DEPARTMENT OF HEALTH CARE ACCESS AND INFORMATION
FACILITIES DEVELOPMENT DIVISION

APPLICATION FOR PREAPPROVED PREFABRICATED COMPONENTS AND SYSTEMS

OFFICE USE ONLY

APPLICATION #: PCS- 0002

HCAI Preapproved Prefabricated Components and Systems (PCS)

Type: ☒ New ☐ Renewal

Manufacturer Information

Manufacturer: Simpson Strong-Tie

Manufacturer's Technical Representative: Brandon Chi

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Product Information

Product Name: Simpson Strong-Tie Yield-Link Moment Connection

Product Type: Pre-fabricated Steel Moment Connectors

General Description: Replaceable pre-fabricated steel connectors used to construct a steel beam-to-column moment connection.

Applicant Information

Applicant Company Name: Simpson Strong-Tie

Contact Person: Louay Shamroukh

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I hereby agree to reimburse the Department of Health Care Access and Information review fees in accordance with the 2019 California Administrative Code.

Signature of Applicant: [signature] Date: 6-10-2022

Title: Director of Engineering Company Name: Simpson Strong-Tie

Registered Design Professional Preparing Engineering Report

Company Name: Simpson Strong-Tie

Name: Brandon Chi California License Number: S4954

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OFFICE USE ONLY

APPLICATION #: PCS- 0002

Disciplines Involved

- [x] Structural
- [ ] Architectural
- [ ] Mechanical
- [ ] Electrical
- [ ] Plumbing
- [ ] Fire Life Safety

OFFICE USE ONLY – HCAI APPROVAL

Signature: ____________________ Date: 6/15/2022

Print Name: Kamalpreet Kalsi

Title: Senior Structural Engineer

Approved Version Number v1.0

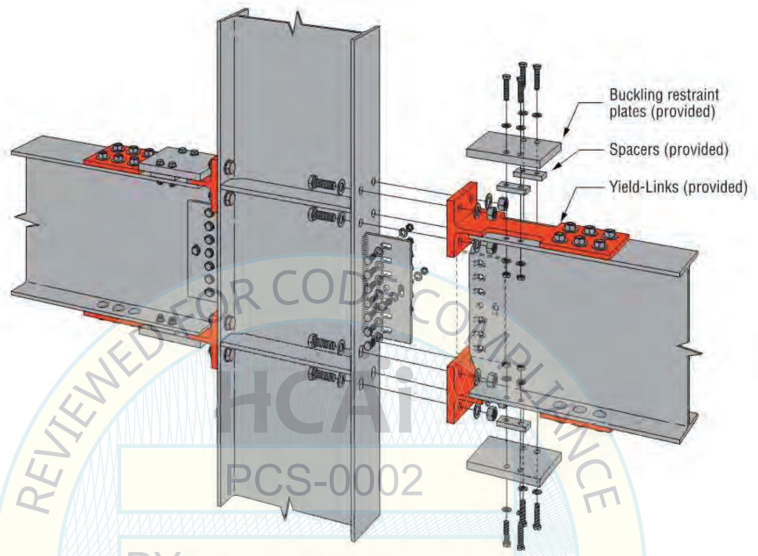
Version History

v 1.0 May 15, 2022 - Original Approval

HCAI PLAN REVIEW GUIDE

SIMPSON STRONG-TIE – YEILD LINK MOMENT CONNECTION

FOR USING SIMPSON STRONG-TIE YIELD-LINK IMF & SMF CONNECTIONS



Detailed View of the Yield-Link Connection

SIMPSON Strong-Tie®

Date: 06/13/2022

Version: 1.0

REGISTERED PROFESSIONAL ENGINEER
BRANDON Y. CHI
No. 84954
EXP: 09/30/2022
STRUCTURAL
STATE OF CALIFORNIA

06/13/2022

SIMPSON STRONG-TIE CO., INC.
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SECTION 1: LIMITS OF PRE-APPROVAL

1.1 Yield-Link Moment Connect Design Criteria for Use Under 2019 California Building Code

A bolted moment connection for Simpson Strong-Tie Yield-Link Moment Connection in accordance with AISC 358-16 Chapter 12, Supplement No.2 shall be permitted under the following conditions:

a. A linear analysis procedure shall be used for design of the SMF and IMF systems using the SST yield link when permitted in accordance with ASCE 7. Nonlinear procedures will be considered as an alternative system.

b. Only T-stub yield links are permitted. End plate yield links are not permitted.

c. The biaxial dual-strong axis and column minor axis configurations of the moment connection shall be considered as an alternative system.

d. Beam flange width-to-thickness ratio shall not exceed λ_p of 0.38*sqrt(E/F_y) per AISC 360 Table B4.1b Case 11.

e. Yield Link stem-to-beam flange connection bolts shall not slip under wind design demand loads. Yield-Link stem to beam flange connection shall be designed to prevent slip using AISC 360 Equation (J3-4), where the slip resistance μ is taken to be 0.3. Story drift limitations per 2019 CBC § 1609A.1.2 shall also be met.

f. Double shear plate connection is permitted to increase connection axial capacity for collector loads. A PJP groove weld for second shear plate is permissible due to space restrictions.

SECTION 2: DESIGN GUIDE

2.1 Yield Link Description

a. Overview and Expected Connection Behavior

The Simpson Strong-Tie® Strong Frame® steel special moment frame connection (YLMC) is a field-bolted connection that uses patented Yield-Link® structural fuse technology to mitigate against damage to beams and columns during seismic events. The inelastic deformations in the moment connections are limited to neck down portion of the Yield-Links that are located on top and bottom of the beam flanges.

b. Parts and Nomenclature

The Yield-Link Moment Connection (YLMC) consisted of a pair Yield-Links bolted to the top and bottom beam flanges. The reduced portion of each Yield-Link is retrained by a pair of spacer plates on each side of the yielding region. A buckling restraint plate connects the spacer plates to the beam flanges. The Yield-Link is then connected to the column flange through the link flange-to-column flange bolts. The shear plate is modified such that all the moment is resisted by the Yield-Links and all the shear and axial load are resisted by the shear plate. The shear plate rotates around the central bolt, which uses a standard size bolt hole, whereas the rest of the vertical holes are slotted horizontally to rotate with the connection movement. Additional horizontal bolts at the beam centerline can be added to increase the axial capacity of the connection. In addition, double shear plates can also be used to increase the axial load capacity for collector demands.

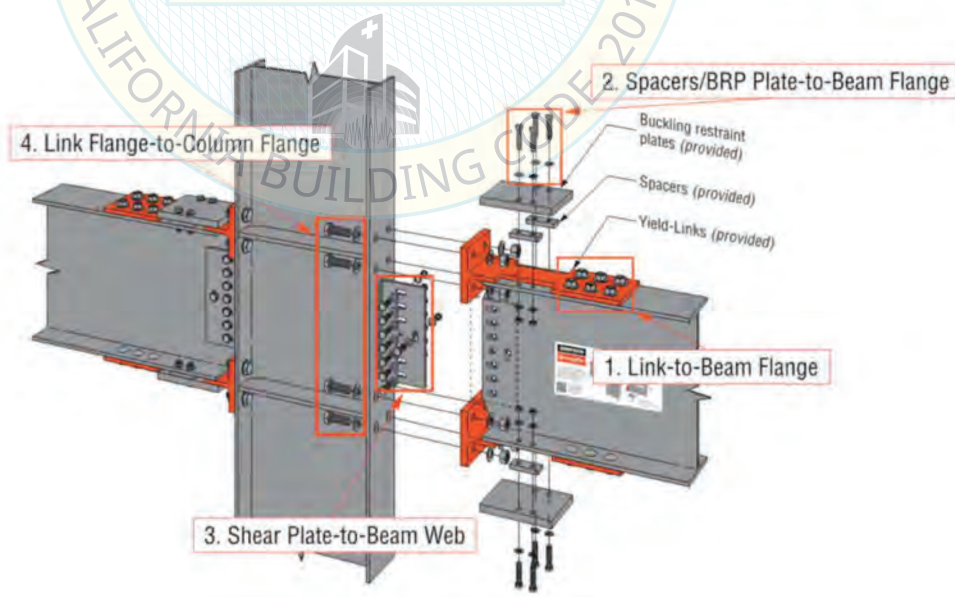
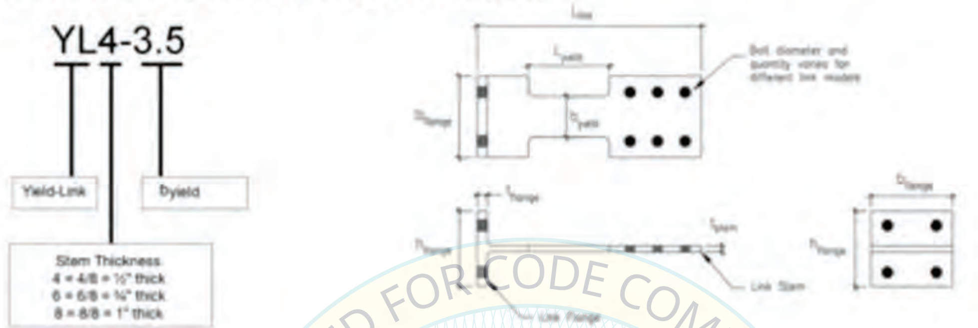


Figure 1.1: Simpson Strong-Tie Yield-Link Connection Components

Currently Simpson Strong-Tie offers three link thicknesses (tstem=0.5", 0.75" and 1"). Link design capacity is governed by the Yield-Link area (byield x tstem). A list of SKU offerings is shown in the table below:

Yield-Link® Moment Connection Product Offering

Yield-Link Moment Connection Product Offering Table



Yield-Link® ID	t_{stem} (in.)	t_{flange} (in.)	b_{yield} (in.)	A_{yield} (in.)²	L_{yield} (in.)	L_{link} (in.)	b_{flange} (in.)	h_{flange} (in.)	$P_{y,link}$ kips	K_{eff} kip/in.	Min. Beam Depth	Max. Beam Depth	BRP ID	Spacer Plate ID
YL4-2	0.50	0.875	2.200	1.000	7	18.6250	6.25	5.75	50.00	2.970	W12	W18	BRP4C	SP4C
YL4-2.5			2.500	1.250					62.50	3.468				
YL4-3			3.000	1.500			6.50		75.00	3.960				
YL4-2.25			2.250	1.125		21.3750	7.00		56.25	3.337			BRP4A	SP4
YL4-2.875			2.875	1.438					71.88	3.953				
YL4-3.5			3.500	1.750			8.00		87.50	4.460			BRP4B	
YL4-3.75			3.750	1.875					93.75	4.651				
YL4-4			4.000	2.000					100.00	4.831				
YL4-2.25-10		0.819	2.250	1.125	10	21.5625	7.00		56.25	2.554	W16	W27	BRP4A-10	SP4-10
YL4-2.875-10			2.875	1.438					71.88	3.077				
YL4-3.5-10			3.500	1.750		24.3125	8.00		87.50	3.529			BRP4B-10	
YL4-3.75-10			3.750	1.875					93.75	3.701				
YL4-4-10			4.000	2.000					100.00	3.865				
YL6-2.5	0.75	1.250	2.500	1.875	10	27.5000	6.50	9.25	93.75	3.426	W16	W27	BRP6D	SP6D
YL6-3			3.000	2.250			8.00		112.50	4.149			BRP6A	SP6
YL6-3.5			3.500	2.625					131.25	4.564				
YL6-4			4.000	3.000					150.00	4.933				
YL6-4.5			4.500	3.375			10.00		168.75	5.801			BRP6B	
YL6-5			5.000	3.750					187.50	6.167				
YL6-5.5			5.500	4.125			12.00		206.25	7.037			BRP6C	
YL6-6			6.000	4.500					225.00	7.400				
YL6-3-13			3.000	2.250	13	30.5000	8.00		112.50	3.484	W24	W36	BRP6A-13	SP6-13
YL6-3.5-13			3.500	2.625					131.25	3.868				
YL6-4-13			4.000	3.000					150.00	4.216				
YL6-4.5-13			4.500	3.375			10.00		168.75	4.925			BRP6B-13	
YL6-5-13			5.000	3.750					187.50	5.270				
YL6-5.5-13			5.500	4.125			12.00		206.25	5.981			BRP6C-13	
YL6-6-13			6.000	4.500					225.00	6.324				
YL8-4	1.00	1.813	4.000	4.000	13	31.0625	9.00	10.75	200.00	6.034	W24	W36	BRP8A	SP8
YL8-4.5			4.500	4.500					225.00	6.524				
YL8-5			5.000	5.000		34.4375	12.00		250.00	7.698			BRP8B	
YL8-5.5			5.500	5.500					275.00	8.213				
YL8-6			6.000	6.000					300.00	8.698				
YL8-4-15			4.000	4.000	15	33.0625	9.00		200.00	5.465	W24	W36	BRP8A-15	SP8-15
YL8-4.5-15			4.500	4.500					225.00	5.931				
YL8-5-15			5.000	5.000		37.4375	12.00		250.00	6.959			BRP8B-15	
YL8-5.5-15			5.500	5.500					275.00	7.446				
YL8-6-15			6.000	6.000					300.00	7.908				

c. Design Responsibility

Structure and moment connection design and sealing of drawings using the Yield-Link Moment Connection are the responsibility of the Structural Engineer of Record (SEOR). Simpson Strong-Tie Yield-Link Moment Connection tools are available free for charge for the SEOR to use.

2.2 Computer Modeling

a. Stiffness and Panel Zone

Since the Yield-Link Moment Connection is considered a PR connection, connection stiffness shall be accounted for in the structural model for analysis. There are couple of ways to model the YL connection, for all the YLMC design tools a rotational spring is modeled at the ends of the beam to model the YLMC. Calculation of the rotational stiffness is included in Step 11 of AISC 358s2-20 and Step 12 of ICC-ESR-2802.

For Panel Zone modeling, rigid end zones shall be ignored (i.e. set to zero).

b. YLMC Plugins

This guide uses examples from ETABS/SAP plugin version 3.2, RAM Structural version 17.03 and RISA-3D V19. Figure below is a screen shot from the YLMC ETABS Plugin. Rotational spring stiffness (K_rot) can be calculated from the beam size and Yield-Link size.

Initial Link Selection | Beam & Link Check | Column Check | Shear Tab Check | Drift Summary | Beam Design | Column Design | Weld Summary

Filter (Elevation-Story): All Elevations All Stories

Elev. ID	Grid ID	Story	Beam Unique Name	Beam Size	Yield-Link® Size	K_rot (k-in/rad)	I_End Column Size	J_End Column Size	Assign Link at I_End	Assign Link at J_End
4	B	Story2	13	W18X50	YL4-3.5	763185	W24X131	W24X162	☑	☑
4	C	Story2	15	W18X50	YL4-3.5	763185	W24X162	W24X131	☑	☑
4	B	Story1	14	W27X102	YL8-5	3039122	W24X131	W24X162	☑	☑
4	C	Story1	16	W27X102	YL8-5	3039122	W24X162	W24X131	☑	☑

c. Excel Spreadsheet

YLMC Excel Design Tool V3.3 is a standalone spreadsheet that can do both frame analysis and design for 2-dimensional frames (i.e. design one line of frame at a time). Currently the SEOR can overwrite the Yield-Link design moment, shear plate shear and shear plate axial load in the "1 Frame Geometries" tab. The SEOR can input their hand calculated collector axial load and overwrite the $P_{u\text{-}sp}$ axial load to design the shear plate connection.

N	O	P	U	V	W	X	Y	Z	AA	AB
						1. INITIAL YIELD-LINK SELECTION SUMMARY				
Beam ID	Beam Size	Link Size	Assign Link at I_End	Assign Link at J_End	$M_{u\text{-}link}$ (kip-in.)	$V_{gravity}$ (kips)	$P_{u\text{-}sp}$ (kips)	Initial t_{bf} check	Initial b_{bf} check	Initial b_{cf} check
1001	W21X55	YL6-4	YES	YES	1457	0.8	45.2	0.77	1.00	1.16

2.3 Connection and Member Design

a. Design Parameters

Structural design parameters are provided by the SEOR for the lateral force resisting system design. For steel frames with YLMC, response modification factor can be taken as 8.0 for Special Steel Moment Resisting Frames. For the YLMC ETABS Plugin, these design parameters are used to generate the structural base shear for wind and seismic loads. The plugin then generates the load cases and load combos for the design of the YLMC and member design.

DESIGN PARAMETERS

No.	Item	Data
Seismics Coefficients		
1	Standard=	ASCE 7-10
2	0.2 Sec Spectral Accel, Ss=	1.50
3	1 Sec Spectral Accel, S1=	0.60
4	Long-Period Transition Period, TL=	8.00
5	Site Class=	D
6	Sds - User defined=	YES
7	Design Spectral Resp. Accel at Short Period, Sds=	1.00
Factors		
1	Frame type=	SMF
2	Response Modification Coefficient, R=	8.00
3	Overstrength Factor, Omega=	3.00
4	Deflection Amplification factor, Cd=	5.50
5	Occupancy Importance, I=	1.00
6	Redundancy Factor, Rho=	1.00
7	Live Load Factor, f1=	0.50
8	Snow Load Factor, f2=	0.20

All the design tools for the Simpson Yield-Link Moment Connection cover the design of the following items. Sections below will cover these individual items in more detail.

- Drift Check
- Beam and Link Strength Check
- Local Column Check
- Shear Tab/Plate Check
- Drags and Collectors Check
- Beam (Global) Design Check
- Column (Global) Design Check

Note: Design output can be from ETABS/SAP2000 plugin, Yield-Link Excel Tool, RAM Structural System or SEOR's calculations. The reference screen shots below are from the Simpson Strong-Tie ETABS plugin.

This approval is only limited to the design methodology under AISC 358s2-20 with additional HCAI Limitations:

b. Prequalification Limits per AISC 358-20 Supplement 2

- Maximum beam and column size: W36
- Maximum Yield-Link thickness: 1 inch
- Maximum Yield-Link yielding width: 6 inch

Please see AISC 358s2-20 for the complete list of prequalification.

c. HCAI Preapproval Limitations

Simpson Yield Link design criteria for use under 2019 California Building Code:

A bolted moment connection for Simpson Strong-tie Yield Link in accordance with AISC 358-16 Chapter 12, Supplement No.2 shall be permitted under these conditions:

- A linear analysis procedure shall be used for design of the SMF and IMF systems using the SST yield link when permitted in accordance with ASCE 7. Nonlinear procedures will be considered as an alternative system.
- Only T-stub yield links are permitted. End plate yield links are not permitted.
- The biaxial dual-strong axis and column minor axis configurations of the moment connection shall be considered as an alternative system.
- Beam flange width-to-thickness ratio shall not exceed λ_p of 0.38*sqrt(E/F_y) per AISC 360 Table B4.1b Case 11.
- Yield Link stem-to-beam flange connection bolts shall not slip under wind design demand load. Yield-Link stem to beam flange connection shall be designed to prevent slip using AISC 360 Equation (J3-4), where the slip resistance μ is taken to be 0.3. Story drift limitations per 2019 CBC § 1609A.1.2 shall also be met.
- Double shear plate connection is permitted to increase connection axial capacity for collector loads. A PJP groove weld for second shear plate is permissible due to space restrictions.

d. Initial Link Selection

In the ***Initial Yield-Link Selection/Frame Geometry Summary*** Output Page of the PDF file (The reference screen shots below are from the Simpson Strong-Tie ETABS plugin).

Initial Yield-Link® Selection Summary

Elev. ID	Grid ID	Story	Beam Unique Name	Beam Size	Yield-Link® Size	K_rot (kip-in/rad)	I_End Col Size	J_End Col Size	Assign K at I_End	Assign K at J_End	tbf Check	bf Check	Lyield Check	PZ I_End Col	PZ J_End Col	Drift DCR
1	B	ROOF	97	W30X99	YL8-4	2843299	W33X221	W33X221	True	True	0.597	0.810	0.848	0.856	0.880	0.353
1	C	ROOF	98	W30X99	YL8-4	2843299	W33X221	W33X221	True	True	0.597	0.810	0.848	0.880	0.880	0.348

- K_rot: Calculate the rotational spring for partially restrained connection
- tbf: Minimum of 0.4" thick beam flange (actual check in later section)
- bf: This is to check if the Yield-Links will fit on the beam/column flange
- Lyield: Initial Yield-Link yielding length check
- PZ (value for reference only; see other tabs noted later, DCR can be higher than 1.0, column shear has not been subtracted out yet, nor doubler plates included at this time)
- Drift (value for reference only; approximately 20% increase in drift is expected after implementation of yield link connection stiffness): Maximum of the Seismic and Wind drift check (see Drift Section for detailed check)

e. Drift Checks

Similar to other moment connections, drift typically governs steel moment frame design. For frames with YLMC, drift should be calculated by accounting for the connection rotation stiffness. This is already taken into account in all the YLMC design tools.

In the ***Drift Summary*** Output Page of the PDF file:

Drift Summary

Elev. ID	Grid ID	Story	Beam Unique Name	Beam Size	Yield-Link® Size	I_End Col Size	J_End Col Size	Assign K at I_End	Assign K at J_End	Node ID @LeftEnd	Node ID Below	Delta_x (in)	Story Height (in)	Allowable Drift (in)	Drift DCR
1	B	ROOF	97	W30X99	YL8-4	W33X221	W33X221	YES	YES	103	1	1.018	144.00	2.880	0.353
1	C	ROOF	98	W30X99	YL8-4	W33X221	W33X221	YES	YES	104	2	1.001	144.00	2.880	0.348

f. Beam and Link Checks

In the ***Beam & Yield-Link Check Summary*** output page of the PDF file

Beam & Yield-Link® Check Summary

Elev. ID	Grid ID	Story	Beam Unique Name	Beam Size	Yield-Link® Size	BRP Size	MU_max (kips.in)	Beam tf_DCR	Link Strength DCR	Lyield DCR	BRP DCR	BRP Bolt DCR
1	B	ROOF	97	W30X99	YL8-4	BRP8A	1980.16	0.769	0.358	0.848	0.836	0.555
1	C	ROOF	98	W30X99	YL8-4	BRP8A	1943.74	0.769	0.352	0.848	0.836	0.555

- Beam flange thickness (tf_DCR): Minimum beam flange thickness check per AISC 358 Step 10.2
- Link Strength: Link strength to resist LRFD combo moment per AISC 358 Step 3
- Lyield_DCR: Minimum yielding length check per AISC 358 EQ. 12.9-4
- BRP DCR: Buckling restraint plate strength check per AISC 358 EQ. 12.9-13
- BRP Bolt DCR: Buckling restraint bolt strength check per AISC 358 Step 10.3

g. Column Local Checks

In the ***Column Check Summary*** output page of the PDF file

Column Check Summary

Elev. ID	Grid ID	Story	Column Unique Name	Column Size	Yield- Link® Left Side	Yield- Link® Right Side	Col Pu (kips)	V_bm Gravity (kips)	Stiffener Provider	Doubler Plate Provider	Stiffener Req.	Min. Stiff. Thk (in)	Min. Dblr Plate Thk (in	SCWL DCR	Col PZ DCR	Col Flange Check
1	B	ROOF	116	W33X221	NA	YL8-4	64.10	12.30	NO	NO	NO	0.000	0.000	NA	0.856	0.686
5	B	ROOF	107	W33X221	NA	YL8-4	63.99	12.30	NO	NO	NO	0.000	0.000	NA	0.856	0.686

- Stiffener Required: Stiffener check per AISC 358 Step 19
- Doubler Plate thickness: Doubler plate thickness per AISC 358 Step 16
- SCWL_DCR: Strong column weak beam/Yield-Link Per AISC 358 Section 12.4. No SCWB check required at top story of multistory building or one-story building per AISC 341 E3.4a (a).
- Col PZ DCR: Per Step 16. Column panel zone capacity check per AISC 360 J10-9 or J10-10.
- Col Flange Check: Column flange flexural yielding check per AISC 358 Eq. 12.9-38

h. Shear Tab Check

In the ***Shear Tab Check Summary*** output page of the PDF file:

Shear Tab Check Summary

Elev. ID	Grid ID	Story	Beam Unique Name	Beam Size	Yield-Link® Size	Axial Pu (kips)	Shear Vu_bm (kips)	No. Vert. Bolts	No. Horz. Bolts	Bolt Size	Bolt Type	Shear Plate Thk (in)	Fillet Weld Size (in)	Beam Web DCR	Shear PL Geo. Check	Shear Plate DCR	Bolt DCR	Fillet Weld DCR (in)
1	B	ROOF	97	W30X99	YL8-4	37.68	12.30	7	2	7/8	A325X	1/2	5/16	0.472	OK	0.466	0.701	0.282
1	C	ROOF	98	W30X99	YL8-4	37.68	12.30	7	2	7/8	A325X	1/2	5/16	0.472	OK	0.466	0.701	0.282

- Beam Web: Beam web strength check per AISC 358 Step 15.5
- Shear Plate geometry: AISC 358 Step 15.2 for slotted holes, and overall depth to fit Yield-Links
- Shear Plate strength: Shear plate strength check per AISC 358 Step 15.3
- Bolt strength: Shear plate bolt strength check per AISC 358 Step 15.1
- Shear tab to column flange fillet weld: Fillet weld size check per AISC 358 Step 15.4

i. Collector Connection Check

Currently all the Yield-Link moment connection tools use Overstrength/Omega combinations to calculate the axial forces in the beams for the connection design. However, beam axial forces depend on how the floor and roof diagrams are modeled. For example, if all the diaphragms are modeled as rigid diaphragms, then axial loads in the beams will be all zero. Floor and Roof diaphragms could be modeled with semi-rigid diagrams (as approved by reviewer) to reflect their actual stiffness and to get more accurate force distribution to the beam and connections.

As noted previously, YLMC connection force input can be overwritten in the Excel tool and ETABS/SAP2000 plugins. The collector load will be designed to transmit through the shear plate by the centerline of bolts with only vertical slots. Double plate shears are permitted for higher demand loads.

File Options Export Help
Design Parameters | DCR Limits | Create LoadCombos | PDF Summary | Slab Depths | DXF Elevation

Initial Link Selection | Beam & Link Check | Column Check | Shear Tab Check | Drift Summary | Beam Design | Column Design | Weld Summary

Filter (Elevation-Story): All Elevations · All Stories

Elev ID	Grid ID	Story	Beam Unique Name	Beam Size	Yield-Link® Size	Axial Pu (kips)	Shear Vg_LC08 (kips)	Shear Vu (kips)	Shear Vg_LC01-07 (kips)	No. Vert. Bolts	No. Horz. Bolts	Bolt Size (in)	Bolt Type	SP Thickness (in)	No. of SP	Fillet Weld Size (in)	Beam Web DCR	Shear Plate Geometry Check	Shear Plate DCR	Bolt DCR	Fillet Weld DCR
1	B	Story2	39	W18X50	YL6-3	50.00	2.78	22.83	4.16	3	2	7/8	A325X	1/2	1	5/16	0.947	OK	0.654	0.852	0.199
1	C	Story2	41	W18X50	YL6-3	55.00	2.78	22.83	4.11	3	2	7/8	A325X	1/2	1	5/16	1.044	OK	0.708	0.930	0.199
1	B	Story1	40	W27X102	YL6-3	70.00	12.80	42.58	17.52	7	3	7/8	A325X	1/2	1	5/16	0.612	OK	0.598	0.786	0.193
1	C	Story1	42	W27X102	YL6-3	43.97	12.87	42.65	17.43	7	2	7/8	A325X	1/2	1	5/16	0.575	OK	0.538	0.744	0.193
4	B	Story2	9	W18X50	YL6-3	30.42	2.78	22.83	4.16	3	2	7/8	A325X	1/2	1	5/16	0.562	OK	0.452	0.555	0.199
4	C	Story2	11	W18X50	YL6-3	30.42	2.78	22.83	4.10	3	2	7/8	A325X	1/2	1	5/16	0.562	OK	0.452	0.555	0.199
4	B	Story1	10	W27X102	YL6-3	43.99	12.83	42.61	17.58	7	2	7/8	A325X	1/2	1	5/16	0.576	OK	0.539	0.744	0.193

j. Beam Design Check

In the ***Beam Design Summary*** output page of the PDF file:

Beam Design

Elev. ID	Grid ID	Story	Beam Unique Name	Beam Size	Yield-Link® ID	Pr_Link (kips)	Mpr_Link (kips.in)	Mu_Omega (kips.in)	Mu_Omega/Pr_Link	Mu_Adj. Fact	Axial DCR	B-maj DCR	B-min DCR	PMM DCR	Adj. B-maj DCR	Total DCR
1	B	ROOF	97	W30X99	YL8-4	312.00	9578.40	4891.11	0.51	1.96	0.042	0.380	0.050	0.472	0.744	0.836
1	C	ROOF	98	W30X99	YL8-4	312.00	9578.40	4731.45	0.49	2.02	0.051	0.371	0.033	0.455	0.751	0.835

Beam design by ETABS steel design module (CSI software) and the summary analysis output are shown for reference. Output summary includes the following:

- Axial: Axial capacity check per AISC 360
- Major bending: Major axis bending capacity check per AISC 360
- Minor bending: Minor axis bending capacity check per AISC 360
- Combined Axial + Major bending: Combined axial + bending per AISC 360 chapter H
- Adj. Major bending: Step 13.1(2) ensure beam is stronger than the Yield-Link Moment Connection
- Total Combined Axial + Adj. Major bending: Step 13.1

k. Column (Global) Design check

In the ***Column Design Summary***:

Column Design

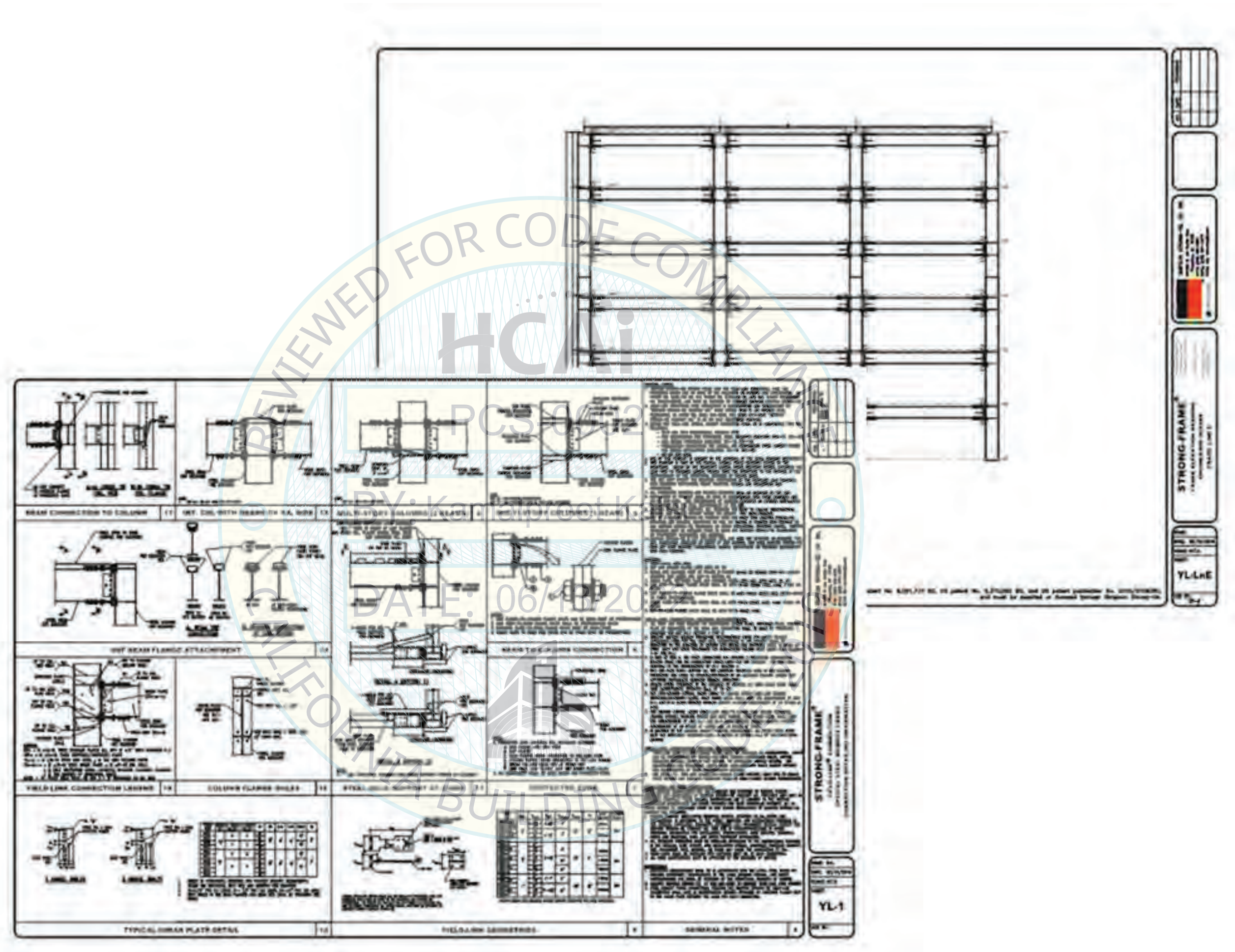
Elev. ID	Grid ID	Story	Column Unique Name	Column Size	Mu_Top (kips.in)	Mu_Bot (kips.in)	Mu_Omega (kips.in)	Pu_Omega (kips)	Bracing at Beam Bot. FLG	bf/tf DCR	h/tw DCR	Axial DCR	B-maj DCR	B-min DCR	PMM DCR	Adj. B-maj DCR	Total DCR
1	B	ROOF	116	W33X221	4661.58	2146.80	4661.58	64.10	YES	N/A	N/A	0.011	0.121	0.030	0.162	0.121	0.162
5	B	ROOF	107	W33X221	4635.19	2110.17	4635.19	63.99	YES	N/A	N/A	0.011	0.120	0.018	0.150	0.120	0.150

Column design by ETABS steel design module (CSI software) and the summary analysis output are shown for reference. Output summary includes the following:

- Axial: Axial capacity check per AISC 360
- Major bending: Major axis bending capacity check per AISC 360
- Minor bending: Minor axis bending capacity check per AISC 360
- Combined Axial + Major bending: Combined axial + bending per AISC 360 chapter H
- Adj. Major bending: Step 13.2
- Total Combined Axial + Adj. Major bending

l. Design Summary Output

After the completion of Yield-Link Moment Design for the specific structure/frame line, all the design tools and plugins have the capability to output a summary calculation package in PDF format. In addition, the design tools can also generate a detailed scaled elevation drawing for each of the moment frame lines. Figure below shows an example output of the frame elevation and the typically installation sheet that accompanies the frame elevation.



2.4 Fabrication and Erection

a. Overview

Yield-Links, buckling restrain plates and spacer plates are fabricated and supplied by Simpson Strong-Tie. All these items are fabricated in Riverside, California, an AISC certified facility. Each item/SKU is uniquely identified with a part number. In addition to the etched SKU ID, YLMC fuse elements also have a batch number (for heat number traceability) and date of manufacturing.

b. Yield Link Plate Fabrication

The reduced section of the Yield-Link shall be cut using the following methods: laser, plasma, or water-jet method. Maximum roughness of the cut surface shall be 250 µ-in. (6.5 microns) in accordance with ASME B46.1. All transitions between the reduced section of the Yield-Link and the nonreduced sections of the Yield-Link shall utilize a smooth radius, R, as shown in Figure 12.2(a) in AISC 358s2-20, where R equals the thickness of the link stem, t_{stem}. Cutting tolerance at the reduced section shall be plus or minus 1/16 in. (2 mm) from the theoretical cut line.

c. Quality Assurance Plan

Quality Assurance plan conforming to 2019 CBC § 1704A.2, AISC Chapter 360 Chapter N, AISC 341, Chapter J and AWS D1.8 Clause 7 shall be included in the construction documents by the registered design professional.

d. Yield Link Specific Inspection Requirements

- Roughness shall be 250 micro inches
- Yield-Link sizes shall match the elevation drawings on the construction document
- Yield-Links at top and bottom of the beam shall be installed with matching heat number at each connection

These additional special inspection requirements are listed in TIO form as part of the construction documents.

e. Bolting:

- Bolt holes in the beam/column and shear plates shall meet AISC 360 Chapter M requirements on surface roughness profile.
- Bolt installation shall meet RCSC Specification for Structural Joints Using High-Strength Bolts.
- See Section 2.6c for specific bolting requirements for the YLMC connection.

f. Tolerances:

- Refer to AISC 303 Code of standard Practice for Steel Buildings and Bridges for fabrication and erection tolerances.

g. Protected Zones

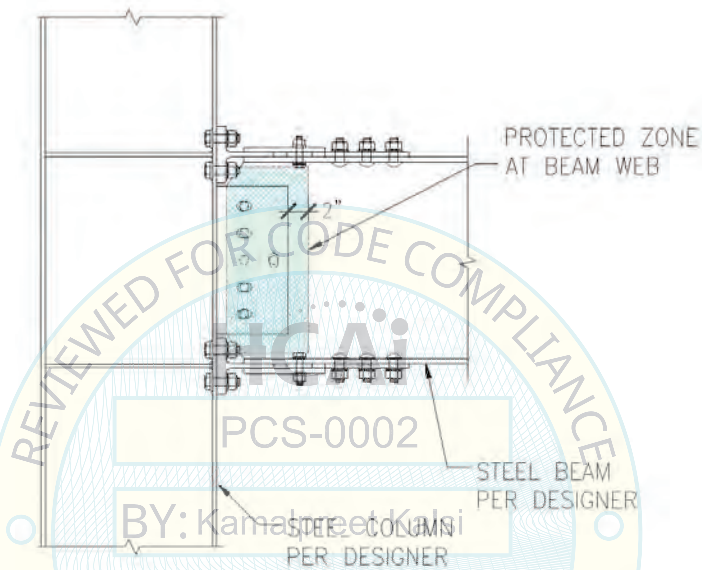
The Protected Zone includes the following elements:

- Link flange and link stem
- Beam flange area connected to the link stem
- Column flange area connected to the link flange
- Shear plate and beam web at shear plate (2" around shear plate)
- Link stem, Link flange BRP and shear plate bolts.
- Marking of Protected Zone:
  Before fireproofing: Each moment connection with Simpson Strong-Tie Yield-Link Moment Connection will have a sticker placed at the beam web. The sticker will indicate the Protected Zones for the connection.
  After fireproofing: Marking shall be made by using any non-destructive method.
- See Detail 7/INST1A Protected Zone elements for the Simpson Strong-Tie Yield-Link moment Connection.

h. Repairs

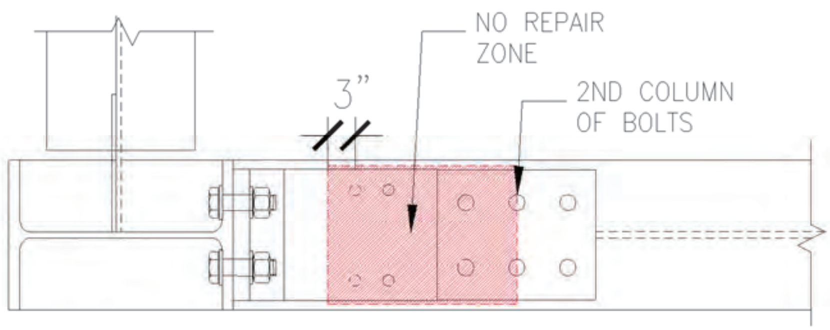
The Structural Engineer of Record shall be notified if any attachment to the Protected Zone in the Yield-Link region occurs. Sections below cover some of the possible repairs at the Protected Zone region of the YLMC.

h.1. At Beam Web near Shear Plate:



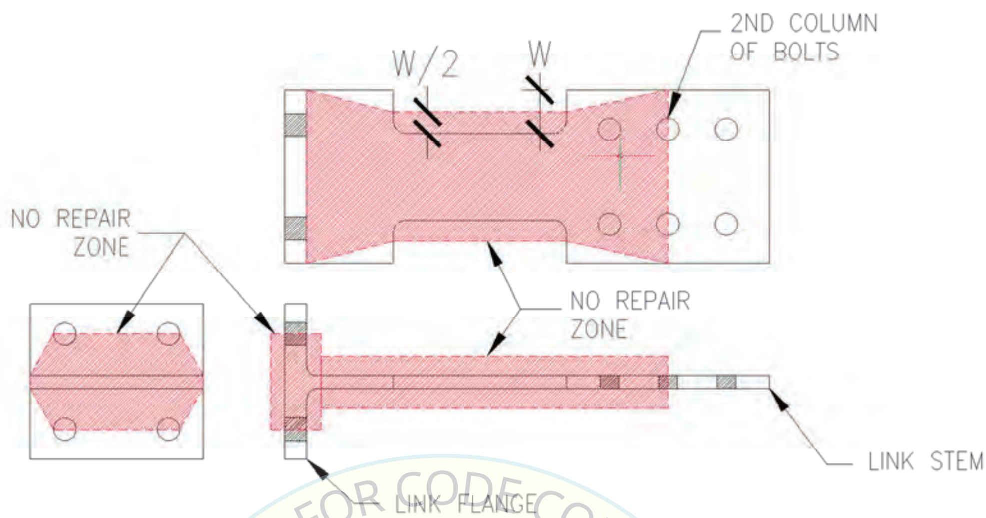
Any notch, accidental weld, or shot pin that occurs within the 2” protected zone from the shear tab shall be removed and beam web ground smooth (500 micro-inch in accordance with ASME B46.1; so the shear plate can rotate with the connection. This applies to the beam web on the opposite side of the shear plate as well.

h.2. At Buckling Restraint Plate (BRP) and Link Stem:



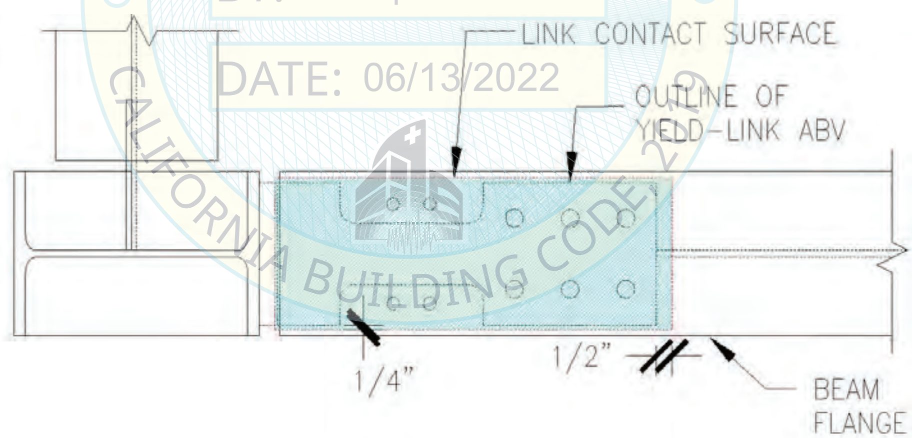
Any notch, accidental weld, or shot pin occurring away from the no repair zone can be removed and surface ground smooth. Otherwise the BRP or Yield-Link shall be replaced. See Item C for more detailed information for Yield-Link requirements.

h.3. At Yield-Link Stem and Flange:



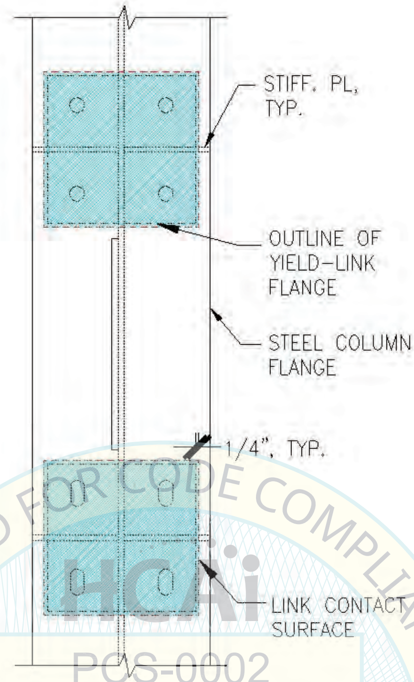
Any notch, accidental weld, or shot pin occurring away from the no repair zone can be removed and surface ground smooth; Otherwise the Yield-Link shall be replaced.

h.4. At Beam Top and Bottom Flange Contract Surfaces:



If any notch, accidental weld, or shot pin occurs within any of Yield-Link to beam flange contact surfaces prior to installation of the Yield-Link, the surfaces shall be grounded smooth.

h.5. At Column Flange Contract Surfaces:



If any notch, accidental weld, or shot pin occurs within any of the Yield-Link to column flange contact surfaces prior to installation of the Yield-Link, the surfaces shall be grounded smooth.

h.6. At Yield-Link Connection Bolts (BRP, Link Flange-to-Column Flange, Link Stem-to Beam Flange, Shear Plate):

If any notch, accidental weld, or shot pin occurs on any of the bolts/washers/nuts, the specific part shall be replaced.

2.5 Details, Specifications and TIO

a. Installation Details (General Notes and Details)

Details from the installation sheets noted below are preapproved.

Drawing Sheet	Detail No.	Detail Name	Date	Notes/Approval ID
YL-INSTL1A	4	General Notes	06/13/2022	
	5	Multi-Story Columns (1-Beam)	06/13/2022	
	6	Connection Bolting Detail	06/13/2022	
	7	Protected Zone	06/13/2022	
	8	Yield-Link Geometries	06/13/2022	
	9	Multi-Story Columns (2-Beams)	06/13/2022	
	11	Steel Deck Support at Conn.	06/13/2022	
	13	Roof Conn. W/Beams on Each Side	06/13/2022	
	15	Yield-Link Concrete Cover	06/13/2022	
	17	Bolt Bearing Near Column Web	06/13/2022	
	19	Yield-Link Connection Legend	06/13/2022	
	20	Typical Shear Plate Detail	06/13/2022	

Drawing Sheet	Detail No.	Detail Name	Date	Notes/Approval ID
YL-INSTL2A	1	YL4 Beam Cope/Hole Detail	06/13/2022	
	2	YL6 Beam Cope/Hole Detail	06/13/2022	
	4	Yield-Link Detailed Geometries	06/13/2022	
	5	Extended YL4 Beam Cope/Hole Detail	06/13/2022	
	6	Extended YL6 Beam Cope/Hole Detail	06/13/2022	
	9	YL8 Beam Cope/Hole Detail	06/13/2022	
	10	Shear Plate Details	06/13/2022	
	11	Stiffener Plate Details	06/13/2022	
	12	Doubler Plate Details	06/13/2022	
	13	Extended YL8 Beam Cope/Hole Detail	06/13/2022	
	17	Column Flange Holes	06/13/2022	
	18	Orthogonal Conn. To Column	06/13/2022	
	20	Alt. Attachments at SMF BM FLG	06/13/2022	Not Approved Alternative system.

Drawing Sheet	Detail No.	Detail Name	Date	Notes/Approval ID
YL-INSTL3A	2	Corner Cond. With Flange Cruciform Column	06/13/2022	Not Approved Alternative system.
	4	Slope Beam Yield-Link Connection (Option 1)	06/13/2022	
	10	Multi-Direction Moment Connection Detail	06/13/2022	Not Approved Alternative system.
	12	Slope Beam Yield-Link Connection (Option 2)	06/13/2022	
	18	Cruciform Col. Clip Details	06/13/2022	Not Approved Alternative system.
	20	Slope Beam Yield-Link Connection (Option 2)	06/13/2022	

b. Master Specifications

Master specification for Yield-Link Moment Connection titled: "*Prefabricated Special Steel Moment Frame*", issue number 3; issue date: 06/13/2022 is preapproved.

c. TIO Sample

On-Site special inspection for Yield-Link Moment Connection shall include:

C-0T3: Yield-Link sizes shall match the elevation drawings on the construction document.

C-0T4: Yield-Links at top and bottom of the beam shall be installed with matching heat number at each connection.

C-0T5: Yield-Link cut surfaces at the reduced section shall have a maximum roughness of 250 micro-inch (6.5 microns), in accordance with ASME B46.1.

SECTION C

NOTE: Approved agencies, individuals, and all changes to the TIO program shall be identified, evaluated by the DPOR and approved by HCAI prior to proceeding with the related work.

Facility #:	Facility Name:	Project #:	Sub #:

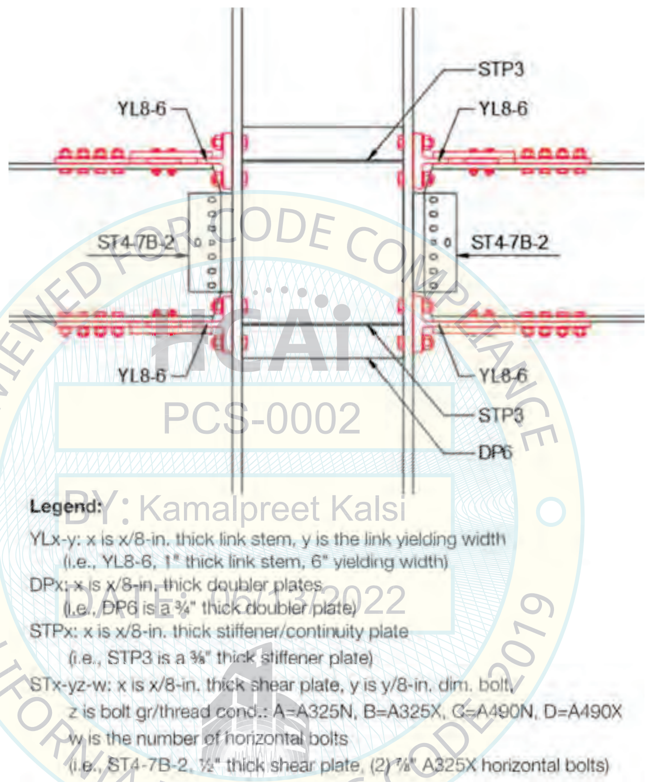
Index #	REQUIRED (Select with "X")	ON-SITE SPECIAL INSPECTIONS	Select with "X" or required information: Samples of test & inspection reports	OPAA No. and Expiration Date	RESPONSIBLE APPROVED AGENCY AND/OR INDIVIDUAL (IDENTIFY SPECIAL INSPECTOR)	COMPLIANCE VERIFICATION BY IOR (Initial/Date)	HCAI/FDD USE (Initial/Date)
OTHER SPECIAL INSPECTIONS							
C-OT1		**Exterior insulation and finish systems (EIFS).** CBC 1705A.16 EIFS applications					
C-OT2		**Water-resistive barrier coating** CBC 1705A.16.1 ASTM E2570 Barrier coating when installed over a sheathing substrate.					
C-OT3	X	**Simpson Strong-Tie Yield-Link Moment Connection:** Yield-Link sizes/spacer plates and buckling restrain plates shall match the elevation drawings on the construction document.					
C-OT4	X	**Simpson Strong-Tie Yield-Link Moment Connection:** Yield-Links at top and bottom of the beam shall be installed with matching heat number at each connection.					
C-OT5	X	**Simpson Strong-Tie Yield-Link Moment Connection:** Yield-Link cut surfaces at the reduced section shall have a maximum roughness of 250 micro-inch (6.5 microns), in accordance with ASME B46.1.					

Page 6

2.6 Plan Review

a. Construction Documents (Typical Callout)

As part of the Yield-Link design tools, scaled elevation drawings with connection detailed showing Yield-Link size and connection plates can be output to a .DXF file. The design drawings should include frame elevations showing the connection details. A typical connection detail is shown below:



a1. Yield-Link sizes should match calculations

Example: YL8-6 is Yield-Link Size shown in connection above (See Legend above, typ.)

a2. Shear tab thickness should match calculations

Example: ***ST4***-7B-2, ST4 is a 4/8" thick shear tab

a3. Shear tab bolt size, grade and number of horizontal bolts should match calculations

Example: ST4-***7B***-**2**, 7B indicates 7/8" A325X bolts, 2 is the number of horizontal bolts

a4. Stiffener plate (continuity plate) thickness should match calculations

Example: STP**3**, 3 indicates 3/8" thick plate

a5. Doubler plate thickness (if required) should match calculations

Example: DP**6**, 6 indicates 6/8" thick plate

b. Frequently asked questions

b1. Maximum beam size and column sizes acceptable?

On the plans or ***Initial Yield-Link Selection/Frame Geometry Summary***

a. Verify that the maximum beam size is a W36 (no weight limit or span-to-depth ratio)

b. Verify that the maximum column size is a W36 (no weight limit)

b2. For fixed base designs, are the 1st story columns seismically compact?

In the ***Column Design*** Output Page of the PDF file

If both b_f/t_f and h/t_w DCR values <=1.0, then it's compact

b3. Is the Column/Beam and Yield-Link combination stiff enough to meet seismic drift check?

Yield-Link Connection stiffness is incorporated in all the design tools per AISC 358 Step 11.
In the ***Drift Summary***, Drift DCR <=1.0, then it's adequate

b4. Are the Yield-Links specified for the moment connection adequate for strength check?

In the ***Beam & Link Check Summary***:
If Link Strength DCR <=1.0, then it's adequate

b5. Will the Yield-Link connection slip for typical wind load combinations?

No, Yield-Link Moment Connection is designed for all the standard wind LRFD combinations to remain elastic. Refer to item 3c for additional check as part of the HCAI approval.

b6. Is stability beam bracing required?

Beam bracing required by AISC 341-16 Section D1-1 2a for Moderately Ductile Members or D1-1 2b for Highly Ductile Members ($L_b=0.095r_yE/(R_yF_y)$) is NOT applicable to beam with Yield-Link Moment Connections since the beam is not designed to Yield. Beam bracing shall be provided per AISC 360 where required.

b7. Can beam bracing be used for beams with Yield-Link connections?

Yes, beam can be designed with bracing to reduce the lateral torsional buckling length of the beam to reduce the beam size/weight.

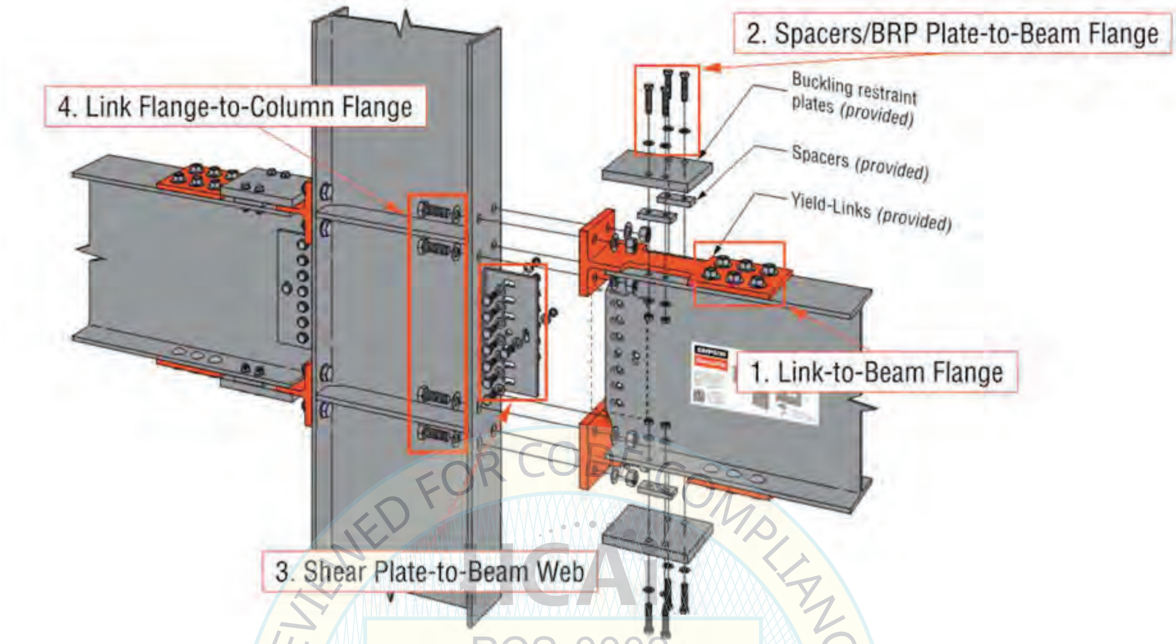
b8. Is there any requirement for special bracing at plastic hinge locations per AISC 341-16?

No, there is no plastic hinge bracing requirement for the Yield-Link Moment Connection.

b9. Is stability bracing of beam-to column connections per AISC 341-16 required?

Yes, column flanges shall be laterally braced at the levels of both the top and bottom beam flanges. However, for the Yield-Link connection, if the column is designed in accordance with Section 12.9 in AISC 358 (Maximum nominal flexural strength is calculated using Sx, instead of Zx, i.e. Mpb=Sx * Fy, instead of Mpb=Zx * Fy), only bracing at the level of the beam top flange is required.

c. Yield Link Connection Bolting



c1. Link stem-to-beam flange bolts must be pretensioned (Not slip-critical)

On the ***Frame Elevation sheets, details or schedules***:

a. Since this is a pretensioned joint only, surface preparation requirement for a slip-critical connection is not required. However, the Link stem-to-beam flange contact surface shall not be painted based on the connection per testing.
b. Link stem-to-beam flange bolts shall be ASTM F3125 grade A490 of F2280 (X or N)
c. Pre-tensioning of this joint can be done with either:
   1. Twisted-off type bolts (F2280)
   2. Direct Tension Indicating (DTI) washers
   2. Turn of nut method
   3. Calibrated wrench method

c2. Spacer/BRP-to beam flange bolts must be snug-tight

On the ***Frame Elevation sheets, details or schedules***:

a. Spacer/BRP bolts must be installed snug-tight
b. BRP-to beam flange bolts shall be ASTM F3125 Grade A325-N (A325-X for YL8-6 & YL8-6-15)

c3. Shear Plate-to-beam web bolts:

On the ***Frame Elevation sheets, details or schedules***:

a. Shear plate-to-beam web bolts can be:
   1. Snug-tight, or
   2. Pretensioned
b. Shear plate-to beam web bolts shall be ASTM F3125 Grade A325, A490, F1852 or F2280 (X or N) per design

c4. Link flange-to-column flange bolts:

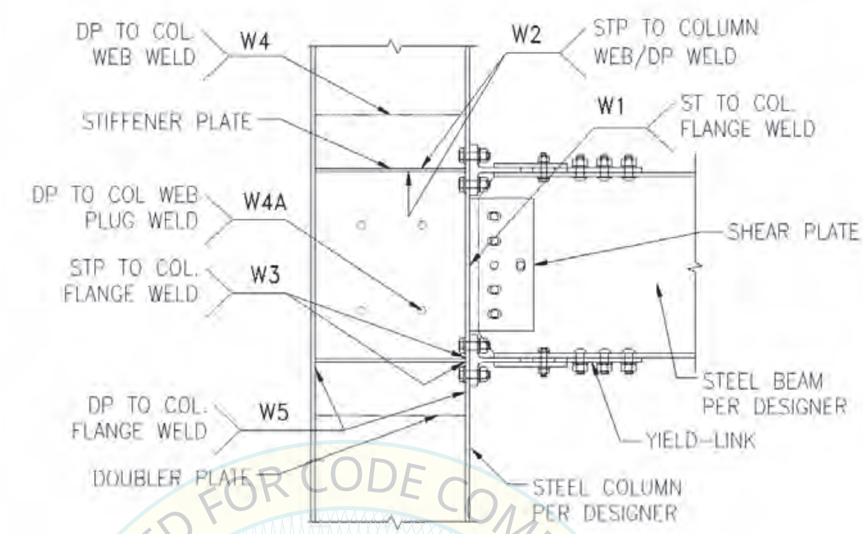
On the ***Frame Elevation sheets, details or schedules***:

a. Link flange-to-column flange bolts can be:

   1. Snug-tight, or
   2. Pretensioned

b. Link flange-to column flange bolts shall be ASTM F3125 Grade A325-N (snug-tight) or F1852-N (pretensioned)

d. Yield Link Connection Welding



In addition to the moment frame elevation output, a weld table is included under each of the frame elevations (see example below).

Elev. ID	Grid ID	Story	Column Unique Name	Column Size	Left ST Thk (in)	Right ST Thk (in)	STP Thk (in)	DP Thk (in)	Left ST Weld W1A (in)	N_sides of W1A	Right ST Weld W1B (in)	N_sides of W1B	STP to Col. Web W2 (in)	N_sides of W2	STP to Col. Flg. W3 (in)	N_sides of W3
1	B	Story2	113	W24X131	N/A	1/2	1/2	N/A	N/A	N/A	5/16	2	3/16	2	3/16	2
1	C	Story2	119	W24X162	1/2	1/2	1/2	N/A	5/16	2	5/16	2	3/16	2	3/16	2
1	D	Story2	125	W24X131	1/2	N/A	1/2	N/A	5/16	2	N/A	N/A	3/16	2	3/16	2
1	B	Story1	114	W24X131	N/A	1/2	1/2	N/A	N/A	N/A	5/16	2	3/16	2	3/16	2
1	C	Story1	120	W24X162	1/2	1/2	1/2	N/A	5/16	2	5/16	2	3/16	2	3/16	2
1	D	Story1	126	W24X131	1/2	N/A	1/2	N/A	5/16	2	N/A	N/A	3/16	2	3/16	2

<u>d1. Shear plate-to-column flange welds (W1):</u>

On the ***Frame Elevation sheets, details or schedules***:

a. Weld is Non-Demand Critical; it can be made with either:
   1. Double sided fillet welds (shown on weld table output)
   2. PJP weld
   3. CJP weld
   4. For (2) shear plates: 1st plate double sided fillet weld, 2nd plate PJP field weld

<u>d2. Continuity/Stiffener plate-to-column flange welds (W2):</u>

On the ***Frame Elevation sheets, details or schedules***:

a. Stiffener plate-to-column web weld is Non-Demand Critical, it can be made with either:
   1. Double sided fillet welds (shown on weld table output)
   2. PJP weld
   3. CJP weld

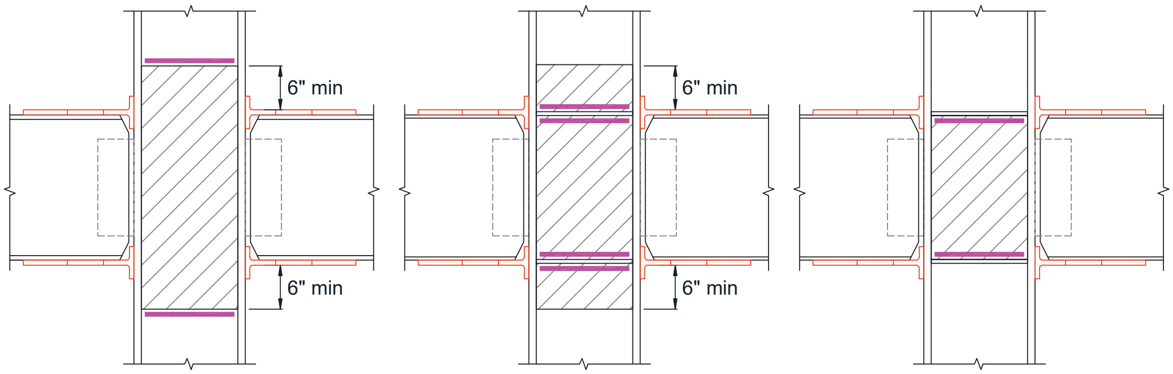
<u>d3. Continuity/Stiffener plate-to-column flange welds (W3):</u>

On the ***Frame Elevation sheets, details or schedules***:

a. Stiffener plate-to-column flange weld is Non-Demand Critical, it can be made with either:
   1. Double sided fillet welds (shown on weld table output)
   2. PJP weld
   3. CJP weld

d4. Doubler plate-to-column web/stiffener (if required) welds (W4):

On the ***Frame Elevation sheets, details or schedules***:



a. Doubler plate-to- column web/stiffener weld can be made with either:
   1. Fillet welds
   2. CJP weld (Demand Critical)

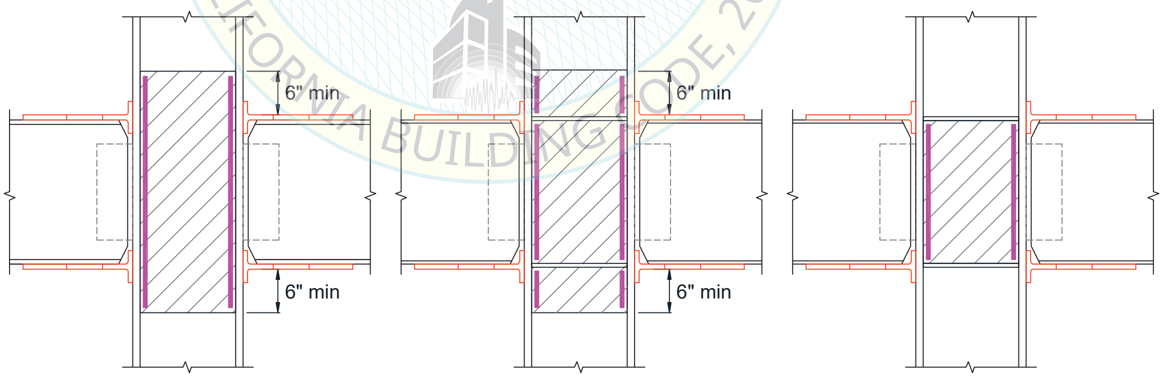
d4A. Doubler plate-to-column web plug welds (W4a):

On the ***Frame Elevation sheets, details or schedules***:

a. Doubler plate-to- column web:
   1. Plug weld

d5. Doubler plate-to-column flange (if required) welds (W5):

On the ***Frame Elevation sheets, details or schedules***:



a. Doubler plate-to- column flange weld can be made with either:
   1. Fillet welds
   2. PJP weld
   3. CJP weld (Demand critical)

SECTION 3: SPECIFICATIONS

SECTION 05 12 24

PREFABRICATED SPECIAL STEEL MOMENT CONNECTIONS

PART 1 GENERAL

1.1 SECTION INCLUDES

A. Prefabricated steel moment frame connections are designed and fabricated as a part of a special steel moment frame that will support gravity loads and resist lateral in-plane wind or earthquake loads.

B. As part of the Yield-Link Moment Connection Simpson Strong-Tie supplies (2) Yield-Links, (2) buckling restraint plates and (4) spacer plates per steel moment connection. Continuity plates, shear plates, doubler plates and all fasteners are provided by others as noted in the Structural Drawings.

1.2 RELATED SECTIONS

A. Section 05 12 00 – Structural Steel Framing

1.3 SUBMITTALS:

A. Shop and Erection Drawings: Show location, fabrication and assembly of structural beam and column with Yield-Link Moment Connections.

B. On each sheet of structural drawings/shop detail drawings that contains technical information showing the Simpson Strong-Tie Yield-Link Moment Connections, the following notice of intellectual property must be fixed before release for intended use:

Simpson Strong-Tie® Strong Frame® and Yield-Link® Structural Fuse are protected under one or more of the following patents and applications: US patent no. 8,001,734 B2, US patent no. 8,375,652 B2, US patent publication no. 2015/0159362, US patent publication no. 2017/0138043, and US patent publication no. 15/935,412 and must be supplied or licensed thru Simpson Strong-Tie Company Inc. Other US and international patents pending. Yield-Link moment connection is manufactured and protected under US patent no. 10,669,718 B2 and cannot be duplicated or fabricated without expressed, written permission from Strong Strong-Tie Co., Inc.

C. Mill certificates of Yield-Links, spacer plates and buckling restraint plates.

1.4 QUALITY ASSURANCE

A. Comply with the applicable provisions of the following specifications and documents:
   1. AISC 303-16 – Code of Standard Practice for Steel Buildings and Bridges
   2. AISC 358s2-20 - Prequalified Connections for Special and Intermediate Steel Moment Frames for Seismic Applications
   3. ASTM F3125-2015 – Standard Specification for High Strength Structural Bolts and Assemblies.
   4. ASTM A992-2020 –Structural Steel Shapes
   5. AWS D1.1-2020 – Structural Welding Code - Steel
   6. AWS D1.8-2016 – Structural Welding Code Seismic Supplement
   7. ESR-2802 – Simpson Strong-Tie Strong Frame Steel Moment Frame Connection
   8. ICC-ES AC129 – Acceptance Criteria for Steel Moment Connection Systems

B. Conduct pre-installation conference at project site.

1.5 DELIVERY, STORAGE, AND HANDLING

A. Deliver products to job site in manufacturer's or distributor's packaging undamaged, complete with installation instructions.

B. Protect and handle materials in accordance with manufacturer's recommendations to prevent damage or deterioration.

C. Store fasteners in a protected place in sealed containers in accordance with manufacturer's requirements.

PART 2 PRODUCTS

2.1 MANUFACTURERS

A. Manufacturer: Simpson Strong-Tie Company, Inc.

2.2 MATERIALS

A. Structural Steel:

1. W-shape for beam and columns: ASTM A992, unless noted otherwise (U.N.O.)
2. Built-up column: W-section (ASTM A992), T-Section (cut from ASTM A992 W-section)
3. Shear plate, Stiffener Plate, Doubler Plate: ASTM A572 Gr. 50 (U.N.O.)

B. Yield-Link Connection:

1. Yield-Links® (W-links): ASTM A992
2. Yield-Links (Welded): ASTM A572, Grade 50
3. Buckling restraint plate (BRP): ASTM A992 or ASTM A572 Gr 50, unless noted otherwise
4. Spacer plates: ASTM A992 or ASTM A572 Gr 50, unless noted otherwise

C. Weld Filler Metal:

1. Low hydrogen type conforming to AWS D1.1 Table 3.1, with a minimum yield of 70 ksi.
2. Notch toughness meet 20-lb-ft at 0° F; in addition, demand critical (DC) welds meet CVN toughness of 40-lb-ft at 70° F per AWS D1.8.

D. Bolts:

1. Yield-Link flange-to-column flange: ASTM F3125 Grade A325-N or F1852-N
2. Yield-Link stem-to- beam flange: ASTM F3125 Grade A490 or F2280
3. Shear Plate-to-beam web: ASTM F3125 Grade A325, A490, F1852 or F2280 (X or N type per design)
4. BRP-to-beam flange bolts: ASTM F3125 Grade A325-N (A325-X for YL8-6 and YL8-6-15)

E. Finishes:

1. Orange paint for Yield-Links (except at the Yield-Link to beam flange faying surface)
2. Gray Primer for buckling restraint plates (BRP) and spacer plates (SP)

2.3 FABRICATION

A. Yield-Link Connection:

1. Shop assembly for welded links to occur per the manufacturer's approved production drawings.
2. Fabrication tolerances per AISC 303 and manufacturer requirements that are more stringent in fabrication of the Yield-Links.

3. The manufacturer's identification shall be stamped into the metal or a label may be attached to the part with adhesive. Three sets of numbers are noted in the link stem: 1) Yield-Link ID; 2) 6 digits of batch number (provides for traceability of the heat number from a mill cert) and; 3) 6 digits of the production date.
4. Yield-Link moment connection is manufactured and protected under US patent no. 10,669,718 B2 and cannot be duplicated or fabricated without expressed, written permission from Strong Strong-Tie Co., Inc.

B. Structural Steel:
1. Fabricate in accordance with AISC 303, AISC 360 and AISC 341.
2. Thermal cutting of bolt holes in the beam/column receiving the Yield-Link connection is not acceptable.

2.4 PRIMING, PAINTING AND GALANIZING

A. Hot-Dip Galvanized Finish: Galvanizing of the Yield-Links is not permitted.

B. Beam flange surfaces in contact with the Yield-Link bolted connection region shall be unpainted. Remove loose rust and mill scale and spatter, slag, or flux deposits. Prepare surface by either SSPC-SP 2 (Hand Tool Cleaning) or SSPC-SP 3 (Power Tool Cleaning).

2.5 DESIGN AND TESTING

A. Connection design shall be per Design Procedure outlined in Annex A of ICC-ESR 2802 or AISC 358 Chapter 12

B. Testing shall be performed as per ICC-ES Acceptance Criteria 129 (AC129).

C. Testing shall be conducted under the supervision of an independent laboratory.

PART 3 EXECUTION

3.1 EXAMINATION

A. Yield-Link moment connection shall be installed on structural steel beam and columns per manufacturer's instructions or Engineer's construction documents.

B. Verify that the dimensions in the beam and column are detailed to receive the specified Yield-Link connection.

3.2 INSTALLATION

A. Top and bottom links for each Yield-Link® connection shall have the same heat number (1st set of number noted in the link stem per Section 2.3.C)

B. Spacer plates shall match the thickness of the link as shown in Table 1.1

C. Buckling restraint plates shall match the Yield-Link Model per Table 1.1.

D. All specified fasteners must be installed according to the manufacturer's instructions or Engineer's construction documents.

E. Install all specified fasteners before loading the Yield-Link Moment Connections.

F. Bolts connecting the Yield-Link to beam and column must be tightened in accordance with the manufacturer's installation instructions or Engineer's construction documents.

G. Do not overload by exceeding the manufacturer's allowable load values, load values obtained from the manufacture's software plugins and design tools.

H. The manufacture's patent label shall be applied adjacent to each moment connection with Yield-Links.

3.3 FIELD QUALITY CONTROL

A. Determine that the proper part is being used in the correct application and has been fabricated by the approved manufacturer by observation of the manufacture's patent label applied to each moment connection near the beam –to-column moment connection.

B. Field bolting of Yield-Link Moment Connection shall be installed in accordance with the manufacturer's instructions or Engineer's construction documents.

C. Structural inspections shall be AISC 360, AISC 341 and per SEOR project specific requirements.

D. Structural inspections shall follow CBC Table 1705A.2.1 requirements for HCAI/OSHPD projects.

3.4 FIELD CONNECTIONS (BOLTING)

A. Yield-Link flange-to-column flange: Snug-tight or pretensioned

B. Yield-Link stem-to- beam flange: Pretensioned

C. Shear Plate-to-beam web: Snug-tight or pretensioned

D. BRP-to-beam flange bolts: Snug-tight

3.5 FIELD MODIFICATIONS

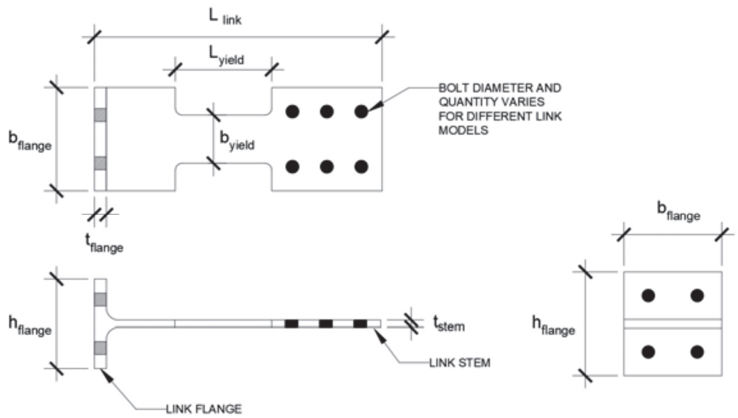
A. Do not cut or enlarge the existing holes in the Yield-Link, BRP, or spacer plates.

B. Do not weld to protected zone as indicated in ESR-2802 or AISC 358.

3.6 REPAIRS

A. Repairs of damaged Yield-Links, spacer plates and buckling restraint plates: All repairs shall be approved by the Structural Engineer of Record and Simpson Strong-Tie.

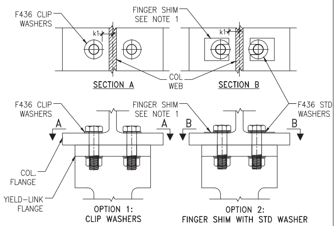
Table 1.1: Simpson Strong-Tie® Strong Frame® Yield-Links®



Yield-Link® ID [1]	Yield Link® Geometry							Design Information				BRP ID	Spacer Plate ID
	t_{stem}	t_{flange}	b_{yield}	L_{yield}	L_{link}	b_{flange}	h_{flange}	$P_{y\ link}$ kips	K_{eff} kip/in.	Min. Beam Depth	Max. Beam Depth		
YL4-2	1/2"	7/8"	2"	7"	1'-6 5/8"	6-1/4"	5-3/4"	50.0	2970	W12	W18	BRP4C	SP4C
YL4-2.5			2.5"					62.5	3468				
YL4-3			3"			6-1/2"		75.0	3960				
YL4-2.25			2.25"			7"		56.25	3337			BRP4A	SP4
YL4-2.875			2.875"					71.88	3953				
YL4-3.5			3.5"		1'-9 3/8"	8"		87.50	4460			BRP4B	
YL4-3.75			3.75"					93.75	4651				
YL4-4			4"					100.00	4831				
YL4-2.25-10		13/16"	2.25"	10"	1'-9 9/16"	7"		56.25	2554	W16	W27	BRP4A-10	SP4-10
YL4-2.875-10			2.875"					71.88	3077				
YL4-3.5-10			3.5"		2'-5/16"	8"		87.50	3529			BRP4B-10	
YL4-3.75-10			3.75"					93.75	3701				
YL4-4-10			4"					100.00	3865				
YL6-2.5	3/4"	1-1/4"	2.5"	10"	2'-3 1/2"	6-1/2"	9-1/4"	93.75	3426	W16	W27	BRP6D	SP6D
YL6-3			3"			8"		112.50	4149			BRP6A	SP6
YL6-3.5			3.5"					131.25	4564				
YL6-4			4"					150.00	4933				
YL6-4.5			4.5"			10"		168.75	5801			BRP6B	
YL6-5			5"					187.50	6167				
YL6-5.5			5.5"			12"		206.25	7037			BRP6C	
YL6-6			6"					225.00	7400				
YL6-3-13			3"	13"	2'-6 1/2"	8"		112.50	3484	W24	W36	BRP6A-13	SP6-13
YL6-3.5-13			3.5"					131.25	3868				
YL6-4-13			4"					150.00	4216				
YL6-4.5-13			4.5"			10"		168.75	4925			BRP6B-13	
YL6-5-13			5"					187.50	5270				
YL6-5.5-13			5.5"			12"		206.25	5981			BRP6C-13	
YL6-6-13			6"					225.00	6324				
YL8-4	1"	1-13/16"	4"	13"	2'-7 1/16"	9"	10-3/4"	200.00	6034	W24	W36	BRP8A	SP8
YL8-4.5			4.5"					225.00	6524				
YL8-5			5"		2'-10 7/16"	12"		250.00	7698			BRP8B	
YL8-5.5			5.5"					275.00	8213				
YL8-6			6"					300.00	8698				
YL8-4-15			4"	15"	2'-9 1/16"	9"		200.00	5465	W24	W36	BRP8A-15	SP8-15
YL8-4.5-15			4.5"					225.00	5931				
YL8-5-15			5"		3'-7/16"	12"		250.00	6959			BRP8B-15	
YL8-5.5-15			5.5"					275.00	7446				
YL8-6-15			6"					300.00	7908				

END OF SECTION 05 12 24

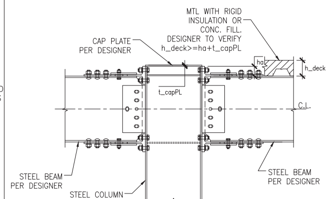
SECTION 4: STRUCTURAL DETAILS



NOTE:
1. FINGER SHIM SHALL BE STRUCTURAL GRADE STEEL MATERIAL. TRIM TO FIT AS REQUIRED.
2. K1 VALUE FOR STEEL SHAPE VARIES. BOLT WASHER/SHIM SHALL BEAR FLAT ON COL. FLANGE.
3. SEE RCSC SPECIFICATION ON USE OF WASHERS ON SLOPING SURFACES

INST1A-17 [06-13-2022]

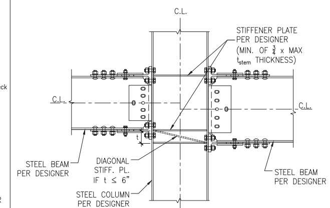
BOLT BEARING NEAR COLUMN WEB — 17



NOTE:
1. SEE 5/– OR 9/– FOR INFO NOT NOTED
2. SEE 8/– FOR ha VALUES FOR DIFFERENT LINK SIZES

INST1A-13 [06-13-2022]

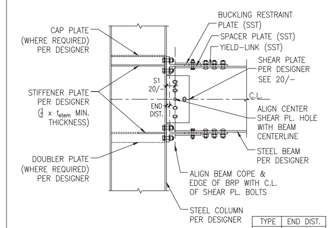
ROOF CONN. W/BEAMS ON EA. SIDE — 13



NOTE:
1. SEE 5/– FOR INFO NOT NOTED

INST1A-09 [06-13-2022]

MULTI-STORY COLUMNS (2 BEAMS) — 9

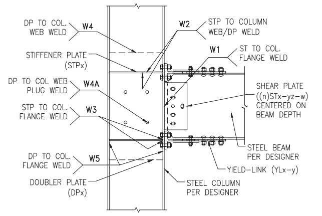


TYPE	END DIST.
YL4	1.00"
YL6	1.75"
YL8	2.25"

NOTE:
SEE 6/– FOR BOLTING REQUIREMENTS
SEE 7/– FOR PROTECTED ZONE AT YIELD-LINK CONNECTION

INST1A-05 [06-13-2022]

MULTI-STORY COLUMNS (1-BEAM) — 5

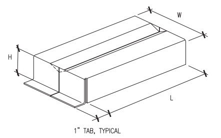


LEGEND:
DPx: x IS x/8-IN. THICK DOUBLER PLATES (i.e. DP4 IS 1/2" THICK DOUBLER PL.)
STPx: x IS x/8-IN. THICK STIFFENER/CONTINUITY PLATE
YLx-y: x IS x/8-IN THICK LINK STEM, y IS THE LINK YIELDING WIDTH
(n)STx-yz-w: n IS THE NUMBER OF SHEAR PLATES WHEN (2) ARE REQUIRED
x IS x/8-IN THICK SHEAR PLATE, y IS y/8-IN DIM. BOLT,
z IS BOLT GR/THREAD COND.: A=A325N, B=A325X, C=A490N, D=A490X
w IS THE NUMBER OF HORIZONTAL BOLTS

NOTES:
1. SEE 20/YL-INST1A FOR SHEAR PLATE GEOMETRY DETAILS
2. SEE 10/YL-INST2A FOR DOUBLE SHEAR PLATE DETAIL, (2)STx-yz-w,
3. SEE 11/YL-INST2A FOR STIFFENER PLATE DETAILS
4. SEE 12/YL-INST2A FOR DOUBLER PLATE WELDING DETAILS
5. SEE 17/YL-INST2A FOR COLUMN FLANGE HOLE DETAILS
6. YIELD-LINKS, BUCKLING RESTRAINT PLATES AND SPACER PLATES PROVIDED BY SIMPSON STRONG-TIE. ALL OTHER PLATES, STEEL AND FASTENERS PROVIDED BY OTHERS

INST1A-19 [06-13-2022]

YIELD-LINK CONNECTION LEGEND — 19

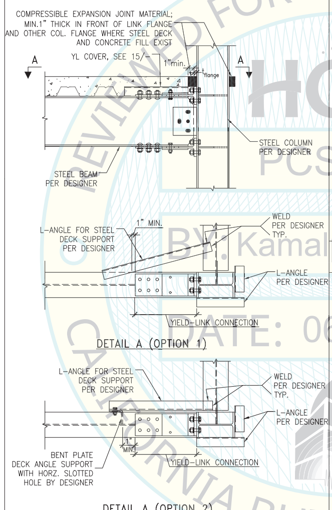


YIELD-LINK ID	CONC. COVER MODEL ID	DIMENSIONS (IN.) WIDTH (W)	LENGTH (L)	HEIGHT (H)
YL4-2, YL4-2.5, YL4-3	YLC-4-6	6-5/8"	18-7/8"	2"
YL4-2.25, YL4-2.25-10; YL4-2.875, YL4-2.875-10	YLC-4-7	7-1/8"	21-7/8"	
YL4-3.5, YL4-3.5-10; YL4-3.75, YL4-3.75-10; YL4-4, YL4-4-10	YLC-4-8	8-1/8"	24-5/8"	
YL6-2.5	YLC-6-6	6-5/8"	27"	2-3/4"
YL6-3; YL6-3.5; YL6-4	YLC-6-8	8-1/8"	27"	
YL6-3-13; YL6-3.5-13; YL6-4-13	YLC-6-8A	8-1/8"	30"	3"
YL6-4.5; YL6-5	YLC-6-10	10-1/8"	27"	2-3/4"
YL6-4.5-13; YL6-5-13	YLC-6-10A	10-1/8"	30"	3"
YL6-5.5; YL6-6	YLC-6-12	12-1/8"	27"	2-3/4"
YL6-5.5-13; YL6-6-13	YLC-6-12A	12-1/8"	30"	3"
YL8-4, YL8-4-15; YL8-4.5, YL8-4.5-15	YLC-8-9	9-1/8"	32-1/8"	3"
YL8-5, YL8-5-15; YL8-5.5, YL8-5.5-15; YL8-6, YL8-6-15	YLC-8-12	12-1/8"	35-1/2"	

NOTES:
1. DIMENSIONAL TOLERANCE SHALL BE +/- 1/16"
2. THICKNESS:
2.1. 18 GAUGE, ASTM A653 GRADE 33 STEEL FOR WIDTH (W) LESS THAN 12"
2.2. 16 GAUGE, ASTM A653 GRADE 33 STEEL FOR WIDTH (W) EQUAL TO OR GREATER THAN 12"
3. MIN. BEND RADIUS: 1T
4. FINISH: GALVANIZED/ZMAX COATING
5. ALTERNATE CONCRETE COVER BY OTHERS, SEE DETAIL 20/YL-INST2A

INST1A-15 [06-13-2022]

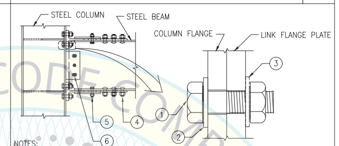
YIELD-LINK CONCRETE COVER — 15



NOTE:
1. SEE STRUCTURAL DRAWINGS FOR OTHER SUPPORT OPTIONS BY DESIGNER

INST1A-11 [06-13-2022]

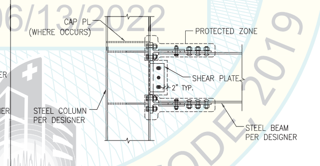
STEEL DECK SUPPORT AT CONN. — 11



NOTES:
1. LINK FLANGE-TO-COLUMN FLANGE BOLTS CAN BE SNUG-TIGHT OR PRETENSIONED
2. AT COL. FLANGE SLOTTED HOLES:
A) SNUG-TIGHT JOINT: USE ASTM F436 WASHER OR $\frac{5}{16}$ IN. THICK PL WASHER.
B) PRETENSIONED JOINT: USE $\frac{5}{16}$ IN. THICK PL WASHER OR CONT. BAR PER RCSC.
3. AT LINK FLANGE HOLES:
A) SNUG-TIGHT JOINT: WASHER NOT REQUIRED.
B) PRETENSIONED JOINT: USE ASTM F436 WASHER.
4. LINK STEM-TO-BEAM FLANGE BOLTS SHALL BE PRETENSIONED.
A) USE ASTM F436 WASHER PER PRETENSIONING METHOD
5. BUCKLING RESTRAINT PLATE-TO-BEAM FLANGE BOLTS MUST BE SNUG-TIGHT. NO WASHERS REQUIRED AT THE BEAM FLANGE SIDE. USE ASTM F436 WASHER AT BRP SIDE.
6. SHEAR PLATE TO BEAM WEB BOLTS CAN BE SNUG-TIGHT OR PRETENSIONED.
A) AT SHEAR PLATE SLOTTED HOLES: USE ASTM F436 WASHER OR $\frac{5}{16}$ IN. THICK PL WASHER FOR SNUG-TIGHT JOINTS.
B) USE USE $\frac{5}{16}$ IN. THICK PL WASHER OR CONT. BAR PER RCSC FOR PRETENSIONED JOINTS. AT BEAM WEB STANDARD HOLES: WASHERS NOT REQUIRED FOR SNUG-TIGHT JOINTS. USE ASTM F436 WASHER FOR PRETENSIONED JOINTS.
7. PRETENSIONING METHODS CAN BE: TURN-OF-NUT; CALIBRATED WRENCH; TWISTED OFF TYPE; OR BY USE OF DIRECT TENSION INDICATING WASHERS PER RCSC. WASHER REQUIREMENTS VARY PER PRETENSIONING METHOD.

INST1A-06 [06-13-2022]

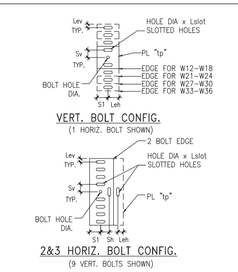
CONNECTION BOLTING DETAIL — 6



NOTE:
1. PROTECTED ZONE INCLUDES THE FOLLOWING ELEMENTS:
A. LINK FLANGE AND LINK STEM
B. BRP PLATES
C. BEAM FLANGE CONTACT AREAS CONNECTED TO THE LINK STEM
D. COLUMN FLANGE CONTACT AREAS CONNECTED TO THE LINK FLANGE
E. SHEAR PLATE AND BEAM WEB AT 2" AROUND SHEAR PLATE
F. LINK STEM, LINK FLANGE, BRP AND SHEAR PLATE BOLTS
2. PROTECTED ZONE SHALL BE MARKED BY USING ANY NON-DESTRUCTIVE METHODS PRIOR TO AND AFTER APPLICATION OF SPRAYED ON FIRE-RESISTING MATERIAL
3. NO ATTACHMENT SHALL BE MADE WITHIN THE PROTECTED ZONE.

INST1A-07 [06-13-2022]

PROTECTED ZONE — 7



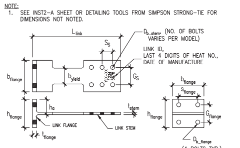
BEAM SIZE	HOLE DIA.	n VERT BOLTS	n HORZ BOLTS	Sv	Sh	Lev	Leh	Lslot	tp
W36	y+1/16"	9	W	2 7/8"	2 3/4"	1 1/4"	1 3/4"	2 7/8"	X
W33				2 1/2"					
W30		7	W	2 3/4"				2 3/8"	
W27				2 5/16"					
W24	y+1/16"	5	W	2 3/4"	2 3/4"	1 1/8"	1 3/4"	1 7/8"	X
W21				2 3/8"					
W18		3	W	3 1/4"				1 1/2"	
W16				2 5/8"					
W14				2 3/4"					
W12				2 3/16"					

1. NUMBER OF VERTICAL AND HORIZONTAL BOLTS SHOWN FOR REFERENCE ONLY. REFER TO STRUCTURAL DRAWINGS FOR PROJECT SPECIFIC REQUIREMENTS.
2. SHEAR PLATE THICKNESS (x), BOLT SIZE (y) AND QUANTITY (w) PER DESIGNER, SEE FRAME ELEVATION SHEETS.
3. S1=2.75" FOR YL4 LINKS; S1= 3.5" FOR YL6 LINKS; S1=4.0" FOR YL8 LINKS
4. BOLT HOLES IN THE BEAM FLANGES AND BEAM WEB SHALL BE STANDARD SIZE HOLES

INST1A-16 [06-13-2022]

TYPICAL SHEAR PLATE DETAIL — 20

NOTE:
1. SEE INST2-A SHEET OR DETAILING TOOLS FROM SIMPSON STRONG-TIE FOR DIMENSIONS NOT NOTED.



TYPE	t_{flange}	t_{stem}	h_{flange}	G_{flange}	h_w
YL4	0.875"	[illegible]	[illegible]	[illegible]	[illegible]
YL4-10	0.83"	[illegible]	[illegible]	[illegible]	[illegible]
YL6/YL6-13	1.25"	[illegible]	[illegible]	[illegible]	[illegible]
YL8/YL8-15	1.81"	[illegible]	[illegible]	[illegible]	[illegible]

SIMPSON STRONG-TIE® STRONG FRAME® AND THE YIELD-LINK® STRUCTURAL FUSE ARE PROTECTED UNDER ONE OR MORE OF THE FOLLOWING US PATENTS AND APPLICATIONS: US PATENT NO. 8,001,734 B2, US PATENT NO. 8,375,652 B2, AND US PATENT PUBLICATION NO. 2015/0159362, AND MUST BE SUPPLIED OR LICENSED THROUGH SIMPSON STRONG-TIE.

STANDARD YIELD-LINKS: LINK ID	L_{link}	b_{yield}	b_{flange}	s_{flange}	D_{b_flg}	D_{b_stem}	No.of Bolts[1]	BRP Model	SPACER Model
YL4-2	1'-6 5/8"	2"	6 1/4"	3 1/2"	7/8"	7/8"	4	BRP4C	SP4C
YL4-2.5		2 1/2"							
YL4-3		3"	6 1/2"						
YL4-2.25		2 1/4"	7"	4 1/2"				BRP4A	SP4
YL4-2.875		2 7/8"							
YL4-3.5	1'-9 3/8"	3 1/2"	8"	5 1/4"			6	BRP4B	
YL4-3.75		3 3/4"							
YL4-4		4"							
YL6-2.5	2'-3 1/2"	2 1/2"	6 1/2"	4"	7/8"	7/8"	6	BRP6D	SP6D
YL6-3		3"	8"	5"	1"	1"		BRP6A	SP6
YL6-3.5		3 1/2"							
YL6-4		4"			1 1/4"				
YL6-4.5		4 1/2"	10"	6"		1 1/4"		BRP6B	
YL6-5		5"							
YL6-5.5		5 1/2"	12"					BRP6C	
YL6-6		6"			1 3/8"				
YL8-4	2'-7 1/16"	4"	9"	5 1/4"	1 1/4"	1 1/4"	6	BRP8A	SP8
YL8-4.5		4 1/2"			1 3/8"				
YL8-5	2'-10 7/16"	5"	12"	6"			8	BRP8B	
YL8-5.5		5 1/2"			1 1/2"				
YL8-6		6"							

EXTENDED YIELD-LINKS: LINK ID	L_{link}	b_{yield}	b_{flange}	s_{flange}	D_{b_flg}	D_{b_stem}	No.of Bolts[1]	BRP Model	SPACER Model
YL4-2.25-10	1'-9 9/16"	2 1/4"	7"	4 1/2"	7/8"	7/8"	4	BRP4A-10	SP4-10
YL4-2.875-10		2 7/8"							
YL4-3.5-10	2'-5/16"	3 1/2"	8"	5 1/4"			6	BRP4B-10	
YL4-3.75-10		3 3/4"							
YL4-4-10		4"							
YL6-3-13	2'-6 1/2"	3"	8"	5"	1"	1"	6	BRP6A-13	SP6-13
YL6-3.5-13		3 1/2"							
YL6-4-13		4"			1 1/4"				
YL6-4.5-13		4 1/2"	10"	6"		1 1/4"		BRP6B-13	
YL6-5-13		5"							
YL6-5.5-13		5 1/2"	12"					BRP6C-13	
YL6-6-13		6"			1 3/8"				
YL8-4-15	2'-9 1/16"	4"	9"	5 1/4"	1 1/4"	1 1/4"	6	BRP8A-15	SP8-15
YL8-4.5-15		4 1/2"			1 3/8"				
YL8-5-15	3'-7/16"	5"	12"	6"			8	BRP8B-15	
YL8-5.5-15		5 1/2"			1 1/2"				
YL8-6-15		6"							

1. NUMBER OF YIELD-LINK STEM BOLTS SHOWN IN THIS COLUMN. NUMBER OF YIELD-LINK FLANGE BOLTS=4 FOR ALL LINKS.
2. CUSTOM DESIGN LINKS AVAILABLE, CONTACT SIMPSON STRONG-TIE FOR MORE INFORMATION.
3. BEAM, COLUMN AND HOLE DIMENSIONAL TOLERANCE SHALL BE PER AISC CODE OF STANDARD PRACTICE FOR STEEL BUILDINGS AND BRIDGES (AISC 303-16)

INST1A-08 [06-13-2022]

YIELD-LINK GEOMETRIES — 8

GENERAL NOTES:
1. SIMPSON STRONG-TIE YIELD-LINK® MOMENT CONNECTION IS PROTECTED UNDER ONE OR MORE OF THE FOLLOWING U.S. PATENTS AND APPLICATIONS: U.S. PATENT NO. 8,001,734 B2, U.S. PATENT NO. 8,375,652 B2, AND U.S. PATENT PUBLICATION NO. 2015/0159362, AND MUST BE SUPPLIED OR LICENSED THROUGH SIMPSON STRONG-TIE. YIELD-LINK MOMENT CONNECTION IS MANUFACTURED AND PROTECTED UNDER U.S. PATENT NO. 10,669,718 B2 AND CANNOT BE DUPLICATED OR FABRICATED WITHOUT EXPRESSED, WRITTEN PERMISSION FROM SIMPSON STRONG-TIE CO., INC.
2. YIELD-LINK MOMENT CONNECTION IS MANUFACTURED AND TRADEMARKED BY "SIMPSON STRONG-TIE COMPANY INC." HOME OFFICE: 5956 W. LAS POSITAS BLVD., PLEASANTON, CA 94588 TEL: (800) 999-5099, FAX: (925) 847-1597. "SIMPSON STRONG-TIE COMPANY INC." IS AN ISO 9001 REGISTERED COMPANY.
3. SIMPSON STRONG-TIE® COMPANY, INC. RESERVES THE RIGHT TO CHANGE SPECIFICATIONS, DESIGNS, AND MODELS WITHOUT NOTICE OR LIABILITY FOR SUCH CHANGES.
4. USE OF A SIMPSON STRONG-TIE PRODUCT DOES NOT IMPLY THAT SIMPSON STRONG-TIE ENDORSES ANY PROJECT, STRUCTURE OR USE. NO LICENSE IS GRANTED WITH RESPECT TO ANY SIMPSON STRONG-TIE TRADEMARK OR OTHER INTELLECTUAL PROPERTY RIGHTS. WRITTEN PERMISSION MUST BE OBTAINED PRIOR TO USING ANY SIMPSON STRONG-TIE TRADEMARKS OR PROPRIETARY DOCUMENTS AND MATERIALS.
5. SIMPSON STRONG-TIE IS NOT AFFILIATED WITH, AND DOES NOT SPONSOR OR ENDORSE, THE SEOR, FABRICATOR, INSTALLER OR USERS OF THIS DRAWING, NOR DOES SIMPSON STRONG-TIE HAVE ANY JOINT VENTURE, PARTNERSHIP, AGENCY, EMPLOYMENT OR FIDUCIARY RELATIONSHIP WITH SUCH PERSONS.

DESIGN NOTES:
1. DESIGN FOR YIELD-LINK SPECIAL MOMENT CONNECTION SHALL BE IN ACCORDANCE WITH THE FOLLOWING:
 - 2019 CALIFORNIA BUILDING CODE (CBC)
 - AISC SPECIFICATION FOR STRUCTURAL STEEL BUILDINGS (ANSI/AISC 360-16)
 - AISC SEISMIC PROVISIONS (ANSI/AISC 341-16)
 - PREQUALIFIED CONNECTIONS FOR SPECIAL AND INTERMEDIATE STEEL MOMENT FRAMES FOR SEISMIC APPLICATIONS (ANSI/AISC 358s2-20)
 - ICC-ES ESR-2802
 - 2014 RCSC SPECIFICATION FOR STRUCTURAL JOINTS USING HIGH-STRENGTH BOLTS
 - STRUCTURAL WELDING CODE-STEEL (AWS D1.1-2015)
 - STRUCTURAL WELDING CODE SEISMIC SUPPLEMENT (AWD D1.8-2016)
2. IN ADDITION TO THE AISC 358s2-20 CHAPTER 12 LIMITS, THE FOLLOWING HCAI/OSHPD PREAPPROVAL LIMITATIONS SHALL APPLY:
 - A LINEAR ANALYSIS PROCEDURE SHALL BE USED FOR DESIGN OF SMF AND IMF SYSTEMS USING THE YIELD-LINK MOMENT CONNECTION WHEN PERMITTED IN ACCORDANCE WITH ASCE 7. NON-LINEAR PROCEDURE WILL BE CONSIDERED AN ALTERNATE SYSTEM.
 - ONLY T-STUB YIELD-LINKS ARE PERMITTED, END PLATE YIELD-LINKS ARE NOT PERMITTED.
 - THE BI-AXIAL DUAL-STRONG AXIS AND COLUMN MINOR AXIS CONFIGURATIONS OF THE MOMENT CONNECTION SHALL BE CONSIDERED AN ALTERNATIVE SYSTEM.
 - BEAM FLANGE WIDTH-TO-THICKNESS RATIO SHALL NOT EXCEED LAMDA-P OF 0.38*SQRT(E/FY) PER AISC 360 TABLE B4.1b CASE 11.
 - YIELD LINK STEM-TO-BEAM FLANGE CONNECTION BOLTS DO NOT SLIP UNDER WIND DESIGN DEMAND LOADS. YIELD-LINK STEM-TO-BEAM FLANGE CONNECTION SHALL BE DESIGNED TO PREVENT SLIP USING AISC 360 EQUATION J3-4, WHERE THE SLIP RESISTANCE MU IS TAKEN AS 0.3.
 - WIND DRIFT LIMITATIONS PER 2019 CBC SECTION 1609A.1.2 SHALL ALSO BE MET.
 - DOUBLE SHEAR PLATE CONNECTION IS PERMITTED TO INCREASE CONNECTION AXIAL CAPACITY FOR COLLECTOR LOADS. A PJP GROOVE WELD FOR SECOND SHEAR PLATE IS PERMISSIBLE DUE TO SPACE RESTRICTIONS.
3. USE OF THIS PRODUCT IS SUBJECT TO THE APPROVAL OF HCAI ALTERNATE MEANS OF COMPLIANCE TO MEET 2019 CBC SECTION 2205A.5 REQUIREMENTS.
4. THIS PRODUCT IS PART OF THE OVERALL LATERAL FORCE RESISTING SYSTEM OF THE STRUCTURE. DESIGN OF THE BUILDING'S LATERAL FORCE RESISTING SYSTEM, INCLUDING THE LOAD PATH TO TRANSFER LATERAL FORCES FROM THE STRUCTURE TO THE GROUND, IS THE RESPONSIBILITY OF THE STRUCTURAL ENGINEER OF RECORD (SEOR).
5. THE SEOR MUST SPECIFY THE REQUIRED COMPONENTS OF THE COMPLETE LOAD TRANSFER PATH INCLUDING DIAPHRAGMS, SHEAR TRANSFER, CHORDS AND COLLECTORS AND FOUNDATIONS.
6. ALL CONNECTED MEMBERS AND RELATED ELEMENTS SHALL BE DESIGNED BY THE SEOR.
7. INSTALLATION OF PRODUCT SHALL BE DONE IN CONFORMANCE WITH THESE DRAWINGS AND ICC-ES ESR-2802 AND AISC 358. THE PERFORMANCE OF MODIFIED PRODUCTS OR ALTERED INSTALLATION PROCEDURES ARE THE RESPONSIBILITY OF THE SEOR AND CONTRACTOR.

MATERIAL:
1. YIELD-LINKS: ASTM A992
2. BRP AND SPACER PLATES: ASTM A572 GR. 50
3. BEAMS AND COLUMNS MUST BE ROLLED W-FLANGE (A992) OR WELDED BUILT-UP I-SHAPE MEMBERS THAT MEETS AISC 360 SECTION A3 AND AISC 358 SECTION 2.3.2.
4. SHEAR PLATE, STIFFENER PLATE AND DOUBLER PLATE: ASTM A572 GR. 50 (U.N.O.)
5. LINK STEM-TO-BEAM FLANGE BOLTS SHALL BE ASTM F3125 A490 OR F2280 (X OR N) (PRETENSIONED)
6. LINK FLANGE-TO-COLUMN FLANGE BOLTS SHALL BE ASTM F3125 GRADE A325-N (SNUG-TIGHT) OR F1852-N (PRETENSIONED)
7. SHEAR PLATE-TO-BEAM WEB BOLTS SHALL BE ASTM F3125 GRADE A325, A490, F1852 OR F2280 (X OR N TYPE PER DESIGN)
8. BRP-TO-BEAM FLANGE BOLTS SHALL BE ASTM F3125 GRADE A325-N (A325-X FOR YL8-6 & YL8-6-15)
9. COMPRESSIBLE EXPANSION JOINT MATERIAL: SPECIFIED COMPRESSIVE STRENGTH SHALL BE LESS THAN 10 PSI UNLESS OTHERWISE INDICATED ELSEWHERE FOR THE PROJECT. SPECIFIED COMPRESSIVE STRENGTH MUST BE NO MORE THAN 100 PSI. RECOMMENDED MATERIALS AT NON-FIRE RATED ASSEMBLIES ARE RIGID FOAM TYPE EXPANDED POLYSTYRENE (EPS) OR TYPE EXTRUDED POLYSTYRENE (XPS). RECOMMENDED MATERIALS AT FIRE-RATED ASSEMBLIES ARE SPRAYED FIRE-RESISTIVE MATERIALS (SPRAY-APPLIED FIREPROOFING) WITH FIRE RESISTANCE DESIGN PARAMETERS AS REQUIRED FOR THE PROJECT.

YIELD-LINK CONNECTION WELDING REQUIREMENTS:
1. ALL SHOP AND FIELD WELDING ON COMPONENTS (SHEAR PLATE, STIFFENER PLATE, DOUBLER PLATE [IF REQUIRED]) OF THE YIELD-LINK CONNECTION SHALL BE DONE BY PERSONNEL QUALIFIED PER AWS D1.1 SECTION 4 PART C, AWS D1.8 ANNEX D (AS REQUIRED), AND AS APPROVED BY HCAI OR AHJ.
2. ALL WELDERS SHALL HAVE EVIDENCE OF PASSING THE AWS D1.1 STANDARD QUALIFICATION TESTING, AND SHALL BE CERTIFIED FOR THE WORK THEY ARE PERFORMING.
3. SPECIFIC WRITTEN WELDING PROCEDURE SPECIFICATIONS (WPS) FOR EVERY WELDING APPLICATION USED FOR THE YIELD-LINK CONNECTIONS IN THE PROJECT SHALL BE SUBMITTED TO THE SEOR, HCAI OR AHJ FOR REVIEW AND APPROVAL PRIOR TO START OF WORK. EACH WPS SHALL REFERENCE THE PREQUALIFIED WELDED JOINT DETAIL FROM SECTION 3 OF AWS D1.1.
4. IN LIEU OF ITEM 2, THE STEEL FABRICATOR MAY PROVIDE A PROCEDURE QUALIFICATION RECORD (PQR) FOR THE CONNECTION WELDS USED FOR THE PROJECT. THE PQR SHALL CONFORM TO THE REQUIREMENTS OF AWS D1.1 SECTION 4.
5. WPS (OR PQR) SHALL CONTAIN ALL THE ESSENTIAL VARIABLES USED IN THE WELDING PROCEDURE AND SHALL BE STAMPED/APPROVED BY AN AMERICAN WELDING SOCIETY (OR APPROVED EQUIVALENT) CERTIFIED WELDING INSPECTOR.
6. UNLESS NOTED OTHERWISE BY THE ENGINEER OF RECORD, ALL WELD FILLER METAL SHALL HAVE CLASSIFICATION STRENGTH (F_{EXX}) OF 70 KSI.
7. FOR NON-DEMAND CRITICAL WELDS (BEAM TO COLUMN SHEAR PLATE AND COLUMN STIFFENER/CONTINUITY PLATE), WELD FILLER METAL SHALL MEET THE REQUIREMENT OF AWS D1.1 AND D1.8 MINIMUM CHARPY V-NOTCH REQUIREMENT OF 20 FT*LBS AT 0°F.
8. FOR DEMAND CRITICAL WELDS, COLUMN SPLICES, DOUBLER PLATES (CJP WELDS) AND BASE PLATES, WELD FILLER METAL SHALL MEET THE REQUIREMENTS OF AWS D1.1 AND D1.8 MINIMUM CHARPY V-NOTCH REQUIREMENT OF 20 FT*LBS AT 0°F AND 40 FT*LBS AT 70°F.
9. IN ADDITION TO THE MINIMUM CHARPY V-NOTCH REQUIREMENT, ALL FILLER METAL SHALL MEET THE DIFFUSIBLE HYDROGEN LEVEL AS NOTED IN SECTION 6.1.3 OF AWS D1.8.
10. SUBMIT WELDER QUALIFICATION, WPS (OR PQR), AND FILLER METAL CERTIFICATION OF CONFORMANCE PROVIDED BY THE WIRE MANUFACTURE TO THE SEOR AND HCAI OR AHJ FOR APPROVAL PRIOR TO FABRICATION.
11. STEEL BASE METAL WHICH WELD METAL IS TO BE DEPOSITED SHALL BE FREE OF DUST, MOISTURE, GREASE SLAG, MILL SCALE AND OTHER FOREIGN MATERIAL THAT WOULD PREVENT PROPER WELD DEPOSIT.
12. FOR SHEAR PLATE WELDS, FILLET WELD TERMINATION MAY EXTEND TO THE ENDS OF THE SHEAR PLATE OR MAYBE STOPPED SHORT AT EACH END OF THE SHEAR PLATE FOR A LENGTH THAT'S THE SAME DIMENSION AS THE WELD SIZE. FILLET WELDS SHALL NOT BE WRAPPED AROUND TO THE TOP AND BOTTOM EDGES OF THE SHEAR PLATE.
13. FOR COLUMN STIFFENER/CONTINUITY PLATES, FILLET WELDS SHALL TERMINATE A DISTANCE OF ONE FILLET WELD SIZE FROM THE k DISTANCE + 1-1/2" FROM THE EXTERIOR COLUMN FLANGE, AND k1 DISTANCE + 1/2" FROM THE CENTERLINE OF THE COLUMN WEB.
14. ALL COMPLETE PENETRATION GROOVE WELDS (CJP) SHALL BE ULTRA-SONIC TESTED PER D1.1 AND D1.8 REQUIREMENTS AS APPLICABLE.
15. WHEN WELDING OF DOUBLER PLATES, CONTINUITY PLATES, OR STIFFENERS HAS BEEN PERFORMED IN THE K-AREA, THE WEB SHALL BE TESTED FOR CRACKS USING MAGNETIC PARTICLE TESTING. THE MAGNETIC PARTICLE TEST AREA SHALL INCLUDE THE K-AREA BASE METAL WITHIN 3" OF THE WELD.
16. CJP GROOVE WELD ON BASE METAL THICKER THAN 1-1/2" SHALL BE ULTRASONICALLY TESTED FOR DISCONTINUITIES (LAMELLAR TEARING). ANY BASE METAL DISCONTINUITIES FOUND SHALL BE ACCEPTED OR REJECTED ON THE BASIS OF CRITERIA OF AWS D1.1 TABLE 6.3.

YIELD-LINK CONNECTION BOLTING REQUIREMENTS:
1. BOLT HOLES SHALL BE MADE BY DRILLING, PUNCHING OR THERMAL CUTTING. MECHANICALLY GUIDED MEANS OF THERMALLY CUT HOLES SHALL HAVE A SURFACE ROUGHNESS PROFILE NOT EXCEEDING 1000 MICRO-INCHES AS DEFINED IN ASME B46.1.
2. FAYING SURFACES ADJACENT TO THE BOLT HEAD AND NUT SHALL BE FREE OF DIRT AND OTHER FOREIGN MATERIAL. BEAM FLANGE SURFACE CONNECT THE YIELD-LINK STEM SHALL BE FREE OF SCALE, COATING OR INADVERTANT OVERSPRAY (SEE DETAIL 1, 2, 5, 6 9 AND 13 ON YL-INST2A SHEET). BURRS LESS THAN OR EQUAL 1/16" IN HEIGHT ARE PERMITTED ON THE FAYING SURFACES OF ALL JOINTS.
3. HIGH STRENGTH BOLTS SHALL HAVE LOT TRACEABILITY, CHEMICAL AND MECHANICAL TEST RESULTS INDICATED ON THE MILL CERTIFICATION/TEST REPORTS. BOLT MILL CERTIFICATIONS SHALL BE SUBMITTED TO SEOR PRIOR TO ASSEMBLY OR ERECTION OF STRUCTURAL STEEL.
4. HIGH STRENGTH BOLTS SHALL BE STORE AND PROTECTED FROM MOISTURE AND DIRT IN CLOSED CONTAINERS.
5. TWIST-OFF TYPE BOLT (F1852 AND F2280) ASSEMBLIES (BOLT, WASHER AND NUT) SHALL NOT BE RE-LUBRICATED, EXCEPT BY THE MANUFACTURER.
6. A490 BOLTS, F1852 AND F2280 TWIST-OFF TYPE ASSEMBLIES SHALL NOT BE REUSED. TOUCHING UP OR RE-TIGHTENING BOLTS THAT MAY HAVE BEEN LOOSENED BY INSTALLATION OF ADJACENT BOLTS NEED NOT BE CONSIDERED AS A REUSE.
7. UP TO 1/4 INCH THICK FINGER SHIMS MAYBE USED WITHOUT RESTRICTION.
8. YIELD-LINK STEM TO BEAM FLANGE BOLTS SHALL BE INSTALLED AS PRETENSIONED. BOLTS CAN BE PRETENSIONED BY TWISTED-OFF-TYPE TENSION CONTROLLED BOLTS, DIRECT TENSION INDICATOR (DTI) WASHERS, TURN-OF-NUT METHOD, OR CALIBRATED WRENCH METHOD. SEE SPECIFICATION FOR STRUCTURAL JOINTS USING HIGH-STRENGTH BOLTS FOR REQUIREMENTS OF THESE PRETENSIONING METHODS.
9. SEE DETAIL 6/YL-INST1A FOR LINK FLANGE-TO-COLUMN FLANGE BOLTING, BEAM WEB-TO-SHEAR PLATE BOLTING AND BUCKLING RESTRAINT PLATE-TO BEAM FLANGE BOLTING REQUIREMENTS.
10. FOR PRETENSIONED JOINTS, MINIMUM BOLT PRETENSION VALUE SHALL BE PER RCSC TABLE 8.1.
11. BOLTING PRE-INSTALLATION VERIFICATION SHALL BE CONDUCTED TO VERIFY THE PRETENSIONING METHOD USED FOR THE SPECIFIC PROJECT CAN ACHIEVE THE MINIMUM BOLT TENSION VALUES.
12. BOLTS IN "X" CONNECTIONS SHALL HAVE THE THREADS EXCLUDED FROM THE SHEAR PLANE. BOLTS IN THE "N" CONNECTIONS MAY HAVE THE THREADS INCLUDED IN THE SHEAR PLANE.
13. MINIMUM BOLT LENGTHS USED SHALL BE SUCH THAT BOLT THREAD IS AT LEAST FLUSH WITH THE OUTER FACE OF THE NUT.

INSTALLATION, FIELD MODIFICATIONS AND REPAIRS:
THESE GENERAL INSTRUCTIONS FOR THE INSTALLER ARE PROVIDED TO ENSURE PROPER SELECTION AND INSTALLATION OF SIMPSON STRONG-TIE COMPANY INC. PRODUCTS AND MUST BE FOLLOWED CAREFULLY. THESE GENERAL INSTRUCTIONS ARE IN ADDITION TO THE SPECIFIC INSTALLATION INSTRUCTIONS AND NOTES PROVIDED FOR EACH PARTICULAR PRODUCT, ALL OF WHICH SHOULD BE CONSULTED PRIOR TO AND DURING INSTALLATION OF SIMPSON STRONG-TIE COMPANY INC. PRODUCTS.
1. PROPER PRODUCT INSTALLATION REQUIRES CAREFUL ATTENTION TO ALL NOTES AND INSTRUCTIONS. IN ADDITIONAL TO THE NOTES, WARNINGS, AND INSTRUCTIONS PROVIDED IN THE CATALOG, CONTRACTORS, INSTALLERS, DESIGNERS, ENGINEERS AND CONSUMERS SHOULD CONSULT THE SIMPSON STRONG-TIE COMPANY INC. WEB SITE AT WWW.STRONGTIE.COM TO OBTAIN ADDITIONAL INFORMATION FOR INSTALLATION, SPECIFICATIONS, CODE REPORTS, TECHNICAL FLIERS AND BULLETINS, FAQS, AND OTHER PERTINENT INFORMATION.
2. FABRICATION AND ERECTION TOLERANCE SHALL BE IN ACCORDANCE WITH AISC 303-16 CODE OF STANDARD PRACTICE FOR STEEL BUILDINGS AND BRIDGES.
3. ALL SPECIFIED FASTENERS MUST BE INSTALLED ACCORDING TO THE INSTRUCTIONS PROVIDED IN THE CATALOG, CODE REPORT, AND INSTALLATION DETAILS. INCORRECT FASTENER QUANTITY, SIZE, PLACEMENT, TYPE, MATERIAL, OR FINISH MAY CAUSE THE CONNECTION TO FAIL.
4. YIELD-LINKS SHALL NOT BE HOT-DIP GALVANIZED.
5. YIELD-LINKS SHALL BE INSTALLED WITH MATCHING HEAT NO. AT EACH CONNECTION.
6. ALL FIELD MODIFICATIONS AND REPAIRS MUST BE APPROVED BY THE SEOR
7. YIELD-LINK SIZE, SPACER AND BRP SHALL BE INSTALLED PER MANUFACTURER OR SEOR'S FRAME ELEVATION DRAWINGS.
8. DO NOT CUT OR ENLARGE EXISTING HOLES IN THE YIELD-LINK, BRP OR SPACER PLATES.

FIREPROOFING:
1. WHEN THE STEEL STRUCTURE IS REQUIRED TO MEET CERTAIN TYPE OF CONSTRUCTION RATING, THE YIELD-LINK CONNECTION SHALL HAVE A FIRE RATING SIMILAR TO THAT OF THE PRIMARY STRUCTURAL FRAME.
2. WHERE SPRAY FIRE-RESISTANT MATERIAL (SFRM) IS USED, MINIMUM THICKNESS OF SFRM FOR THE YIELD-LINK FLANGE AND STEM SHALL BE SIMILAR TO THAT OF THE BEAM AND COLUMN SECTIONS.
3. FOR CASES WHEN THE CONCRETE THICKNESS ABOVE THE YIELD-LINK COVER PLATE IS SHALLOWER THAN THE CONCRETE THICKNESS ABOVE THE METAL DECK RIB, YIELD-LINK AND COVER PLATE AT THE BEAM TOP FLANGE CAN BE COVERED WITH FIRE BARRIER PAINT WHICH HAS A COATING THICKNESS THAT PROVIDES THE SAME FIRE RATING PROTECTION AS THE PRIMARY STRUCTURAL FRAME. FIRE BARRIER PAINT SHALL BE TESTED AND CERTIFIED TO ASTM E119.
4. FOR HCAI/OSHPD PROJECTS, FINAL FIREPROOFING APPROVAL SHALL BE BY THE HCAI/OSHPD FIRE-LIFE SAFETY OFFICER.

QUALITY CONTROL AND QUALITY ASSURANCE:
1. STRUCTURAL STEEL INSPECTION SHALL FOLLOW CBC TABLE 1705A.2.1 REQUIREMENTS.
2. THE FABRICATOR AND ERECTOR SHALL BE RESPONSIBLE FOR QUALITY CONTROL (Q/C) BY ENSURING WORKMANSHIP MEETS THE REQUIREMENTS OF THE CONTRACT DOCUMENTS. THE OWNER'S REPRESENTATIVE SHALL BE RESPONSIBLE FOR QUALITY ASSURANCE (Q/A) INSPECTIONS. Q/C AND Q/A REQUIREMENTS SHALL BE IN ACCORDANCE WITH AISC 341 CHAPTER J. AND CBC 2019 CHAPTER 17A.
3. CONTACT SIMPSON STRONG-TIE® AT 800-999-5099 TO REQUEST YIELD-LINK MILL CERTS.
4. STEEL MOMENT FRAME CONNECTIONS UTILIZING THE SIMPSON STRONG-TIE YIELD-LINK CONNECTION SHALL HAVE AN IDENTIFICATION STICKER (PROVIDED BY SIMPSON) PLACED NEXT TO THE SHEAR PLATE BETWEEN THE LINKS FOR EACH CONNECTION.
5. YIELD-LINK SIZE, SPACER AND BRP SHALL BE INSTALLED PER MANUFACTURER OR SEOR'S FRAME ELEVATION DRAWINGS.
6. YIELD-LINKS SHALL BE INSTALLED WITH MATCHING HEAT NO. AT EACH CONNECTION.
7. THE REDUCED SECTION OF THE YIELD-LINK SHALL HAVE MAXIMUM SURFACE ROUGHNESS OF 250 MICRO-INCHES IN ACCORDANCE WITH ASME B46.1. WHEN REQUIRED, 3RD PARTY QUALITY ASSURANCE SHALL COORDINATE WITH SIMPSON STRONG-TIE IN DOCUMENTATION OF THE YIELD-LINK ROUGHNESS MEASUREMENT.

INST1A-04 [06-13-2022]

GENERAL NOTES — 4

NO.	DATE	REVISIONS

06/13/2022

SIMPSON STRONG-TIE, CO. INC.
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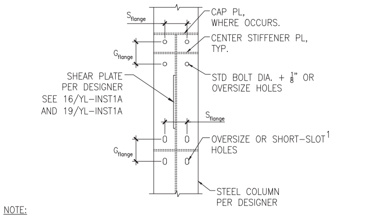
Fax: (925) 847-1597
Web site: www.strongtie.com
SIMPSON Strong-Tie
THERE IS NO EQUAL

YIELD-LINK® MOMENT CONNECTION
STEEL SPECIAL MOMENT FRAME
CONNECTION DETAILING INFORMATION

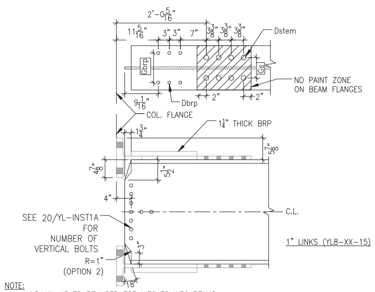
NAME: B.C.
DATE: 06-13-2022
SCALE: N.T.S.
SHEET:
YL-INST1A
JOB NO.



NOTE:
1. REFER TO RCSC FOR WASHER REQUIREMENTS ON SLOTTED HOLES
2. SEE DETAIL 17/INST1A FOR WASHER OPTIONS WHEN WASHER ENCROACH INTO COLUMN K REGION.
3. SEE 4/YL-INST2A FOR Sflange AND Gflange DIMENSIONS

INST2A-17 [06-13-2022]

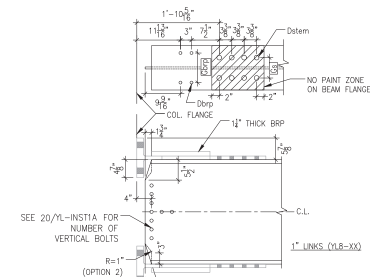
COLUMN FLANGE HOLES 17



NOTE:
1. YL8-XX-15 TO BE USED FOR W30 TO W36 BEAMS
2. SEE 4/YL-INST2A FOR Gs, Dbrp AND Dstem, TYP.

INST2A-13 [06-13-2022]

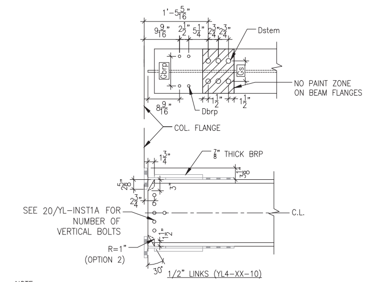
EXTENDED YL8 BEAM COPE/HOLE 13



NOTE:
1. YL8-XX TO BE USED FOR W24 TO W36 BEAMS
2. SEE 4/YL-INST2A FOR Gs, Dbrp AND Dstem, TYP.

INST2A-09 [06-13-2022]

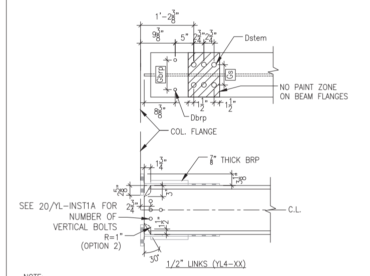
YL8 BEAM COPE/HOLE DETAILS 9



NOTE:
1. YL4-XX-10 TO BE USED FOR W21 TO W27 BEAMS
2. SEE 4/YL-INST2A FOR Gs, Dbrp AND Dstem, TYP.

INST2A-05 [06-13-2022]

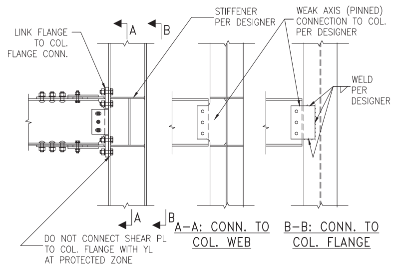
EXTENDED YL4 BEAM COPE/HOLE 5



NOTE:
1. YL4-XX TO BE USED FOR W12 TO W18 BEAMS
2. SEE 4/YL-INST2A FOR Gs, Dbrp AND Dstem, TYP.

INST2A-01 [06-13-2022]

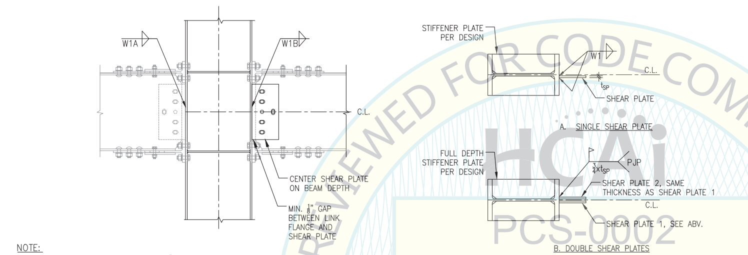
YL4 BEAM COPE/HOLE DETAILS 1



NOTE:
1. FOR HCAI/OSHPD PRE-APPROVAL, INTENT OF DETAIL IS TO ALLOW WELDING OF STIFFENER PLATE BETWEEN BOLTS. SECTION A AND B ARE PER SEOR.

INST2A-18 [06-13-2022]

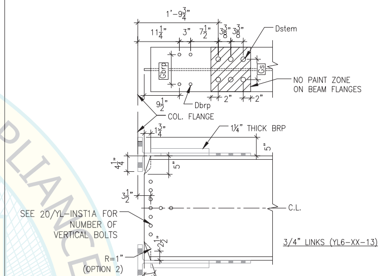
ORTHOGONAL CONN. TO COL. 18



NOTE:
1. W1 FILLET WELD SHALL BE MIN. 5/16 X SHEAR PLATE THICKNESS ON E.S.
2. SHEAR PLATE GEOMETRY SEE 20/YL-INST1A AND FRAME ELEVATION SHEETS

INST2A-10 [06-13-2022]

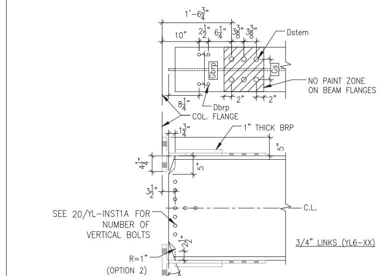
SHEAR PLATE DETAILS 10



NOTE:
1. YL6-XX-13 TO BE USED FOR W30 TO W36 BEAMS
2. SEE 4/YL-INST2A FOR Gs, Dbrp AND Dstem, TYP.

INST2A-06 [06-13-2022]

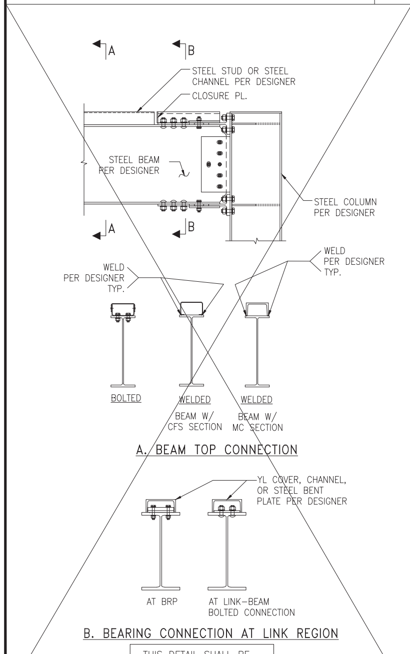
EXTENDED YL6 BEAM COPE/HOLE 6



NOTE:
1. YL6-XX TO BE USED FOR W16 TO W27 BEAMS
2. SEE 4/YL-INST2A FOR Gs, Dbrp AND Dstem, TYP.

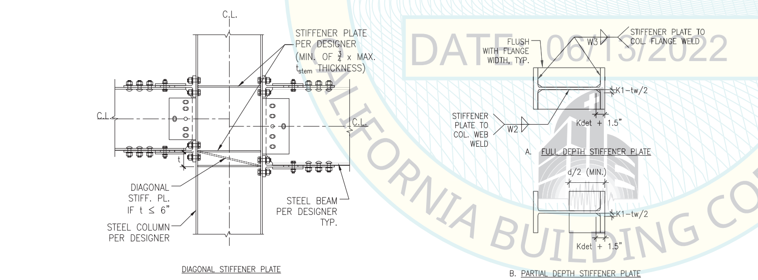
INST2A-02 [06-13-2022]

YL6 BEAM COPE/HOLE DETAILS 2



INST2A-20 [06-13-2022]

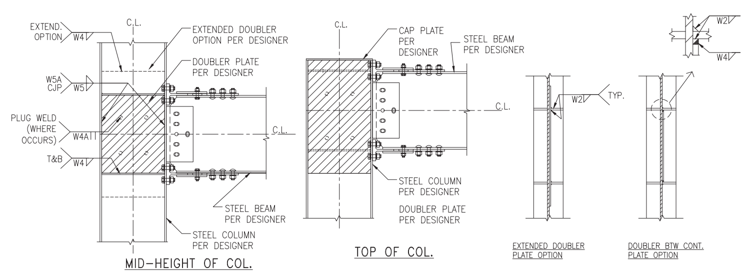
ALT. ATTACHMENTS AT SMF BM FLG 20



NOTE:
1. W2, W3 WELDS PER DRAWING ELEVATION WELD TABLE, SEE STRUCTURAL DRAWINGS AND 19/YL-INST1A

INST2A-11 [06-13-2022]

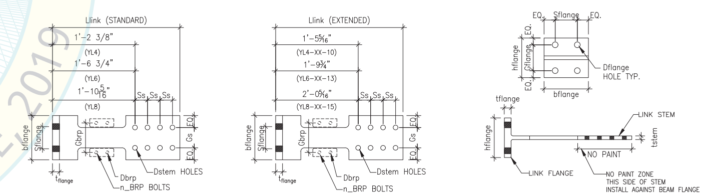
STIFFENER PLATE DETAILS 11



NOTE:
1. W4, W5 WELDS PER DRAWING ELEVATION WELD TABLE, SEE STRUCTURAL DRAWINGS
2. WHEN W5 IS A CJP WELD, IT SHALL MEET D1.8 DC WELD REQUIREMENTS (SEE 19/YL-INST1A)

INST2A-12 [06-13-2022]

DOUBLER PLATE DETAILS 12



STANDARD YIELD-LINKS				EXTENDED YIELD-LINKS				SHARED PARAMETERS											
LINK ID	tstem / tflange	Llink	n_BRP BOLTS	LINK ID	tstem / tflange	Llink	n_BRP BOLTS	bflange	hflange	Sflange	Gflange	Dflange	Dstem[3]	Dbrp	Gbrp	Gs	Ss	n STEM BOLTS	n FLG BOLTS
YL4-2	1/2" / 7/8"	1'-6 5/8"	2 (TOTAL)					6 1/4"	5 3/4"	3 1/2"	3 1/4"	15/16"	15/16"	1 1/16"	3 3/4"	3 1/2"	2 3/4"	4	4
YL4-2.5										3 1/2"	3 1/4"	15/16"	15/16"	1 1/16"	4 1/4"	3 1/2"	2 3/4"		
YL4-3								6 1/2"		3 1/2"	3 1/4"	15/16"	15/16"	1 1/16"	4 3/4"	3 1/2"	2 3/4"		
YL4-2.25				YL4-2.25-10	1/2" / 13/16"	1'-9 9/16"	4 (TOTAL)	7"		4 1/2"	3 1/4"	15/16"	15/16"	1 1/16"	4 3/8"	3 1/2"	2 3/4"		
YL4-2.875				YL4-2.875-10						4 1/2"	3 1/4"	15/16"	15/16"	1 1/16"	5"	3 1/2"	2 3/4"		
YL4-3.5		1'-9 3/8"		YL4-3.5-10		2'-0 5/16"		8"		5 1/4"	3 1/4"	15/16"	15/16"	1 1/16"	5 5/8"	5 1/4"	2 3/4"	6	
YL4-3.75				YL4-3.75-10						5 1/4"	3 1/4"	15/16"	15/16"	1 1/16"	5 7/8"	5 1/4"	2 3/4"		
YL4-4				YL4-4-10						5 1/4"	3 1/4"	15/16"	15/16"	1 1/16"	6 1/8"	5 1/4"	2 3/4"		
YL6-2.5	3/4" / 1 1/4"	2'-3 1/2"	4 (TOTAL)					6 1/2"	9 1/4"	4"	5 3/4"	15/16"	15/16"	13/16"	4 5/8"	4"	3 3/8"	6	4
YL6-3				YL6-3-13	3/4" / 1 1/4"	2'-6 1/2"	4 (TOTAL)	8"		5"	5 3/4"	1 1/16"	1 1/16"	13/16"	5 1/4"	5"	3 3/8"		
YL6-3.5				YL6-3.5-13						5"	5 3/4"	1 1/16"	1 1/16"	13/16"	5 3/4"	5"	3 3/8"		
YL6-4				YL6-4-13						5"	5 3/4"	1 5/16"	1 1/16"	13/16"	6 1/4"	5"	3 3/8"		
YL6-4.5				YL6-4.5-13				10"		6"	5 3/4"	1 5/16"	1 5/16"	13/16"	6 3/4"	6"	3 3/8"		
YL6-5				YL6-5-13						6"	5 3/4"	1 5/16"	1 5/16"	13/16"	7 1/4"	6"	3 3/8"		
YL6-5.5				YL6-5.5-13				12"		6"	5 3/4"	1 5/16"	1 5/16"	13/16"	7 3/4"	6"	3 3/8"		
YL6-6				YL6-6-13						6"	5 3/4"	1 7/16"	1 5/16"	13/16"	8 1/4"	6"	3 3/8"		
YL8-4	1" / 1 13/16"	2'-7 1/16"	4 (TOTAL)	YL8-4-15	1" / 1 13/16"	2'-9 1/16"	6 (TOTAL)	9"	10 3/4"	5 1/4"	7"	1 5/16"	1 5/16"	13/16"	6 1/2"	5 1/4"	3 3/8"	6	4
YL8-4.5				YL8-4.5-15						5 1/4"	7"	1 7/16"	1 5/16"	13/16"	7"	5 1/4"	3 3/8"		
YL8-5		2'-10 7/16"		YL8-5-15		3'-0 7/16"		12"		6"	7"	1 7/16"	1 5/16"	13/16"	7 1/2"	6"	3 3/8"	8	
YL8-5.5				YL8-5.5-15						6"	7"	1 9/16"	1 5/16"	13/16"	8"	6"	3 3/8"		
YL8-6				YL8-6-15						6"	7"	1 9/16"	1 5/16"	13/16"	8 1/2"	6"	3 3/8"		

NOTES:
1. FOR CUSTOM LINKS NOT NOTED CONTACT SIMPSON STRONG-TIE FOR CUSTOM DETAILING GEOMETRIES
2. BEAM, COLUMN AND HOLE DIMENSIONAL TOLERANCE SHALL BE PER AISC CODE OF STANDARD PRACTICE FOR STEEL BUILDINGS AND BRIDGES (AISC 303-16)
3. Dstem HOLES ON BEAM FLANGES ARE PERMITTED TO BE STANDARD SIZES HOLES PER AISC 360-16 TABLE J3.3, ALL LINK HOLE SIZES SHOWN ARE 1/16" LARGER THAN BOLT DIAMETER.
4. SEE DETAILS 1/-, 2/-, 5/-, 6/-, 9/- AND 13/- FOR BEAM COPE AND BOLT HOLE DETAILS
5. SEE DETAIL 17/- FOR COLUMN FLANGE BOLT HOLE DETAILS

INST2A-04 [06-13-2022]

YIELD-LINK DETAILED GEOMETRIES 4

NO.	DATE	REVISIONS

06/13/2022

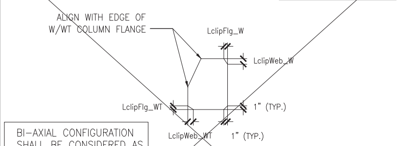
SIMPSON Strong-Tie
THERE IS NO EQUAL
SIMPSON STRONG-TIE, CO. INC.
• 5956 W. Las Positas Blvd. Pleasanton, CA 94588
• Tel: (800) 999-5099
• Fax: (925) 847-1597
• Web site: www.strongtie.com

YIELD-LINK® MOMENT CONNECTION
STEEL SPECIAL MOMENT FRAME
CONNECTION DETAILING INFORMATION

NAME: B.C.
DATE: 06-08-2022
SCALE: N.T.S.
SHEET: YL-INST2A
JOB NO.

CRUCIFORM COL. STP CLIP DETAILS — 18

INST3A-18 [06-13-2022]



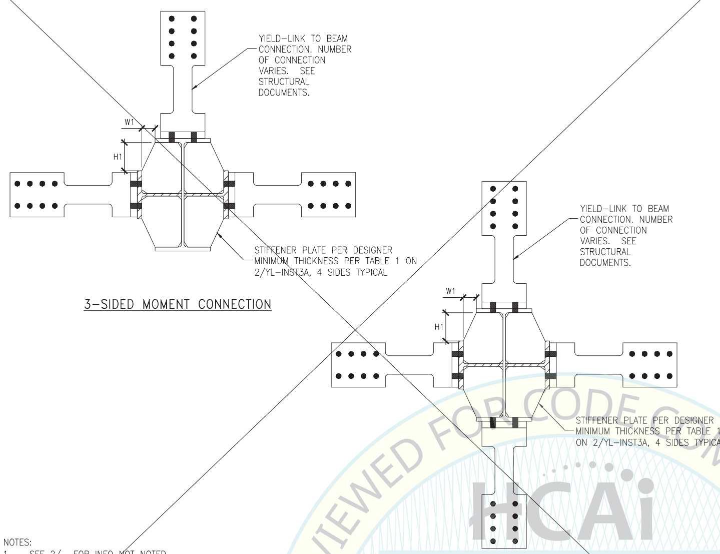
BI-AXIAL CONFIGURATION SHALL BE CONSIDERED AS ALTERNATE SYSTEM BY HCAI/OSHPD

NOTES:

1. WEB COPE AT WT AND W SECTIONS (LclipWeb_WT, LclipWeb_W) = $k+1.5"-t_f$
2. FLANGE COPE AT WT AND W SECTIONS (LclipFlg_WT, LclipFlg_W) = $k_1+0.5"-t_w/2$
3. SEE AWS D1.8 CHAPTER C-4 FOR MORE INFORMATION
4. SEE AISC STEEL MANUAL FOR k, k_1, t_f AND t_w VALUES FOR THE COLUMNS

MULTI-DIRECTION MOMENT CONNECTION DETAIL — 10

INST3A-10 [06-13-2022]



NOTES:

1. SEE 2/- FOR INFO NOT NOTED

BI-AXIAL CONFIGURATION SHALL BE CONSIDERED AS ALTERNATE SYSTEM BY HCAI/OSHPD

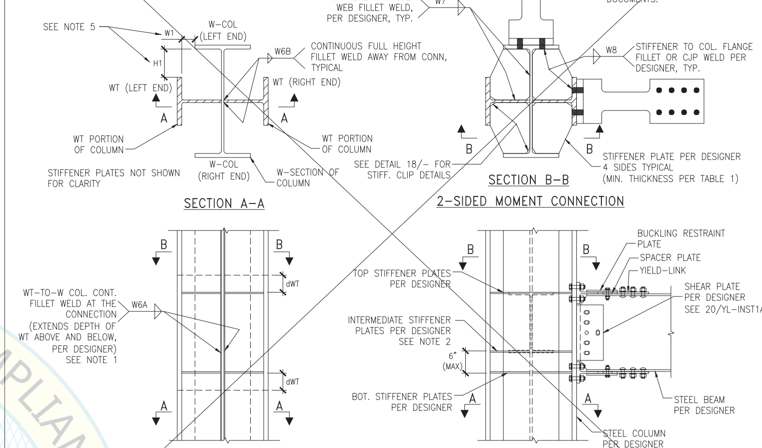
CORNER CONDITION WITH FLANGE CRUCIFORM COLUMN — 2

INST3A-02 [06-13-2022]

TABLE 1

YIELD-LINK	MIN. STIFF. THK (IN)
YL4	3/8
YL6	1/2
YL8	5/8

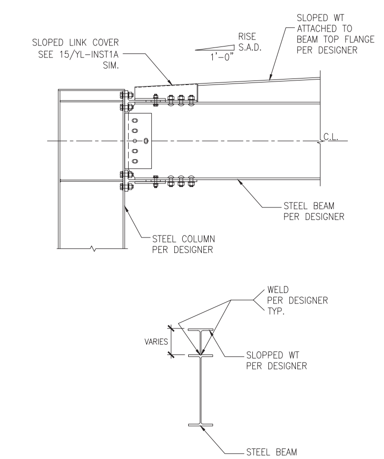
BI-AXIAL CONFIGURATION SHALL BE CONSIDERED AS ALTERNATE SYSTEM BY HCAI/OSHPD



NOTES:

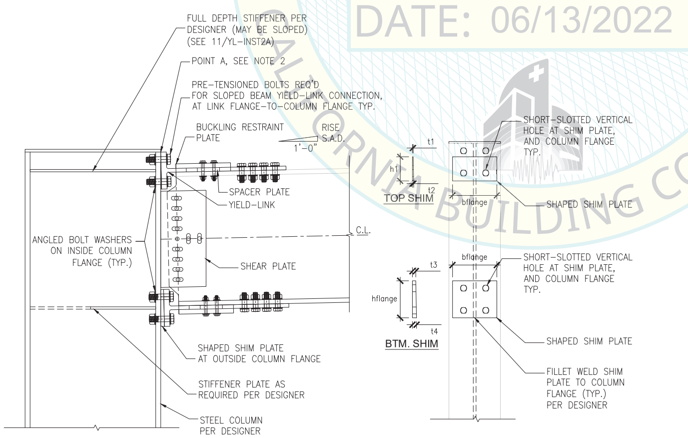
1. DOUBLE SIDED FILLET WELD SHALL DEVELOP SHEAR AT COLUMN FLANGE-TO-WEB CONNECTION STRENGTH. SMALLER OF: (A) NOMINAL SHEAR STRENGTH OF WT COLUMN WEB, (B) MAXIMUM SHEAR FORCE AT PR_LINK OF CONNECTION.
2. INTERMEDIATE STIFFENER REQUIRED IF DIFFERENCE BETWEEN DEPTH OF BEAMS CONNECTING W-COL AND WT SECTION IS GREATER THAN 6".
3. SEE 6/YL-INST1A FOR BOLTING REQUIREMENTS
4. SEE 7/YL-INST1A FOR PROTECTED ZONE AT YIELD-LINK CONNECTION
5. W1 AND H1 ARE DIMENSIONS FOR WELDING CLEARANCE. DESIGNER TO COORDINATE WITH FABRICATOR FOR MINIMUM DIMENSIONS REQUIRED.

SLOPED BEAM CONN (OPTION 3) — 20



INST3A-20 [06-13-2022]

SLOPED BEAM YIELD-LINK CONNECTION (OPTION 2) — 12

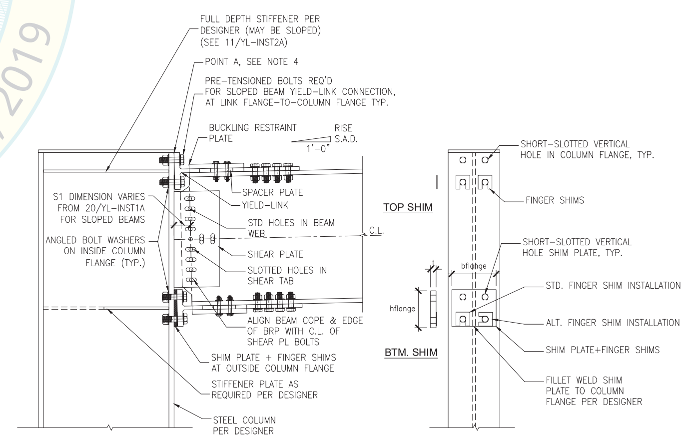


NOTES:

1. SEE 4/- FOR INFO NOT NOTED
2. THICKNESS OF SHIMS (t1, t2, t3 AND t4) TO BE DETERMINED BY DESIGNER/DETAILER ASSUMING TOP OF LINK FLANGE IN CONTACT WITH COLUMN FLANGE (POINT A), WITH MINIMUM SHIM THICKNESS OF 1/16" AT (t1 and t3 LOCATIONS)
3. t1 SHALL BE A MAXIMUM OF 1/8"

INST3A-12 [06-13-2022]

SLOPED BEAM YIELD-LINK CONNECTION (OPTION 1) — 4



NOTES:

1. MAXIMUM RISE = 1" PER FOOT. CONTACT SIMPSON STRONG-TIE FOR STEEPER SLOPES.
2. LINK FLANGE-TO-COLUMN FLANGE BOLTS SHALL BE PRE-TENSIONED FOR SLOPED BEAM-TO-COLUMN YIELD-LINK CONNECTIONS
3. SHIM PLATE SIZE TO MATCH LINK FLANGE LENGTH AND WIDTH DIMENSIONS.
4. THICKNESS OF SHIM TO BE DETERMINED BY DESIGNER/DETAILER ASSUMING TOP OF LINK FLANGE IN CONTACT WITH COLUMN FLANGE (POINT A)
5. SHIM PLATE MATERIAL: A36 OR A572 GR. 50
6. FINGER SHIM MATERIAL: STRUCTURAL-GRADE STEEL MATERIAL, BUT NEED NOT BE HARDENED
7. ANGLED BOLT WASHER MATERIAL: F436
8. BEAM SLOPED DOWN CONDITION SIMILAR
9. SEE YL-INST1A SHEET FOR FURTHER INFORMATION

INST3A-04 [06-13-2022]

NO.	DATE	REVISIONS

06/13/2022

SIMPSON Strong-Tie
SIMPSON STRONG-TIE, CO. INC.
• 5956 W. Las Positas Blvd. Pleasanton, CA 94588
• Tel: (800) 999-5099
• Fax: (925) 847-1597
• Web site: www.strongtie.com
THERE IS NO EQUAL

YIELD-LINK® MOMENT CONNECTION

STEEL SPECIAL MOMENT FRAME

CONNECTION DETAILING INFORMATION

NAME: B.C.
DATE: 06-08-2022
SCALE: N.T.S.
SHEET: YL-INST3A
JOB NO.

APPENDIX (FOR REFERENCE ONLY)

APPENDIX A1: ANSI/AISC 358s2-20

For YLMC in AISC 358, please see Chapter 12 PDF document

https://www.aisc.org/globalassets/aisc/publications/standards/a358-20w.pdf

APPENDIX A2: YIELD-LINK MOMENT CONNECTION EXCEL TOOL USER GUIDE

Simpson Strong-Tie

Yield-Link® Moment Connection Excel Design Tool

User Manual for V3.3

Date: 05/2022

Directions to use this Design Aid:

Follow the Steps in sequential order indicated on the VBA Buttons in the custom Ribbon under "**SST Yield-Links**":

Provide input field where it's shaded in gray with black text: User Input

Users can also overwrite the values shaded in pink with black text: User Overwrite

Current spreadsheet performs initial analysis to calculate the drift and frame forces. User to confirm with their structural design software and overwrite these values as applicable. Once the VBA buttons along the Ribbon are pressed, tabs along the bottom of the tool will appear. Once sheet tabs have populated, pressing the main buttons along the Ribbon of the tool will reset the sheets, resetting any user inputs and overrides. User should use the tabs created at bottom of page for navigating tool. See below for updating calculations at each step.

Figure 1: VBA Buttons at top of the screen

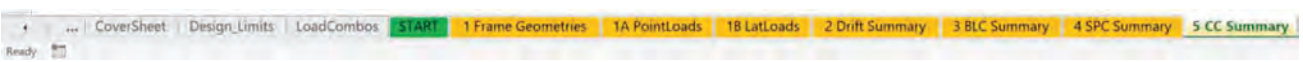
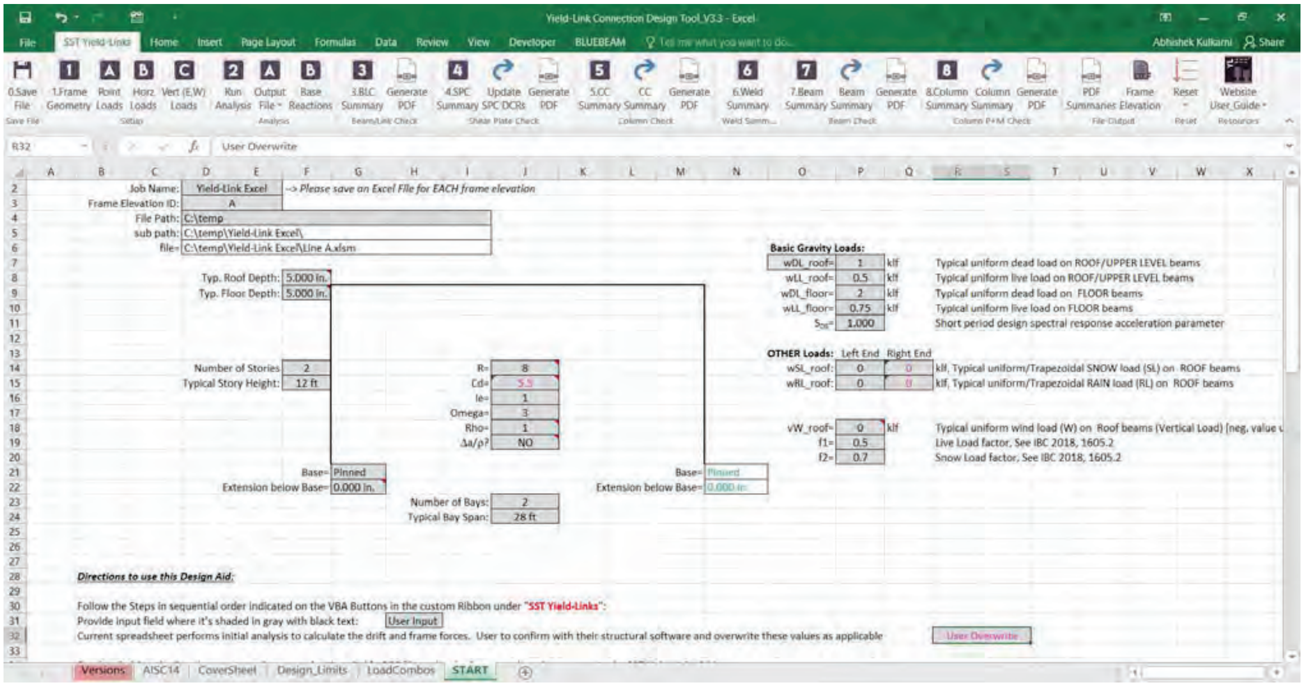
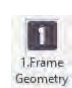


Figure 2: Tabs at bottom of sheet

Step 0:



1. Input **Job Name**, **Frame Elevation ID**, and **File Path** for where to save the Excel File.
2. One file should be used for each frame line. Push the "***0. Save File***" 

button when finished with input.
3. Input Basic Frame Geometries in the "***START***" tab. **Typical Story Height** and **Bay Span** for initial frame geometry and can be refined in Step 1.
   a. **Typ. Roof Depth** and **Typ. Floor Depth** are used for drawing purposes only to show the top of steel beam height below the finished level height and not used in calculations.
   b. **Basic Gravity Loads** assume uniform full width loading. On cell O7, user may select whether the top frame level is assigned as roof loading or as upper level for floor loading. This will affect the Live Load uniform load and Live Load point loads in the ***"1A PointLoads"*** tab.
   c. Floor level uniform loading input will apply the same loading to all levels under the upper level and can be adjusted in the user's software output file.
   d. Frame **Base** fixity can be selected as fully pinned or fully fixed for initial design and output file base restraint.
   e. The **Extension Below Base** input is for drawing function only and the frame design height should be evaluated based on required design height and can be adjusted in the user's software output file.
4. Once all initial inputs are entered, push the ***"1. Frame Geometry"*** button. 

Step 1:

1. Input/modify detailed frame geometries in the "***1 Frame Geometries***" tab.

	A	B	C	E	G	H	I	M	N	O	P	U	V
1				Frame Geometry									
2	Elev ID	Grid ID	Story	Story Height (in.)	Left Beam Size	Left Beam Bay Span (in.)	Column size	Fixed Base Col width-to-thickness DCR	Beam ID	Beam Size	Link Size	Assign Link at I_End	Assign Link at J_End
3	A	1	Story2	144	N/A	N/A	W27x114	N/A					
4	A	2	Story2	144	W24x84	336	W27x114	N/A	2001	W24x84	YL6-4	YES	YES
5	A	3	Story2	144	W24x84	336	W27x114	N/A	2002	W24x84	YL6-4	YES	YES
6	A	1	Story1	144	N/A	N/A	W27x114	N/A					
7	A	2	Story1	144	W24x84	336	W27x114	N/A	1001	W24x84	YL6-4	YES	YES
8	A	3	Story1	144	W24x84	336	W27x114	N/A	1002	W24x84	YL6-4	YES	YES

W	X	Y	Z	AA	AB	AC	AD	AE	AF	AG	AH	AI	AJ
1. INITIAL YIELD-LINK SELECTION SUMMARY													
$M_{u\text{-}link}$ (kip-in.)	$V_{gravity}$ (kips)	$P_{u\text{-}sp}$ (kips)	Initial t_{bf} check	Initial b_{bf} check	Initial b_{cf} check	Initial $L_{y\text{-}link}$ check	Link Strength DCR	PZ I-End Col. DCR	PZ J-End Col. DCR	K_rot (kip-in./rad)	phi * Mn (k-in)	M_{cap_link} (kip-in.)	Beam Slope Condition (x" per foot)
812	17.7	21.5	0.52	0.91	0.82	0.88	0.24	0.56	1.11	1.52E+06	3355	5815	0.00
448	12.6	13.5	0.52	0.91	0.82	0.88	0.13	1.11	0.56	1.52E+06	3355	5815	0.00
1544	17.9	10.5	0.52	0.91	0.82	0.88	0.46	0.56	1.11	1.52E+06	3355	5815	0.00
629	12.3	12.9	0.52	0.91	0.82	0.88	0.19	1.11	0.56	1.52E+06	3355	5815	0.00

Figure 3: Step 1 (1 Frame Geometries Input)

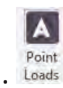
a. Model SST-Yield Link Connections with rotational spring value **(K_rot)** given in column "AG", refer to Modeling Guide for more information. Rotational spring stiffness is calculated automatically once beam size and Yield-Link (YL) sizes are chosen and will auto apply to RISA-3D and SAP2000 output files (Step 2A). Changes to beam and / or column size will change the K_rot and need to be adjusted in user's output file.
b. Adjust **Grid ID** as applicable for calculation and ***"Frame Elevation"*** outputs.
c. **Story Height** and **Bay Span** may be refined on this tab. **Story Height** in column "E" is the level height used in the frame drawings. **Bay Span** is considered as column centerline dimensions. The calculations will use member centerlines for design within the Excel Tool and when generating the output files in Step 2A.
d. For YL connections, 1st level columns with a fixed base condition will need to satisfy the AISC 341-16 highly ductile **width-to-thickness** requirement. Calculation in column "M" requires frame analysis output, see Step 2.
e. **Link Size** in column "P" shall be coordinated with the allowable minimum and maximum beam depth shown on the Yield-Link Moment Connection Geometry table (page 11) found in the F-L-YLMCDG20 ***Design Guide***. Pull down list will filter based on beam depth. Beam flange width should be compared to link stem width for compatibility. For a list of initial beam and column size selections based on geometric restraints please refer to the ***YLMC Initial Beam-Col Matches*** PDF.

f. ***M_{u_link}*, *$V_{gravity}$* and *$P_{u\text{-}sp}$*** start with default values, and will update when Step 2 is performed. Overwrite from user analysis software as applicable where loading or configuration varies from Excel Tool inputs. See the ***"LoadCombos"*** tab, column "N" for ***M_{u_link}***, column "Q" for ***$V_{gravity}$***, column "P" for ***$P_{u\text{-}sp}$***, for load combinations to envelope in analysis software. Note that when values are overwritten, they will not refresh when the ***"Run Analysis"*** button is performed again unless the sheet is refreshed by pressing the main button in the Ribbon for that step, resetting that tab with initial default inputs.

g. ***Initial t_{bf}, b_{bf}, b_{cf},*** and **$L_{y\text{-}link}$** DCR should be less than the limiting value before proceeding to Step 2.

h. ***PZ*** checks on this step are initial panel zone checks. At lower levels, upper story column shear has not been subtracted out and will be completed on Step ***"5. CC Summary"***. Doubler plates can also be selected on step 5 as applicable to reduce PZ DCR.

i. ***Beam Slope Condition*** can be used to slope beams up or down up to 1" per foot.

2. Once all initial checks in columns "Z" through "AC" are satisfied, push "***1A. Point Loads***". 

Step 1A:

1. Input **Column Point loads** at the top of each column member (in columns "F", "G", "H").

A	B	C	D	E	F	G	H
Column Point Loads (DL, LL/LR, SL)							
Elev ID	Grid ID	Story	Column ID	Column Size	Column DL kips	Column LL kips	Column SL kips
A	1	Story1	1	W27x114			

I	J	K	L	M	N	O	P	Q	R	S
Beam Point Load (Dead)										
Beam ID	Beam Size	Beam Span (in)	P_{DL1} (kips)	DL_X1 (in)	P_{DL2} (kips)	DL_X2 (in)	P_{DL3} (kips)	DL_X3 (in)	P_{DL4} (kips)	DL_X4 (in)
1001	W24x84	336								

T	U	V	W	X	Y	Z	AA	AB	AC	AD	AE	AF	AG	AH	AI
Beam Point Loads (Live/Roof Live)								Beam Point Loads (Snow)							
P_{LL1} (kips)	LL_X1 (in)	P_{LL2} (kips)	LL_X2 (in)	P_{LL3} (kips)	LL_X3 (in)	P_{LL4} (kips)	LL_X4 (in)	P_{SL1} (kips)	SL_X1 (in)	P_{SL2} (kips)	SL_X2 (in)	P_{SL3} (kips)	SL_X3 (in)	P_{SL4} (kips)	SL_X4 (in)

Figure 3: Step 1A PointLoads Input

2. Input **Beam Point Loads** for **Dead Load** (in columns "L through S"), **Live/Roof Live** (in columns "T through AA"), and **Snow Load** (in columns "AB through AI"). Loading should be entered in kips and location in inches for each point load. Additional point loads over (4) per load case will need to be applied in user's analysis software.

a. **Live/Roof Live** will auto default based on top level of beams and setting in the ***"START"*** tab cell O7 for Roof / Upper Level.
b. Seismic and Wind vertical point loads will be applied in Step 1C.
c. Update M_{u_link}, $V_{gravity}$ *and* $P_{u\text{-}sp}$ on the ***"1 Frame Geometries"*** tab as required.

3. Push "***1B. Horz Loads***". 

Step 1B:

A	C	D	E	F	G	H	I	J	K	L	M	N	O
	1B. LATERAL LOADS AND DEFLECTION PARAMETERS												
Elev ID	Story	Column Location ID	Story Height in	Total Frame Width in	Fx Strength (kips)	Fx Drift (kips)	Ni Strength (kips)	Allowable Drift Limit	C_d	I_e	Pi (kips)	F_wind (kips)	Allowable Drift Limit
A	Story2	A-1-Story2	144	672	1.0	1.0	0.0	0.025 Hx	5.5	1	1	1.0	hx/ 100
A	Story1	A-1-Story1	144	672	1.0	1.0	0.0	0.025 Hx	5.5	1	1	1.0	hx/ 100

Figure 4: Step 1B Horz. Loads Input

1. Input lateral seismic load (1.0E) for **Fx Strength** in column "G" and **Allowable Drift Limit** (seismic) in column "J". Lateral seismic load (1.0E) for **Fx Drift** in column "H" is initially set equal to input for Fx Strength, but can be adjusted as needed if lateral seismic drift force is less than strength, such as when program calculated periods for drift are used.
2. Adjust Notional Load **Ni Strength** in column "I" as applicable and enter **Pi** (total vertical design load) on column "M". Pi is the tributary mass (DL+LL) for the frame; see ASCE 7-16 section 12.8.7 for more information.
3. Input lateral **F_wind** load (1.0W) in column "N" and **Allowable Drift Limit** (wind) in column "O".

Step 1C:

A	B	C	D	E	F	G	H
Column Vertical Point Loads (EQ, EQ_Drift and Wind)							
Elev ID	Grid ID	Story	Column ID	Column Size	Column EQ kips	Column EQ_Drift kips	Column Wind kips
A	1	Story2	4	W27x114			

I	J	K	L	M	N	O	P	Q	R	S
			Beam Vertical Point Load (EQv)							
Beam ID	Beam Size	Beam Span (in)	P_{EV1} (kips)	EV_X1 (in)	P_{EV2} (kips)	EV_X2 (in)	P_{EV3} (kips)	EV_X3 (in)	P_{EV4} (kips)	EV_X4 (in)
2001	W24x84	336								

T	U	V	W	X	Y	Z	AA	AB	AC	AD	AE	AF	AG	AH	AI
Beam Vertical Point Load (Eqv_Drift)								Beam Vertical Point Loads (Wind)							
P_{EVD1} (kips)	EVD_X1 (in)	P_{EVD2} (kips)	EVD_X2 (in)	P_{EVD3} (kips)	EVD_X3 (in)	P_{EVD4} (kips)	EVD_X4 (in)	P_{WV1} (kips)	WV_X1 (in)	P_{WV2} (kips)	WV_X2 (in)	P_{WV3} (kips)	WV_X3 (in)	P_{WV4} (kips)	WV_X4 (in)

Figure 4: Step 1C Vert. (E,W) Loads Input

1. Input **Column EQ** and **Column Wind** vertical point loads at the top of each column member (in columns “F”, “G”, “H”).
   a. **Column EQ_Drift** loads will automatically be applied based on **Column EQ** inputs and can be adjusted as required.
2. Input **Beam Point Loads** for **EQv Load** (in columns “L through S”), **EQv_Drift** (in columns “T through AA”), and **Wind** (in columns “AB through AI”). Loading should be entered in kips and the location in inches for each point load. Additional point loads over (4) per load case will need to be applied in user’s analysis software.
   a. **EQv_Dift** loads will automatically be applied based on **EQv** inputs and can be adjusted as required.

Step 2:

1. Push the "***2 Run Analysis***" button. Review **Drift DCR**s in column "O" and in column “AK”.

A	B	C	D	E	F	G	H	I	J	K	L	M	N	O
2. DRIFT CHECK SUMMARY														
Elev ID	Grid ID	Story	Column Location ID	Story Height in	Fx_Drift (kips)	Drift Node ID	Frame Seismic Displacement (in.) [LC 1&2]	δ_{xe} (in.)	C_d	I_e	δ_x (in.)	Allowable Drift Limit	Allowable Drift (in.)	Drift DCR
A	1	Story1	A-1-Story1	138	10.00	2	0.106	0.106	5.5	1	0.581	0.025 Hx	3.45	0.168

P	Q	R	S	T	U	V	W	X	AE	AF	AG	AH	AI	AJ	AK
2a. Stability Check for P-delta Effects									2b. Wind deflection						
Pi (kips)	P_x (kips)	Fx (kips)	Vx (kips)	hsx (in)	Stability Coeff, θ	β (max Link DCR)	θmax	Stability θ, DCR	F_wind (kips)	Drift Node ID	Frame Wind Displacement (in.) [LC 9&10]	drift (in.)	Allowable Drift Limit	Allowable Drift (in)	Wind Drift DCR
1	1	20.00	20	138	0.0000	1	0.09	0.000	10.00	2	0.074	0.074	hx/ 100	1.38	0.054

Figure 5: Step 2 Drift Summary

2. Review the **Link Strength DCR** in column “AD” of the ***"1 Frame Geometries"*** tab.

Step 2A:

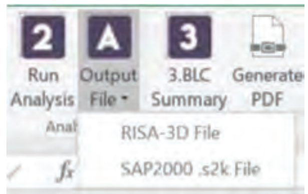


Figure 6: Step 2A RISA 3-D and SAP2000 .s2k file ouput

1. Click on "***A Output File***"  to generate RISA-3D or SAP2000 (.s2k) output file for the frame.
   a. In RISA-3D, set adjust stiffness reduction to "YES (Iterative)" under the Global/Codes setting menu when running non-drift check combinations.
   b. Adjust beam top and bottom flange bracing as applicable for the beam design. Note that lateral torsional beam bracing is not required for use with the Yield-Link Moment Connections.
   c. In RISA-3D, the user can run load combinations 1 through 28 for connection Moment, $\mathbf{M_{u\text{-}link}}$, then overwrite column "W" in the "***1 Frame Geometries"*** tab, as required. Please note, $\mathbf{M_{u\text{-}link}}$ should be taken at the face of the column.
   d. In RISA-3D, the user can run load combination 8 for beam end reaction, $\mathbf{V_{gravity}}$, then overwrite column "X" in the "***1 Frame Geometries"*** tab as required.
   e. In RISA-3D, the user can run load combinations 1 through 22 and 33 through 36 for connection axial load, $\mathbf{P_{u\text{-}sp}}$, then overwrite values in column "Y" on the "***1 Frame Geometries"*** tab as required. Please note, $\mathbf{P_{u\text{-}sp}}$ is the maximum beam axial load at the face of each column.
   f. In RISA-3D, the user can run load combination 29 through 32 for seismic drift checks, then overwrite **Frame Seismic Displacement** values in column "H" on the "***2 Drift Summary"*** tab as required.
   g. In RISA-3D, the user can run load combination 23 and 24 for wind drift checks, then overwrite **Frame Wind Displacement** values in column "AG" on the "***2 Drift Summary"*** tab as required.
   h. In RISA-3D, the user can run an envelope of **ALL** load combinations to design the beam and column for omega level forces. See steps ***"7. Beam Summary"*** and ***"8. Column Summary"***.
   i. Please note, for the beam design, the moment demand need not be more than $\mathbf{M_{cap\_link}}$ noted in the "***1 Frame Geometries"*** tab column "AI" since $\mathbf{M_{cap\_link}}$ is the maximum moment the connection can deliver.
   j. In addition, the user should ratio up the demand load (**Mu_Omega**) to $\mathbf{M_{cap\_link}}$ if **Mu_Omega** is less than $\mathbf{M_{cap\_link}}$, to verify the beam is able to develop the demand from the links. Step "***7. Beam Summary"*** will automatically run this calculation. (e.g. if the Flexural DCR=0.69, and Mu_Omega=1000 k-in, M_{cap_link}=1250 k-in, then the new Flexural

DCR=1250/1000*0.69=0.86, user than combines the existing Axial DCR + new Flexural DCR to see if it's still under their design limit).

k. For SAP2000, load combinations are the same as those in RISA-3D, they are also noted in the "***LoadCombos***" tab and are shown in Figure 7 below:

ID	Load Combinations	Load Multipliers									
		DL	LL	LR	SL	RL	WL	NL	EL_D	EL	
SST_LC1	1.4 DL + NL	1.4	0	0	0	0	0	1	0	0	Gravity
SST_LC2	1.2 DL + 1.6 LL + 0.5 LR + NL	1.2	1.6	0.5	0	0	0	1	0	0	
SST_LC3	1.2 DL + 1.6 LL + 0.5 SL + NL	1.2	1.6	0	0.5	0	0	1	0	0	
SST_LC4	1.2 DL + 1.6 LL + 0.5 RL + NL	1.2	1.6	0	0	0.5	0	1	0	0	
SST_LC5	1.2 DL + 1.6 LR + 0.5 LL + NL	1.2	0.5	1.6	0	0	0	1	0	0	
SST_LC6	1.2 DL + 1.6 SL + 0.5 LL + NL	1.2	0.5	0	1.6	0	0	1	0	0	
SST_LC7	1.2 DL + 1.6 RL + 0.5 LL + NL	1.2	0.5	0	0	1.6	0	1	0	0	
SST_LC8	(1.2 + 0.2*SDS)DL + f1*LL + f2*SL	1.4	0.5	0.5	0.7	0.5	0	0	0	0	
SST_LC9	1.2 DL + 1.6 LR + 0.5 WL	1.2	0	1.6	0	0	0.5	0	0	0	Wind
SST_LC10	1.2 DL + 1.6 LR - 0.5 WL	1.2	0	1.6	0	0	-0.5	0	0	0	
SST_LC11	1.2 DL + 1.6 SL + 0.5 WL	1.2	0	0	1.6	0	0.5	0	0	0	
SST_LC12	1.2 DL + 1.6 SL - 0.5 WL	1.2	0	0	1.6	0	-0.5	0	0	0	
SST_LC13	1.2 DL + 1.6 RL + 0.5 WL	1.2	0	0	0	1.6	0.5	0	0	0	
SST_LC14	1.2 DL + 1.6 RL - 0.5 WL	1.2	0	0	0	1.6	-0.5	0	0	0	
SST_LC15	1.2 DL + 1.0 WL + 0.5 LL + 0.5 LR	1.2	0.5	0.5	0	0	1	0	0	0	
SST_LC16	1.2 DL - 1.0 WL + 0.5 LL + 0.5 LR	1.2	0.5	0.5	0	0	-1	0	0	0	
SST_LC17	1.2 DL + 1.0 WL + 0.5 LL + 0.5 SL	1.2	0.5	0	0.5	0	1	0	0	0	
SST_LC18	1.2 DL - 1.0 WL + 0.5 LL + 0.5 SL	1.2	0.5	0	0.5	0	-1	0	0	0	
SST_LC19	1.2 DL + 1.0 WL + 0.5 LL + 0.5 RL	1.2	0.5	0	0	0.5	1	0	0	0	
SST_LC20	1.2 DL - 1.0 WL + 0.5 LL + 0.5 RL	1.2	0.5	0	0	0.5	-1	0	0	0	
SST_LC21	0.9 DL + 1.0 WL	0.9	0	0	0	0	1	0	0	0	
SST_LC22	0.9 DL - 1.0 WL	0.9	0	0	0	0	-1	0	0	0	
SST_LC23	1.0 DL + 0.5 LL + 0.5 LR +0.7 WL	1.0	0.5	0.5	0	0	0.7	0	0	0	
SST_LC24	1.0 DL + 0.5 LL + 0.5 LR -0.7 WL	1.0	0.5	0.5	0	0	-0.7	0	0	0	
SST_LC25	(1.2 + 0.2*SDS)DL + EL*rho + f1*LL + f2*SL	1.4	0.5	0.5	0.7	0	0	0	0	1.0	Seismic
SST_LC26	(1.2 + 0.2 SDS)DL - EL*rho + f1*LL + f2* SL	1.4	0.5	0.5	0.7	0	0	0	0	-1.0	
SST_LC27	(0.9 - 0.2 SDS) DL + EL*ρ	0.7	0	0	0	0	0	0	0	1.0	
SST_LC28	(0.9 - 0.2 SDS) DL - EL*ρ	0.7	0	0	0	0	0	0	0	-1.0	
SST_LC29	(1.2 + 0.2*SDS)DL + EL_D+ f1*LL + f2*SL	1.4	0.5	0.5	0.7	0	0	0	1.0	0	
SST_LC30	(1.2 + 0.2 SDS)DL - EL_D+ f1*LL + f2* SL	1.4	0.5	0.5	0.7	0	0	0	-1.0	0	
SST_LC31	(0.9 - 0.2 SDS) DL + EL_D	0.7	0	0	0	0	0	0	1.0	0	
SST_LC32	(0.9 - 0.2 SDS) DL - EL_D	0.7	0	0	0	0	0	0	-1.0	0	
SST_LC33	(1.2 + 0.2 SDS)DL + OmegaEL + f1*LL + f2*SL	1.4	0.5	0.5	0.7	0	0	0	0	3.0	Ω
SST_LC34	(1.2 + 0.2 SDS)DL -OmegaEL + f1*LL + f2*SL	1.4	0.5	0.5	0.7	0	0	0	0	-3.0	
SST_LC35	(0.9 - 0.2 SDS) DL + OmegaEL	0.7	0	0	0	0	0	0	0	3.0	
SST_LC36	(0.9 - 0.2 SDS) DL - OmegaEL	0.7	0	0	0	0	0	0	0	-3.0	

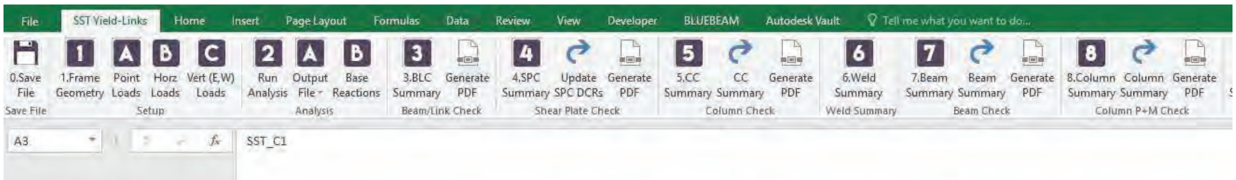
Figure 7: Load Combinations use for Yield-Link Connection Design

l. In addition to the elastic model output for SAP2000, there is an option to generate a model with nonlinear hinges under the "***Design Limits***" tab by setting cell E80 to "YES".

70	**SAP2000 Nonlinear Parameters for Pushover/Time History:**	
71		
72	**Beam Hinges modeled with M3 (Deformation Control) type hinge**	
73		
74	**Column Hinges modeled with Fiber (P+M2+M3) hinge**	
75	RelativeHingeLength=	0.05
76		
77	Relative Distance (top)=	0.9
78	Relative Distance (bottom)=	0.1
79		
80	Generate NL Hinges in .e2k file?	$\boxed{\text{YES}}$

Step 2B:

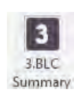
1. After ***"Run Analysis"*** is complete, push the ***"2B Base Reactions"*** button to view base reactions for individual load cases and for the Load Combinations run in the Excel Design Tool.
   a. Base reactions for all load combinations can be viewed in user's output design file.



	Base Reactions (Column Nodes from Left to Right)												
Combo ID	Load Combination	Node	Fx (kips)	Fy (kips)	M (k-in)	Node	Fx (kips)	Fy (kips)	M (k-in)	Node	Fx (kips)	Fy (kips)	M (k-in)
SST_C1	DL	1	4.6	46.3	0.0	4	0.0	92.6	0.0	7	-4.6	46.3	0.0
SST_C2	LL	1	1.8	10.4	0.0	4	0.0	21.2	0.0	7	-1.8	10.4	0.0
SST_C3	LR	1	-0.1	6.8	0.0	4	0.0	14.3	0.0	7	0.1	6.8	0.0
SST_C4	SL	1	0.0	0.0	0.0	4	0.0	0.0	0.0	7	0.0	0.0	0.0
SST_C5	RL	1	0.0	0.0	0.0	4	0.0	0.0	0.0	7	0.0	0.0	0.0
SST_C6	WL	1	-0.6	-0.6	0.0	4	-0.8	0.0	0.0	7	-0.6	0.6	0.0
SST_C7	NL	1	0.0	0.0	0.0	4	0.0	0.0	0.0	7	0.0	0.0	0.0
SST_C8	EL_D	1	-0.6	-0.6	0.0	4	-0.8	0.0	0.0	7	-0.6	0.6	0.0
SST_C9	EL	1	-0.6	-0.6	0.0	4	-0.8	0.0	0.0	7	-0.6	0.6	0.0
SST_LC02	1.2 DL + 1.6 LL + 0.5 LR + NL	1	8.5	75.4	0.0	4	0.0	152.5	0.0	7	-8.6	75.5	0.0
SST_LC08	(1.2 + 0.2*SDS)DL + f1*LL + f2*SL	1	7.5	73.2	0.0	4	0.0	147.7	0.0	7	-7.5	73.2	0.0
SST_LC13	1.2 DL + 1.6 RL + 0.5 WL	1	5.4	55.1	0.0	4	-0.4	111.3	0.0	7	-6.0	55.7	0.0
SST_LC14	1.2 DL + 1.6 RL - 0.5 WL	1	6.0	55.7	0.0	4	0.4	111.3	0.0	7	-5.4	55.1	0.0
SST_LC15	1.2 DL + 1.0 WL + f1 LL + 0.5 LR	1	5.9	63.4	0.0	4	-0.8	129.1	0.0	7	-7.1	64.6	0.0
SST_LC16	1.2 DL - 1.0 WL + f1 LL + 0.5 LR	1	7.1	64.6	0.0	4	0.8	129.1	0.0	7	-5.9	63.4	0.0
SST_LC23	1.0 DL + 0.5 LL + 0.5 LR +0.7 WL	1	5.0	54.5	0.0	4	-0.6	110.4	0.0	7	-5.8	55.3	0.0
SST_LC24	1.0 DL + 0.5 LL + 0.5 LR -0.7 WL	1	5.8	55.3	0.0	4	0.6	110.4	0.0	7	-5.0	54.5	0.0
SST_LC25	(1.2 + 0.2*SDS)DL + EL*rho + f1*LL + f2*SL	1	6.9	72.6	0.0	4	-0.8	147.7	0.0	7	-8.1	73.8	0.0
SST_LC26	(1.2 + 0.2 SDS)DL - EL*rho + f1*LL + f2* SL	1	8.1	73.8	0.0	4	0.8	147.7	0.0	7	-6.9	72.6	0.0
SST_LC29	(1.2 + 0.2*SDS)DL + EL_D+ f1*LL + f2*SL	1	6.6	72.8	0.0	4	-0.8	147.4	0.0	7	-7.8	74.0	0.0
SST_LC30	(1.2 + 0.2 SDS)DL - EL_D+ f1*LL + f2* SL	1	7.8	74.0	0.0	4	0.8	147.4	0.0	7	-6.6	72.8	0.0
SST_LC33	(1.2 + 0.2 SDS)DL + OmegaEL + f1*LL + f2*SL	1	5.7	71.5	0.0	4	-2.4	147.7	0.0	7	-9.3	75.0	0.0
SST_LC34	(1.2 + 0.2 SDS)DL -OmegaEL + f1*LL + f2*SL	1	9.3	75.0	0.0	4	2.4	147.7	0.0	7	-5.7	71.5	0.0

AISC14 | CoverSheet | Design_Limits | LoadCombos | START | 1 Frame Geometries | 1A PointLoads | 1B LatLoads | 1C LatVertLoads | 2 Drift Summary | Base Reaction

Step 3:

1. Once all the design checks from Step 2 and Step 2A are satisfied, proceed to push the "***3. BLC Summary"*** button. 

   a. If any DCRs in columns "J" through "O" are NOT satisfied in the ***"3. BLC Summary"*** tab, then update column/beam/links sizes as needed on Step 1 and repeat Step 2.

3. BEAM AND YIELD-LINK CHECK SUMMARY																
Conn Count	Elev ID	Grid ID	Story	Beam ID	Beam Size	Link Size	$M_{u\text{-}link}$ (kip-in.)	BRP Size	t_{BRP} DCR	Beam tf, bf DCR	BRP Bolt DCR	Link Strength DCR	Link Slip DCR	$L_{y\text{-}link}$ DCR	Beam b/t	Beam h/tw	Detailed PDF Output
1	A	1	Story2	2001	W24x84	YL6-4	883	BRP6A	0.838	0.663	0.513	0.263	0.273	0.881	Compact	Compact	View PDF

   b. If design changes are made, press the "***3. BLC Summary***" button at the top Ribbon to refresh calculation sheet.

2. Once all DCRs in columns "J" though "O" are satisfied, Push "***Generate PDF***" 

to view detailed PDF files for beam and yield-link checks. Please note these files are saved in the file path created in Step ***"0. Save File"***.

Step 4:

1. When all the DCR numbers for ***"BLC Summary"*** are satisfied, push the "***4. SPC Summary***" button 4.SPC Summary to setup ***"4 SPC Summary"*** sheet.

A	B	C	D	E	F	G	H	I	J	K	L	M	N	O	P	Q	R	S	T	U	V	W	X	Y
	4. SHEAR PLATE CHECK SUMMARY																							
Conn Count	Elev ID	Grid ID	Story	Beam ID	Beam Size	Beam Bay Span (in.)	Link Size	P_{u_sp} (kips)	V_{E_LC08} (kips)	$V_u=2M_{pr}/L_h+V_g$ (kips)	$V_{E\_LC01\text{-}07}$ (kips)	No. vert. bolts	No. horz. bolts	Bolt Size (in.)	Bolt Type	SP Plate Thickness (in.)	No. of SP	Weld Size (in.)	Beam Web DCR	Shear Plate Geometry Check	Shear Plate DCR	Bolt Shear DCR	Fillet Weld DCR	Detailed PDF Output
1	A	1	Story1	1001	W24x84	336	YL6-4	7.71	46.74	85.29	48.3	5	2	7/8	A325-N	3/8	1	4/16	0.432	OK	0.946	0.718	0.601	View PDF
1	A	2	Story1	1002	W24x84	336	YL6-4	7.71	46.74	85.29	48.3	5	2	7/8	A325-N	3/8	1	4/16	0.432	OK	0.946	0.718	0.601	View PDF

2. Verify and revise inputs in columns "N" through "R" as required if DCRs in columns "T" through "X" are NOT satisfied.
   a. Push "***Update SPC DCRs***" button Update SPC DCRs to update calculation values on screen if you make any modifications to the gray user input cells.
   b. Do not push the ***"4. SPC Summary"*** button again unless wanting to reset shear plate inputs to default values.
   c. Push "***Generate PDF***" Generate PDF to view detailed PDF files for shear plate checks.

Step 5:

1. Once all DCRs for the shear plate checks are satisfied, push the "***5. CC Summary***" button 5.CC Summary to setup column check summary tab.

D	E	F	G	H	S	T	U	V	W	X	Y	Z	AA	AB	AC	AD
					5. COLUMN CHECK SUMMARY											
Story	Column Location	Story Height in	Column size	Pu Column (kips)	Bottom Stiffener Provided?	Doubler Plate Provided?	Stiffener Required for AISC 360 J10?	Min. Stiffener Thickness (in)	Min. Doubler Thickness (in)	SCWB DCR Check	Column Pz DCR Check	Column Flange Check	Min. Stiff to flange fillet size	Min. Stiff to web fillet size	Column b/t	Column h/tw
Story1	A-1-Story1	138	W27x114	36.3	YES	NO	YES	3/8	0	N/A	0.936	0.809	3/16	3/16	Compact	Compact
Story1	A-2-Story1	138	W27x114	60.6	YES	NO	YES	3/8	0	N/A	1.114	0.806	5/16	5/16	Compact	Compact
Story1	A-3-Story1	138	W27x114	36.3	YES	NO	YES	3/8	0	N/A	0.936	0.809	3/16	3/16	Compact	Compact

2. Overwrite the **Pu Column** axial load from output file in column "H" as applicable (envelope of load combinations 1 through 22 and 33 through 36).
3. Input if **Bottom Stiffener** and **Doubler Plates** are to be provided. Minimum stiffener size for concentrated force check will be calculated and returned in column "V". Stiffener at beam top flange is assumed for all Yield-Link moment connections.
   a. For the YL moment connection, there is an exception to not require the column lateral bracing at the level of the beam bottom flange and the stiffener may be omitted if not required for capacity checks. See AISC 358-18 Chapter 12 step 12.2 (7) for more information. Option to design with column stability bracing at bottom beam flange can

be selected on the "***Design_Limits"*** tab, cell J30. Column design on step ***"8. Column Summary"*** will adjust based on column bracing selection.

b. When providing doubler plates, it's considered economical to increase column weight 70-100 plf before selecting "Yes" to provide a doubler plate. If doubler plates are included, user can select welding preferences on "***Design_Limits"*** tab.

c. Push "***Update CC Summary***" button 

to calculate and update column DCRs on screen if you make any modifications to the gray user input cells.

d. Do not push ***"5. CC Summary"*** button again unless wanting to reset column connection check inputs to default values.

e. Push "***Generate PDF***" button 

to view detailed PDF files for column connection checks.

Step 6:

1. Once all the DCRs for the column connection checks are satisfied. Push the "***6. Weld Summary"*** button 

to see tabulated summary for 1) shear tab welds, 2) stiffener plate welds, and 3) doubler plate welds. This weld table will be exported to the ***"Frame Elevation"***.

6. Weld Summary

A	B	C	D	E	H	I	J	K	L	M	N	O	P	Q	R
Col.	Elev.	Grid	Story	Column	Left Shear PL		Right Shear PL		Stiff. PL	Doubler PL	1A. Left ST to col flange Weld				
ID	ID	Id	ID	Size	no. SP	thickness, (in.)	no. SP	thickness, (in.)	thickness, (in.)	thickness, (in.)	Fillet Size, (in.)	# of sides	PJP Size, (in.)	# of sides	Fillet Size, (in.)
1	A	1	Story1	W27x114	N/A	N/A	1	3/8	3/8	N/A	N/A	N/A	N/A	N/A	4/16
2	A	2	Story1	W27x114	1	3/8	1	3/8	4/8	N/A	4/16	2	N/A	N/A	4/16
3	A	3	Story1	W27x114	1	3/8	N/A	N/A	3/8	N/A	4/16	2	N/A	N/A	N/A

S	T	U	V	W	X	Y	Z	AA	AB	AC	AD	AE
1B. Righ ST to col flange Weld			2. STP to col web/DP weld		3. STP to col flange weld		4. DP to col web Weld		4A. DP to col web Plug Weld		5. DP to col flange weld	
# of sides	PJP Size, (in.)	# of sides	Fillet Size, (in.)	# of sides	Fillet Size, (in.)	# of sides	Fillet Size, (in.)	# of sides	Diameter, (in.)	Depth, (in.)	Fillet Size, (in.)	# of sides
2	N/A	N/A	3/16	2	3/16	2	N/A	N/A	N/A	N/A	N/A	N/A
2	N/A	N/A	3/16	2	3/16	2	N/A	N/A	N/A	N/A	N/A	N/A
N/A	N/A	N/A	3/16	2	3/16	2	N/A	N/A	N/A	N/A	N/A	N/A

2. Please see the "***Design_Limits"*** tab for the different welding options available for the stiffener and doubler plates. Refresh the weld summary tab by pushing "***6. Weld Summary"*** button.

Step 7:

1. Push the "***7. Beam Summary***" button 7.Beam Summary to set up the "***7 Beam Summary***" tab.

D	E	F	G	L	M	N	AE	AF	AG	AH	AI	AJ	AK	AL	AM	AN	AO
7. Preliminary Beam Design Summary																	LC 7
Beam ID	Beam Size	Link Size	Beam Bay Span (in.)	Lb (in.)	Beam Interm. Bracing	Mcap_link (k-in)	Pu Omega (kips)	Mu Omega (k-in)	Vu Omega (k-in)	Mcap_Link /Mu_Omega	Axial DCR	Flexural DCR	P+M DCR	V DCR	Adj. Flexure DCR	Total P+M DCR	Detailed PDF Output
1001	W24x84	YL6-4	336	303.7	None	5815	22.4	3105	38	1.873	0.097	0.350	0.395	0.124	0.656	0.701	View PDF
1002	W24x84	YL6-4	336	303.7	None	5815	22.4	3105	38	1.873	0.097	0.359	0.403	0.124	0.672	0.716	View PDF

AQ	AR	AS	AT	AU	AV	AW	AX	AY	AZ	BA	BB	BC	BD	BE
7B. Beam Design Per Analyais Software (i.e. RISA 3D) with Mpr_Link Check														
Beam ID	Pu Omega (kips)	Mu Omega (k-in)	Vu Omega (k-in)	Phi*Pn (kips)	Phi*Mn (kip-in)	Phi*Vn (kips)	Mcap_link (kip-in)	M_{cap_link}/ Phi*Mn	Pu_omega/ Phi*Pn	Mu_omega/ Phi*Mn	M_{cap_link}/ M_{u_omega}	Vu DCR	P+M DCR	Adj P+M DCR
1001	22.4	3105.1	37.83	230.0	8890.7	305.8	5815	0.654	0.097	0.349	1.873	0.124	0.398	0.703
1002	22.4	3105.1	37.83	230.0	9786.7	305.8	5815	0.594	0.097	0.317	1.873	0.124	0.366	0.643

a. **Preliminary Beam Design Summary** per AISC 360-16 is performed for the Omega Level load combinations (+/- directions) in the "***7 Beam Summary***" tab. Beams are designed for P+M under the Omega Level demand loads.

b. User may select **Beam Intermediate Bracing** locations (top and bottom flange) in column "M" as needed per AISC *Standard Provisions*. User to apply beam stability bracing locations in user's analysis software as applicable.

c. Beam design data from user software may be overwritten in columns "AR" through "AW" for final output **PDF Summary**. Use envelope of all load combinations for beam end reactions and **Phi*Pn** (Axial Compression Analysis), **Phi*Mn** (Flexural Analysis (Strong Axis)), and **Phi*Vn** (Shear Analysis (Major Axis)).

d. Push the "***Update Beam Summary***" button Beam Summary to calculate and update beam design DCRs on screen, if you make any modifications to the gray user input cells.

e. Do not push the ***"7. Beam Summary"*** button again unless wanting to reset beam inputs to default values. If beam sizes are changed after running this step, it is recommended to refresh complete sheet and re-input user values.

f. Push the "***Generate PDF***" button Generate PDF to view detailed PDF files for beam checks.

Step 8:

1. Push the "***8. Column Summary***" button to set up the "***8 Column Summary***" tab.

D	E	F	G	H	AG	AH	AI	AJ	AK	AL	AM	AN	AO	AP	AQ
8. Preliminary Column Design Summary															LC 7
Story	Column Location	Story Height (in)	Column size	Col Bracing at Bm Bot Flg?	Cb_user	Pu_Ω Column (kips)	Vu_Ω Column (kips)	Mu_Ω(top) Column (kip-in)	Mu_Ω(bot.) Column (kip-in)	Mu_Ω(max) Column (kip-in)	Axial DCR	Flexural DCR	P+M DCR	V DCR	Detailed PDF Output
Story1	A-1-Story1	138	W27x114	YES	1.00	36.3	22.4	3088	0	3088	0.033	0.200	0.217	0.053	View PDF

   a. **Preliminary Column Design** per AISC 360-16 is performed for the Omega Level load combinations (+/- directions) in the "***8 Column Summary***" tab. Columns are designed for P+M under the Omega Level demand loads.
   b. Final member design will need to be performed using the user's analysis and design software (i.e. RISA-3D). Use envelope of all load combinations and update column forces in columns "AH" through "AK" as applicable. Column **Cb** value to be updated in column "AG".
   c. Push the "***Update Column Summary***" button to calculate and update column design DCRs on screen , if you make any modifications to the gray user input cells.
   d. Do not push the ***"8. Column Summary"*** button again unless wanting to reset beam inputs to default values. If column sizes are changed after running this step, it is recommended to refresh complete sheet and re-input user values.
   e. Push the "***Generate PDF***" button to view detailed PDF files for column checks.

Step 9:

1. Click the ***"PDF Summaries"*** button to print out all the design summary tabs.
2. Click the "***Frame Elevation"*** button to generate a DXF file of the frame elevation showing the Yield-Link Moment Connections, shear plate size and geometry, continuity plate size, doubler plates, and weld summary table. Detail Sheets YL-INST1 and YL-INST2 are also created on Paper Space tabs.

Step 10:

Clicking the "***Reset"*** button 

will offer the user options to either reset the complete spreadsheet and delete all the PDF output files or reset only the Moment, Shear and Axial forces per Step 1. (Columns W, X and Y per Figure 3)

Additional Setting:

Under the "***Design Limits***" Tab at the base of the sheet the user has the option to change the different design limit states DCR, material properties and welding preferences.

	A–D	E	F	G–I	J	K
1	User Input for Various Design Limit States:					
2						
3	Initial Link Check:			Beam and Link Check (BLC Details):		
4	DCR	Limit		DCR	Limit	
5	tbf Check:	1.00		Beam tbf_DCR:	1.00	
6	bf Check:	1.00		Link strength DCR:	1.00	
7	Lyield Check:	1.00		Lyield Check:	1.00	
8	Panel Zone DCR:	1.00		tbrp_DCR:	1.00	
9	Drift DCR:	1.00		brp_Bolt_DCR:	1.00	
10						
11	Column Check (BLC Details):			Shear Plate Check (SPC Details):		
12	DCR	Limit		DCR	Limit	
13	SCWL_DCR:	1.00		Beam Web DCR:	1.00	
14	DCR_PZ:	1.00		Shear Plate DCR:	1.00	
15	Stiffener DCR:	1.00		Bolt DCR:	1.00	
16	Column Flange DCR:	1.00		Fillet Weld DCR:	1.00	
17						
18	User Input for Material Properties:					
19						
20	Beam:			Shear Plate:		
21	Fy=	50	ksi	Fy=	50	ksi
22	Fu=	65	ksi	Fu=	65	ksi
23	Column:					
24	Fy=	50	ksi	Doubler Plate:		
25	Fu=	65	ksi	Fy=	50	ksi
26	Stiffener Plate:			Fu=	65	ksi
27	Fy=	50	ksi	tdp_min=	0.25	in
28	Fu=	65	ksi			
29	tstp_min=	0.375	in	Column Bracing at Beam Bot		
30	Bot. Stiffener Depth (1-sided connection)=	Full Depth		Flange (Yes/NO):	YES	
31						

APPENDIX A3: RISA-3D YIELD-LINK MOMENT CONNECTION MODELING

Yield-Link Moment Connection Modeling

in RISA 3-D (V18, 19, 20)

Version 1.0

Date: 04/11/2022

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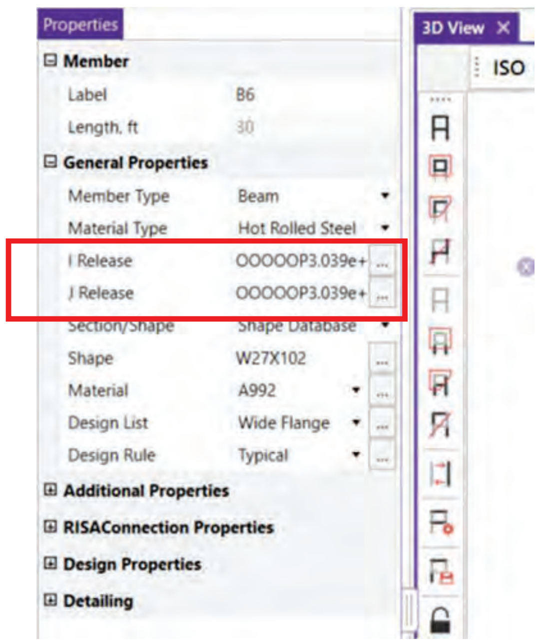
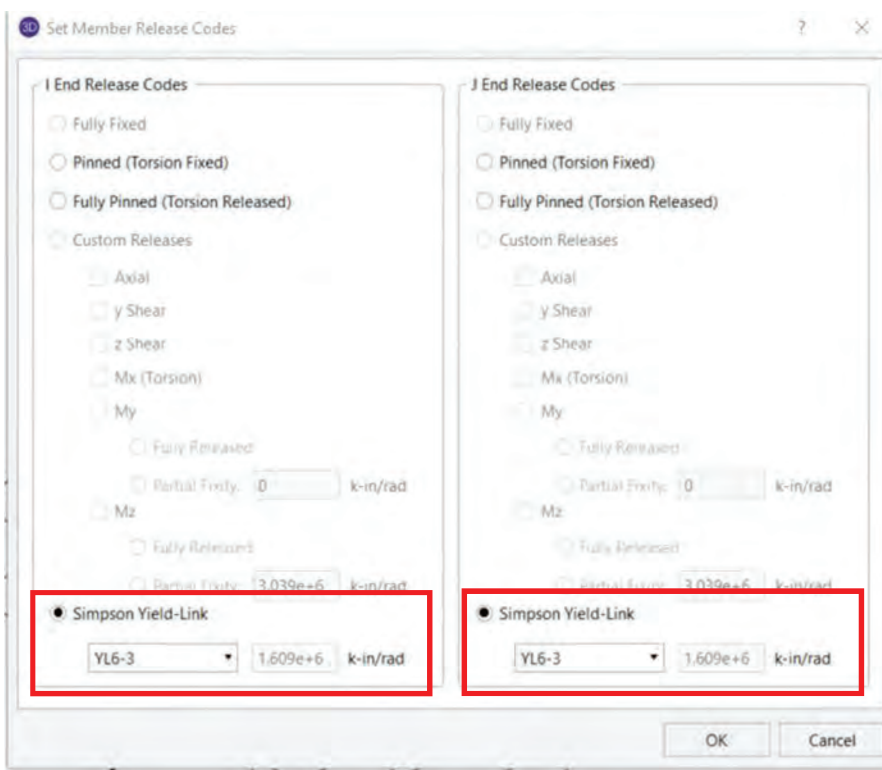
1.0 Yield-Link Moment Connection Input:

1.1. Yield-Link Moment Connection assignment under "Properties"

Yield-Link assignment to a beam-to-column connection is under the Properties Tab in RISA-3D. Under the General Properties, I and J Releases, the user can find the user menu for Simpson Yield-Link Assignment.



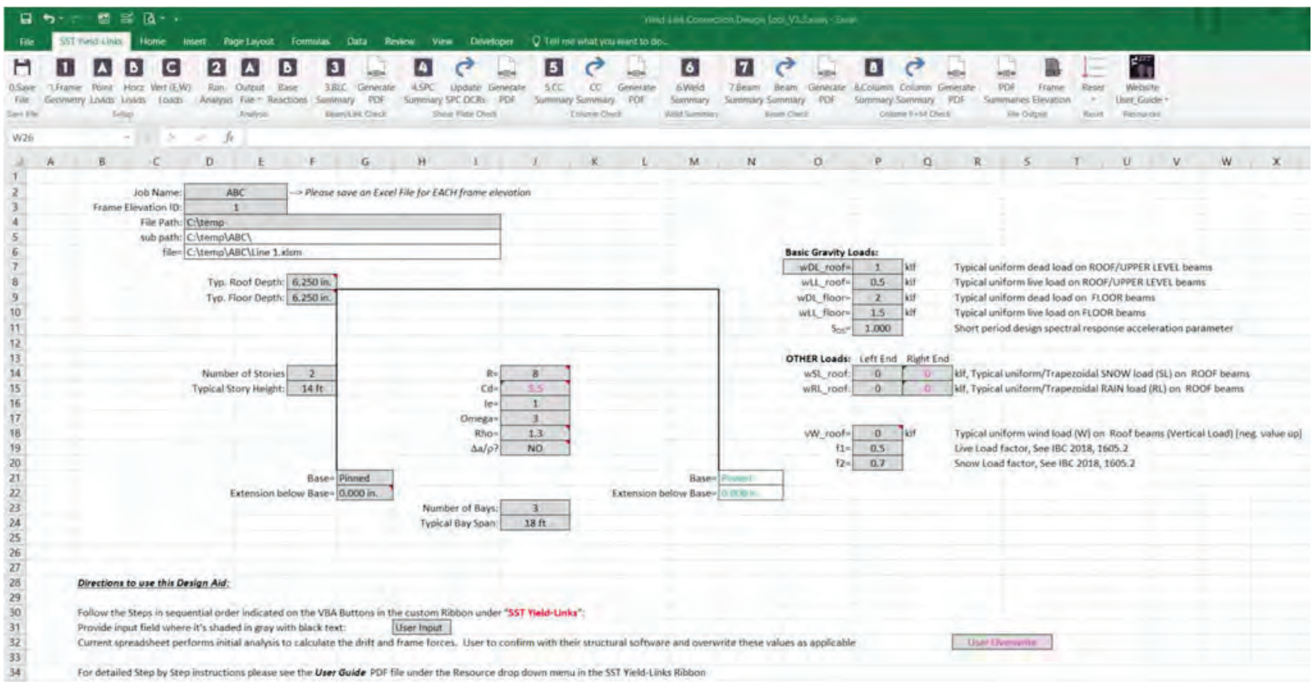


2.0 Yield-Link Beam/Column and Connection Design:

Please note, once the Yield-Link connection has been assigned to the beam RISA-3D will calculate the connection stiffness and apply to the beam-to-column connection. From this point on the user can:

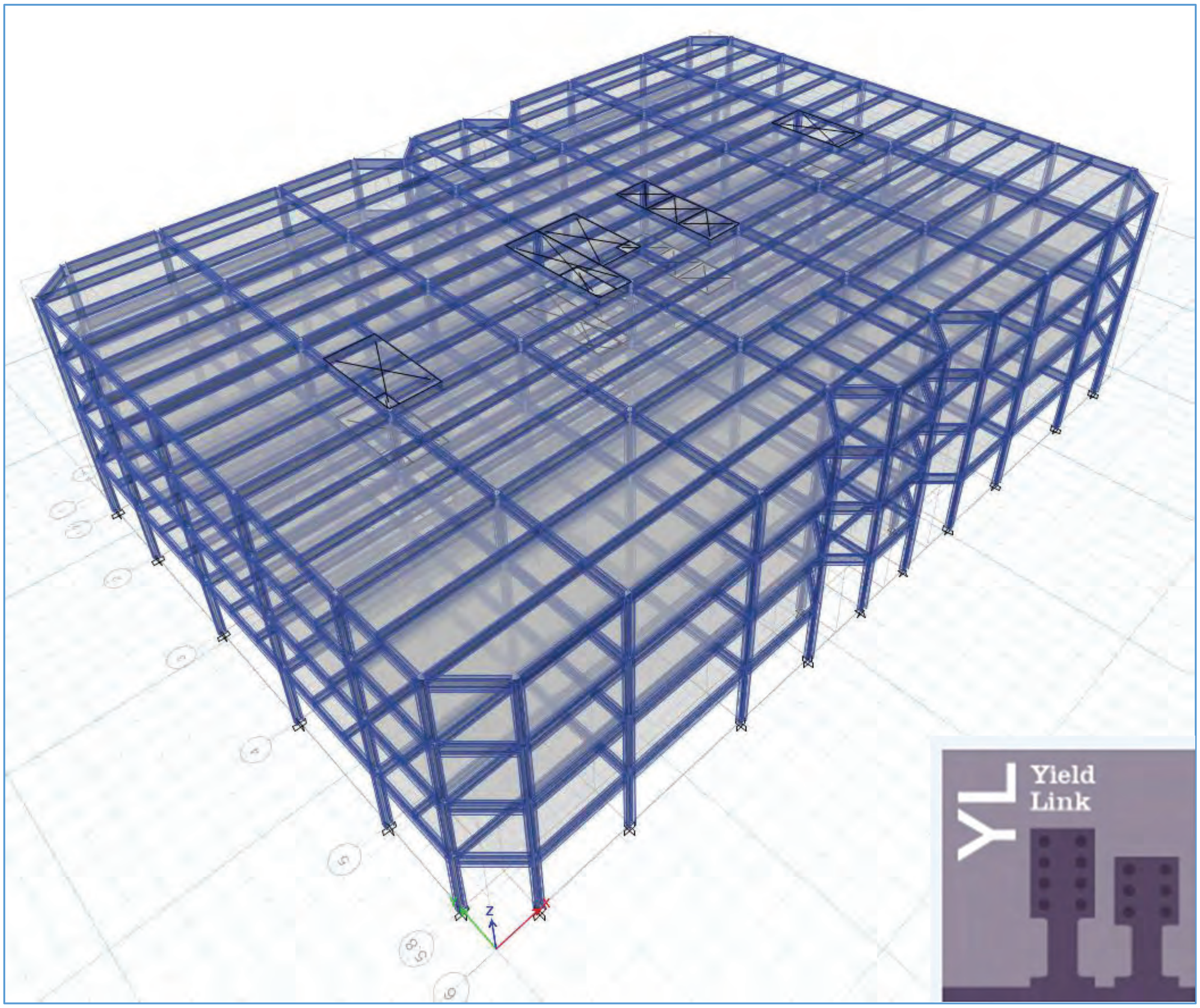
1. Run Drift Check of the structure with the Yield-Link moment connection stiffness incorporated
2. Get lateral force distribution to each of the frames.

Currently RISA-3D do not do Yield-Link strength check, beam/column design or Yield-Link connection design within RISA-3D. The user will have to use the Yield-Link Excel Tool to complete the frame designs. Please refer to Appendix B for user guide on the Yield-Link Moment Connection Excel Design Tool.



APPENDIX A4: ETABS/SAP2000 PLUGIN USER GUIDE

Simpson Strong-Tie Yield Link® Connection SAP2000®/ETABS® Plugin User Guide



Version: 3.2.0

Release Date: 3rd May, 2022

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1.0 Welcome to Simpson Strong-Tie Yield Link® Plugin User Guide

This guide helps navigate through the Simpson Strong-Tie Yield Link® Connection Plugin developed for SAP2000® & ETABS® structural engineering software.

Quick Resources:

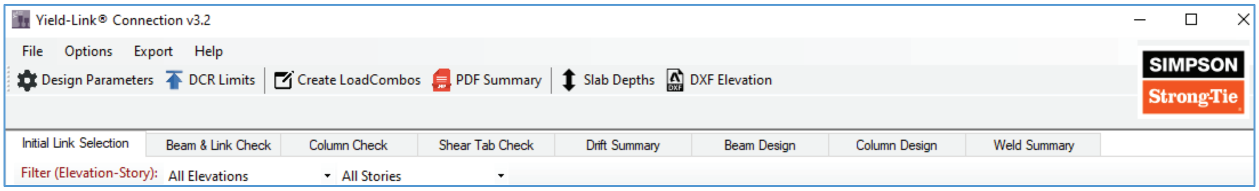
1. Contact a Yield-Link Professional
2. View Product Information
3. Visit our website www.strongtie.com
4. Read ANSI/AISC 358-16 with ANSI/AISC 358s2-20
5. Read ESR-2802

1.1 Overview

Welcome to Yield-Link® Connection Plugin. For first time users, please read License Agreement before proceeding to use this Plugin. SAP2000® & ETABS® are integrated software packages developed by Computers & Structures, Inc. for structural analysis and design of buildings. Yield-Link® Connection Plugin developed by Simpson Strong-Tie makes use of SAP2000® & ETABS® Application Programming Interface (API) to create a seamless user experience for applying the Yield-Link® technology for designing Steel Special Moment Connections. See Yield-Link® Plugin Installation Manual for instructions on how to install the Plugin.

2.0 Yield-Link® Plugin Application Interface

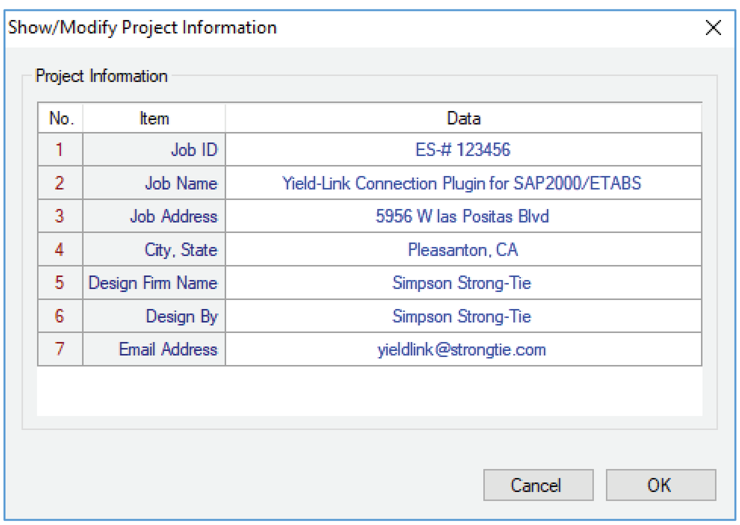
Main Menu



All functions of the Yield-Link® Connection Plugin may be accessed through the main menu at the top of the interface starting with "File" on the far left and ending with "Help" on the far right. Clicking on each of these menus will display a set of sub-menus (listed below). The sub-menus provide various inputs for project information / design parameters as required by the design engineer. Easy access is provided for frequently used features.

2.1 File Menu

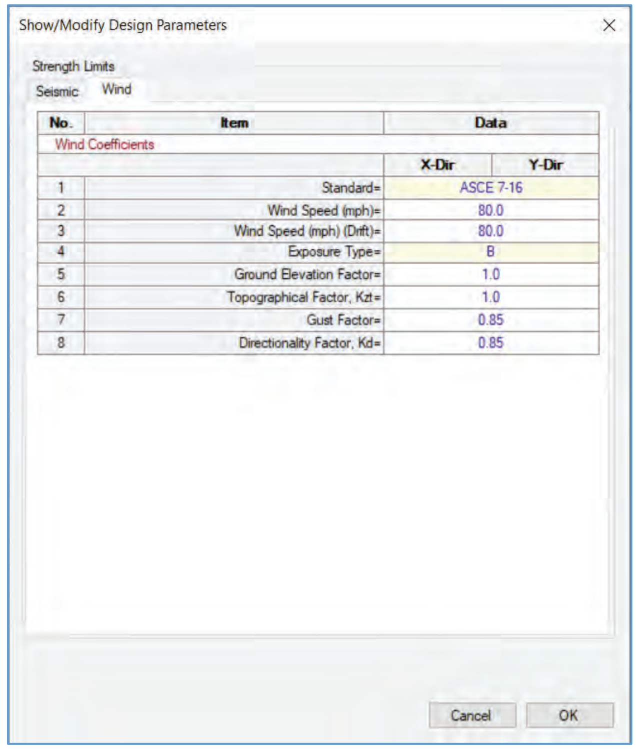
1. ***Show/Modify Project Information*** feature provides the option of entering all necessary project details like Job ID and design firm information for documentation.



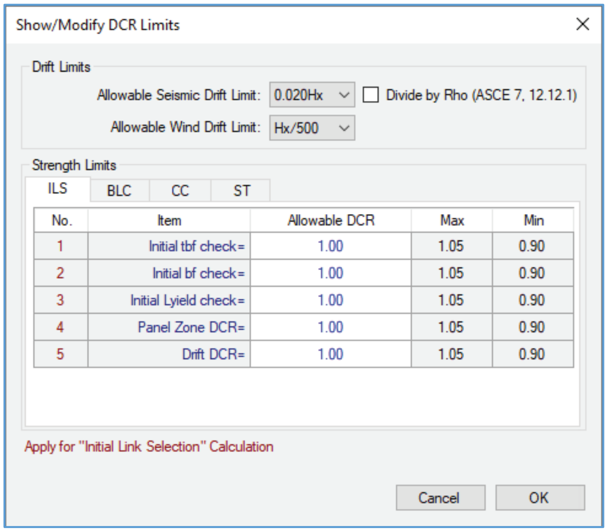
2. ***Save*** option saves the selected frame information as a .XML file.
3. ***Load*** feature can read the saved .XML files.
4. ***Exit*** option will close the Plugin, prompting for saving the results if required.

2.2 Option Menu

1. ***Design Parameters*** feature allows the user to set Seismic Design Parameters, Wind Coefficients as well as Load Factors.
    a. Load factors (f1 & f2), Redundancy factor (Rho) and Overstrength factor (Omega) are used in the load combinations generated by the Plugin using Create Load Combinations feature. (See section 2.2 (3) for detailed information).
    b. Importance factor (I), Response modification coefficient (R) and Deflection Amplification factor (C_d) are used for computing story drift.
    c. For member design, the user is required to manually select the required design preferences like Design code, Frame type, Analysis method etc. through ETABS® as indicated by the warning in red.



2. ***Design Demand Capacity Ratio (DCR) Limits*** feature allows the user to vary allowable demand capacity ratios with a minimum value of 0.9 and maximum of 1.05. This feature also provides the option to select the allowable seismic drift limit from a list of dropdown options as per ASCE/SEI 7-16, Table 12.12-1. as well as the allowable wind drift limit. Note that Hx/125 and Hx/142 wind drift limits are applicable for HCAI projects only.

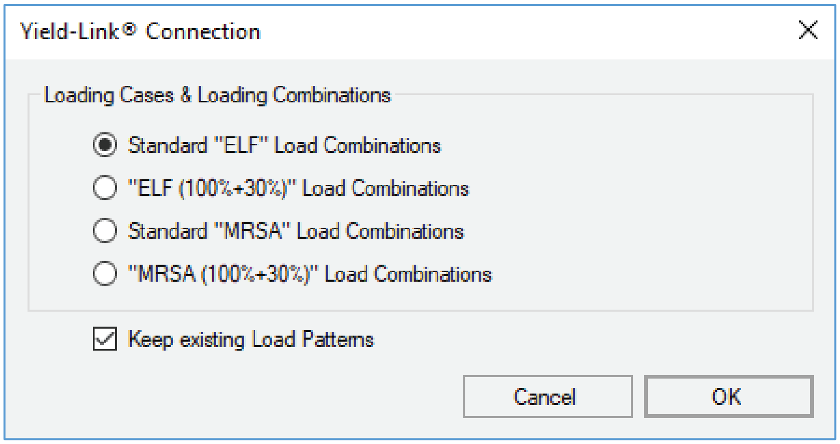


3. ***Create Load Combinations*** feature provides option to create Load Patterns, Load Cases and Load Combinations required for calculating various demand loads used for seismic design. User can select type of analysis as required based on the structure's seismic design category, structural system, dynamic properties and regularity.

   Currently, the Plugin offers two methods based on the type of analysis procedure. First is the Equivalent Lateral Force (ELF) procedure (ASCE/SEI 7-16, Chapter 12.8) with additional option of analyzing corner columns subjected to 100% lateral loads in the one direction + 30% additional loads in the perpendicular direction for each procedure. (ASCE/SEI 7-16, Chapter 12.5.3.1a) Second is the Modal Response Spectrum Analysis (MRSA) (ASCE/SEI 7-16, Chapter 12.9) also with the additional option of analyzing corner columns subjected to 100% lateral loads in the one direction + 30% additional loads in the perpendicular direction for each procedure. (ASCE/SEI 7-16, Chapter 12.5.3.1a)

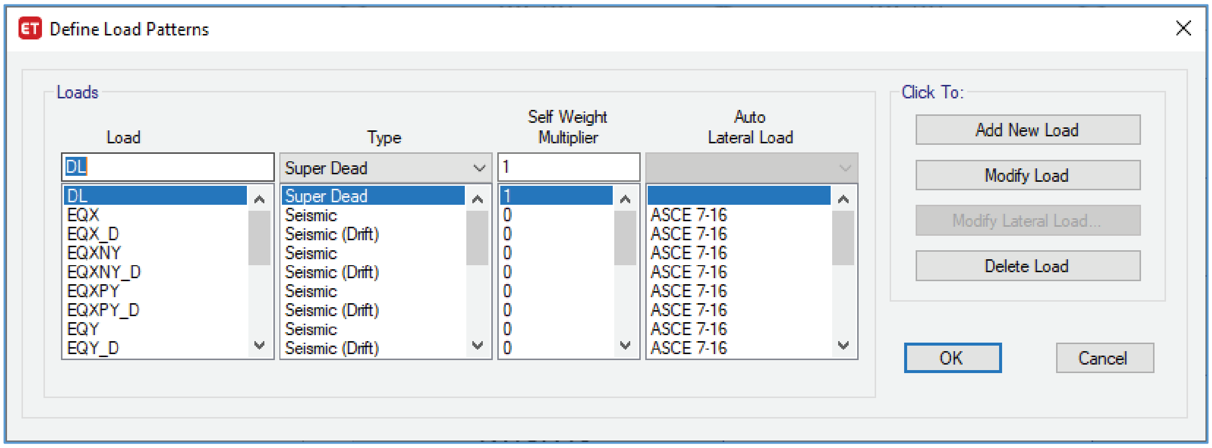
   Selecting one of the two available (ELF or MRSA) analytical procedures creates a set of load patterns, load cases and load combinations as required per ASCE 7-16 as explained in the following section.

A. *Load Combinations per Equivalent Lateral Force (ELF) Procedure:*



a. Load Patterns: A total of 21 load patterns are created for generating load cases and required load combinations. Load patterns will auto-populate in the "Load Patterns" option under "Define" menu.

ID	Load Patterns	Type
1	DL	Dead
2	LL	Live
3	LR	Roof Live
4	SL	Snow
5	RL	Rain
6	W	Wind (Strength)
7	NLX	Notional
8	NLY	
9	EQX	Seismic (Strength)
10	EQXPY	
11	EQXNY	
12	EQY	
13	EQYPX	
14	EQYNX	
15	EQX_D	Seismic (Drift)
16	EQXPY_D	
17	EQXNY_D	
18	EQY_D	
19	EQYPX_D	
20	EQYNX_D	
21	W_D	Wind (Drift)



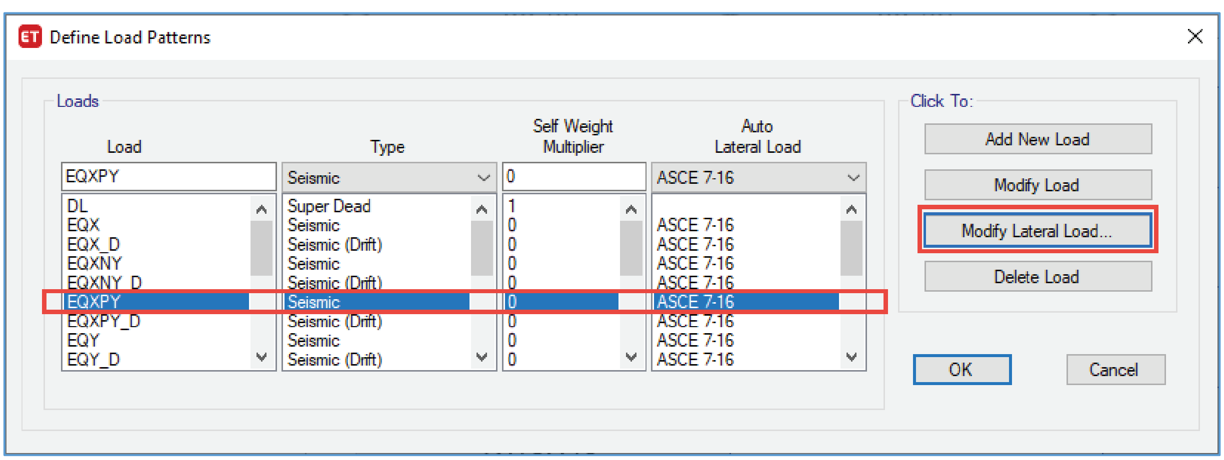
User Note [1]:

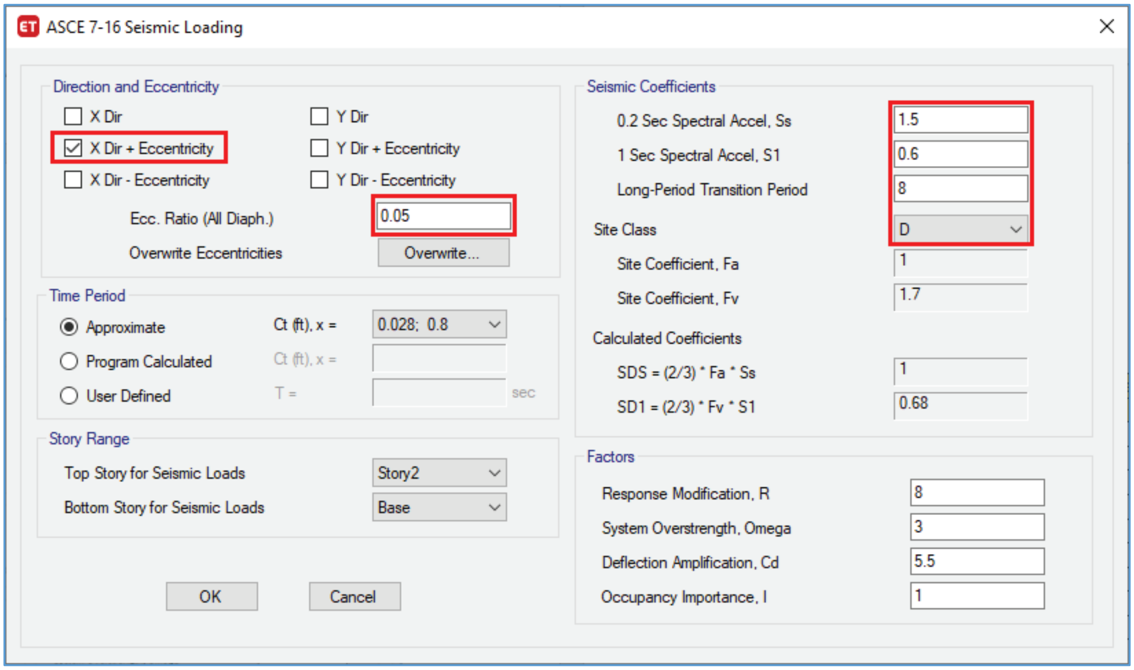
I. Response Modification Coefficient (R), Overstrength Factor (Omega), Deflection Amplification Factor (C_d) and Occupancy Importance Factor (I_e) are used to calculate parameters like seismic base shear and inter-story drift. These factors can be defined using Plugin interface as explained in section 2.2(1).

II. Direction + Eccentricity applies to Rigid and Semi-Rigid diaphragms. See SAP2000® & ETABS® user manuals for more information.

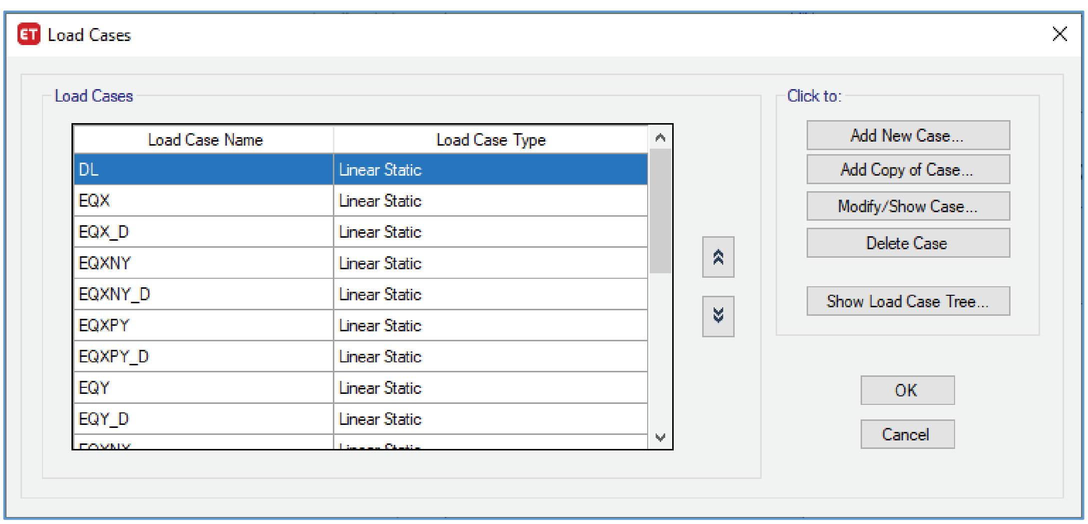
III. Seismic coefficients are calculated based on parameters like mapped MCE_R (5% damped) spectral response acceleration at a period of 1 second (S_1), mapped MCE_R (5% damped) spectral response acceleration at short periods (S_S) and Site Class. Currently, the ETABS®/SAP2000® API limits the modification lateral load parameters such as direction, eccentricity and site class. Hence, user is required to manually enter this information. See Appendix A for alternative approach when using ETABS®.

IV. Example shows case of Seismic Loading along X-Axis with 5% positive eccentricity (EQXPY). User must select "Modify Lateral Load" option and check the box for X-Dir + Eccentricity, Specify Eccentricity Ratio of 0.05, Seismic Coefficients and required Site Class based on type of soils and their engineering properties.





b. ***Load Cases:*** Total of 22 Load Cases corresponding to the Load Patterns listed above are created. All load cases are linear static with the exception of a P-DELTA Case which is a Nonlinear Static Load Case. Load Cases will auto-populate in the "Load Cases" option under "Define" menu.



ID	Load Cases	Load Pattern Multipliers										Action
		PDELTA	DL	LL	LR	SL	RL	W	W_D	NLX	NLY	
1	DL	0	1	0	0	0	0	0	0	0	0	Additive
2	LL	0	0	1	0	0	0	0	0	0	0	Additive
3	LR	0	0	0	1	0	0	0	0	0	0	Additive
4	SL	0	0	0	0	1	0	0	0	0	0	Additive
5	RL	0	0	0	0	0	1	0	0	0	0	Additive
6	W	0	0	0	0	0	0	1	0	0	0	Additive
7	W_D	0	0	0	0	0	0	0	1	0	0	Additive
8	NLX	0	0	0	0	0	0	0	0	1	0	Additive
9	NLY	0	0	0	0	0	0	0	0	0	1	Additive
10	PDELTA	0	1	0.5	0.5	0.5	0.5	0	0	0	0	Additive

ID	Load Cases	Load Pattern Multipliers							Action
		PDELTA	EQX	EQXPX	EQXNX	EQY	EQYPY	EQYNY	
11	EQX	1	1	0	0	0	0	0	Additive
12	EQXPY	1	0	1	0	0	0	0	Additive
13	EQXNY	1	0	0	1	0	0	0	Additive
14	EQY	1	0	0	0	1	0	0	Additive
15	EQYPX	1	0	0	0	0	1	0	Additive
16	EQYNX	1	0	0	0	0	0	1	Additive

ID	Load Cases	Load Pattern Multipliers							Action
		PDELTA	EQX_D	EQXPX_D	EQXNX_D	EQY_D	EQYPY_D	EQYNY_D	
17	EQX_D	1	1	0	0	0	0	0	Additive
18	EQXPY_D	1	0	1	0	0	0	0	Additive
19	EQXNY_D	1	0	0	1	0	0	0	Additive
20	EQY_D	1	0	0	0	1	0	0	Additive
21	EQYPX_D	1	0	0	0	0	1	0	Additive
22	EQYNX_D	1	0	0	0	0	0	1	Additive

c. ***Load Combinations:*** For analysis using standard Equivalent Lateral Force (ELF) procedure, (ASCE/SEI 7-16, Section 12.8) a total of 40 load combinations are generated by the Plugin. These combinations are required for determining the load demands on the lateral force resisting frames which help design the Yield-Link® Moment Connection and verify beam and column sizes.

User Note [2]:

I. Live load factor (f1), Snow load factor (f2), Design (5% damped) spectral response acceleration at short periods (S_{DS}) and Redundancy Factor (ρ) can all be modified using Show/Modify Design Parameters feature as explained in step 2.2(1) of this User Guide.

ID	Load Combinations	Load Pattern Multipliers				Action
		NLX	NLY	W	W_D	
NL	NL	1	1	0	0	Envelope
WL	WL	0	0	1	0	Envelope
WL_D	WL_D	0	0	0	1	Envelope

ID	Load Combinations	Load Pattern Multipliers						Action
		EQX	EQXPX	EQXNX	EQY	EQXPY	EQXNY	
EL	EL	1	1	1	1	1	1	Envelope

ID	Load Combinations	Load Pattern Multipliers						Action
		EQX_D	EQXPX_D	EQXNX_D	EQY_D	EQYPY_D	EQYNY_D	
EL_D	EL_D	1	1	1	1	1	1	Envelope

ID	Load Combinations	Load Multipliers						Action
		DL	LL	LR	SL	RL	NL	
SST_LC1	1.4 DL + NL	1.4	0	0	0	0	1	Additive
SST_LC2	1.2 DL + 1.6 LL + 0.5 LR + NL	1.2	1.6	0.5	0	0	1	Additive
SST_LC3	1.2 DL + 1.6 LL + 0.5 SL + NL	1.2	1.6	0	0.5	0	1	Additive
SST_LC4	1.2 DL + 1.6 LL + 0.5 RL + NL	1.2	1.6	0	0	0.5	1	Additive
SST_LC5	1.2 DL + 1.6 LR + 0.5 LL + NL	1.2	0.5	1.6	0	0	1	Additive
SST_LC6	1.2 DL + 1.6 SL + 0.5 LL + NL	1.2	0.5	0	1.6	0	1	Additive
SST_LC7	1.2 DL + 1.6 RL + 0.5 LL + NL	1.2	0.5	0	0	1.6	1	Additive
SST_LC8	$(1.2 + 0.2*S_{DS})$DL + f1*LL + f2*SL	$(1.2 + 0.2*S_{DS})$	f1	0	f2	0	0	Additive

ID	Load Combinations	Load Multipliers							Action
		DL	LL	LR	SL	RL	WL	WL_D	
SST_LC9	1.2 DL + 1.6 LR + 0.5 WL	1.2	0	1.6	0	0	0.5	0	Additive
SST_LC10	1.2 DL + 1.6 LR - 0.5 WL	1.2	0	1.6	0	0	-0.5	0	Additive
SST_LC11	1.2 DL + 1.6 SL + 0.5 WL	1.2	0	0	1.6	0	0.5	0	Additive
SST_LC12	1.2 DL + 1.6 SL - 0.5 WL	1.2	0	0	1.6	0	-0.5	0	Additive
SST_LC13	1.2 DL + 1.6 RL + 0.5 WL	1.2	0	0	0	1.6	0.5	0	Additive
SST_LC14	1.2 DL + 1.6 RL - 0.5 WL	1.2	0	0	0	1.6	-0.5	0	Additive
SST_LC15	1.2 DL + 1.0 WL + 0.5 LL + 0.5 LR	1.2	0.5	0.5	0	0	1.0	0	Additive
SST_LC16	1.2 DL - 1.0 WL + 0.5 LL + 0.5 LR	1.2	0.5	0.5	0	0	-1.0	0	Additive
SST_LC17	1.2 DL + 1.0 WL + 0.5 LL + 0.5 SL	1.2	0.5	0	0.5	0	1.0	0	Additive
SST_LC18	1.2 DL - 1.0 WL + 0.5 LL + 0.5 SL	1.2	0.5	0	0.5	0	-1.0	0	Additive
SST_LC19	1.2 DL + 1.0 WL + 0.5 LL + 0.5 RL	1.2	0.5	0	0	0.5	1.0	0	Additive
SST_LC20	1.2 DL - 1.0 WL + 0.5 LL + 0.5 RL	1.2	0.5	0	0	0.5	-1.0	0	Additive
SST_LC21	0.9 DL + 1.0 WL	0.9	0	0	0	0	1.0	0	Additive
SST_LC22	0.9 DL - 1.0 WL	0.9	0	0	0	0	-1.0	0	Additive
SST_LC23	1.0 DL + 0.5 LL + 0.5 LR + 1.0 WL_D	1.0	0.5	0.5	0	0	0	1.0	Additive
SST_LC24	1.0 DL + 0.5 LL + 0.5 LR – 1.0 WL_D	1.0	0.5	0.5	0	0	0	-1.0	Additive

ID	Load Combinations	Load Multipliers					Action
		DL	LL	SL	EL	EL_D	
SST_LC25	$(1.2 + 0.2*S_{DS})$DL + EL*ρ + f1*LL + f2*SL	$(1.2 + 0.2*S_{DS})$	f1	f2	ρ	0	Additive
SST_LC26	$(1.2 + 0.2*S_{DS})$DL - EL*ρ + f1*LL + f2* SL	$(1.2 + 0.2*S_{DS})$	f1	f2	$-\rho$	0	Additive
SST_LC27	$(0.9 - 0.2*S_{DS})$ DL + EL*ρ	$(0.9 - 0.2*S_{DS})$	0	0	ρ	0	Additive
SST_LC28	$(0.9 - 0.2*S_{DS})$ DL - EL*ρ	$(0.9 - 0.2*S_{DS})$	0	0	$-\rho$	0	Additive
SST_LC29	$(1.2 + 0.2*S_{DS})$DL + EL_D+ f1*LL + f2*SL	$(1.2 + 0.2*S_{DS})$	f1	f2	0	1.0	Additive
SST_LC30	$(1.2 + 0.2*S_{DS})$DL - EL_D+ f1*LL + f2* SL	$(1.2 + 0.2*S_{DS})$	f1	f2	0	-1.0	Additive
SST_LC31	$(0.9 - 0.2*S_{DS})$ DL + EL_D	$(0.9 - 0.2*S_{DS})$	0	0	0	1.0	Additive
SST_LC32	$(0.9 - 0.2*S_{DS})$ DL - EL_D	$(0.9 - 0.2*S_{DS})$	0	0	0	-1.0	Additive
SST_LC33	$(1.2 + 0.2*S_{DS})$DL + ΩEL + f1*LL + f2*SL	$(1.2 + 0.2*S_{DS})$	f1	f2	Ω	0	Additive
SST_LC34	$(1.2 + 0.2*S_{DS})$DL -ΩEL + f1*LL + f2*SL	$(1.2 + 0.2*S_{DS})$	f1	f2	$-\Omega$	0	Additive
SST_LC35	$(0.9 - 0.2*S_{DS})$ DL + ΩEL	$(0.9 - 0.2*S_{DS})$	0	0	Ω	0	Additive
SST_LC36	$(0.9 - 0.2*S_{DS})$ DL - ΩEL	$(0.9 - 0.2*S_{DS})$	0	0	$-\Omega$	0	Additive

B. Load Combinations for case of corner columns subjected to 100% lateral loads in one direction and additional 30% in the perpendicular direction using ELF procedure.

User Note [3]:

I. Additional 16 Load Cases & 5 Load Combinations are created when "ELF (100%+30%)" Load Combinations option is selected as listed below.

ID	Load Combinations	Load Case Multipliers						Action
		EQX	EQXPY	EQXNY	EQY	EQYPX	EQYNX	
22	ELF 100XPY+30Y	0	1	0	0.3	0	0	Additive
23	ELF 100XNY+30Y	0	0	1	0.3	0	0	Additive
24	ELF 100XPY-30Y	0	1	0	-0.3	0	0	Additive
25	ELF 100XNY-30Y	0	0	1	-0.3	0	0	Additive
26	ELF -100XPY+30Y	0	-1	0	0.3	0	0	Additive
27	ELF -100XNY+30Y	0	0	-1	0.3	0	0	Additive
28	ELF -100XPY-30Y	0	-1	0	-0.3	0	0	Additive
29	ELF -100XNY-30Y	0	0	-1	-0.3	0	0	Additive
30	ELF 30X+100YPX	0.3	0	0	0	1	0	Additive
31	ELF 30X+100YNX	0.3	0	0	0	0	1	Additive
32	ELF 30X-100YPX	-0.3	0	0	0	1	0	Additive
33	ELF 30X-100YNX	-0.3	0	0	0	0	1	Additive
34	ELF -30X+100YPX	0.3	0	0	0	-1	0	Additive
35	ELF -30X+100YNX	0.3	0	0	0	0	-1	Additive
36	ELF -30X-100YPX	-0.3	0	0	0	-1	0	Additive
37	ELF -30X-100YNX	-0.3	0	0	0	0	-1	Additive

ID	Load Combinations	Load Case Multipliers																Action
		22	23	24	25	26	27	28	29	30	31	32	33	34	35	36	37	
EL_100+30	EL_100+30	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	Envelope

ID	Load Combinations	Load Multipliers				Action
		DL	LL	SL	EL_100+30	
SST_LC37	(1.2 + 0.2 SDS)DL + Ω(EL_100+30) + f1*LL + f2*SL	(1.2 + 0.2 SDS)	f1	f2	Ω	Additive
SST_LC38	(1.2 + 0.2 SDS)DL -Ω(EL_100+30) + f1*LL + f2*SL	(1.2 + 0.2 SDS)	f1	f2	-Ω	Additive
SST_LC39	(0.9 - 0.2 SDS) DL + Ω(EL_100+30)	(0.9 - 0.2 SDS)	0	0	Ω	Additive
SST_LC40	(0.9 - 0.2 SDS) DL - Ω(EL_100+30)	(0.9 - 0.2 SDS)	0	0	-Ω	Additive

Tables 1.1 & 1.2 illustrate the use of SST Load combinations for various design checks.

Structural elements including Yield-Links, Beams & Columns are designed for maximum demand loads from LRFD Load Combinations for Strength Design and Structural Integrity per Sections 2.3 & 2.6, ASCE 7-16. Beams and columns are designed for seismic load effects including system overstrength. Seismic story drift is determined using strength level seismic forces per Section 12.8.6, ASCE 7-16.

Table 1.1: Design Check Load Combinations (LRFD)

ID	Load Combinations (LRFD)	Design Check					
		Link Strength	Seismic Drift	Wind Drift	Beam (P+M)	Column (P+M)	V_bm Gravity
SST_LC1	1.4 DL + NL	x	x	x	x	x	x
SST_LC2	1.2 DL + 1.6 LL + 0.5 LR + NL	x	x	x	x	x	x
SST_LC3	1.2 DL + 1.6 LL + 0.5 SL + NL	x	x	x	x	x	x
SST_LC4	1.2 DL + 1.6 LL + 0.5 RL + NL	x	x	x	x	x	x
SST_LC5	1.2 DL + 1.6 LR + 0.5 LL + NL	x	x	x	x	x	x
SST_LC6	1.2 DL + 1.6 SL + 0.5 LL + NL	x	x	x	x	x	x
SST_LC7	1.2 DL + 1.6 RL + 0.5 LL + NL	x	x	x	x	x	x
SST_LC8	(1.2 + 0.2*SDS)DL + f1*LL + f2*SL	x	x	x	x	x	x
SST_LC9	1.2 DL + 1.6 LR + 0.5 WL	x			x	x	
SST_LC10	1.2 DL + 1.6 LR - 0.5 WL	x			x	x	
SST_LC11	1.2 DL + 1.6 SL + 0.5 WL	x			x	x	
SST_LC12	1.2 DL + 1.6 SL - 0.5 WL	x			x	x	
SST_LC13	1.2 DL + 1.6 RL + 0.5 WL	x			x	x	
SST_LC14	1.2 DL + 1.6 RL - 0.5 WL	x			x	x	
SST_LC15	1.2 DL + 1.0 WL + 0.5 LL + 0.5 LR	x			x	x	
SST_LC16	1.2 DL - 1.0 WL + 0.5 LL + 0.5 LR	x			x	x	
SST_LC17	1.2 DL + 1.0 WL + 0.5 LL + 0.5 SL	x			x	x	
SST_LC18	1.2 DL - 1.0 WL + 0.5 LL + 0.5 SL	x			x	x	
SST_LC19	1.2 DL + 1.0 WL + 0.5 LL + 0.5 RL	x			x	x	
SST_LC20	1.2 DL - 1.0 WL + 0.5 LL + 0.5 RL	x			x	x	
SST_LC21	0.9 DL + 1.0 WL	x			x	x	
SST_LC22	0.9 DL - 1.0 WL	x			x	x	
SST_LC23	1.0 DL + 0.5 LL + 0.5 LR + 1.0 WL_D			x			
SST_LC24	1.0 DL + 0.5 LL + 0.5 LR – 1.0 WL_D			x			
SST_LC25	(1.2 + 0.2*SDS)DL + EL*ρ + f1*LL + f2*SL	x					
SST_LC26	(1.2 + 0.2 SDS)DL - EL*ρ + f1*LL + f2* SL	x					
SST_LC27	(0.9 - 0.2 SDS) DL + EL*ρ	x					
SST_LC28	(0.9 - 0.2 SDS) DL - EL*ρ	x					
SST_LC29	(1.2 + 0.2*SDS)DL + EL_D+ f1*LL + f2*SL		x				
SST_LC30	(1.2 + 0.2 SDS)DL - EL_D+ f1*LL + f2* SL		x				
SST_LC31	(0.9 - 0.2 SDS) DL + EL_D		x				
SST_LC32	(0.9 - 0.2 SDS) DL - EL_D		x				

Table 1.2: Design Check Load Combinations (System Overstrength)

ID	Load Combinations (Omega)	Design Check					
		Link Strength	Seismic Drift	Wind Drift	Beam (P+M)	Column (P+M)	V_bm Gravity
SST_LC33	(1.2 + 0.2 SDS)DL + ΩEL + f1*LL + f2*SL				x	x	
SST_LC34	(1.2 + 0.2 SDS)DL -ΩEL + f1*LL + f2*SL				x	x	
SST_LC35	(0.9 - 0.2 SDS) DL + ΩEL				x	x	
SST_LC36	(0.9 - 0.2 SDS) DL - ΩEL				x	x	
SST_LC37	(1.2 + 0.2 SDS)DL + Ω(EL_100+30) + f1*LL + f2*SL					x	
SST_LC38	(1.2 + 0.2 SDS)DL -Ω(EL_100+30) + f1*LL + f2*SL					x	
SST_LC39	(0.9 - 0.2 SDS) DL + Ω(EL_100+30)					x	
SST_LC40	(0.9 - 0.2 SDS) DL - Ω(EL_100+30)					x	

C. Load Combinations per Modal Response Spectrum Analysis (MRSA) Procedure:

User Note [4]:

I. A function "SPEC" is defined which represents a response spectrum per seismic coefficients and design code defined using the Show/Modify Design Parameters feature in the Plugin UI.

II. A total of 12 additional response spectrum load cases are defined with a default scale factor of 1xg (=386.09 in./sec^2)

III. Scale Factors are modified using the ratio of calculated base shear using ELF procedure to calculated base shear using MRSA procedure.

SFX = Base Shear_EQX / Base Shear_SPECX * 1xg

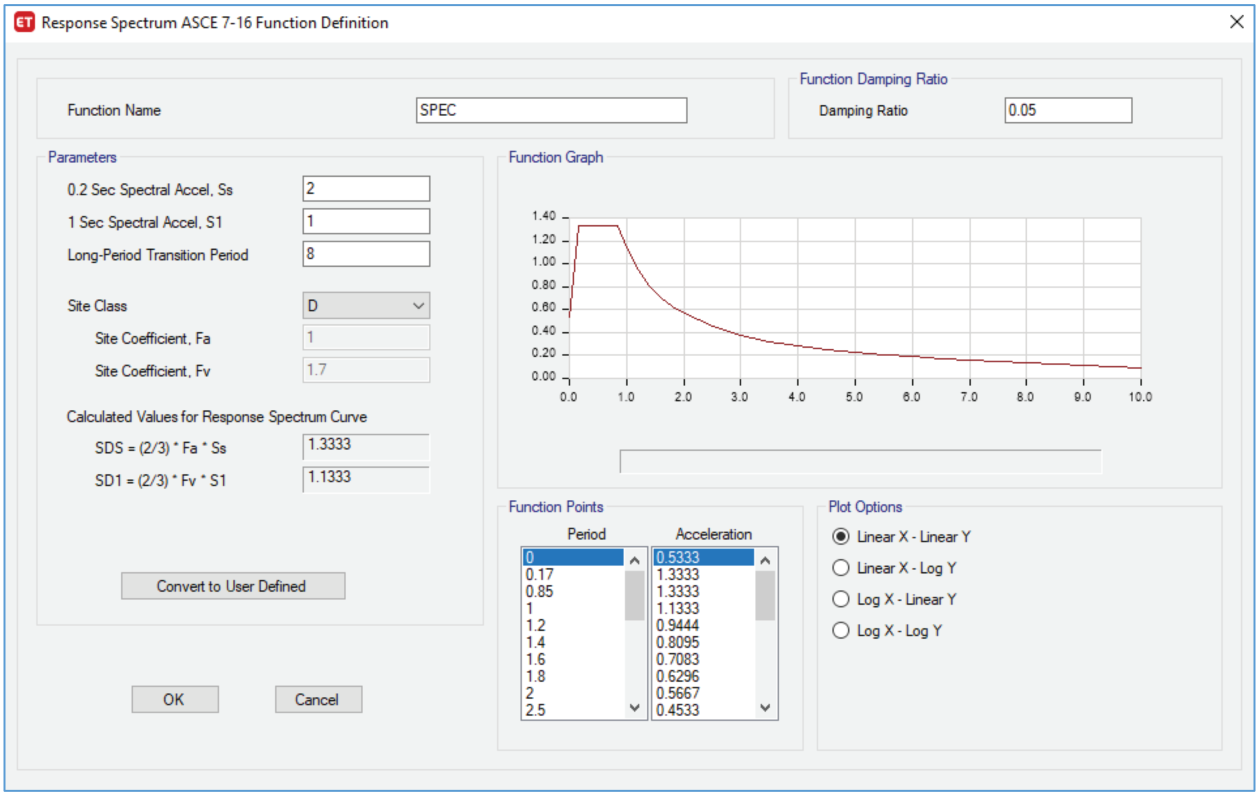
SFY = Base Shear_EQY / Base Shear_SPECY * 1xg

SFX_D = Base Shear_EQX_D / Base Shear_SPECX_D * 1xg

SFY_D = Base Shear_EQY_D / Base Shear_SPECY_D * 1xg

IV. Modal Mass Participation along X and Y directions must be greater than 90%

V. Load Combinations EL and EL_D are defined using the newly created response spectrum load cases.



ID	Load Cases	Load Type	Load Name	Function	Scale Factor	Eccentricity Ratio
22	SPECX	Acceleration	U1	SPEC	SFX	0
23	SPECX_PY	Acceleration	U1	SPEC	SFX	0.05
24	SPECX_NY	Acceleration	U1	SPEC	SFX	-0.05
25	SPECY	Acceleration	U2	SPEC	SFY	0
26	SPECY_PX	Acceleration	U2	SPEC	SFY	0.05
27	SPECY_NX	Acceleration	U2	SPEC	SFY	-0.05

ID	Load Cases	Load Type	Load Name	Function	Scale Factor	Eccentricity Ratio
28	SPECX_D	Acceleration	U1	SPEC	SFX_D	0
29	SPECX_PY_D	Acceleration	U1	SPEC	SFX_D	0.05
30	SPECX_NY_D	Acceleration	U1	SPEC	SFX_D	-0.05
31	SPECY_D	Acceleration	U2	SPEC	SFY_D	0
32	SPECY_PX_D	Acceleration	U2	SPEC	SFY_D	0.05
33	SPECY_NX_D	Acceleration	U2	SPEC	SFY_D	-0.05

ID	Load Combinations	Load Pattern Multipliers						Action
		SPECX	SPECX_PY	SPECX_NY	SPECY	SPECY_PX	SPECY_NX	
EL	EL	1	1	1	1	1	1	Envelope

ID	Load Combinations	Load Pattern Multipliers						Action
		SPECX_D	SPECX_PY_D	SPECX_NY_D	SPECY_D	SPECY_PX_D	SPECY_NX_D	
EL_D	EL_D	1	1	1	1	1	1	Envelope

D. Load Combinations for case of corner columns subjected to 100% lateral loads in one direction and additional 30% in the perpendicular direction using MRSA procedure.

User Note [5]:

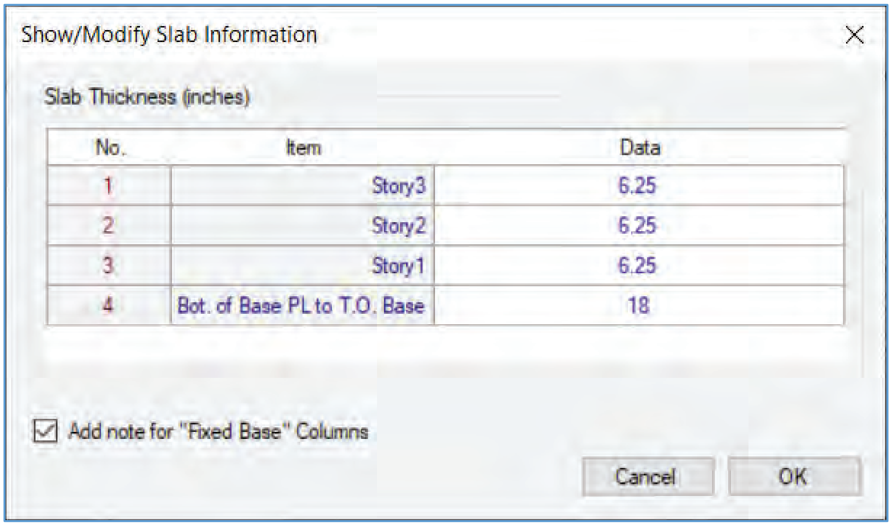
I. Additional 16 Load Cases & 5 Load Combinations are created when "MRSA (100%+30%)" Load Combinations option is selected as listed below.

ID	Load Combinations	Load Case Multipliers						Action
		SPECX	SPECX_PY	SPECX_NY	SPECY	SPECY_PX	SPECY_NX	
22	MRSA 100XPY+30Y	0	1	0	0.3	0	0	Additive
23	MRSA 100XNY+30Y	0	0	1	0.3	0	0	Additive
24	MRSA 100XPY-30Y	0	1	0	-0.3	0	0	Additive
25	MRSA 100XNY-30Y	0	0	1	-0.3	0	0	Additive
26	MRSA -100XPY+30Y	0	-1	0	0.3	0	0	Additive
27	MRSA -100XNY+30Y	0	0	-1	0.3	0	0	Additive
28	MRSA -100XPY-30Y	0	-1	0	-0.3	0	0	Additive
29	MRSA -100XNY-30Y	0	0	-1	-0.3	0	0	Additive
30	MRSA 30X+100YPX	0.3	0	0	0	1	0	Additive
31	MRSA 30X+100YNX	0.3	0	0	0	0	1	Additive
32	MRSA 30X-100YPX	-0.3	0	0	0	1	0	Additive
33	MRSA 30X-100YNX	-0.3	0	0	0	0	1	Additive
34	MRSA -30X+100YPX	0.3	0	0	0	-1	0	Additive
35	MRSA -30X+100YNX	0.3	0	0	0	0	-1	Additive
36	MRSA -30X-100YPX	-0.3	0	0	0	-1	0	Additive
37	MRSA -30X-100YNX	-0.3	0	0	0	0	-1	Additive

ID	Load Combinations	Load Case Multipliers																Action
		22	23	24	25	26	27	28	29	30	31	32	33	34	35	36	37	
EL_100+30	EL_100+30	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	Envelope

ID	Load Combinations	Load Multipliers				Action
		DL	LL	SL	EL_100+30	
SST_LC37	(1.2 + 0.2 SDS)DL + Ω(EL_100+30) + f1*LL + f2*SL	(1.2 + 0.2 SDS)	f1	f2	Ω	Additive
SST_LC38	(1.2 + 0.2 SDS)DL -Ω(EL_100+30) + f1*LL + f2*SL	(1.2 + 0.2 SDS)	f1	f2	-Ω	Additive
SST_LC39	(0.9 - 0.2 SDS) DL + Ω(EL_100+30)	(0.9 - 0.2 SDS)	0	0	Ω	Additive
SST_LC40	(0.9 - 0.2 SDS) DL - Ω(EL_100+30)	(0.9 - 0.2 SDS)	0	0	-Ω	Additive

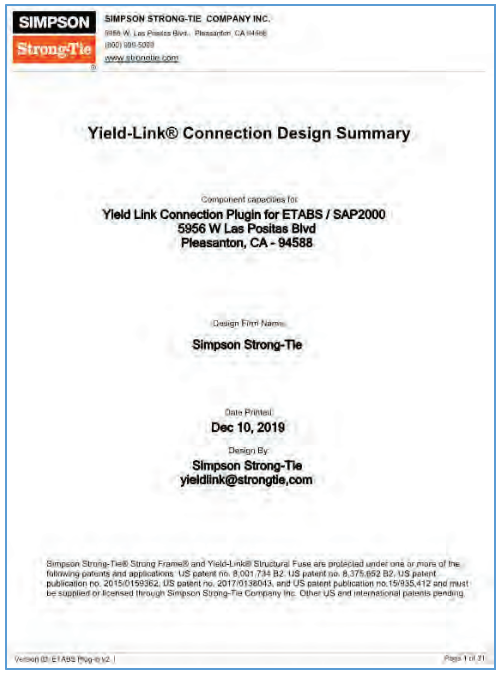
4. ***Change Slab Depths*** feature provides the functionality to set or modify slab thicknesses in the elevation drawings generated using the DXF Elevation feature. The default slab thickness is set to 6.25 in. and the thickness from bottom of base plate to top of base is set to 18 in. User should note that these values are not used for analysis.



5. **Turn off Auto Select Design** option excludes set of design combinations created by ETABS®/SAP2000® used for Member Design as necessary Load Combinations are already created in the Create Load Combinations step.

2.3 Export Menu

1. ***PDF Summary Report*** feature provides the functionality to export a summary of the seven screen tabs after connection and frame member design are completed.



SIMPSON STRONG-TIE COMPANY INC.
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Yield-Link® Connection Design Summary

Component capacities for:
Yield Link Connection Plugin for ETABS / SAP2000
5956 W Las Positas Blvd
Pleasanton, CA - 94588

Design Firm Name:
Simpson Strong-Tie

Date Printed:
Dec 10, 2019

Design By:
Simpson Strong-Tie
yieldlink@strongtie,com

Simpson Strong-Tie® Strong Frame® and Yield-Link® Structural Fuse are protected under one or more of the following patents and applications: US patent no. 8,001,734 B2, US patent no. 8,375,652 B2, US patent publication no. 2015/0159362, US patent no. 2017/0138043, and US patent publication no.15/935,412 and must be supplied or licensed through Simpson Strong-Tie Company Inc. Other US and international patents pending.

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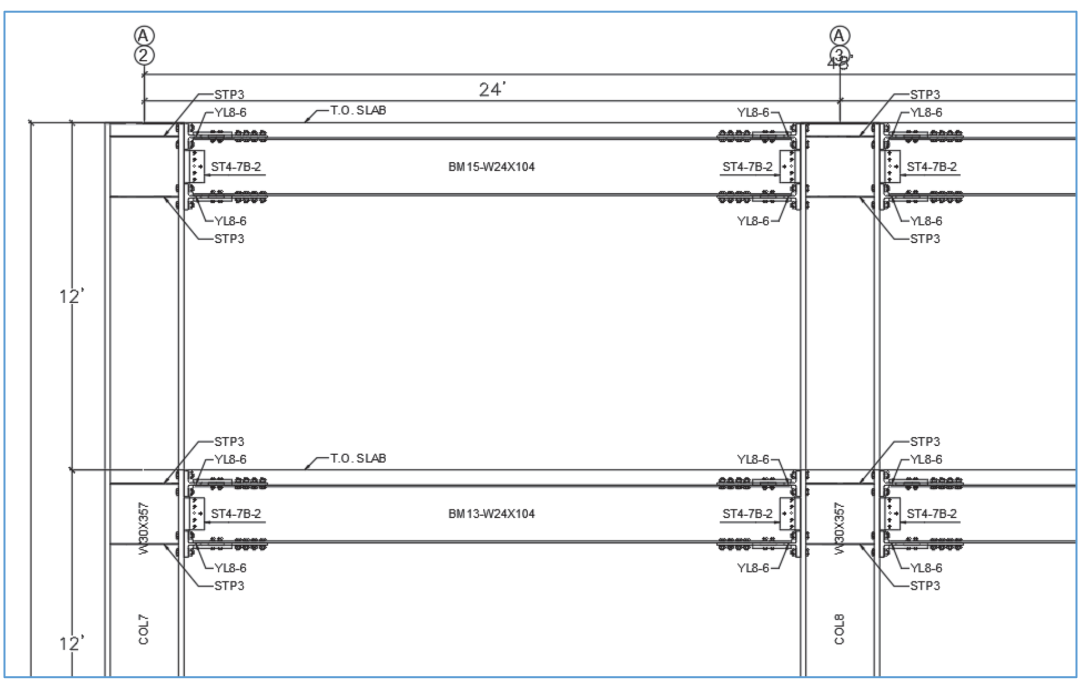
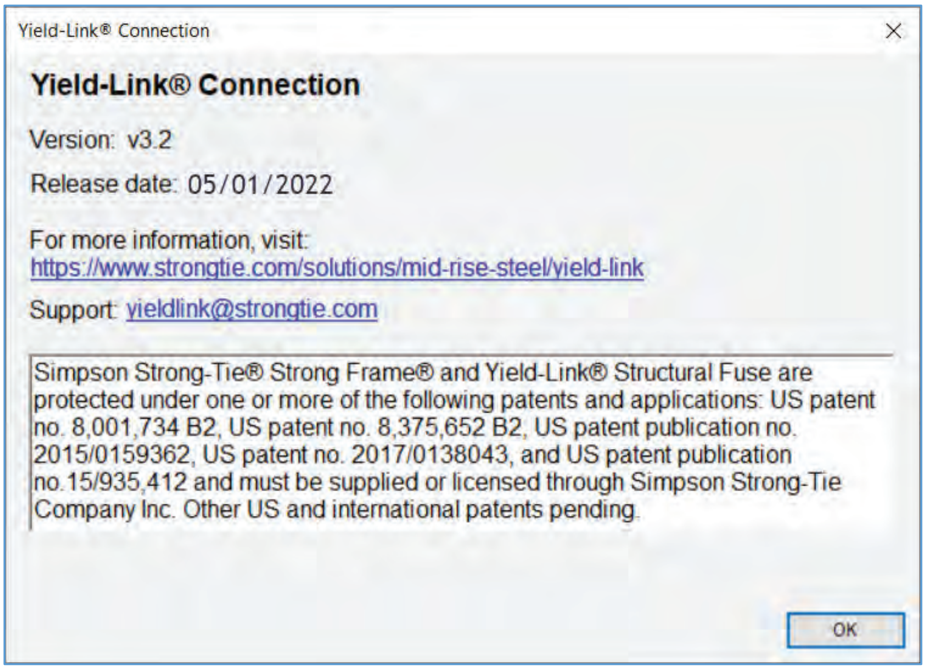
2. ***DXF Frame Elevations*** option creates DXF files for all Elevation IDs selected with all member component and geometry details.

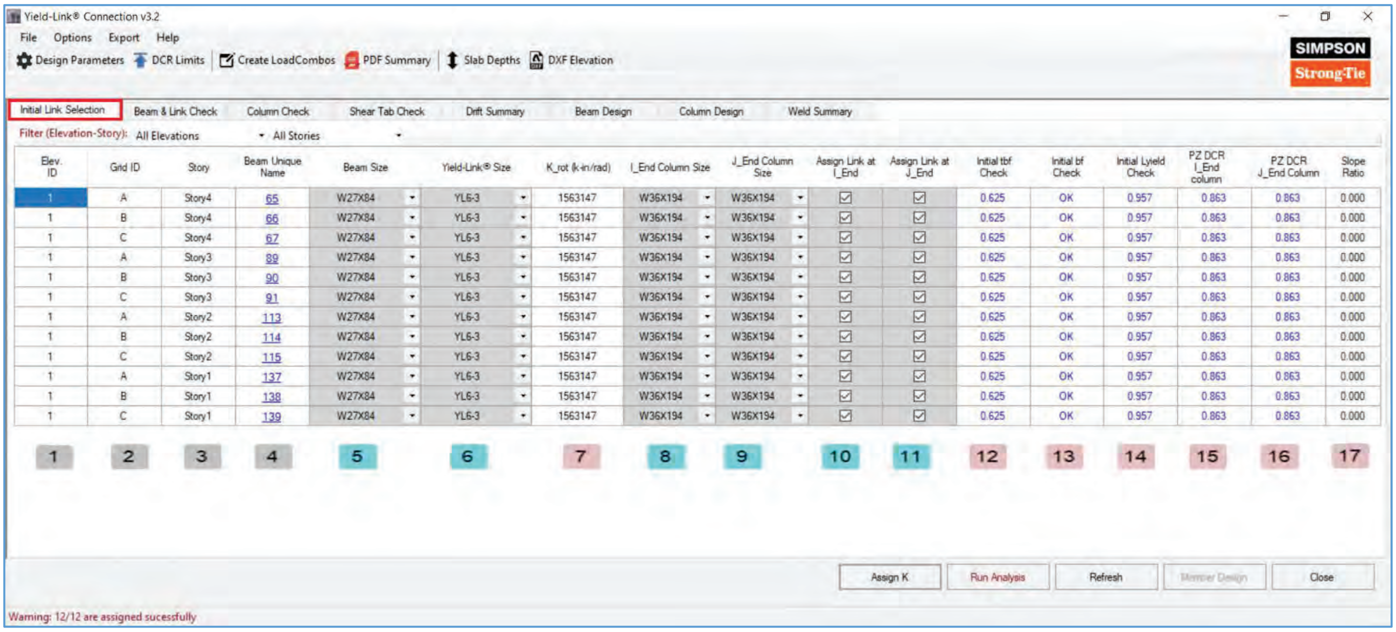


2.4 Help Menu

1. ***About*** opens up window with Plugin details like version ID, Release date, Support, Patent information.
2. ***User Guide*** option opens up this document.
3. ***AISC 358s2-16 (Chapter 12)*** provides quick access to Simpson Strong-Tie Strong Frame Moment Connection Chapter 12 in ANSI/AISC 358s2-20.
4. ***ESR-2802*** provides a link to the latest evaluation report by ICC-ES.



Plug-in User Interface



X = Value auto-populated from SAP2000® / ETABS® structural building model.

X = User input.

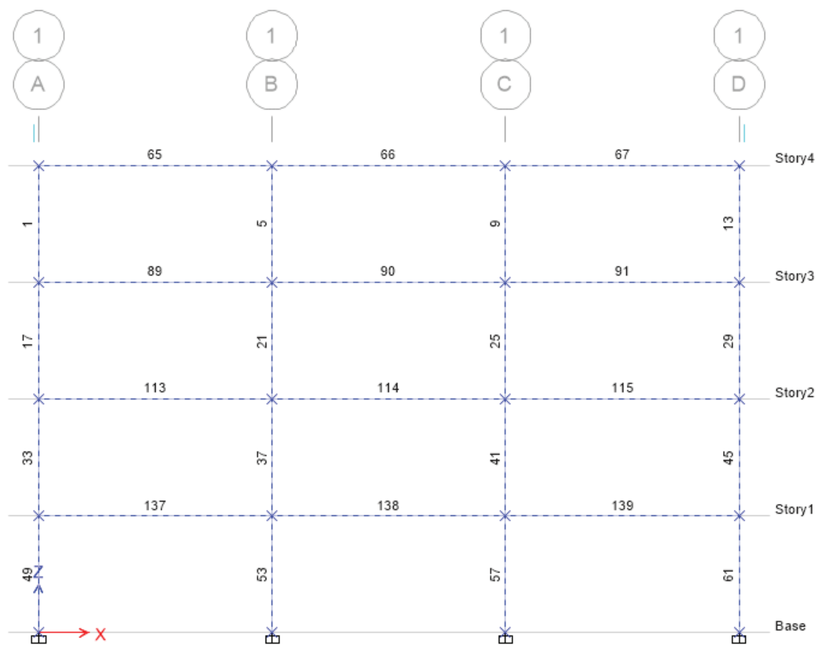
X = Value calculated by Plugin based on user input.

X = Value calculated by SAP2000® / ETABS® based on user input.

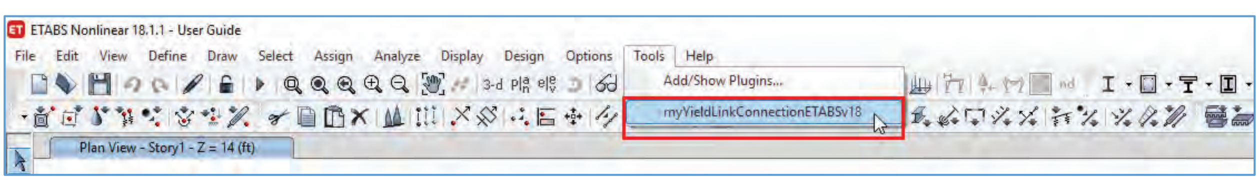
The Plugin interface consists of seven tabs allocated for different design calculations. The following sections explain the working of these design tabs.

3.0 Assigning Yield-Links® to Lateral Force Resisting Frames

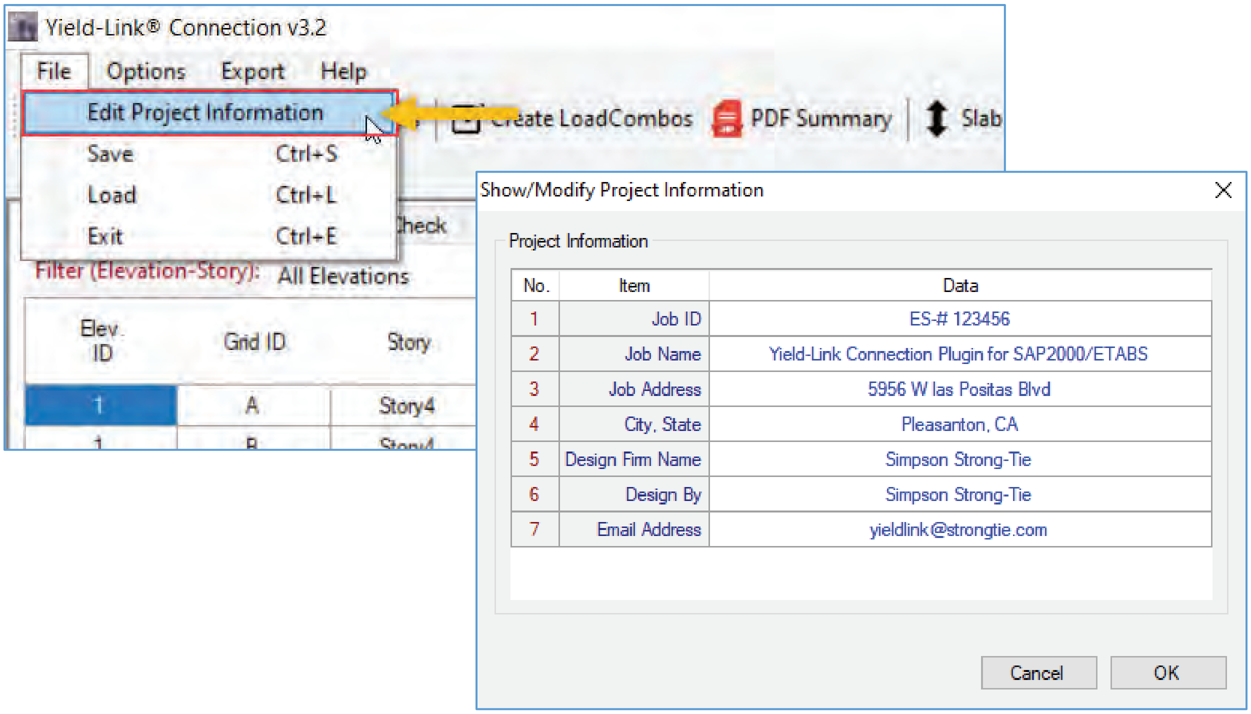
Step 1: Select all lateral force resisting frames where Yield-Link® moment connections are desired. Note that a minimum of 2 W-shape columns and 1 W-Shape connecting beam is required to be selected to constitute a frame. Note that any moment frames are grouped together, should have the group name start with either “YL*”, “GL*” or “MF*” for compatibility with this Plugin.



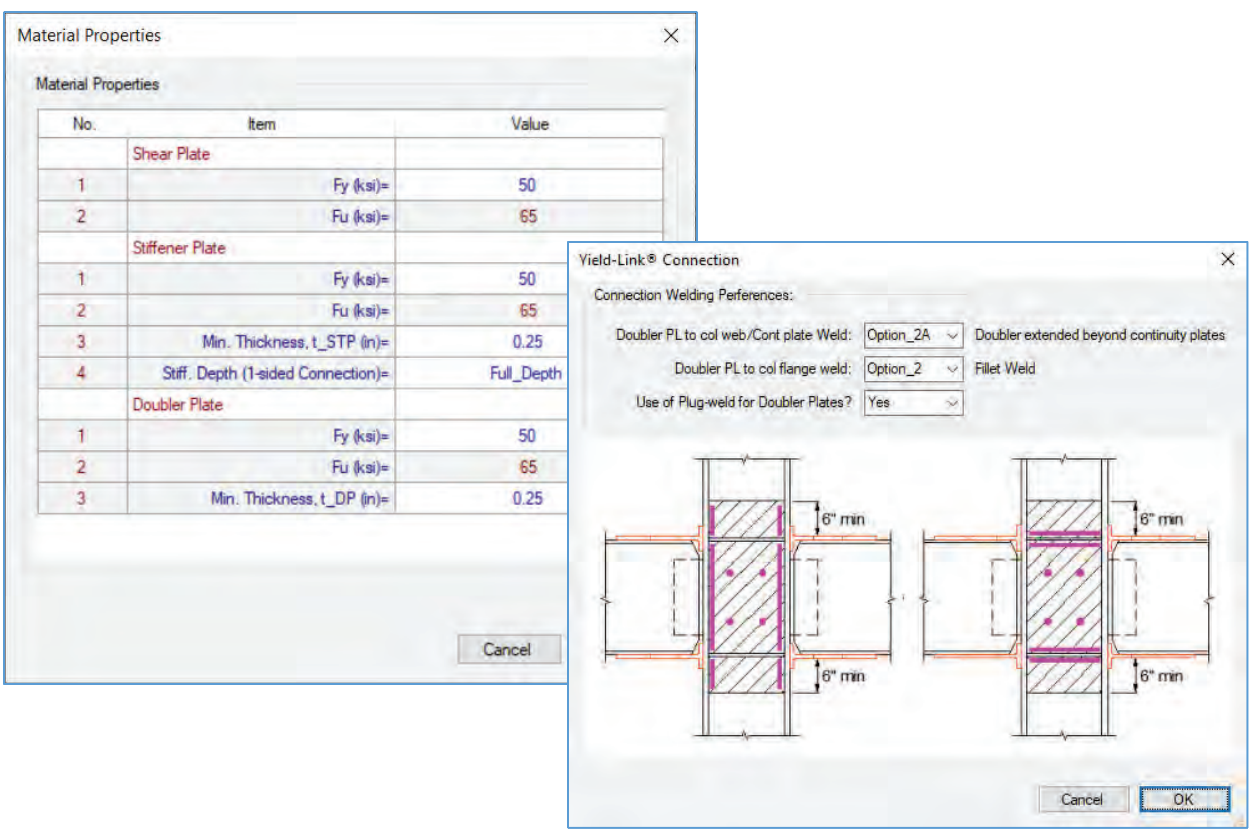
Step 2: Select “myYieldLinkConnection” plugin available under the Tools menu dropdown.



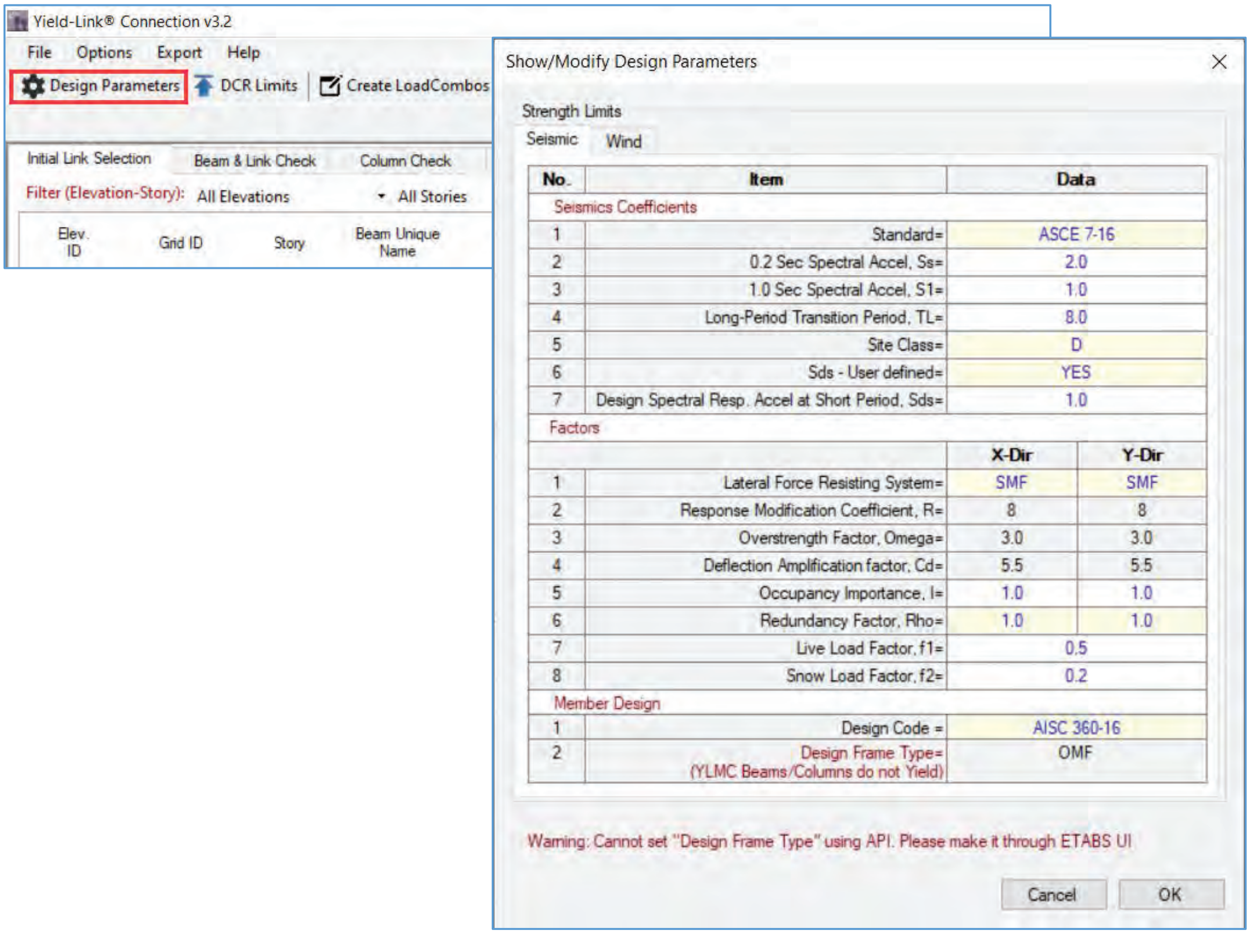
Step 3: Input project information from File Menu of Plugin interface.



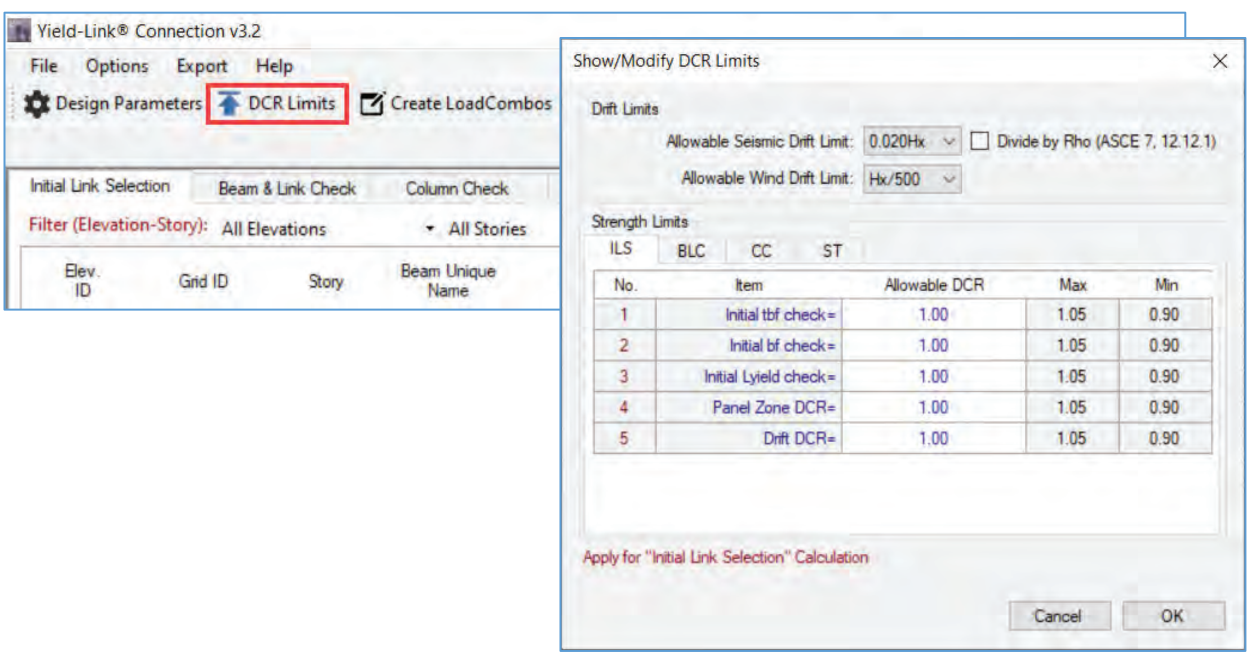
Step 4: Input material properties & welding preferences.



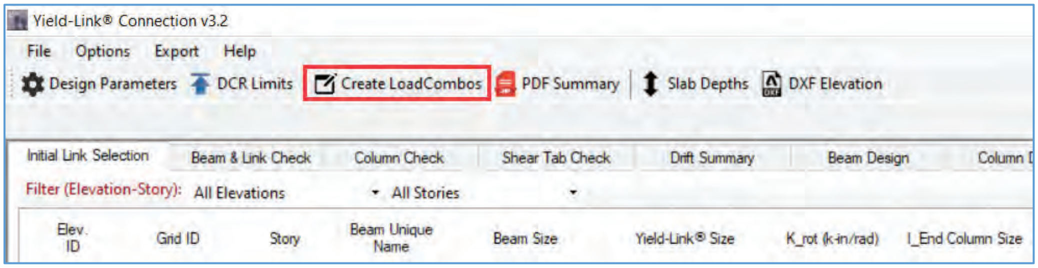
Step 5: Input Seismic Design Parameters



Step 6: Set Allowable DCR Limits



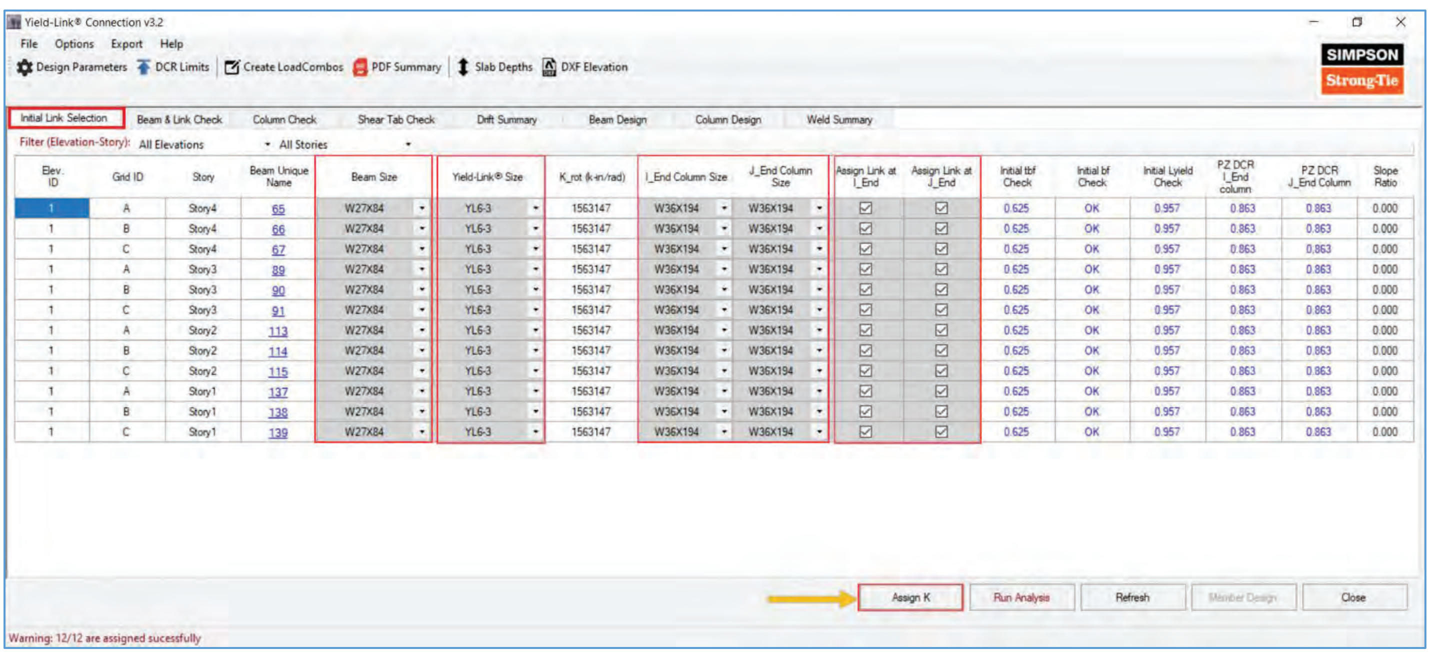
Step 7: Create Load Combinations

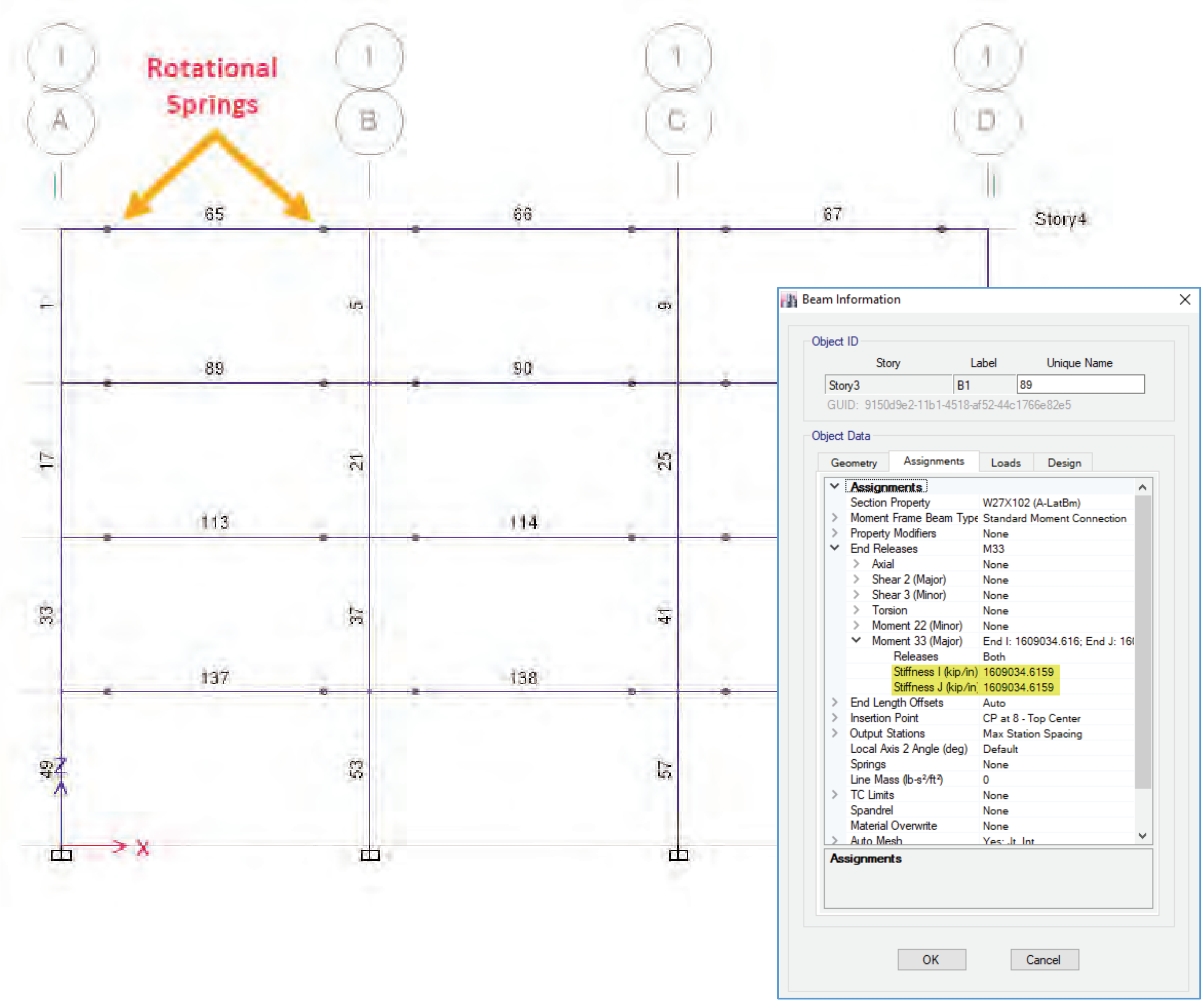


Currently, the ETABS® API limits the modification of lateral load parameters such as direction, eccentricity and site class. Hence, user is required to manually enter this information. See Appendix A for alternative approach. User should verify the building weight and base shear (Strength Check and Drift Check) calculated by ETABS®.

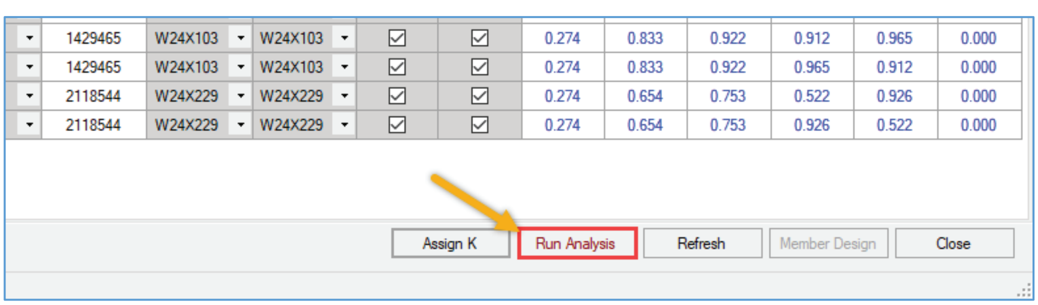
Step 8: From Initial Link Selection Tab, select Yield-Link® size for each connection and check boxes where moment connections are desired. Unchecked box will assign a pinned connection. User can interactively select beam and column sections from Plugin interface. (User Note: Yield-Link® sizes are selected such that all initial checks as explained in the next section are satisfied.)

Step 9: Click on "Assign K" button on the bottom to assign Yield-Links® to all selected frames where moment connections are desired.



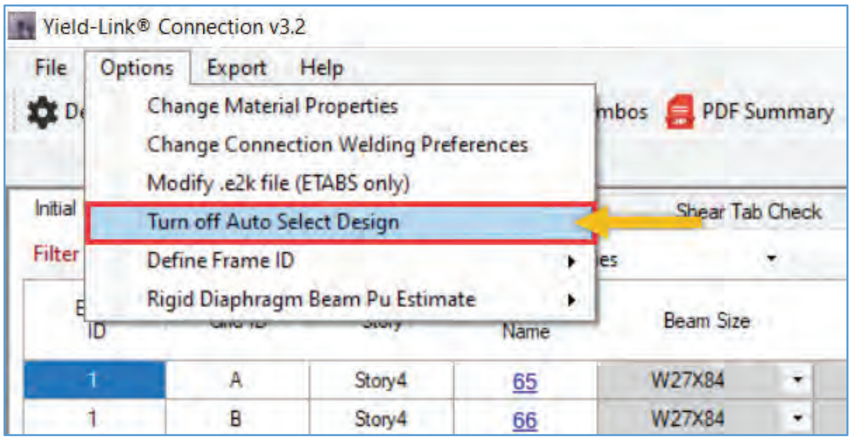


Step 10: Once Yield-Links® are assigned to the frames, click on "Run Analysis" button to analyze the structure. (User Note: Make sure all external loads are applied to the model before clicking the Run Analysis option).

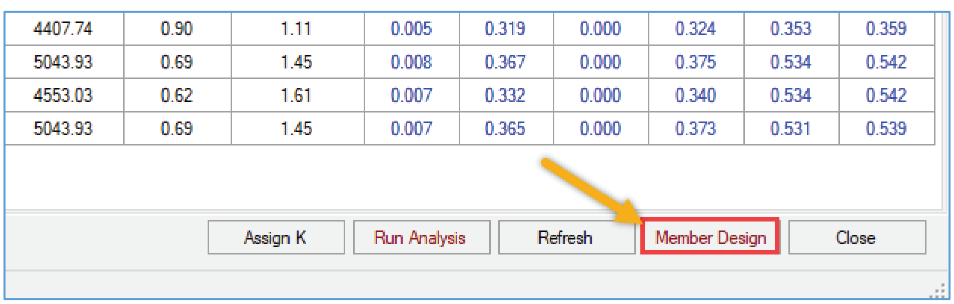


Step 11: Check that all strength and drift requirements are satisfied. If not, go to Initial Link Selection tab and change necessary input. Reiterate using Assign K and Run Analysis again. Once all strength and drift DCRs are satisfied, proceed to step 10.

Step 12: Deselect the design load combinations created by ETABS® or SAP2000® using the "*Turn Off Auto Select Design*" feature under the Options Menu.



Step 13: Go to Beam Design Tab and click Member Design option at the bottom. Selected Beams and Columns will be designed per code requirements.

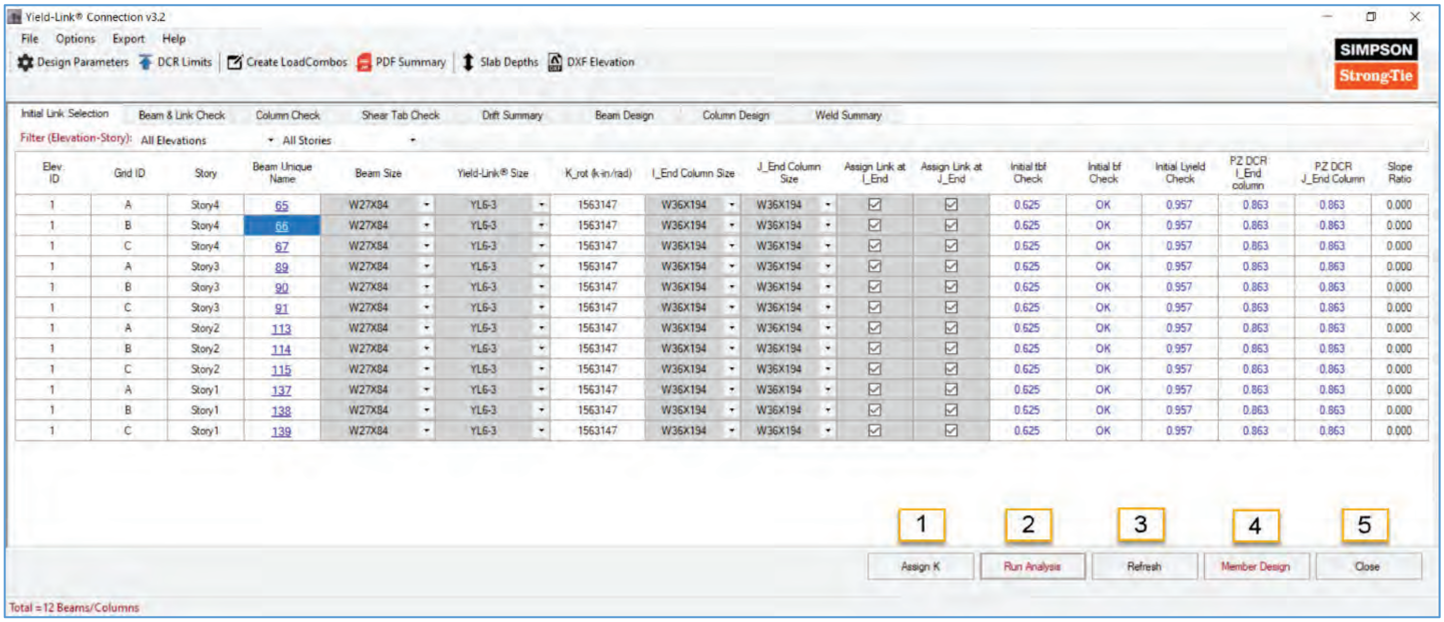


Step 14: Export a detailed report PDF for any member by right clicking on the unique ID and selecting detailed report.

Step 15: Export a PDF summary of all seven design tabs.

Step 16: Set/modify slab thicknesses and export DXF Elevations of the selected frames where Yield-Link® moment connections are designed.

Plug-in User Interface



Elev ID	Grid ID	Story	Beam Unique Name	Beam Size	Yield-Link® Size	K_rot (k-in/rad)	I_End Column Size	J_End Column Size	Assign Link at I_End	Assign Link at J_End	Initial tbf Check	Initial bf Check	Initial Lyield Check	PZ DCR I_End column	PZ DCR J_End Column	Slope Ratio
1	A	Story4	65	W27X84	YL6-3	1563147	W36X194	W36X194	☑	☑	0.625	OK	0.957	0.863	0.863	0.000
1	B	Story4	66	W27X84	YL6-3	1563147	W36X194	W36X194	☑	☑	0.625	OK	0.957	0.863	0.863	0.000
1	C	Story4	67	W27X84	YL6-3	1563147	W36X194	W36X194	☑	☑	0.625	OK	0.957	0.863	0.863	0.000
1	A	Story3	89	W27X84	YL6-3	1563147	W36X194	W36X194	☑	☑	0.625	OK	0.957	0.863	0.863	0.000
1	B	Story3	90	W27X84	YL6-3	1563147	W36X194	W36X194	☑	☑	0.625	OK	0.957	0.863	0.863	0.000
1	C	Story3	91	W27X84	YL6-3	1563147	W36X194	W36X194	☑	☑	0.625	OK	0.957	0.863	0.863	0.000
1	A	Story2	113	W27X84	YL6-3	1563147	W36X194	W36X194	☑	☑	0.625	OK	0.957	0.863	0.863	0.000
1	B	Story2	114	W27X84	YL6-3	1563147	W36X194	W36X194	☑	☑	0.625	OK	0.957	0.863	0.863	0.000
1	C	Story2	115	W27X84	YL6-3	1563147	W36X194	W36X194	☑	☑	0.625	OK	0.957	0.863	0.863	0.000
1	A	Story1	137	W27X84	YL6-3	1563147	W36X194	W36X194	☑	☑	0.625	OK	0.957	0.863	0.863	0.000
1	B	Story1	138	W27X84	YL6-3	1563147	W36X194	W36X194	☑	☑	0.625	OK	0.957	0.863	0.863	0.000
1	C	Story1	139	W27X84	YL6-3	1563147	W36X194	W36X194	☑	☑	0.625	OK	0.957	0.863	0.863	0.000

X = Action buttons as explained below.

1. **Assign K** feature will assign a rotational spring value (calculated from Yield-Link® and beam properties) to beams where Yield-Link® moment connections are desired.
2. **Run Analysis** will analyze the model using user assigned external loads, seismic design parameters as selected in step 5 and load combinations created in step 7.
3. **Refresh** button will reset design values and input to reflect any changes in the 3D model.
4. **Member Design** will design the Beams and Columns per Load Combinations created in step 7. See User Note 6.
5. **Close** button will close the Plugin interface and exit the application.

User Note [6]:

Following design preferences are required to be defined before design of members.

I. Framing Type = OMF

Since plastic hinging occurs in the yield-links and not in the beam, the beam can be analyzed as OMF.

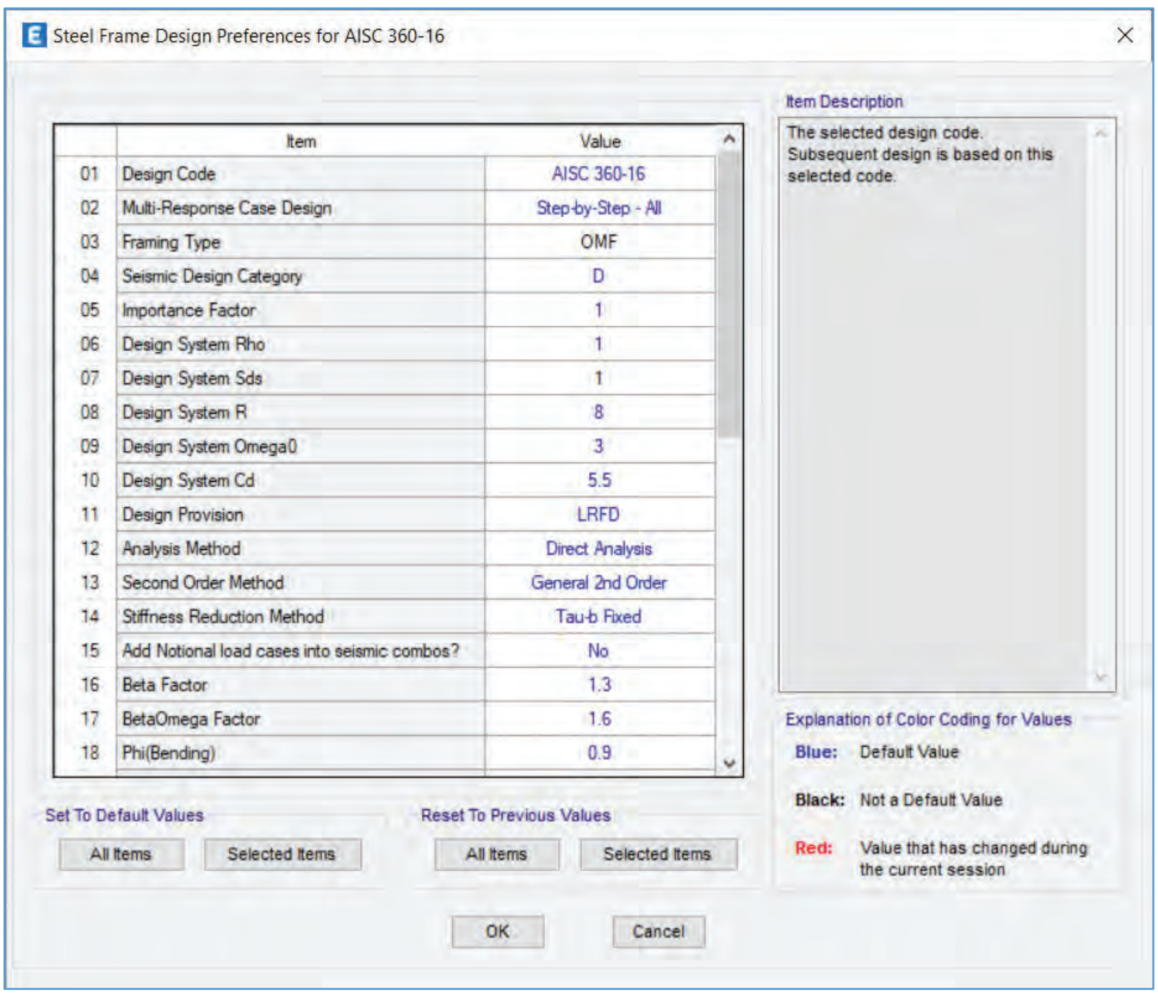
II. Analysis Method = Direct Analysis

Direct Analysis Method described in AISC 360, Chapter C is the default analysis option used to check/design steel members as it offers the following advantages:

a. Accurately model frame behavior.
b. Factor loads (LRFD or ASD)
c. Second-order effects. (including P-Δ and P-δ effects)
d. Geometric imperfections. (Use of Notional Loads)
e. Stiffness reduction due to inelasticity (including effect of residual stresses)
f. Effective column length, K = 1 for member design.

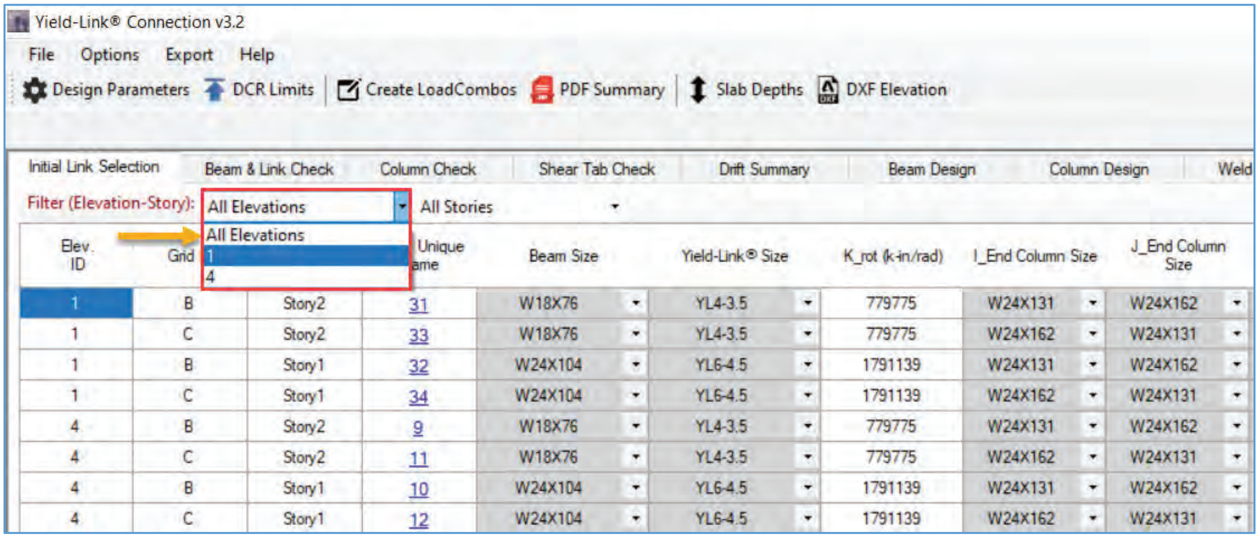
III. Stiffness Reduction Method = Tau-b Variable

Refer Chapter C2.3, AISC 360 for more information regarding adjustment to stiffness. User may use "Tau-b Fixed" option by changing the coefficient of Notional Load Patterns NLX and NLY from 0.002 to 0.003. (Chapter C2.3 (c), AISC 360-16)

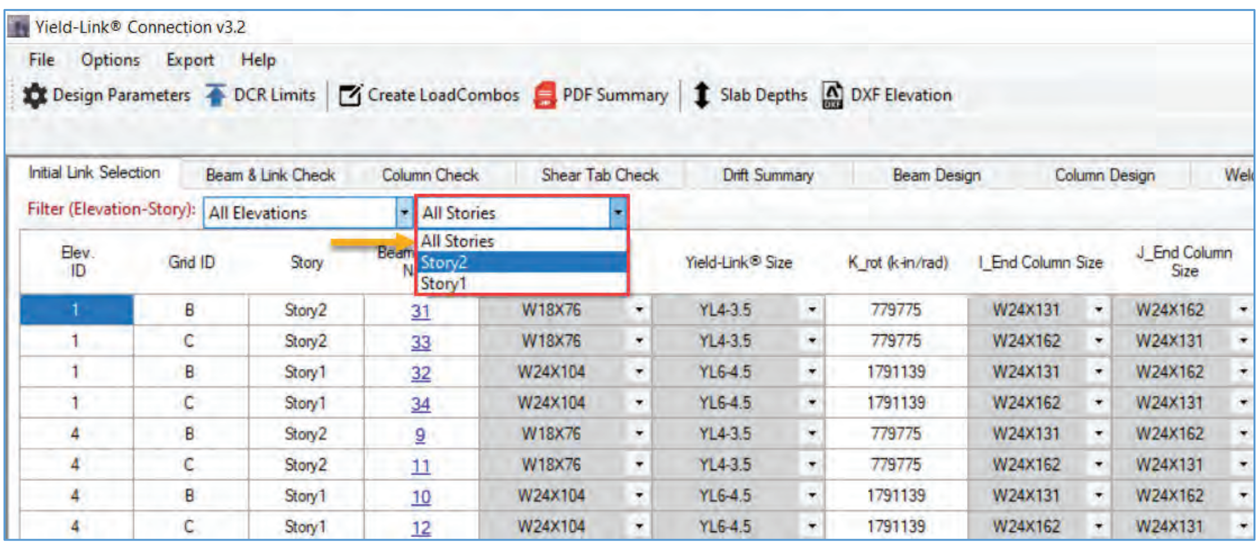


4.0 Additional Features

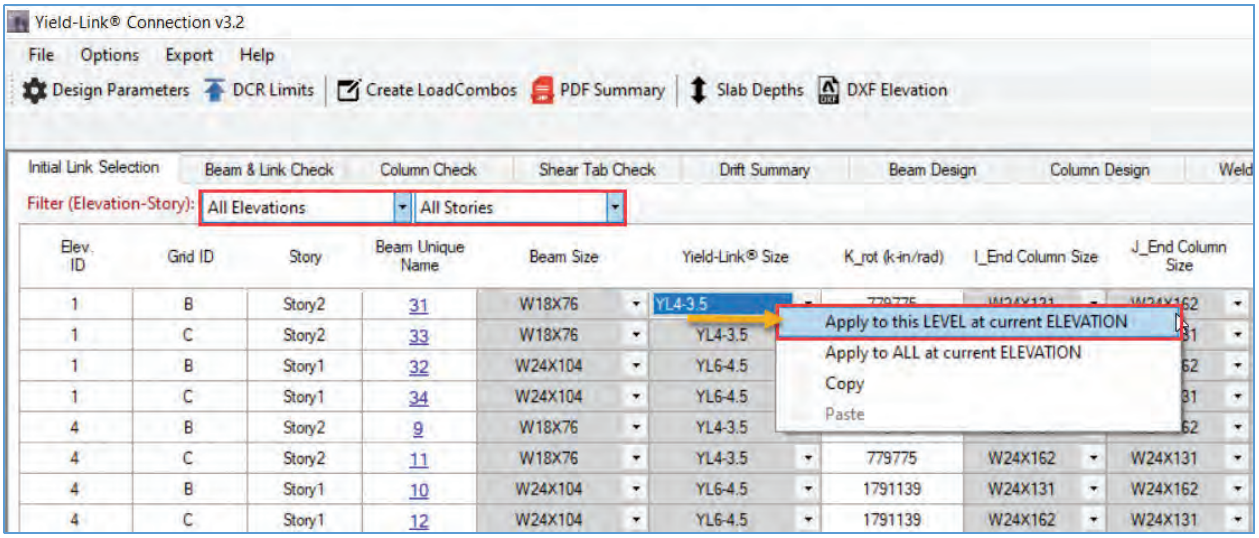
1. Filter using Elevation/Story ID:



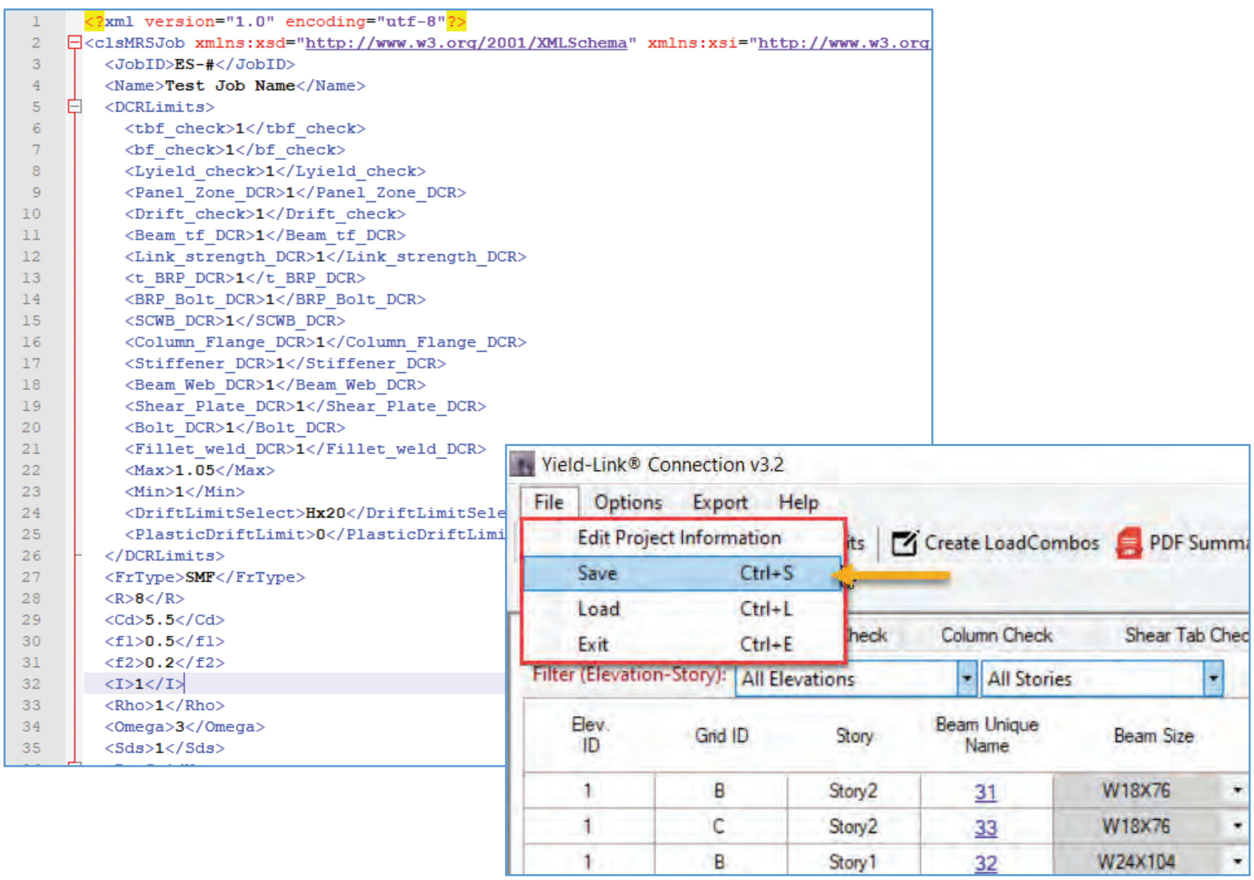
2. Right click on any Input Field to see additional options as shown:



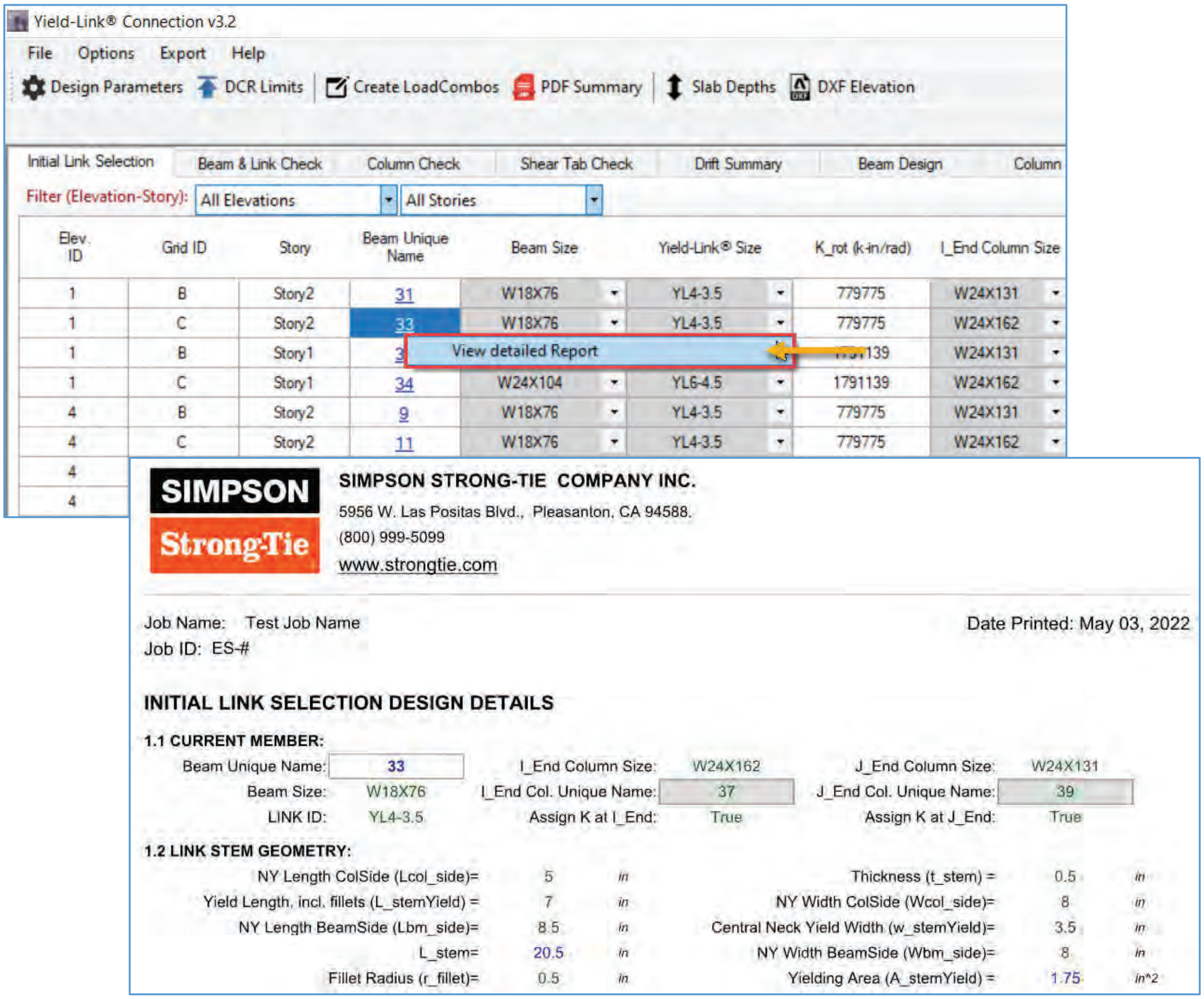
"APPLY value to this LEVEL at current ELEVATION" or "APPLY value to ALL at current ELEVATION"



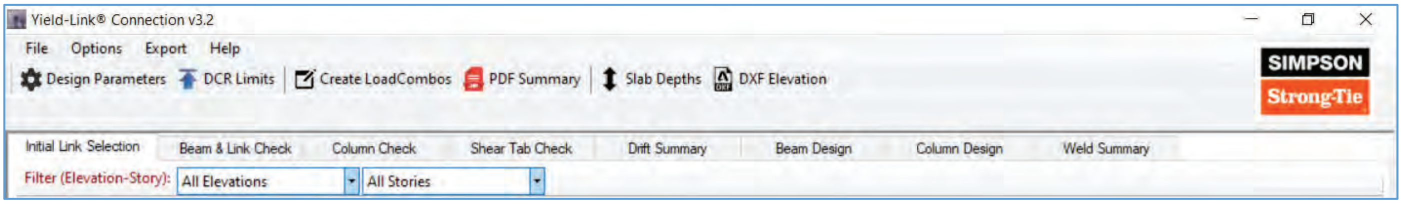
3. Save & Load Input Files in XML format:



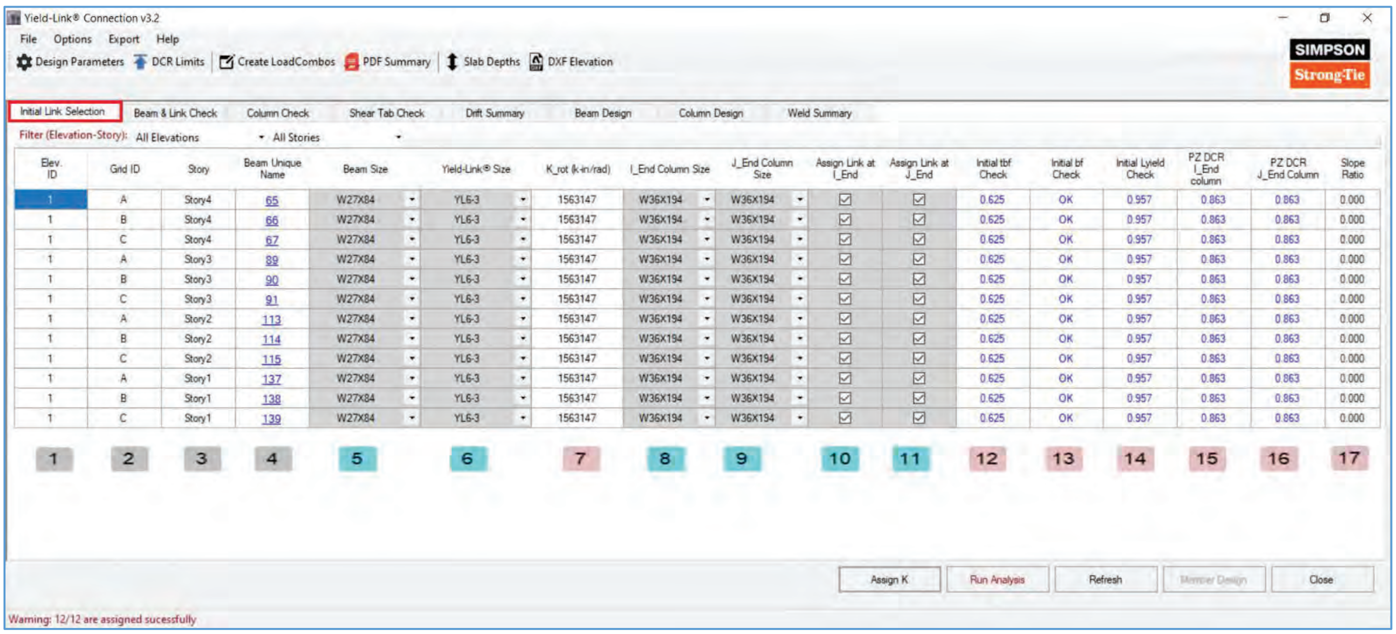
4. View Detailed Calculation Report PDF by Right Clicking any Unique Member ID:



5.0 Yield-Link® Plugin Output Tabs



5.1 Initial Link Selection



Size your moment frame beams/columns similar to your other connection design (i.e. RBS etc.)

1. **Elevation ID:** Structural modeling in SAP2000® & ETABS® is based on a grid system. The elevation ID gives information regarding the elevation plane on which the selected frame is located.
2. **Grid ID:** This feature shows the grid located on the left node of the selected beam.
3. **Story:** This column provides details regarding the story on which the selected beam is located.
4. **Beam Unique Name:** SAP2000® & ETABS® assigns a unique ID for every structural element.
5. **Beam Size:** Displays the structural shape assign to the corresponding beam element.

6. **Yield-Link® Size:** User Input based on the frame geometry, load demand and member sizes. See Table 2 below for current Yield-Link® offerings.

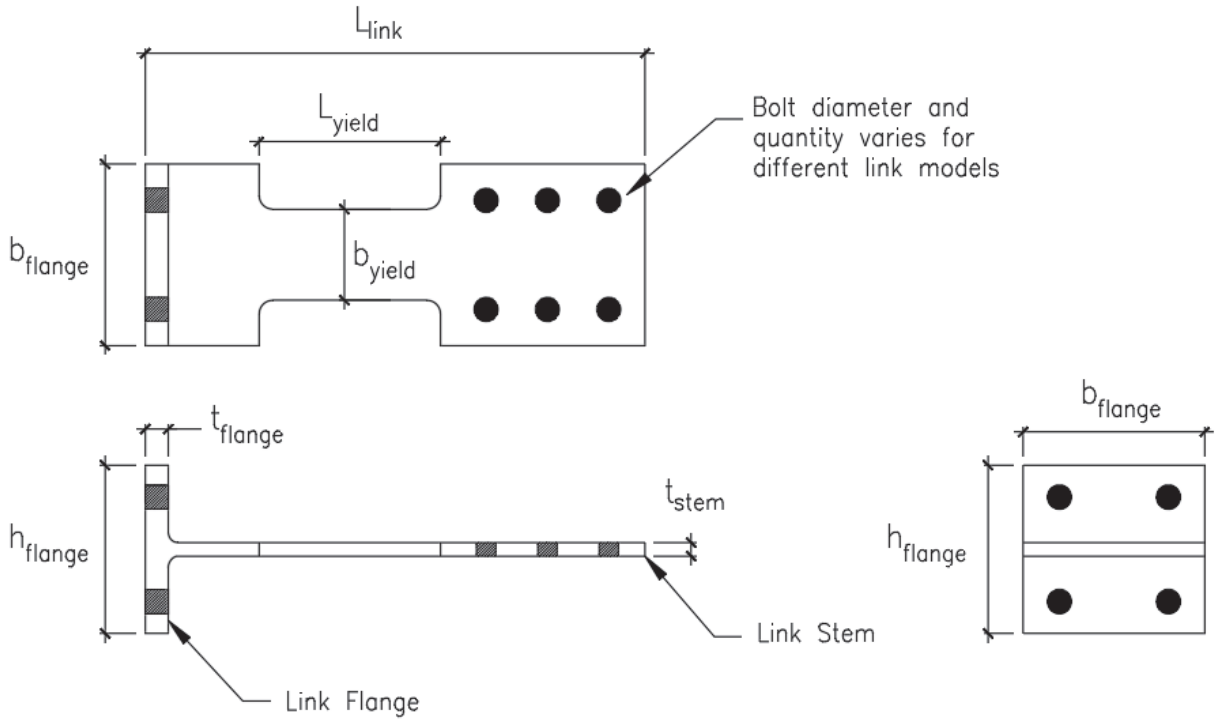
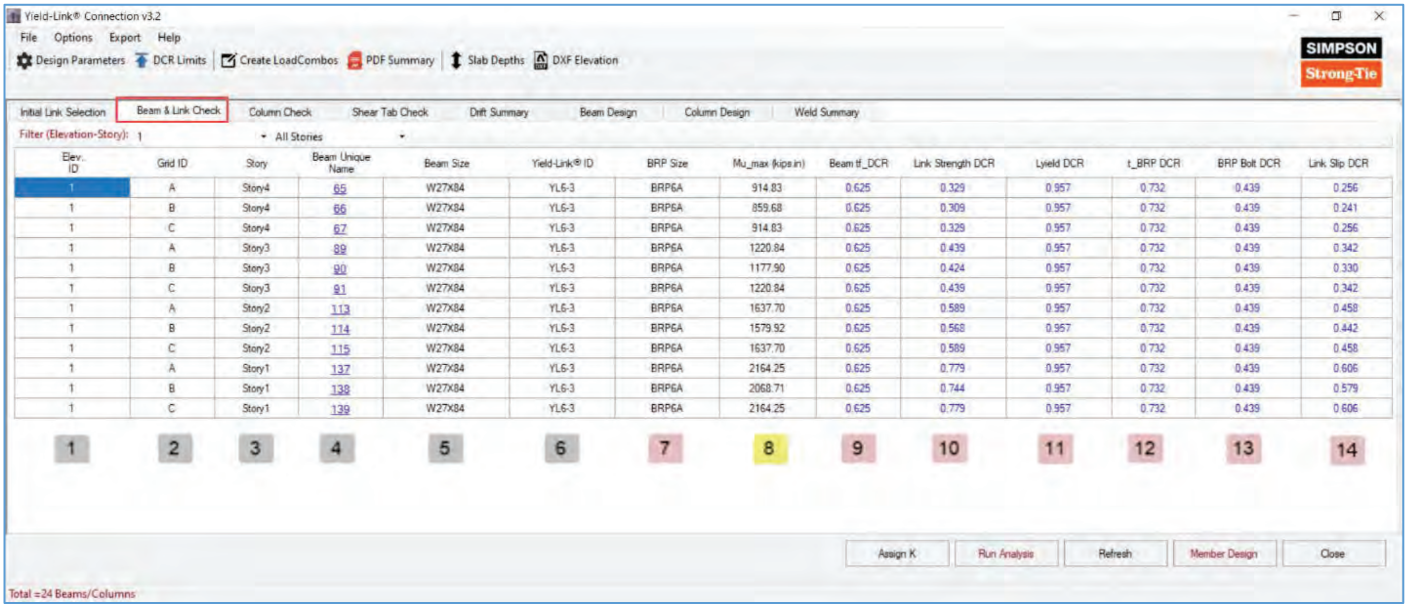


Table 2: Yield-Link Geometry & Design Information

Yield-Link® ID[1]	Yield Link® Geometry							Design Information				BRP ID	Spacer Plate ID
	t_{stem}	t_{flange}	b_{yield}	L_{yield}	L_{link}	b_{flange}	h_{flange}	P_{y_link} kips	K_{eff} kip/in.	Min. Beam Depth	Max. Beam Depth		
YL4-2	1/2"	7/8"	2"	7"	1'-6 5/8"	6-1/4"	5-3/4"	50.0	2970	W12	W18	BRP4C	SP4C
YL4-2.5			2.5"					62.5	3468				
YL4-3			3"			6-1/2"		75.0	3960				
YL4-2.25			2.25"			7"		56.25	3337			BRP4A	SP4
YL4-2.875			2.875"					71.88	3953				
YL4-3.5			3.5"		1'-9 3/8"	8"		87.50	4460			BRP4B	
YL4-3.75			3.75"					93.75	4651				
YL4-4			4"					100.00	4831				
YL4-2.25-10		13/16"	2.25"	10"	1'-9 9/16"	7"		56.25	2554	W16	W27	BRP4A-10	SP4-10
YL4-2.875-10			2.875"					71.88	3077				
YL4-3.5-10			3.5"		2'-5/16"	8"		87.50	3529			BRP4B-10	
YL4-3.75-10			3.75"					93.75	3701				
YL4-4-10			4"					100.00	3865				
YL6-2.5	3/4"	1-1/4"	2.5"	10"	2'-3 1/2"	6-1/2"	9-1/4"	93.75	3426	W16	W27	BRP6D	SP6D
YL6-3			3"			8"		112.50	4149			BRP6A	SP6
YL6-3.5			3.5"					131.25	4564				
YL6-4			4"					150.00	4933				
YL6-4.5			4.5"			10"		168.75	5801			BRP6B	
YL6-5			5"					187.50	6167				
YL6-5.5			5.5"			12"		206.25	7037			BRP6C	
YL6-6			6"					225.00	7400				
YL6-3-13			3"	13"	2'-6 1/2"	8"		112.50	3484	W24	W36	BRP6A-13	SP6-13
YL6-3.5-13			3.5"					131.25	3868				
YL6-4-13			4"					150.00	4216				
YL6-4.5-13			4.5"			10"		168.75	4925			BRP6B-13	
YL6-5-13			5"					187.50	5270				
YL6-5.5-13			5.5"			12"		206.25	5981			BRP6C-13	
YL6-6-13			6"					225.00	6324				
YL8-4	1"	1-13/16"	4"	13"	2'-7 1/16"	9"	10-3/4"	200.00	6034	W24	W36	BRP8A	SP8
YL8-4.5			4.5"					225.00	6524				
YL8-5			5"		2'-10 7/16"	12"		250.00	7698			BRP8B	
YL8-5.5			5.5"					275.00	8213				
YL8-6			6"					300.00	8698				
YL8-4-15			4"	15"	2'-9 1/16"	9"		200.00	5465	W24	W36	BRP8A-15	SP8-15
YL8-4.5-15			4.5"					225.00	5931				
YL8-5-15			5"		3'-7/16"	12"		250.00	6959			BRP8B-15	
YL8-5.5-15			5.5"					275.00	7446				
YL8-6-15			6"					300.00	7908				

7. **K_rot:** Rotational Stiffness (kip-in./rad) associated with the moment connection calculated per Step 11.1, Chapter 12 of ANSI/AISC 358s2-20.
8. **I_End Column Size:** Shows the steel structural shape assigned to the column on the left side of the selected beam.
9. **J_End Column Size:** Shows the steel structural shape assigned to the column on the right side of the selected beam.
10. **Assign Link at I_End:** User input corresponding to the moment connection desired at left end of beam. If box is unchecked, a pinned connection with K_rot=0 (kip-in./rad) is assigned to I_End of the beam.
11. **Assign Link at J_End:** User input corresponding to the moment connection desired at right end of beam. If box is unchecked, a pinned connection with K_rot=0 (kip-in./rad) is assigned to J_End of the beam.
12. **Beam flange thickness, tbf check:** Refer ANSI/AISC 358s2-20 Chapter 12.3.1 (3)
13. **Width of Yield-Link® reduced region, bf check:** Refer Annex A, EQ. A-3, ICC-ES ESR-2802 or Chapter 12, EQ. 12.9-3, ANSI/AISC 358s2-20.
14. **Minimum yielding length, L_{yield} check:** Refer Chapter 12, EQ. 12.9-4, ANSI/AISC 358s2-20.
15. **PZ DCR I_End column:** Panel zone check for the column at left end of beam due to demand load from axial force in Yield-Link® calculated per J10-9 or J10-10, AISC Steel Construction Manual, 14th Edition.
16. **PZ DCR J_End column:** Panel zone check for the column at right end of beam due to demand load from axial force in Yield-Link® calculated per J10-9 or J10-10, AISC Steel Construction Manual, 14th Edition.
17. **Slope Ratio:** Plugin will auto-calculate a beam slope ratio based on difference in vertical distance per foot if the beam is modeled to have a slope. For example, if the beam is modeled to have a slope of 0.25” per foot, slope ratio is calculated as 0.25/12 = 0.021.

5.2 Beam & Link Check

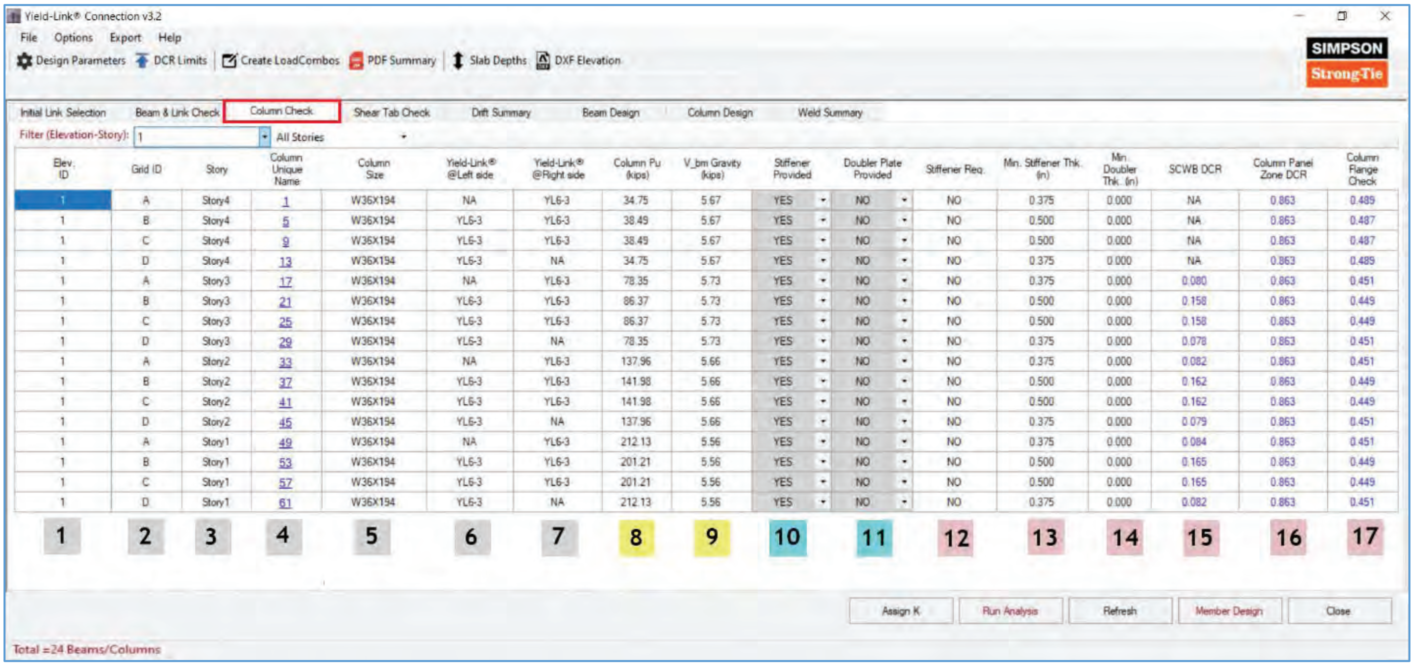


Elev. ID	Grid ID	Story	Beam Unique Name	Beam Size	Yield-Link® ID	BRP Size	Mu_max (kips.in)	Beam tf_DCR	Link Strength DCR	Lyield DCR	t_BRP DCR	BRP Bolt DCR	Link Slip DCR
1	A	Story4	65	W27X84	YL6-3	BRP6A	914.83	0.625	0.329	0.957	0.732	0.439	0.256
1	B	Story4	66	W27X84	YL6-3	BRP6A	859.68	0.625	0.309	0.957	0.732	0.439	0.241
1	C	Story4	67	W27X84	YL6-3	BRP6A	914.83	0.625	0.329	0.957	0.732	0.439	0.256
1	A	Story3	89	W27X84	YL6-3	BRP6A	1220.84	0.625	0.439	0.957	0.732	0.439	0.342
1	B	Story3	90	W27X84	YL6-3	BRP6A	1177.90	0.625	0.424	0.957	0.732	0.439	0.330
1	C	Story3	91	W27X84	YL6-3	BRP6A	1220.84	0.625	0.439	0.957	0.732	0.439	0.342
1	A	Story2	113	W27X84	YL6-3	BRP6A	1637.70	0.625	0.589	0.957	0.732	0.439	0.458
1	B	Story2	114	W27X84	YL6-3	BRP6A	1579.92	0.625	0.568	0.957	0.732	0.439	0.442
1	C	Story2	115	W27X84	YL6-3	BRP6A	1637.70	0.625	0.589	0.957	0.732	0.439	0.458
1	A	Story1	137	W27X84	YL6-3	BRP6A	2164.25	0.625	0.779	0.957	0.732	0.439	0.606
1	B	Story1	138	W27X84	YL6-3	BRP6A	2068.71	0.625	0.744	0.957	0.732	0.439	0.579
1	C	Story1	139	W27X84	YL6-3	BRP6A	2164.25	0.625	0.779	0.957	0.732	0.439	0.606
1	2	3	4	5	6	7	8	9	10	11	12	13	14

Columns 1-6 are explained in Initial Link Selection Tab details.

7. **BRP Size:** A buckling restraint plate is bolted on top of the spacers and yield-link as a part of the moment connection. BRP Size depends on the Yield-Link® geometry as shown in Table 1.
8. **Mu_max:** Maximum moment calculated by ETABS® at beam ends using SST_LC 1-7, SST_LC 9-22 & SST_LC 25-28 for link strength design check.
9. **Beam flange thickness, tbf DCR:** Refer calculation of thickness of the Yield-Link flange, t_{flange}, required to prevent prying action per EQ. 12.9-10, Chapter 12 of ANSI/AISC 358s2-20.
10. **Link Strength DCR:** This value is determined by taking a ratio of axial demand load on the link, P_{u_link} and the link axial capacity at yield, P_{y_link} multiplied by a safety factor, ϕ of 0.9 (User Note: See Table 1 for link capacity, P_{y_link})
11. **Minimum yielding length, L_{yield} check:** Refer Chapter 12, EQ. 12.9-4, ANSI/AISC 358s2-20.
12. **BRP thickness, t_BRP DCR:** Determine the minimum thickness of the Buckling Restraint Plate (BRP) to prevent link stem buckling during compression of the link stem per EQ. 12.9-13, Chapter 12 of ANSI/AISC 358s2-20 or EQ A-20, ICC-ES ESR-2802.
13. **BRP Bolt Size, BRP_Bolt DCR:** Determine the BRP bolt size per EQ A-28, ESR 2802.
14. **Link Slip DCR:** Determine Yield-Link stem-to-beam flange connection slip, per AISC 360-16, EQ J3.4 (μ = 0.3)

5.3 Column Check



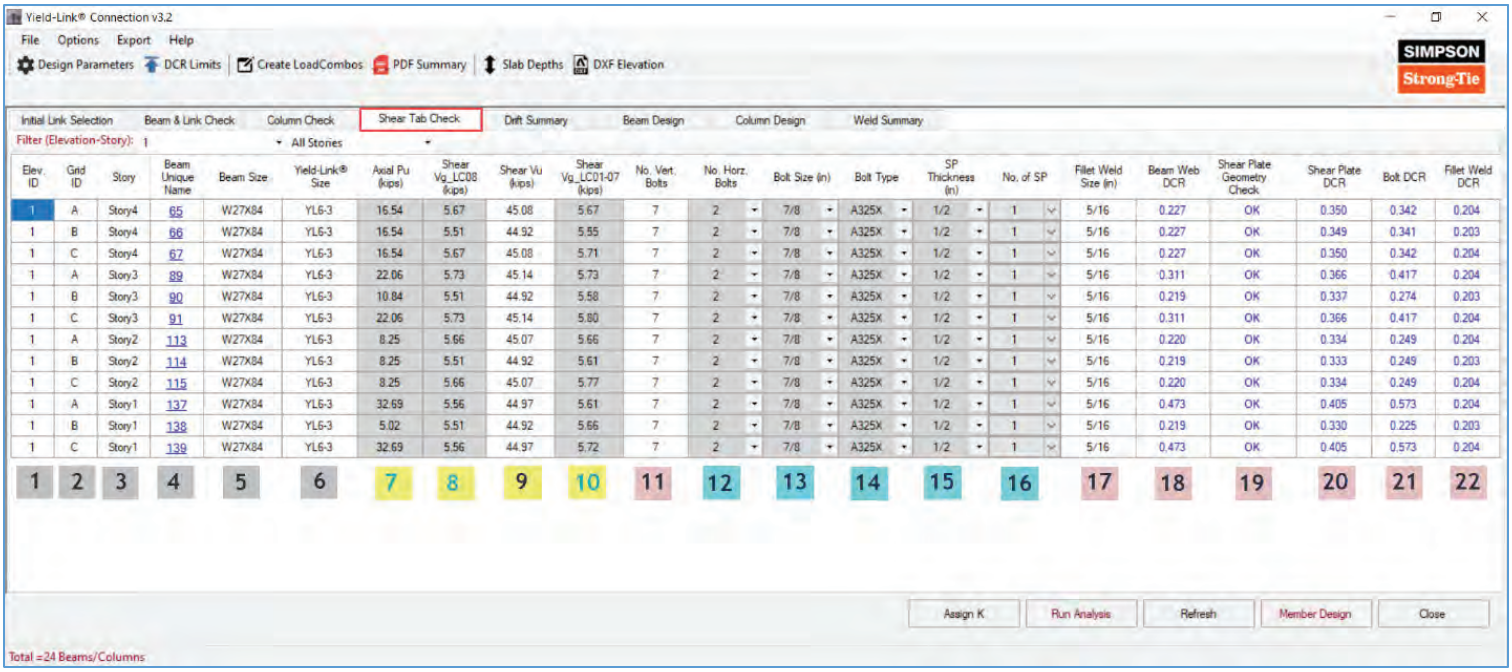
Elev. ID	Grid ID	Story	Column Unique Name	Column Size	Yield-Link® @Left side	Yield-Link® @Right side	Column Pu (kips)	V_bm Gravity (kips)	Stiffener Provided	Doubler Plate Provided	Stiffener Req.	Min. Stiffener Thk. (in)	Min Doubler Thk. (in)	SCWB DCR	Column Panel Zone DCR	Column Flange Check
1	A	Story4	1	W36X194	NA	YL6-3	34.75	5.67	YES	NO	NO	0.375	0.000	NA	0.863	0.489
1	B	Story4	5	W36X194	YL6-3	YL6-3	38.49	5.67	YES	NO	NO	0.500	0.000	NA	0.863	0.487
1	C	Story4	9	W36X194	YL6-3	YL6-3	38.49	5.67	YES	NO	NO	0.500	0.000	NA	0.863	0.487
1	D	Story4	13	W36X194	YL6-3	NA	34.75	5.67	YES	NO	NO	0.375	0.000	NA	0.863	0.489
1	A	Story3	17	W36X194	NA	YL6-3	78.35	5.73	YES	NO	NO	0.375	0.000	0.080	0.863	0.451
1	B	Story3	21	W36X194	YL6-3	YL6-3	86.37	5.73	YES	NO	NO	0.500	0.000	0.158	0.863	0.449
1	C	Story3	25	W36X194	YL6-3	YL6-3	86.37	5.73	YES	NO	NO	0.500	0.000	0.158	0.863	0.449
1	D	Story3	29	W36X194	YL6-3	NA	78.35	5.73	YES	NO	NO	0.375	0.000	0.078	0.863	0.451
1	A	Story2	33	W36X194	NA	YL6-3	137.96	5.66	YES	NO	NO	0.375	0.000	0.082	0.863	0.451
1	B	Story2	37	W36X194	YL6-3	YL6-3	141.98	5.66	YES	NO	NO	0.500	0.000	0.162	0.863	0.449
1	C	Story2	41	W36X194	YL6-3	YL6-3	141.98	5.66	YES	NO	NO	0.500	0.000	0.162	0.863	0.449
1	D	Story2	45	W36X194	YL6-3	NA	137.96	5.66	YES	NO	NO	0.375	0.000	0.079	0.863	0.451
1	A	Story1	49	W36X194	NA	YL6-3	212.13	5.56	YES	NO	NO	0.375	0.000	0.084	0.863	0.451
1	B	Story1	53	W36X194	YL6-3	YL6-3	201.21	5.56	YES	NO	NO	0.500	0.000	0.165	0.863	0.449
1	C	Story1	57	W36X194	YL6-3	YL6-3	201.21	5.56	YES	NO	NO	0.500	0.000	0.165	0.863	0.449
1	D	Story1	61	W36X194	YL6-3	NA	212.13	5.56	YES	NO	NO	0.375	0.000	0.082	0.863	0.451

Columns 1-5 are explained in Initial Link Selection Tab details.

6. **Yield-Link® @Left Side:** User Input based on the frame geometry, load demand and member sizes from Initial Link Selection tab.
7. **Yield-Link® @Right Side:** User Input based on the frame geometry, load demand and member sizes from Initial Link Selection tab.
8. **Column Pu (kips):** This value is determined as maximum column axial force from SST_LC 1-7, SST_LC 9-22 & SST_LC 33-36 for standard "ELF" or "MRSA" procedure and SST_LC1-7, SST_LC 9-22 & SST_LC 37-40 for orthogonal load combinations when structural irregularities are present.
9. **V_bm Gravity (kips):** This value is determined as maximum shear force in beam due to gravity loads from SST_LC 8.
10. **Stiffener Provided:** User input based on column flange and web strength for concentrated forces. If capacity less than demand load, provide Stiffener.
11. **Doubler Plate Provided:** If capacity less than demand load, provide Doubler Plate. User should note that it is sometimes more economical to increase column size/weight by 70-100 plf. than welding on a doubler plate.

12. **Stiffener Required:** (Yes/No) This value is auto-calculated by Plugin based on the demand loads exerted from link axial forces. If stiffener is required, column 10 (stiffener provided) is default to yes.
13. **Minimum Stiffener Thickness (in):** This value is auto-calculated by Plugin based on whether a stiffener is required or not in order to satisfy the design DCR. Minimum stiffener thickness cannot be less than 3/8".
14. **Minimum Doubler Thickness (in):** This value is auto-calculated by Plugin based on whether a doubler plate is required or not in order to satisfy the design DCR. If doubler plate is required, column 11 (doubler plate provided) is default to yes.
15. **Strong Column – Weak Beam Check, SCWB DCR:** Refer Section 4.1.5 (2a), ICC-ES ESR-2802.
16. **Column Panel Zone Check:** Panel zone check for the column due to demand loads from axial force in Yield-Link® calculated per J10-9 or J10-10, AISC Steel Construction Manual, 14th Edition.
17. **Column Flange Check:** Determine the minimum column flange thickness for flexural yielding per EQ. A-41 of ICC-ES ESR-2802 or EQ. 12.9-38, Chapter 12 of ANSI/AISC 358s2-20.

5.4 Shear Tab Check



Elev. ID	Grid ID	Story	Beam Unique Name	Beam Size	Yield-Link® Size	Axial Pu (kips)	Shear Vg_LC08 (kips)	Shear Vu (kips)	Shear Vg_LC01-07 (kips)	No. Vert. Bolts	No. Horz. Bolts	Bolt Size (in)	Bolt Type	SP Thickness (in)	No. of SP	Fillet Weld Size (in)	Beam Web DCR	Shear Plate Geometry Check	Shear Plate DCR	Bolt DCR	Fillet Weld DCR
1	A	Story4	65	W27X84	YL6-3	16.54	5.67	45.08	5.67	7	2	7/8	A325X	1/2	1	5/16	0.227	OK	0.350	0.342	0.204
1	B	Story4	66	W27X84	YL6-3	16.54	5.51	44.92	5.55	7	2	7/8	A325X	1/2	1	5/16	0.227	OK	0.349	0.341	0.203
1	C	Story4	67	W27X84	YL6-3	16.54	5.67	45.08	5.71	7	2	7/8	A325X	1/2	1	5/16	0.227	OK	0.350	0.342	0.204
1	A	Story3	89	W27X84	YL6-3	22.06	5.73	45.14	5.73	7	2	7/8	A325X	1/2	1	5/16	0.311	OK	0.366	0.417	0.204
1	B	Story3	90	W27X84	YL6-3	10.84	5.51	44.92	5.58	7	2	7/8	A325X	1/2	1	5/16	0.219	OK	0.337	0.274	0.203
1	C	Story3	91	W27X84	YL6-3	22.06	5.73	45.14	5.80	7	2	7/8	A325X	1/2	1	5/16	0.311	OK	0.366	0.417	0.204
1	A	Story2	113	W27X84	YL6-3	8.25	5.66	45.07	5.66	7	2	7/8	A325X	1/2	1	5/16	0.220	OK	0.334	0.249	0.204
1	B	Story2	114	W27X84	YL6-3	8.25	5.51	44.92	5.61	7	2	7/8	A325X	1/2	1	5/16	0.219	OK	0.333	0.249	0.203
1	C	Story2	115	W27X84	YL6-3	8.25	5.66	45.07	5.77	7	2	7/8	A325X	1/2	1	5/16	0.220	OK	0.334	0.249	0.204
1	A	Story1	137	W27X84	YL6-3	32.69	5.56	44.97	5.61	7	2	7/8	A325X	1/2	1	5/16	0.473	OK	0.405	0.573	0.204
1	B	Story1	138	W27X84	YL6-3	5.02	5.51	44.92	5.66	7	2	7/8	A325X	1/2	1	5/16	0.219	OK	0.330	0.225	0.203
1	C	Story1	139	W27X84	YL6-3	32.69	5.56	44.97	5.72	7	2	7/8	A325X	1/2	1	5/16	0.473	OK	0.405	0.573	0.204
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22

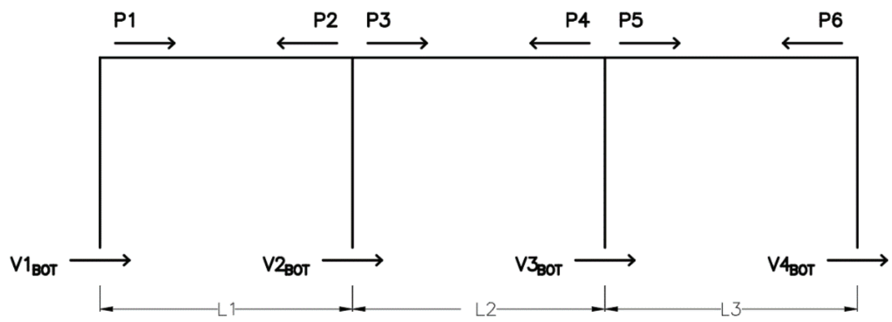
Columns 1-6 are explained in Initial Link Selection Tab details.

7. **Axial, Pu (kips):** Maximum axial force in beam per Overstrength Load Combinations (SST_LC 33-36) from SAP2000® / ETABS® output. User has the option to overwrite this value if different.

User Note [7]:

- **i.** When rigid/semi-rigid diaphragms are used, axial force in beam is conservatively approximated using column shear at the story above and below the beam as explained below.
- **ii.** Column shear is calculated per Overstrength Load Combinations (SST_LC 31-34/SST_LC 35-38 per type of analytical procedure).

CASE 1: Roof level



$$P1 = V1_{BOT}$$

$$P2 = V2_{BOT} \text{ x } \left[\frac{L1}{L1+L2}\right]$$

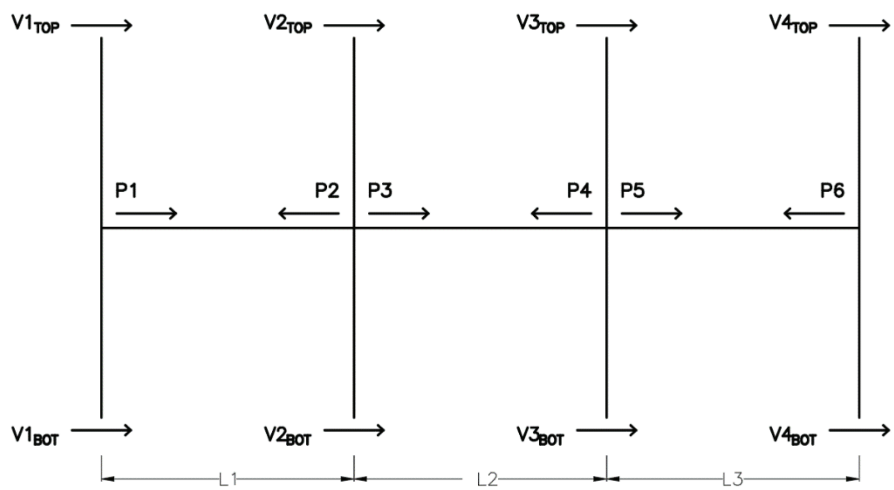
$$P3 = V2_{BOT} \text{ x } \left[\frac{L2}{L1+L2}\right]$$

$$P4 = V3_{BOT} \text{ x } \left[\frac{L2}{L2+L3}\right]$$

$$P5 = V3_{BOT} \text{ x } \left[\frac{L3}{L2+L3}\right]$$

$$P6 = V4_{BOT}$$

CASE 2: Mid-floor level



$$P1 = (V1_{BOT} - V1_{TOP})$$

$$P2 = (V2_{BOT} - V2_{TOP}) \text{ x } \left[\frac{L1}{L1+L2}\right]$$

$$P3 = (V2_{BOT} - V2_{TOP}) \text{ x } \left[\frac{L2}{L1+L2}\right]$$

$$P4 = (V3_{BOT} - V3_{TOP}) \text{ x } \left[\frac{L2}{L2+L3}\right]$$

$$P5 = (V3_{BOT} - V3_{TOP}) \text{ x } \left[\frac{L3}{L2+L3}\right]$$

$$P6 = (V4_{BOT} - V4_{TOP})$$

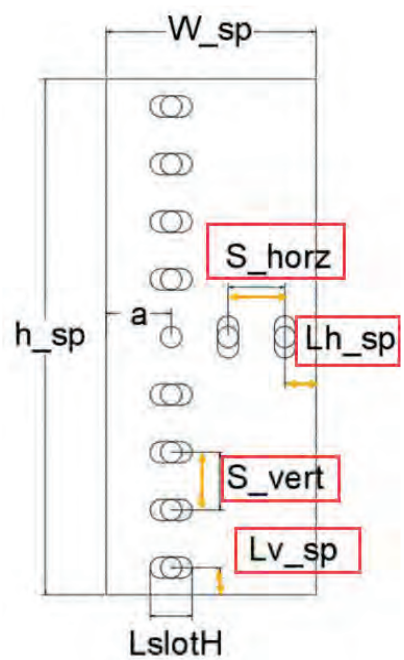
8. **Shear, Vg_LC08 (kips):** Total shear demand due to gravity load combination, SST_LC08 per Table 1.1 of this guide. It takes into account load combination, 1.2DL + f_1*L+0.2**S* per $V_{gravity}$ component from EQ. 12.9-34, ANSI/AISC 358s2-20. User has the option to overwrite this value if different.
9. **Shear, Vu (kips):** Total shear demand (Gravity + Seismic) on the beam and beam web-to-column flange connection, V_u per Chapter 12, EQ 12.9-34 of ANSI/AISC 358s2-20.
10. **Shear, Vg_LC01-07 (kips):** Total shear demand due to gravity load combinations, SST_LC01 to SST_LC07 per Table 1.1 of this guide for cases where higher gravity loads are anticipated. User has the option to overwrite this value if different.
11. **No. of Vertical Bolts:** Value defined per shear plate geometry using Table 2.
12. **No. of Horizontal Bolts:** User input based on shear tab design.
13. **Bolt Size:** User input based on shear tab design.
14. **Bolt Type:** User input based on shear tab design.
15. **Shear Plate Thickness:** User input based on shear tab design.
16. **No. of Shear Plates:** User input based on number of shear plates used in design.
17. **Fillet Weld Size (in.):** Weld at the shear plate to column flange is sized to develop the plate in shear. For double sided fillet welds, the minimum leg size is calculated as follows per Chapter 12, Step 15.4 of ANSI/AISC 358s2-20 (rounded up to the nearest 16ths of an inch.)

   Fillet weld size = 5/8 * Shear Plate Thickness

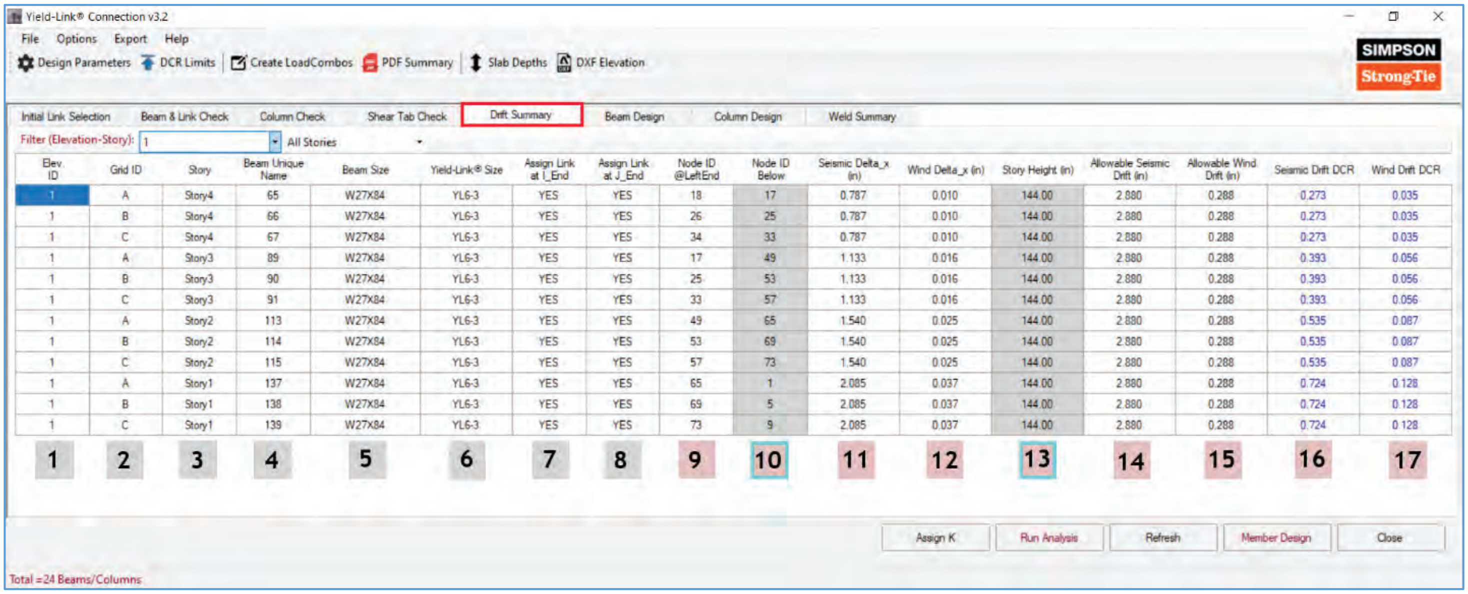
18. **Beam Web DCR:** Check strength of beam web for bolt bearing, tearout and block shear per Section J3.10 and J4.3. of ANSI/AISC 360-16.
19. **Shear Plate Geometry Check:** Shear Plate dimension should satisfy edge distance & bolt spacing requirements and allow the beam to fit between the link flanges.
20. **Shear Plate DCR:** Check strength of Shear Plate for yielding, rupture, bolt bearing, tearout and block shear per, Section J3.10 and J4.3 of ANSI/AISC 360-16.
21. **Bolt DCR:** Check shear strength of bolt per ANSI/AISC 360-16, EQ. J3-1.
22. **Fillet Weld DCR:** Check strength of fillet weld against demand load, Vu_bm (kip).

Table 3: Sample Shear Plate Bolt Details

<table>
<tr><th>Beam Size</th><th>Hole Dia. (in.)</th><th>n_Vbolts</th><th>S_vert (in.)</th><th>S_horz (in.)</th><th>Lv_sp (in.)</th><th>Lh_sp (in.)</th><th>L_slotH (in.)</th></tr>
<tr><td>W12</td><td rowspan="6">0.9375</td><td rowspan="4">3</td><td>2.1875</td><td rowspan="10">2.75</td><td rowspan="6">1.125</td><td rowspan="10">1.75</td><td rowspan="4">1.5</td></tr>
<tr><td>W14</td><td>2.750</td></tr>
<tr><td>W16</td><td>2.625</td></tr>
<tr><td>W18</td><td>3.250</td></tr>
<tr><td>W21</td><td rowspan="2">5</td><td>2.375</td><td rowspan="2">1.875</td></tr>
<tr><td>W24</td><td>2.750</td></tr>
<tr><td>W27</td><td rowspan="4">1.0625</td><td rowspan="2">7</td><td>2.3125</td><td rowspan="4">1.25</td><td rowspan="2">2.375</td></tr>
<tr><td>W30</td><td>2.750</td></tr>
<tr><td>W33</td><td rowspan="2">9</td><td>2.500</td><td rowspan="2">2.875</td></tr>
<tr><td>W36</td><td>2.875</td></tr>
</table>



5.5 Drift Summary

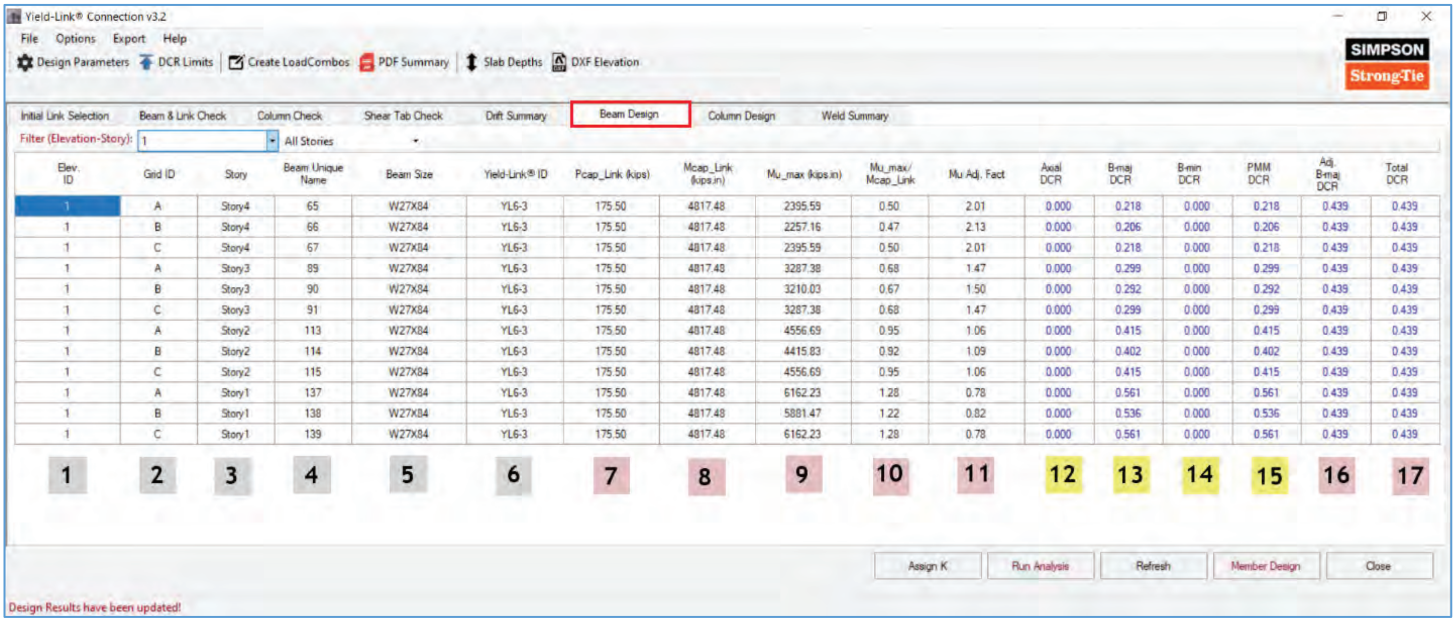


Elev. ID	Grid ID	Story	Beam Unique Name	Beam Size	Yield-Link® Size	Assign Link at I_End	Assign Link at J_End	Node ID @LeftEnd	Node ID Below	Seismic Delta_x (in)	Wind Delta_x (in)	Story Height (in)	Allowable Seismic Drift (in)	Allowable Wind Drift (in)	Seismic Drift DCR	Wind Drift DCR
1	A	Story4	65	W27X84	YL6-3	YES	YES	18	17	0.787	0.010	144.00	2.880	0.288	0.273	0.035
1	B	Story4	66	W27X84	YL6-3	YES	YES	26	25	0.787	0.010	144.00	2.880	0.288	0.273	0.035
1	C	Story4	67	W27X84	YL6-3	YES	YES	34	33	0.787	0.010	144.00	2.880	0.288	0.273	0.035
1	A	Story3	89	W27X84	YL6-3	YES	YES	17	49	1.133	0.016	144.00	2.880	0.288	0.393	0.056
1	B	Story3	90	W27X84	YL6-3	YES	YES	25	53	1.133	0.016	144.00	2.880	0.288	0.393	0.056
1	C	Story3	91	W27X84	YL6-3	YES	YES	33	57	1.133	0.016	144.00	2.880	0.288	0.393	0.056
1	A	Story2	113	W27X84	YL6-3	YES	YES	49	65	1.540	0.025	144.00	2.880	0.288	0.535	0.087
1	B	Story2	114	W27X84	YL6-3	YES	YES	53	69	1.540	0.025	144.00	2.880	0.288	0.535	0.087
1	C	Story2	115	W27X84	YL6-3	YES	YES	57	73	1.540	0.025	144.00	2.880	0.288	0.535	0.087
1	A	Story1	137	W27X84	YL6-3	YES	YES	65	1	2.085	0.037	144.00	2.880	0.288	0.724	0.128
1	B	Story1	138	W27X84	YL6-3	YES	YES	69	5	2.085	0.037	144.00	2.880	0.288	0.724	0.128
1	C	Story1	139	W27X84	YL6-3	YES	YES	73	9	2.085	0.037	144.00	2.880	0.288	0.724	0.128
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17

Columns 1-8 are explained in Initial Link Selection Tab details.

9. **Node ID @Left End:** Value used to determine story displacement after running analysis.
10. **Node ID Below:** Value used to determine story displacement after running analysis. User can modify this value to enter correct node ID at bottom if Column is divided into 2 or more parts.
11. **Seismic Delta_x (in.):** Design story drift, Δ is calculated as the difference of top and bottom story deflections times the deflection amplification factor, C_d. The deflection is calculated using EQ. 12.8 15 of ASCE/SEI 7-16.
12. **Wind Delta_x (in.):** Wind drift is calculated using SST_LC 23-24 as explained in Section 2.2 (3) of the User Guide.
13. **Story Height (in.):** Value auto-populated from ETABS® per selected story. User can modify this value to enter the correct story height if Column is divided into 2 or more parts.
14. **Allowable Seismic Drift (in.):** Value calculated per ASCE/SEI 7-16, Table 12.12-1 and defined as explained in section 2.2 (2) of the User Guide.
15. **Allowable Wind Drift (in.):** Value calculated per equation defined as explained in section 2.2 (2) of the User Guide.
16. **Seismic Drift DCR:** Check the design story drift value (Column 11) with the allowable seismic drift (Column 14) to determine Seismic Drift DCR.
17. **Wind Drift DCR:** Check the wind drift (Column 12) value with the allowable wind drift (Column 15) to determine Wind Drift DCR.

5.6 Beam Design

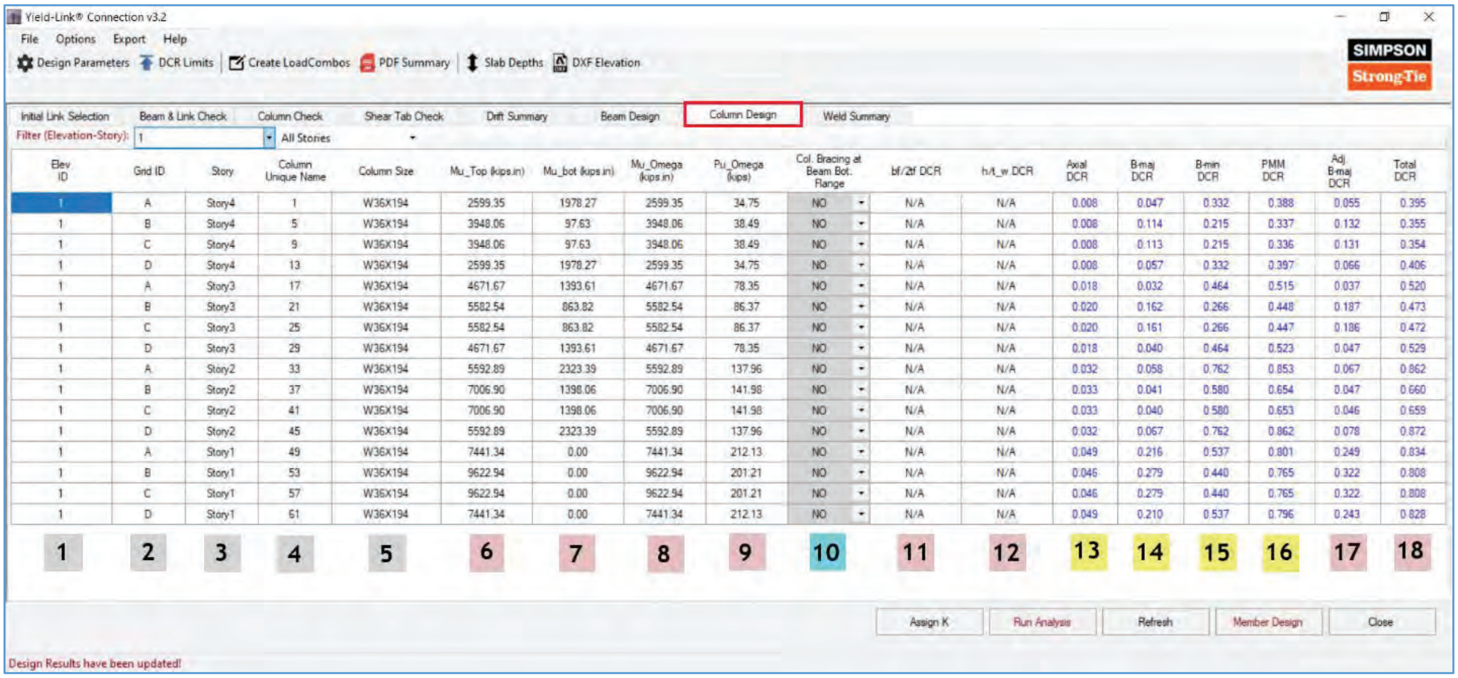


Elev. ID	Grid ID	Story	Beam Unique Name	Beam Size	Yield-Link® ID	Pcap_Link (kips)	Mcap_Link (kips.in)	Mu_max (kips.in)	Mu_max/ Mcap_Link	Mu Adj. Fact	Axial DCR	B-maj DCR	B-min DCR	PMM DCR	Adj. B-maj DCR	Total DCR
1	A	Story4	65	W27X84	YL6-3	175.50	4817.48	2395.59	0.50	2.01	0.000	0.218	0.000	0.218	0.439	0.439
1	B	Story4	66	W27X84	YL6-3	175.50	4817.48	2257.16	0.47	2.13	0.000	0.206	0.000	0.206	0.439	0.439
1	C	Story4	67	W27X84	YL6-3	175.50	4817.48	2395.59	0.50	2.01	0.000	0.218	0.000	0.218	0.439	0.439
1	A	Story3	89	W27X84	YL6-3	175.50	4817.48	3287.38	0.68	1.47	0.000	0.299	0.000	0.299	0.439	0.439
1	B	Story3	90	W27X84	YL6-3	175.50	4817.48	3210.03	0.67	1.50	0.000	0.292	0.000	0.292	0.439	0.439
1	C	Story3	91	W27X84	YL6-3	175.50	4817.48	3287.38	0.68	1.47	0.000	0.299	0.000	0.299	0.439	0.439
1	A	Story2	113	W27X84	YL6-3	175.50	4817.48	4556.69	0.95	1.06	0.000	0.415	0.000	0.415	0.439	0.439
1	B	Story2	114	W27X84	YL6-3	175.50	4817.48	4415.83	0.92	1.09	0.000	0.402	0.000	0.402	0.439	0.439
1	C	Story2	115	W27X84	YL6-3	175.50	4817.48	4556.69	0.95	1.06	0.000	0.415	0.000	0.415	0.439	0.439
1	A	Story1	137	W27X84	YL6-3	175.50	4817.48	6162.23	1.28	0.78	0.000	0.561	0.000	0.561	0.439	0.439
1	B	Story1	138	W27X84	YL6-3	175.50	4817.48	5881.47	1.22	0.82	0.000	0.536	0.000	0.536	0.439	0.439
1	C	Story1	139	W27X84	YL6-3	175.50	4817.48	6162.23	1.28	0.78	0.000	0.561	0.000	0.561	0.439	0.439
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17

Columns 1-6 are explained in Initial Link Selection Tab details.

7. **Pr_link (kips):** Probable maximum tensile strength of Yield-Link calculated per EQ. 12.9-6, Chapter 12 of ANSI/AISC 358s2-20.
8. **Mpr_link (kip-in.):** Probable maximum moment capacity of Yield-Link calculated per EQ. 12.9-28, Chapter 12 of ANSI/AISC 358s2-20.
9. **Mu_Omega (kip-in.):** This value is determined as maximum column moment from SST_LC 1-22, & SST_LC 33-36.
10. **Mu_Omega/Mpr_link:** Ratio of values from columns 8 and 9.
11. **Mu Adjustment Factor:** This value is calculated by taking the reciprocal of the ratio in column 10.
12. **Axial DCR:** Component of total demand over capacity ratio contributed by the axial force in the beam as calculated by ETABS® or SAP2000®.
13. **B-major DCR:** Component of total demand over capacity ratio contributed by bending along the major axis of the beam as calculated by ETABS® or SAP2000®.
14. **B-minor DCR:** Component of total demand over capacity ratio contributed by bending along the minor axis of the beam as calculated by ETABS® or SAP2000®.
15. **PMM DCR:** Sum of axial and bending along major and minor axes demand over capacity ratios as calculated by ETABS® or SAP2000®.
16. **Adjusted B-major DCR:** This value is calculated by taking the product of Mu adjustment Factor (Column 11) and B-major DCR (Column 13). Beam is designed by taking into account the moment capacity of the Yield-Link® as Mu cannot exceed Mpr_link. If Mu is less than Mpr_link, Mu is factored up to match Mpr_link for beam flexural design. If Mu is greater than Mpr_link, then Mu is factored down to Mpr_link for beam flexural design.
17. **Total DCR:** Sum of axial and bending along adjusted major axis and minor axis demand over capacity ratios as calculated by ETABS® or SAP2000®.

5.7 Column Design



Elev ID	Grid ID	Story	Column Unique Name	Column Size	Mu_Top (kips.in)	Mu_bot (kips.in)	Mu_Omega (kips.in)	Pu_Omega (kips)	Col. Bracing at Beam Bot. Flange	bf/2tf DCR	h/t_w DCR	Axial DCR	B-maj DCR	B-min DCR	PMM DCR	Adj B-maj DCR	Total DCR
1	A	Story4	1	W36X194	2599.35	1978.27	2599.35	34.75	NO	N/A	N/A	0.008	0.047	0.332	0.388	0.055	0.395
1	B	Story4	5	W36X194	3948.06	97.63	3948.06	38.49	NO	N/A	N/A	0.008	0.114	0.215	0.337	0.132	0.355
1	C	Story4	9	W36X194	3948.06	97.63	3948.06	38.49	NO	N/A	N/A	0.008	0.113	0.215	0.336	0.131	0.354
1	D	Story4	13	W36X194	2599.35	1978.27	2599.35	34.75	NO	N/A	N/A	0.008	0.057	0.332	0.397	0.066	0.406
1	A	Story3	17	W36X194	4671.67	1393.61	4671.67	78.35	NO	N/A	N/A	0.018	0.032	0.464	0.515	0.037	0.520
1	B	Story3	21	W36X194	5582.54	863.82	5582.54	86.37	NO	N/A	N/A	0.020	0.162	0.266	0.448	0.187	0.473
1	C	Story3	25	W36X194	5582.54	863.82	5582.54	86.37	NO	N/A	N/A	0.020	0.161	0.266	0.447	0.186	0.472
1	D	Story3	29	W36X194	4671.67	1393.61	4671.67	78.35	NO	N/A	N/A	0.018	0.040	0.464	0.523	0.047	0.529
1	A	Story2	33	W36X194	5592.89	2323.39	5592.89	137.96	NO	N/A	N/A	0.032	0.058	0.762	0.853	0.067	0.862
1	B	Story2	37	W36X194	7006.90	1398.06	7006.90	141.98	NO	N/A	N/A	0.033	0.041	0.580	0.654	0.047	0.660
1	C	Story2	41	W36X194	7006.90	1398.06	7006.90	141.98	NO	N/A	N/A	0.033	0.040	0.580	0.653	0.046	0.659
1	D	Story2	45	W36X194	5592.89	2323.39	5592.89	137.96	NO	N/A	N/A	0.032	0.067	0.762	0.862	0.078	0.872
1	A	Story1	49	W36X194	7441.34	0.00	7441.34	212.13	NO	N/A	N/A	0.049	0.216	0.537	0.801	0.249	0.834
1	B	Story1	53	W36X194	9622.94	0.00	9622.94	201.21	NO	N/A	N/A	0.046	0.279	0.440	0.765	0.322	0.808
1	C	Story1	57	W36X194	9622.94	0.00	9622.94	201.21	NO	N/A	N/A	0.046	0.279	0.440	0.765	0.322	0.808
1	D	Story1	61	W36X194	7441.34	0.00	7441.34	212.13	NO	N/A	N/A	0.049	0.210	0.537	0.796	0.243	0.828
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18

Columns 1-5 are explained in Column Check Tab details.

6. **Mu_Top (kip-in.):** Absolute maximum moment demand from elastic analysis at top of column calculated using load combinations per Table 1.1 & 1.2.
7. **Mu_Bottom (kip-in.):** Absolute maximum moment demand from elastic analysis at bottom of column calculated using load combinations per Table 1.1 & 1.2.
8. **Mu_Omega (kip-in.):** This value is calculated as the maximum of Mu_Top and Mu_Bottom.
9. **Pu_Omega (kip):** Absolute maximum axial load demand from elastic analysis in column calculated using load combinations per Table 1.1 & 1.2.
10. **Bracing at Beam Bottom Flange (Yes/No):** Provide lateral bracing of columns in accordance with Chapter 12, Section 12.3.2 (7) of ANSI/AISC 358s2-20. Refer column 17 below.
11. **bf/2tf check:** Limiting width-to-thickness ratio for unstiffened elements subjected to compression per Table D1.1, ANSI/AISC 341-16. (Applicable only at story 1 of fixed base system per Chapter 12, Section 12.3.2 (6a) of ANSI/AISC 358s2-20)
12. **h/tw check:** Limiting width-to-thickness ratio for stiffened elements subjected to compression per Table D1.1 of ANSI/AISC 341-16. (Applicable only at story 1 of fixed base system per Chapter 12, Section 12.3.2 (6a) of ANSI/AISC 358s2-20)
13. **Axial DCR:** Component of total demand over capacity ratio contributed by the axial force (calculated from Overstrength Load Combinations) in the column as designed by ETABS® or SAP2000®.

14. **B-major DCR:** Component of total demand over capacity ratio contributed by bending along the major axis of the column (calculated from Overstrength Load Combinations) as designed by ETABS® or SAP2000®.
15. **B-minor DCR:** Component of total demand over capacity ratio contributed by bending along the minor axis of the column (calculated from Overstrength Load Combinations) as designed by ETABS® or SAP2000®.
16. **PMM DCR:** Sum of axial and bending along major and minor axes demand over capacity ratios as calculated by ETABS® or SAP2000®.
17. **Adjusted B-major DCR:** Depending upon the location of Mu_Omega and column bracing at beam bottom flange, the major bending (B-Major) DCR can be adjusted as follows:

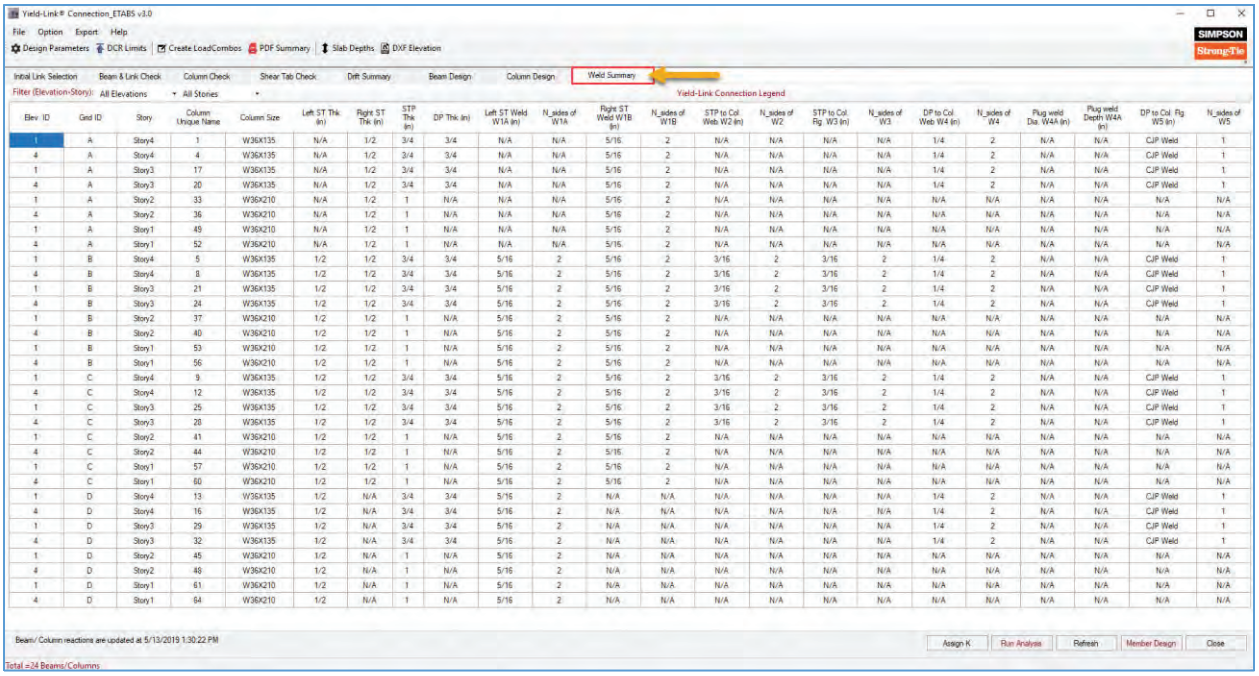
<table>
<tr><th rowspan="4">Column Design</th><th colspan="3">Case</th></tr>
<tr><th rowspan="3">Mu_Top < Mu_Bottom</th><th colspan="2">Mu_Top > = Mu_Bottom</th></tr>
<tr><th colspan="2">Column Bracing at Beam Bottom Flange?</th></tr>
<tr><th>NO</th><th>YES</th></tr>
<tr><td>Adjusted B-Major DCR</td><td>B-Major DCR x 1</td><td>B-Major DCR x 1</td><td>B-Major DCR x $\frac{Z_x}{S_x}$</td></tr>
</table>

User Note [8]:

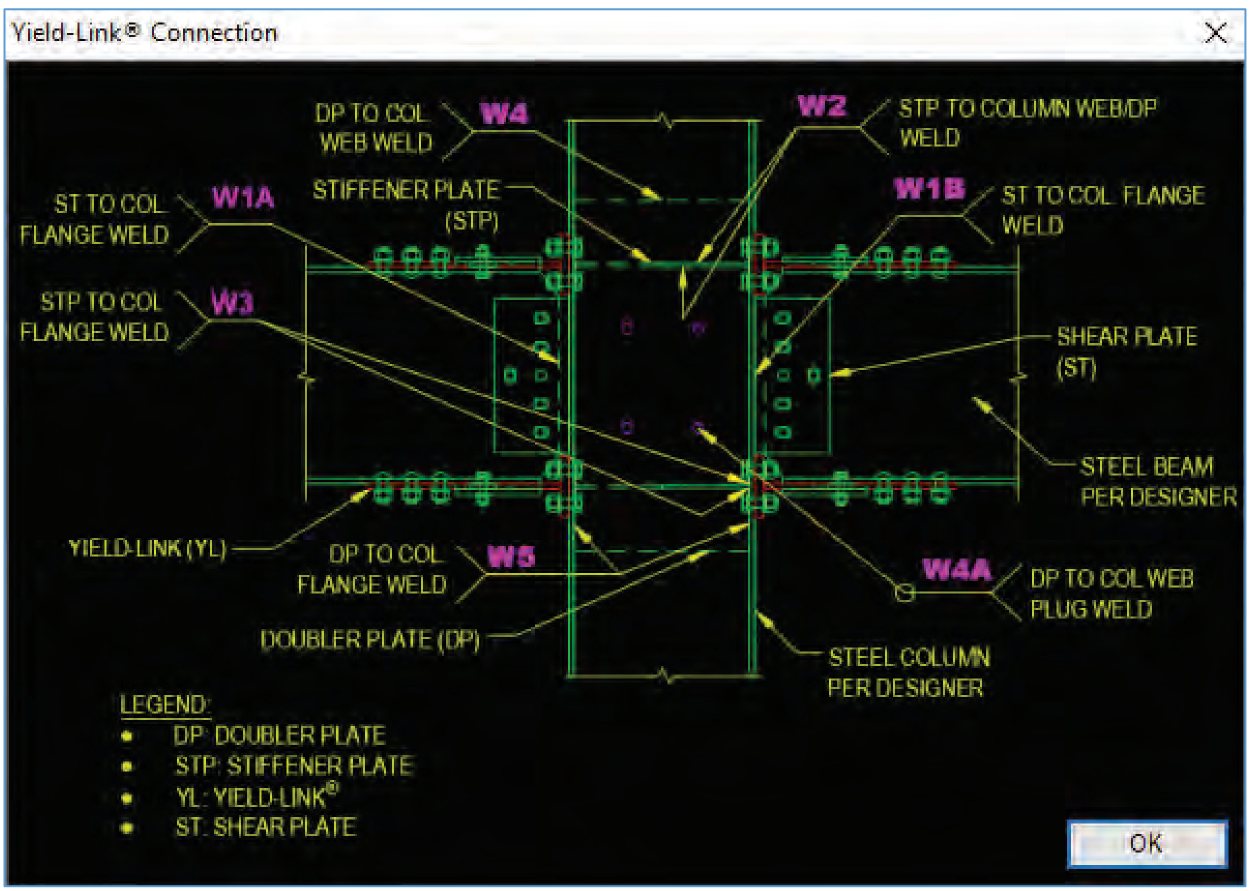
Sx is the elastic section modulus (in^3) and Zx is the plastic section modulus (in^3) of the column section.

18. **Total DCR:** Sum of axial and bending along adjusted major axis and minor axis demand over capacity ratios as calculated by ETABS® or SAP2000®.

5.8 Weld Summary



Elev ID	Grid ID	Story	Column Unique Name	Column Size	Left ST Thk (in)	Right ST Thk (in)	STP Thk (in)	DP Thk (in)	Left ST Weld W1A (in)	N_sides of W1A	Right ST Weld W1B (in)	N_sides of W1B	STP to Col Web W2 (in)	N_sides of W2	STP to Col Flg W3 (in)	N_sides of W3	DP to Col Web W4 (in)	N_sides of W4	Plug weld Dia. W4A (in)	Plug weld Depth W4A (in)	DP to Col Flg W5 (in)	N_sides of W5
1	A	Story4	1	W36X135	N/A	1/2	3/4	3/4	N/A	N/A	5/16	2	N/A	N/A	N/A	N/A	1/4	2	N/A	N/A	CJP Weld	1
4	A	Story4	4	W36X135	N/A	1/2	3/4	3/4	N/A	N/A	5/16	2	N/A	N/A	N/A	N/A	1/4	2	N/A	N/A	CJP Weld	1
1	A	Story3	17	W36X135	N/A	1/2	3/4	3/4	N/A	N/A	5/16	2	N/A	N/A	N/A	N/A	1/4	2	N/A	N/A	CJP Weld	1
4	A	Story3	20	W36X135	N/A	1/2	3/4	3/4	N/A	N/A	5/16	2	N/A	N/A	N/A	N/A	1/4	2	N/A	N/A	CJP Weld	1
1	A	Story2	33	W36X210	N/A	1/2	1	N/A	N/A	N/A	5/16	2	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
4	A	Story2	36	W36X210	N/A	1/2	1	N/A	N/A	N/A	5/16	2	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
1	A	Story1	49	W36X210	N/A	1/2	1	N/A	N/A	N/A	5/16	2	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
4	A	Story1	52	W36X210	N/A	1/2	1	N/A	N/A	N/A	5/16	2	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
1	B	Story4	5	W36X135	1/2	1/2	3/4	3/4	5/16	2	5/16	2	3/16	2	3/16	2	1/4	2	N/A	N/A	CJP Weld	1
4	B	Story4	8	W36X135	1/2	1/2	3/4	3/4	5/16	2	5/16	2	3/16	2	3/16	2	1/4	2	N/A	N/A	CJP Weld	1
1	B	Story3	21	W36X135	1/2	1/2	3/4	3/4	5/16	2	5/16	2	3/16	2	3/16	2	1/4	2	N/A	N/A	CJP Weld	1
4	B	Story3	24	W36X135	1/2	1/2	3/4	3/4	5/16	2	5/16	2	3/16	2	3/16	2	1/4	2	N/A	N/A	CJP Weld	1
1	B	Story2	37	W36X210	1/2	1/2	1	N/A	5/16	2	5/16	2	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
4	B	Story2	40	W36X210	1/2	1/2	1	N/A	5/16	2	5/16	2	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
1	B	Story1	53	W36X210	1/2	1/2	1	N/A	5/16	2	5/16	2	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
4	B	Story1	56	W36X210	1/2	1/2	1	N/A	5/16	2	5/16	2	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
1	C	Story4	9	W36X135	1/2	1/2	3/4	3/4	5/16	2	5/16	2	3/16	2	3/16	2	1/4	2	N/A	N/A	CJP Weld	1
4	C	Story4	12	W36X135	1/2	1/2	3/4	3/4	5/16	2	5/16	2	3/16	2	3/16	2	1/4	2	N/A	N/A	CJP Weld	1
1	C	Story3	25	W36X135	1/2	1/2	3/4	3/4	5/16	2	5/16	2	3/16	2	3/16	2	1/4	2	N/A	N/A	CJP Weld	1
4	C	Story3	28	W36X135	1/2	1/2	3/4	3/4	5/16	2	5/16	2	3/16	2	3/16	2	1/4	2	N/A	N/A	CJP Weld	1
1	C	Story2	41	W36X210	1/2	1/2	1	N/A	5/16	2	5/16	2	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
4	C	Story2	44	W36X210	1/2	1/2	1	N/A	5/16	2	5/16	2	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
1	C	Story1	57	W36X210	1/2	1/2	1	N/A	5/16	2	5/16	2	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
4	C	Story1	60	W36X210	1/2	1/2	1	N/A	5/16	2	5/16	2	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
1	D	Story4	13	W36X135	1/2	N/A	3/4	3/4	5/16	2	N/A	N/A	N/A	N/A	N/A	N/A	1/4	2	N/A	N/A	CJP Weld	1
4	D	Story4	16	W36X135	1/2	N/A	3/4	3/4	5/16	2	N/A	N/A	N/A	N/A	N/A	N/A	1/4	2	N/A	N/A	CJP Weld	1
1	D	Story3	29	W36X135	1/2	N/A	3/4	3/4	5/16	2	N/A	N/A	N/A	N/A	N/A	N/A	1/4	2	N/A	N/A	CJP Weld	1
4	D	Story3	32	W36X135	1/2	N/A	3/4	3/4	5/16	2	N/A	N/A	N/A	N/A	N/A	N/A	1/4	2	N/A	N/A	CJP Weld	1
1	D	Story2	45	W36X210	1/2	N/A	1	N/A	5/16	2	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
4	D	Story2	48	W36X210	1/2	N/A	1	N/A	5/16	2	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
1	D	Story1	61	W36X210	1/2	N/A	1	N/A	5/16	2	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
4	D	Story1	64	W36X210	1/2	N/A	1	N/A	5/16	2	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A

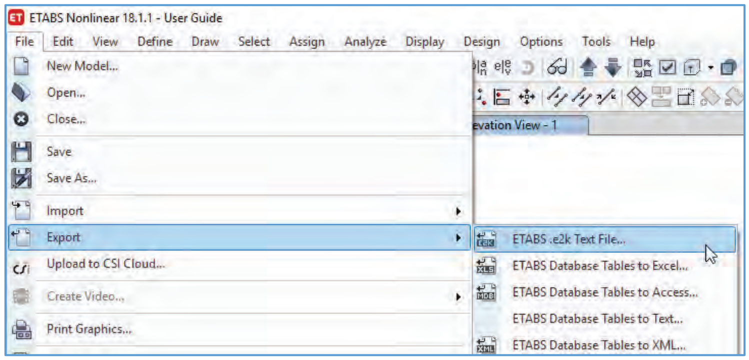


Columns 1-5 are explained in Column Design Tab details.

6. **No. Left ST:** Number of shear plates used on left side of column per input from Shear Tab Check.
7. **Left ST Thk (in.):** Thickness of shear plates on left side of column per input from Shear Tab Check.
8. **No. Right ST:** Number of shear plates used on right side of column per input from Shear Tab Check.
9. **Right ST Thk (in.):** Thickness of shear tab on right side of column per user input from Shear Tab Check.
10. **Left ST Fillet Weld W1A (in.):** Thickness of weld connecting left shear plate to col. flange per design.
11. **N_Sides of Fillet W1A:** Number of sides of weld connecting left shear plate to column flange.
12. **Left ST PJP Weld W1A (in.):** Thickness of weld connecting left shear plate to col. flange per design. Applies for the field welded shear plate only if No. of Left ST = 2.
13. **N_sides of PJP W1A:** Number of sides of weld connecting left shear plate to column flange. Applies for the field welded shear plate only if No. of Left ST = 2.
14. **Right ST Fillet Weld W1B (in.):** Thickness of weld connecting right shear plate to column flange.
15. **N_Sides of Fillet W1B:** Number of sides of weld connecting right shear plate to column flange.
16. **Right ST PJP Weld W1B (in.):** Thickness of weld connecting right shear plate to col. flange per design. Applies for the field welded shear plate only if No. of Right ST = 2.
17. **N_sides of PJP W1B:** Number of sides of weld connecting right shear plate to column flange. Applies for the field welded shear plate only if No. of Right ST = 2.
18. **STP Thk (in.):** Thickness of Stiffener Plate per design from Column Check.
19. **DP Thk (in.):** Thickness of Doubler Plate per design from Column Check.
20. **STP to Col. Web W2 (in.):** Thickness of weld connecting stiffener plate to column web per design.
21. **N_Sides of W2:** Number of sides of weld connecting stiffener plate to column web.
22. **STP to Col. Flg. W3 (in.):** Thickness of weld connecting stiffener plate to column flange per design.
23. **N_Sides of W3:** Number of sides of weld connecting stiffener plate to column flange.
24. **DP to Col. Web W4 (in.):** Thickness of weld connecting doubler plate to column web per design.
25. **N_Sides of W4:** Number of sides of weld connecting doubler plate to column web.
26. **Plug Weld Dia. W4A (in.):** Diameter of plug welds used to connect doubler plate to column web.
27. **Plug Weld Depth W4A (in.):** Depth of plug welds used to connect doubler plate to column web.
28. **DP to Col. Flg. W5 (in.):** Thickness of weld connecting doubler plate to column flange per design.
29. **N_Sides of W5:** Number of sides of weld connecting doubler plate to column flange per design.

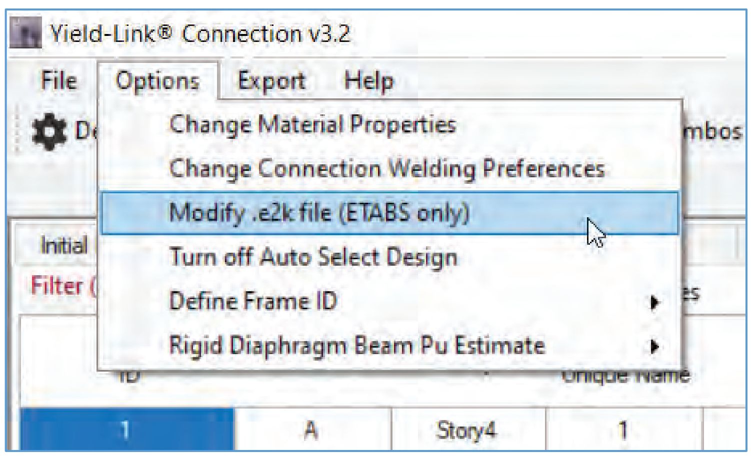
Appendix A: Define Load Patterns using ETABS® .e2k Text File

Step 1: Export ETABS® Model as .e2k Text File.

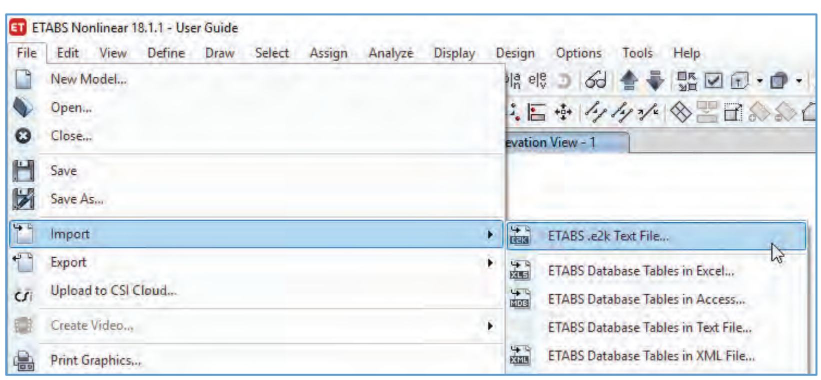


Step 2: Using Plugin User Interface, define design parameters as explained in section 2.2 (1)

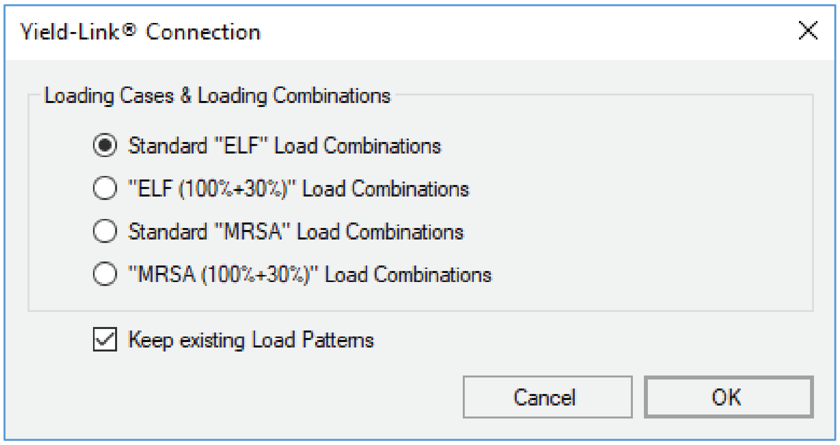
Step 3: Select the exported .e2k Text File using "Create/Modify Load Pattern in .e2k file" feature under Options Menu.



Step 4: Import modified .e2k text file and save updated ETABS® model.



Step 5: Create Load Cases & Combinations as explained in section 2.2 (3). Make sure to check the box for "Keep existing Load Patterns" to save load patterns created using the Modify.e2k file feature.



APPENDIX A5: YIELD-LINK MOMENT CONNECTION IN RAM STRUCTURAL SYSTEMS

SIMPSON Strong-Tie®

Yield-Link Moment Connection

Design in RAM Structural Systems v 17.03

Version 1.0

Date: 3/25/2022

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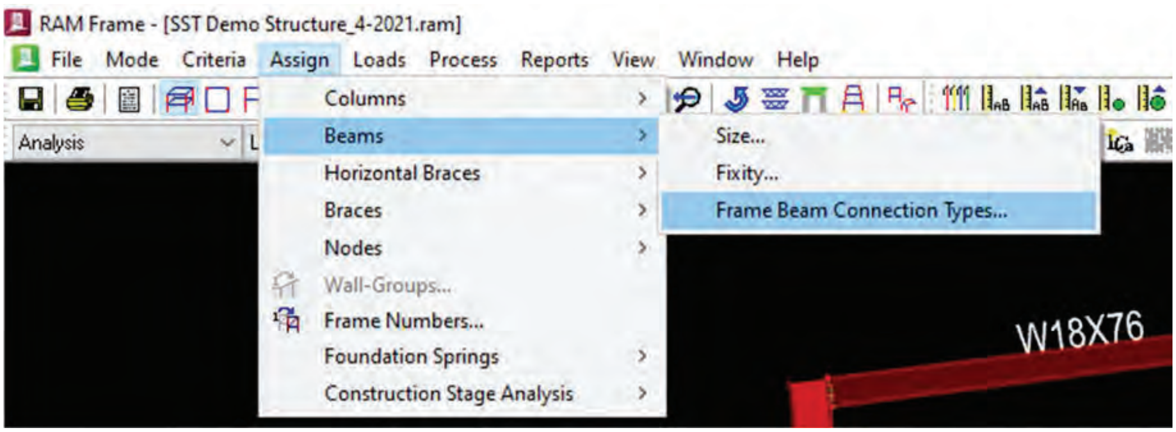
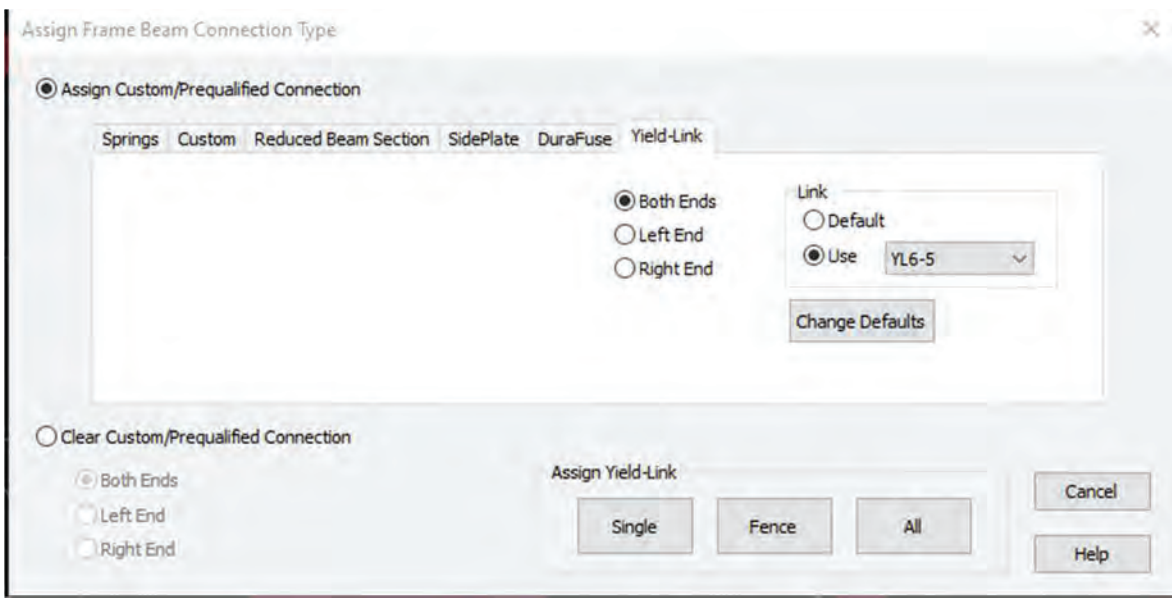
1.0 Yield-Link Moment Connection Input:

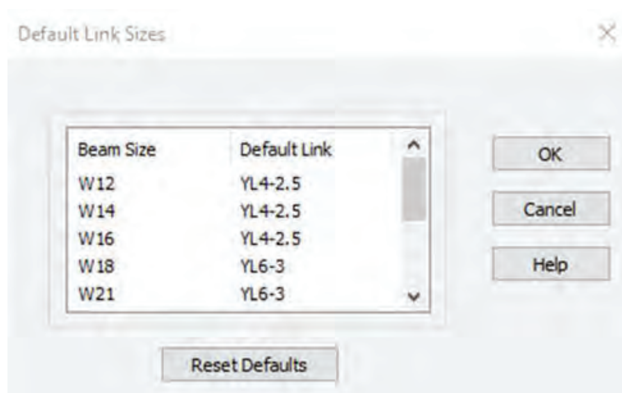
1.1. Yield-Link Moment Connection assignment under the "Analysis Module"

Yield-Link assignment to a beam-to-column assignment is under the Analysis Module in RAM. Under the Assign Beams, Frame Beam Connection Types, the user can find the user menu for Yield-Link Assignment.



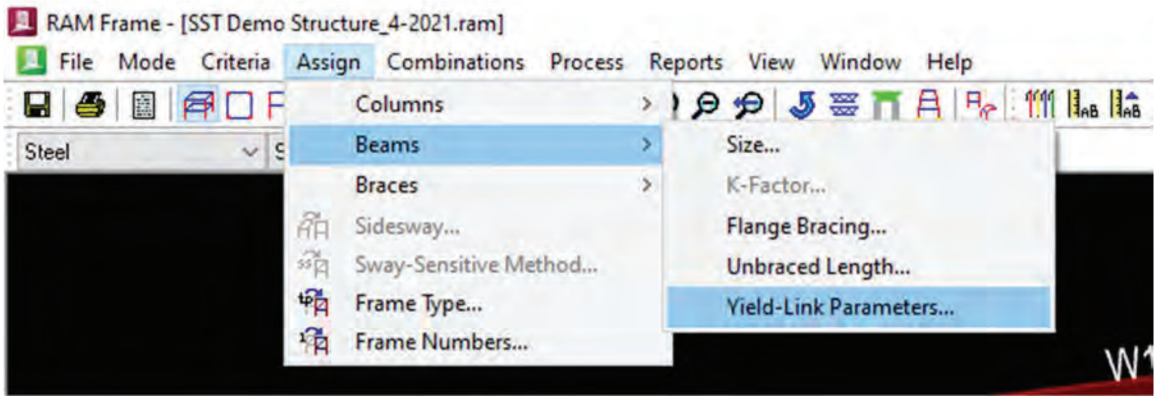


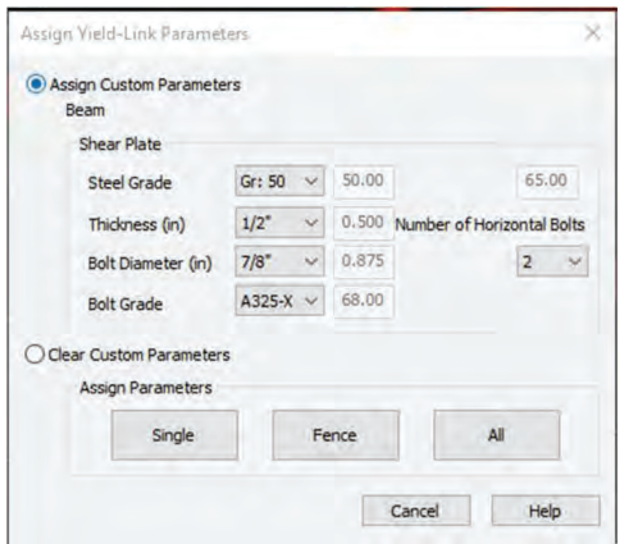
Users can start the assignment by assigning a default set of Links to each beam type based on the nominal beam size. The user can then modify each connection by applying a specific link to the beam.



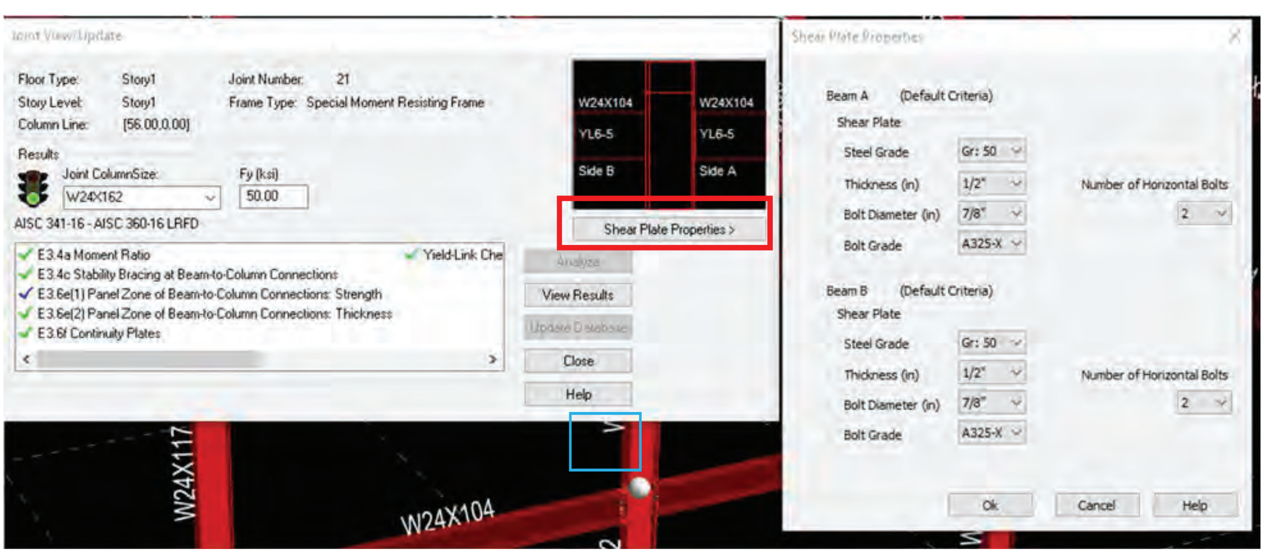
1.2. Shear plate assignment under Steel *Seismic Provisions* Module

Yield-Link shear plate parameters can be set under the Steel Standard Design Module in RAM. The user can set the shear tab thickness, bolt diameter etc. and apply the requirement to all the Yield-Link shear plate design.





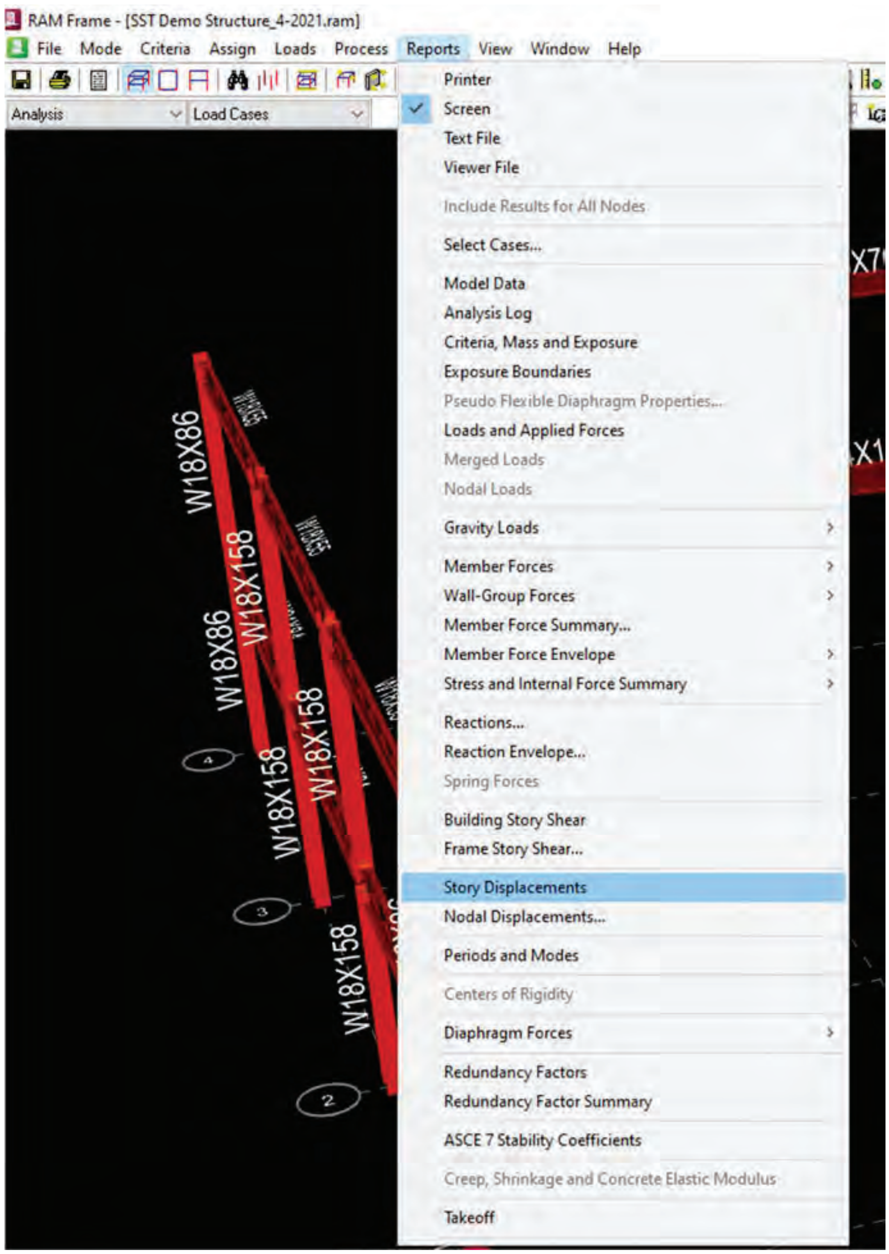
In addition, user can assign specific shear plate connection properties to each connection by right clicking on connection icon to bring up the Joint View/Update after connection design



2.0 Yield-Link Moment Connection Output:

2.1. Story Drift Check

Similar to the all other lateral system design for buildings. Story Drift check for RAM Structural Systems can be check by viewing the *Story Displacements* under the Analysis Module *Reports* Tab.



Story Displacements

RAM Frame 17.03.00.285
DataBase: SST Demo Structure_4-2021 03/24/22 13:35:47
Building Code: IBC

CRITERIA:

Rigid End Zones: Ignore Effects
Member Force Output: At Face of Joint
P-Delta: Yes Scale Factor (DL): 1.00 Scale Factor (LL): 0.50
Scale Factor (Roof): 0.50 Scale Factor (Snow): 0.50
Ground Level: Base
Mesh Criteria :
Max. Distance Between Nodes on Mesh Line (ft) : 4.92
Merge Node Tolerance (in) : 0.0098
Geometry Tolerance (in) : 0.0049
Walls Out-of-plane Stiffness Included in Analysis.
Rotational Fixities Released at Wall Foundation Nodes.
Use Reduced Stiffness for Steel Members (AISC 360): τb = 1.00
Sign considered for Dynamic Load Case Results.
Rigid Links Included at Fixed Beam-to-Wall Locations
Eigenvalue Analysis : Eigen Vectors (Subspace Iteration)

LOAD CASE DEFINITIONS:

E5	EQ_D	EQ_ASCE716_X_+E_Drft
E6	EQ_D	EQ_ASCE716_X_-E_Drft
E7	EQ_D	EQ_ASCE716_Y_+E_Drft
E8	EQ_D	EQ_ASCE716_Y_-E_Drft

Level: Story2, Diaph: 1

Center of Mass (ft): (56.00, 36.00)

LdC	Disp X in	Disp Y in	Theta Z rad
E5	$\boxed{1.64134}$	-0.00004	-0.00010
E6	1.64134	0.00004	0.00010
E7	-0.00000	1.43505	0.00016
E8	0.00000	1.43494	-0.00015

Level: Story1, Diaph: 1

Center of Mass (ft): (56.90, 36.00)

LdC	Disp X in	Disp Y in	Theta Z rad
E5	$\boxed{0.99587}$	-0.00072	-0.00006
E6	0.99587	0.00072	0.00006
E7	0.00000	0.93443	0.00011
E8	-0.00000	0.93219	-0.00009

An example of the displacements at the center of mass for each of the drift load cases is shown above. Please note to calculate the story drift, one would need to subtract the current story displacement from the story above (i.e. in the X-direction, the drift at the 2nd and 1st levels for load case E5 are):

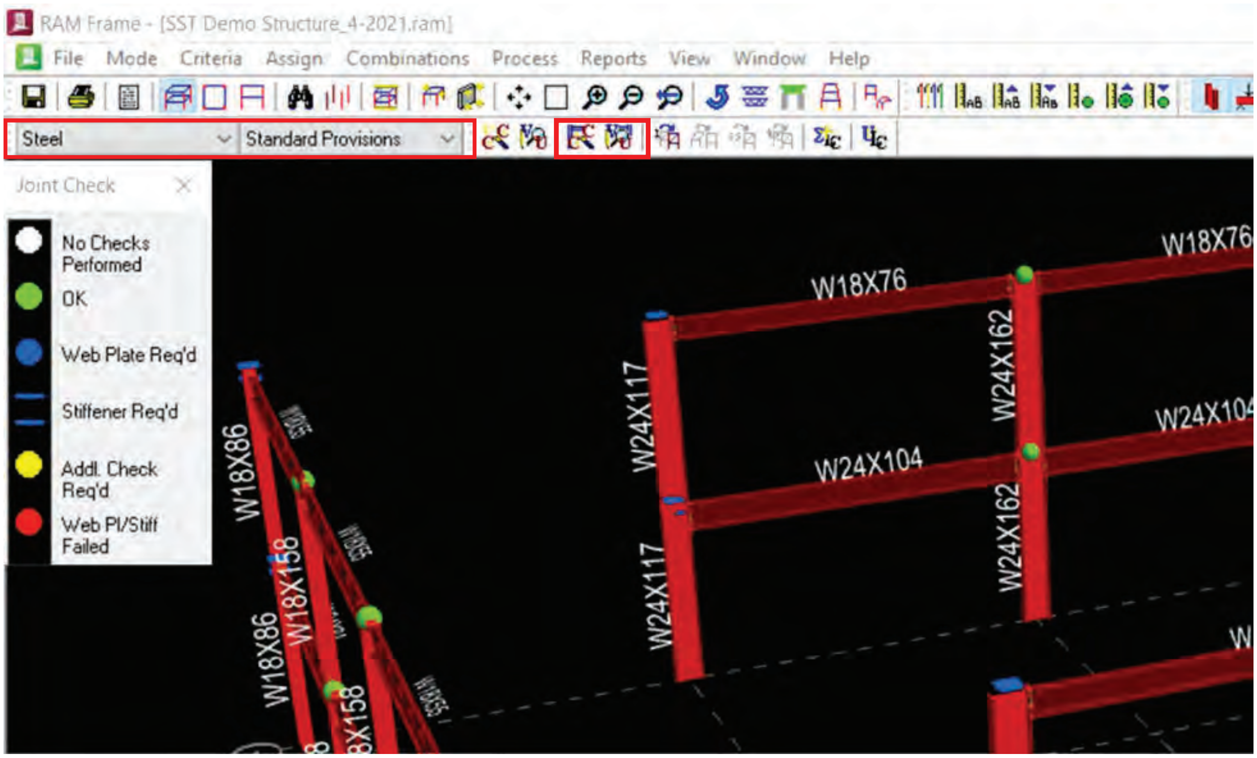
Delta_xe (2nd Level) =1.64134” - 0.99587” =0.64547”
Delta_xe (1st Level) = 0.99587” – 0” (base) = 0.99587”

If the story height is 14’-0” for both levels, and the allowable drift is 2% and Cd=5.5 for SMF, then the allowable drift will be Delta_allow=14’-0” x 0.02 / 5.5 = 0.61091”.

Since story drift at each level is more than the allowable drift. Then drift check is not adequate and the user will need to upsize Yield-Link, beam or column until the drift limit is met. This check will need to be done for all the drift load combinations and in each direction of the building (X and Y directions).

2.2 Link Strength Check

Link strength check in RAM is under the Steel Standard Provision module.



Once the connection design is completed, users can view all the results on the screen as shown in the figure above. In addition, users can also view individual connections by clicking on a join symbol to see detailed results. *Joint Check* PDF shown on the next page shows the Yield-Link strength to demand ratio.

Joint Code Check

RAM Structural System 17.03.00.285
DataBase: SST Demo Structure_4-2021 03/24/22 15:58:24
Building Code: IBC Steel Code: AISC360-16 LRFD

Story Number: 2 **Joint Number:** 8

Final Design

Yield-Link Design Check - **OK**
No Web Plate Required
No Bot Flange Stiffener Required

Joint Data and Material Properties

Web Plate Nominal Yield (ksi) 50.00
Stiffener Nominal Yield (ksi) 50.00

	Size	Plan Angle	Elev Angle	Yield(ksi)
Col. At Jnt:	W24X162	0.00	---	50.00
Beam SideA :	W18X76	0.00	0.00	50.00
Beam SideB :	W18X76	180.00	0.00	50.00

Criteria

Force on column flange is from beam moment, axial and shear forces.
Use actual beam moments to determine panel zone shear at the joint.
Optimize design of each stiffener at a joint

Yield-Link Design

Side	Size			Ratio
A	YL6-3:	Pu (kip) = 60.20	0.9 Pylink (kip) = 101.25	0.595
B	YL6-3:	Pu (kip) = 60.20	0.9 Pylink (kip) = 101.25	0.595

2.3 Column Stiffeners Check:

Stiffener check in RAM is under the Steel Standard Provision Connection Design.

Stiffener Required Area

Side	Flange	Ast Reqd (two stiffeners) (in2)	
SideA	Top	1.98	OK
SideA	Bot	1.98	OK

Compression

	Flange	Side A Force (kip)	Side A LCo	Side A Cap. (kip)	Side B Force (kip)	Side B LCo	Side B Cap. (kip)	Stiffen
Local Web Yld	Top	292.5	0	247.5	---	---	---	YES
	Bot	292.5	0	247.5	---	---	---	YES
Web Crippling	Top	292.5	0	311.0	---	---	---	NO
	Bot	292.5	0	311.0	---	---	---	NO

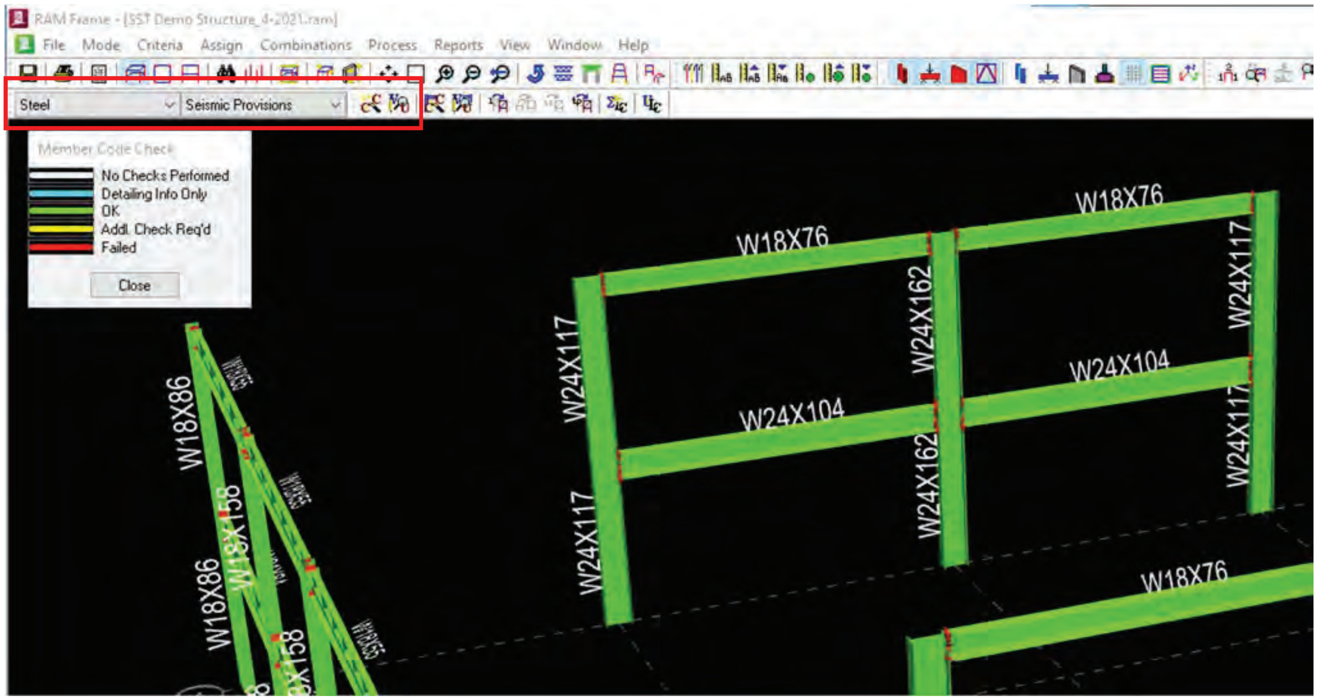
Tension

	Flange	Side A Force (kip)	Side A LCo	Side A Cap. (kip)	Side B Force (kip)	Side B LCo	Side B Cap. (kip)	Stiffen
Local Web Yld	Top	292.5	0	247.5	---	---	---	YES
	Bot	292.5	0	247.5	---	---	---	YES
Flange Bend.	Top	292.5	0	203.2	---	---	---	YES
	Bot	292.5	0	203.2	---	---	---	YES

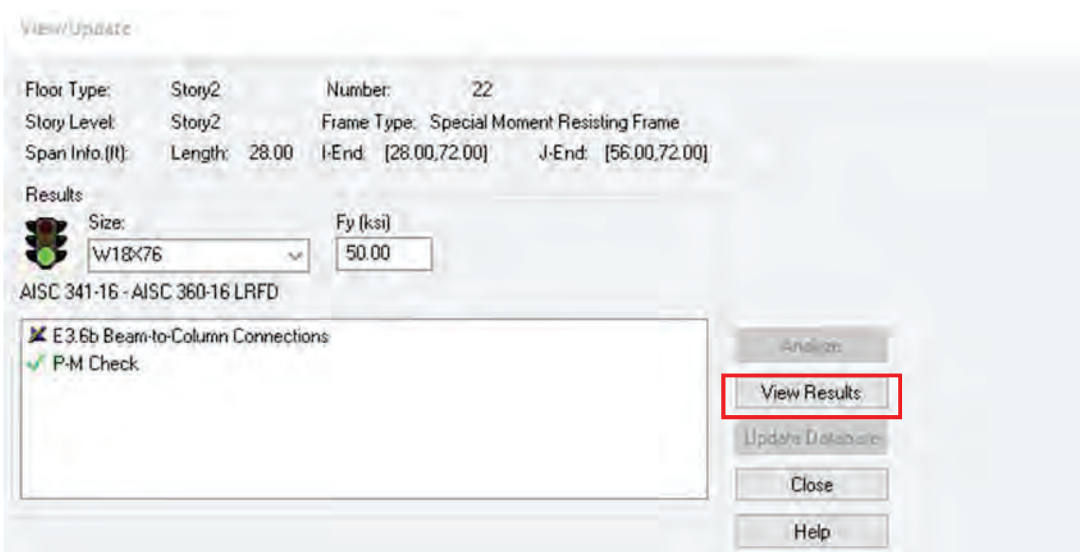
Note: LCo numbers correspond to the Numbers on the Load Combination Printout

2.4 Beam Check for Omega Forces and to Develop Connection Capacity (Mpr)

Yield-Link Moment Connection beam design in RAM is under the Steel Seismic Provisions module. Once the member design is done, member code check status for all the beams can be seen graphically as show below (green means OK).



In addition, the user can click on the individual members and bring up a screen to see detailed results. The user can then view a detailed PDF output of the beam design (as shown in the next page)



Please note, beam design for Yield-Link uses Omega/Overstrength load combinations. Making sure the beam can develop the link capacity (Mpr) is shown on page 2/2 of the output.

Seismic Provisions Member Code Check

RAM Structural System 17.03.00.285
DataBase: SST Demo Structure_4-2021
Building Code: IBC

03/24/22 16:09:33
Steel Code: AISC341-16 - LRFD

Beam Parameters

Story: Story2 Frame No: 2 Member No: 22
Fy (ksi): 50.00 Size: W18X76
Frame Type: Special Moment Resisting Frame
Left Connection - YL6-3
Right Connection - YL6-3

E3.6b Beam-to-Column Connection Requirements

Required flexural strength of connection (kip-ft) = 0.8Mp = 221.72 at story drift angle in (1)

See Additional AISC 358 Requirements for Assigned Connection Type.

INPUT DESIGN PARAMETERS:

	X-Axis	Y-Axis
Lu for Axial (ft)	5.00	5.00
Lu for Bending (ft)	5.00	5.00
K	1.00	1.00
Top Flange Continuously Braced	No	
Bottom Flange Continuously Braced	No	

CONTROLLING BEAM SEGMENT FORCES - SHEAR

Load Combination: 1.400 D + 1.400 ND1 + 0.500 Lp + 0.500 NL1 + 3.000 E5

Segment distance (ft) i - end 0.00
j - end 28.00

SHEAR CHECK:

Vux (kip) =	-22.52	1.00Vnx (kip)	=	232.05	Vux/1.00Vnx	=	0.097
Vuy (kip) =	-0.00	0.90Vny (kip)	=	403.92	Vuy/0.90Vny	=	0.000

CONTROLLING BEAM SEGMENT FORCES - AXIAL

Load Combination: 1.400 D + 1.400 ND1 + 0.500 Lp + 0.500 NL1 + 3.000 E1

Segment distance (ft) i - end 0.00
j - end 28.00

AXIAL CHECK:

Pu (kip) =	0.00	0.90Pnx (kip)	=	1003.50	Pu/0.90Pnx	=	0.000
		0.90Pny (kip)	=	1003.50	Pu/0.90Pny	=	0.000
		0.90Pn (kip)	=	1003.50	Pu/0.90Pn	=	0.000

CONTROLLING BEAM SEGMENT FORCES - FLEXURE

Load Combination: 1.400 D - 1.400 ND1 + 0.500 Lp - 0.500 NL1 - 3.000 E5

Segment distance (ft) i - end 0.00
j - end 28.00

CALCULATED PARAMETERS:

Pu (kip)	=	0.00	0.90Pn (kip)	=	1003.50
Mux (kip-ft)	=	-263.02	0.90Mnx (kip-ft)	=	611.25
Muy(kip-ft)	=	0.00	0.90Mny (kip-ft)	=	158.25
Cbx	=	2.28			

Seismic Provisions Member Code Check

RAM Structural System 17.03.00.285
DataBase: SST Demo Structure_4-2021
Building Code: IBC

Page 2/2
03/24/22 16:09:33
Steel Code: AISC341-16 - LRFD

INTERACTION EQUATION:

Pu/0.90*Pn=0.000
Mrx/Mcx = 0.430
Mpr (kip-ft) = 277.14 0.90Mnx (kip-ft) = 611.25 Ratio = 0.453

Verify Yield-Link Connections with Simpson Strong-Tie prior to finalizing structural design.

2.5. Column Check for Omega Axial + Moment

Similar to the beam design, column design for the Yield-Link Moment Connection is performed under the Steel Seismic Provision Module in RAM. Yield-Link Moment Connection column design requires the use Omega/Overstrength load combos for the Axial + Moment design check. This is shown on page 2/2 of the PDF output.

Seismic Provisions Member Code Check

Bentley

RAM Structural System 17.03.00.285 | Page 2/2
DataBase: SST Demo Structure_4-2021 | 03/24/22 16:09:33
Building Code: IBC | Steel Code: AISC341-16 - LRFD

SHEAR CHECK:

Vux (kip)	=	-53.25	1.00Vnx (kip)	=	400.95	Vux/1.00Vnx	= 0.133
Vuy (kip)	=	0.06	0.90Vny (kip)	=	587.52	Vuy/0.90Vny	= 0.000

CONTROLLING COLUMN FORCES - AXIAL

Load Combination: 1.400 D - 1.400 ND1 + 0.500 Lp - 0.500 NL1 - 3.000 E6

AXIAL CHECK:

Pu (kip)	=	147.68	0.90Pnx (kip)	=	1488.35	Pu/0.90Pnx	= 0.099
			0.90Pny (kip)	=	1218.89	Pu/0.90Pny	= 0.121
			0.90Pn (kip)	=	1218.89	Pu/0.90Pn	= 0.121

CONTROLLING COLUMN FORCES - FLEXURE

Load Combination: 1.400 D - 1.400 ND1 + 0.500 Lp - 0.500 NL1 - 3.000 E6

Axial		Load (kip)	147.68
Moment	Top	Mux (kip-ft)	692.08
		Muy (kip-ft)................	-0.87
Moment	Bot.	Mux (kip-ft)	0.00
		Muy (kip-ft)................	0.00

CALCULATED PARAMETERS:

Pu (kip)	=	147.68	0.90Pnx (kip)	=	1488.35
			0.90Pny (kip)	=	1218.89
Mux (kip-ft)	=	692.08	0.90Mnx (kip-ft)	=	1226.25
Muy(kip-ft)	=	-0.87	0.90Mny (kip-ft)	=	267.75
			Mcx (kip-ft)	=	1142.80
KL/Rx	=	16.56	KL/Ry	=	57.18
Cbx	=	1.67			

INTERACTION EQUATION:
Pu/0.90*Pn=0.099
Eq H1-3: 0.174 + 0.132 = 0.306
Eq H1-1b Per H1.3: 0.050 + 0.564 + 0.000 = 0.614

Verify Yield-Link Connections with Simpson Strong-Tie prior to finalizing structural design.

2.6 Column Panel Zone Check

Yield-Link Moment Connection column panel zone check in RAM is performed under the Steel Seismic Provision Module. Output below shows the panel zone check perform in RAM. When the column web alone is not adequate, doubler plate thickness and welding is also shown in the calculation output.

E3.6e(1) Panel-Zone of Beam-to-Column Connections: Strength --- OK With Web Plate

Bm.	Size	Bf Red. (in)	Z (in^3)	Z hng (in^3)	Mpr (kip-ft)
28	W24X104	0.000	289.00	289.00	605.72
27	W24X104	0.000	289.00	289.00	605.72
Bm.	**Mpr (kip-ft)**	**Sh (in)**	**V hng (kip)**	**Mf (kip-ft)**	**Vpz (kip)**
28	605.72	3.50	57.823	619.67	299.24
27	605.72	3.50	57.824	602.80	291.09

Sh = Distance face-of-column to location of hinge.
Vhng = Shear at hinge or col face from applicable load combinations with Ecl
Mpr = Pr-Link x (d+tstem). Zhng = Z at hinge.

Required Panel Zone Shear Strength (kip) = 503.01
Panel Zone Shear Reduced by Vc (kip) = 87.32
Axial Comp. for Panel Zone Calc. (kip) = 76.97 - Combination 1.400 D - 1.400 ND2 + 0.500 Lp - 0.500 NL2 - 3.000 E8
Panel Zone Strength Without Web Plate (kip) = 475.88 NG
Panel Zone Strength With Web Plate (kip) = 855.56 OK
No plug welds specified so both column and web plate must meet Equation 9-2
dz (in) = 22.600 wz (in) = 22.560
Column web thickness reqd (in) = 0.502 Thickness provided = 0.705 OK
Web Plate Thickness Used (in) = 0.625
Web PL thickness reqd (in) = 0.502 Thickness provided = 0.625 OK

Doubler and Continuity Plate Welds
Option: Extended Doubler Plates
Doubler to Cont. Plate fillet weld (in): 0.4375 Doubler to column flange weld (in): 0.8750

Stiffener Thicknesses (in): Side A Top: 0.500 Bot: 0.000 Side B Top: 0.500 Bot: 0.000

2.7 BRP, Beam Flange Thickness, BRP bolt, Beam Flange geometry and net section check

Yield-Link Moment Connection check in RAM is performed under the Steel Seismic Provision module. Output below shows the various checks done in RAM under the Seismic Provisions Joint Code Check.

Buckling Restraint Plate Thickness - OK

Side A	tbrp-min (in)	=	0.815	tbrp (in)	=	1.000	Ratio	=	0.815
Side B	tbrp-min (in)	=	0.815	tbrp (in)	=	1.000	Ratio	=	0.815

Beam Flange Thickness - OK

Side A	Pe (in)	=	10.362	tf-min (in)	=	0.516	tf (in)	=	0.750	Ratio	=	0.689
Side B	Pe (in)	=	10.362	tf-min (in)	=	0.516	tf (in)	=	0.750	Ratio	=	0.689

Buckling Restraint Plate Bolt Size and Quantity - OK

Side A	Vux-bolt (kip)	=	4.49	Vuy-bolt (kip)	=	9.89	# Bolts per side	=	2
	RnVx(T+V) (kip)	=	23.79	RnVy(T+V) (kip)	=	29.82	Ratio	=	0.332
Side B	Vux-bolt (kip)	=	4.49	Vuy-bolt (kip)	=	9.89	# Bolts per side	=	2
	RnVx(T+V) (kip)	=	23.79	RnVy(T+V) (kip)	=	29.82	Ratio	=	0.332

Beam Edge - OK

Side A	bf (in)	=	12.800	bf-min (in)	=	9.250	Ratio	=	0.723
Side B	bf (in)	=	12.800	bf-min (in)	=	9.250	Ratio	=	0.723

Beam Net Section - OK

Side A	Mpb-net (kip-ft)	=	1113.89	Mpr (kip-ft)	=	605.72	Ratio	=	0.544
Side B	Mpb-net (kip-ft)	=	1113.89	Mpr (kip-ft)	=	605.72	Ratio	=	0.544

2.8 Column Flange Width and Minimum Column Flange for Flexural Yielding check

Yield-Link Moment Connection check in RAM is performed under the Steel Seismic Provision Module. Output below shows the column flange checks done in RAM under the Seismic Provisions Joint Code Check.

Column Flange Width - OK

Side A	bcf (in)	=	13.000	bcf-min (in)	=	7.500	Ratio	=	0.577
Side B	bcf (in)	=	13.000	bcf-min (in)	=	7.500	Ratio	=	0.577

Column flange checked for smaller edge distance.

Side A	Dflange (in)	=	1.000	Lcol-edge (in)	=	4.000	Ratio	=	0.250
Side B	Dflange (in)	=	1.000	Lcol-edge (in)	=	4.000	Ratio	=	0.250

Column Connection

Side A (Top)	Yp (in)	=	265.82	tcf (in)	=	1.220	tcf-reqd (in)	=	0.632	Ratio	=	0.518
Side A (Bot)	Yc (in)	=	265.82	tcf (in)	=	1.220	tcf-min (in)	=	0.553	Ratio	=	0.453
Side B (Top)	Yp (in)	=	265.82	tcf (in)	=	1.220	tcf-reqd (in)	=	0.632	Ratio	=	0.518
Side B (Bot)	Yc (in)	=	265.82	tcf (in)	=	1.220	tcf-min (in)	=	0.553	Ratio	=	0.453

2.9 Shear Plate Check

Yield-Link Moment Connection shear plate check in RAM is performed under the Steel Seismic Provision Module. *Seismic Provisions Joint Code Check* output shows the shear plate connection design in detail as shown in the output below. Please note, all the failure modes as noted in the Yield-Link Moment Connection Excel Tool and ETABS/SAP2000 plugin is also done in RAM.

Seismic Provisions Joint Code Check

Bentley

RAM Structural System 17.03.00.285 — Page 2/4
DataBase: SST Demo Structure_4-2021 — 03/24/22 16:47:52
Building Code: IBC — Steel Code: AISC341-16 - LRFD

Yield-Link connection's top stiffener is required for Stability Bracing at Beam-to-Column Connections per AISC 341 Section E3.4c.

See Additional AISC 358 Requirements for Assigned Connection Type.

Yield-Link Connection Design - OK

Shear Plate Bolt Size - OK

Side A: A325-X Bolt Diameter (in) = 0.875 No. Horiz. Bolts = 2 No. Vert. Bolts = 5

Pu (kip)	= 51.20	Beam Vu (kip)	= 47.82		
Bolt Vu (kip)	= 27.33	ϕRn-bolt (kip)	= 40.59	Ratio	= 0.673

Side B: A325-X Bolt Diameter (in) = 0.875 No. Horiz. Bolts = 2 No. Vert. Bolts = 5

Pu (kip)	= 51.20	Beam Vu (kip)	= 57.82		
Bolt Vu (kip)	= 28.09	ϕRn-bolt (kip)	= 40.59	Ratio	= 0.692

Shear Plate Geometry - OK

Side A	Wsp (in)	= 8.000	hsp (in)	= 13.250	tsp (in)	= 0.500
	Svert (in)	= 2.750	Shoriz (in)	= 2.750	Horiz Lslot (in)	= 1.813
Side B	Wsp (in)	= 8.000	hsp (in)	= 13.250	tsp (in)	= 0.500
	Svert (in)	= 2.750	Shoriz (in)	= 2.750	Horiz Lslot (in)	= 1.813

Shear Plate Yielding (Vertical) - OK

Side A	Vu (kip)	= 47.82	ϕVy (kip)	= 198.75	Ratio	= 0.241
Side B	Vu (kip)	= 57.82	ϕVy (kip)	= 198.75	Ratio	= 0.291

Shear Plate Rupture (Vertical) - OK

Side A	Vu (kip)	= 47.82	ϕVr (kip)	= 120.66	Ratio	= 0.396
Side B	Vu (kip)	= 57.82	ϕVr (kip)	= 120.66	Ratio	= 0.479

Shear Plate Axial and Moment - OK

Side A	Vuy (kip)	= 47.82	Mecc (kip-ft)	= 13.95		
	L-Whitmore (in)	= 8.092	A-Whit. (in2)	= 4.046	θ-Whit. (Deg)	= 30.000
	fmax (ksi)	= 16.90	ϕb.Fy (ksi)	= 45.00	Ratio	= 0.376
Side B	Vuy (kip)	= 57.82	Mecc (kip-ft)	= 16.87		
	L-Whitmore (in)	= 8.092	A-Whit. (in2)	= 4.046	θ-Whit. (Deg)	= 30.000
	fmax (ksi)	= 17.79	ϕb.Fy (ksi)	= 45.00	Ratio	= 0.395

Shear Plate To Column Flange Fillet Weld - OK

Side A	Thickness (in)	= 0.313	Length (in)	= 12.750		
	Vu (kip)	= 47.82	ϕRn (kip)	= 177.47	Ratio	= 0.269
Side B	Thickness (in)	= 0.313	Length (in)	= 12.750		
	Vu (kip)	= 57.82	ϕRn (kip)	= 177.47	Ratio	= 0.326

Beam Web and Shear Tab Bearing: OK

	Horizontal React.			Vertical React.			Combined React.			
Side A	Pu	ϕRn	DCR	Vu	ϕRn	DCR	θ	Pr	ϕRn	DCR

Seismic Provisions Joint Code Check

RAM Structural System 17.03.00.285 Page 3/4
DataBase: SST Demo Structure_4-2021 03/24/22 16:47:52
Building Code: IBC Steel Code: AISC341-16 - LRFD

	(kip)	(kip)		(kip)	(kip)		(Deg)	(kip)	(kip)	
Beam Web	51.20	90.49	0.57	47.82	255.94	0.19	20.49	27.33	41.85	0.653
Shear Plate	51.20	90.49	0.57	47.82	231.26	0.21	20.49	27.33	102.38	0.267

	Horizontal React.			Vertical React.			Combined React.			
Side B	Pu	ϕRn	DCR	Vu	ϕRn	DCR	θ	Pr	ϕRn	DCR
	(kip)	(kip)		(kip)	(kip)		(Deg)	(kip)	(kip)	
Beam Web	51.20	90.49	0.57	57.82	255.94	0.23	24.31	28.09	43.37	0.648
Shear Plate	51.20	90.49	0.57	57.82	231.26	0.25	24.31	28.09	102.38	0.274

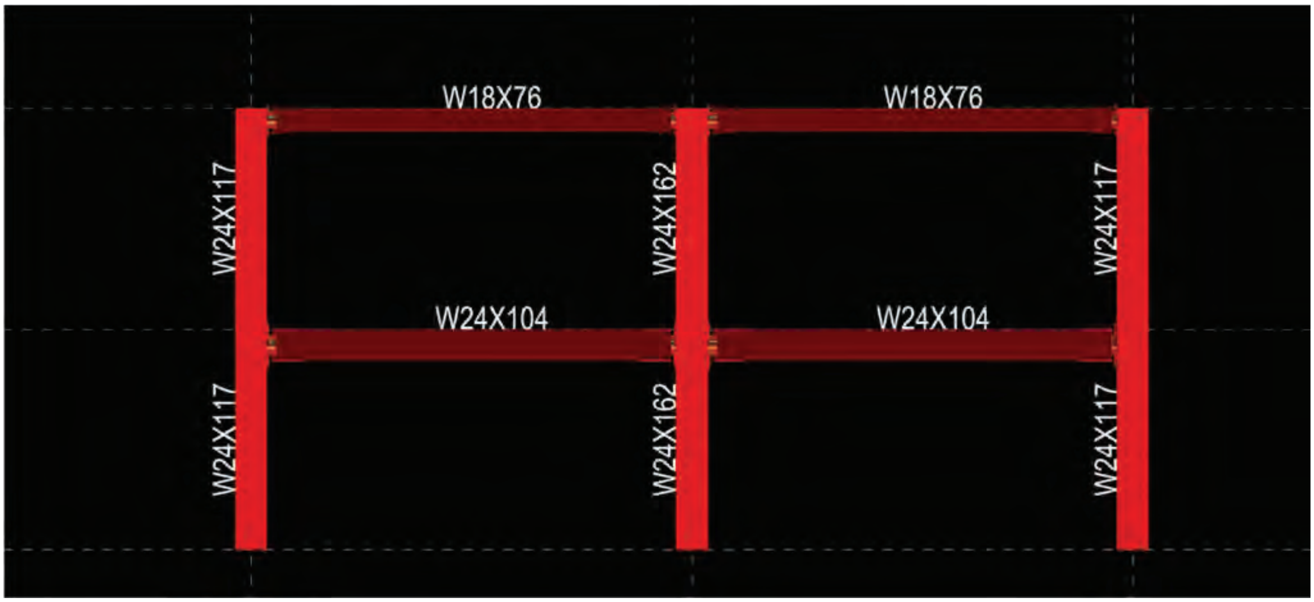
Beam Web and Shear Tab Block Shear: OK

	Horizontal React.			Vertical React.			Combined React.			
Side A	Pu	ϕRn	DCR	Vu	ϕRn	DCR	θ	Pr	ϕRn	DCR
	(kip)	(kip)		(kip)	(kip)		(Deg)	(kip)	(kip)	
Beam Web	51.20	101.25	0.51	47.82	210.36	0.23	20.49	70.06	232.38	0.307
Shear Plate	51.20	201.09	0.38	47.82	215.57	0.22	20.49	58.69	133.45	0.581

	Horizontal React.			Vertical React.			Combined React.			
Side B	Pu	ϕRn	DCR	Vu	ϕRn	DCR	θ	Pr	ϕRn	DCR
	(kip)	(kip)		(kip)	(kip)		(Deg)	(kip)	(kip)	
Beam Web	51.20	101.25	0.51	57.82	210.36	0.27	24.31	77.23	232.38	0.332
Shear Plate	51.20	201.09	0.38	57.82	215.57	0.27	24.31	61.85	133.45	0.640

2.10 Collector and Chore Forces Check for Shear Plate Design:

Since there is no user input for collector/chore forces in RAM for connection design. The user can use the Yield-Link Moment Connection Excel Tool to design/check the YL shear plate connection. User will need to input the frame geometry, member sizes, link sizes and loading for each frame and use Step 4 in the YL Excel Tool to check/design the shear plate. Figures below shows a 2 story x 2 bay frame elevation from RAM and the input screen from the Yield-Link Moment Connection Excel Tool for Step 4. Please note that the user can overwrite column I in the Excel sheet (Pu_sp) with their own calculated Collector/Chore axial forces. To finish the shear plate design the user can adjust the values in columns N thru R until all the DCR checks are adequate.



F	G	H	I	J	K	L	M	N	O	P	Q	R	S	T
4. SHEAR PLATE CHECK SUMMARY														
Beam Size	Beam Bay Span (in.)	Link Size	Pu_sp (kips)	V_g_LC08 (kips)	$V_u=2M_{pr}/L_h+V_g$ (kips)	V_g_LC01-07 (kips)	No. vert. bolts	No. horz. bolts	Bolt Size (in.)	Bolt Type	SP Plate Thickness (in.)	No. of SP	Weld Size (in.)	Beam Web DCR
W18X76	360	YL6-4	27.75	1.63	28.61	1.5	3	2	7/8	A325-N	3/8	1	4/16	0.417
W18X76	360	YL6-4	27.75	1.63	28.61	1.5	3	2	7/8	A325-N	3/8	1	4/16	0.417
W24X104	360	YL8-4	28.23	2.23	50.02	2.1	5	2	7/8	A325-N	3/8	1	4/16	0.360
W24X104	360	YL8-4	28.23	2.23	50.02	2.1	5	2	7/8	A325-N	3/8	1	4/16	0.360

APPENDIX A6: YIELD-LINK MOMENT CONNECTION INITIAL BEAM-COL MATCHES

Link Model	Initial Target Beam Size for YL Connection									
	W12X	W14X	W16X	W18X	W21X	W24X	W27X	W30X	W33X	W36X
YL4-2	30	34	36	35	-	-	-	-	-	-
YL4-2.5	30	34	36	35	-	-	-	-	-	-
YL4-3	30	34	36	50	-	-	-	-	-	-
YL4-2.25	30	34	36	50	-	-	-	-	-	-
YL4-2.875	40	34	36	50	-	-	-	-	-	-
YL4-3.5	40	43	67	50	-	-	-	-	-	-
YL4-3.75	40	43	67	71	-	-	-	-	-	-
YL4-4	40	43	67	76	-	-	-	-	-	-
YL4-2.25-10	-	-	-	-	44	55	84	-	-	-
YL4-2.875-10	-	-	-	-	48	55	84	-	-	-
YL4-3.5-10	-	-	-	-	48	68	84	-	-	-
YL4-3.75-10	-	-	-	-	48	68	84	-	-	-
YL4-4-10	-	-	-	-	48	68	84	-	-	-
YL6-2.5	-	-	36	50	44	55	84	-	-	-
YL6-3	-	-	67	50	48	68	84	-	-	-
YL6-3.5	-	-	67	76	48	68	84	-	-	-
YL6-4	-	-	67	76	68	68	84	-	-	-
YL6-4.5	-	-	67	76	101	104	84	-	-	-
YL6-5	-	-	67	76	101	104	84	-	-	-
YL6-5.5	-	-	67	76	101	104	84	-	-	-
YL6-6	-	-	77	76	101	104	146	-	-	-
YL6-3-13	-	-	-	-	-	-	-	90	118	135
YL6-3.5-13	-	-	-	-	-	-	-	90	118	135
YL6-4-13	-	-	-	-	-	-	-	90	118	135
YL6-4.5-13	-	-	-	-	-	-	-	90	118	135
YL6-5-13	-	-	-	-	-	-	-	90	118	135
YL6-5.5-13	-	-	-	-	-	-	-	90	118	135
YL6-6-13	-	-	-	-	-	-	-	90	118	135
YL8-4	-	-	-	-	-	68	84	90	118	135
YL8-4.5	-	-	-	-	-	84	84	90	118	135
YL8-5	-	-	-	-	-	104	84	90	118	135
YL8-5.5	-	-	-	-	-	104	94	99	118	135
YL8-6	-	-	-	-	-	104	146	108	118	135
YL8-4-15	-	-	-	-	-	-	-	90	118	135
YL8-4.5-15	-	-	-	-	-	-	-	90	118	135
YL8-5-15	-	-	-	-	-	-	-	90	118	135
YL8-5.5-15	-	-	-	-	-	-	-	99	118	135
YL8-6-15	-	-	-	-	-	-	-	108	118	135

Link Model	Initial Target Column Size for YL connection											
	W8X	W10X	W12X	W14X	W16X	W18X	W21X	W24X	W27X	W30X	W33X	W36X
YL4-2	31	33	30	34	36	40	44	55	84	90	118	135
YL4-2.5	35	39	40	38	40	40	50	55	84	90	118	135
YL4-3	35	39	40	38	45	46	50	62	84	90	118	135
YL4-2.25	31	33	40	34	40	50	48	55	84	90	118	135
YL4-2.875	35	39	40	38	40	50	55	55	84	90	118	135
YL4-3.5	40	39	45	43	67	50	55	68	84	90	118	135
YL4-3.75	40	45	45	48	67	50	62	68	84	90	118	135
YL4-4	40	45	45	48	67	55	62	68	84	90	118	135
YL4-2.25-10	31	33	40	34	40	50	48	55	84	90	118	135
YL4-2.875-10	35	39	40	38	40	50	55	55	84	90	118	135
YL4-3.5-10	40	39	45	43	67	50	55	68	84	90	118	135
YL4-3.75-10	40	45	45	48	67	50	62	68	84	90	118	135
YL4-4-10	40	45	45	48	67	55	62	68	84	90	118	135
YL6-2.5	48	54	58	53	57	60	68	76	84	90	118	135
YL6-3	-	60	72	68	77	65	73	84	94	99	118	135
YL6-3.5	-	68	79	74	77	71	83	84	94	108	118	135
YL6-4	-	-	79	74	89	86	101	94	102	116	130	135
YL6-4.5	-	-	87	82	89	97	101	117	102	116	130	135
YL6-5	-	-	96	99	89	97	111	117	114	124	130	150
YL6-5.5	-	-	96	109	100	106	111	117	114	124	141	150
YL6-6	-	-	96	109	-	106	122	131	129	132	141	150
YL6-3-13	-	60	72	68	77	65	73	84	94	99	118	135
YL6-3.5-13	-	68	79	74	77	71	83	84	94	108	118	135
YL6-4-13	-	-	79	74	89	86	101	94	102	116	130	135
YL6-4.5-13	-	-	87	82	89	97	101	117	102	116	130	135
YL6-5-13	-	-	96	99	89	97	111	117	114	124	130	150
YL6-5.5-13	-	-	96	109	100	106	111	117	114	124	141	150
YL6-6-13	-	-	96	109	-	106	122	131	129	132	141	150
YL8-4	-	-	106	109	100	106	122	103	129	132	141	150
YL8-4.5	-	-	106	120	-	119	122	131	129	132	152	160
YL8-5	-	-	120	132	-	119	132	146	129	148	152	160
YL8-5.5	-	-	252	132	-	-	147	146	161	148	169	170
YL8-6	-	-	252	145	-	-	147	146	161	148	169	182
YL8-4-15	-	-	106	109	100	106	122	103	129	132	141	150
YL8-4.5-15	-	-	106	120	-	119	122	131	129	132	152	160
YL8-5-15	-	-	120	132	-	119	132	146	129	148	152	160
YL8-5.5-15	-	-	252	132	-	-	147	146	161	148	169	170
YL8-6-15	-	-	252	145	-	-	147	146	161	148	169	182

APPENDIX A7: DESIGN EXAMPLE-DEMO MODELS

Appendix 7A

YLMC EXCEL Tool Demo

Date: 5-12-2022

Simpson Strong-Tie Yield-Link
Moment Connection Calculations

Project Name:	Sample Calc
Frame Elevation ID:	DMO
Project Address:	5956 W Las Positas Blvd Pleasanton, CA - 94588
Design Firm:	Simpson Strong-Tie
Design Engineer:	Simpson Strong-Tie
Designed for:	Simpson Strong-Tie
Design Date:	5/12/2022
Tool Version:	3.3.0

User Input for Various Design Limit States:

Initial Link Check:

	DCR Limit
tbf Check:	1.00
bf Check:	1.00
Lyield Check:	1.00
Panel Zone DCR:	1.00
Drift DCR:	1.00

Beam and Link Check (BLC Details):

	DCR Limit
Beam bf, tbf_DCR:	1.04
Link strength DCR:	1.00
Lyield Check:	1.00
tbrp_DCR:	1.00
brp_Bolt_DCR:	1.00
Link slip DCR:	1.00

Column Check (BLC Details):

	DCR Limit
SCWL_DCR:	1.00
DCR_PZ:	1.00
Stiffener DCR:	1.00
Column Flange DCR:	1.00

Shear Plate Check (SPC Details):

	DCR Limit
Beam Web DCR:	1.00
Shear Plate DCR:	1.00
Bolt DCR:	1.00
Fillet Weld DCR:	1.00

User Input for Material Properties:

Beam:

Fy=	50	ksi
Fu=	65	ksi

Column:

Fy=	50	ksi
Fu=	65	ksi

Stiffener Plate:

Fy=	50	ksi
Fu=	65	ksi
tstp_min=	0.375	in.
Bot. Stiffener Depth (1-sided connection)=	Full Depth	

Shear Plate:

Fy=	50	ksi
Fu=	65	ksi

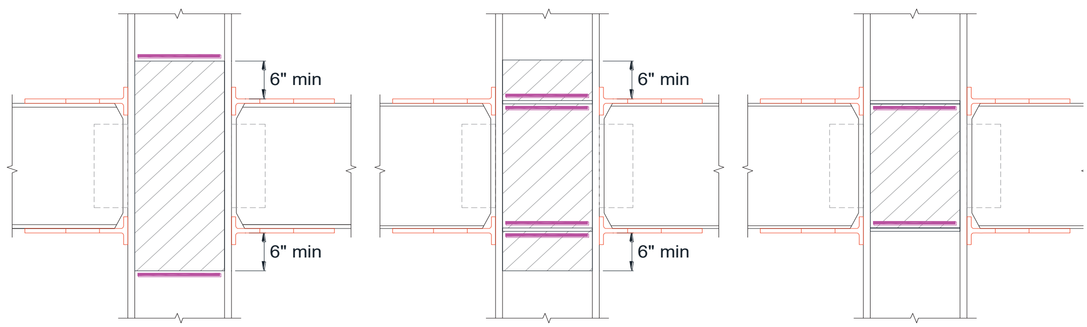
Doubler Plate:

Fy=	50	ksi
Fu=	65	ksi
tdp_min=	0.375	in.

Column Bracing at Beam Bot Flange (Yes/NO): YES

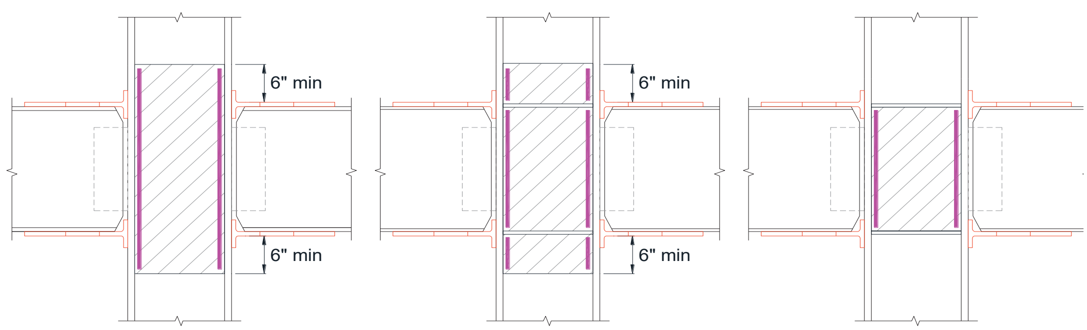
User Input for Connection Welding Perferences:

Doubler PL to col web/Cont plate Weld: Option_2B | Doubler placed between continuity plates



Option 1 Option 2A Option 2B

Doubler PL to col flange weld: Option_1 | Fillet Weld



Use of Plug-weld for Doubler Plates? NO

Equivalent Lateral Force Method (RISA-3D/SAP2000 Load Combinations)

	Design Parameters
f1=	0.5
f2=	0.7
ρ=	1.0
Ω=	3.0
S_{DS}=	1.000

Total LC= 36

ID	Load Combinations	Load Multipliers										Design Check					YL
		DL	LL	LR	SL	RL	WL	NL	EL_D	EL		Link Strength	Seismic Drift	Beam + Col. (P+M)	V_bm Gravity	Wind Defl.	Excel Tool ID
SST_C1	DL	1	0	0	0	0	0	0	0	0	Load Case						
SST_C2	LL	0	1	0	0	0	0	0	0	0							
SST_C3	LR	0	0	1	0	0	0	0	0	0							
SST_C4	SL	0	0	0	1	0	0	0	0	0							
SST_C5	RL	0	0	0	0	1	0	0	0	0							
SST_C6	WL	0	0	0	0	0	1	0	0	0							
SST_C7	NL	0	0	0	0	0	0	1	0	0							
SST_C8	EL_D	0	0	0	0	0	0	0	1	0							
SST_C9	EL	0	0	0	0	0	0	0	0	1							
SST_LC01	1.4 DL + NL	1.4	0	0	0	0	0	1	0	0	Gravity	x		x	x		
SST_LC02	1.2 DL + 1.6 LL + 0.5 LR + NL	1.2	1.6	0.5	0	0	0	1	0	0		x		x	x		4
SST_LC03	1.2 DL + 1.6 LL + 0.5 SL + NL	1.2	1.6	0	0.5	0	0	1	0	0		x		x	x		
SST_LC04	1.2 DL + 1.6 LL + 0.5 RL + NL	1.2	1.6	0	0	0.5	0	1	0	0		x		x	x		
SST_LC05	1.2 DL + 1.6 LR + f1 LL + NL	1.2	0.5	1.6	0	0	0	1	0	0		x		x	x		
SST_LC06	1.2 DL + 1.6 SL + f1 LL + NL	1.2	0.5	0	1.6	0	0	1	0	0		x		x	x		
SST_LC07	1.2 DL + 1.6 RL + f1 LL + NL	1.2	0.5	0	0	1.6	0	1	0	0		x		x	x		
SST_LC08	(1.2 + 0.2*SDS)DL + f1*LL + f2*SL	1.4	0.5	0.5	0.7	0.5	0	0	0	0		x		x	x		3
SST_LC09	1.2 DL + 1.6 LR + 0.5 WL	1.2	0	1.6	0	0	0.5	0	0	0	Wind	x		x			11
SST_LC10	1.2 DL + 1.6 LR - 0.5 WL	1.2	0	1.6	0	0	-0.5	0	0	0		x		x			12
SST_LC11	1.2 DL + 1.6 SL + 0.5 WL	1.2	0	0	1.6	0	0.5	0	0	0		x		x			
SST_LC12	1.2 DL + 1.6 SL - 0.5 WL	1.2	0	0	1.6	0	-0.5	0	0	0		x		x			
SST_LC13	1.2 DL + 1.6 RL + 0.5 WL	1.2	0	0	0	1.6	0.5	0	0	0		x		x			
SST_LC14	1.2 DL + 1.6 RL - 0.5 WL	1.2	0	0	0	1.6	-0.5	0	0	0		x		x			
SST_LC15	1.2 DL + 1.0 WL + f1 LL + 0.5 LR	1.2	0.5	0.5	0	0	1	0	0	0		x		x			9
SST_LC16	1.2 DL - 1.0 WL + f1 LL + 0.5 LR	1.2	0.5	0.5	0	0	-1	0	0	0		x		x			10
SST_LC17	1.2 DL + 1.0 WL + f1 LL + 0.5 SL	1.2	0.5	0	0.5	0	1	0	0	0		x		x			
SST_LC18	1.2 DL - 1.0 WL + f1 LL + 0.5 SL	1.2	0.5	0	0.5	0	-1	0	0	0		x		x			
SST_LC19	1.2 DL + 1.0 WL + f1 LL + 0.5 RL	1.2	0.5	0	0	0.5	1	0	0	0		x		x			
SST_LC20	1.2 DL - 1.0 WL + f1 LL + 0.5 RL	1.2	0.5	0	0	0.5	-1	0	0	0		x		x			
SST_LC21	0.9 DL + 1.0 WL	0.9	0	0	0	0	1	0	0	0		x		x			
SST_LC22	0.9 DL - 1.0 WL	0.9	0	0	0	0	-1	0	0	0		x		x			
SST_LC23	1.0 DL + 0.5 LL + 0.5 LR + 0.7 WL	1.0	0.5	0.5	0	0	0.7	0	0	0						x	13
SST_LC24	1.0 DL + 0.5 LL + 0.5 LR - 0.7 WL	1.0	0.5	0.5	0	0	-0.7	0	0	0						x	14
SST_LC25	(1.2 + 0.2*SDS)DL + EL*rho + f1*LL + f2*SL	1.4	0.5	0.5	0.7	0	0	0	0	1.0	Seismic	x					5
SST_LC26	(1.2 + 0.2 SDS)DL - EL*rho + f1*LL + f2* SL	1.4	0.5	0.5	0.7	0	0	0	0	-1.0		x					6
SST_LC27	(0.9 - 0.2 SDS) DL + EL*rho	0.7	0	0	0	0	0	0	0	1.0		x					
SST_LC28	(0.9 - 0.2 SDS) DL - EL*rho	0.7	0	0	0	0	0	0	0	-1.0		x					
SST_LC29	(1.2 + 0.2*SDS)DL + EL_D+ f1*LL + f2*SL	1.4	0.5	0.5	0.7	0	0	0	1.0	0			x				1
SST_LC30	(1.2 + 0.2 SDS)DL - EL_D+ f1*LL + f2* SL	1.4	0.5	0.5	0.7	0	0	0	-1.0	0			x				2
SST_LC31	(0.9 - 0.2 SDS) DL + EL_D	0.7	0	0	0	0	0	0	1.0	0			x				
SST_LC32	(0.9 - 0.2 SDS) DL - EL_D	0.7	0	0	0	0	0	0	-1.0	0			x				
SST_LC33	(1.2 + 0.2 SDS)DL + OmegaEL + f1*LL + f2*SL	1.4	0.5	0.5	0.7	0	0	0	0	3.0	Ω			x			7
SST_LC34	(1.2 + 0.2 SDS)DL -OmegaEL + f1*LL + f2*SL	1.4	0.5	0.5	0.7	0	0	0	0	-3.0				x			8
SST_LC35	(0.9 - 0.2 SDS) DL + OmegaEL	0.7	0	0	0	0	0	0	0	3.0				x			
SST_LC36	(0.9 - 0.2 SDS) DL - OmegaEL	0.7	0	0	0	0	0	0	0	-3.0				x			

Job Name:	Sample Calc	--> *Please save an Excel File for EACH frame elevation*
Frame Elevation ID:	DMO	
File Path:	C:\Users\akulkarni\Desktop\2022 Goals\01 - YLMC Tools & Plugins_Pa	
sub path:	C:\Users\akulkarni\Desktop\2022 Goals\01 - YLMC Tools & Plugins_Pa	
file=	C:\Users\akulkarni\Desktop\2022 Goals\01 - YLMC Tools & Plugins_Pa	

Typ. Roof Depth:	3.500 in.
Typ. Floor Depth:	5.500 in.
Number of Stories	2
Typical Story Height:	12 ft
Base=	Pinned
Extension below Base=	0.000 in.

HCAI/DSA?	YES
R=	8
Cd=	5.5
Ie=	1
Omega=	3
Rho=	1
Δa/ρ?	NO
Number of Bays:	2
Typical Bay Span:	30 ft

Base=	Pinned
Extension below Base=	0.000 in.

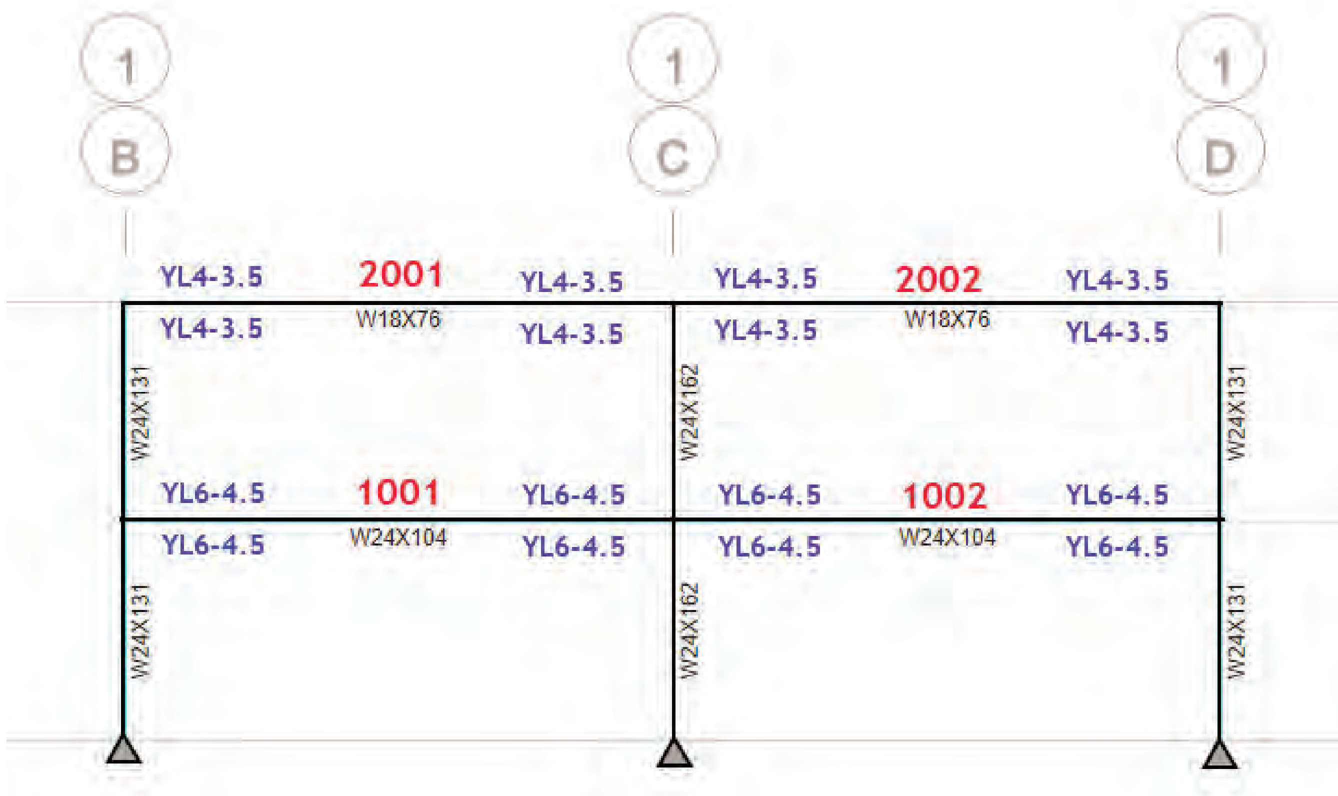
Basic Gravity Loads:

wDL_roof=	0.229	klf	Typical uniform dead load on ROOF/UPPER LEVEL beams
wLL_roof=	0.072	klf	Typical uniform live load on ROOF/UPPER LEVEL beams
wDL_floor=	0.454	klf	Typical uniform dead load on FLOOR beams
wLL_floor=	0.359	klf	Typical uniform live load on FLOOR beams
S_{DS}=	1.000		Short period design spectral response acceleration parameter

OTHER Loads:	Left End	Right End	
wSL_roof:	0	0	klf, Typical uniform/Trapezoidal SNOW load (SL) on ROOF beams
wRL_roof:	0	0	klf, Typical uniform/Trapezoidal RAIN load (RL) on ROOF beams

vW_roof=	0	klf	Typical uniform wind load (W) on Roof beams (Vertical Load) [neg.
f1=	0.5		Live Load factor, See IBC 2018, 1605.2
f2=	0.7		Snow Load factor, See IBC 2018, 1605.2

Frame Geometry												
Elev ID	Grid ID	Story	Story Height (in.)	Left Beam Size	Left Beam Bay Span (in.)	Column size	Fixed Base Col width-to-thickness DCR	Beam ID	Beam Size	Link Size	Assign Link at I_End	Assign Link at J_End
DMO	1	Story2	144	N/A	N/A	W24X131	N/A	N/A	N/A			
DMO	2	Story2	144	W18X76	360	W24X162	N/A	2001	W18X76	YL4-3.5	YES	YES
DMO	3	Story2	144	W18X76	360	W24X131	N/A	2002	W18X76	YL4-3.5	YES	YES
DMO	1	Story1	144	N/A	N/A	W24X131	N/A	N/A	N/A			
DMO	2	Story1	144	W24X104	360	W24X162	N/A	1001	W24X104	YL6-4.5	YES	YES
DMO	3	Story1	144	W24X104	360	W24X131	N/A	1002	W24X104	YL6-4.5	YES	YES



1. INITIAL YIELD-LINK SELECTION SUMMARY																	
Elev ID	Grid ID	Story	$M_{u\text{-link}}$ (kip-in.)	$V_{gravity}$ (kips)	$P_{u\text{-sp}}$ (kips)	Initial t_{bf} check	Initial b_{bf} check	Initial b_{cf} check	Initial $L_{y\text{-link}}$ check	Link Strength DCR	PZ I-End Col. DCR	PZ J-End Col. DCR	K_rot (kip-in./rad)	phi * Mn (kip-in.)	M_{cap_link} (kip-in.)	Beam Slope Condition (x" per foot)
DMO	1	Story2				N/A	N/A	N/A	N/A	N/A	N/A	N/A	#VALUE!	#N/A	#N/A	
DMO	2	Story2	1008	7.1	31.0	0.59	0.68	0.68	0.93	0.68	0.34	0.57	7.80E+05	1473	2553	0.00
DMO	3	Story2	1008	7.1	31.0	0.59	0.68	0.66	0.93	0.68	0.57	0.34	7.80E+05	1473	2553	0.00
DMO	1	Story1				N/A	N/A	N/A	N/A	N/A	N/A	N/A	#VALUE!	#N/A	#N/A	
DMO	2	Story1	2477	14.7	24.6	0.53	0.72	0.72	0.88	0.66	0.66	1.11	1.79E+06	3774	6542	0.00
DMO	3	Story1	2484	14.7	24.6	0.53	0.72	0.72	0.88	0.66	1.11	0.66	1.79E+06	3774	6542	0.00

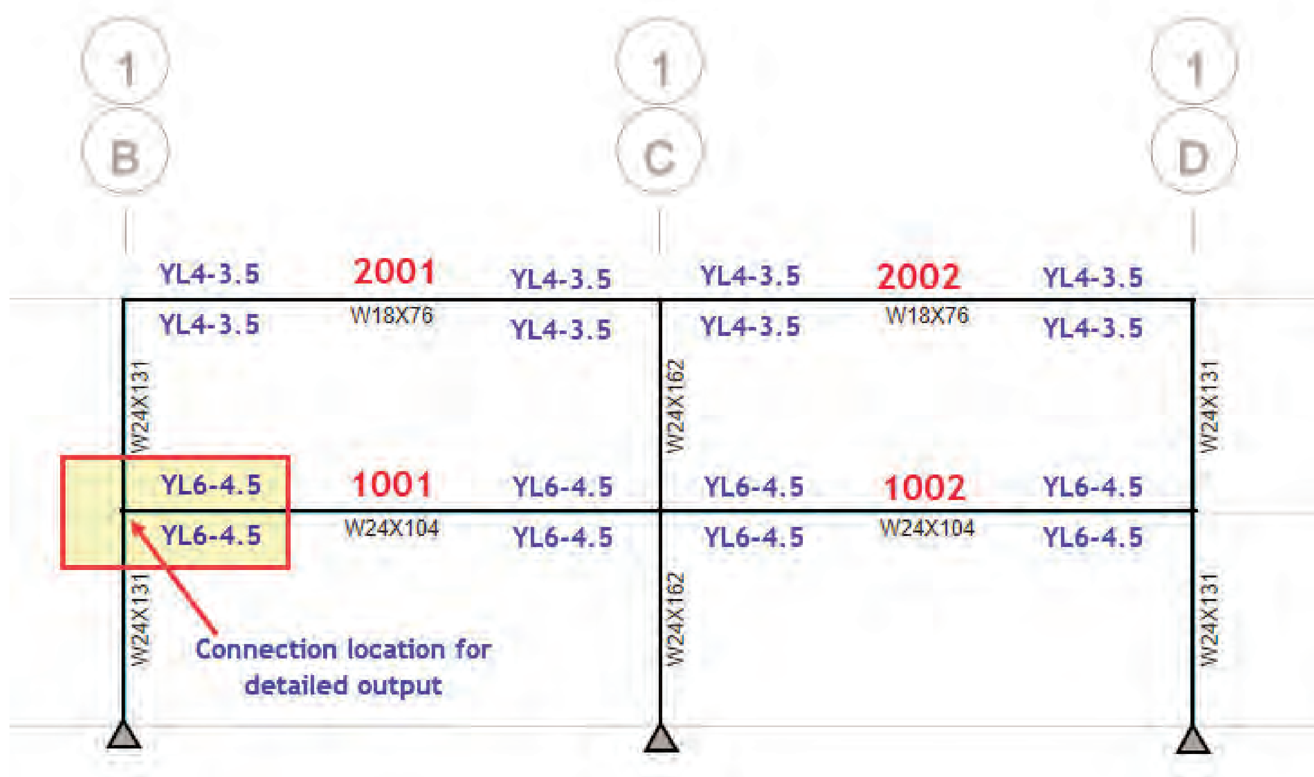
Column Vertical Point Loads (DL, LL/LR, SL)								Beam Vertical Point Load (Dead)											Beam Vertical Point Loads (Live/Roof Live)								Beam Vertical Point Loads (Snow)							
Elev ID	Grid ID	Story	Column ID	Column Size	Column DL kips	Column LL kips	Column SL kips	Beam ID	Beam Size	Beam Span (in.)	P_{DL1} (kips)	DL_X1 (in.)	P_{DL2} (kips)	DL_X2 (in.)	P_{DL3} (kips)	DL_X3 (in.)	P_{DL4} (kips)	DL_X4 (in.)	P_{LL1} (kips)	LL_X1 (in.)	P_{LL2} (kips)	LL_X2 (in.)	P_{LL3} (kips)	LL_X3 (in.)	P_{LL4} (kips)	LL_X4 (in.)	P_{SL1} (kips)	SL_X1 (in.)	P_{SL2} (kips)	SL_X2 (in.)	P_{SL3} (kips)	SL_X3 (in.)	P_{SL4} (kips)	SL_X4 (in.)
DMO	1	Story2	4	W24X131	10.6	6.2																												
DMO	2	Story2	5	W24X162	7.6	5.1		2001	W18X76	360																								
DMO	3	Story2	6	W24X131	10.6	6.2		2002	W18X76	360																								
DMO	1	Story1	1	W24X131	23.4	31.3																												
DMO	2	Story1	2	W24X162	17.5	25.8		1001	W24X104	360																								
DMO	3	Story1	3	W24X131	23.1	30.8		1002	W24X104	360																								

1B. LATERAL LOADS AND DEFLECTION PARAMETERS													
Elev ID	Story	Column Location ID	Story Height (in.)	Total Frame Width (in.)	Fx Strength (kips)	Fx Drift (kips)	Ni Strength (kips)	Allowable Drift Limit	C_d	I_e	Pi (kips)	F_wind (kips)	Allowable Drift Limit
DMO	Story2	DMO-1-Story2	144	720	32.3	32.3	1.5	0.025 Hx	5.5	1	1	3.9	hx/500
DMO	Story1	DMO-1-Story1	144	720	21.5	21.5	1.0	0.025 Hx	5.5	1	1	3.6	hx/500

2. DRIFT CHECK SUMMARY														
Elev ID	**Grid ID**	**Story**	**Column Location ID**	**Story Height (in.)**	**Fx_Drift (kips)**	**Drift Node ID**	**Frame Seismic Displacement (in.) [LC 1&2]**	**δ_{xe} (in.)**	**C_d**	**I_e**	**δ_x (in.)**	**Allowable Drift Limit**	**Allowable Drift (in.)**	**Drift DCR**
DMO	1	Story2	DMO-1-Story2	144	32.33	3	0.858	0.411	5.5	1	2.259	0.025 Hx	3.60	0.627
DMO	1	Story1	DMO-1-Story1	144	21.51	2	0.448	0.448	5.5	1	2.462	0.025 Hx	3.60	0.684

					2a. Stability Check for P-delta Effects									2b. Wind deflection						
Elev ID	Grid ID	Story	Column Location ID	Story Height (in.)	Pi (kips)	P_x (kips)	Fx (kips)	Vx (kips)	hsx (in.)	Stability Coeff, θ	β (max Link DCR)	θmax	Stability θ, DCR	F_wind (kips)	Drift Node ID	Frame Wind Displacement (in.) [LC 9&10]	Drift (in.)	Allowable Drift Limit	Allowable Drift (in.)	Wind Drift DCR
DMO	1	Story2	DMO-1-Story2	144	1	1	32.33	32	144	0.0001	1	0.09	0.001	3.91	3	0.081	0.039	hx/500	0.29	0.134
DMO	1	Story1	DMO-1-Story1	144	1	2	21.51	54	144	0.0001	1	0.09	0.001	3.64	2	0.043	0.043	hx/500	0.29	0.148

3. BEAM AND YIELD-LINK CHECK SUMMARY														
Elev ID	Grid ID	Story	Beam ID	Beam Size	Link Size	$M_{u\text{-link}}$ (kip-in.)	BRP Size	t_{BRP} DCR	Beam t_f, b_f DCR	BRP Bolt DCR	Link Strength DCR	Link Slip DCR	$L_{y\text{-link}}$ DCR	Beam b/t	Beam h/tw
DMO	1	Story2	2001	W18X76	YL4-3.5	1008	BRP4B	0.686	0.588	0.954	0.684	0.541	0.929	Compact	Compact
DMO	2	Story2	2002	W18X76	YL4-3.5	1008	BRP4B	0.686	0.588	0.954	0.684	0.541	0.929	Compact	Compact
DMO	1	Story1	1001	W24X104	YL6-4.5	2477	BRP6B	0.774	0.636	0.513	0.656	0.481	0.881	Compact	Compact
DMO	2	Story1	1002	W24X104	YL6-4.5	2484	BRP6B	0.774	0.636	0.513	0.658	0.482	0.881	Compact	Compact



4. SHEAR PLATE CHECK SUMMARY

Elev ID	Grid ID	Story	Beam ID	Beam Size	Beam Bay Span (in.)	Link Size	P_{u_sp} (kips)	V_{g_LC08} (kips)	V_u=2M_{pr}/L_h+ V_g (kips)	$V_{g\_LC01-07}$ (kips)	No. vert. bolts	No. horz. bolts	Bolt Size (in.)	Bolt Type	SP Plate Thickness (in.)	No. of SP	Weld Size (in.)	Beam Web DCR	Shear Plate Geometry Check	Shear Plate DCR	Bolt Shear DCR	Fillet Weld DCR
DMO	1	Story2	2001	W18X76	360	YL4-3.5	30.98	7.07	22.55	6.8	3	2	7/8	A325-X	1/2	1	5/16	0.479	OK	0.456	0.561	0.196
DMO	2	Story2	2002	W18X76	360	YL4-3.5	30.98	7.07	22.55	6.8	3	2	7/8	A325-X	1/2	1	5/16	0.479	OK	0.456	0.561	0.196
DMO	1	Story1	1001	W24X104	360	YL6-4.5	24.61	14.71	54.57	17.7	5	2	7/8	A325-X	1/2	1	5/16	0.306	OK	0.496	0.536	0.307
DMO	2	Story1	1002	W24X104	360	YL6-4.5	24.61	14.71	54.57	17.8	5	2	7/8	A325-X	1/2	1	5/16	0.306	OK	0.496	0.536	0.307

		5. COLUMN CHECK SUMMARY																
Elev ID	Grid ID	Story	Column Location	Story Height (in.)	Column size	Pu Column (kips)	Bottom Stiffener Provided?	Doubler Plate Provided?	Stiffener Required for AISC 360 J10?	Min. Stiffener Thickness (in.)	Min. Doubler Thickness (in.)	SCWB DCR Check	Column Pz DCR Check	Column Flange Check	Min. Stiff to flange fillet size	Min. Stiff to web fillet size	Column b/t	Column h/tw
DMO	1	Story2	DMO-1-Story2	144	W24X131	40.3	YES	NO	NO	4/8	0	N/A	0.724	0.564	3/16	3/16	Compact	Compact
DMO	2	Story2	DMO-2-Story2	144	W24X162	30.1	YES	NO	NO	4/8	0	N/A	0.621	0.443	3/16	3/16	Compact	Compact
DMO	3	Story2	DMO-3-Story2	144	W24X131	40.4	YES	NO	NO	4/8	0	N/A	0.724	0.564	3/16	3/16	Compact	Compact
DMO	1	Story1	DMO-1-Story1	144	W24X131	139.9	YES	NO	NO	4/8	0	0.216	0.830	0.751	3/16	3/16	Compact	Compact
DMO	2	Story1	DMO-2-Story1	144	W24X162	128.8	YES	NO	NO	4/8	0	0.335	0.915	0.589	3/16	3/16	Compact	Compact
DMO	3	Story1	DMO-3-Story1	144	W24X131	139.2	YES	NO	NO	4/8	0	0.216	0.830	0.751	3/16	3/16	Compact	Compact

Col.	Elev.	Grid	Story	Column	Left Shear PL		Right Shear PL		Stiff. PL	Doubler PL	1A. Left ST to col flange Weld				1B. Righ ST to col flange Weld				2. STP to col web/DP weld		3. STP to col flange weld		4. DP to col web Weld		A. DP to col web Plug Wel		5. DP to col flange weld	
ID	ID	Id	ID	Size	no. SP	thickness, (in.)	no. SP	thickness, (in.)	thickness, (in.	thickness, (in.	Fillet Size, (in.)	# of sides	PJP Size, (in.)	# of sides	Fillet Size, (in.)	# of sides	PJP Size, (in.)	# of sides	Fillet Size, (in.)	# of sides	Fillet Size, (in.)	# of sides	Fillet Size, (in.)	# of sides	Diameter, (in.	Depth, (in.	Fillet Size, (in.)	# of sides
4	DMO	1	Story2	W24X131	N/A	N/A	1	4/8	4/8	N/A	N/A	N/A	N/A	N/A	5/16	2	N/A	N/A	3/16	2	3/16	2	N/A	N/A	N/A	N/A	N/A	N/A
5	DMO	2	Story2	W24X162	1	4/8	1	4/8	4/8	N/A	5/16	2	N/A	N/A	5/16	2	N/A	N/A	3/16	2	3/16	2	N/A	N/A	N/A	N/A	N/A	N/A
6	DMO	3	Story2	W24X131	1	4/8	N/A	N/A	4/8	N/A	5/16	2	N/A	N/A	N/A	N/A	N/A	N/A	3/16	2	3/16	2	N/A	N/A	N/A	N/A	N/A	N/A
1	DMO	1	Story1	W24X131	N/A	N/A	1	4/8	4/8	N/A	N/A	N/A	N/A	N/A	5/16	2	N/A	N/A	3/16	2	3/16	2	N/A	N/A	N/A	N/A	N/A	N/A
2	DMO	2	Story1	W24X162	1	4/8	1	4/8	4/8	N/A	5/16	2	N/A	N/A	5/16	2	N/A	N/A	3/16	2	3/16	2	N/A	N/A	N/A	N/A	N/A	N/A
3	DMO	3	Story1	W24X131	1	4/8	N/A	N/A	4/8	N/A	5/16	2	N/A	N/A	N/A	N/A	N/A	N/A	3/16	2	3/16	2	N/A	N/A	N/A	N/A	N/A	N/A

7. Preliminary Beam Design Summary

Elev ID	Grid ID	Story	Beam ID	Beam Size	Link Size	Beam Bay Span (in.)	Lb (in.)	Beam Interm. Bracing	Mcap_link (k-in)	P_{u_sp} (kips)	Mu Omega (k-in)	V_u (kips)	Mcap_Link/ Mu_Omega	Axial DCR	Flexural DCR	P+M DCR	V DCR	Adj. Flexure DCR	Total P+M DCR
DMO	1	Story2	2001	W18X76	YL4-3.5	360	330.25	None	2553	31.0	2749	20	0.928	0.098	0.378	0.425	0.098	0.351	0.400
DMO	2	Story2	2002	W18X76	YL4-3.5	360	330.25	None	2553	31.0	2749	20	0.928	0.098	0.378	0.425	0.098	0.351	0.400
DMO	1	Story1	1001	W24X104	YL6-4.5	360	330.25	None	6542	24.6	7063	50	0.926	0.046	0.544	0.564	0.153	0.505	0.527
DMO	2	Story1	1002	W24X104	YL6-4.5	360	330.25	None	6542	24.6	7063	50	0.926	0.046	0.544	0.564	0.153	0.505	0.527

			7B. Beam Design Per Analyais Software (i.e. RISA 3D) with Mpr_Link Check													
Elev ID	Grid ID	Story	Beam ID	Pu Omega (kips)	Mu Omega (k-in)	Vu Omega (k-in)	Phi*Pn (kips)	Phi*Mn (kip-in)	Phi*Vn (kips)	Mcap_link (kip-in)	M_{cap_link}/ Phi*Mn	Pu_omega/ Phi*Pn	Mu_omega/ Phi*Mn	M_{cap_link}/ M_{u_omega}	Vu DCR	P+M DCR	Adj P+M DCR
DMO	1	Story2	2001	31.0	2749.2	20.45	314.7	7335.0	208.8	2553	0.348	0.098	0.375	0.928	0.098	0.424	0.397
DMO	2	Story2	2002	31.0	2749.2	20.45	314.7	7335.0	208.8	2553	0.348	0.098	0.375	0.928	0.098	0.424	0.397
DMO	1	Story1	1001	24.6	7063.0	49.76	538.5	13005.0	325.4	6542	0.503	0.046	0.543	0.926	0.153	0.566	0.526
DMO	2	Story1	1002	24.6	7063.0	49.76	538.5	13005.0	325.4	6542	0.503	0.046	0.543	0.926	0.153	0.566	0.526

8. Preliminary Column Design Summary																	
Col ID	Elev ID	Grid ID	Story	Column Location	Story Height (in.)	Column size	Col Bracing at Bm Bot Flg?	Cb	Pu_Ω Column (kips)	Vu_Ω Column (kips)	Mu_Ω(top) Column (kip-in)	Mu_Ω(bot.) Column (kip-in)	Mu_Ω(max) Column (kip-in)	Axial DCR	Flexural DCR	P+M DCR	V DCR
4	DMO	1	Story2	DMO-1-Story2	144	W24X131	YES	2.17	40.3	26.0	2749	1127	2749	0.028	0.165	0.179	0.065
5	DMO	2	Story2	DMO-2-Story2	144	W24X162	YES	2.22	30.1	53.4	4758	3203	4758	0.016	0.226	0.234	0.112
6	DMO	3	Story2	DMO-3-Story2	144	W24X131	YES	2.17	40.4	26.0	2749	1127	2749	0.028	0.165	0.179	0.065
1	DMO	1	Story1	DMO-1-Story1	144	W24X131	YES	1.66	139.9	46.9	5936	0	5936	0.096	0.357	0.405	0.117
2	DMO	2	Story1	DMO-2-Story1	144	W24X162	YES	1.66	128.8	72.5	9162	0	9162	0.070	0.436	0.463	0.152
3	DMO	3	Story1	DMO-3-Story1	144	W24X131	YES	1.66	139.2	46.9	5936	0	5936	0.095	0.357	0.405	0.117

Base Reactions (Column Nodes from Left to Right)														
Combo ID	Load Combination	Node	Fx (kips)	Fy (kips)	M (k-in)	Node	Fx (kips)	Fy (kips)	M (k-in)	Node	Fx (kips)	Fy (kips)	M (k-in)	
SST_C1	DL	1	1.3	49.7	0.0	4	0.0	55.1	0.0	7	-1.3	49.4	0.0	
SST_C2	LL	1	0.9	36.6	0.0	4	0.0	36.8	0.0	7	-0.9	36.1	0.0	
SST_C3	LR	1	0.0	7.2	0.0	4	0.0	7.3	0.0	7	0.0	7.2	0.0	
SST_C4	SL	1	0.0	0.0	0.0	4	0.0	0.0	0.0	7	0.0	0.0	0.0	
SST_C5	RL	1	0.0	0.0	0.0	4	0.0	0.0	0.0	7	0.0	0.0	0.0	
SST_C6	WL	1	-2.1	-2.1	0.0	4	-3.4	0.0	0.0	7	-2.1	2.1	0.0	
SST_C7	NL	1	-0.7	-0.7	0.0	4	-1.1	0.0	0.0	7	-0.7	0.7	0.0	
SST_C8	EL_D	1	-14.8	-16.1	0.0	4	-24.2	0.0	0.0	7	-14.8	16.1	0.0	
SST_C9	EL	1	-14.8	-16.1	0.0	4	-24.2	0.0	0.0	7	-14.8	16.1	0.0	
SST_LC02	1.2 DL + 1.6 LL + 0.5 LR + NL	1	2.5	121.0	0.0	4	-1.1	128.8	0.0	7	-3.9	121.3	0.0	
SST_LC08	(1.2 + 0.2*SDS)DL + f1*LL + f2*SL	1	2.4	91.4	0.0	4	0.0	99.3	0.0	7	-2.4	90.8	0.0	
SST_LC13	1.2 DL + 1.6 RL + 0.5 WL	1	0.6	58.5	0.0	4	-1.7	66.2	0.0	7	-2.7	60.4	0.0	
SST_LC14	1.2 DL + 1.6 RL - 0.5 WL	1	2.7	60.7	0.0	4	1.7	66.2	0.0	7	-0.6	58.2	0.0	
SST_LC15	1.2 DL + 1.0 WL + f1 LL + 0.5 LR	1	0.0	79.4	0.0	4	-3.4	88.3	0.0	7	-4.2	83.0	0.0	
SST_LC16	1.2 DL - 1.0 WL + f1 LL + 0.5 LR	1	4.2	83.6	0.0	4	3.4	88.3	0.0	7	0.0	78.8	0.0	
SST_LC23	1.0 DL + 0.5 LL + 0.5 LR + 0.7 WL	1	0.3	70.1	0.0	4	-2.4	77.2	0.0	7	-3.3	72.6	0.0	
SST_LC24	1.0 DL + 0.5 LL + 0.5 LR - 0.7 WL	1	3.3	73.1	0.0	4	2.4	77.2	0.0	7	-0.3	69.6	0.0	
SST_LC25	(1.2 + 0.2*SDS)DL + EL*rho + f1*LL + f2*SL	1	-12.4	75.3	0.0	4	-24.2	99.3	0.0	7	-17.3	106.9	0.0	
SST_LC26	(1.2 + 0.2 SDS)DL - EL*rho + f1*LL + f2* SL	1	17.3	107.6	0.0	4	24.2	99.3	0.0	7	12.4	74.6	0.0	
SST_LC29	(1.2 + 0.2*SDS)DL + EL_D+ f1*LL + f2*SL	1	-12.5	75.3	0.0	4	-24.2	99.2	0.0	7	-17.2	107.0	0.0	
SST_LC30	(1.2 + 0.2 SDS)DL - EL_D+ f1*LL + f2* SL	1	17.2	107.6	0.0	4	24.2	99.2	0.0	7	12.5	74.7	0.0	
SST_LC33	(1.2 + 0.2 SDS)DL + OmegaEL + f1*LL + f2*SL	1	-42.1	43.0	0.0	4	-72.5	99.3	0.0	7	-46.9	139.2	0.0	
SST_LC34	(1.2 + 0.2 SDS)DL -OmegaEL + f1*LL + f2*SL	1	46.9	139.9	0.0	4	72.5	99.3	0.0	7	42.1	42.4	0.0	

3.1 CURRENT MEMBER:

Beam Unique Name:	1001	Link ID:	YL6-4.5	R=	8
Beam Size:	W24X104	Mu_max:	2477	kips.in	
Beam b/t	Compact	Beam h/tw:	Compact	(AISC 360 Table B4.1b)	

3.2 LINK STEM GEOMETRY: (From YL Database)

NY Length ColSide ($_{Lcol\_side}$)=	5.00	in	Thickness (t_{stem}) =	0.75	in
Yield Length, incl. fillets (L_{y_yield}) =	10.00	in	NY Width ColSide (b_{col_side})=	10.00	in
NY Length BeamSide (L_{bm_side})=	11.25	in	Central Neck Yield Width (b_{yield})=	4.50	in
L_stem=	26.25	in	NY Width BeamSide (b_{bm_side})=	10.00	in
Link Radius (r_link)=	0.75	in	Yielding Area (A_stemYield) =	3.38	in^2

3.3 LINK STEM BOLTS: (From YL Database)

Num. Bolts (n_bolt_linkBm)=	6.00		Gauge Along Width (g_{stem})=	6.00	in
Bolt Type (Bolt_Gr_linkBm)=	A490		Spacing Along Length (s_{stem}) =	3.38	in
Stem Bolt Dia=	1.25	in	First Bolt distance to Neck (S_c)=	2.50	in
Min. Bolt length =	3.125	in	Last Bolt distance to Edge (S_b) =	2.00	in

3.4 LINK FLANGE GEOMETRY: (From YL Database)

Thickness (t_{flange})=	1.25	in
Flange Width (b_{flange})=	10.00	in
Flange height (h_{flange}) =	9.25	in

3.5 LINK FLANGE BOLTS: (From YL Database)

Num. Bolts (n_bolt_linkCol) =	4		Gauge Along Width (vertical) (g_{flange})=	5.75	in
Bolt Type (Bolt_Gr_linkCol)=	A325		Spacing Along Length (horiz) (s_{flange})=	6.00	in
Bolt Dia (boltD_linkCol)_=	1.25	in			

3.6 LINK MATERIAL: (AISC 358, 12.8.2)

Fy_link =	50	ksi	Ry_link=	1.1
Fu_link =	65	ksi	Rt_link =	1.2

3.7 LINK CHECK: (AISC 358, 12.9, Step 3 and Step 6)

Limit Rotation (r_linkLimit) =	0.05	radians	*Connection rotation per 12.9 Step 6*
Limit Strain ε (e_linkLimit) =	8.50%		*Link strain limit per 12.9 Step 6*
Rotation Arm (d_arm)=	12.425	in	*= (db + t_stem)/2*
Link Extension (linkDelta)=	0.621	in	*=d_arm* r_linkLimit*
Lyield_req=	8.81	in	*= r_linkLimit* d_arm/ 0.085 + 2* r_link (EQ 12.9-4)*
Lyield=	10.00	in	*from previous value*
Lyield_DCR=	**0.88**		*= Lstem_yield_req/ L_stemYield*
Check=	OK		*OK, if L_stem_DCR < DCR_allowed*

Strength Check: Step 3

Mu=	2477	k-in	Moment Demand (LRDF combos)
d=	24.1	in	Beam Depth from database
$d+t_{stem}$=	24.85	in	
P_{u_link}=	99.69	kips	Link Axial demand, Mu/(d+tstem)
P_{y_link}=	168.75	kips	Link Yield Strength from YL database
Link stem DCR=	**0.656**	OK	*$=P_{u\_link}/(0.9*P_{y\_link})$*

Krot: **AISC 358, 12.9 Step 11:**

K1=	35752	k/in
K2=	23673	k/in
K3=	9787.5	k/in
Keff=	5801	k/in
Mye=	4612.78125	k*in
Δ_y=	0.03199853	in
θy=	0.00257533	rad
Krot=	1791139	k*in/rad

Slip-Critical Check:

Pu_slip=	99.69	kips	*=Pu_link*
ϕ=	1.0		
μ=	0.30		
Du=	1.13		
hf=	1.00		
ns=	1.00		
Tb=	102.00	kips	*Minimum Bolt Pretension per Table J3.1*
n_bolt_linkBm=	6.000		
ϕRn_slip=	207.5	kips	*= ϕ * μ *Du *hf *Tb *ns *n_bolt_linkBm*
Link slip DCR=	**0.481**	OK	*$=P_{u\_link}/\phi Rn\_slip$*

3.8 BUCKLING RESTRAINT PLATE AND SPACER GEOMETRY: (From YL Database)

D_brp=	0.750	in	*from Database*
nbolt_brp=	2.00		*per Spacer plate*
S_{BRP}=	2.50	in	*from Database*
t_{BRP}=	1.00	in	*from Database*
Lbe=	3.75	in	*from Database*
L_brp_cont=	6.500	in	*=Lbrp - Lend - S_{BRP}*
Lbrp=	12.750	in	*from Database*
Wbrp_min=	10.000	in	*= Wbm_side*
Fy_BRP=	50.00	ksi	*User Input*
Ry_BRP=	1.10		*User Input*
Fu_BRP=	65.00	ksi	*User Input*
Rt_BRP=	1.20		*User Input*

3.9 BRP THICKNESS CHECK: (AISC 358, Chapter 12, Design Step 10.1)

Pcap_Link=	263.25	kips	*'=if(R=3,Py_link, Pr_link)*
Lelong=	0.621	in	*Elongated distance at 0.05 rad*
L1st_bolt=	3.75	in	*Start of yielding region to 1st BRP bolt centerline*
Lcant=	3.97	in	*Lever arm from edge of yield link to edge of bolt hole + elongated dist at 0.05 rad*
bn_brp=	8.25	in	*Net width of BRP, without bolt holes*
tbrp,min=	0.774	in	*AISC 358, EQ 12.9-13*
tbrp_use=	1.000	in	*value previously defined*
tbrp_DCR=	**0.774**		OK

3.10 BEAM FLANGE THICKNESS CHECK: (AISC 358, Chapter 12, Design Step 10.2)

Joint Rotation=	0.04	rad	
$e_{0.04}$=	0.05847	in/in	
g=	0.01096	in	*gap between BRP and beam flange*
Iy=	0.158	in^4	
l_o=	1.071	in	*effective buckling wave length*
Qi=	10.78	kips	
N=	9.34		*=LstemYield / lo*
N=	9.0		*Round down to next integer*
Ndesign=	5.0		*N used for design (Contact points with BRP)*
Tux=	53.9	kips	*=Ndesign * Qi*
Tux_bolt=	13.47	kips	*=Tux/Nbolts*
ϕ=	1.0		
Ledge=	6.500	in	*1st bolt to edge of BRP*
S_brpBolt_min=	6.25	in	*=byield + 2*(0.125 + d_brp_bolt)*
gbrp=	6.750	in	
b=	3.125	in	*BRP bolt to face of web*
c=	6.150	in	*edge of beam flange to face of web*
a=	3.025	in	*center of BRP bolt to edge of beam flange*
x=sqrt(b*c)=	4.384	in	
4*x=	17.54	in	
2*x=	8.77	in	
S_brp_bolt=	2.500	in	
pe=	10.02	in	
b'=	2.750	in	
tf_min=	0.477	in	*=sqrt(4*b'*T/(f*pe*Fu_bm))*
tbf=	0.750	in	*beam flange thickness*
tbf_DCR=	**0.636**	OK	

1 row of BRP bolts:		2 rows of BRP bolts:		3 rows of BRP bolts:	
pe (in)=	17.54	pe1 (in)=	10.02	pe1 (in)=	10.02
		pe2 (in)=	17.54	pe2 (in)=	3.44
		pe (in)=	10.02	pe3 (in)=	10.02
				pe (in)=	7.83

3.11 BRP BOLT SIZE AND QUANTITY CHECK: (AISC 358, Chapter 12, Design Step 10.3)

I_x=	5.695	in^4	*=byield^3 * tstem /12,* *strong axis moment of inertia of reduced link region*
gap=	0.125	in	*gap between link and spacer plate*
Vuy/Spacer=	18.37	kips	
Vuy per bolt=	9.184	kips	

	Tx (k)	Vx (kip)	Ty (kips)	Vy (kips)
Bolt Forces	13.47	4.042	0	9.184

Bolt Type (Bolt_Gr_linkBm)=	A325		
Thread Condition=	N		
ϕbolt=	0.75		
Fnv_325	54.0	ksi	*AISC 360 Table J3.2*
Ab_stem=	0.442	in^2	*= Pi* (boltD_linkBm/2)^ 2*
Strength per Bolt (ϕVbolt) =	17.89	kips	*= ϕbolt* Fnv_325* Ab_stem*
Fnt_325	90	ksi	*AISC 360 Table J3.2*
ϕT_{bolt}=	29.82	kips	*=ϕbolt* Fnt_325* Ab_stem*

X-Dir. DCR	Y-Dir. DCR

Tension	0.45	0.00
Shear	0.226	0.513
RnV_(V+T), kips	15.18	17.89
T+V	0.266	0.513

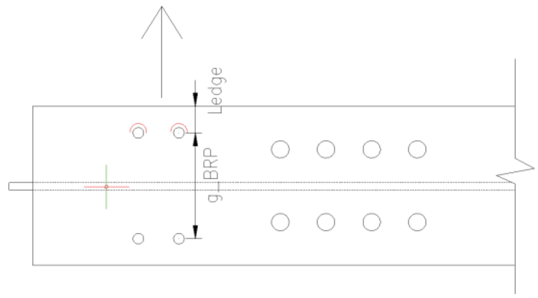
Max DCR=	**0.513**	OK

3.12 LINK STEM HOLE/BRP HOLE FIT ON BEAM FLANGE (Geometry Check)

bbf=	12.80	in	= beam flange width
bbf_BRP_bolt=	9.25	in	=minimum flnage width for BRP bolt spacing + min. bolt edge distance
bbf_stem_bolt=	8.75	in	=minimum flange width for stem bolt spacing + min. bolt edge distance
bbf_min=	9.25	in	=max (bbf_BRP_bolt, bbf_stem_bolt)
bf_DCR=	0.723	OK	

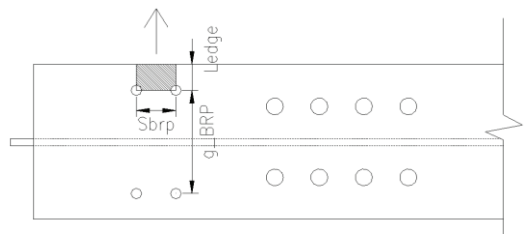
3.13 BRP BOLT BEAM FLANGE BEARING AND TEAROUT IN WEAK-AXIS DIRECTION (AISC 360, J3.10)

bbf=	12.8	in
g_brp=	6.75	in
d_brp=	0.750	in
Ledge=	3.025	in
Ledge_min=	1.00	in
Ledge>1 bolt Dia?	OK	
Ledge>=Ledge_min?	OK	
Vy=	9.18	kips/bolt
Rn_bearing=	87.75	kips
ϕRn_bearing=	65.81	kips
DCR_bearing=	0.140	OK
Lc=	3.025	in
Rn_tearout=	177.0	kips
ϕRn_tearout=	132.7	kips
DCR_tearout=	0.069	OK



3.14 BRP BOLT BEAM FLANGE BLOCK SHEAR IN WEAK-AXIS DIRECTION (AISC 360, J4)

Sbrp=	2.5	in
d_brp=	0.8125	in
Ledge=	3.025	in
tbf=	0.750	in
Vuy=	18.37	kips
Lgv=	6.05	in
Agv=	4.5375	in^2
Rn_yield=	136.1	kips
ϕRn_yield=	136.1	kips
DCR_bearing=	0.135	OK
Lnv=	5.238	in
Anv=	3.928	in^2
Rn_tearout=	153.2	kips
ϕRn_tearout=	114.90	kips
DCR_tearout=	0.160	OK



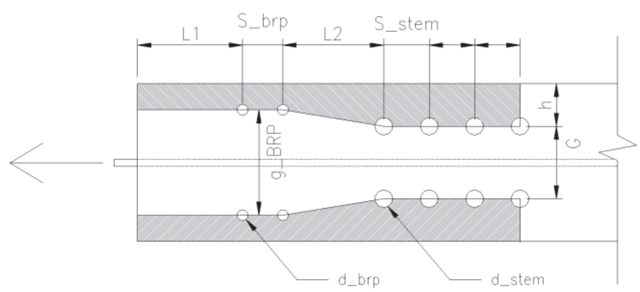
Lnt=	1.6875	in	
Ant=	1.266	in^2	
Rn_BS1=	235.46	kips	
Rn_BS2=	218.39	kips	
Rn_BS=	218.39	kips	(Rn_BS1,Rn_BS2)
ϕRn_BS=	163.79	kips	
DCR_BS=	0.112	OK	

3.15 BEAM FLANGE CHECK FOR LINK STEM-TO-BEAM FLANGE BOLT BEARING AND TEAROUT (AISC 360, J3.10)

Pr_link=	263.25	kips	
tbf=	0.750	in	Beam Flange thickness
bolt_Dia=	1.25	in	Link stem-to-beam flange bolt diameter
n_bolts=	6		Number of link-stem bolts
S_stem_bolts=	3.375	in	stem bolt spacing
Lc=	2.0625	in	
Fy_bm=	50	ksi	from Design_Limits
Fu_bm=	65	ksi	from Design_Limits
Rn_bearing=	877.5	kips	=2.4*bolt_Dia*tbf*Fu_bm*n_bolts
ϕRn_bearing=	658.13	kips	=0.75*Rn_bearing
DCR_bearing=	0.400	OK	
Rn_tearout=	723.9	kips	=1.2*Lc*tbf*Fu_bm*n_bolts
ϕRn_tearout=	543.0	kips	=0.75*Rn_tearout
DCR_tearout=	0.485	OK	

3.16 LINK STEM-TO-BEAM FLANGE BOLT BLOCK SHEAR CHECK (AISC 360, J4)

L1=	6.5	in
Sbrp=	2.5	in
L2=	6.25	in
Stem=	3.375	in
nstem_bolts=	6	
Gstem=	6	in
gbrp=	6.75	in
d_brp=	0.8125	in
d_stem=	1.3125	in
bf=	12.8	in
h=	3.400	in
h_min=	1.625	in
h>1 bolt Dia?	OK	
h>=h_min?	OK	



Lgv=	26.25	in		Lnt=	5.4875	in	
Agv=	19.688	in^2		Ant=	4.116	in^2	
Rn_yield=	1181.3	kips					
ϕRn_yield=	885.9	kips		Rn_BS1=	892	kips	
DCR_yield=	0.297	OK		Rn_BS2=	858.1	kips	
				Rn_BS=	858	kips	(Rn_BS1,Rn_BS2)
Lnv=	21.34	in		ϕRn_BS=	643.6	kips	
Anv=	16.01	in^2		DCR_BS=	0.409	OK	
Rn_rupture=	624.3	kips					
ϕRn rupture=	468.2	kips					
DCR_rupture=	0.562	OK					

3.17 BEAM FLANGE NET SECTION CHECK (AISC 360 F13.1)

Z_{bx}=	289	in^3		Afg=	9.60	in^2
Z_{holes}=	46.0	in^3	$=2*D_{holes}*tbf*(d-tbf)$	Afn=	7.63	in^2
Z_{net}=	243	in^3	$=Z_{bx} - Z_{holes}$	Yt=	1	
$M_{Pb\ net}$=	13367	k-in	$=Znet * Fy * R_y$	Fu*Afn=	496	kips
$P_{r\ link}$=	263.3	kips		Yt*Fy*Afg=	480	kips
d+tstem=	24.85	in		Sx=	258	in^3
$M_{pr\ link}$=	6542	k-in		ϕ*Mn=	13005	k-in
DCR_NetSection=	0.489	$=M_{pr\_link}/M_{pb\_net}$		DCR_NetSection2=	0.503	$=M_{pr\_link}/\phi*Mn$

3.18 DESIGN SUMMARY:

	Design Sections								
	3.7	3.9	3.10	3.11	3.13 & 3.14	3.15 & 3.16	3.17	Max DCR	Check
t_BRP DCR	-	0.774	-	-	-	-	-	0.774	OK
Beam tf DCR	-	-	0.636		0.160	0.562	0.503	0.636	OK
BRP Bolt DCR	-	-	-	0.513	-	-	-	0.513	OK
Link Strength DCR	0.656	-	-	-	-	-	-	0.656	OK
Link Slip DCR	0.481	-	-	-	-	-	-	0.481	OK
Ly_yield DCR	0.881	-	-	-	-	-	-	0.881	OK
BRP Bolt Edge Dist>1d	-	-	-	-	OK	-	-	-	OK
Stem Bolt Edge Dist>1d	-	-	-	-	-	OK	-	-	OK
								Overall Check=	OK

5.1 CURRENT MEMBER:

Column ID:	1	Top Story=	Story2	Use Doubler PL?=	NO
Column size:	W24X131	Left Beam ID=	N/A	Right Beam ID=	1001
Pu (kips):	139.86	Left Beam size=	N/A	Right Beam size=	W24X104
Hcc_b (in):	144	LeftEndLink=	N/A	LeftEndLink=	YES
Hcc_t (in):	144	RightEndLink=	N/A	RightEndLink=	YES
Stiffener (Y/N):	YES	db_left (in)=	#N/A	db_right (in)=	24.1
Ext. Stiffener depth=	Full Depth	Left Link Size:	N/A	Right Link Size:	YL6-4.5
Current Story=	Story1	Left Beam Lh (in):	N/A	Right Beam Lh (in):	360.00
Story Above=	Story2	Vu_Gravity (kips):	0.00	Vu_Gravity (kips):	14.71

5.2 LINK PROPERTIES: (From YL Database)

	Left Link	Right Link					
Thickness (t_stem) =	0.000	0.750	in		R=	8	
Yield Width (b_yield)=	0.000	4.500	in		Fy_link=	50	ksi
Thickness (t_flange)	0.000	1.250	in		Fu_link=	65	ksi
Spacing Along Length (horiz) (bolt_s_flange)	0.000	6.000	in		Rt=	1.2	
Gauge Along Width (vertical) (bolt_g_flange)=	0.000	5.750	in		Py_linkL=	0	kips
Flange height (H_flange) =	0.000	9.250	in		Pr_linkL=	0	kips
					Py_linkR=	168.75	kips
a=	0.000	3.500	in		Pr_linkR=	263.25	kips
Pcap_Link=	0.0	263	kips	= if (R=3, Py_link, Pr_Link)			
Mcap=	0	6542	k-in	= Pcap_Link *(db+tstem)			

5.3 CHECK STRONG COLUMN WEAK BEAM REQUIREMENTS: (AISC 358 Chapter 12, Step 14)

	Left Side	Right Side					
						AISC 360	Table B4.1b
Mcap=	0.00	6541.76	kip*in	*Value previously defined*	Column Size=	W24X131	
Lh=	N/A	328.5	in		Col b/t=	6.7	Compact
Vbm_gravity=	0.00	14.71					
Vu_bm=	0.00	54.54	kip	*= n_sides* Mpr/ Lh + Vbm_gravity*	Col h/tw=	35.6	Compact
Column Size=	W24X131	W24X131		*Value previously defined*			
Fyc=	50	50	ksi	*Value previously defined*		AISC 341	Table D1.1
a=	0.00	3.50	in	*Value previously defined*	Col b/t=	6.7	Hi.Ductile
Agc=	38.6	38.6	in^2	*Looked up value*			
Zcx=	370.00	370.00	in^3	*Looked up value*	Col h/tw=	35.6	Hi.Ductile
dc=	24.5	24.5	in	*Looked up value*			
tcf=	0.96	0.96	in	*Looked up value*			
Pu_colPositive=	139.86	139.86	kips	*Max Axial Force of column per Overstrength combo*			
Muv=	0.0	859.0	kips*in	*= Vu_bm* (dc/2 + a)*			
ΣMp_col=	34319	34319	kip*in	*= Zcx* (Fyc - Pu_colPositive/ Agc) * if(top story,1 else 2)*			
ΣMpb=	0	7401	kip*in	*= Mpr + Muv*			
Pu<0.3*Pc?=	YES	YES		*AISC 341-16 E3.4a Exception (a)*			
Top Story?=	NO	NO		*AISC 341-16 E3.4a Exception (a) (1)*	SCWB not applicable at top story		
SCWB_DCR=	0.000	0.216					
SCWB_DCR_total=	**0.216**	OK					

5.4 COLUMN PANEL ZONE CHECK (WITHOUT DOUBLER PLATE): (AISC 358 Chapter 12, Step 16)

ϕv_pz=	0.90			
Fyc=	50.00		ksi	*Value previously defined*
dc=	24.50		in	*Value previously defined*
tcw=	0.61		in	*Value previously defined*
Agc=	38.60		in^2	*Value previously defined*
Pc=	1930		kips	*= Fyc* Agc*
Pu=	139.86		kips	*Value previously defined*
ϕRn_PZ=	400.21		kips	*= ϕv_pz* if (Pu < 0.4* Pc, 0.6* Fy_col* dc* tcw, 0.6* Fy_col* dc* tcw* (1.4 - Pu/Pc))*
	Left Side	**Right Side**		
Vu_c=	0.00	45.43	kips	*= ΣMpb/((Hcc_b+Hcc_t)/2), 0 for top story*
Pcap_Link=	0.00	263.25	kips	*= Pu_link*
Ru=	0.00	217.82	kips	
DCR_PZ=	0.000	0.544		*= Pr_link - Vu_c / ϕRn_PZ*
DCR_PZ_geometry=	0.000	0.830		=[(dz+wz)/90] / tcw, 0 for R=3 and R=4.5 AISC 341 column web PZ geometry check
DCR_PZ_total=	**0.830**	**OK**		*Ok, if DCR_PZ <= DCR_PZ_Limit*

5.4.1 COLUMN WEB DOUBLER PLATE CHECK: (AISC 358 Chapter 12, Step 16)

Doubler Plate Required=	**NO**		<----If NO, Skip 3.4.1	USE Plug Weld? **NO**
Fy_dpl=	50		ksi	*=Doubler Yield Strength*
ϕRn_PZ_NODBLR=	400.21		kips	*= ϕv_pz* if (Pu < 0.4* Pc, 0.6* Fy_col* dc* tcw, 0.6* Fy_col* dc* tcw* (1.4 - Pu/Pc))*
ϕRn_PZ_req=	0.00		kips	= Additional capcaity required
ϕRn_PZ_NEW=	400.2		kips	= ϕRn_PZ_NODBLR + ϕRn_PZ_req
t_dblr_strength=	0.000		in	
dz=	22.60		in	*$=d_b-2t_{bf}$, if different d_b, use average*
wz=	22.58		in	*$=d_c-2t_{cf}$*
t_dblr_geom=	0.625		in	*=ceiling(((dz+wz)/90),1/16), 0 for R=3 and R=4.5*
t_dblr=	0.625		in, W/O plug weld	
t_dblr_min=	0.375		in, with plug weld	=max([(dz+wz)/90] - tcw,tdp_min,t_dblr_strength)
t_dblr_use=	**0.625**		in	Doubler Plate thickness
ϕRn_PZ_DBLR=	781.25		kips	*= ϕv_pz* if (Pu < 0.4* Pc, 0.6* (Fy_col* dc* tcw + Fy_dpl*(dc-2*tcf)*t_dblr),* 0.6* (Fy_col* dc*tcw + Fy_dpl*(dc-2*tcf)*t_dblr)* (1.4 - Pu/Pc))
	Left Side	**Right Side**		
DCR_PZ_NEW=	0.000	0.279		
DCR_PZ_total_NEW=	**0.279**	**OK**		
DCR_PZ_geometry_Web=	0.830			=[(dz+wz)/90] / tcw, 0 for R=3 and R=4.5
DCR_PZ_geometry_Total=	0.408		in, with plug weld	=[(dz+wz)/90] / (tcw+t_dblr)
DCR_PZ_Total=	**0.408**			=max of DCR_PZ (strength), DCR_PZ (geometry_web), DCR_PZ (geometry_DP)

5.4.2 DOUBLER TO COLUMN WEB/CONTINUITY PLATE WELD (AISC 358 Chapter 12, Step 19)

Option 1: Doubler without Continuity plates

t_dblr=	0.625	in	doubler plate thickness
tcw=	0.61	in	column web thickness
min t=	0.625	in	=min of tDP and tcw, rounded up to nearest 1/16
wfillet=	0.25	in	From AISC 360 Table J2.4

Option 2A: Extended Doubler Plate

Continuity plate to doubler per Section E3.6f.2(c)

t_dblr=	0.625	in	doubler plate thickness (c.3, available strength of doubler plate)
wfillet=	0.3906	in	=5/8*tDP (develop shear strength of the doubler Plate), rounded to nearst 1/16
wfillet_use=	0.4375	in	
# of sides=	2		

Option 2B: Doubler plates placed between continuity plates

75% of the available shear yield strength of the full doubler plate thickness (AISC 341 E3.3(2))

t_dblr=	0.625	in	doubler plate thickness
wfillet_use=	0.6250	in	t_dblr, same thickness to develop doubler Plate capacity
# of sides=	1		

5.4.3 DOUBLER TO WEB PLUG WELD (IF REQUIRED) (AISC 358 Chapter 12, Step 16)

Doubler Plate Plug-Weld Used?	NO	
dz_mod=	11.300	in, 0.5*dz
wz_mod=	11.29	in, 0.5*wz
(dz_mod+wz_mod)/90	0.251	OK
PlugWeld_Dia=	0.938	in

Weld Depth= 0.625 in

5.4.4 DOUBLER TO COLUMN FLANGE WELD (AISC 358 Chapter 12, Step 16)

Option 1: Fillet weld to develop doubler plate shear capacity

t_dblr=	0.625	in	doubler plate thickness
Vn=	18.75	kips	0.6*Fy*Agv (per 1 in length)
wfillet=	14.0	/16	Vn/1.39, rounded up to 1/16"

Option 2: Groove weld per AWS D1.8 Clause 4.3

5.5 CHECK UNSTIFFENED COLUMN FLANGE AND WEB:

SMF_Sides= 1 *(1 for singled sided SMF, 2 for double sided SMF)*

	Left Side	Right Side		
dbm=	#N/A	24.1	in	*d beam*
tbf=	#N/A	0.75		
d_stiff_cap=	#N/A	3.5		
d_stiffTop=	49	49	in	*Location of top stifferener to top of column cap, assume 2 dc if not at top story*
d_stiffBot=	#N/A	73.1	in	*Location of bottom stiffener to top of column cap = d + d_stiffTop*

5.6 COLUMN WEB LOCAL YIELDING (AISC 360 J10.2):

	Left Side	Right Side		
ϕWLY=	1	1		
dc=	24.5	24.5	in	*Value previously defined*
kdes=	1.46	1.46	in	*Looked up value*
t_flange=	0.000	1.250	in	*Link flange thickness*
t_stem=	0.000	0.750	in	*Link stem thickness*
l_b=	0	3.25	in	*= 2* t_flange + t_stem*
Ct_top=	1	1		*= if (d_stiffTop <= dc, 0.5, 1)*
Ct_bot=	#N/A	1		*= if (d_botTop <= dc, 0.5, 1)*
ϕRn_WLYmoreD=	220.83	319.14	kips	*= ϕWLY* Fy_col* tcw* (5* kdes + l_b)*
ϕRn_WLYlessD=	110.41	208.73	kips	*= ϕWLY* Fy_col* tcw* (2.5* kdes + l_b)*
ϕRn_WLY_top=	#N/A	319.14	kips	*= if (d_stiffTop > d, fRn_WLYmoreD, fRn_WLYlessD)*
ϕRn_WLY_bot=	#N/A	319.14	kips	*= if (d_stiffBot > d, fRn_WLYmoreD, fRn_WLYlessD)*

5.7 COLUMN WEB LOCAL CRIPPLING (AISC 360 J10.3):

ϕWLC=	0.75		
Es=	29000	ksi	
tcf=	0.96	in	*Value previously defined*
tcw=	0.605	in	*Value previously defined*
ϕRn_WLC_more0.5D=	399.4	kip	*= ϕWLC* 0.8* tcw^2* (1 + 3*(l_b/dc)*(tcw/tcf)^1.5)* Sqrt(Es* Fy_col* tcf/ tcw)*
AISC J10-5a=	199.72	kips	*= 0.5* ϕRn_WLC_more0.5D*
AISC J10-5b=	149.9	kips	*= ϕWLC* 0.4* tcw^2* (1 + 3*(4* l_b/dc - 0.2)*(tcw/tcf)^1.5)* Sqrt(Es* Fy_col* tcf/ tcw)*
ϕRn_WLC_less0.5D=	199.72	kips	*= if (l_b/ dc <= 0.2, AISC J10-5a, AISC J10-5b)*

	Left Side	Right Side		
ϕRn_WLC_top=	#N/A	399.4	kips	*= if (d_stiffTop > d, fRn_WLC_more0.5D, fRn_WLC_less0.5D)*
ϕRn_WLC_bot=	#N/A	399.4	kips	*= if (d_stiffBot > d, fRn_WLC_more0.5D, fRn_WLC_less0.5D)*

5.8 COLUMN WEB COMPRESSION BUCKLIING (AISC 360 J10.5):

Note: This check is for 2-sided SMF connection only

ϕWCB=	0.9		
h=	21.58	in	*= dc - 2*kdes*
ϕRn_WCB=	266.9	kips	*AISC J10-8, = ϕWCB* 24* tcw^3* sqrt(Es* Fy_col)/h*

	Left Side	Right Side		
ϕRn_WCB_top=	#N/A	266.90	kips	*= if (d_stiffTop < d, ϕRn_WCB*0.5, ϕRn_WCB)*
ϕRn_WCB_bot=	#N/A	266.90	kips	*= if (d_botTop < d, ϕRn_WCB*0.5, ϕRn_WCB)*

5.9 SUMMARY OF LOCAL CAPACITIES:

Left Side

At Top Stiffener			At Bottom Stiffener		
ϕRn_WLY_top=	#N/A	kips	ϕRn_WLY_bot=	#N/A	kips
ϕRn_WLC_top=	#N/A	kips	ϕRn_WLC_bot=	#N/A	kips
ϕRn_WCB_top=	#N/A	kips	ϕRn_WCB_bot=	#N/A	kips

if (SMF_Sides = 1, ϕRn_Stiff_top = min (ϕRn_WLY_top, ϕRn_WLC if (SMF_Sides = 1, ϕRn_Stiff_bot = min (ϕRn_WLY_bot, ϕRn_WLC_bot),
min (ϕRn_WLY_top, ϕRn_WLC_top, ϕRn_WCB_top)) min (ϕRn_WLY_bot, ϕRn_WLC_bot, ϕRn_WCB_bot))

ϕRn_Stiff_top=	#N/A	kips	ϕRn_Stiff_bot=	#N/A	kips		
Top Stiffener Required=	#N/A		Bottom Stiffener Required=	#N/A		Stiff Req'd=	**#N/A**
Top Stiff Provided?:	YES		Bottom Stiff Provided?:	YES			

Right Side

At Top Stiffener			At Bottom Stiffener		
ϕRn_WLY_top=	319.14	kips	ϕRn_WLY_bot=	319.14	kips
ϕRn_WLC_top=	399.45	kips	ϕRn_WLC_bot=	399.4	kips
ϕRn_WCB_top=	266.90	kips	ϕRn_WCB_bot=	266.90	kips

if (SMF_Sides = 1, ϕRn_Stiff_top = min (ϕRn_WLY_top, ϕRn_WLC if (SMF_Sides = 1, ϕRn_Stiff_bot = min (ϕRn_WLY_bot, ϕRn_WLC_bot),
min (ϕRn_WLY_top, ϕRn_WLC_top, ϕRn_WCB_top)) min (ϕRn_WLY_bot, ϕRn_WLC_bot, ϕRn_WCB_bot))

ϕRn_Stiff_top=	319.14	kips	ϕRn_Stiff_bot=	319.14	kips		
Top Stiffener Required=	No		Bottom Stiffener Required=	No		Stiff Req'd=	**NO**
Top Stiff Provided?:	YES		Bottom Stiff Provided?:	YES			

5.10 WEB SIDESWAY BUCKLING (AISC 360-J10.4):

Note: Assume compression flange is NOT restrained against rotation

	Left Side	Right Side		
ϕWSB=	0.85	0.85		
Hcc=	144.00	144.00	in	*Value previously defined*
bcf=	12.9	12.9	in	*Column flange width*
Scx=	329.00	329.0	in^3	*Looked up value*
Lb=	#N/A	132.0	in	*= Hcc - d/2*
My=	16450.0	16450.0	kips*in	*= Scx* Fyc*
Mu=	0	6542	kips*in	*=Mpr*
Cr=	960000	960000	ksi	*= if (Mu < My, 960000, 480000)*
(h/tcw)/(Lb/bcf)=	#N/A	3.49		*= (h/ tcw)/ (Lb/ bcf)*
ϕRn_WSB=	#N/A	6318	kips	*AISC J10-7, = (ϕWSB* Cr* tcw^3* tcf/ h^2)/ (0.4* Cr^3)*
WSB_check=	N/A	N/A		*Not applicable*
BotStiffener_Bracing_Req'd=	#N/A	No		*= if (And (ϕRn_WSB < Pr_link, WSB_check = "Applicable"), "Yes", "No")*

5.11 COLUMN STIFFENER DESIGN: (AISC 358 Chapter 12, Step 19)

	Left Side	Right Side		
Fsu_top=	#N/A	0.0	kips	*= if (Pr_link - ϕRn_Stiff_top < 0, 0, Pr_link - ϕRn_Stiff_top)*
Fsu_bot=	#N/A	0.0	kips	*= if (Pr_link - ϕRn_Stiff_bot < 0, 0, Pr_link - ϕRn_Stiff_bot)*
Fsu=	#N/A	0.0	kips	*= max (Fsu_top, Fsu_bot)* SMF_Sides*

5.12 CONTINUITY PLATE REQUIREMENTS BASED ON GEOMETRY AND TENSION YIELDING: (AISC 358 Chapter 12, Step 19)

ϕt=	0.9		*Value previously defined*
Fy_stiff=	50	ksi	*Value previously defined*
Kdet=	1.875	in	*Looked up value*
K1=	1.125	in	*Looked up value*
tcwdet/2 =	0.313	in	*Looked up value*
tcf(det) =	0.938	in	*Looked up value*

	Left Side	Right Side		
As_min=	#N/A	0.00	in^2	*= Fsu/ (ft* Fy_Stiff)*
Lclip_web_min=	2.438	2.438	in	*= Kdet - tcf + 1.5*
Lclip_web=	**2.438**	**2.438**	in	*= Ceiling (Lclip_web_min, 0.0625)*
Lclip_flange_min=	0.813	0.813	in	*= K1 - tcwdet/2*
Lclip_flange=	**0.813**	**0.813**	in	*= Ceiling (Lclip_flange_min, 0.0625)*
bstiff_min=	3.9975	3.9975	in	*AISC 360 J10.8, = bcf/3 - tcw/2*
bstiff_max=	6.1475	6.1475	in	*= (bcf - tcw)/2*
bstiff=	**6.1250**	**6.1250**	in	*= Floor (bstiff_max,0.125), Each side of column web*
tstiff_USE?	YES	YES	in	*Input value*
t_stiff_tension=	#N/A	0.000		*= As_min/ (bstiff_Lclip_flange)/ 2/ SMF_Sides, OK, if t_stiff_tension <= tstiff*
tstiff_min=	0.000	0.500	in	*= max (t_stem/2, bstiff/16, looked up value), OK, if tstiff_min <= tstiff*
tstiff_max=	0.000	0.750	in	*= t_stem, OK, if tstiff_max >= tstiff*
t_stiff_Use=	#N/A	0.500		

Lstiff_Pdepth_ini=	11.29	in	*= 0.5* dc - tf, For Single sided connections (partial depth)*
Lstiff_Fdepth_ini=	22.58	in	*= dc - 2*tcf, For double sided connections*
Lstiff_ini=	22.58	in	*= if (and(SMF_Sides = 1,Ext. Stiffener depth="Partial"), Lstiff_Pdepth_ini, Lstiff_Fdepth_ini)*
Lstiff=	22.56	in	*= Floor (Lstiff_ini, 0.0625)*

5.13 FULL DEPTH STIFFENER PLATE FOR 2-SIDED MOMENT CONNECTIONS ONLY: THIS SECTION NOT APPLICABLE TO 1-SIDED SMF

	Left Side	Right Side			
ϕc=	0.9	0.9			
Kstiff=	0.75	0.75			
Astiff_fd=	#N/A	10.52	in^2		*= 2* bstiff* tstiff + 12* tcw^2*
Istiff_fd=	#N/A	88.64	in^4		*= 12* tcw* tcw^3/12 + 2* (tstiff* bstiff^3/ 12+ bstiff* tstiff* (bstiff/2 +tc/2)^2)*
rstiff_fd=	#N/A	2.90	in		*= Sqrt (Istiff_fd/ Astiff_fd)*
Kstiff*Lstiff_fd/rstiff_fd=	#N/A	5.83			*= Kstiff* Lstiff_fd/ rstiff_fd*
Fe=	#N/A	8424	ksi		*= Pi^2* 29000/ (Kstiff* Lstiff_fd/ rstiff_fd)^2*
Fcr=	#N/A	49.88	ksi		*= if (Kstiff* Lstiff_fd/ rstiff_fd <= 4.71* Sqrt (29000/ Fy_stiff), 0.658^ (Fy_stiff/ Fc)* Fy_stiff,*
Pn_stiff=	#N/A	525.9	kips		*= Kstiff* Lstiff_fd* rstiff_fd > 25, Fcr* Astiff_fd, Astiff_fd* Fy_stiff 0.877* Astiff_fd)*
ϕc*Pn_stiff=	#N/A	473.3	kips		*= fc* Pn_stiff*
DCR_stiff_comp=	**0.000**	**0.000**		OK	*= if (SMF_Sides = 1, 0, Fsu/ (fc* Pn_stiff)) OK if DCR_Stiff_comp < stiff_DCR*

5.14 STIFFENER PLATE DOUBLE SIDE FILLET WELD TO COLUMN FLANGE: (AISC 358 Chapter 12, Step 19.2)

φfillet=	0.75			
Fexx=	70		ksi	
tstiff=	0.500		in	*Value previously defined*
wfillet_min=	0.1875		in	*Minimum fillet weld size per AISC 360 Table J2.4*
	Left Side	**Right Side**		
wfillet_flange_min=	#N/A	0.000	in	*= (0.5* Fsu/ SMF_Sides)/ (φfillet* 1.5* 0.6* Fexx* (bstiff - Lclip_flange-0.5)*2^(1/2))*
wfillet_flange=	#N/A	0.188	in	*=Max (wfillet_min, ceililing(wfillet_flange_min, 1/16"))*
wfillet_flange_DCR=	**#N/A**	**0.000**		*= wfillet_flange_min/ wfillet_flang*

5.15 STIFFENER PLATE DOUBLE SIDE FILLET WELD TO COLUMN WEB: (AISC 358 Chapter 12, Step 19.2)

	Left Side	Right Side		
wfillet_min=	0.1875	0.1875	in	*Minimum fillet weld size per AISC 360 Table J2.4*
wfillet_min_web=	#N/A	0.000	in	*= 0.5* Fsu* SMF_Sides/ (ffillet* 0.6* Fexx* (Lstiff - Lclip_web-0.5)*2^(1/2))*
wfillet_web=	#N/A	0.188	in	*=max(wfillet_min, ceiling(wfillet_web,1/16)*
wfillet_web_DCR=	**#N/A**	**0.000**		

5.16 CHECK MINIMUM COLUMN FLANGE WIDTH AND THICKNESS: (Geometry Check)

	Left Side	Right Side		
bcf=	12.9	12.9	in	=column flange width
bcf_LinkFlg_bolt=	0	9.25	in	=Link Sflange+2*Ledge_min
bcf_DCR=	**0.000**	**0.717**	OK	

5.16.1 CONNECTION AWAY FROM COLUMN ENDS (Step 18 Table 1.1):

	Left Side	Right Side		
φb=	0.9	0.9		
tcf=	0.96	0.96	in	*Value previously defined*
c=	0.000	5.750	in	*= bolt_g_flange*
g=	0.000	6.000	in	*= bolt_s_flange*
H_flange=	0.000	9.250	in	*Value previously defined*
s=	0.000	4.399	in	*= 0.5* Sqrt (bcf* g)*
pso=	-0.250	2.750	in	*= (g - tstiff)/2*
psi_tmp=	-0.250	2.750	in	*= pso*
psi=	-0.250	2.750	in	*= if (psi_tmp > s, s, psi_tmp)*
h1=	#N/A	21.85	in	*= db + t_stem - g/2*
h0=	#N/A	27.85	in	*= db + t_stem + g/2*
Yc_unstiffened=	#N/A	199.01	in	*= bcf/2* (h1/s + h0/s) + 2/g* (h1* (s + 3*c/4) + h0* (s +c/4) + c^2/4) +g/2*
tcf_req_unstiffened=	#N/A	0.90	in	*= Sqrt (1.1* Mcap/ (φb* Fyc* Yc_unstiffened))*
Yc_stiffened=	#N/A	307.88	in	*= bcf/2* (h1* (1/s + 1/psi) + h0* (1/s + 1/pso)) + 2/g* (h1* (s + psi) + h0* (s + pso))*
tcf_req_stiffened=	#N/A	0.72	in	*= Sqrt (1.1* Mcap/ (fb* Fyc* Yc_stiffened))*
Stiffened?=	YES	YES		*Bottom Stiffener Required per 17.3*
tcf_min=	#N/A	0.721	in	*= if (Stiffened? = YES, tcf_req_stiffened, tcf_req_unstiffened)*
DCR_colFLB1=	**N/A**	**0.751**		*= tcf_min/ tcf*

5.16.2 CONNECTION AT STIFFENED COLUMN END (STEP 18, TABLE 1.2, CASE 1):

	Left Side	Right Side		
de=	1.25	1.25	in	*centerline of top link bolt to top of column*
pfi=	-0.25	2.75	in	*= (g - tstiff)/ 2*
pso=	-0.25	2.75	in	*Value previously defined*
Yp=	#N/A	258.226	in	*= bcf/2* (h1* (1/pfi + 1/pso) + h0* (1/pso + 1/2/s)) + 2/g* (h1* (pfi + s) + h0* (d2 + pso))*
tcf_req=	#N/A	0.787	in	*= Sqrt* (1.1* Mcap/ (fb* Fy* Yp))*
DCR_colFLB2=	**N/A**	**0.820**		*= tcf_req/ tcf*

5.17 STABILITY BRACING AT BEAM-TO-COLUMN CONNECTIONS (AISC 341-16 SECTION E3 4.c (b))

	Left Side	Right Side		
Link Size=	N/A	YL6-4.5		
byield=	0.000	4.500	in	
tstem=	0.000	0.750	in	
Fy_link=	50	50	ksi	
Ry=	1.1	1.1		
Pye_link=	0	185.625	kips	AISC 341, E3.4c.1(b)
0.02 * Pye_link=	0.000	3.713	kips	*Bracing Force (LRFD)*
Pbrace (ASD)=	0.000	2.599	kips	*=0.7*Bracing Force (LRFD)*

5.18 DESIGN SUMMARY:

DCR Summary:

			Report:
SCWB_DCR_total=	0.216		0.216
DCR_PZ_total=	0.830	Overall PZ chk:	OK
DCR_PZ_total_NEW=	0.279	NO	0.830
DCR_stiff_comp_DCR=	0.000	0.000	0.000
wfillet_flange_DCR=	#N/A	0.000	0.000
wfillet_web_DCR=	#N/A	0.000	0.000
DCR_colFLB1=	N/A	0.751	0.751
DCR_colFLB2=	N/A	0.820	0.820
DCR_colFLB=			0.751

Geometry/Weld Summary:

			Report:
DP used?	NO	PlugWeld?	NO
t_dblr_use=	0.625		0.000
Stiffener Req'd=	NO	NO	NO
t_stiff_Use=	#N/A	0.500	0.500
wfillet_flange=	#N/A	0.188	0.188
wfillet_web=	#N/A	0.188	0.188

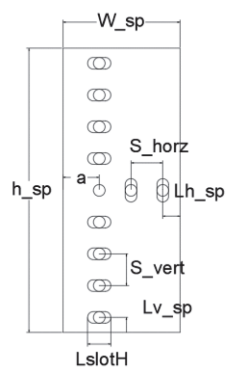
4.1 CURRENT MEMBER:

Beam Unique Name:	1001	Shear Plate Info:					
Beam Size:	W24X104	Thickness (tsp) (in):	0.50	in	Fy_bm=	50	ksi
Axial Pu (kips):	24.61	Bolt Dia (db_sp):	0.875	in	Fu_bm=	65	ksi
Shear V_Grav. SST_LC08 (kips):	14.71	No. of SP (n_sp):	1		Left Col=	W24X131	
Shear Vu_Grav. SST_LC01-07(kips):	17.73	Bolt Type:	A325-X		Right Col=	W24X162	
Link:	YL6-4.5	n_Vbolts:	5		Left_Conn=	YES	
tstem (in):	0.75	n_Hbolts:	2		Right_Conn=	YES	
		R=	8		no. of Conn =	2	

4.2 SHEAR PLATE BOLT SIZE: (AISC 358 Chapter 12, Step 15.1)

Beam Size=	W24X104		*User Input Beam size*	
d_b=	24.10	in	*From AISC Database*	
t_{bf}=	0.75	in	*From AISC Database*	
t_{bw}=	0.50	in	*From AISC Database*	
Fy_sp=	50	ksi	*User Input/Selection*	
Fu_sp =	65	ksi	*User Input/Selection*	
Axial Load (Pu_sp)=	24.61	kips	*Maximum Beam Axial Force from Overstrength Load Combinations*	
Lcc=	360	in	*Column-to-column Centerline dimension. User Input value*	
Lh=	328.25	in	*=Lcc - dc - 2*a*	
Pcap_link=	263.25	kips	*=If (R=3, Py_link, Pr_link)*	
Mcap=	6542	kip-in	*=Pcap_link*(d_b + t_{stem})*	
Vcap_link=	39.86	kips	*=Mcap'*(No. of Connections) / Lh*	
Vertical Load (Vu_bm)=	54.57	kips	*= Vcap_link + Vbm_gravity*	(EQ 12.9-34)
No. of vertical Bolts (n_Vbolts)=	5.000		*Looked up value*	
No. of horizontal Bolts (n_Hbolts)=	2.000		*User Input*	
Vu_bolt=	16.45	kips	*= Max(Sqrt ((Pu_sp/ n_Hbolt)^2 + (Vu_bm/ n_Vbolt)^2), Vu_grav.SSTLC01-07/n_Vbolts)*	(EQ 12.9-35)
ϕbolt=	0.75			
Bolt Type (Bolt_Gr_shearTab) =	A325-X			
Bolt Dia (db_sp)=	7/8	in		
Fnv_A325N=	68.00	ksi		
Anb_sp=	0.601	in^2	*= Pi* (db_sp/2)^2*	
Rn_stBolt_shear=	40.89	kips	*= Fnv_A325N* Anb_sb*	
ϕRn_stBolt_Shear=	30.67	kips	*= Rn_stBolt_shear* ϕbolt * n_sp*	
DCR_shearTab_Bolt=	0.536		*= Vu_bolt/ ϕRn_stBolt_shear*	
Check=	OK		*=OK if DCR_shearTab_Bolt < DCR_allowed*	

4.3 SHEAR PLATE GEOMETRY: (AISC 358 Chapter 12, Step 15.2)



a=	3.500	in	*Value previously defined*
tsp=	0.500	in	*[default to try and match beam web thickness]*
db =	24.10	in	*Value previously defined*
Bolt Spacing (S_min)=	2.000	in	*= 2.0*$d_{b\_sp\text{-}hole}$ (clear distance not less than $d_{b\_sp\text{-}hole}$), see section J3.3*
S_max=	3.275	in	
Bolt Spacing (Svert)=	2.750	in	OK
Bolt Spacing (Shorz)=	2.750	in	
h_flange =	9.25	in	*Value previously defined*
t_stem =	0.750	in	*Value previously defined*
h_clear =	0.125	in	*minimum clearance between link flange and shear tab*
Bolt Vert. Edge dist (Lv_min)=	1.125	in	*Minimum edge distance equal to one bolt diameter per AISC Table J3.4 note [a]*
Lv_sp=	1.125	in	
h_sp max =	15.35	in	*= db + tstem - h_flange - (2*h_clear)*
Plate Depth (h_sp)=	13.25	in	OK *= (n_Vbolt - 1)*Svert + Lv_sp* 2 <= DCR_Plate Depth*
Bolt Horz. Edge dist (Lh_min)=	1.125	in	*Minimum edge distance for 7/8" bolts per AISC Table J3.4*
Lh_sp=	1.750	in	OK
Plate Width (W_sp) =	8.00	in	*= (n_Hbolt_SST - 1)*Shorz + Lh_sp + a*
Lslot_min=	1.770	in	*= db_sp + 1/8 + (2*0.07* Svert*((n_Vbolts - 1)/2)*
LslotH = LslotV =	1.875	in	OK *OK if Lslot_min/ Lslot <= 1.03*
Shear Plate Geometry Check=	OK		*OK if And (h_sp=OK, Lh_sp = OK, Lslot = OK)*

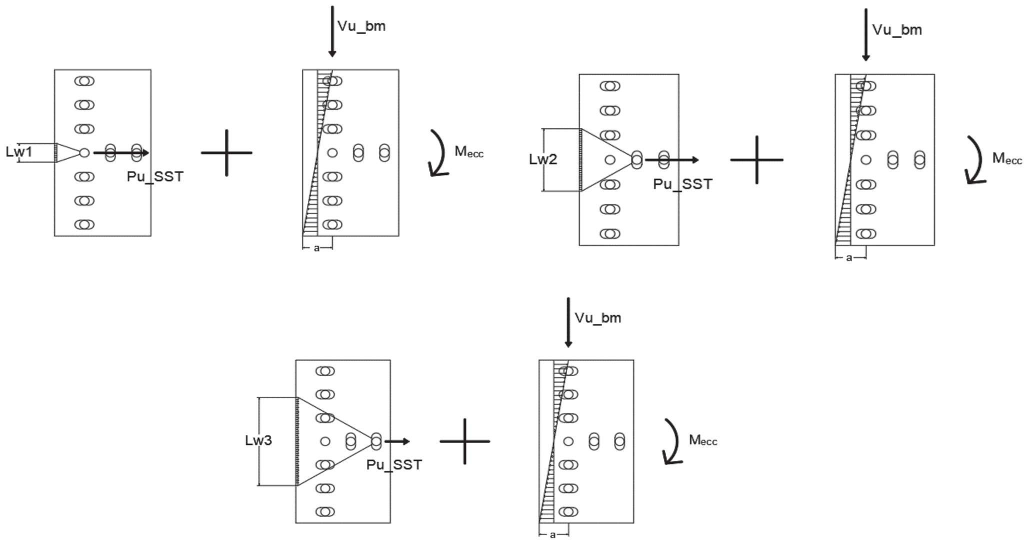
4.4 SHEAR PLATE YIELDING (VERTICAL): (AISC 358 Chapter 12, Step 15.3)

ϕyield=	1		
dHole_sp=	1.00	in	= db_sp + 1/8
Asp_Agv=	6.625	in^2	= tsp* h_sp
ϕVy_sp=	198.75	kips	= ϕyield* 0.6* Asp_Agv* Fy_sp *n_sp
DCR_spYield=	0.275		= Vu_bm/ ϕVy_sp
Check=	OK		OK if DCR_spYield <= DCR_spYield_allowed

4.5 SHEAR PLATE RUPTURE (VERTICAL): (AISC 358 Chapter 12, Step 15.3)

ϕrupture=	0.75		
Asp_nv=	4.13	in^2	= hsp* tsp - dHole_sp* n_Vbolts* tsp
ϕVrupture_sp=	120.66	kips	= ϕrupture* 0.6* Fu_shearTab* Asp_nv *n_sp
DCR_spRupture=	0.452		= Vu_bm/ ϕVrupture_sp
Check=	OK		OK if DCR_spRupture <= DCR_spRupture_allowed

4.6 SHEAR PLATE CHECK FOR AXIAL AND MOMENT: (AISC 358 Chapter 12, Step 15)



ϕb=	0.90		
a=	3.50	in	Value previously defined
Vuy=	54.57	kips	= Vu_bm
Mecc=	191.00	kip-in	= Vuy* a
$\theta_{whitmore}$=	30	deg	
Lwhitmore1 =	4.916	in	= tan ($\theta_{whitmore}$)* a* 2 + db_sp
Awhitmore1 =	2.458	in^2	= Lwhitmore1* tsp *n_sp
Lwhitmore2 =	8.092	in	= min(tan ($\theta_{whitmore}$)* (a+Shorz)*2 + db_sp, hsp)
Awhitmore2 =	4.046	in^2	= Lwhitmore2* tsp *n_sp
Lwhitmore3 =	11.267	in	= min(tan ($\theta_{whitmore}$)* (a+2*Shorz)*2 + db_sp, hsp)
Awhitmore3 =	5.634	in^2	= Lwhitmore3* tsp *n_sp
Lwhitmore4 =	13.250	in	= min(tan ($\theta_{whitmore}$)* (a+3*Shorz)*2 + db_sp, hsp)
Awhitmore4 =	6.625	in^2	= Lwhitmore4* tsp *n_sp
Ssp=	14.630	in^3	= (tsp* hsp^2 *n_sp) / 6
Isp=	96.925	in^4	= (tsp* hsp^3 *n_sp) / 12
fb1=	13.055	ksi	= Mecc / Ssp
Awhitmore=	4.046	in^2	= IF(n_Hbolts=1, Awhitmore1, IF(n_Hbolts=2, Awhitmore2, IF(n_Hbolts=3, Awhitmore3, Awhitemore4)))
Lwhitmore=	8.092	in	= IF(n_Hbolts=1, Lwhitmore1, IF(n_Hbolts=2, Lwhitmore2, IF(n_Hbolts=3, Lwhitmore3, Lwhitemore4)))
yb=	4.046	in	= Lwhitmore/ 2
fb2=	7.973	ksi	= Mecc* yb/ Isp
fa2=	6.082	ksi	= Pu_sp/ Awhitmore
ftot2=	14.055	ksi	= fa2 + fb2
fmax_sp=	14.055	ksi	= max (fb1, ftot2)
ϕb*Fy_sp=	45.000	ksi	= ϕb* Fy_sp
DCR_sp=	0.312		= fmax_sp/ (ϕb* Fy_sp)
Check=	OK		= OK if DCR_sp <= DCR_sp_allowed

4.7 SHEAR PLATE TO COLUMN FLANGE FILLET WELD (Plate 1): (AISC 358 Chapter 12, Step 15.4)

ϕ=	0.75		
tsp=	0.500	in	*Value previously defined*
tw_sp_min=	0.313	in	*= 5/8*tsp (Fillet size required to develop plate capacity as per AISC Steel Manual 14th edition)*
tw_sp=	0.313	in	*= CEILING(tw_sp_min, 1/16)*
Check=	OK		*= OK if DCR_tw_spWeld <= DCR_tw_spWeld_allowed*
Vu_bm=	54.571	kips	*Value previously defined*
Lweld=	12.750	in	*= Length per side assuming 1/4" hold off top and bottom*
F_{EXX}=	70.00	ksi	*= Filler metal classification strength*
Fnw=	42.000	ksi	*= 0.6*F_{EXX}*
Awe=	5.634	in^2	*= 0.707*(2*Lweld)*tw_sp*
ϕRn=	177.468	kips	*= ϕ*Fnw*Awe*
DCR_spWeld=	0.307		*= Vu_bm / ϕRn*
Check=	OK		*= OK if DCR_spWeld <= DCR_spWeld_allowed*

4.7a SHEAR PLATE TO COLUMN FLANGE PJP WELD (Plate 2): NOT APPLICABLE FOR ONE SIDE SHEAR PLATE

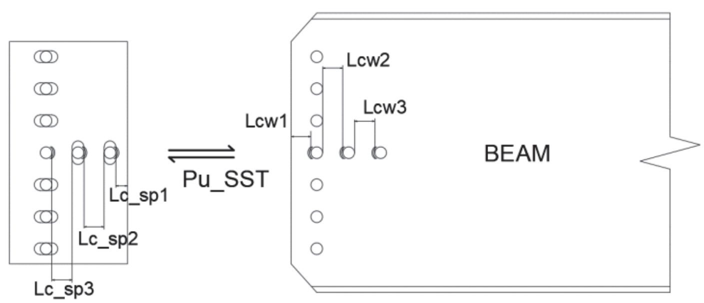
tsp=	0.500	in	*Same thickness for both shear plates*
ϕ_{PJP}=	0.75		
nsides=	1		
t_{eff_PJP}=	0.375	in	*= 3/4*tsp*
Vu_PJP=	27.285	kips	*= Vu_bm / 2*
Aw=	4.781	in^2	*= t_{eff_PJP} * Lweld *nsides*
ϕRn=	150.609	kips	*= ϕ_{PJP}*0.6*F_{EXX}*Aw*
DCR_PJPWeld1=	0.181		*= Vu_PJP / ϕRn*

PJP weld P+M Check:

ϕ=	0.8		
A_{PJP}=	4.969	in	*= t_{eff_PJP} * h_sp*
I_{PJP}=	72.694	in^4	*= t_{eff_PJP} * h_sp^3/12*
f_{PJP}=	7.791	ksi	*=0.5* Pu_sp/ A_{PJP} + 0.5* Mecc* yb/ I_{PJP}*
f_{PJP_allow}=	33.600	ksi	*= ϕ*0.6*F_{EXX}*
DCR_PJPWeld2=	0.232		*= Vu_PJP / ϕRn*
DCR_PJPWeld=	0.232		*= max(DCR_PJPWeld1,DCR_PJPWeld2)*
Check=	OK		*= OK if DCR_PJPWeld <= DCR_PJPWeld_allowed*

4.8 BEAM WEB AND SHEAR TAB BEARING: (AISC 358 Chapter 12, Step 15.5)

CASE 1: HORIZONTAL REACTIONS



BEAM WEB:

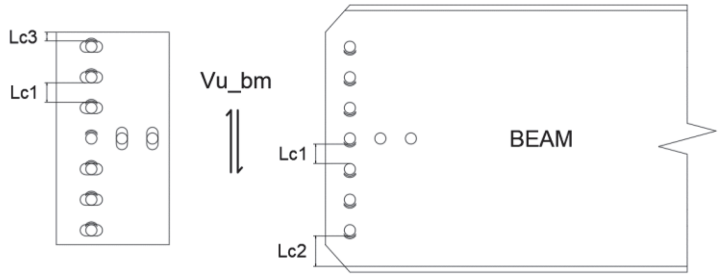
ϕbolt=	0.75		*Value previously defined*
t_{bw}=	0.50	in	*Value previously defined*
Lb_edge=	1.75	in	*Edge distance for beam web bolt hole*
Lcw1=	1.281	in	*= Lb_edge - (db_sp + 1/16)* 0.5*
Lcw2=	1.813	in	*= Shorz - (db_sp + 1/16)*
Lcw3=	0.000	in	*= if (n_Hbot_SST = 2, 0, Lcw2)*
Lcw4=	0.000	in	*= if (n_Hbot_SST = 3, 0, Lcw3)*
Lc_bmWeb=	3.094	in	*= Lcw1 + Lcw2 + Lcw3 + Lcw4*
ϕRn_beamWeb1=	90.49	kips	*= ϕbolt* 1.2* Lc_bmWeb* tbw* Fu_bm*
ϕRn_beamWeb2=	102.38	kips	*= fbolt* 2.4* db_sp* tbw* Fu_bm *n_Hbolts*
ϕRn_beamWeb=	90.49	kips	*= min (ϕRn_beamWeb1, ϕRn_beamWeb2)*
DCR_bmWebX=	0.272		*= Pu_sp/ fRn_beamWeb*
Check=	OK		*= OK if DCR_bmWebX <= DCR_bmWebX_allowed*

SHEAR PLATE:

tsp=	1/2	in	*Value previously defined*
Lsp_edge =	1 3/4	in	*Edge distance for shear tab bolt hole*
Lc_sp1 =	1.281	in	*= Lsp_edge - (db_sp + 1/16)* 0.5*
Lc_sp2 =	1.813	in	*= Shorz - (db_sp + 1/16)*
Lc_sp3 =	0.000	in	*= if (n_Hbolt_SST = 2, 0, Lc_sp2)*
Lc_sp4 =	0.000	in	*= if (n_Hbolt_SST = 3, 0, Lc_sp3)*

Lc_sp=	3.094	in	= Lc_sp1 + Lc_sp2 + Lc_sp3 + Lc_sp4
φRn_sp1=	84.25	kips	=IF(n_Hbolt_SST=1, φbolt*(1.2* Lc_sp1 * tsp* Fu_sp *n_sp), φbolt*(1.2* Lc_sp2+1.0*(Lc_sp1+Lc_sp3+Lc_sp4)) * tsp* Fu_sp *n_sp)
φRn_sp2=	93.84	kips	= φbolt*(2.4*1+2.0*(n_Hbolts-1))* db_sp* tsp* Fu_sp *n_sp
φRn_sp=	84.25	kips	= min (φRn_sp1, φRn_sp2)
DCR_spX=	0.292		= Pu_sp/ fRn_sp
Check=	OK		= OK if DCR_spX <= DCR_spX_allowed

CASE 2: VERTICAL REACTIONS



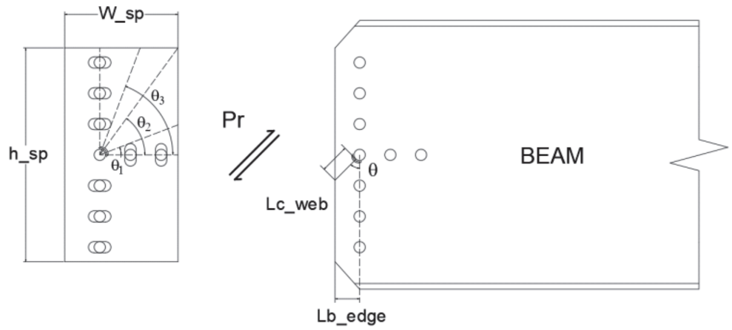
BEAM WEB:

ϕbolt=	0.75		*Value previously defined*
Lc1=	1.813	in	*= Svert - (db_sp + 1/16)*
Lc2=	5.800	in	*= (db - (n_Vbolts - 1)*Svert - 2tbf)*0.5*
Lcb_Vert=	13.050	in	*= Lc2 + (n_Vbolts - 1)*Lc1*
ϕ*Rn_bmWebY1=	381.71	kips	*= ϕbolt* 1.2* Lcb_Vert* tbw* Fu_bm*
ϕ*Rn_bmWebY2=	255.94	kips	*= ϕbolt* 2.4* (db_sp* n_bolts_SST)* tbw* Fu_bm*
ϕ*Rn_bmWebY=	255.94	kips	*= min (ϕ*Rn_bmWebY1, ϕ*Rn_bmWebY2)*
DCR_bmWebY=	**0.213**		*= Vu_bm / (ϕ* Rn_bmWebY)*
Check=	**OK**		*= OK if DCR_bmWebY <= DCR_bmWebY_allowed*

SHEAR PLATE:

Lc3=	0.656	in	*= Lv_sp - (db_sp + 1/16)* 0.5*
Lct_Vert=	7.906	in	*= Lc3 + (n_Vbolts - 1)*Lc1*
ϕ*Rn_spY1=	201.55	kips	*= ϕbolt* (1.2*Lc1+ 1.0*(Lc3 + (n_Vbolts - 2))*Lc1)*tsp* Fu_sp *n_sp*
ϕ*Rn_spY2=	221.81	kips	*= ϕbolt* (2.4*1+2.0*(n_bolts_SST-1))* db_sp* tsp* Fu_sp *n_sp*
ϕ*Rn_spY=	201.55	kips	*= min (ϕ*Rn_spY1, ϕ*Rn_spY2)*
DCR_spY=	**0.271**		*= Vu_bm / (ϕ* Rn_spY)*
Check=	**OK**		*= OK if DCR_spY <= DCR_spY_allowed*

CASE 3: COMBINED AXIAL AND VERTICAL REACTIONS



Px=	12.30	kips	= Pu_sp/ n_Hbolts
Py=	10.91	kips	= Vu_bm/ n_Vbolts
Resultant (Pr)=	16.45	kips	= Sqrt (Px^2 + Py^2)
θ=	0.726	radians	= min(Atan (Py/ Px),1.571))

BEAM WEB:

Method 1: T and V Circular Interaction

DCR_bmWebθBearing1 =	0.119		= DCR_bmWebX^2 + DCR_bmWebY^2

Method 2: Bearing in a Diagonal Line

Lvg_web =	2.339	in	= Lb_edge/ (cos θ)
Lc_web =	1.839	in	= Lvg_web - dHole_sp/2
ϕ*Rn_bmWebθ1=	53.799	kips	= ϕbolt* 1.2* Lc_web* tbw* Fu_bm
ϕ*Rn_bmWebθ2=	102.375	kips	= fbolt* 2.4* db_sp* tbw* Fu_bm*n_Hbolts
ϕ*Rn_bmWebθ=	53.80	kips	= min (ϕ* Rn_bmWeb θ1, ϕ* Rn_bmWeb θ2)
DCR_bmWebθBearing2 =	0.306		= Pr / ϕ*Rn_bmWeb θ
DCR_bmWebθBearing =	0.306		= Min (DCR_bmWeb θBearing1, DCR_bmWeb θBearing2)
	OK		= OK if DCR_bmWeb θ <= DCR_bmWeb θ_allowed

SHEAR PLATE:

Method 1: T and V Circular Interaction

DCR_spθBearing1 =	0.159		= DCR_spX^2 + DCR_spY^2

Method 2: Bearing in a Diagonal Line

θ1=	0.3286	radians	= arctan((0.5*LslotH)/Shorz)
θ2=	0.9741	radians	= arctan((0.5*h_sp)/(w_sp-a))
θ'=	0.3286	radians	= arctan((0.5*LslotV)/Svert)
θ3=	1.2422	radians	= (PI()*0.5)- θ'
θ=	0.7256	radians	Value previously defined

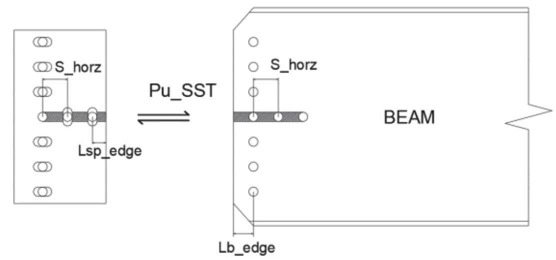
	CASE 1	CASE 2	CASE 3	CASE 4	
min. θ=	0	θ1	θ2	θ3	radians
max θ=	θ1	θ2	θ3	1.571	radians
Check=	NO	YES	NO	NO	
La_sp1=	-	6.015	-	-	in

La_sp=	6.015	in	
ϕ*Rn_spθ1=	175.947	kips	= ϕbolt* 1.2* La_sp* tsp* Fu_sp *n_sp
ϕ*Rn_spθ2=	102.375	kips	= fbolt* 2.4* db_sp* tsp* Fu_sp*n_Hbolts *n_sp
ϕ*Rn_spθ=	102.38	kips	= min (ϕ* Rn_sp θ1, ϕ* Rn_sp θ2)
DCR_spθbearing2=	0.161		= Pr / (ϕ* Rn_sp θ)
DCR_spθbearing=	0.161		= max(DCR_sp θBearing1, DCR_sp θbearing2)
Check=	OK		OK if DCR_sp θbearing <= DCR_sp θbearing_allowed

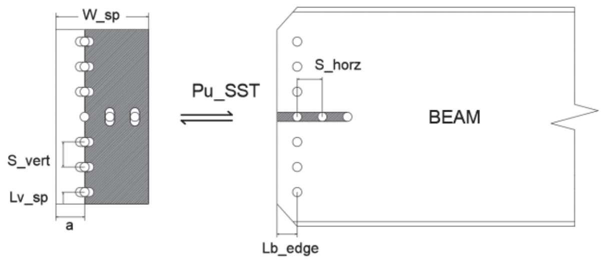
4.9 BEAM WEB AND SHEAR TAB BLOCKSHEAR CHECK: (AISC 358 Chapter 12, Step 15.5)

CASE 1: HORIZONTAL REACTIONS

CASE 1



CASE 2



BEAM WEB:

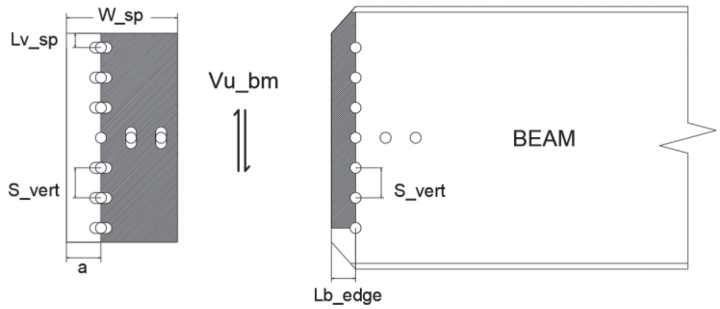
ϕblockshear=	0.75		
Ubs=	1.00		
Lb_edge=	1.750	in	*value previously defined*
Lc_bmWeb =	3.094	in	*value previously defined*
Lh_bmWeb=	9.000	in	*= 2* (Lb_edge+Shorz*(n_Hbolt_SST - 1))*
Agv_bmWebHorz=	4.500	in^2	*= Lh_bmWeb*tbw*
Ant_bmWebHorz=	0.000	in^2	*= 0*
Anv_bmWebHorz=	4.500	in^2	*= Agv_bmWebHorz*
Rn_bmWebHorz1=	175.50	kips	*= 0.6* Fu_bmWeb* Anv_bmWebHorz + Ubs* Fu_bmWeb* Ant_bmWebHorz*
Rn_bmWebHorz2=	135.00	kips	*= 0.6* Fy_bmWeb* Agv_bmWebHorz + Ubs* Fu_bmWeb* Ant_bmWebHorz*
Rn_bmWebHorz=	101.25	kips	*= ϕblocksher* MIN (Rn_bmWebHorz1, Rn_bmWebHorz2)*
DCR_bmWebHorz_BS=	0.243		*= Pu_sp/ Rn_bmWebHorz*
Check=	OK		*= OK if DCR_bmWebHorz_BS <= DCR_bmWebHorz_BS_allowed*

SHEAR PLATE:

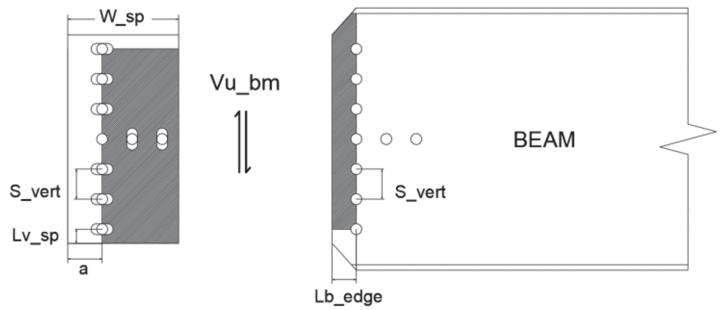
CASE 1:			
Lsp_edge=	1.750	in	*value previously defined*
Lc_sp =	3.094	in	*value previously defined*
Lh_sp1=	9.000	in	*= 2* (Lsp_edge + Shorz*(n_Hbolt_SST - 1))*
Agv_spHorz1=	4.500	in^2	*= Lh_sp1* tsp *n_sp*
Ant_spHorz1=	0.000	in^2	*= 0*
Anv_spHorz1=	4.500	in^2	*= Agv_spHorz1*
Rn_spHorz1=	175.50	kips	*= 0.6* Fu_sp* Anv_spHorz1 + Ubs* Fu_sp* Ant_spHorz1*
Rn_spHorz2=	135.00	kips	*= 0.6* Fy_sp* Agv_spHorz1 + Ubs* Fu_sp* Ant_spHorz1*
ϕRn_spHorz1=	101.25	kips	*= ϕblockshear* min (Rn_spHorz1, Rn_spbHorz2)*
DCR_spHorz_BS1=	0.243		*= Pu_sp/ Rn_spHorz1*
CASE 2:			
Lh_sp2=	8.250	in	*=h_sp-(n_Vbolts*dHole_sp)*
Agv_spHorz2=	0.000	in^2	*= 0*
Ant_spHorz2=	4.125	in^3	*= Lh_sp2 *tsp *n_sp*
Anv_spHorz2=	0.000	in^2	*= 0*
Rn_spHorz3=	268.13	kips	*= 0.6* Fu_sp* Anv_spHorz2 + Ubs* Fu_sp* Ant_spHorz2*
Rn_spHorz4=	268.13	kips	*= 0.6* Fy_sp* Agv_spHorz2 + Ubs* Fu_sp* Ant_spHorz2*
ϕRn_spHorz2=	201.09	kips	*= ϕblockshear* min (Rn_spHorz3, Rn_spbHorz4)*
DCR_spHorz_BS2=	0.122		*= Pu_sp/ Rn_spHorz2*
DCR_spHorz_BS=	0.243		*= MAX(DCR_spHorz_BS1, DCR_spHorz_BS2)*
Check=	OK		*= OK if DCR_spHorz_BS <= DCR_spHorz_BS_allowed*

CASE 2: VERTICAL REACTIONS

CASE 1



CASE 2



BEAM WEB:

S_vert=	2.75	in	*value previously defined*
Lb_edge=	1.75	in	*value previously defined*
h_bmWeb=	16.800	in	*= (d- (n_Vbolt - 1)*Svert - 2tbf)*0.5 + 2*(n_Vbolt - 1)*Svert)*
Agv_bmWeb1=	8.400	in^2	*= tbw* h_bmWeb*
Anv_bmWeb1=	6.150	in^2	*= tbw* (h_bmWeb - (n_Vbolt-0.5)* dHole_sp)*
Ant_bmWeb1=	0.625	in^2	*= (Lb_edge - dHole_sp/2)* tbw*
Rn_bmWebVert1=	280.475	kips	*= 0.6* Fu_bmWeb* Anv_bmWeb1 + Ubs*Fu_bmWeb* Ant_bmWeb1*
Rn_bmWebVert2=	292.625	kips	*= 0.6* Fy_bmWeb* Agv_bmWeb1 + Ubs*Fu_bmWeb* Ant_bmWeb1*
ϕRn_bmWebVert=	210.36	kips	*= ϕblockshear* min (Rn_bmWebVert1, Rn_bmWebVert2)*
DCR_bmWebVert_BS=	**0.259**		*= Vu_bm/ ϕRn_bmWebVert*
Check=	**OK**		*= OK if DCR_bmWebVert_BS <= DCR_bmWebVert_BS_allowed*

SHEAR PLATE:

CASE 1:

hsp=	13.250	in	*value previously defined*
Agv_sp1=	6.625	in^2	*= hsp* tsp *n_sp*
Anv_sp1=	4.125	in^2	*= tsp* (hsp - (n_Vbolt)* dHole_sp) *n_sp*
Ant_sp1=	0	in	*= 0*
Rn_spVert1=	160.88	kips	*= 0.6* Fu_sp* Anv_sp1 + Ubs* Fu_sp* Ant_sp1*
Rn_spVert2=	198.75	kips	*= 0.6* Fy_sp* Agv_sp1 + Ubs* Fu_sp* Ant_sp1*
ϕRn_spVert=	120.66	kips	*= ϕblockshear* min (Rn_spVert1, Rn_spVert2)*
DCR_spVert_BS1=	**0.452**		*= Vu_bm/ ϕRn_spVert*

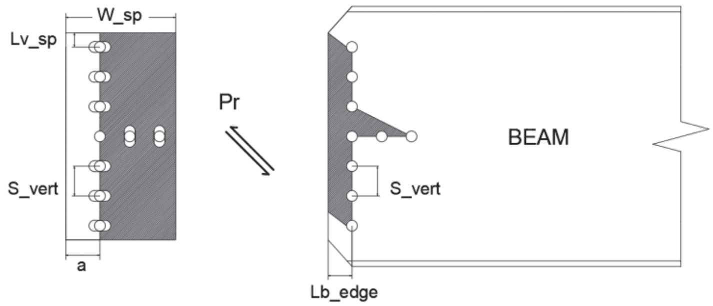
CASE 2:

Lv_sp=	1.125	in	*value previously defined*
hsp=	13.250	in	*value previously defined*
Lslot=	1.875	in	*value previously defined*
Agv_sp1=	5.813	in^2	*= (hsp-Lv_sp-(0.5*dHole_sp))* tsp *n_sp*
Anv_sp1=	4.375	in^2	*= tsp* (hsp - (n_Vbolt-0.5)* dHole_sp) *n_sp*
Ant_sp1=	1.781	in^2	*= (W_sp-a-(0.5*LslotH)* tsp *n_sp*
Rn_spVert1=	286.406	kips	*= 0.6* Fu_sp* Anv_sp1 + Ubs* Fu_sp* Ant_sp1*
Rn_spVert2=	290.156	kips	*= 0.6* Fy_sp* Agv_sp1 + Ubs* Fu_sp* Ant_sp1*
ϕRn_spVert=	214.80	kips	*= ϕblockshear* min (Rn_spVert1, Rn_spVert2)*
DCR_spVert_BS2=	**0.254**		*= Vu_bm/ ϕRn_spVert*

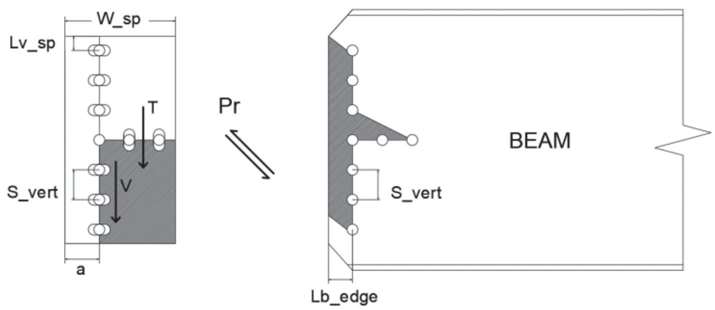
DCR_spVert_BS=	**0.452**		*= MAX(DCR_spVert_BS1, DCR_spVert_BS1)*
Check=	**OK**		*= OK if DCR_spVert_BS <= DCR_spVert_BS_allowed*

CASE 3: COMBINED AXIAL AND VERTICAL REACTIONS

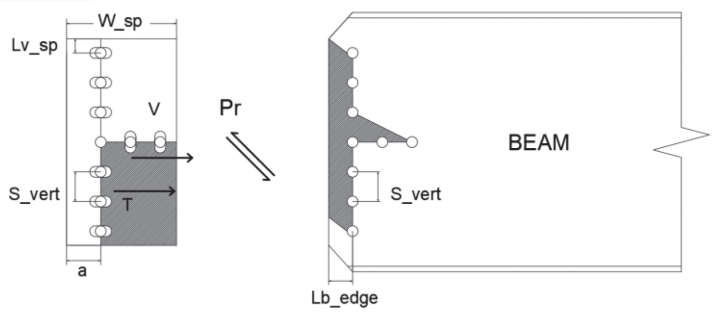
CASE 1



CASE 2



CASE 3



BEAM WEB:

Method 1: T and V Circular Interaction

DCR_bmWebθ_BS1 =	0.126		$= DCR\_bmWebHorz\_BS^2 + DCR\_bmWebVert\_BS^2$

Method 2: Failure along Hole Edges

Pr_BS=	59.86	kips	=SQRT((Pu_sp^2)+(Vu_bm)^2)
θ=	0.726	radians	*value previously defined*
Lb_edge=	1.750	in	*value previously defined*
Lbw_diag=	10.14	in	*min(sqrt(Svert^2 +Shorz^2) + Lb_edge, (Lb_edge + Shorz)/cos θ)*
Lvg_web=	10.14	in	*= 2*Lb_edge + (n_Hbolt_SST - 1)*Shorz + sqrt(Svert^2 + ((n_Hbolt_SST - 1)*Shorz))^2)*
Lvn_web=	7.14	in	*= Lvg_web - 2*dHole_sp - (n_Hbolt_SST - 1)*dHole_sp*
Ltg_web=	8.25	in	*= (n_Vbolt - 2)*Svert*
Ltn_web=	5.25	in	*= Ltg_web - (n_Vbolt - 2)*dHole_sp*
Agv_web=	5.07	in^2	*= Lvg_web* tbw*
Anv_web=	3.57	in^2	*= Lvn_web* tbw*
Agt_web=	4.13	in^2	*= Ltg_web* tbw*
Ant_web=	2.63	in^2	*= Ltn_web* tbw*
Rn_bmWeb_BS1=	309.84	kips	*= 0.6* Fu_bmWeb* Agv_web + Ubs* Fu_bmWeb* Ant_web*
Rn_bmWeb_BS2=	322.71	kips	*= 0.6* Fy_bmWeb* Agv_web + Ubs* Fu_bmWeb* Ant_web*
ϕRn_bmWeb_BS=	232.38	kips	*= ϕblockshear* min (Rn_bmWeb_BS1, Rn_bmWeb_BS2)*
DCR_bmWebθ_BS2=	0.258		*= Pr_BS / ϕRn_bmWeb_BS*
DCR_bmWebq_BS=	**0.258**		*= max(DCR_bmWeb θ_BS1, DCR_bmWeb θ_BS2)*
Check=	**OK**		*= OK if DCR_bmWeb θ_BS <= DCR_bmWeb θ_BS_allowed*

SHEAR PLATE:

Method 1: T and V Circular Interaction

DCR_spθ_BS1 =	0.264		*= DCR_spVert_BS^2 + DCR_spHorz_BS^2*

Method 2: Failure along Hole Edges

CASE 1:

Lvg_sp1=	13.25	in	*= h_sp*
Lvn_sp1=	8.25	in	*= Lvg_sp1 - (n_Vbolts*dHole_sp)*
Ltn_sp1=	0.00	in	*= 0*
Agv_sp1=	6.625	in	*= Lvg_sp* tsp *n_sp*
Anv_sp1=	4.125	in	*= Lvn_sp* tsp *n_sp*
Ant_sp1=	0.00	in	*= Ltn_sp* tsp *n_sp*
Rn_sp_BS1=	160.88	kips	*= 0.6* Fu_sp* Anv_sp1 + Ubs* Fu_sp* Ant_sp1*
Rn_sp_BS2=	198.75	kips	*= 0.6* Fy_sp* Agv_sp1 + Ubs* Fu_sp* Ant_sp1*
φRn_sp_BS1=	120.66	kips	*= φblockshear* min (Rn_sp_BS1, Rn_sp_BS2)*
DCR_spθ_BS2=	0.496		*= Pr_BS / φRn_sp_BS1*

CASE 2:

Pv=	32.7		=ceiling(n_vBolts/2)/n_vBolts*Vu_bm
Pr_BS1=	41.0		*=SQRT(Pv^2 + Pu_sp^2)*
Lvg_sp2=	6.63	in	*= h_sp/2*
Lvn_sp2=	4.13	in	*= Lvg_sp2 - ((n_Vbolt-1)/2)*dHole_sp) - (0.5*dHole_sp)*
Ltn_sp2=	3.00	in	*= (W_sp - a) - ((n_Hbolts-1)*dHole_sp) - (0.5*dHole_sp)*
Agv_sp2=	3.313	in	*= Lvg_sp* tsp *n_sp*
Anv_sp2=	2.063	in	*= Lvn_sp* tsp *n_sp*
Ant_sp2=	1.500	in	*= Ltn_sp* tsp *n_sp*
Rn_sp_BS3=	177.94	kips	*= 0.6* Fu_sp* Anv_sp2 + Ubs* Fu_sp* Ant_sp2*
Rn_sp_BS4=	196.88	kips	*= 0.6* Fy_sp* Agv_sp2 + Ubs* Fu_sp* Ant_sp2*
φRn_sp_BS2=	133.45	kips	*= φblockshear* min (Rn_sp_BS3, Rn_sp_BS4)*
DCR_spθ_BS3=	0.307		*= Pr_BS / φRn_sp_BS2*

CASE 3:

Lvg_sp3=	4.50	in	*= W_sp - a*
Lvn_sp3=	3.00	in	*= (W_sp - a) - ((n_Hbolts-0.5)*dHole_sp)*
Ltn_sp3=	4.13	in	*= Lvg_sp2 - (((n_Vbolt-1)/2)*dHole_sp) - (0.5*dHole_sp)*
Agv_sp3=	2.250	in	*= Lvg_sp* tsp *n_sp*
Anv_sp3=	1.500	in	*= Lvn_sp* tsp *n_sp*
Ant_sp3=	2.063	in	*= Ltn_sp* tsp *n_sp*
Rn_sp_BS5=	192.56	kips	*= 0.6* Fu_sp* Anv_sp3 + Ubs* Fu_sp* Ant_sp3*
Rn_sp_BS6=	201.56	kips	*= 0.6* Fy_sp* Agv_sp3 + Ubs* Fu_sp* Ant_sp3*
φRn_sp_BS3=	144.42	kips	*= φblockshear* min (Rn_sp_BS5, Rn_sp_BS6)*
DCR_spθ_BS4=	0.284		*= Pr_BS1 / φRn_sp_BS3*

DCR_spθ_BS=	**0.496**	*= max(DCR_sp θ_BS1, DCR_sp θ_BS2, DCR_sp θ_BS3, DCR_sp θ_BS4)*
Check=	**OK**	*= OK, if DCR_sp θ_BS <= DCR_sp θ_BS_allowed*

DCR Summary:

Component	Shear	Yielding	Rupture	P+M	Bearing	BS	Max
Beam Web	-	-	-	-	0.306	0.259	0.306
Shear Plate	-	0.275	0.452	0.312	0.292	0.496	0.496
Shear Plate Bolt	0.536	-	-	-	-	-	0.536
SP Fillet Weld	0.307	-	-	-	-	-	0.307

7. AISC 360-16 Steel Beam Design

Beam ID:	1001	Beam Size= W24X104	L_b= 330.3 in		R= 8
LC:	8	Beam Type= W-Section	Link Size= YL6-4.5		

7.1. Loading

M_u=	7063	kip-in	P_u=	20.91	kip	V_u=	49.16	kip
M_{max}=	7063	kip-in	M_{cap_link}=	6542		Mcap/Mu=	0.926	=1 for wind loads, R=3
M_A=	3567	kip-in	M_B=	155	kip-in	M_C=	3160	kip-in
Beam Bracing=	None		LeftLink=	YES		RightLink=	YES	

7.2. Material properties

F_y=	50	ksi	E=	29000	ksi	A_g=	30.7	in^2
F_u=	65	ksi	G=	11200	ksi	Z_x=	289	in^3
bf=	12.8	in	r_x=	10.100	in	Z_y=	62.4	in^3
d=	24.1	in	r_y=	2.910	in	S_x=	258.00	in^3
k_{des}=	1.25	in	t_f=tfi=	0.75	in	S_{xc}=	258	in^3
I_x=	3100	in^4	t_{fe}=	0.75	in	S_{xt}=	258	in^3
I_y=	259	in^4	t_w=	0.5	in	H_z=	1.0	
J=	4.72	in^4	C_w=	35200.00	in^6	L=	330.25	in

h=	22.6	in	=d-2*$_{tf}$
h_c=	22.76	in	=2*(d-t_{fi}-(b_f*t_{fi}^2/2+t_w*(d-t_{fe}-t_{fi})*(d-t_{fe}+t_{fi})/2+b_f*t_{fe}*(d-t_{fe}/2))/A_g)
h_p=	24.10	in	=2*(d-(b_f*(t_{fe}-t_{fi})/2+t_w*(d+t_{fi}-t_{fe}))/(2*t_w))

7.3. Flange compactness check Flexure: (AISC 360 -16 Table B4.1b: Case 10 for W-sections, Case 11 for Built-up shapes)

k_c=	0.59	=max(0.35,min(4/sqrt(h/t_w),0.76)	For built-up sections
λ=	8.53	b_f/(2*t_f)	
$λ_p$=	9.15	=0.38*sqrt(E/F_y)	
F_L=	35.00	=IF(S_{xt}/S_{xc}>=0.7,0.7*F_y,max(F_y*S_{xt}/S_{xc},0.5*F_y))	
$λ_{r\_case10}$=	24.08	=1*sqrt(E/F_y)	Case 10-W-Sections
$λ_{r\_case11}$=	21.09	=0.95*sqrt(Kc*E/F_L)	Case 11 BU-Sections
$λ_r$=	24.08		
Flange:	Compact	=IF(λ<λp,"Compact",IF(AND(λ<λr,λ>λp),"Non-Compact","Slender"))	

7.3.1.a Web compactness check flexture (Table B4.1b: Case 15):

λ=	45.20	=h/t_w
$λ_p$=	90.55	=3.76*sqrt(E/F_y)
$λ_r$=	137.27	=5.70*sqrt(E/F_y)
Web:	Compact	=IF(λ<$λ_p$,"Compact",IF(AND(λ<$λ_r$,λ>$λ_p$),"Non-Compact","Slender"))

Per Table F1.1 for Flexure

Section:	F2	
$Φ_b$=	0.9	=IF(AND(Flange="Compact",Web="Compact"),"F2",IF(AND(OR(Flange="Non-Compact",Flange="Slender"),Web="Compact"), "F3",IF(AND(OR(Flange="Compact",Flange="Non-Compact",Flange="Slender"),Web="Slender"),"F5","F4")))
$Φ_c$=	0.9	
$Φ_v$=	0.9	

7.4. Flange compactness check Axial Compression: (AISC 360 -16 Table B4.1a: Case 1 for W-sections, Case 2 for Built-up shapes)

λ=	8.53	b_f/(2*t_f)
$λ_{r\_case1}$=	13.49	=0.56*sqrt(E/F_y)
k_c=	0.59	From Section 3
$λ_{r\_case2}$=	11.89	=0.64*sqrt(Kc*E/F_y)
$λ_r$=	13.49	
Flange:	Nonslender	

7.4.1.a Web compactness check Axial Compression (Table B4.1a: Case 5):

λ=	45.20	=h/t_w
$λ_r$=	35.88	=1.49*sqrt(E/F_y)
Web:	Slender	

7.5. Flexural Capacity

7.5.1. Yielding (F2.1)

$M_{n\_F2.1}$=	14450	kip-in	$=M_p=F_y * Z_x$	(F2-1)

7.5.2. Lateral-torsional buckling (F2.2)

L_b=	330.25	in	Lateral torsional bracing length	
L_p=	123.34	in	$=1.76*r_y*sqrt(E/F_y)$	(F2-5)
r_{ts}=	3.42	in	$=sqrt(sqrt(I_y*C_w)/S_x)$	(F2-7)
h_o=	23.350	in	$=d-t_f$ Distance between centroids	
c=	1		1 for doubly symmetic I-Shapes	(EQ F2-8a)
L_r=	350.4	in	$=1.95*r_{ts}*E/(0.7*F_y)*sqrt((J*c)/(S_x*h_o))*sqrt(1+sqrt(1+6.76*(0.7*F_y/E*S_x*h_o/(J*c))^2)$	(F2-6)
C_b=	2.30		$=12.5*M_{max}/(2.5*M_{max}+3*M_A+4*M_B+3*M_C)$	(F1-1)
F_{cr}=	88.3	ksi	$=C_b*\pi^2*E/(L_b/r_{ts})^2*sqrt(1+0.078*J_{col}*c_{col}/(S_x*h_o)*(L_b/r_{ts})^2)$	(F2-4)
$M_{n\_F2.2}$=	14450	kip-in	$=IF(L_b<=L_p,M_p,IF(AND(L_p<L_b,L_b<=L_r),MIN(C_b*(M_p-(M_p-0.7*F_y*S_x)*((L_b-L_p)/(L_r-L_p))),M_p),MIN(F_{cr}*S_x,M_p)))$	(F2-2, 2-3)

7.5.3. Lateral torsional buckling (F3.1)

$M_{n\_F3.1}$=	14450		$=M_{n\_F2.2}$	

7.5.4. Compression flange local buckling (F3.2)

K_c=	0.59		$=min(max(0.35,4/sqrt(h/t_w)),0.76)$	
λ=	8.533		$=b_f/(2*t_f)$	
λ_{pf}=	9.15		$=0.38*sqrt(E/F_y)$	
λ_{rf}=	24.1		$=1.0*sqrt(E/F_y)$	
$M_{n\_F3.2}$=	14450		=IF(Flange="Non-Compact",$M_p-(M_p-0.7*S_x*F_y)*(\lambda-\lambda_{pf})/(\lambda_{rf}-\lambda_{pf})$,IF(Flange="Slender",$0.9*E*k_c*S_x/\lambda^2,M_p$))	(F3-1, 3-2)

7.5.5. Compression flange yielding (F4.1)

h_c=	22.60	in	$=d-2*t_f$	
λ_{w_F4}=	45.20		$=h_c/t_w$	
λ_{pw}=	90.55		$=3.76*sqrt(E/F_y)$	
λ_{rw}=	137.27		$=5.7*sqrt(E/F_y)$	
M_{yc}=	12900	kip-in	$=F_y*S_{xc}$	
R_{pc}=	1.12		$=IF(h_c/t_w<=\lambda_{pw},M_p/M_{yc},MIN(M_p/M_{yc}-(M_p/M_{yc}-1)*(\lambda-\lambda_{pw})/(\lambda_{rw}-\lambda_{pw}),M_p/M_{yc}))$	
$M_{n\_F4.1}$=	14450		$=R_{pc}*My_c$	(F4-1)

7.5.6. Lateral torsional buckling (F4.2)

a_{w_F42}=	1.18		$=h_c*t_w/(b_f*t_f)$	(F4-12)
r_{t_F4}=	3.38	in	$=b_f/sqrt(12*(1+1/6*a_{w\_F42}))$	(F4-11)
L_{p_F4}=	89.50		$=1.1*r_{t\_F4}*sqrt(E/F_y)$	(F4-7)
L_{r_F4}=	346.00	in	$=1.95*r_{t\_F4}*E/F_L*sqrt(J/(S_{xc}*h_o))+sqrt((J/S_{xc}*h_o)$^2 + 6.76 * (F_L/E)^2)	(F4-8)
F_{cr_F4}=	86.54	ksi	$=C_b*\pi^2*E/(L_b/r_{t\_F4})^2*sqrt(1+0.078*J/(S_{xc}*h_o)*(L_b/r_{t\_F4})^2)$	(F4-5)
$M_{n\_F4.2}$=	14450	kip-in	$=IF(L_b<=L_p,M_p,IF(L_b>L_r,F_{cr\_F4}*S_{xc},MIN(C_b*(R_{pc}*M_{yc}-(R_{pc}*M_{yc}-F_L*S_{xc})*(L_b-L_p)/(L_r-L_p)),R_{pc}*M_{yc})))$	(F4-2, 4-3)

7.5.7. Compression flange local buckling (F4.3)

λ_{f_F43}=	8.53		$=b_f/(2*t_f)$	
λ_{pf}=	9.15		From Section 3 above	
λ_{rf}=	24.08		From Section 3 above	
M_{n_F13}=	14674		=Rpc*Myc-(Rpc*Myc-FL*Sxc)*(λf_F43-λpf)/(λrf-λpf)	(F4-13)
M_{n_F14}=	55019		$0.9*E*kc*Sxc/\lambda_{f\_F432}{}^2$	(F4-14)
M_{n_F43}=	14450	kip-in	=IF(Flange="Compact",M_p,IF(Flange="Non-Compact", M_{n_F13}, M_{n_F14})	

7.5.8. Tension flange yielding (F4.4)

λ_{F44}=	45.20		$=h_c/t_w$	
M_{yt}=	12900	kip-in	$=F_y*S_{xt}$	(F4-16a)
λ_{pw}=	91		From Section 3 above	
λ_{rw}=	137		From Section 3 above	
$R_{pt\_F4\text{-}16a}$=	1.12		$=M_p/M_{yt}$	(F4-16b)
$R_{pt\_F4\text{-}16b}$=	1.24		$=M_p/M_{yt}-(M_p/M_{yt}-1)*(\lambda_{F44}-\lambda_{pw})/(\lambda_{rw}-\lambda_{pw})$	
R_{pt}=	1.12		$=IF(\lambda_{F44}<=\lambda_{pw},R_{pt\_F4\text{-}16a}, R_{pt\_F4\_16b})$	
$M_{n\_F4.4}$=	14450	kip-in	$=R_{pt} * M_{yt}$	(F4-15)

7.5.9. Compression flange yielding (F5.1)

a_w=	1.18		$=min(10,a_{w\_F42})$	
r_{t_F5}=	3.38	in	$=r_{t\_F4}$	
R_{pg}=	1.00		$=min(1,1-a_w/(1200+300*a_w)*(h_c/t_w-5.7*sqrt(E/F_y))$	(F5-6)
$M_{n\_F5.1}$=	12900	kip-in	$=R_{pg}*F_y*S_{xc}$	(F5-1)

7.5.10. Lateral-torsional buckling (F5.2)

L_{p_F5}=	89.5	in	$=1.1*r_{t\_F5}*sqrt(E/F_y)$	(F4-7)
L_{r_F5}=	306	in	$=\pi*r_{t\_F5}*sqrt(E/0.7/F_y)$	(F5-5)
$F_{cr\_F5.3}$=	50	ksi	$=min(F_y,C_b*(F_y-0.3*F_y*(L_b-L_{p\_F5})/(L_{r\_F5}-L_{p\_F5}))$	(F5-3)
$F_{cr\_F5.4}$=	50	ksi	$=min(F_y,C_b*\pi^2*E/(L_b/r_{t\_F5})^2)$	(F5-4)
$F_{cr\_LTB\_F5}$=	50	ksi	$=IF(L_b<=L_{p\_F5},"NA",IF(L_b>L_{r\_F5},F_{cr\_F5.4},F_{cr\_F5.3}))$	
$M_{n\_F5.2}$=	12900	kip-in	$=IF(L_b<=L_{p\_F5},Mp,R_{pg}*F_{cr\_LTB\_F5}*S_{xc})$	(F5-2)

7.5.11. Compression flange local buckling (F5.3)

λ_f=	8.53		From Section 3 above	
λ_{fp}=	9.15		From Section 3 above	
λ_{fr}=	24.08		From Section 3 above	
$F_{cr\_F5\_8}$=	51	ksi	$=F_y-0.3*F_y*(\lambda_f-\lambda_{fp})/(\lambda_{fr}-\lambda_{fp})$	(F5-8)
$F_{cr\_F5\_9}$=	213	ksi	$=0.9*E*k_c/(b_f/(2*t_f))^2$	(F5-9)
F_{cr_F5}=	50	ksi	=if(Flange="Compact",50, if(Flange="Non-Compact",$F_{cr\_F5\_8}$,$F_{cr\_F5\_9}$)	
$M_{n\_F5.3}$=	12900	kip-in	$=R_{pg}*F_{cr\_F5}*S_{xc}$	(F5-7)

7.5.12. Tension flange yielding (F5.4)

$M_{n\_F5.4}$=	14450	kip-in	$=M_p$ Both flanges are symmetric	(F5-10)

7.5.13. Flange hole reduction (F13.1)

D_{HoleT}=	0.000	in		
A_{fg}=	9.60	in^2	$=b_f*t_f$	
A_{fn}=	9.60	in^2	$=t_f*(b_f-2*D_{HoleT})$	
Y_t=	1.00		$=IF(F_y/F_u<=0.8,1,1.1)$	
$M_{n\_F13.1}$=	16770	kip-in	$=F_u*A_{fn}/A_{fg}*S_x$	(F13-1)

Flexural Capacity Summary

M_{n_F2}=	14450	kip-in	$=min(M_{n\_F2.1},M_{n\_F2.2})$
M_{n_F3}=	14450	kip-in	=min(Mn_F3.1,Mn_F3.2)
M_{n_F4}=	14450	kip-in	$=min(M_{n\_F4.1},M_{n\_F4.2},M_{n\_F4.3},M_{n\_F4.4})$
M_{n_F5}=	12900	kip-in	$=min(M_{n\_F5.1},M_{n\_F5.2},M_{n\_F5.3},M_{n\_F5.4})$
M_n=	14450	kip-in	IF(B3="Beam",IF(DesignSection="F2",MIN(M_{n_F2},$M_{n\_F13.1}$), IF(DesignSection="F3",MIN(M_{n_F3},$M_{n\_F13.1}$),IF(DesignSection="F4", MIN(M_{n_F4},$M_{n\_F13.1}$),MIN(M_{n_F5},$M_{n\_F13.1}$)))),M_{n_F4})
ΦM_n=	13005	kip-in	$=\Phi_b*M_n$

7.6. Axial Capacity

Section=	E7		=IF(OR(Flange="Slender",Web="Slender"),"E7","E3")	

7.6.1. E.3 Flexural buckling (member without slender elements)

K_x=	1		K_y=	1		K_z=	1	
L_x=	330.25	in	L_y=	330.3	in	L_z=	330.3	in

K_{LR}=	113.49		=max(K_x*Lx/r_x,K_y*Ly/r_y)	
F_e=	22.22	ksi	=π^2*E/(K_{LR})^2	(E3-4)
$F_{cr\_E.3}$=	19.49	ksi	=IF(K_{LR}<=4.71*SQRT(E/F_y),F_y*0.658$^{(Fy/Fe)}$,0.877*F_e)	(E3-2, 3-3)
P_{n_E3}=	598.32	kip	=$F_{cr\_E.3}$*A_g	(E3-1)

7.6.2. E.4 Torsional and flexural-torsional buckling: (Applies to singly symmetric shapes)

F_{ey}=	22.22	ksi	=π^2*E/(K_y*L_y/r_y)^2	(E3-4)
F_{ez}=	43.24	ksi	=π^2*E*C_w/(K_z*L_z)^2+G*J)/(1/(Ix+Iy))	(E4-2)
$F_{e\_E.4}$=	22.22	ksi	=(F_{ey}+F_{ez})/(2*H_z)*(1-sqrt(1-4*F_{ey}*F_{ez}*H_z/(F_{ey}+F_{ez})^2))	(E4-3)
$F_{cr\_E.4}$=	19.49	ksi	=IF(K_{Lr}<=4.71*SQRT(E/F_y),F_y*0.658$^{(Fy/Fe_E.4)}$,0.877*$F_{e\_E.4}$)	(E3-2, 3-3)
$P_{n\_E.4}$=	598.3	kip	=A_g*$F_{cr\_E.4}$	(E4-1)

7.6.2. E.7 Flexural, torsional and flexural-torsional buckling (members with slender elements)

7.6.2a Calculation for effective flange width

Flange=	Nonslender			
λ=	8.53		From Section 3 above	
λ_r=	13.49		From Section 3 above	
c1=	0.22		AISC 360-16 Table E7.1 all elements	
c2=	1.49		AISC 360-16 Table E7.1 all elements	
F_{el}=	277.3	ksi	=(c2 * λrw/λw)^2 * Fy	
Fcr=	19.5	ksi	=Min (Fcr_E3, Fcr_E4)	
bf=	12.80	in	=Full Flange Width from AISC Database	
$b_{e\_E7\text{-}3}$=	8.2	in	=bf*(1-c1*sqrt(F_{el}/F_{cr}))*sqrt(F_{el}/F_{cr}), but can't be more than bf	(E7-3)
be=	12.80	in	=If (Flange="Nonslender", b_f, $b_{e\_E7\text{-}3}$)	
Aweb=	11.50	in^2	=Ag-2*tf*bf	
Af=	19.20	in^2	=2*tf*be	
Ae1=	30.7	in^2	=Aweb+Af	

7.6.2b Calculation for effective web depth

Web=	Slender			
λ_w=	45.20		From Section 4.1 above	
λ_{rw}=	35.88		From Section 4.1 above	
c1=	0.18		AISC 360-16 Table E7.1 stiffened elements	
c2=	1.31		AISC 360-16 Table E7.1 stiffened elements	
F_{el}=	54.1	ksi	=(c2 * λrw/λw)^2 * Fy	
h=	22.6	in		
$h_{e\_E7\text{-}3}$=	22.6	in	=h*(1-c1*sqrt(F_{el}/F_{cr}))*sqrt(F_{el}/F_{cr}), but can't be more than h	(E7-3)
h_e=	22.6	in	=If (Web="Nonslender", h, $h_{e\_E7\text{-}3}$)	
A_{flange}=	19.2	in^2	=2*bf*tf	
A_{web_radius}=	11.5	in^2	=A_g-A_{flange}	
A_{web}=	11.3	in^2	=h*t_w	
A_{radius}=	0.2	in^2	=A_{web_radius} - A_{Web}	
A_{e_web}=	11.30	in^2	=h_e * t_w	
A_{e2}=	30.7	in^2	=Aflange+Aradius+Ae_web	
A_e=	30.7	in^2	=Min (Ae1, Ae2)	
F_{cr}=	19.5	ksi		
P_{n_E7}=	598.3	kips		
P_n=	598.3	kip	=IF(AxialDesignSection="E3",P_{n_E3},P_{n_E7})	
Φ_c*P_n=	538.5	kip		

7.7. Demand per AISC 360-16 Appendix 8 (to include p-small delta)

P_y=	1535	kip	$=A_g*F_y$	
P_r=	21	kip	$=P_{u\_omega}$	
α=	1		1 for LRFD, 1.6 for ASD	
C_m=	1		conservative assumption	(A-8-5)
P_{e1y}=	8135	kip	$=\pi^2*E*I_x/(Lb)^2$	
B_1=	1.003		$=C_m/(1-\alpha*P_r/P_{e1y})$	(A-8-3)

7.8. Axial + flexural interaction (Chapter H)

M_r=	7081	kip-in	$=M_u*B_1$	
M_c=	13005	kip-in	$=\Phi*M_n$	
P_r=	20.9	kip	$=P_u$	
P_c=	538.5	kip	$=\Phi_c*P_n$	
DCR_Axial=	**0.039**		=Pr/Pc	
DCR_Flexural=	**0.544**		=Mr/Mc Adj_DCR_Flexural= **0.504** For R>3	
DCR_P+M=	**0.564**		$=IF(P_r/P_c>=0.2,P_r/P_c+8/9*M_r/M_c,P_r/(2*P_c)+M_r/M_c)$	(H1-1)
Adj P+M=	**0.524**		$=IF(P_r/P_c>=0.2,P_r/P_c+8/9*$Adj_DCR_Flexural$,P_r/(2*P_c)+$Adj_DCR_Flexural$)$	

7.9. Shear capacity (Chapter G)

Vu=	49.16	kips		
A_w=	12.05	in^2	$=t_w*(d-Hole_{web})$	
h/tw=	45			
k_v=	5.34		Webs without transverse stiffeners	
$2.24sqrt(E/F_y)$=	53.9			
C_{v1_G2-2}=	1			(G2-2)
$1.1sqrt(k_vE/F_y)$=	61.2			
C_{v1_G2-3}=	1.00			(G2-3)
C_{v1_G2-4}=	1.4		$=1.1*sqrt(k_v*E/F_y)/(h/t_w)$	(G2-4)
$C_{v1\_G2-3\_4?}$=	1.0		$=if(h/tw <= 1.1*sqrt(k_v*E/F_y), C_{v1\_G2-3}, C_{v1\_G2-4})$	
C_{v1}=	1.0		$=if(C_{v1\_G2-2}=\text{"N/A"},C_{v1\_G2-3\_4?})$	
ϕ_vV_n=	325.4	kip	$=\phi_v*0.6*F_y*A_w*C_{v1}$	(G2-1)
DCR_Shear=	**0.151**		$=V_u/(\Phi_v*V_n)$	

7.10. Design Summary:

		Demand-to-Capacity Ratios					
LC ID	Load Combination	P	M	P+M	V	Adj M	Adj P+M
LC 8	(1.2 + 0.2 SDS)DL -OmegaEL + f1*LL + f2*SL	0.039	0.544	0.564	0.151	0.504	0.524

8. AISC 360-16 Steel Column Design

Column ID:	1	Column Size=	W24X131	L_b=	144	in
LC:	7	ColumnType=	W-Section	Col Beam Bot Bracing?	YES	

8.1. Loading

M_u=	5327	kip-in	P_u=	43.0	kip		V_u=	42.1	kip
M_{max}=	5327	kip-in	M_{u_top}=	5327	kip-in	TopGov	M_{u_bot}=	0	kip-in
M_A=	1402	kip-in	M_B=	2523.2	kip-in		M_C=	4205.4	kip-in

8.2. Material properties

F_y=	50	ksi	E=	29000	ksi	A_g=	38.600	in^2
F_u=	65	ksi	G=	11200	ksi	Z_x=	370.000	in^3
bf=	12.9	in	r_x=	10.200	in	Z_y=	81.500	in^3
d=	24.5	in	r_y=	2.97	in	S_x=	329.000	in^3
k_{des}=	1.46	in	t_f=tfi=	0.96	in	S_{xc}=	329.00	in^3
I_x=	4020.0	in^4	t_{fe}=	0.96	in	S_{xt}=	329.00	in^3
I_y=	340.0	in^4	t_w=	0.605	in	H_Z=	1.0	
J=	9.500	in^4	C_w=	47100	in^6			

h=	22.58	in	$=d-2*_{tf}$
h_c=	22.69	in	$=2*(d-t_{fi}-(b_f*t_{fi}^2/2+t_w*(d-t_{fe}-t_{fi})*(d-t_{fe}+t_{fi})/2+b_f*t_{fe}*(d-t_{fe}/2))/A_g)$
h_p=	24.50	in	$=2*(d-(b_f*(t_{fe}-t_{fi})/2+t_w*(d+t_{fi}-t_{fe}))/(2*t_w))$

8.3. Flange compactness check Flexure: (AISC 360 -16 Table B4.1b: Case 10 for W-sections, Case 11 for Built-up shapes)

k_c=	0.65	$=max(0.35,min(4/sqrt(h/t_w),0.76)$	For built-up sections
λ=	6.72	$b_f/(2*t_f)$	
λ_p=	9.15	$=0.38*sqrt(E/F_y)$	
F_L=	35.00	$=IF(S_{xt}/S_{xc}>=0.7,0.7*F_y,max(F_y*S_{xt}/S_{xc},0.5*F_y))$	
λ_{r_case10}=	24.08	$=1*sqrt(E/F_y)$	Case 10-W-Sections
λ_{r_case11}=	22.13	$=0.95*sqrt(Kc*E/F_L)$	Case 11 BU-Sections
λ_r=	24.08		
Flange:	Compact	=IF(λ<λp,"Compact",IF(AND(λ<λr,λ>λp),"Non-Compact","Slender"))	

8.3.1.a Web compactness check flexture (Table B4.1b: Case 15):

λ=	37.32	$=h/t_w$
λ_p=	90.55	$=3.76*sqrt(E/F_y)$
λ_r=	137.27	$=5.70*sqrt(E/F_y)$
Web:	Compact	=IF(λ<λ_p,"Compact",IF(AND(λ<λ_r,λ>=λ_p),"Non-Compact","Slender"))

Per Table F1.1 for Flexure

Section:	F2	
Φ_b=	0.9	=IF(AND(Flange="Compact",Web="Compact"),"F2",IF(AND(OR(Flange="Non-Compact",Flange="Slender"),Web="Compact"), "F3",IF(AND(OR(Flange="Compact",Flange="Non-Compact",Flange="Slender"),Web="Slender"),"F5","F4")))
Φ_c=	0.9	
Φ_v=	0.9	

8.4. Flange compactness check Axial Compression: (AISC 360 -16 Table B4.1a: Case 1 for W-sections, Case 2 for Built-up shapes)

λ=	6.72	$b_f/(2*t_f)$
λ_{r_case1}=	13.49	$=0.56*sqrt(E/F_y)$
k_c=	0.65	From Section 3
λ_{r_case2}=	12.47	$=0.64*sqrt(Kc*E/F_y)$
λ_r=	13.49	
Flange:	Nonslender	

8.4.1.a Web compactness check Axial Compression (Table B4.1a: Case 5):

λ=	37.32	$=h/t_w$
λ_r=	35.88	$=1.49*sqrt(E/F_y)$
Web:	Slender	

8.5. Flexural Capacity

8.5.1. Yielding (F2.1)

$M_{n\_F2.1}$=	18500	kip-in	M_p=F_y * Z_x (Col braced at Bm bottom), M_y=F_y * S_x (Col not braced at Bm bottom)	(F2-1)

8.5.2. Lateral-torsional buckling (F2.2)

L_p=	125.89	in	=1.76*r_y*sqrt(E/F_y)	(F2-5)
r_{ts}=	3.49	in	=sqrt(sqrt(I_y*C_w)/S_x)	(F2-7)
h_o=	23.540	in	=d-t_f Distance between centroids	
c=	1		1 for doubly symmetic I-Shapes	(EQ F2-8a)
L_r=	382.01	in	=1.95*r_{ts}*E/(0.7*F_y)*sqrt((J*c)/(S_x*h_o))* sqrt(1+sqrt(1+6.76*(0.7*F_y/E*S_x*h_o/(J*c))^2)	(F2-6)
$C_{b\_calculated}$=	1.66		=12.5*M_{max}/(2.5*M_{max}+3*M_A+4*M_B+3*M_C)	
C_b=	1.66		=IF(Cb=$C_{b\_calculated}$,$C_{b\_calculated}$,Cb)	(F1-1)
F_{cr}=	299.7	ksi	=C_b*π^2*E/(L_b/r_{ts})^2*sqrt(1+0.078*J_{col}*c_{col}/(S_x*h_o)*(L_b/r_{ts})^2)	(F2-4)
$M_{n\_F2.2}$=	18500	kip-in	=IF(L_b<=L_p,M_p,IF(AND(L_p<L_b,L_b<=L_r),MIN(C_b*(M_p-(M_p-0.7*F_y*S_x)*((L_b-L_p)/(L_r-L_p))),M_p),MIN(F_{cr}*S_x,M_p)))	(F2-2, 2-3)

8.5.3. Lateral torsional buckling (F3.1)

$M_{n\_F3.1}$=	18500		=$M_{n\_F2.2}$	

8.5.4. Compression flange local buckling (F3.2)

K_c=	0.65		=min(max(0.35,4/sqrt(h/t_w)),0.76)	
λ=	6.719		=b_f/(2*t_f)	
λ_{pf}=	9.15		=0.38*sqrt(E/F_y)	
λ_{rf}=	24.1		=1.0*sqrt(E/F_y)	
$M_{n\_F3.2}$=	18500		=IF(Flange="Non-Compact",M_p-(M_p-0.7*S_x*F_y)*(λ-λ_{pf})/(λ_{rf}-λ_{pf}),IF(Flange="Slender",0.9*E*k_c*S_x/λ^2,M_p))	(F3-1, 3-2)

8.5.5. Compression flange yielding (F4.1)

h_c=	22.58	in	=d-2*t_f	
λ_{w_F4}=	37.32		=h_c/t_w	
λ_{pw}=	90.55		=3.76*sqrt(E/F_y)	
λ_{rw}=	137.27		=5.7*sqrt(E/F_y)	
M_{yc}=	16450	kip-in	=F_y*S_{xc}	
R_{pc}=	1.12		=IF(h_c/t_w<=λ_{pw},M_p/M_{yc},MIN(M_p/M_{yc}-(M_p/M_{yc}-1)*(λ-λ_{pw})/(λ_{rw}-λ_{pw}),M_p/M_{yc}))	
$M_{n\_F4.1}$=	18500		=R_{pc}*My_c	(F4-1)

8.5.6. Lateral torsional buckling (F4.2)

a_{w_F42}=	1.10		=h_c*t_w/(b_f*t_f)	(F4-12)
r_{t_F4}=	3.42	in	=b_f/sqrt(12*(1+1/6*a_{w_F42}))	(F4-11)
L_{p_F4}=	90.67		=1.1*r_{t_F4}*sqrt(E/F_y)	(F4-7)
L_{r_F4}=	374.88	in	=1.95*r_{t_F4}*E/F_L*sqrt(J/(S_{xc}*h_o))+sqrt((J/S_{xc}*h_o)^2 + 6.76 * (F_L/E)^2)	(F4-8)
F_{cr_F4}=	289.38	ksi	=C_b*π^2*E/(L_b/r_{t_F4})^2*sqrt(1+0.078*J/(S_{xc}*h_o)*(L_b/r_{t_F4})^2)	(F4-5)
$M_{n\_F4.2}$=	18500	kip-in	=IF(L_b<=L_p,M_p,IF(L_b>L_r,F_{cr_F4}*S_{xc}, MIN(C_b*(R_{pc}*M_{yc}-(R_{pc}*M_{yc}-F_L*S_{xc})*(L_b-L_p)/(L_r-L_p)),R_{pc}*M_{yc})))	(F4-2, 4-3)

8.5.7. Compression flange local buckling (F4.3)

λ_{f_F43}=	6.72		=b_f/(2*t_f)	
λ_{pf}=	9.15		From Section 3 above	
λ_{rf}=	24.08		From Section 3 above	
M_{n_F13}=	19638		=Rpc*Myc-(Rpc*Myc-FL*Sxc)*(λf_F43-λpf)/(λrf-λpf)	(F4-13)
M_{n_F14}=	124548		0.9*E*kc*Sxc/$\lambda_{f\_F432}^{\ 2}$	(F4-14)
M_{n_F43}=	18500	kip-in	=IF(Flange="Compact",M_p,IF(Flange="Non-Compact", M_{n_F13}, M_{n_F14})	

8.5.8. Tension flange yielding (F4.4)

λ_{F44}=	37.32		$=h_c/t_w$	
M_{yt}=	16450	kip-in	$=F_y*S_{xt}$	(F4-16a)
λ_{pw}=	91		From Section 3 above	
λ_{rw}=	137		From Section 3 above	
$R_{pt\_F4\text{-}16a}$=	1.12		$=M_p/M_{yt}$	(F4-16b)
$R_{pt\_F4\text{-}16b}$=	1.27		$=M_p/M_{yt}-(M_p/M_{yt}-1)*(\lambda_{F44}-\lambda_{pw})/(\lambda_{rw}-\lambda_{pw})$	
R_{pt}=	1.12		$=IF(\lambda_{F44}<=\lambda_{pw},R_{pt\_F4\text{-}16a}, R_{pt\_F4\_16b})$	
$M_{n\_F4.4}$=	18500	kip-in	$=R_{pt} * M_{yt}$	(F4-15)

8.5.9. Compression flange yielding (F5.1)

a_w=	1.10		$=min(10,a_{w\_F42})$	
r_{t_F5}=	3.42	in	$=r_{t\_F4}$	
R_{pg}=	1.00		$=min(1,1-a_w/(1200+300*a_w)*(h_c/t_w-5.7*sqrt(E/F_y))$	(F5-6)
$M_{n\_F5.1}$=	16450	kip-in	$=R_{pg}*F_y*S_{xc}$	(F5-1)

8.5.10. Lateral-torsional buckling (F5.2)

L_{p_F5}=	90.7	in	$=1.1*r_{t\_F5}*sqrt(E/F_y)$	(F4-7)
L_{r_F5}=	310	in	$=\pi*r_{t\_F5}*sqrt(E/0.7/F_y)$	(F5-5)
$F_{cr\_F5.3}$=	50	ksi	$=min(F_y,C_b*(F_y-0.3*F_y*(L_b-L_{p\_F5})/(L_{r\_F5}-L_{p\_F5}))$	(F5-3)
$F_{cr\_F5.4}$=	50	ksi	$=min(F_y,C_b*\pi^2*E/(L_b/r_{t\_F5})^2)$	(F5-4)
$F_{cr\_LTB\_F5}$=	50	ksi	$=IF(L_b<=L_{p\_F5},\text{"NA"},IF(L_b>L_{r\_F5},F_{cr\_F5.4},F_{cr\_F5.3}))$	
$M_{n\_F5.2}$=	16450	kip-in	$=IF(L_b<=L_{p\_F5},Mp,R_{pg}*F_{cr\_LTB\_F5}*S_{xc})$	(F5-2)

8.5.11. Compression flange local buckling (F5.3)

λ_f=	6.72		From Section 3 above	
λ_{fp}=	9.15		From Section 3 above	
λ_{fr}=	24.08		From Section 3 above	
$F_{cr\_F5\_8}$=	52	ksi	$=F_y-0.3*F_y*(\lambda_f-\lambda_{fp})/(\lambda_{fr}-\lambda_{fp})$	(F5-8)
$F_{cr\_F5\_9}$=	379	ksi	$=0.9*E*k_c/(b_f/(2*t_f))^2$	(F5-9)
F_{cr_F5}=	50	ksi	=if(Flange="Compact",50, if(Flange="Non-Compact",$F_{cr\_F5\_8}$,$F_{cr\_F5\_9}$)	
$M_{n\_F5.3}$=	16450	kip-in	$=R_{pg}*F_{cr\_F5}*S_{xc}$	(F5-7)

8.5.12. Tension flange yielding (F5.4)

$M_{n\_F5.4}$=	18500	kip-in	$=M_p$ Both flanges are symmetric	(F5-10)

8.5.13. Flange hole reduction (F13.1)

D_{HoleT}=	0.000	in		
A_{fg}=	12.38	in^2	$=b_f*t_f$	
A_{fn}=	12.38	in^2	$=t_f*(b_f-2*D_{HoleT})$	
Y_t=	1.00		$=IF(F_y/F_u<=0.8,1,1.1)$	
$M_{n\_F13.1}$=	21385	kip-in	$=F_u*A_{fn}/A_{fg}*S_x$	(F13-1)

Flexural Capacity Summary

M_{n_F2}=	18500	kip-in	$=min(M_{n\_F2.1},M_{n\_F2.2})$
M_{n_F3}=	18500	kip-in	=min(Mn_F3.1,Mn_F3.2)
M_{n_F4}=	18500	kip-in	$=min(M_{n\_F4.1},M_{n\_F4.2},M_{n\_F4.3},M_{n\_F4.4})$
M_{n_F5}=	16450	kip-in	$=min(M_{n\_F5.1},M_{n\_F5.2},M_{n\_F5.3},M_{n\_F5.4})$
M_n=	18500	kip-in	IF(B3="Beam",IF(DesignSection="F2",MIN(M_{n_F2},$M_{n\_F13.1}$), IF(DesignSection="F3",MIN(M_{n_F3},$M_{n\_F13.1}$),IF(DesignSection="F4", MIN(M_{n_F4},$M_{n\_F13.1}$),MIN(M_{n_F5},$M_{n\_F13.1}$)))),M_{n_F4})
ΦM_n=	16650	kip-in	$=\Phi_b*M_n$

8.6. Axial Capacity

Section= E7 =IF(OR(Flange="Slender",Web="Slender"),"E7","E3")

8.6.1. E.3 Flexural buckling (member without slender elements)

K_x=	1		K_y=	1		K_z=	1	
L_b=	144	in	L_y=	144	in	L_z=	144	in

K_{LR}=	48.48		=max(K_x*Lb/r_x,K_y*Lb/r_y)	
F_e=	121.75	ksi	=π^2*E/(KLR)^2	(E3-4)
$F_{cr\_E.3}$=	42.10	ksi	=IF(KLR<=4.71*SQRT(E/F_y),F_y*0.658$^{(Fy/Fe)}$,0.877*F_e)	(E3-2, 3-3)
P_{n_E3}=	1625.21	kip	=$F_{cr\_E.3}$*A_g	(E3-1)

8.6.2. E.4 Torsional and flexural-torsional buckling: (Applies to singly symmetric shapes)

F_{ey}=	121.75	ksi	=π^2*E/(K_y*L_y/r_y)^2	(E3-4)
F_{ez}=	173.51	ksi	=π^2*E*C_w/(K_z*L_z)^2+G*J)/(1/(Ix+Iy))	(E4-2)
$F_{e\_E.4}$=	121.75	ksi	=(F_{ey}+F_{ez})/(2*H_z)*(1-sqrt(1-4*F_{ey}*F_{ez}*H_z/(F_{ey}+F_{ez})^2))	(E4-3)
$F_{cr\_E.4}$=	42.10	ksi	=IF(K_{Lr}<=4.71*SQRT(E/F_y),F_y*0.658$^{(Fy/Fe_E.4)}$,0.877*$F_{e\_E.4}$)	(E3-2, 3-3)
$P_{n\_E.4}$=	1625.2	kip	=A_g*$F_{cr\_E.4}$	(E4-1)

8.6.2. E.7 Flexural, torsional and flexural-torsional buckling (members with slender elements)

8.6.2a Calculation for effective flange width

Flange=	Nonslender			
λ=	6.72		From Section 3 above	
λ_r=	13.49		From Section 3 above	
c1=	0.22		AISC 360-16 Table E7.1 all elements	
c2=	1.49		AISC 360-16 Table E7.1 all elements	
F_{el}=	447.3	ksi	=(c2 * λrw/λw)^2 * Fy	
Fcr=	42.1	ksi	=Min (Fcr_E3, Fcr_E4)	
bf=	12.90	in	=Full Flange Width from AISC Database	
b_{e_E7-3}=	11.9	in	=bf*(1-c1*sqrt(F_{el}/F_{cr}))*sqrt(F_{el}/F_{cr}), but can't be more than bf	(E7-3)
be=	12.90	in	=If (Flange="Nonslender", b_f, b_{e_E7-3})	
Aweb=	13.83	in^2	=Ag-2*tf*bf	
Af=	24.77	in^2	=2*tf*be	
Ae1=	38.6	in^2	=Aweb+Af	

8.6.2b Calculation for effective web depth

Web=	Slender			
λ_w=	37.32		From Section 4.1 above	
λ_{rw}=	35.88		From Section 4.1 above	
c1=	0.18		AISC 360-16 Table E7.1 stiffened elements	
c2=	1.31		AISC 360-16 Table E7.1 stiffened elements	
F_{el}=	79.3	ksi	=(c2 * λrw/λw)^2 * Fy	
h=	22.6	in		
h_{e_E7-3}=	22.6	in	=h*(1-c1*sqrt(F_{el}/F_{cr}))*sqrt(F_{el}/F_{cr}), but can't be more than h	(E7-3)
h_e=	22.6	in	=If (Web="Nonslender", h, h_{e_E7-3})	
A_{flange}=	24.8	in^2	=2*bf*tf	
A_{web_radius}=	13.8	in^2	=A_g-A_{flange}	
A_{web}=	13.7	in^2	=h*t_w	
A_{radius}=	0.2	in^2	=A_{web_radius} - A_{Web}	
A_{e_web}=	13.66	in^2	=h_e * t_w	
A_{e2}=	38.6	in^2	=Aflange+Aradius+Ae_web	
A_e=	38.6	in^2	=Min (Ae1, Ae2)	
F_{cr}=	42.1	ksi		
P_{n_E7}=	1625.2	kips		
P_n=	1625.2	kip	=IF(AxialDesignSection="E3",P_{n_E3},P_{n_E7})	
Φ_c*P_n=	1462.7	kip		

8.7. Demand per AISC 360-16 Appendix 8 (to include p-small delta)

P_y=	1930	kip	$=A_g*F_y$	
P_r=	43	kip	$=P_{u\_omega}$	
α=	1		1 for LRFD, 1.6 for ASD	
C_m=	1		conservative assumption	(A-8-5)
P_{e1y}=	55488	kip	$=\pi^2*E*I_x/(Lb)^2$	
B_1=	1.001		$=C_m/(1-\alpha*P_r/P_{e1y})$	(A-8-3)

8.8 Axial + flexural interaction (Chapter H)

M_r=	5331.0	kip-in	$=M_u*B_1$	
M_c=	16650	kip-in	$=\Phi*M_n$	
P_r=	43.0	kip	$=P_u$	
P_c=	1462.7	kip	$=\Phi_c*P_n$	
DCR_Axial=	**0.029**		=Pr/Pc	
DCR_Flexural=	**0.320**		=Mr/Mc	
DCR_P+M=	**0.335**		$=IF(P_r/P_c>=0.2,P_r/P_c+8/9*M_r/M_c,P_r/(2*P_c)+M_r/M_c)$	(H1-1)

8.9. Shear capacity (Chapter G)

$Hole_{Web}$=	0	in		
Vu=	42.13	kips		
A_w=	14.82	in^2	$=t_w*(d-Hole_{web})$	
h/tw=	37			
k_v=	5.34		Webs without transverse stiffeners	
$2.24sqrt(E/F_y)$=	53.9			
C_{v1_G2-2}=	1			(G2-2)
$1.1sqrt(k_vE/F_y)$=	61.2			
C_{v1_G2-3}=	1.00			(G2-3)
C_{v1_G2-4}=	1.6		$=1.1*sqrt(k_v*E/F_y)/(h/t_w)$	(G2-4)
$C_{v1\_G2-3\_4?}$=	1.0		$=if(h/tw <= 1.1*sqrt(k_v*E/F_y), C_{v1\_G2-3}, C_{V1\_G2-4})$	
C_{v1}=	1.0		$=if(C_{v1\_G2-2}$="N/A",$C_{v1\_G2-3\_4?})$	
V_n=	444.7	kip	$=0.6*F_y*A_w*C_{v1}$	(G2-1)
DCR_Shear=	**0.105**		$=V_u/(\Phi_v*V_n)$	

8.10. Design Summary:

		Demand-to-Capacity Ratios			
LC ID	Load Combination	P	M	P+M	V
LC 7	(1.2 + 0.2 SDS)DL + OmegaEL + f1*LL + f2*SL	0.029	0.320	0.335	0.105

Appendix 7B

YLMC ETABS Demo

Date: 5-23-2022

SIMPSON STRONG-TIE COMPANY INC.
5956 W. Las Positas Blvd., Pleasanton, CA 94588.
(800) 999-5099
www.strongtie.com

Yield-Link® Connection Design Summary

Component capacities for

Demo Structure - Sample Calculations
5956 W Las Positas Blvd
Pleasanton, CA - 94568

Design Firm Name:

Simpson Strong-Tie

Date Printed:

May 23, 2022

Design By:

Simpson Strong-Tie
yieldlink@strongtie.com

MATERIAL PROPERTIES

No.	Item	Value
Shear Plate		
1	Fy (ksi)=	50.00
2	Fu (ksi)=	65.00
Stiffener Plate		
1	Fy (ksi)=	50.00
2	Fu (ksi)=	65.00
3	Min. Thickness, t_STP (in)=	0.25
4	Stiff. Depth (1-sided Connection)=	Full_Depth
Doubler Plate		
1	Fy (ksi)=	50.00
2	Fu (ksi)=	65.00
3	Min. Thickness, t_DP (in)=	0.25

DESIGN PARAMETERS

No.	Item	Data	
Seismics Coefficients			
1	Standard=	ASCE 7-16	
2	0.2 Sec Spectral Accel, Ss=	2.00	
3	1.0 Sec Spectral Accel, S1=	1.00	
4	Long-Period Transition Period, TL=	8.00	
5	Site Class=	D	
6	Sds - User defined=	YES	
7	Design Spectral Resp. Accel at Short Period, Sds=	1.00	
Factors			
	HCAI/DSA Project?	NO	
		X-Dir	Y-Dir
1	Lateral Force Resisting System=	SMF	SMF
2	Response Modification Coefficient, R=	8.00	8.00
3	Overstrength Factor, Omega=	3.00	3.00
4	Deflection Amplification factor, Cd=	5.50	5.50
5	Occupancy Importance, I=	1.00	1.00
6	Redundancy Factor, Rho=	1.00	1.00
7	Live Load Factor, f1=	0.50	
8	Snow Load Factor, f2=	0.20	
Member Design			
1	Design Code=	AISC 360-16	
2	Design Frame Type=	OMF	

No.	Item	Data	
Wind Coefficients			
		X-Dir	Y-Dir
1	Standard=	ASCE 7-16	
2	Wind Speed (mph)=	80.00	
3	Wind Speed Drift (mph)=	80.00	
4	Exposure Type=	B	
5	Ground Elevation Factor=	1.00	
6	Topographical Factor, Kzt=	1.00	
7	Gust Factor=	0.85	
8	Directionality Factor, Kd=	0.85	

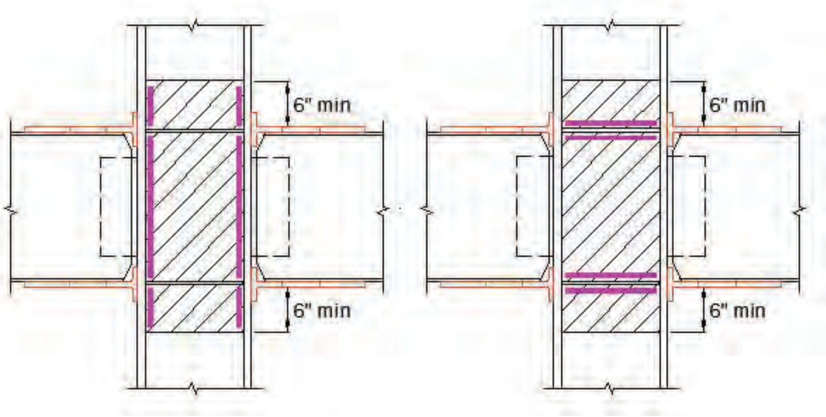
DESIGN DCR's

Allowable Seismic Drift Limit:	0.025Hx	Allowable Wind Drift Limit:	Hx/500

No.	Item	Allowable DCR	Max.	Min.
ILS (INITIAL LINK SELECTION)				
1	Initial tbf check=	1.00	1.05	0.90
2	Initial bf check=	1.00	1.05	0.90
3	Initial Lyield check=	1.00	1.05	0.90
4	Panel Zone DCR=	1.00	1.05	0.90
5	Drift DCR=	1.00	1.05	0.90
BLC (BEAM & LINK CHECK)				
1	Beam tf DCR=	1.00	1.05	0.90
2	Link strength DCR=	1.00	1.05	0.90
3	Link Slip DCR=	1.00	1.05	0.90
4	Lyield check=	1.00	1.05	0.90
5	t_BRP DCR=	1.00	1.05	0.90
6	BRP Bolt DCR=	1.00	1.05	0.90
CC (COLUMN CHECK)				
1	SCWB DCR=	1.00	1.05	0.90
2	Panel Zone DCR=	1.00	1.05	0.90
3	Column Flange DCR=	1.00	1.05	0.90
4	Stiffener DCR=	1.00	1.05	0.90
ST (SHEAR TAB CHECK)				
1	Beam Web DCR=	1.00	1.05	0.90
2	Shear Plate DCR=	1.00	1.05	0.90
3	Bolt DCR=	1.00	1.05	0.90
4	Fillet Weld DCR=	1.00	1.05	0.90

WELDING PREFERENCES

1	Doubler PL to col web/Cont plate Weld:	Option_2A	Doubler extended beyond continuity plates
2	Doubler PL to col flange weld:	Option_2	Fillet Weld
3	Use of Plug-weld for Doubler Plates?	NO	



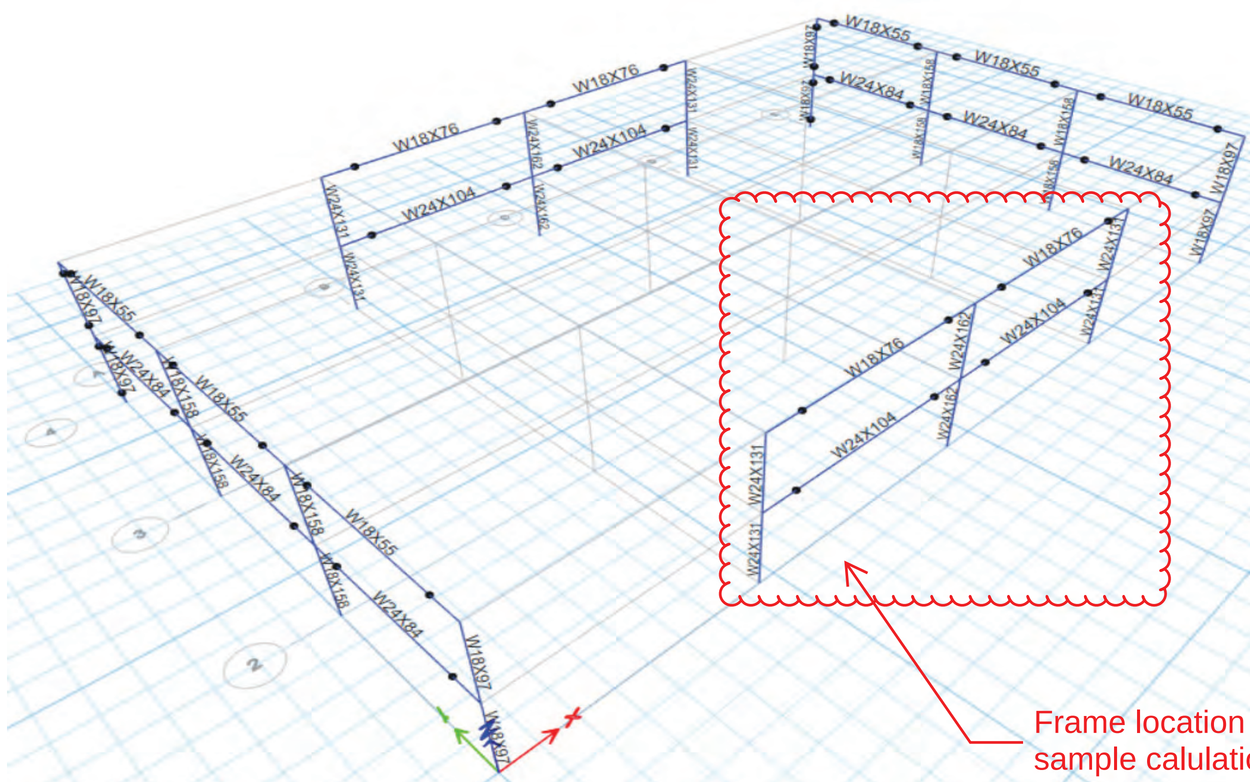
Job Name: Demo Structure - Sample Calculations
Job ID: ES-# 221804

Date Printed: May 23, 2022

Initial Yield-Link® Selection Summary

Elev. ID	Grid ID	Story	Beam Unique Name	Beam Size	Yield-Link® Size	K_rot (kip-in/rad)	I_End Col Size	J_End Col Size	Assign K at I_End	Assign K at J_End	Initial tbf Check	Initial bf Check	Initial Lyield Check	PZ I_End Col	PZ J_End Col
1	B	Story2	31	W18X76	YL4-3.5	779775	W24X131	W24X162	True	True	0.588	OK	0.929	0.341	0.621
1	C	Story2	33	W18X76	YL4-3.5	779775	W24X162	W24X131	True	True	0.588	OK	0.929	0.621	0.724
1	B	Story1	32	W24X104	YL6-4.5	1791139	W24X131	W24X162	True	True	0.533	OK	0.881	0.658	1.106
1	C	Story1	34	W24X104	YL6-4.5	1791139	W24X162	W24X131	True	True	0.533	OK	0.881	1.106	0.830

Beam ID for detailed output



Frame location for sample calulations

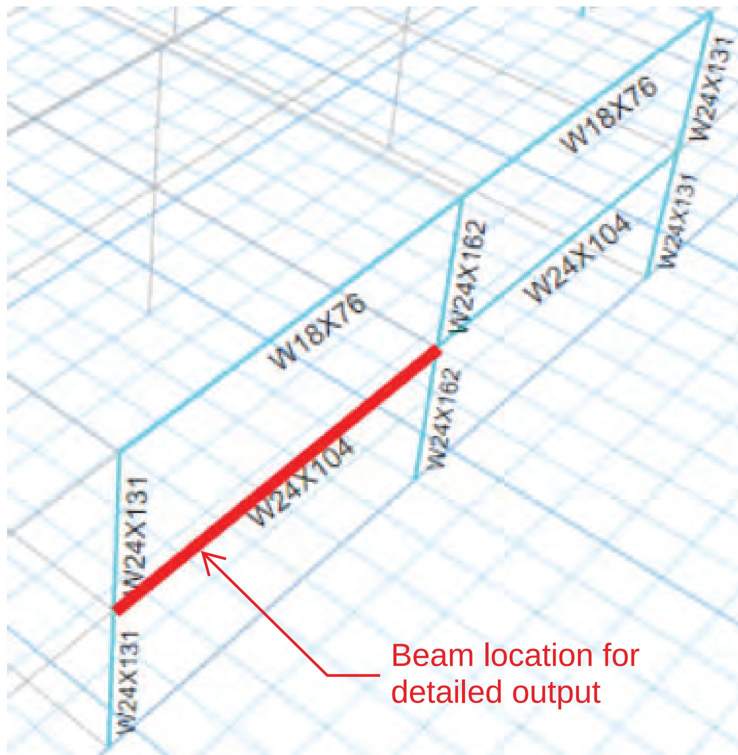
Job Name: Demo Structure - Sample Calculations
Job ID: ES-# 221804

Date Printed: May 23, 2022

Beam & Yield-Link® Check Summary

Elev. ID	Grid ID	Story	Beam Unique Name	Beam Size	Yield-Link® Size	BRP Size	MU_max (kips.in)	Beam tf_DCR	Link Strength DCR	Lyield DCR	BRP DCR	BRP Bolt DCR	Link Slip DCR
1	B	Story2	31	W18X76	YL4-3.5	BRP4B	1035.40	0.588	0.703	0.929	0.686	0.954	0.556
1	C	Story2	33	W18X76	YL4-3.5	BRP4B	1035.52	0.588	0.703	0.929	0.686	0.954	0.556
1	B	Story1	32	W24X104	YL6-4.5	BRP6B	2810.37	0.636	0.745	0.881	0.774	0.513	0.545
1	C	Story1	34	W24X104	YL6-4.5	BRP6B	2810.90	0.636	0.745	0.881	0.774	0.513	0.545

Beam ID for detailed output



Beam location for detailed output

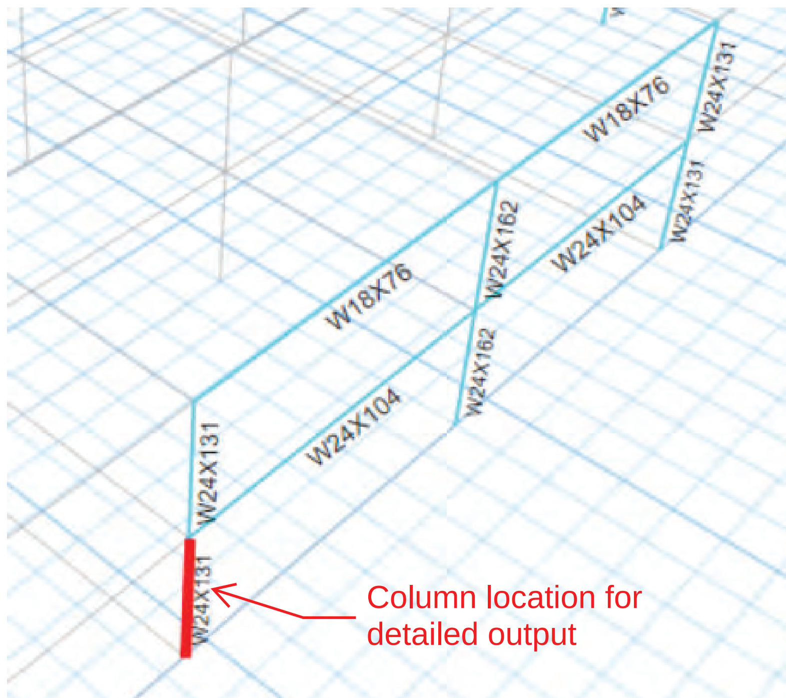
Job Name: Demo Structure - Sample Calculations
Job ID: ES-# 221804

Date Printed: May 23, 2022

Column Check Summary

Elev. ID	Grid ID	Story	Column Unique Name	Column Size	Yield- Link® Left Side	Yield- Link® Right Side	Col Pu (kips)	V_bm Gravity (kips)	Stiffener Provider	Doubler Plate Provider	Stiffener Req.	Min. Stiff. Thk (in)	Min. Dblr Plate Thk (in)	SCWB DCR	Col PZ DCR	Col Flange Check
1	B	Story2	35	W24X131	NA	YL4-3.5	36.55	5.91	YES	NO	NO	0.500	0.000	NA	0.724	0.564
1	C	Story2	37	W24X162	YL4-3.5	YL4-3.5	31.63	5.91	YES	NO	NO	0.500	0.000	NA	0.621	0.443
1	D	Story2	39	W24X131	YL4-3.5	NA	36.60	5.84	YES	NO	NO	0.500	0.000	NA	0.724	0.564
1	B	Story1	36	W24X131	NA	YL6-4.5	139.59	17.00	YES	NO	NO	0.500	0.000	0.219	0.830	0.751
1	C	Story1	38	W24X162	YL6-4.5	YL6-4.5	121.98	17.00	YES	NO	NO	0.500	0.000	0.332	0.915	0.589
1	D	Story1	40	W24X131	YL6-4.5	NA	139.42	16.91	YES	NO	NO	0.500	0.000	0.217	0.830	0.751

Column ID for detailed output



Job Name: Demo Structure - Sample Calculations

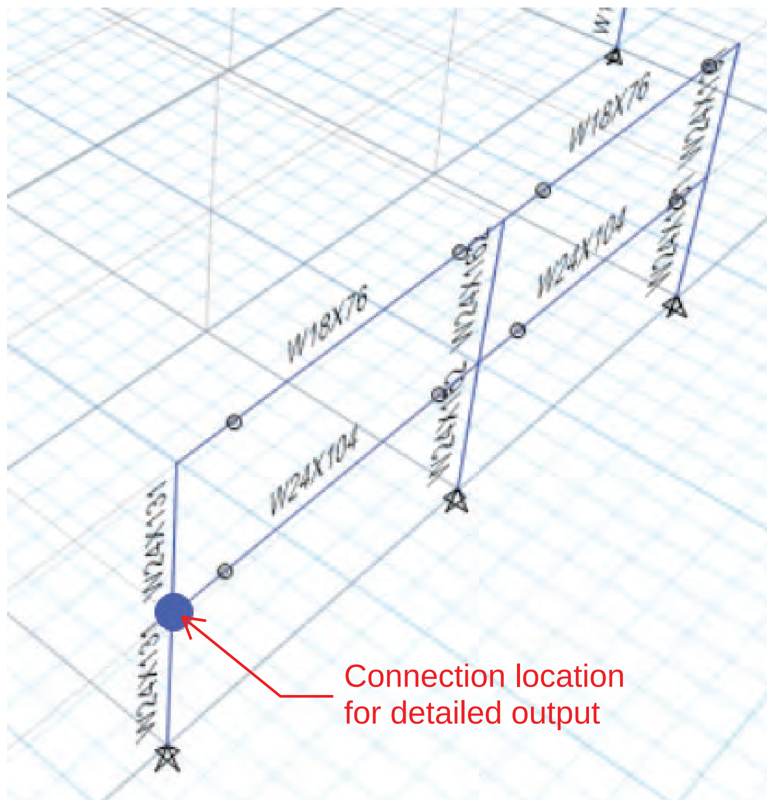
Date Printed: May 23, 2022

Job ID: ES-# 221804

Shear Tab Check Summary

Elev. ID	Grid ID	Story	Beam Unique Name	Beam Size	Yield-Link® Size	Axial Pu (kips)	Shear Vg_LC08 (kips)	Shear Vu (kips)	Shear Vg_LC01-07 (kips)	No. Vert. Bolts	No. Horz. Bolts	Bolt Size	Bolt Type	Shear Plate Thk (in)	No. of SP	Fillet Weld Size (in)	Beam Web DCR	Shear PL Geo. Check	Shear Plate DCR	Bolt DCR	Fillet Weld DCR (in)
1	B	Story2	31	W18X76	YL4-3.5	28.93	4.81	20.29	5.91	3	2	7/8	A325X	1/2	1	5/16	0.449	OK	0.420	0.521	0.177
1	C	Story2	33	W18X76	YL4-3.5	28.93	4.81	20.29	5.84	3	2	7/8	A325X	1/2	1	5/16	0.449	OK	0.420	0.521	0.177
1	B	Story1	32	W24X104	YL6-4.5	25.82	12.23	52.09	17.00	5	2	7/8	A325X	1/2	1	5/16	0.324	OK	0.482	0.541	0.294
1	C	Story1	34	W24X104	YL6-4.5	25.82	12.30	52.16	16.91	5	2	7/8	A325X	1/2	1	5/16	0.324	OK	0.482	0.541	0.294

Beam ID for detailed output



Connection location for detailed output

Job Name: Demo Structure - Sample Calculations
Job ID: ES-# 221804
Date Printed: May 23, 2022

Drift Summary

Elev. ID	Grid ID	Story	Beam Unique Name	Beam Size	Yield-Link® Size	Assign K at I_End	Assign K at J_End	Node ID @LeftEnd	Node ID Below	Seismic Delta_x (in)	Wind Delta_x (in)	Story Height (in)	Allowable Seismic Drift (in)	Allowable Wind Drift (in)	Seismic Drift DCR	Wind Drift DCR
1	B	Story2	31	W18X76	YL4-3.5	YES	YES	28	29	2.233	0.050	144.00	3.600	0.288	0.620	0.174
1	C	Story2	33	W18X76	YL4-3.5	YES	YES	30	31	2.233	0.050	144.00	3.600	0.288	0.620	0.174
1	B	Story1	32	W24X104	YL6-4.5	YES	YES	29	34	3.295	0.081	144.00	3.600	0.288	0.915	0.281
1	C	Story1	34	W24X104	YL6-4.5	YES	YES	31	35	3.295	0.081	144.00	3.600	0.288	0.915	0.281

Beam Design

Elev. ID	Grid ID	Story	Beam Unique Name	Beam Size	Yield-Link® ID	Pcap_Link (kips)	Mcap_Link (kips.in)	Mu_max (kips.in)	Mu_max/ Pcap_Link	Mu_Adj. Fact	Axial DCR	B-maj DCR	B_min DCR	PMM DCR	Adj. B_maj DCR	Total DCR
1	B	Story2	31	W18X76	YL4-3.5	136.50	2552.55	2678.30	1.05	0.95	0.000	0.365	0.000	0.365	0.348	0.348
1	C	Story2	33	W18X76	YL4-3.5	136.50	2552.55	2678.12	1.05	0.95	0.000	0.365	0.000	0.365	0.348	0.348
1	B	Story1	32	W24X104	YL6-4.5	263.25	6541.76	7442.46	1.14	0.88	0.000	0.572	0.000	0.572	0.503	0.503
1	C	Story1	34	W24X104	YL6-4.5	263.25	6541.76	7442.99	1.14	0.88	0.000	0.572	0.000	0.572	0.503	0.503

Job Name: Demo Structure - Sample Calculations
Job ID: ES-# 221804

Date Printed: May 23, 2022

Column Design

Elev. ID	Grid ID	Story	Column Unique Name	Column Size	Mu_Top (kips.in)	Mu_Bot (kips.in)	Mu_ Omega (kips.in)	Pu_ Omega (kips)	Bracing at Beam Bot. FLG	bf/tf DCR	h/tw DCR	Axial DCR	B_maj DCR	B_min DCR	PMM DCR	Adj. B_maj DCR	Total DCR
1	B	Story2	35	W24X131	2359.64	674.18	2359.64	36.55	YES	N/A	N/A	0.007	0.021	0.183	0.211	0.021	0.211
1	C	Story2	37	W24X162	4002.25	3148.03	4002.25	31.63	YES	N/A	N/A	0.006	0.149	0.172	0.328	0.149	0.328
1	D	Story2	39	W24X131	2359.37	674.95	2359.37	36.60	YES	N/A	N/A	0.012	0.041	0.184	0.236	0.041	0.236
1	B	Story1	36	W24X131	6098.00	0.00	6098.00	139.59	YES	N/A	N/A	0.045	0.366	0.145	0.556	0.366	0.556
1	C	Story1	38	W24X162	9283.95	0.00	9283.95	121.98	YES	N/A	N/A	0.022	0.441	0.138	0.601	0.441	0.601
1	D	Story1	40	W24X131	6097.94	0.00	6097.94	139.42	YES	N/A	N/A	0.027	0.345	0.146	0.518	0.345	0.518

Job Name: Demo Structure - Sample Calculations
Job ID: ES-# 221804

Date Printed: May 23, 2022

Weld Summary

Elev. ID	Grid ID	Story	Col. Unique Name	Column Size	No. Left ST	Left ST Thk (in)	No. Right ST	Right ST Thk (in)	Left ST Fillet Weld W1A (in)	N_sides of Fillet W1A	Left ST PJP Weld W1A (in)	N_sides of PJP W1A	Right ST Fillet Weld W1B (in)	N_sides of Fillet W1B	Right ST PJP Weld W1B (in)	N_sides of PJP W1B
1	B	Story2	35	W24X131	N/A	N/A	1	1/2	N/A	N/A	N/A	N/A	5/16	2	N/A	N/A
1	C	Story2	37	W24X162	1	1/2	1	1/2	5/16	2	N/A	N/A	5/16	2	N/A	N/A
1	D	Story2	39	W24X131	1	1/2	N/A	N/A	5/16	2	N/A	N/A	N/A	N/A	N/A	N/A
1	B	Story1	36	W24X131	N/A	N/A	1	1/2	N/A	N/A	N/A	N/A	5/16	2	N/A	N/A
1	C	Story1	38	W24X162	1	1/2	1	1/2	5/16	2	N/A	N/A	5/16	2	N/A	N/A
1	D	Story1	40	W24X131	1	1/2	N/A	N/A	5/16	2	N/A	N/A	N/A	N/A	N/A	N/A

Weld Summary 2

Elev. ID	Grid ID	Story	Col. Unique Name	Column Size	STP Thk (in)	DP Thk (in)	STP to Col. Web W2 (in)	N_sides of W2	STP to Col. Flg. W3 (in)	N_sides of W3
1	B	Story2	35	W24X131	1/2	N/A	3/16	2	3/16	2
1	C	Story2	37	W24X162	1/2	N/A	3/16	2	3/16	2
1	D	Story2	39	W24X131	1/2	N/A	3/16	2	3/16	2
1	B	Story1	36	W24X131	1/2	N/A	3/16	2	3/16	2
1	C	Story1	38	W24X162	1/2	N/A	3/16	2	3/16	2
1	D	Story1	40	W24X131	1/2	N/A	3/16	2	3/16	2

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Job Name: Demo Structure - Sample Calculations

Job ID: ES-# 221804

Date Printed: May 23, 2022

INITIAL LINK SELECTION DESIGN DETAILS

1.1 CURRENT MEMBER:

Beam Unique Name:	32	I_End Column Size:	W24X131	J_End Column Size:	W24X162
Beam Size:	W24X104	I_End Col. Unique Name:	36	J_End Col. Unique Name:	38
LINK ID:	YL6-4.5	Assign K at I_End:	True	Assign K at J_End:	True

1.2 LINK STEM GEOMETRY:

NY Length ColSide (Lcol_side)=	5	in	Thickness (t_stem) =	0.75	in
Yield Length, incl. fillets (L_stemYield) =	10	in	NY Width ColSide (Wcol_side)=	10	in
NY Length BeamSide (Lbm_side)=	11.25	in	Central Neck Yield Width (w_stemYield)=	4.5	in
L_stem=	26.25	in	NY Width BeamSide (Wbm_side)=	10	in
Fillet Radius (r_fillet)=	0.75	in	Yielding Area (A_stemYield) =	3.38	in^2

1.3 LINK STEM BOLTS:

Num. Bolts (n_bolt_linkBm)=	6		Gauge Along Width (bolt_g_stem)=	6	in
Bolt Type (Bolt_Gr_linkBm)=	A490		Spacing Along Length (bolt_s_stem) =	3.375	in
Bolt Dia (boltD_linkBm)=	1.25	in	First Bolt distance to Neck (Sc)=	2.5	in
Min. Bolt length =	3.125	in	Last Bolt distance to Edge (Sb) =	2	in

1.4 LINK FLANGE GEOMETRY:

Thickness (t_flange)=	1.25	in
Flange Width (W_flange)=	10	in
Flange height (H_flange) =	9.25	in

1.5 LINK FLANGE BOLTS:

Num. Bolts (n_bolt_linkCol) =	4		Gauge Along Width (vertical) (bolt_g_flange)=	5.75	in
Bolt Type (Bolt_Gr_linkCol)=	A325		Spacing Along Length (horiz) (bolt_s_flange)=	6	in
Bolt Dia (boltD_linkCol)_=	1.25	in			

1.6 LINK MATERIAL:

Fy_link =	50	ksi	Material Overstrength Factor (Rt_link) =	1.2
Fu_link =	65	ksi	Ry_link=	1.1

1.7 LINK K_ROT:

Pye_link=	185.63	kips	*=Expected yield strength of link*
Link Flange (K1) =	35752	kips/in	*=0.75 * 192 *E *(Wcol_side*t_flange^3/ 12) / bolt_g_flange^3*
Link Stem NY Portion (K2) =	23673	kips/in	*=t_stem*Wcol_side * E / [(Lcol_side + Sc + if (n_bolt_linkBm > 4, bolt_s_stem/2,0)]*
Link Stem Neck (K3) =	9788	in	*=t_stem * w_stemYield * E / L_stemYield*
Effective Stiffness (K_eff)=	5801	kips/in	*=K1*K2*K3 / (K1*K2 + K2*K3 + K1*K3)*
Mye_link=	4613	k-in	
Dy=	0.0320	in	
qy=	0.002575	rad	
Krot=	1791139	k-in/rad	

1.8 BEAM FLANGE THICKNESS CHECK:

Beam flange thickness (tbf)=	0.75	in		
SUM_bf=	0.533	in	*= 0.4 / (tbf)*	*(AISC 358-16, Chapter 12.3.1.3)*
Adequate =	OK	*= OK, if tbf>=0.4"*		

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1.9 BEAM & COLUMN FLANGE WIDTH CHECK:

bbf_min=	9.25	in	*Looked up value*
bcf_min=	9.25	in	*Looked up value*
Beam flange width (bbf)=	12.8	in	*Looked up value*
I_End Column flange width (bc1f)=	12.9	in	*Looked up value*
J_End Column flange width (bc2f)=	13	in	*Looked up value*
k1=	1.2	in	*Looked up value*
B=	2.5	in	*= bolt washer diameter*
E=	1.09375	in	*= min. edge distance*
ColGageReq =	4.9	in	*= 2*K1 + bolt washer diameter (B)*
sflange=	6	in	*Previous define*
DCR_bf=	0.723		*=max(bbf_min/bbf, bcf_min/bc1f, bcf_min/bc2f)*
ColGageReq_washer =	4.6	in	*= 2*K1 + 2*E*
Clip washers =	Not Required		*Required if (S_flange + 0.5)<ColGageReq, NG if (S_flange + 0.5)<ColGageReq_washer*
Adequate =	OK		*OK, if DCR_bbf <= DCR_allowed and Clip washers<> N/G*

1.10 LINK STEM YIELDING LENGTH CHECK:

Limit Rotation (r_linkLimit) =	0.05	radians	*Connection rotation per ESR-2802*
Limit Strain ε (e_linkLimit) =	8.50%		*Link strain limit per ESR-2802*
Rotation Arm (d_arm)=	12.425	in	*= (db + t_stem)/2*
Link Extension (linkDelta)=	0.621	in	*=d_arm* r_linkLimit*
Lstem_yield_req=	8.809	in	*= r_linkLimit* d_arm/ 0.085 + 2* r_fillet*
L_stem_DCR=	0.881		*= Lstem_yield_req/ L_stemYield*
Check=	OK		*OK, if L_stem_DCR < SUM_allowed*

1.11 PRELIMINARY COLUMN PANEL ZONE CHECK

Column Unique Name=	36		38			
	Left Side	Right Side	Left Side	Right Side		
Φv_Pz=	0.90	0.90	0.90	0.90		
Fyc=	50.00	50.00	50.00	50.00	ksi	
dc=	24.50	24.50	25.00	25.00	in	*Looked up value*
tcw=	0.61	0.61	0.71	0.71	in	*Looked up value*
Agc=	38.60	38.60	47.80	47.80	in^2	*Looked up value*
Pc=	1930.00	1930.00	2390.00	2390.00	kips	*= Fyc* Agc*
Pu=	139.59	139.59	88.19	88.19	kips	*Max Axial Force of column per Omega Combo (SST LC31-34) From ETABS*
ΦRn_PZ=	400.21	400.21	475.88	475.88	kips	*= Φv_pz* if (Pu < 0.4* Pc, 0.6* Fy_col* dc* tcw, 0.6* Fy col* dc* tcw* (1.4 - Pu/Pc))*
Vu_c=	0	0	0	0	kips	*= Column Shear (= 0 , Before Analysis)*
Pcap_link=	0.00	263.25	263.25	263.25	kips	*= if(R=3,Py_link, Pr_link)*
Ru=	0.00	263.25	263.25	263.25	kips	*= Pcap_link + Vu_c*
SUM_PZ=	0.000	0.658	0.553	0.553		*= Pcap_link/ ΦRn_PZ*
Total_SUM_PZ=	0.658		1.106			
SUM_PZ_geometry=	N/A		0.712			*=[(dz+wz)/90] / tcw, N/A for R=3*
Final_SUM_PZ=	0.658		1.106			
Adequate =	OK		NG			*Ok, if SUM_PZ <= DCR_PZ_Allowable*

DCR Summary:

Component	Check	DCR	Limit	Result
Beam	Initial tbf Check=	0.533	1.00	OK
Beam/Column	Initial bf Check=	0.723	1.00	OK
Link	Inittial Lyield Check=	0.881	1.00	OK
Column	Pz I_End column=	0.658	1.00	OK
	Pz J_End column=	1.106	1.00	NG

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Job Name: Demo Structure - Sample Calculations

Date Printed: May 23, 2022

Job ID: ES-# 221804

BEAM & YIELD-LINK® DESIGN DETAILS

2.1 CURRENT MEMBER:

		R=	8		AISC 360 Table B4.1b			
Beam Unique Name:	32	Link ID:	YL6-4.5		Beam b/2/t =	8.5	Compact	
Beam Size:	W24X104	Mu_max:	2810.37	kips.in	Beam h/tw =	43.1	Compact	

2.2 LINK STEM GEOMETRY:

NY Length ColSide (Lcol_side)=	5	in	Thickness (t_stem) =	0.75	in
Yield Length, incl. fillets (L_stemYield) =	10	in	NY Width ColSide (Wcol_side)=	10	in
NY Length BeamSide (Lbm_side)=	11.25	in	Central Neck Yield Width (w_stemYield)=	4.5	in
L_stem=	26.25	in	NY Width BeamSide (Wbm_side)=	10	in
Fillet Radius (r_fillet)=	0.75	in	Yielding Area (A_stemYield) =	3.38	in^2

2.3 LINK STEM BOLTS:

Num. Bolts (n_bolt_linkBm)=	6		Gauge Along Width (bolt_g_stem)=	6	in
Bolt Type (Bolt_Gr_linkBm)=	A490		Spacing Along Length (bolt_s_stem) =	3.375	in
Bolt Dia (boltD_linkBm)=	1.25	in	First Bolt distance to Neck (Sc)=	2.5	in
Min. Bolt length =	3.125	in	Last Bolt distance to Edge (Sb) =	2	in

2.4 LINK FLANGE GEOMETRY:

Thickness (t_flange)=	1.25	in
Flange Width (W_flange)=	10	in
Flange height (H_flange) =	9.25	in

2.5 LINK FLANGE BOLTS:

Num. Bolts (n_bolt_linkCol) =	4		Gauge Along Width (vertical) (bolt_g_flange)=	5.75	in
Bolt Type (Bolt_Gr_linkCol)=	A325		Spacing Along Length (horiz) (bolt_s_flange)=	6	in
Bolt Dia (boltD_linkCol)_=	1.25	in			

2.6 LINK MATERIAL:

Fy_link =	50	ksi	Material Overstrength Factor (Ry_link) =	1.1
Fu_link =	65	ksi	Rt_link=	1.2

2.7 LINK STEM STRAIN DEMAND CHECK:

Limit Rotation (r_linkLimit) =	0.05	radians	Connection rotation per ESR-2802
Limit Strain ε (e_linkLimit) =	0.08		Link strain limit per ESR-2802
Rotation Arm (d_arm)=	12.425	in	= (db + t_stem)/2
Link Extension (linkDelta)=	0.621	in	=d_arm* r_linkLimit
Lyield_req=	8.81	in	= r_linkLimit* d_arm/ 0.085 + 2* r_fillet
Lyield_DCR=	0.881		= Lstem_yield_req/ L_stemYield
Check=	OK		OK, if L_stem_DCR < SUM_allowed

Strength Check:

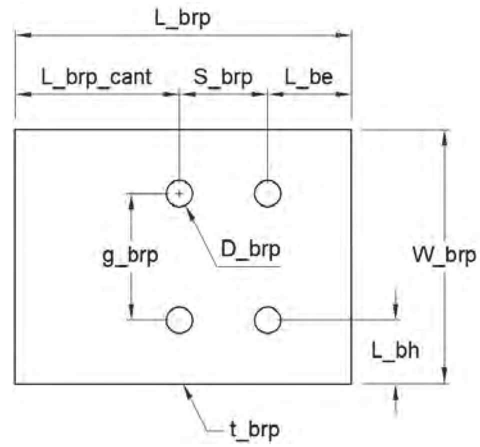
Mu=	2810.37	k-in	Moment Demand
db=	24.10	in	Beam Depth from database
db+tstem=	24.85	in	
Pu_link=	113.09	kips	Link Axial demand
Py_link=	168.75	kips	Link Capacity from database
Φ*Py_link=	151.88	kips	*Φ = 0.9*
Link stem DCR=	0.745		= Pu_link/(0.9*Py_link)
Check=	OK		OK, if Link stem DCR < SUM_allowed

Slip-Critical Check:

Pu_slip=	113.09	kips	=Pu_link
ϕ=	1		
μ=	0.30		
Du=	1.13		
hf=	1.00		
ns=	1.00		
Tb=	102.00	kips	Minimum Bolt Pretension per Table J3.1
n_bolt_linkBm=	6.00		
ΦRn_slip=	207.47	kips	=f *m *Du *hf *Tb *ns *n_bolt_linkBm
Link Slip DCR=	0.545	kips	=Pu_link/fRn_slip
Check=	OK		OK, if Link Slip DCR < SUM_allowed

2.8 BUCKLING RESTRAINT PLATE AND SPACER CHECK:

D_brp=	0.750	in	from Database
nbolt_brp=	2.00		per Spacer plate
S_brp=	2.5	in	from Database
t_brp=	1	in	from Database
L_be=	3.75	in	from Database
L_brp_cant=	6.500	in	=Lbrp - L_be - S_brp
L_brp=	12.75	in	from Database
Wbrp_min=	10	in	= Wbm_side
Fy_brp=	50.00	ksi	User Input
Ry_brp=	1.10		User Input
Fu_brp=	65.00	ksi	User Input
Rt_brp=	1.20		User Input



2.9 BRP THICKNESS CHECK:

Pcap_link=	263.25	kips	=if(R=3,Py_link, Pr_link)
Lelong=	0.621	in	Elongated distance at 0.05 rad
L1st_bolt=	3.75	in	Start of yielding region to 1st BRP bolt centerline
Lcant=	3.97	in	Lever arm from edge of yield link to edge of bolt hole + elongated dist at 0.05 rad
bn_brp=	8.25	in	Net width of BRP, without bolt holes
tbrp,min=	0.774	in	EQ 10.1-1
tbrp_use=	1.000	in	value previously defined
tbrp_DCR=	0.774		
Check=	OK		OK, if tbrp_DCR < SUM_allowed

2.10 BEAM FLANGE THICKNESS CHECK:

Joint Rotation=	0.04	rad	
e0.04=	0.05847	in/in	
g=	0.01096	in	gap between BRP and beam flange
Iy=	0.158	in^4	
lo=	1.071	in	effective buckling wave length
Qi=	10.78	kips	
N=	9.34		=LstemYield / lo
N=	9.00		Round down to next integer
Ndesign=	5.0		N used for design (Contact points with BRP)
Tux=	53.89	kips	=Ndesign * Qi
Tux_bolt=	13.47	kips	=Tux/Nbolts
Φ=	1.00		

Ledge=	6.500	in	1st bolt to edge of BRP					
S_brpBolt_min=	6.250	in	=byield + 2*(0.125 + d_brp_bolt)					
gbrp=	6.750	in						
b=	3.125	in	BRP bolt to face of web					
c=	6.150	in	edge of beam flange to face of web					
a=	3.025	in	center of BRP bolt to edge of beam flange					
x=sqrt(b*c)=	4.384	in	1 row of BRP bolts:			2 rows of BRP bolts:		
4*x=	17.536	in	pe_1row=	17.536	in	pe1=	10.018	in
2*x=	8.768	in				pe2=	10.018	in
S_brp_bolt=	3	in				pe_2row=	10.018	in
pe=	10.018	in				3 rows of BRP bolts:		
b'=	2.750	in				pe1=	10.018	in
tf_min=	0.477	in	=max(sqrt(4*b'*T/(Φ*pe*Fu_bm)),0.4)			pe2=	3.442	in
tbf=	0.750	in	beam flange thickness			pe3=	10.018	in
tbf_DCR=	0.636					pe_3row=	7.826	in
Check=	OK		OK, if tbrp_DCR < SUM_allowed					

2.11 BRP BOLT SIZE AND QUANTITY CHECK:

Ix=	5.695	in4	=byield^3 * tstem /12, strong axis moment of inertia of reduced link region
Vuy=	18.368	kips	
Vuy per bolt=	9.184	kips	

	Tx (kips)	Vx (kips)	Ty (kips)	Vy (kips)
Bolt Forces=	13.472	4.042	0.000	9.184

Bolt Type (Bolt_Gr_linkBm)=	A325		
Thread Condition=	N		
Φbolt=	0.75		
Fnv_325	54	ksi	AISC 360 Table J3.2
Ab_stem=	0.442	in^2	= Pi* (boltD_linkBm/2)^ 2
Strength per Bolt (ΦVbolt) =	17.892	kips	=Φbolt* Fnv_325* Ab_stem
Fnt_325	90	ksi	AISC 360 Table J3.2
ΦTbolt=	29.821	kips	=Φbolt* Fnt_325* Ab_stem

	X-Dir DCR	Y-Dir DCR
Tension=	0.452	0.000
Shear=	0.226	0.513
RnV_(V+T), kips=	15.177	17.892
T+V=	0.266	0.513

Max DCR=	0.513	
Check=	OK	OK, if tbrp_DCR < SUM_allowed

2.12 LINK STEM HOLE/BRP HOLE FIT ON BEAM FLANGE

bff=	12.8	in	
bff_BRP_bolt=	9.3	in	=minimum flnage width for BRP bolt spacing + min. bolt edge distance
bff_stem_bolt=	8.750	in	=minimum flange width for stem bolt spacing + min. bolt edge distance
bff_min=	9.3	in	=max (bbf_BRP_bolt, bbf_stem_bolt)
bf_DCR=	0.723		
Check=	OK		

2.13 BRP BOLT BEAM FLANGE BEARING AND TEAROUT IN WEAK-AXIS DIRECTION (AISC 360, J3.10)

Φbr=	0.8			
bbf=	12.8	in		
g_brp=	6.750	in		
d_brp=	0.750	in		
Ledge=	3.025	in		
Ledge_min=	1.000	in		
Ledge>1 bolt Dia?	OK			
Check=	OK			
Vy=	9.2	kips/bolt		
Rn_bearing=	87.75	kips		
ΦRn_bearing=	65.8	kips		
DCR_bearing=	0.140			
Check=	OK			
Lc=	3.03	in		
Rn_tearout=	177.0	kips	=1.2*Lc*tbf*Fu_bm	(AISC 360)
ΦRn_tearout=	132.7	kips	=Φbr*Rn_tearout	
DCR_tearout=	0.069			
Check=	OK			

2.14 BRP BOLT BEAM FLANGE BLOCK SHEAR IN WEAK-AXIS DIRECTION (AISC 360, J4)

Φu=	0.75	
Sbrp=	2.5	in
d_brp=	0.8125	in
Ledge=	3.025	in
tbf=	0.750	in
Vuy=	18.37	kips
Lgv=	6.05	in
Agv=	4.54	in^2
Rn_bearing=	136.13	kips
ΦRn_yield=	102.09	kips
DCR_yield=	0.180	
Check=	OK	
Lnv=	5.24	
Anv=	3.93	in^2
Rn_tearout=	153.20	kips
ΦRn_tearout=	114.90	kips
DCR_tearout=	0.160	
Check=	OK	

Lnt=	1.69		
Ant=	1.27	in^2	
Rn_BS1=	235.46	kips	
Rn_BS2=	218.39	kips	
Rn_BS=	218.39	kips	min(Rn_BS1, Rn_BS2)
ΦRn_BS=	163.79	kips	
DCR_BS=	0.112		
Check=	OK		

2.15 BEAM FLANGE CHECK FOR LINK STEM-TO-BEAM FLANGE BOLT BEARING AND TEAROUT (AISC 360, J3.10)

Pr_link=	263.3	kips	
tbf=	0.750	in	Beam Flange thickness
bolt_Dia=	1.250	in	Link stem-to-beam flange bolt diameter
n_bolts=	6	in	Number of link-stem bolts
S_stem_bolts=	3.38	in	stem bolt spacing
Lc=	2.0625	in	
Fy_bm=	50	ksi	from Design_Limits
Fu_bm=	65	ksi	from Design_Limits
Rn_bearing=	877.5	kips	=2.4*bolt_Dia*tbf*Fu_bm*n_bolts
ΦRn_bearing=	658.1	kips	=fu*Rn_bearing
DCR_bearing=	0.400		
Check=	OK		
Rn_tearout=	723.9	kips	=1.2*Lc*tbf*Fu_bm*n_bolts
ΦRn_tearout=	543.0	kips	=fu*Rn_tearout
DCR_tearout=	0.485		
Check=	OK		

2.16 LINK STEM-TO-BEAM FLANGE BOLT BLOCK SHEAR CHECK (AISC 360, J4)

L1=	6.500	in
Sbrp=	2.5	in
L2=	6.3	in
Stem=	3.38	in
nstem_bolts=	6.00	
Gstem=	6.000	in
gbrp=	6.750	in
d_brp=	0.813	in
d_stem=	1.313	in
bbf=	12.8	in
h=	3.400	in
h_min=	1.625	in
h>1 bolt Dia?	OK	
h>=h_min?	OK	

Lgv=	26.25	in
Agv=	19.69	in
Rn_yield=	1181.3	in^2
ΦRn_yield=	885.9	kips
DCR_yield=	0.297	kips
Check=	OK	
Lnv=	21.34	in
Anv=	16.01	in^2
Rn_rupture=	624.30	kips
ΦRn_rupture=	468.23	kips
DCR_rupture=	0.562	
Check=	OK	

Lnt=	5.49	in	
Ant=	4.12	in^2	
Rn_BS1=	891.8	kips	
Rn_BS2=	858.1	kips	
Rn_BS=	858.1	kips	min(Rn_BS1, Rn_BS2)
ΦRn_BS=	643.6	kips	
DCR_BS=	0.409		
Check=	OK		

2.17 BEAM FLANGE NET SECTION CHECK

Zbx=	289	in^3	
Zholes=	46.0	in^3	=2*Dholes*tbf*(d-tbf)
Znet=	243	in^3	=Zbx-Zholes
Mpb_net=	13367	k-in	=Znet*Fy*Ry
Pr_link=	263.3	kips	
d+tstem=	24.85	in	
Mpr_link=	6542	k-in	
DCR_NetSection=	0.489		=Mpr_link/Mpb_net

DCR Summary:

Component	Check	DCR	Limit	Result
Beam	Beam tf_DCR=	0.636	1.00	OK
Link	Link Strength DCR=	0.745	1.00	OK
	Link Slip DCR=	0.545	1.00	OK
	Ly_yield DCR=	0.881	1.00	OK
BRP	t_BRP DCR=	0.774	1.00	OK
	BRP Bolt DCR=	0.513	1.00	OK
	BRP Bolt Edge Dist>1d	OK		OK
	Stem Bolt Edge Dist>1d	OK		OK

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Job Name: Demo Structure - Sample Calculations

Date Printed: May 23, 2022

Job ID: ES-# 221804

SHEAR PLATE DESIGN DETAILS

4.1 CURRENT MEMBER:

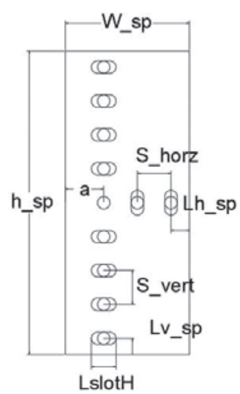
Beam Unique Name:	32	Thickness (tsp) (in):	0.50	in	Fy_bm=	50	ksi
Beam Size:	W24X104	Bolt Dia (db_sp):	0.875	in	Fu_bm=	65	ksi
Axial Pu (kips):	25.82	No. of SP(n_sp):	1		Left Conn=	YES	
Shear V_Grav.SST_LC08 (kips):	12.23	Bolt Type:	A325_X		Right_Conn=	YES	
Shear V_Grav.SST_LC01-07 (kips):	17.00	n_Vbolts_SST:	5.00		No. of Conn =	2	
Link ID:	YL6-4.5	n_Hbolts_SST:	2.00				
tstem (in):	0.75						

Shear Plate Info: R= 8

4.2 SHEAR PLATE BOLT SIZE:

Beam Section (bmSize) =	W24X104		Value previously defined
Beam Depth (db)=	24.10		Value previously defined
tbf=	0.750		Value previously defined
tbw=	0.500		Value previously defined
Fy_sp=	50	ksi	
Fu_sp =	65	ksi	
Axial Load (Pu_sp)=	25.82	kips	Max axial force per LC 31-34 from ETABS
Lcc=	360.00	in	Value previously defined
Lh=	328.25	in	=Lcc-dc_left/2 - dc_right/2-2*a
Pcap_link=	263.25	kips	=If (R=3, Py_link, Pr_link)
Mcap=	6541.76	kip-in	=Pcap_link*(db + tstem)
Vcap_link=	39.86	kips	=Mcap'*(No. of Connections) / Lh
Vertical Load (Vu_bm)=	52.09	kips	= Vcap_link + Vbm_gravity
No of vertical Bolts (n_Vbolts_SST)=	5		Looked up value
No of horizontal Bolts (n_Hbolts_SST)=	2		User Input
Vu_bolt=	16.59	kips	= Max(Sqrt ((Pu_sp/ n_Hbolt_SST)^2 + (Vu_bm/ n_Vbolt)^2), Vu_grav.SSTLC01-07/n_Vbolts)
Φbolt=	0.75		
Bolt Type (Bolt_Gr_shearTab) =	A325-X		[A325-X default]
Bolt Dia (db_sp)=	0.875	in	
Fnv_bolt=	68.00	ksi	
Anb_sp=	0.601	in^2	= Pi* (db_sp/2)^2
Rn_stBolt_shear=	40.89	kips	= Fnv_bolt* Anb_sb
ΦRn_stBolt_Shear=	30.67	kips	= Rn_stBolt_shear* Φbolt*n_sp
SUM_shearTab_Bolt=	0.541		= Vu_bolt/ ΦRn_stBolt_shear
Check=	OK		OK if SUM_shearTabl_Cbolt <= SUM_Allowed

4.3 SHEAR PLATE GEOMETRY:



a=	3.500	in	Value previously defined
tsp=	0.500	in	[default to try and match beam web thickness]
db =	24.10	in	Value previously defined
Bolt Spacing (S_min)=	2.000	in	= 2 x db_sp-hole See AISC 360 J3.3
S_max=	3.275	in	
Bolt Spacing (Svert)=	2.750	in	OK Check Table J3.4 Note [a], See Section 4.4, 4.8 & 4.9 below
Bolt Spacing (Shorz)=	2.750	in	
h_flange =	9.25	in	Value previously defined
t_stem =	0.750	in	Value previously defined
h_clear =	0.125	in	minimum clearance between link flange and shear tab
Bolt Vert. Edge dist (Lv_min)=	1.125	in	Minimum edge distance equal to one bolt diameter per AISC Table J3.4 note [a]
Lv_sp=	1.125	in	
h_sp max =	15.35	in	= db + tstem - h_flange - (2*h_clear)
Plate Depth (h_sp)=	13.25	in	OK = (n_Vbolt_SST - 1)*Svert + Lv_sp* 2 <= SUM_Plate Depth
Bolt Horz. Edge dist (Lh_min)=	1.125	in	Minimum edge distance for 7/8" bolts per AISC Table J3.4
Lh_sp=	1.750	in	OK
Plate Width (W_sp) =	8.00	in	= (n_Hbolt_SST - 1)*Shorz + Lh_sp + a
Lslot_min=	1.770	in	= db_sp + 1/8 + 0.14* Svert*((n_Vbolts_SST - 1)/2)
LslotH = LslotV =	1.875	in	OK OK if Lslot_min/ Lslot <= 1.03
Shear Plate Geometry Check=	OK		OK if And (Svert = OK, Lv_sp = OK, Lh_sp = OK, Lslot = OK)

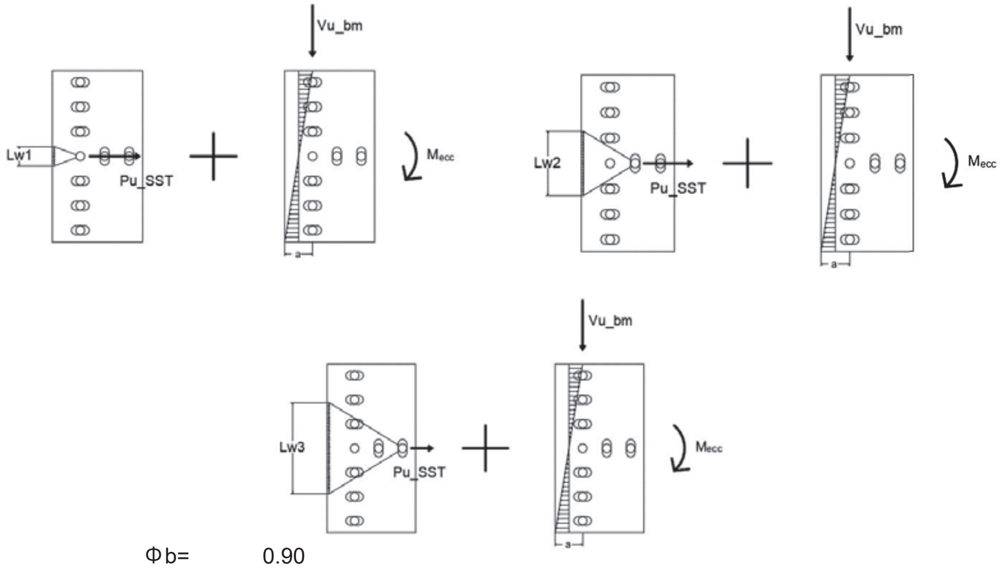
4.4 SHEAR PLATE YIELDING (VERTICAL):

Φyield=	1		
dHole_sp=	1.00	in	= db_sp + 1/8
Asp_Agv=	6.625	in^2	= tsp* h_sp
ΦVy_sp=	198.75	kips	= Φyield* 0.6* Asp_Agv* Fy_sp*n_sp
SUM_spYield=	0.262		= Vu_bm/ ΦVy_sp
Check=	OK		OK if SUM_spYield <= SUM_allowed

4.5 SHEAR PLATE RUPTURE (VERTICAL):

Φrupture=	0.75		
Asp_nv=	4.13	in^2	= hsp* tsp - dHole_sp* n_Vbolts_SST* tsp
ΦVrupture_sp=	120.66	kips	= Φrupture* 0.6* Fu_shearTab* Asp_nv*n_sp
SUM_spRupture=	0.432		= Vu_bm/ ΦVrupture_sp
Check=	OK		OK if SUM_spRupture <= SUM_allowed

4.6 SHEAR PLATE CHECK FOR AXIAL AND MOMENT:



Φb=	0.90		
a=	3.50	in	Value previously defined
Vuy=	52.09	kips	= Vu_bm
Mecc=	182.31	kips*in	= Vuy* a
θ Whitmore=	30	deg	
Lwhitmore1 =	4.916	in	= tan (θ Whitmore)* a* 2 + db_sp
Awhitmore1 =	2.458	in^2	= Lwhitmore1* tsp*n_sp
Lwhitmore2 =	8.092	in	= min(tan (θ Whitmore)* (a+Shorz)*2 + db_sp, hsp)
Awhitmore2 =	4.046	in^2	= Lwhitmore2* tsp*n_sp
Lwhitmore3 =	11.267	in	= min(tan (θ Whitmore)* (a+2*Shorz)*2 + db_sp, hsp)
Awhitmore3 =	5.634	in^2	= Lwhitmore3* tsp*n_sp
Lwhitmore4 =	13.250	in	= min(tan (θ Whitmore)* (a+3*Shorz)*2 + db_sp, hsp)
Awhitmore4 =	6.625	in^2	= Lwhitmore4* tsp*n_sp
Ssp=	14.630	in^3	= tsp* hsp^2*n_sp/ 6
Isp=	96.925	in^4	= tsp* hsp^3*n_sp/ 12
fb1=	12.461	ksi	= Mecc/ Ssp
Awhitmore=	4.046	in^2	= IF(n_Hbolts=1, Awhitmore1, IF(n_Hbolts=2, Awhitmore2, IF(n_Hbolts=3, Awhitmore3, Awhitemore4)))
yb=	2.458	in	= Lwhitmore1/ 2
fb2=	4.624	ksi	= Mecc* yb/ Isp
fa2=	6.382	ksi	= Pu_sp/ Awhitmore
ftot2=	11.005	ksi	= fa2 + fb2
fmax_sp=	12.461	ksi	= max (fb1, ftot2)
Φb*Fy_sp=	45.000	ksi	= Φb* Fy_sp
SUM_sp=	0.277		= fmax_sp/ (Φb* Fy_shearTab), OK if SUM_sp <= SUM_Allowed
Check=	OK		OK if SUM_sp <= SUM_allowed

4.7 SHEAR PLATE TO COLUMN FLANGE FILLET WELD (Plate 1): (AISC 358 Chapter 12, Step 15.4)

ϕ=	0.75		
tsp=	0.500	in	Value previously defined
tw_sp_min=	0.313	in	= 5/8*tsp (Fillet size required to develop plate capacity as per AISC Steel Manual 14th edition)
tw_sp=	0.313	in	= CEILING(tw_sp_min, 1/16)
Vu_bm=	52.088	kips	Value previously defined
Lweld=	12.750	in	= Length per side assuming 1/4" hold off top and bottom
FEXX=	70.00	ksi	= Filler metal classification strength

Fnw=	42.000	ksi	= 0.6*FEXX
Awe=	5.634	in2	= 0.707*(2*Lweld)*tw_sp
ΦRn=	177.468	kips	= Φ*Fnw*Awe
SUM_spWeld=	0.294		= Vu_bm / ΦRn
Check=	OK		= OK if SUM_spWeld <= SUM_spWeld_allowed

4.7a SHEAR PLATE TO COLUMN FLANGE PJP WELD (Plate 2): NOT APPLICABLE FOR ONE SIDE SHEAR PLATE

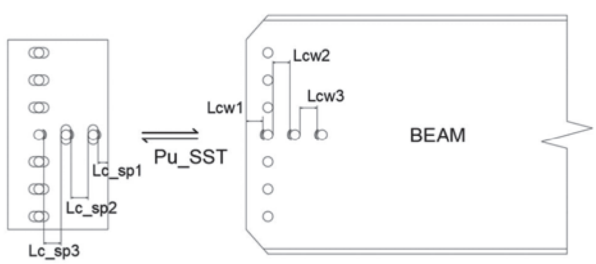
tsp=	0.500	in	Same thickness for both shear plates
ϕ=	0.75		
nsides=	1		
teff_PJP=	0.375	in	= 3/4*tsp
Vu_PJP=	26.044	kips	Vu_bm / 2
Aw=	4.781	in2	= teff_PJP* Lweld *nsides
ΦRn=	150.609	kips	= Φ*0.6*FEXX*Aw
SUM_PJPWeld1=	0.173		= Vu_PJP / ΦRn

PJP Weld P+M Check:

ϕ=	0.8		
APJP=	4.969	in2	= teff_PJP* h_sp
IPJP=	72.694	in	= teff_PJP* h_sp3/12
fPJP=	5.681	ksi	= 0.5* Pu_sp/ APJP + 0.5* Mecc* yb/ IPJP
fPJP_allow=	33.600	ksi	= Φ*0.6*FEXX
SUM_PJPWeld2=	0.169		= Vu_PJP / ΦRn
SUM_PJPWeld=	0.173		= max(SUM_PJPWeld1,SUM_PJPWeld2)
Check=	OK		= OK if SUM_PJPWeld <= SUM_PJPWeld_allowed

4.8 BEAM WEB AND SHEAR TAB BEARING:

CASE 1: HORIZONTAL REACTIONS



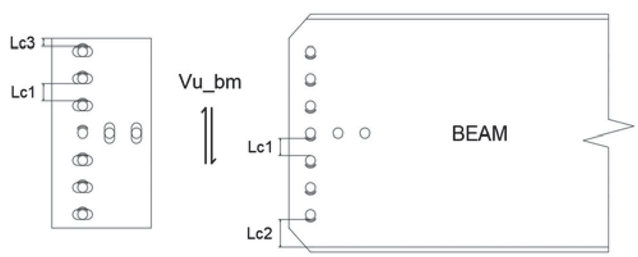
BEAM WEB:

Φbolt=	0.75		
tbw=	0.50	in	value previously defined
Lb_edge=	1.75	in	Edge distance for beam web bolt hole
Lcw1=	1.281	in	= Lb_edge - (db_sp + 1/16)* 0.5
Lcw2=	1.813	in	= Shorz - (db_sp + 1/16)
Lcw3=	0.000	in	= if (n_Hbot_SST = 2, 0, Lcw2)
Lcw4=	0.000	in	= if (n_Hbot_SST = 3, 0, Lcw3)
Lc_bmWeb=	3.094	in	= Lcw1 + Lcw2 + Lcw3 + Lcw4
ΦRn_beamWeb1=	90.49	kips	= Φbolt* 1.2* Lc_bmWeb* tbw* Fu_bmWeb
ΦRn_beamWeb2=	102.38	kips	= Φbolt* 2.4* db_sp* tbw* Fu_bmWeb *n_Hbolt
ΦRn_beamWeb=	90.49	kips	= min (ΦRn_beamWeb1, ΦRn_beamWeb2)
SUM_bmWebX=	0.285		= Pu_SST/ ΦRn_beamWeb
Check=	OK		OK if SUM_spWeld <= SUM_allowed

SHEAR PLATE:

tsp=	0.500	in	value previously defined
Lb_edge=	1.750	in	Edge distance for beam web bolt hole
Lc_sp1 =	1.281	in	= Sp_edgeR - (db_sp + 1/16)* 0.5
Lc_sp2 =	1.813	in	= Shorz - (db_sp + 1/16)
Lc_sp3 =	0.000	in	= if (n_Hbot_SST = 2, 0, Lc_sp2)
Lc_sp4 =	0.000	in	= if (n_Hbot_SST = 3, 0, Lc_sp3)
Lc_sp=	3.094	in	= Lc_sp1 + Lc_sp2 + Lc_sp3 + Lc_sp4
ΦRn_sp1=	84.246	kips	= IF(n_Hbolt_SST=1, fbolt*(1.2* Lc_sp1 * tsp* Fu_sp *n_sp), fbolt*(1.2* Lc_sp2+1.0*(Lc_sp1+Lc_sp3+Lc_sp4)) * tsp*Fu_sp *n_sp)
ΦRn_sp2=	93.844	kips	= Φbolt*(2.4*1+2.0*(n_Hbolts-1))* db_sp* tsp* Fu_sp *n_sp
ΦRn_sp=	84.246	kips	= min (ΦRn_sp1, ΦRn_sp2)
SUM_spX=	0.306		= Pu_SST/ ΦRn_sp
Check=	OK		OK if SUM_spX <= SUM_Allowed

CASE 2: VERTICAL REACTIONS



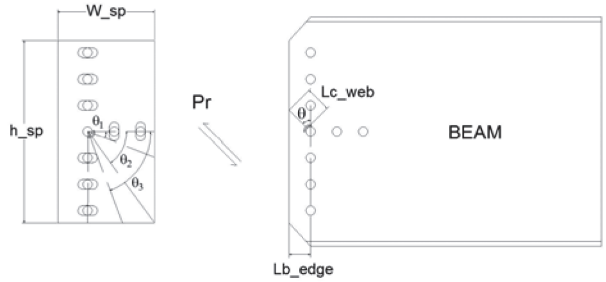
BEAM WEB:

Φbolt=	0.75		Value previously defined
Lc1=	1.813	in	= Svert - (db_sp + 1/16)
Lc2=	5.800	in	= (db - (n_Vbolts - 1)*Svert - 2tbf)*0.5
Lcb_Vert=	13.050	in	= Lc2 + (n_Vbolts - 1)*Lc1
Φ*Rn_bmWebY1=	381.71	kips	= Φbolt* 1.2* Lcb_Vert* tbw* Fu_bm
Φ*Rn_bmWebY2=	255.94	kips	= Φbolt* 2.4* (db_sp* n_bolts_SST)* tbw* Fu_bm
Φ*Rn_bmWebY=	255.94	kips	= min (Φ*Rn_bmWebY1, Φ*Rn_bmWebY2)
SUM_bmWebY=	0.204		= Vu_bm / (Φ*Rn_bmWebY)
Check=	OK		OK if SUM_spWeld <= SUM_allowed

SHEAR PLATE:

Lc3=	0.656	in	= Lv_sp - (db_sp + 1/16)* 0.5
Lct_Vert=	7.906	in	= Lc4 + (n_Vbolts_SST - 1)*Lc1
Φ*Rn_spY1=	201.55	kips	= Φbolt* (1.2*Lc1+ 1.0*(Lc3 + (n_Vbolts - 2))*Lc1)*tsp* Fu_sp *n_sp
Φ*Rn_spY2=	262.03	kips	= Φbolt* (2.4*1+2.0*(n_bolts_SST-1))* db_sp* tsp* Fu_sp *n_sp
Φ*Rn_spY=	201.55	kips	= min (Φ*Rn_spY1, Φ*Rn_spY2)
SUM_spY=	0.258		= Vu_bm/ (Φ*Rn_spY)
Check=	OK		OK if SUM_spY <= SUM_Allowed

CASE 3: COMBINED AXIAL AND VERTICAL REACTIONS



Px=	12.91	kips	= Pu_SST/ n_Hbolts_SST
Py=	10.42	kips	= Vu_bm/ n_Vbolts_SST
Resultant (Pr)=	16.59	kips	= Sqrt (Px^2 + Py^2)
θ =	0.679	radians	= min(Atan (Py/ Px),1.571))

BEAM WEB:

Method 1: T and V circular interaction

SUM_bmWeb θ Bearing1 =	0.123		= SUM_bmWebX^2 + SUM_bmWebY^2

Method 2: Bearing in a diagonal line

Lvg_web =	2.249	in	= Lb_edge/ (cos θ)
Lc_web =	1.749	in	= Lvg_web - dHole_sp/2
Φ*Rn_bmWeb θ 1=	51.150	kips	= Φbolt* 1.2* Lc_web* tbw* Fu_bmWeb
Φ*Rn_bmWeb θ 2=	102.375	kips	= Φbolt* 2.4* db_sp* tbw* Fu_bmWeb*n_Hbolt
Φ*Rn_bmWeb θ =	51.150	kips	= min (Φ*Rn_bmWeb θ 1, Φ*Rn_bmWeb θ 2)
SUM_bmWeb θ Bearing2 =	0.324		= Pr/ Φ*Rn_bmWeb θ
SUM_bmWeb θ Bearing =	0.324		= Max(SUM_bmWeb θ Bearing1, SUM_bmWeb θ Bearing2)
Check=	OK		OK if SUM_spY <= SUM_Allowed

SHEAR PLATE:

Method 1: T and V circular interaction

SUM_sp θ Bearing1 =	0.161		= SUM_spX^2 + SUM_spY^2

Method 2: Bearing in a diagonal line

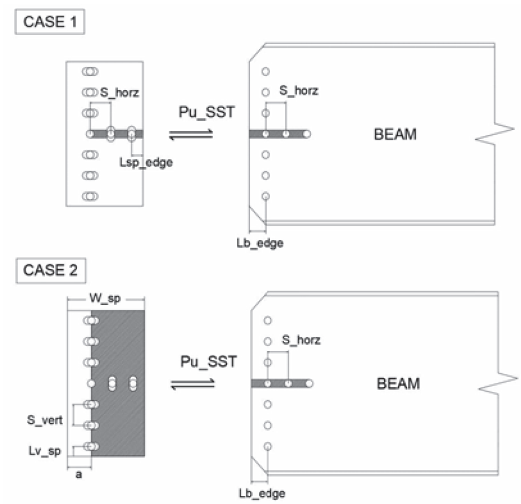
θ 1=	0.329	radians	= arctan((0.5*LslotH)/Shorz)
θ 2=	0.974	radians	= arctan((0.5*h_sp)/(w_sp-a))
θ '=	0.329	radians	= arctan((0.5*LslotV)/Svert)
θ 3=	1.242	radians	= (PI()*0.5)- θ '
θ =	0.679	radians	Value previously defined

	CASE 1	CASE 2	CASE 3	CASE 4
min. θ =	0	θ1	θ2	θ3
max θ =	θ1	θ2	θ3	1.571
Check=	NO	YES	NO	NO
La_sp1=	-	5.782	-	-

La_sp=	5.782	in	
Φ*Rn_sp θ 1=	169.124	kips	= Φbolt* 1.2* La_sp* tsp* Fu_sp*n_sp
Φ*Rn_sp θ 2=	102.375	kips	= Φbolt* 2.4* db_sp* tsp* Fu_sp*n_Hbolts*n_sp
Φ*Rn_sp θ =	102.375	kips	= min (Φ*Rn_sp θ 1, Φ*Rn_sp θ 2)
SUM_sp θ bearing2=	0.162		= Pr / (Φ*Rn_sp θ)
SUM_sp θ bearing=	0.162		= max(SUM_sp θ Bearing1, SUM_sp θ bearing2)
Adequate =	OK		OK if SUM_sp θ bearing <= SUM_sp θ bearing_allowed

4.9 BEAM WEB AND SHEAR TAB BLOCKSHEAR CHECK:

CASE 1: HORIZONTAL REACTIONS



BEAM WEB:

Φblockshear=	0.75		
Ubs=	1.00		
Lb_edge=	1.750	in	value previously defined
Lc_bmWeb =	3.094	in	value previously defined
Lh_bmWeb=	9.000	in	= 2* (Lb_edge+Shorz*(n_Hbolt_SST - 1))
Agv_bmWebHorz=	4.500	in^2	= Lh_bmWeb*tbw
Ant_bmWebHorz=	0.000	in^2	= 0
Anv_bmWebHorz=	4.500	in^2	= Agv_bmWebHorz
Rn_bmWebHorz1=	175.500	kips	= 0.6* Fu_bmWeb* Anv_bmWebHorz + Ubs* Fu_bmWeb* Ant_bmWebHorz
Rn_bmWebHorz2=	135.000	kips	= 0.6* Fy_bmWeb* Agv_bmWebHorz + Ubs* Fu_bmWeb* Ant_bmWebHorz
Rn_bmWebHorz=	101.250	kips	= Φblocksher*min (Rn_bmWebHorz1, Rn_bmWebHorz2)
SUM_bmWebHorz_BS=	0.255		= Pu_SST/ Rn_bmWebHorz
Adequate =	OK		OK if SUM_bmWebHorz <= SUM_Allowed

SHEAR PLATE:

CASE 1:

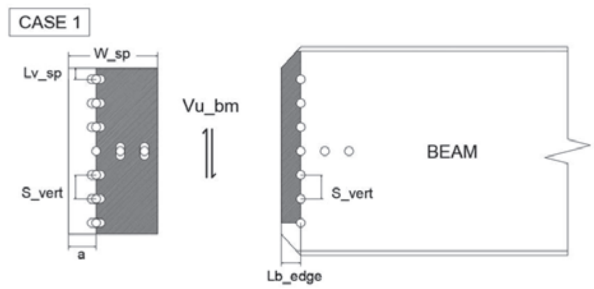
Lsp_edge=	1.750	in	value previously defined
Lc_sp =	3.094	in	value previously defined
Lh_sp1=	9.000	in	= 2* (Lsp_edge + Shorz*(n_Hbolt_SST - 1))
Agv_spHorz1=	4.500	in^2	= Lh_sp* tsp*n_sp
Ant_spHorz1=	0.000	in^2	= 0
Anv_spHorz1=	4.500	in^2	= Agv_bmWebHorz
Rn_spHorz1=	175.50	kips	= 0.6* Fu_sp* Anv_spHorz + Ubs* Fu_sp* Ant_spHorz
Rn_spbHorz2=	135.00	kips	= 0.6* Fy_sp* Agv_spHorz + Ubs* Fu_sp* Ant_spHorz
ΦRn_spHorz1=	101.25	kips	= Φblocksher* min (Rn_spHorz1, Rn_spbHorz2)
SUM_spHorz_BS1=	0.255		= Pu_SST/ Rn_spHorz

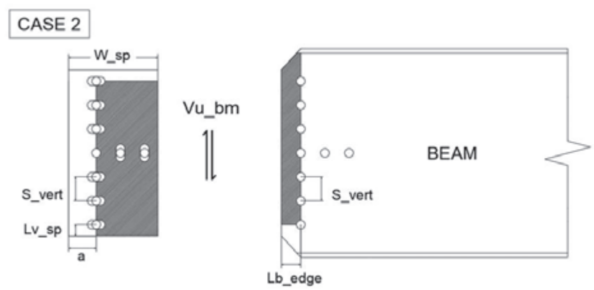
CASE 2:

Lh_sp2=	8.250	in	=h_sp-(n_Vbolts*dHole_sp)
Agv_spHorz2=	0.000	in^2	= 0
Ant_spHorz2=	4.125	in^3	= Lh_sp2*tsp*n_sp
Anv_spHorz2=	0.000	in^2	= 0
Rn_spHorz3=	268.13	kips	= 0.6* Fu_sp* Anv_spHorz2 + Ubs* Fu_sp* Ant_spHorz2

Rn_spHorz4=	268.13	kips	= 0.6* Fy_sp* Agv_spHorz2 + Ubs* Fu_sp* Ant_spHorz2
ΦRn_spHorz2=	201.09	kips	= Φblockshear* min (Rn_spHorz3, Rn_spbHorz4)
SUM_spHorz_BS2=	0.128		= Pu_sp/ Rn_spHorz2
SUM_spHorz_BS=	0.255		= Pu_SST/ Rn_spHorz
Check=	OK		= OK if SUM_spHorz_BS <= SUM_spHorz_BS_allowed

CASE 2: VERTICAL REACTIONS





BEAM WEB:

S_vert=	2.750	in	= Svert
Lb_edge=	1.75	in	value previously defined
h_bmWeb=	16.800	in	= (d- (n_Vbolt_SST - 1)*Svert - 2tbf)*0.5 + (n_Vbolt_SST - 1)*Svert)
Agv_bmWeb1=	8.400	in^2	= tbw* h_bmWeb
Anv_bmWeb1=	6.150	in^2	= tbw* (h_bmWeb - (n_Vbolt_SST-0.5)* dHole_sp)
Ant_bmWeb1=	0.625	in^2	= (Lb_edge - dHole_sp/2)* tbw
Rn_bmWebVert1=	280.475	kips	= 0.6* Fu_bmWeb* Anv_bmWeb1 + Ubs*Fu_bmWeb* Ant_bmWeb1
Rn_bmWebVert2=	292.625	kips	= 0.6* Fy_bmWeb* Agv_bmWeb1 + Ubs*Fu_bmWeb* Ant_bmWeb1
ΦRn_bmWebVert=	210.36	kips	= Φblockshear* min (Rn_bmWebVert1, Rn_bmWebVert2)
SUM_bmWebVert_BS=	0.248		= Vu_bm/ ΦRn_bmWebVert
Check=	OK		OK if SUM_bmWebVert_BS <= SUM_Allowed

SHEAR PLATE:

CASE 1:

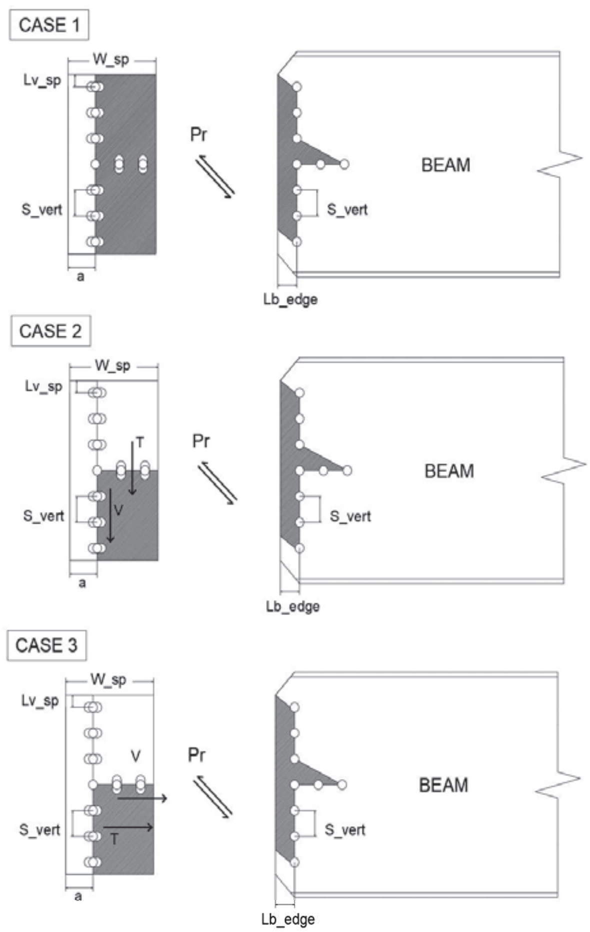
hsp=	13.250	in	value previously defined
Agv_sp1=	6.625	in^2	= hsp* tsp*n_sp
Anv_sp1=	4.125	in^2	= tsp* (hsp - (n_Vbolt)* dHole_sp)*n_sp
Ant_sp1=	0	in^2	= (Lsp_edge - dHole_sp/2)* tsp
Rn_spVert1=	160.88	kips	= 0.6* Fu_shearTab* Anv_sp1 + Ubs* Fu_shearTab* Ant_sp1
Rn_spVert2=	198.75	kips	= 0.6* Fy_shearTab* Agv_sp1 + Ubs* Fu_shearTab* Ant_sp1
ΦRn_spVert=	120.66	kips	= Φblockshear* min (Rn_spVert1, Rn_spVert2)
SUM_spVert_BS1=	0.432		= Vu_bm/ ΦRn_spVert

CASE 2:

Lv_sp=	1.125	in	value previously defined

hsp=	13.250	in	value previously defined
Lslot=	1.875	in	value previously defined
Agv_sp1=	5.813	in^2	= hsp* tsp*n_sp
Anv_sp1=	4.375	in^2	= tsp* (hsp - (n_Vbolt_SST-0.5)* dHole_sp)*n_sp
Ant_sp1=	1.781	in^2	= (W_sp - a - LslotH/2)* tsp*n_sp
Rn_spVert1=	286.406	kips	= 0.6* Fu_shearTab* Anv_sp1 + Ubs* Fu_shearTab* Ant_sp1
Rn_spVert2=	290.156	kips	= 0.6* Fy_shearTab* Agv_sp1 + Ubs* Fu_shearTab* Ant_sp1
ΦRn_spVert=	214.80	kips	= Φblockshear* min (Rn_spVert1, Rn_spVert2)
SUM_spVert_BS2=	0.242		= Vu_bm/ ΦRn_spVert
SUM_spVert_BS=	0.432		= MAX(SUM_spVert_BS1, SUM_spVert_BS1)
Check=	OK		= OK if SUM_spVert_BS <= SUM_spVert_BS_allowed

CASE 3: COMBINED AXIAL AND VERTICAL REACTIONS



BEAM WEB:

Method 1: N and V circular interaction

SUM_bmWeb θ _BS1 =	0.126		= SUM_bmWebHorz_BS2 + SUM_bmWebVert_BS2

Method 2: Failure along hole edges

Pr_BS=	58.14	kips	=SQRT((Pu_sp^2)+(Vu_bm)^2)
θ =	0.679	radians	value previously defined
Lb_edge=	1.750	in	value previously defined
Lbw_diag=	10.14	in	min(sqrt(Svert2+Shorz2) + Lb_edge, (Lb_edge + Shorz)/cosq)

Lvg_web=	10.14	in	= 2*Lb_edge + (n_Hbolt_SST - 1)*Shorz + sqrt(Svert^2 + ((n_Hbolt_SST - 1)*Shorz))^2)
Lvn_web=	7.14	in	= Lvg_web - 2*dHole_sp - (n_Hbolt - 1)*dHole_sp
Ltg_web=	8.25	in	= (n_Vbolt_SST - 2)*Svert
Ltn_web=	5.25	in	= Ltg_web - (n_Vbolt_SST - 2)*dHole_sp
Agv_web=	5.07	in^2	= Lvg_web* tbw
Anv_web=	3.57	in^2	= Lvn_web* tbw
Agt_web=	4.13	in^2	= Ltg_web* tbw
Ant_web=	2.63	in^2	= Ltn_web* tbw
Rn_bmWeb_BS1=	309.84	kips	= 0.6* Fu_bmWeb* Agv_web + Ubs* Fu_bmWeb* Ant_web
Rn_bmWeb_BS2=	322.71	kips	= 0.6* Fy_bmWeb* Agv_web + Ubs* Fu_bmWeb* Ant_web
ΦRn_bmWeb_BS=	232.38	kips	= Φblockshear* min (Rn_bmWeb_BS1, Rn_bmWeb_BS2)
SUM_bmWebθ_BS2=	0.250		= 2*Pr/ ΦRn_bmWeb_BS
SUM_bmWebq_BS=	0.250		= max(SUM_bmWebθ_BS1, SUM_bmWebθ_BS2)
Check=	OK		= OK if SUM_bmWebθ_BS <= SUM_bmWebθ_BS_allowed

SHEAR PLATE:

Method 1: N and V circular interaction

SUM_spθ_BS1 =	0.251		= SUM_spVert_BS^2 + SUM_spHorz_BS^2

Method 2: Failure along hole edges

CASE 1:

Lvg_sp1=	13.25	in	= h_sp
Lvg_sp1=	8.25	in	= Lvg_sp1 - (n_Vbolts*dHole_sp)
Ltn_sp1=	0.00	in	= 0
Agv_sp1=	6.625	in	= Lvg_sp* tsp*n_sp
Anv_sp1=	4.125	in	= Lvn_sp* tsp*n_sp
Ant_sp1=	0.00	in	= Ltn_sp* tsp*n_sp
Rn_sp_BS1=	160.88	kips	= 0.6* Fu_sp* Anv_sp1 + Ubs* Fu_sp* Ant_sp1
Rn_sp_BS2=	198.75	kips	= 0.6* Fy_sp* Agv_sp1 + Ubs* Fu_sp* Ant_sp1
ΦRn_sp_BS1=	120.66	kips	= Φblockshear* min (Rn_sp_BS1, Rn_sp_BS2)
SUM_spθ_BS2=	0.482		= Pr_BS / ΦRn_sp_BS1

CASE 2:			
Pv=	31.3		=Ceiling(n_vBolts/2)/n_vBolts*Vu_bm
Pr_BS1=	40.5		=SQRT(Pv^2 + Pu_sp^2)
Lvg_sp2=	6.63	in	= h_sp/2
Lvn_sp2=	4.13	in	= Lvg_sp2 - (((n_Vbolt_SST-1)/2)*dHole_sp) - (0.5*dHole_sp)
Ltn_sp2=	3.00	in	= (W_sp - a) - ((n_Hbolts-1)*dHole_sp) - (0.5*dHole_sp)
Agv_sp2=	3.313	in	= Lvg_sp* tsp*n_sp
Anv_sp2=	2.063	in	= Lvn_sp* tsp*n_sp
Ant_sp2=	1.500	in	= Ltn_sp* tsp*n_sp
Rn_sp_BS3=	177.94	kips	= 0.6* Fu_sp* Anv_sp2 + Ubs* Fu_sp* Ant_sp2
Rn_sp_BS4=	196.88	kips	= 0.6* Fy_sp* Agv_sp2 + Ubs* Fu_sp* Ant_sp2
ΦRn_sp_BS2=	133.45	kips	= Φblockshear* min (Rn_sp_BS1, Rn_sp_BS2)
SUM_spθ_BS3=	0.304		= Pr_BS / ΦRn_sp_BS1
CASE 3:			
Lvg_sp3=	4.50	in	= W_sp - a
Lvn_sp3=	3.00	in	= (W_sp - a) - ((n_Hbolts-0.5)*dHole_sp)
Ltn_sp3=	4.13	in	= Lvg_sp2 - (((n_Vbolt_SST-1)/2)*dHole_sp) - (0.5*dHole_sp)
Agv_sp3=	2.250	in	= Lvg_sp* tsp*n_sp
Anv_sp3=	1.500	in	= Lvn_sp* tsp*n_sp
Ant_sp3=	2.063	in	= Ltn_sp* tsp*n_sp
Rn_sp_BS5=	192.56	kips	= 0.6* Fu_sp* Anv_sp3 + Ubs* Fu_sp* Ant_sp3
Rn_sp_BS6=	201.56	kips	= 0.6* Fy_sp* Agv_sp3 + Ubs* Fu_sp* Ant_sp3
ΦRn_sp_BS3=	144.42	kips	= Φblockshear* min (Rn_sp_BS5, Rn_sp_BS6)
SUM_spθ_BS4=	0.281		= Pr_BS1 / ΦRn_sp_BS3
SUM_spθ_BS=	0.482		= max(SUM_spθ_BS1, SUM_spθ_BS2, SUM_spθ_BS3, SUM_spθ_BS4)
Check=	OK		= OK, if SUM_spθ_BS <= SUM_spθ_BS_allowed

DCR Summary:

Component	Shear	Yielding	Rupture	P+M	Bearing	BS	Max	Check
Beam Web:	-	-	-	-	0.324	0.255	0.324	OK
Shear Plate:	-	0.262	0.432	0.277	0.306	0.482	0.482	OK
Shear Plate Bolt:	0.541	-	-	-	-	-	0.541	OK
SP Fillet Weld:	0.294	-	-	-	-	-	0.294	OK

SIMPSON STRONG-TIE COMPANY INC.
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(800) 999-5099
www.strongtie.com

Job Name: Demo Structure - Sample Calculations

Date Printed: May 23, 2022

Job ID: ES-# 221804

COLUMN DESIGN DETAILS

3.1 CURRENT MEMBER:

				R=	8
Column Unique Name:	36	Current Story=	Story1	Top Story=	Story2
Column size:	W24X131	Story Above=	Story2	Use Doubler PL?=	NO
Pu (kips):	139.59	Left Beam size:	NA	Right Beam size:	W24X104
Hcc_b (in):	144	Left Beam ID=	NA	Right Beam ID=	32
Hcc_t (in):	144	db_left (in)=	NA	db_right (in)=	24.1
Stiffener (Y/N):	YES	Left Link Size:	NA	Right Link Size:	YL6-4.5
Doubler (Y/N):	NO	No. of conn.=	0	No. of conn.=	2
		Left Beam Lh (in):	0	Right Beam Lh (in):	328.25
		Vu_Gravity (kips):	0	Vu_Gravity (kips):	17

3.2 LINK PROPERTIES:

	Left Link	Right Link		
Thickness (t_stem) =	0.000	0.750	in	
Yield Width (w_stemYield)=	0.000	4.500	in	
Thickness (t_flange)=	0.000	1.250	in	
Spacing Along Length (horiz) (bolt_s_flange)=	0.000	6.000	in	
Gauge Along Width (vert.) (bolt_g_flange)=	0.000	5.750	in	
Flange height (H_flange) =	0.000	9.250	in	
a=	0.000	3.500	in	
Pcap_Link=	0.0	263.3	kips	= if (R=3, Py_link, Pr_Link)
Mcap=	0.00	6541.76	k-in	= Pcap_Link *(db+tstem)

3.3 CHECK STRONG BEAM WEAK LINK REQUIREMENTS:

Column Size=	W24X131		Value previously defined
dc=	24.5	in	Looked up value
tcf=	0.96	in	Looked up value
Fyc=	50	ksi	Value previously defined
Agc=	38.6	in^2	Looked up value
Zcx=	370	in^3	Looked up value
Top Story?=	NO		=YES, IF Current Story = Top Story, Note: AISC 341-16 E3.4a Exception (a) (1)
Σ Mp_col=	34324	kip*in	= Zcx* (Fyc - Pu_colPositive/ Agc) * if(top story,1 else 2)
Column Pu=	139.59	kips	Max Axial Force of column per Omega Combo (LC31-34) From ETABS
Column Pc=	1930.00	kips	= Fyc* Agc
Pu<0.3*Pc?=	YES		AISC 341-16 E3.4a Exception (a)

AISC 360		Table B4.1b
Col b/2/t=	6.7	Compact
Col h/tw=	35.6	Compact

	Left Side	Right Side		
Mcap=	0.00	6541.76	kip*in	Value previously defined
Lh=	0.00	296.8	in	=Lcc-dc
Vbm_gravity=	0.00	17.00		=Beam shear from SST_LC8
Vu_bm=	0.00	61.09	kip	= (No. of conn.* Mpr)/ Lh + Vbm_gravity
a=	0.00	3.50	in	Value previously defined
Muv=	0.0	962.2	kips*in	= Vu_bm* (dc/2 + a)
Σ Mpb=	0	7504	kip*in	= Mpr + Muv
SCWB_DCR=	0.000	0.219		= Σ Mpb / Σ Mp_col
SCWB_SUM_total=	0.219			Note: SCWB not applicable at top story or IMF Frame or R=3
Check=	OK			Ok, if SCWB_SUM_total <= SCWB_SUM_allowed

3.4 COLUMN PANEL ZONE CHECK (WITHOUT DOUBLER PLATE):

Φv_PZ=	0.90		
Fyc=	50.00	ksi	Value previously defined
dc=	24.50	in	Value previously defined
tcw=	0.61	in^2	Value previously defined
Agc=	38.60	in^2	Value previously defined
Pc=	1930.00	kips	= Fyc* Agc
Pu=	139.59	kips	Value previously defined
ΦRn_PZ=	400.21	kips	= Φv_PZ* if (Pu < 0.4* Pc, 0.6* Fyc* dc* tcw, 0.6* Fyc* dc* tcw* (1.4 - Pu/Pc)) (J10-9 & J10-10 AISC 360-16)

	Left Side	Right Side		
Vu_c=	0.00	45.43	kips	= Σ Mpr/((Hcc_b+Hcc_t)/2), 0 for top column
Pr_link=	0.00	263.25	kips	= Pu_link
Ru=	0.00	217.82	kips	= Pr_link - Vu_c
DCR_PZ_NODBLR=	0.000	0.544		= Pr_link - Vu_c / ΦRn_PZ
DCR_PZ_NODBLR_Total=	0.544			

dz=	22.60	in	=db-2tbf
wz=	22.58	in	=dc-2tcf
tcw=	0.605	in	Value previously defined
DCR_PZ_geometry=	0.000		=[(dz+wz)/90] / tcw, 0 for R=3 and R=4.5
DCR_PZ_total=	0.544	kips	= Σ Mpb/((Hcc_b+Hcc_t)/2), 0 for top story
Check=	OK		Ok, if SUM_PZ <= DCR_PZ_Allowable

3.4.1 COLUMN WEB DOUBLER PLATE CHECK:

Plate Required=	NO	<----If NO, Skip 3.4.1	USE Plug Weld? NO
Fy_dpl=	50	ksi	=Doubler Yield Strength
ΦRn_PZ_NODBLR=	400.21	kips	= Φv_pz* if (Pu < 0.4* Pc, 0.6* Fy_col* dc* tcw, 0.6* Fy_col* dc* tcw* (1.4 - Pu/Pc))
ΦRn_PZ_req=	0.00	kips	= Additional capcaity required
ΦRn_PZ_NEW=	400.21	kips	= ΦRn_PZ_NODBLR + fRn_PZ_req
t_dblr_strength=	0.625	in	
dz=	22.60	in	=db-2tbf, if different db, use average
wz=	22.58	in	=dc-2tcf
t_dblr_geom=	0.63	in	=if(R=3 or R=4.5, 0, Ceiling(((dz+wz)/90),0.125))
t_dpl=	0.625	in	=Ceiling(((dz+wz)/90),0.125)
t_dblr_min=	0.625	in, with plug weld	=max([(dz+wz)/90] - tcw,0.25,t_dblr_strength)
t_dblr_use=	0.625	in	Doubler Plate thickness
ΦRn_PZ_DBLR=	781.25	kips	= Φv_pz* if (Pu < 0.4* Pc, 0.6* Fy_col* dc* (tcw+t_dblr), 0.6* Fy_col* dc* (tcw+t_dblr)* (1.4 - Pu/Pc))

	Left Side	Right Side		
DCR_PZ_NEW=	0.000	0.279	kips	=Total demand on column web
DCR_PZ_total_NEW=	0.279	OK		
DCR_PZ_geometry_Web=	0.830			=[(dz+wz)/90] / tcw, 0 for R=3 and R=4.5
DCR_PZ_geometry_Total=	0.408		in, with plug weld	=[(dz+wz)/90] / (tcw+t_dblr) , 0 for R=3 and R=4.5
DCR_PZ_Total=	0.830			=max of DCR_PZ (strength), DCR_PZ (geometry_web), DCR_PZ (geometry_DP)

3.4.2 DOUBLER TO COLUMN WED/CONTINUITY PLATE WELD

Option 1: Doubler without Continuity plates

t_dblr=	0.625	in	doubler plate thickness
tcw=	0.605	in	column web thickness
min t=	0.625	in	=min of tDP and tcw, rounded up to nearest 1/16
wfillet=	4/16	in	From AISC 360 Table J2.4

Option 2A: Extended Doubler Plate

t_dblr=	0.625	in	doubler plate thickness
wfillet=	0.391	in	=5/8*tDP (develop shear strenght of the doubler Plate), rounded to nearst 1/16
wfillet_use=	7/16	in	
# of sides=	2		

Option 2B: Doubler plates placed between continuity plates

75% of the available shear yield strength of the full doubler plate thickness (AISC 341 E3.3(2))

t_dblr=	0.625	in	doubler plate thickness
Vn_doubler=	0.625	kips	=t_dblr, same thickness to develop doubler Plate capacity
wfillet_use=	10/16	in	
# of sides=	1		

3.4.3 DOUBLER TO WEB PLUG WELD (IF REQUIRED)

Doubler Plate Plug-Weld Used?	NO			
dz_mod=	11.30	in, 0.5*dz		
wz_mod=	11.29	in, 0.5*wz		
(dz_mod+wz_mod)/90	0.251	OK		
PlugWeld_Dia=	15/16	in	Weld Depth=	10/16 in

3.4.4 DOUBLER TO COLUMN FLANGE WELD

Option 1: Fillet weld to develop doubler plate shear capacity

t_dblr=	0.625	in	doubler plate thickness
Vn=	18.75	kips	0.6*Fy*Agv (per 1 in length)
wfillet=	14/16	in	Vn/1.39, rounded up to 1/16"

Option 2: Groove weld per AWS D1.8 Clause 4.3

3.5 CHECK UNSTIFFENED COLUMN FLANGE AND WEB:

SMF_Sides= 1

	Left Side	Right Side		
dbm=	NA	24.1	in	dbeam
tbf=	NA	0.75	in	
d_stiff_cap=	NA	5.5	in	
d_stiffTop=	NA	49.0	in	Loc. of top stifferener to top of column cap, assume 2 dc if not at top story
d_stiffBot=	NA	73.1	in	Loc. of bottom stiffener to top of column cap = d + d_stiffTop

3.6 COLUMN WEB LOCAL YIELDING (AISC 360 J10.2):

ΦWLY=	1		=J10.2 AISC 360-16
dc=	24.5	in	Value previously defined
kdes=	1.46	in	Looked up value

	Left Side	Right Side		
t_flange=	0.000	1.250	in	Link flange thickness
t_stem=	0.000	0.750	in	Link stem thickness
l_b=	0.00	3.25	in	= 2* t_flange + t_stem
Ct_top=	NA	1.00		= if (d_stiffTop <= dc, 0.5, 1)
Ct_bot=	NA	1.00		= if (d_botTop <= dc, 0.5, 1)
ΦRn_WLYmoreD=	220.83	319.14	kips	= ΦWLY* Fy_col* tcw* (5* kdes + l_b)
ΦRn_WLYlessD=	110.41	208.73	kips	= ΦWLY* Fy_col* tcw* (2.5* kdes + l_b)
ΦRn_WLY_top=	NA	319.14	kips	= if (d_stiffTop > d, ΦRn_WLYmoreD,ΦRn_WLYlessD)
ΦRn_WLY_bot=	NA	319.14	kips	= if (d_stiffBot > d, ΦRn_WLYmoreD, ΦRn_WLYlessD)

3.7 COLUMN WEB LOCAL CRIPPLING (AISC 360 J10.3):

ΦWLC=	0.75		
Es=	29000	ksi	
tcf=	0.96	in	Value previously defined
tcw=	0.605	in	Value previously defined
ΦRn_WLC_more0.5D=	399.4	kip	= ΦWLC* 0.8* tcw^2* (1 + 3*(l_b/dc)*(tcw/tcf)^1.5)* Sqrt(Es* Fy_col* tcf/ tcw)
AISC J10-5a=	199.72	kips	= 0.5* fRn_WLC_more0.5D
AISC J10-5b=	194.1	kips	= fWLC* 0.4* tcw^2* (1 + 3*(4* l_b/dc - 0.2)*(tcw/tcf)^1.5)* Sqrt(Es* Fy_col* tcf/ tcw)
ΦRn_WLC_less0.5D=	199.72	kips	= if (l_b/ dc <= 0.2, AISC J10-5a, AISC J10-5b)

	Left Side	Right Side		
ΦRn_WLC_top=	NA	399.45	kips	= if (d_stiffTop > d, ΦRn_WLC_more0.5D, ΦRn_WLC_less0.5D)
ΦRn_WLC_bot=	NA	399.45	kips	= if (d_stiffBot > d, ΦRn_WLC_more0.5D, ΦRn_WLC_less0.5D)

3.8 COLUMN WEB COMPRESSION BUCKLING (AISC 360 J10.5):

Note: This check is for 2-sided SMF connection only

ΦWCB=	0.9		
h=	21.58	in	= dc - 2*kdes
ΦRn_WCB=	266.9	kips	AISC J10-8, = ΦWCB* 24* tcw^3* sqrt(Es* Fy_col)/h

	Left Side	Right Side		
ΦRn_WLC_top=	NA	266.90	kips	= if (d_stiffTop < d, Φ*Rn_WCB*0.5, ΦRn_WCB)
ΦRn_WLC_bot=	NA	266.90	kips	= if (d_botTop < d, ΦRn_WCB*0.5, ΦRn_WCB)

3.9 SUMMARY OF LOCAL CAPACITIES:

Left Side:

At Top Stiffener:

ΦRn_WLY_top=	NA	kips
ΦRn_WLC_top=	NA	kips
ΦRn_WCB_top=	NA	kips

if (SMF_Sides = 1, ΦRn_Stiff_top = min (ΦRn_WLY_top, ΦRn_WLC_top), min (ΦRn_WLY_top, ΦRn_WLC_top, ΦRn_WCB_top))

ΦRn_Stiff_top=	0.00	kips
Top Stiffener Required=	NO	
Top Stiff Provided?:	YES	

At Bottom Stiffener:

ΦRn_WLY_bot=	NA	kips
ΦRn_WLC_bot=	NA	kips
ΦRn_WCB_bot=	NA	kips

if (SMF_Sides = 1, ΦRn_Stiff_bot = min (ΦRn_WLY_bot, ΦRn_WLC_bot), min (ΦRn_WLY_bot, ΦRn_WLC_bot, ΦRn_WCB_bot))

ΦRn_Stiff_bot=	0.00	kips
Bottom Stiffener Required=	NO	
Bottom Stiff Provided?:	YES	
Stiff Req'd=	NO	

Right Side:

At Top Stiffener:

ΦRn_WLY_top=	319.14	kips
ΦRn_WLC_top=	399.45	kips
ΦRn_WCB_top=	266.90	kips

if (SMF_Sides = 1, ΦRn_Stiff_top = min (ΦRn_WLY_top, ΦRn_WLC_top), min (ΦRn_WLY_top, ΦRn_WLC_top, ΦRn_WCB_top))

ΦRn_Stiff_top=	319.14	kips
Top Stiffener Required=	NO	
Top Stiff Provided?:	YES	

At Bottom Stiffener:

ΦRn_WLY_bot=	319.14	kips
ΦRn_WLC_bot=	399.45	kips
ΦRn_WCB_bot=	266.90	kips

if (SMF_Sides = 1, ΦRn_Stiff_bot = min (ΦRn_WLY_bot, ΦRn_WLC_bot), min (ΦRn_WLY_bot, ΦRn_WLC_bot, ΦRn_WCB_bot))

ΦRn_Stiff_bot=	319.14	kips
Bottom Stiffener Required=	NO	
Bottom Stiff Provided?:	YES	
Stiff Req'd=	NO	

3.10 WEB SIDESWAY BUCKLING (AISC 360-J10.4):

Note: Assume compression flange is NOT restrained against rotation

	Left Side	Right Side		
ΦWSB=	0.85	0.85		
Hcc=	144.00	144.00	in	Value previously defined
bcf=	12.9	12.9	in	Column flange width
Scx=	329	329	in^3	Looked up value
Lb=	NA	131.95	in	= Hcc - d/2
My=	16450	16450	kips*in	= Scx* Fyc
Mu=	0	6542	kips*in	=Mcap
Cr=	960000	960000	ksi	= if (Mu < My, 960000, 480000)
(h/tcw)/(Lb/bcf)=	NA	3.4872		= (h/ tcw)/ (Lb/ bcf)
ΦRn_WSB=	NA	6318.52	kips	AISC J10-7, = (ΦWSB* Cr* tcw^3* tcf/ h^2)/ (0.4* Cr^3)
WSB_check=	N/A	N/A		Not applicable
BotStiffener_Bracing_Req'd=	NO	NO		= if (And (ΦRn_WSB < Pr_link, WSB_check = "Applicable"), "YES", "NO")

3.11 COLUMN STIFFENER DESIGN:

	Left Side	Right Side		
Fsu_top=	0.00	0.00	kips	= if (Pcap_link - ΦRn_Stiff_top < 0, 0, Pcap_link - ΦRn_Stiff_top)
Fsu_bot=	0.00	0.00	kips	= if (Pcap_link - ΦRn_Stiff_bot < 0, 0, Pcap_link - ΦRn_Stiff_bot)
Fsu=	0.00	0.00	kips	= max (Fsu_top, Fsu_bot)* SMF_Sides

3.12 CONTINUITY PLATE REQUIREMENTS BASED ON GEOMETRY AND TENSION YIELDING:

Φt=	0.9		Value previously defined
Fy_stiff=	50	ksi	Value previously defined
Kdet=	1.875	in	Looked up value
K1=	1.125	in	Looked up value
tcwdet/2 =	0.313	in	Looked up value
tcf(det) =	0.938	in	Looked up value

	Left Side	Right Side		
As_min=	0.000	0.00	in^2	= Fsu/ (Φt* Fy_Stiff)
Lclip_web_min=	2.438	2.438	in	= Kdet - tcf + 1.5
Lclip_web=	2.438	2.438	in	= Ceiling (Lclip_web_min, 0.0625)
Lclip_flange_min=	0.813	0.813	in	= K1 - tcwdet/2
Lclip_flange=	0.813	0.813	in	= Ceiling (Lclip_flange_min, 0.0625)
bstiff_min=	3.9975	3.9975	in	AISC J10.8, = bcf/3 - tcw/2
bstiff_max=	6.1475	6.1475	in	= (bcf - tcw)/2
bstiff=	6.1250	6.1250	in	= Floor (bstiff_max,0.125), Each side of column web

tstiff_USE?	YES	YES	in	Input value
t_stiff_tension=	0.000	0.000	in	= As_min/ (bstiff_Lclip_flange)/ 2/ SMF_Sides, OK, if t_stiff_tension <= tstiff
tstiff_min=	0.500	0.500	in	= max (t_stem/2, bstiff/16, t_STP_user), OK, if tstiff_min <= tstiff
tstiff_max=	0.000	0.750	in	= t_stem, OK, if tstiff_max >= tstiff
t_stiff_Use=	0.000	0.500		

Lstiff_Pdepth_ini=	11.29	in	= 0.5* dc - tf, For Single sided connections
Lstiff_Fdepth_ini=	22.58	in	= dc - 2*tcf, For double sided connections
Lstiff_ini=	22.58	in	= if (SMF_Sides = 1, Lstiff_Pdepth_ini, Lstiff_Fdepth_ini)
Lstiff=	22.56	in	= Floor (Lstiff_ini, 0.0625)

3.13 FULL DEPTH STIFFENER PLATE FOR 2-SIDED MOMENT CONNECTIONS ONLY:

	Left Side	Right Side		
Φc=	0.9	0.9		
Kstiff=	0.75	0.75		
Astiff_fd=	4.4	10.5173	in^2	= 2* bstiff* tstiff + 12* tcw^2
Istiff_fd=	0.1	88.637	in^4	= 12* tcw* tcw^3/12 + 2* (tstiff* bstiff^3/ 12+ bstiff* tstiff* (bstiff/2 +tc/2)^2)
rstiff_fd=	0.175	2.903	in	= Sqrt (Istiff_fd/ Astiff_fd)
Kstiff*Lstiff_fd/rstiff_fd=	96.891	5.829		= Kstiff* Lstiff_fd/ rstiff_fd
Fe=	30	8424	ksi	= Pi^2* 29000/ (Kstiff* Lstiff_fd/ rstiff_fd)^2
Fcr=	25.17	49.88	ksi	= if (Kstiff* Lstiff_fd/ rstiff_fd <= 4.71* Sqrt (29000/ Fy_stiff), 0.658^ (Fy_stiff/ Fc)* Fy_stiff,0.877* Astiff_fd)
Pn_stiff=	110.5	525.9	kips	= Kstiff* Lstiff_fd* rstiff_fd > 25, Fcr* Astiff_fd, Astiff_fd* Fy_stiff
Φc*Pn_stiff=	99.5	473.3	kips	= Φc* Pn_stiff
SUM_stiff_comp=	0.000	0.00		= if (SMF_Sides = 1, 0, Fsu/ (fc* Pn_stiff))
		OK		OK if SUM_Stiff_comp < SUM_Allowed

3.14 STIFFENER PLATE DOUBLE SIDE FILLET WELD TO COLUMN FLANGE:

Φfillet=	0.75		
Fexx=	70	ksi	
tstiff=	0.500	in	Value previously defined
wfillet_min=	0.2	in	Minimum fillet weld size per AISC 360 Table J2.4

	Left Side	Right Side		
wfillet_flange_min=	0.000	0.000	in	= (0.5* Fsu/ SMF_Sides)/ (ffillet* 1.5* 0.6* Fexx* (bstiff - Lclip_flange-0.5)*2^(1/2))
wfillet_flange=	0.000	0.188	in	
wfillet_flange_DCR=	0.000	0.000		= wfillet_flange_min/ wfillet_flang

3.15 STIFFENER PLATE DOUBLE SIDE FILLET WELD TO COLUMN WEB:

wfillet_min=	0.188	0.188	in	Minimum fillet weld size per AISC 360 Table J2.4
wfillet_min_web=	0.000	0.000	in	= 0.5* Fsu* SMF_Sides/ (ffillet* 0.6* Fexx* (Lstiff - Lclip_web-0.5)*2^(1/2))
wfillet_web=	0.000	0.188	in	=max(wfillet_min, ceiling(wfillet_web,1/16)
wfillet_web_DCR=	0.000	0.000		= wfillet_min_web/ wfillet_web

3.16 CHECK MINIMUM COLUMN FLANGE THICKNESS:

3.16.1 CONNECTION AWAY FROM COLUMN ENDS (SST Step 18 Table 1.1):

	Left Side	Right Side		
Φc=	0.9	0.9		
tcf=	0.96	0.96	in	Value previously defined
c=	NA	5.75	in	= bolt_g_flange
g=	NA	6	in	= bolt_s_flange
H_flange=	NA	9.25	in	Value previously defined
s=	NA	4.399	in	= 0.5* Sqrt (bcf* g)
pso=	NA	2.75	in	= (g - tstiff)/2
psi_tmp=	NA	2.75	in	= pso
psi=	NA	2.75	in	= if (psi_tmp > s, s, psi_tmp)
h1=	NA	21.85	in	= db + t_stem - g/2
h0=	NA	27.85	in	= db + t_stem + g/2
Yc_unstiffened=	0.00	199.01	in	= bcf/2* (h1/s + h0/s) + 2/g* (h1* (s + 3*c/4) + h0* (s +c/4) + c^2/4) +g/2
tcf_req_unstiffened=	0.000	0.90	in	= Sqrt (1.1* Mpr/ (fb* Fyc* Yc_unstiffened))
Yc_stiffened=	0.00	307.88	in	= bcf/2* (h1* (1/s + 1/psi) + h0* (1/s + 1/pso)) + 2/g* (h1* (s + psi) + h0* (s + pso))
tcf_req_stiffened=	0.000	0.72	in	= Sqrt (1.1* Mpr/ (fb* Fyc* Yc_stiffened))
Stiffened?=	YES	YES		Input value
tcf_min=	0.000	0.721	in	= if (Stiffened? = YES, tcf_req_stiffened, tcf_req_unstiffened)
SUM_colFLB1=	0.000	0.751		= tcf_min/ tcf
		OK		OK if SUM_colFLB1 < SUM_Allowed

3.16.2 CONNECTION AT STIFFENED COLUMN END (SST STEP 18, TABLE 1.2, CASE 1):

	Left Side	Right Side		
de=	1.25	1.25	in	centerline of top link bolt to top of column
pfi=	NA	2.75	in	= (g - tstiff)/ 2
pso=	NA	2.75	in	Value previously defined
Yp=	0.0	258.226	in	= bcf/2* (h1* (1/pfi + 1/pso) + h0* (1/pso + 1/2/s)) + 2/g* (h1* (pfi + s) + h0* (d2 + pso))
tcf_req=	0.000	0.787	in	= Sqrt* (1.1* Mpr/ (fb* Fy* Yp))
SUM_colFLB2=	0.000	0.820		= tcf_req/ tcf
		OK		OK, if SUM_colFLB2 < SUM_Allowed

3.17 STABILITY BRACING AT BEAM-TO-COLUMN CONNECTIONS (AISC 314-16 SECTION E3.4.C(B))

	Left Side	Right Side		
Link Size=	NA	YL6-4.5		
byield=	0.0	4.5	in	
tstem=	0.0	0.8	in	
Fy_link=	50	50	ksi	
Ry=	1.1	1.1		
Pye_link=	0.000	185.625	kips	
0.02 * Pye_llink=	0.000	3.713	kips	Bracing Force (LRFD)
Pbrace (ASD)=	0.000	2.599	kips	=0.7*Bracing Force (LRFD)

3.18 DESIGN SUMMARY:

DCR Summary:			Report:
SCWB_DCR_total=	0.219		0.219
DCR_PZ_total=	0.544	Overall PZ chk:	OK
DCR_PZ_total_NEW=	0.279	NO	0.544
DCR_stiff_comp_DCR=	0.000	0.000	NA
wfillet_flange_DCR=	0.000	0.000	NA
wfillet_web_DCR=DCR_colFLB1=	0.000	0.000	NA
DCR_colFLB2=	0.000	0.751	0.751
DCR_colFLB=	0.000	0.820	0.820
			0.751

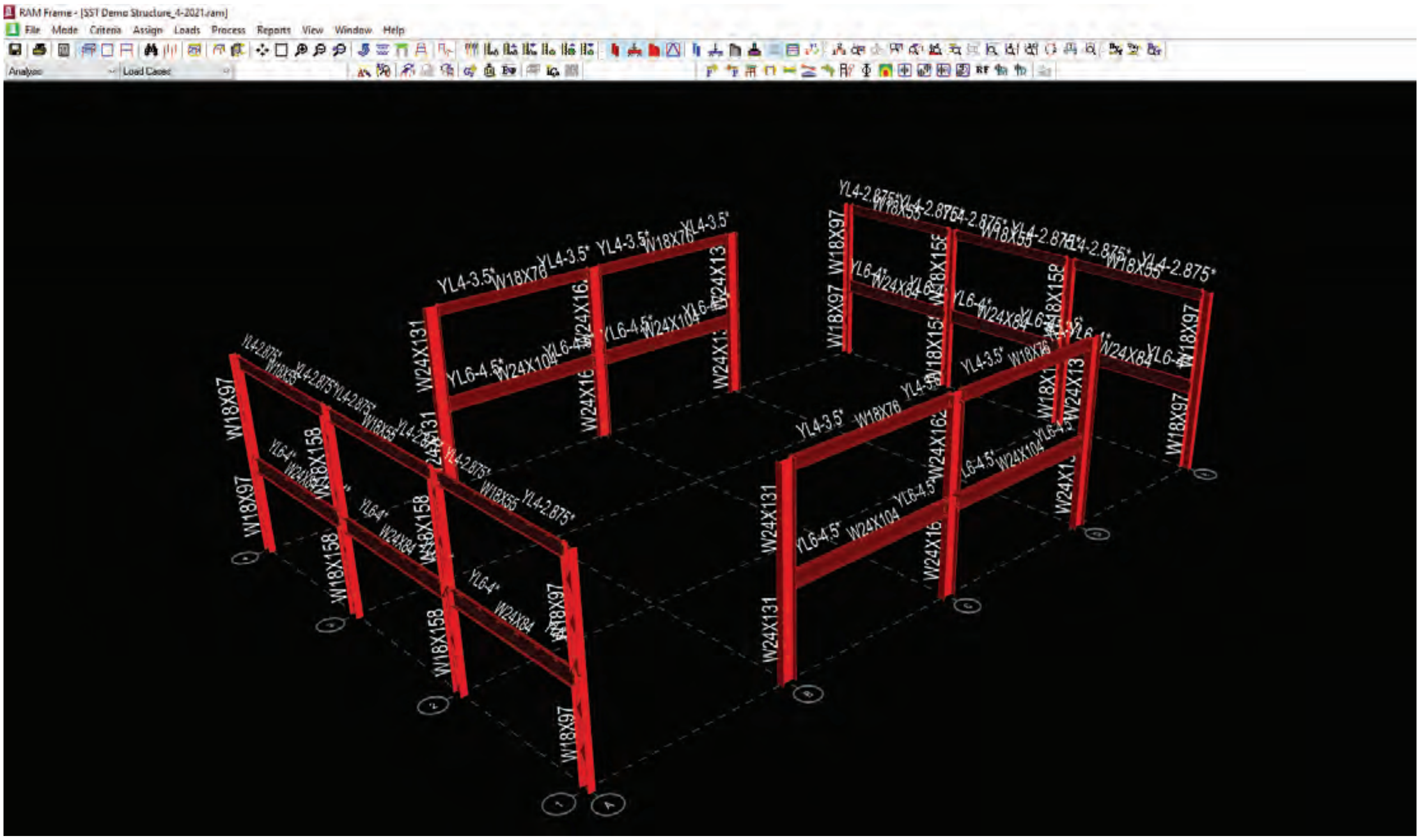
Geometry/Weld Summary:			Report:
DP used?	NO	PlugWeld?	NO
t_dblr_use=	0.625		0.000
Stiffener Req'd=	NO	NO	NO
t_stiff_Use=	0.000	0.500	0.500
wfillet_flange=	0.000	0.188	0.188
wfillet_web=	0.000	0.188	0.188

Appendix 7C

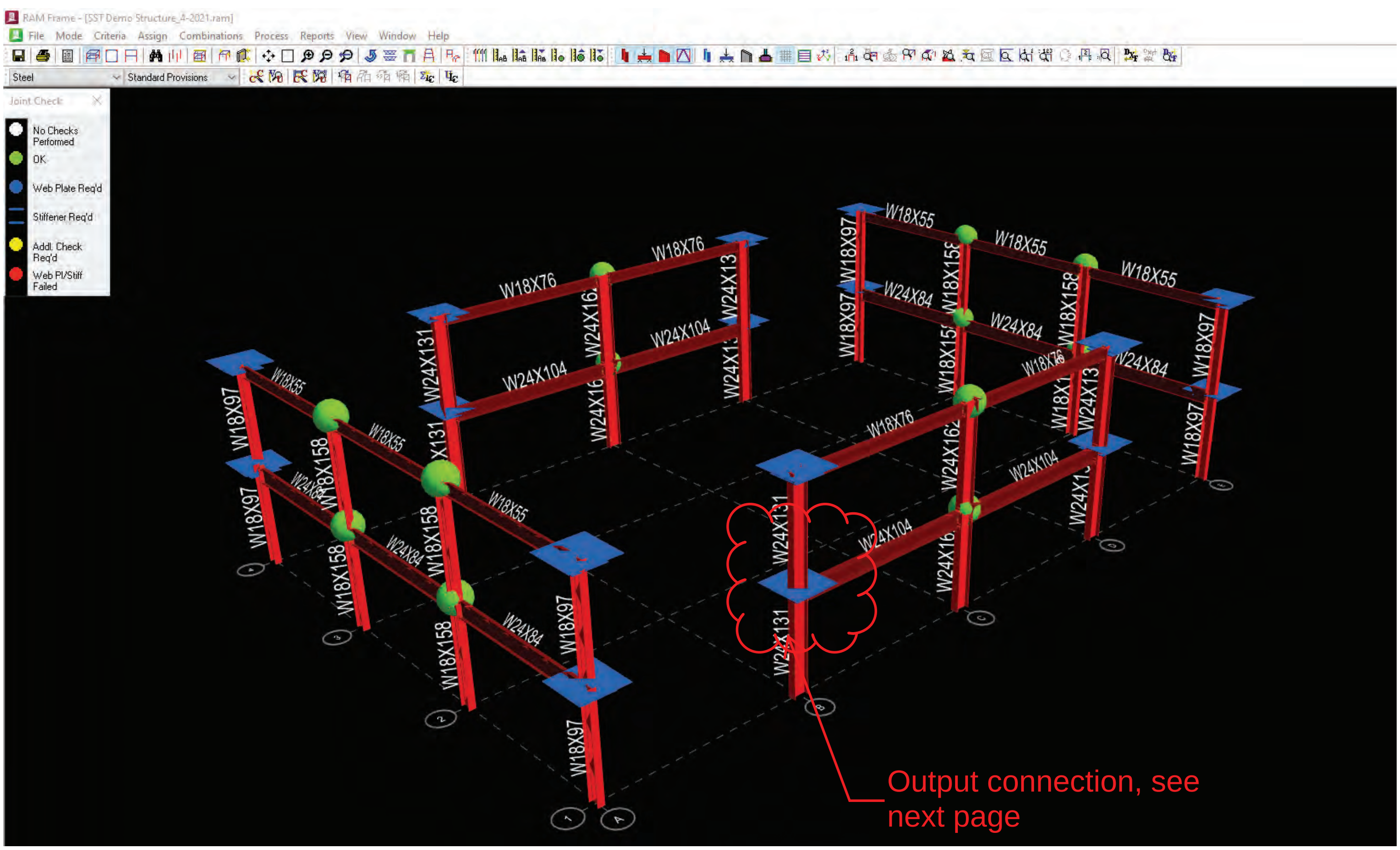
YLMC RAM Demo

Date: 4-22-2022

1. Column/Beam and Link Sizes



2. Yield-Link Strength Check (Standard Provision, LRDF combos)



Joint Code Check

RAM Structural System 17.03.00.285
DataBase: SST Demo Structure_4-2021
Building Code: IBC

04/22/22 15:42:03
Steel Code: AISC360-16 LRFD

Story Number: 1 **Joint Number:** 19

Final Design

Yield-Link Design Check - **OK**
No Web Plate Required

Top	SideA Stiffener LxWxT (in) :	11.417	x	6.102	x	0.591
Bot	SideA Stiffener LxWxT (in) :	11.417	x	6.102	x	0.591

Joint Data and Material Properties

Web Plate Nominal Yield (ksi) ---------- 50.00
Stiffener Nominal Yield (ksi) ----------- 50.00

	Size	Plan Angle	Elev Angle	Yield(ksi)
Col. At Jnt:	W24X131	0.00	---	50.00
Beam SideA :	W24X104	0.00	0.00	50.00

Criteria

Force on column flange is from beam moment, axial and shear forces.
Use actual beam moments to determine panel zone shear at the joint.
Optimize design of each stiffener at a joint

Yield-Link Design

Side	Size			Ratio
A	YL6-4.5:	Pu (kip) = 106.66	0.9 Pylink (kip) = 151.88	0.702

Stiffener Required Area

Side	Flange	Ast Reqd (two stiffeners) (in2)	
SideA	Top	0.09	OK
SideA	Bot	0.09	OK

Compression

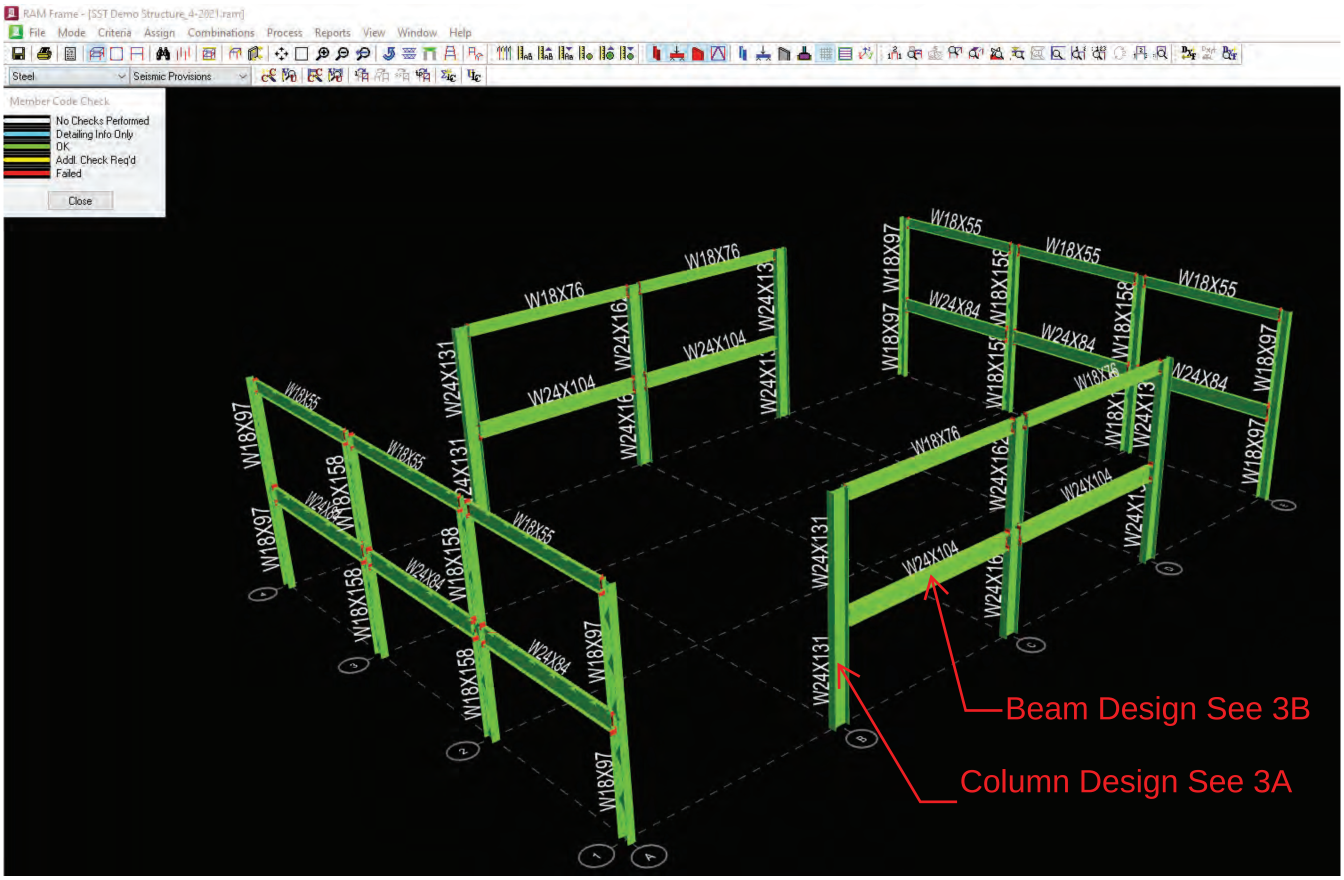
		Side A			Side B			
	Flange	Force (kip)	LCo	Cap. (kip)	Force (kip)	LCo	Cap. (kip)	Stiffen
Local Web Yld	Top	263.3	0	288.9	---	---	---	NO
	Bot	263.3	0	288.9	---	---	---	NO
Web Crippling	Top	263.3	0	379.0	---	---	---	NO
	Bot	263.3	0	379.0	---	---	---	NO

Tension

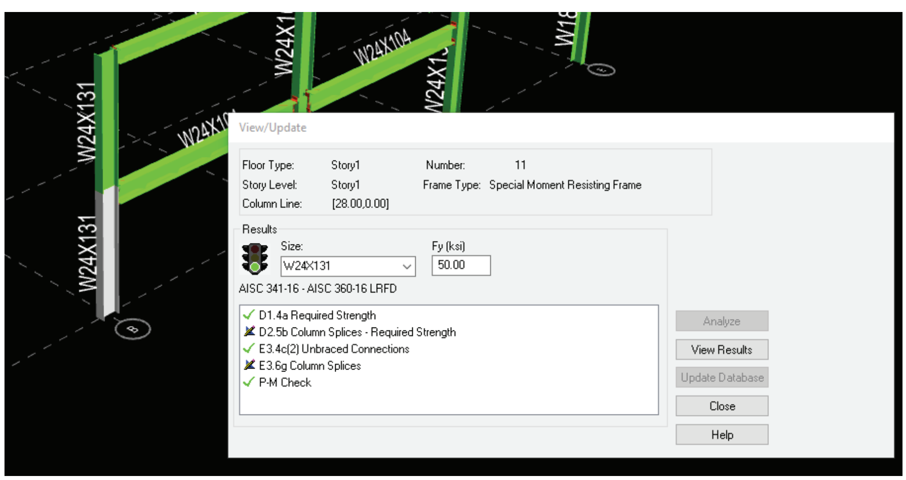
		Side A			Side B			
	Flange	Force (kip)	LCo	Cap. (kip)	Force (kip)	LCo	Cap. (kip)	Stiffen
Local Web Yld	Top	263.3	0	288.9	---	---	---	NO
	Bot	263.3	0	288.9	---	---	---	NO
Flange Bend.	Top	263.3	0	259.2	---	---	---	YES
	Bot	263.3	0	259.2	---	---	---	YES

Note: LCo numbers correspond to the Numbers on the Load Combination Printout

3. Yield-Link Beam And Column Strength Check (Seismic Provision, Overstrength/Omega combos)



3A. Yield-Link Column Strength Check (Seismic Provision, Overstrength/Omega combos)



3A. Yield-Link Column Strength Check (page 1/2)

Seismic Provisions Member Code Check

RAM Structural System 17.03.00.285
DataBase: SST Demo Structure_4-2021 04/22/22 15:45:56
Building Code: IBC Steel Code: AISC341-16 - LRFD

Column Parameters
Story: Story1 Frame No: 1 Member No: 11
Fy (ksi): 50.00 Size: W24X131
Frame Type: Special Moment Resisting Frame

D1.4a Required Strength --- OK
Compression: Max Pu (kip) = 132.72 --- Combination: 1.400 D - 1.400 ND1 + 0.500 Lp - 0.500 NL1 - 3.000 E6
Max Pu/φPn = 0.10 OK
Tension: Max Pu (kip) = 32.70 --- Combination: 0.700 D + 0.700 ND1 + 3.000 E6
Max Pu/φPn = 0.02 OK

D2.5b Column Splices - Required Strength
Design strength of column splices must meet or exceed the following forces:
Required tension and compression strength from D1.4a.
Shear in major axis (kip)= 51.35 --- Combination: 1.400 D - 1.400 ND1 + 0.500 Lp - 0.500 NL1 - 3.000 E2
Shear in minor axis (kip)= 5.73 --- Combination: 1.400 D - 1.400 ND2 + 0.500 Lp - 0.500 NL2 - 3.000 E4
Moment in major axis (kip-ft)= 667.28 --- Combination: 1.400 D - 1.400 ND1 + 0.500 Lp - 0.500 NL1 - 3.000 E2
Moment in minor axis (kip-ft)= 80.26 --- Combination: 1.400 D - 1.400 ND2 + 0.500 Lp - 0.500 NL2 - 3.000 E4
Required shear for column splice is max result from D2.5b and D2.5c
Refer to AISC 341 section D2.5b for additional detailing requirements.

See Additional AISC 358 Requirements for Assigned Connection Type.

E3.4c(1) Unbraced Connections --- OK
Joint Above Column is Restrained
Joint Below Column is Restrained

E3.5a Basic Requirements (D1.1 Moderately Ductile) --- OK
Columns is pinned at base and is exempt from ductility requirements per Section D1.1

E3.6g Column Splices (D2.5c Required Shear Strength)
Column is at the lowest story. No column splice required

INPUT DESIGN PARAMETERS:

	X-Axis	Y-Axis
Lu for Axial (ft)	14.00	14.00
Lu for Bending (ft)	14.00	14.00
K	1.00	1.00

CONTROLLING COLUMN FORCES - SHEAR
Load Combination: 1.400 D - 1.400 ND1 + 0.500 Lp - 0.500 NL1 - 3.000 E2

Shear	Top	Vux (kip)	-51.35
		Vuy (kip)	0.84
Shear	Bot.	Vux (kip)	-51.35
		Vuy (kip)	0.84

Seismic Provisions Member Code Check

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DataBase: SST Demo Structure_4-2021 04/22/22 15:45:56
Building Code: IBC Steel Code: AISC341-16 - LRFD

SHEAR CHECK:

Vux (kip)	=	-51.35	1.00Vnx (kip)	=	444.68	Vux/1.00Vnx	=	0.115
Vuy (kip)	=	0.84	0.90Vny (kip)	=	668.74	Vuy/0.90Vny	=	0.001

CONTROLLING COLUMN FORCES - AXIAL

Load Combination: 1.400 D - 1.400 ND1 + 0.500 Lp - 0.500 NL1 - 3.000 E6

AXIAL CHECK:

Pu (kip)	=	132.72	0.90Pnx (kip)	=	1702.92	Pu/0.90Pnx	=	0.078
			0.90Pny (kip)	=	1374.20	Pu/0.90Pny	=	0.097
			0.90Pn (kip)	=	1374.20	Pu/0.90Pn	=	0.097

CONTROLLING COLUMN FORCES - FLEXURE

Load Combination: 1.400 D - 1.400 ND1 + 0.500 Lp - 0.500 NL1 - 3.000 E2

Axial		Load (kip) ------------------	131.60
Moment	Top	Mux (kip-ft) ----------------	667.28
		Muy (kip-ft)----------------	-11.81
Moment	Bot.	Mux (kip-ft) ----------------	0.00
		Muy (kip-ft)----------------	0.00

CALCULATED PARAMETERS:

Pu (kip)	=	131.60	0.90Pnx (kip)	=	1702.92
			0.90Pny (kip)	=	1374.20
Mux (kip-ft)	=	667.28	0.90Mnx (kip-ft)	=	1387.50
Muy(kip-ft)	=	-11.81	0.90Mny (kip-ft)	=	305.63
			Mcx (kip-ft)	=	1301.21
KL/Rx	=	16.46	KL/Ry	=	56.61
Cbx	=	1.67			

INTERACTION EQUATION:

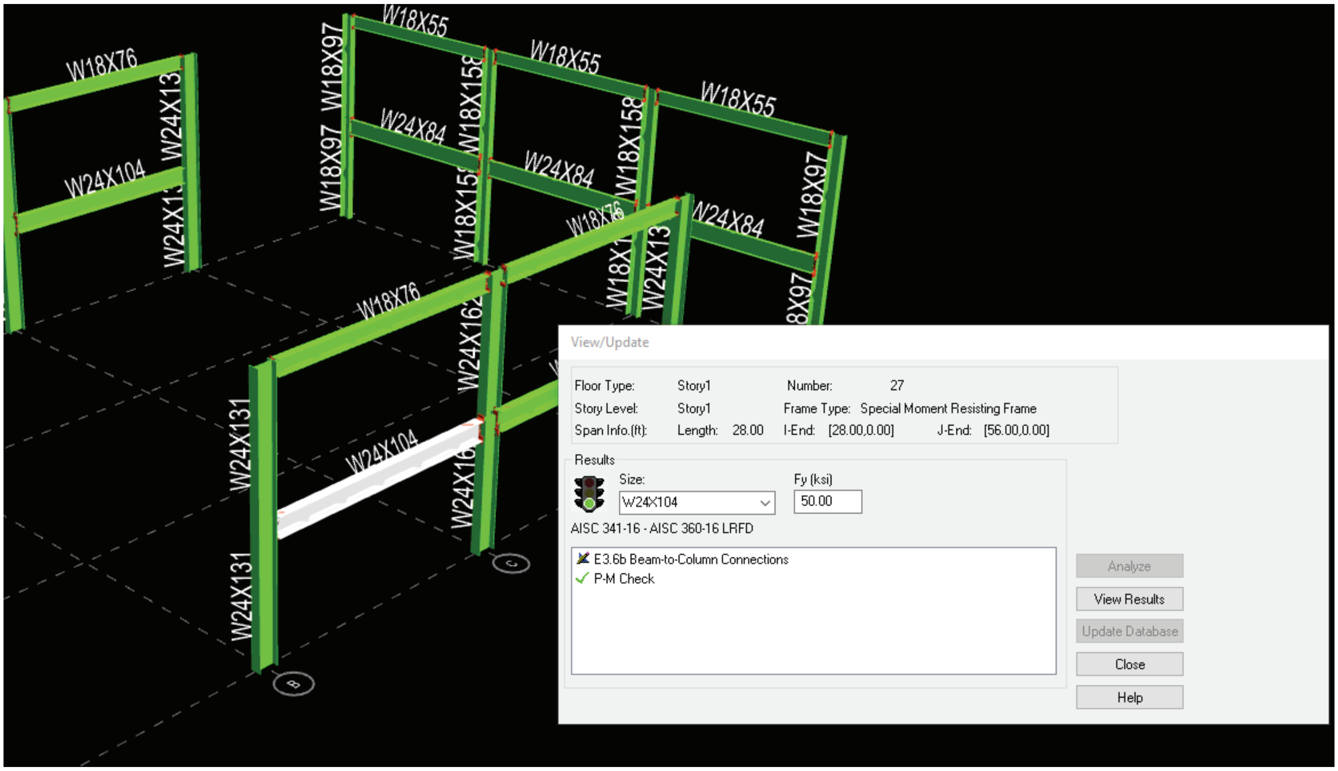
Pu/0.90*Pn=0.077

~~Eq H1-3: 0.139 + 0.095 = 0.234~~

$\boxed{\text{Eq H1-1b Per H1.3: } 0.039 + 0.481 + 0.000 = 0.520}$

Verify Yield-Link Connections with Simpson Strong-Tie prior to finalizing structural design.

3B. Yield-Link Beam Strength Check (Seismic Provision, Overstrength/Omega combos)



3B. Yield-Link Beam Strength Check (page 1/2)

Seismic Provisions Member Code Check

RAM Structural System 17.03.00.285
DataBase: SST Demo Structure_4-2021 — 04/22/22 15:45:56
Building Code: IBC — Steel Code: AISC341-16 - LRFD

Beam Parameters

Story: Story1 — Frame No: 1 — Member No: 27
Fy (ksi): 50.00 — Size: W24X104
Frame Type: Special Moment Resisting Frame
Left Connection - YL6-4.5
Right Connnection - YL6-4.5

E3.6b Beam-to-Column Connection Requirements

Required flexural strength of connection (kip-ft) = 0.8Mp = 436.12 at story drift angle in (1)

See Additional AISC 358 Requirements for Assigned Connection Type.

INPUT DESIGN PARAMETERS:

	X-Axis	Y-Axis
Lu for Axial (ft)	5.00	5.00
Lu for Bending (ft)	5.00	5.00
K	1.00	1.00
Top Flange Continuously Braced	No	
Bottom Flange Continuously Braced	No	

CONTROLLING BEAM SEGMENT FORCES - SHEAR

Load Combination: 1.400 D + 1.400 ND1 + 0.500 Lp + 0.500 NL1 + 3.000 E6

Segment distance (ft) i - end: 0.00
j - end: 28.00

SHEAR CHECK:

Vux (kip)	=	-52.79	1.00Vnx (kip)	=	361.50	Vux/1.00Vnx	=	0.146
Vuy (kip)	=	-0.21	0.90Vny (kip)	=	518.40	Vuy/0.90Vny	=	0.000

CONTROLLING BEAM SEGMENT FORCES - AXIAL

Load Combination: 0.700 D + 0.700 ND2 + 3.000 E3

Segment distance (ft) i - end: 0.00
j - end: 28.00

AXIAL CHECK:

Pu (kip)	=	8.94	0.90Pnx (kip)	=	1317.23	Pu/0.90Pnx	=	0.007
			0.90Pny (kip)	=	1284.56	Pu/0.90Pny	=	0.007
			0.90Pn (kip)	=	1284.56	Pu/0.90Pn	=	0.007

CONTROLLING BEAM SEGMENT FORCES - FLEXURE

Load Combination: 1.400 D + 1.400 ND1 + 0.500 Lp + 0.500 NL1 + 3.000 E1

Segment distance (ft) i - end: 0.00
j - end: 28.00

CALCULATED PARAMETERS:

Pu (kip)	=	-9.33	0.90Pn (kip)	=	1284.56
Mux (kip-ft)	=	-545.15	0.90Mnx (kip-ft)	=	1083.75
Muy(kip-ft)	=	0.45	0.90Mny (kip-ft)	=	234.00
L/r	=	20.66			
Cbx	=	2.12			

3B. Yield-Link Beam Strength Check (page 2/2)

Seismic Provisions Member Code Check

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Building Code: IBC	Steel Code: AISC341-16 - LRFD

INTERACTION EQUATION:

Pu/0.90*Pn=0.007

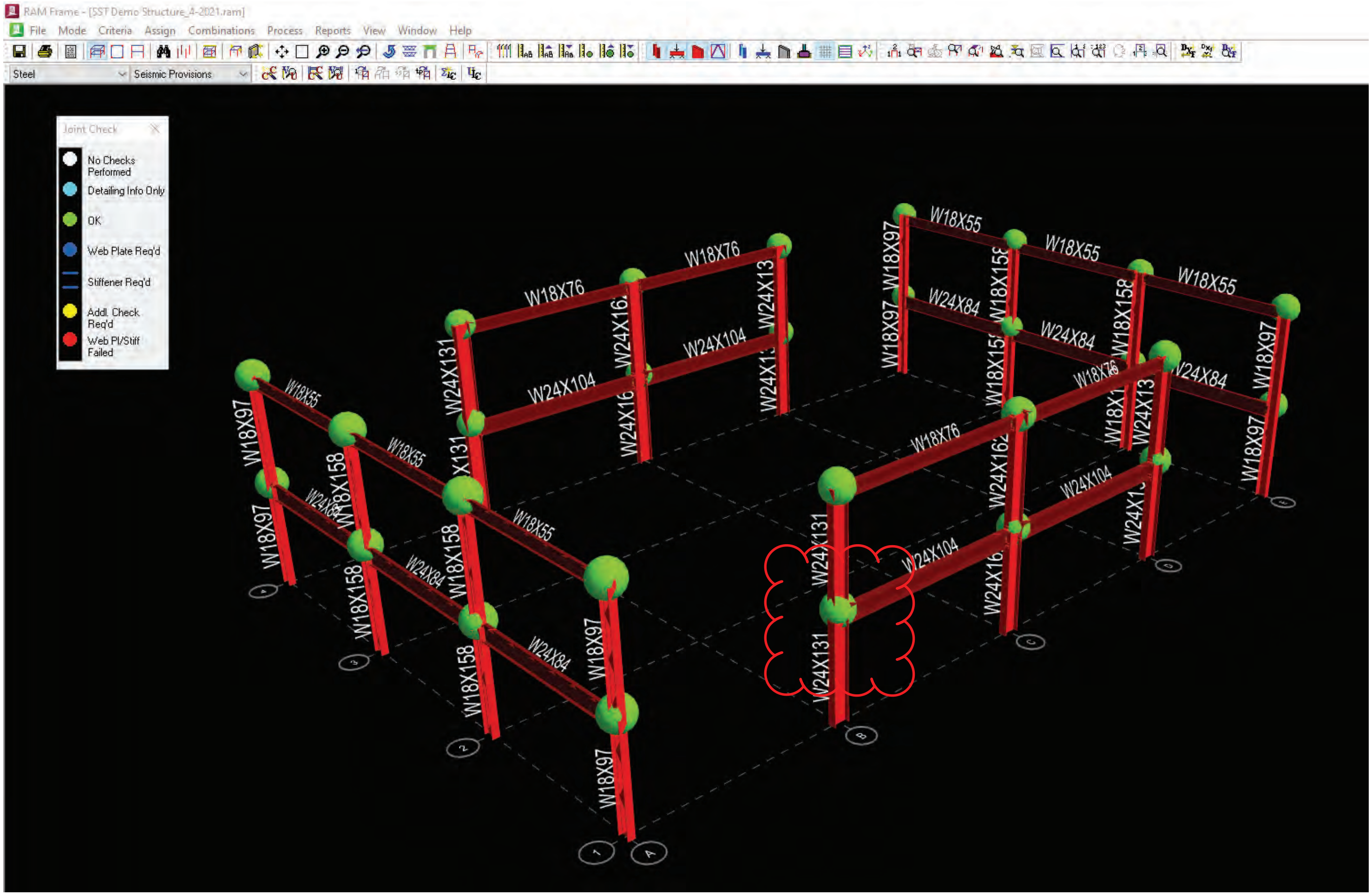
Eq H1-1b: 0.003 + 0.503 + 0.002 = 0.508

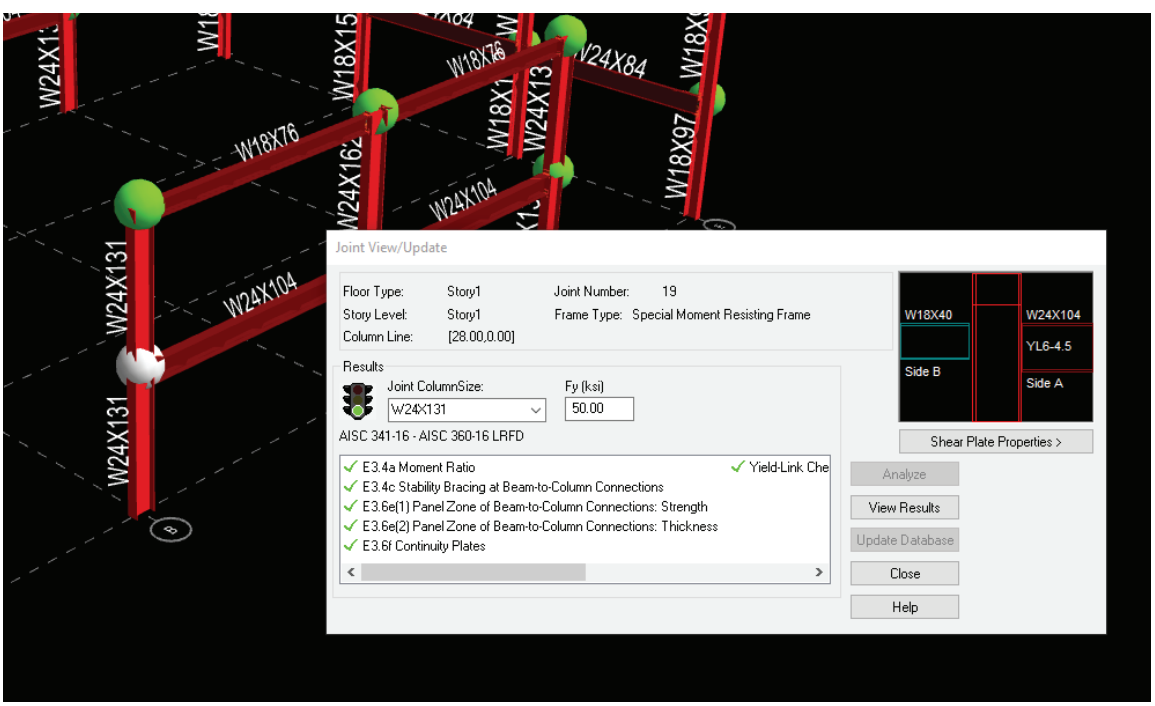
Mpr (kip-ft) = 545.15 0.90Mnx (kip-ft) = 1083.75 Ratio = 0.503

Verify Yield-Link Connections with Simpson Strong-Tie prior to finalizing structural design.

Check against Mpr to make sure beam is stronger than connection

3C. Yield-Link Connection Design/Check (Seismic Provisions)





Seismic Provisions Joint Code Check

RAM Structural System 17.03.00.285
DataBase: SST Demo Structure_4-2021 — 04/22/22 16:02:53
Building Code: IBC — Steel Code: AISC341-16 - LRFD

Joint Parameters

Story: Story1 — Frame No: 1 — Joint No: 19
Fy (ksi): 50.00 — Column Size: W24X131
Joint Frame Type: Special Moment Resisting Frame

E3.4a Moment Ratio --- OK

Col.	Size	M*pc (kip-ft)	Zc (in³)	Pu (kip)	Combo#
Top	W24X131	1638.20	370.00	28.69	14
Bot	W24X131	1548.57	370.00	132.72	14

Bm.	Size	M*pb (kip-ft)	Mpr (kip-ft)	Muv (kip-ft)	Sh+dc/2 (in)
27	W24X104	614.69	545.15	69.54	15.75

SCWB Check

ΣM*pc = ΣZc(Fy-Puc/Ag) (kip-ft) = 3186.77
ΣM*pb = ΣPr-link(d+tstem) + Muv (kip-ft) = 614.69
ΣM*pc/ Σ M*pb (Eqn E3-1) = 5.18 > 1.0 OK

E3.4c Stability Bracing at Beam-to-Column Connections --- OK

Joint is restrained in the minor axis at both the top and bottom flange of the moment-frame beams.

E3.6e(1) Panel-Zone of Beam-to-Column Connections: Strength --- OK

Bm.	Size	Bf Red. (in)	Z (in³)	Z hng (in³)	Mpr (kip-ft)
27	W24X104	0.000	289.00	289.00	545.15

Bm.	Mpr (kip-ft)	Sh (in)	V hng (kip)	Mf (kip-ft)	Vpz (kip)
27	545.15	3.50	52.987	545.15	263.25

Panel Zone Check

Sh = Distance face-of-column to location of hinge.
Vhng = Shear at hinge or col face from applicable load combinations with Ecl
Mpr = Pr-Link x (d+tstem). Zhng = Z at hinge.

Required Panel Zone Shear Strength (kip) = 224.31
Panel Zone Shear Reduced by Vc (kip) = 38.94
Axial Comp. for Panel Zone Calc. (kip) = 132.72 - Combination 1.400 D - 1.400 ND1 + 0.500 Lp - 0.500 NL1 - 3.000 E6
Panel Zone Strength Without Web Plate (kip) = 400.21 OK
Column web thickness reqd (in) = 0.502 Thickness provided = 0.605 OK

See Additional AISC 358 Requirements for Assigned Connection Type.

Yield-Link Connection Design - OK

Shear Plate Bolt Size - OK

Side A: A325-X Bolt Diameter (in) = 0.875 No. Horiz. Bolts = 2 No. Vert. Bolts = 5

Pu (kip) = 2.94	Beam Vu (kip) = 52.99		
Bolt Vu (kip) = 10.70	ϕRn-bolt (kip) = 40.59	Ratio = 0.264	

Shear Plate Check

Shear Plate Geometry - OK

Side A	Wsp (in) = 8.000	hsp (in) = 13.250	tsp (in) = 0.500
	Svert (in) = 2.750	Shoriz (in) = 2.750	Horiz Lslot (in) = 1.813

Shear Plate Yielding (Vertical) - OK

Seismic Provisions Joint Code Check

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Side A Vu (kip) = 52.99 ϕVy (kip) = 198.75 Ratio = 0.267

Shear Plate Rupture (Vertical) - OK
Side A Vu (kip) = 52.99 ϕVr (kip) = 120.66 Ratio = 0.439

Shear Plate Axial and Moment - OK

Side A	Vuy (kip) = 52.99	Mecc (kip-ft) = 15.45				
	L-Whitmore (in) = 8.092	A-Whit. (in2) = 4.046	θ-Whit. (Deg) = 30.000			
	fmax (ksi) = 12.68	ϕb.Fy (ksi) = 45.00	Ratio = 0.282			

Shear Plate To Column Flange Fillet Weld - OK
Side A Thickness (in) = 0.313 Length (in) = 12.750
Vu (kip) = 52.99 ϕRn (kip) = 177.47 Ratio = 0.299

Shear plate weld check

Beam Web and Shear Tab Bearing: OK

Side A	Horizontal React. Pu (kip)	ϕRn (kip)	DCR	Vertical React. Vu (kip)	ϕRn (kip)	DCR	Combined React. θ (Deg)	Pr (kip)	ϕRn (kip)	DCR
Beam Web	2.94	90.49	0.03	52.99	255.94	0.21	82.10	10.70	102.38	0.105
Shear Plate	2.94	90.49	0.03	52.99	231.26	0.23	82.10	10.70	51.19	0.054

Beam Web Check

Beam Web and Shear Tab Block Shear: OK

Side A	Horizontal React. Pu (kip)	ϕRn (kip)	DCR	Vertical React. Vu (kip)	ϕRn (kip)	DCR	Combined React. θ (Deg)	Pr (kip)	ϕRn (kip)	DCR
Beam Web	2.94	101.25	0.03	52.99	210.36	0.25	82.10	53.07	232.38	0.228
Shear Plate	2.94	201.09	0.02	52.99	215.57	0.25	82.10	31.93	133.45	0.440

Buckling Restraint Plate Thickness - OK
Side A tbrp-min (in) = 0.774 tbrp (in) = 1.000 Ratio = 0.774

BRP check

Beam Flange Thickness - OK
Side A Pe (in) = 10.018 tf-min (in) = 0.477 tf (in) = 0.750 Ratio = 0.636

Beam Flange check

Buckling Restraint Plate Bolt Size and Quantity - OK
Side A Vux-bolt (kip) = 4.04 Vuy-bolt (kip) = 9.89 # Bolts per side = 2
RnVx(T+V) (kip) = 25.29 RnVy(T+V) (kip) = 29.82 Ratio = 0.332

Beam Edge - OK
Side A bf (in) = 12.800 bf-min (in) = 9.250 Ratio = 0.723

Beam net section check

Beam Net Section - OK
Side A Mpb-net(kip-ft) = 1113.89 Mpr (kip-ft) = 545.15 Ratio = 0.489

Column Flange Width - OK
Side A bcf (in) = 12.900 bcf-min (in) = 9.250 Ratio = 0.717

Col Flange Geometry Check

Seismic Provisions Joint Code Check

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Column flange checked for smaller edge distance.

Side A	Dflange (in)	=	1.250	Lcol-edge (in)	=	3.450	Ratio =	0.362

Column Connection

Side A (Top)	Yp (in)	=	315.20	tcf (in)	=	0.960	tcf-reqd (in)	=	0.820	Ratio =	0.854
Side A (Bot)	Yc (in)	=	315.20	tcf (in)	=	0.960	tcf-min (in)	=	0.712	Ratio =	0.742

Col Flange Prying Check

Verify Yield-Link Connections with Simpson Strong-Tie prior to finalizing structural design.