



Structural Calculations for:

Zimmer Residence

4661 Forest Ave SE, Mercer Island, WA 98040

Client: Ripple Design Studio

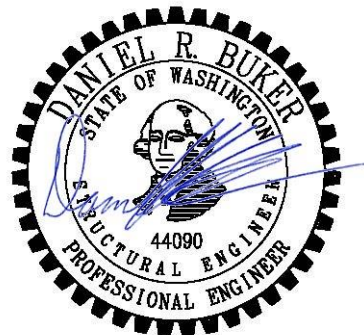
Code: 2021 International Building Code

Table of Contents

- C1 – Design Criteria
- L1 – L109 – Lateral Calculations
- F1 – F21 – Framing Calculations
- S1 – S6 – Shoring Calculations

Scope: Structural Design of Single-Family Residence

November 11th, 2024



Seismic Design Loads (ASCE 7-16)

for a Wood Framed Structure

RISK CATEGORY II

OCCUPANCY CAT. II Table 1.5-1

IMP. FACTOR 1 Table 1.5-2

SITE CLASS D Table 20.3-1

R = 6.5 Table 12.2-1

SEISMIC

DESIGN CATEGORY D 11.6

$$S_s = 1.441$$

$$S_1 = 0.5$$

$$F_a = 1.00 \quad \text{Table 11.4-1}$$

$$F_v = 1.80 \quad \text{Table 11.4-2}$$

$$S_{DS} = 0.961$$

$$S_{D1} = 0.600$$

$$T_0 = 0.12$$

$$T_s = 0.62$$

$$T_L = 6 \quad \text{Fig 22-14}$$

$$T = 0.21$$

$$\text{Seismic Dead Load: } 15^{\text{psf}} \text{ Roof}$$

$$15^{\text{psf}} \text{ Floor}$$

$$20^{\text{psf}} \text{ Walls}$$

$$C_{S_{ULT}} = 0.148 \quad \text{Eqn. 12.8-2}$$

$$C_{S_{ASD}} = 0.106$$

$$W_{\text{roof}} = 15 + 10 = 25^{\text{psf}}$$

$$W_{\text{floor}} = 10 + 10 + 10 = 30^{\text{psf}}$$

Vertical Design Loads

Criteria

ASCE 7-16

IBC 2018

Dead Loads

Roof (Composit) 2.5 psf

1/2" Ply 1.5 psf

Rafter/Truss 2 psf

Insulation 1 psf

5/8" GWB 3.1 psf

Roof PV 5 psf

Misc./Mech. 2 psf

17.1 psf

Use 20 psf

Flooring 1 psf

Sheathing 2.3 psf

Joist 2.6 psf

5/8" GWB 3.1 psf

Misc. Mech 1 psf

10 psf

Use 15 psf

Live Loads

Snow 25 psf

Floor 40 psf

Soil Bearing

1500 psf



Project: Zimmer Residence
4661 Forest Ave SE
Yarrow Point, WA 980

Date: 9/12/2024
Design: CRD

Wind Design Loads (ASCE 7-16)

Directional Procedure - Part 1

Exposure **C**
V= 100 mph
K_d= 0.85
I= 1
G= 0.85
K_e= 1.00

Table 26.6-1

26.11.1

Table 26.9-1

Roof Angle = 18.43 degrees
Ground to top of roof 31.17 ft
Bottom of roof to top of roof 9 ft
(mean roof height) h= 26.7 ft

Pressure Coefficients
from Figure 27.4-1:

Bldg Face	C _p
Windward Wall	0.8
Leeward Wall	-0.5
Windward Roof	0.3
Leeward Roof	-0.6

*Note= C_p values are conservative
worst case values

K_{zt}= 1.00

Pressures:	Calculated using ASCE7-16 Ch. 27 (Directional Procedure)					
Ht	K _z	q _z	P _{ww walls}	P _{lw walls}	Ultimate P _{walls} (psf)	Allowable P _{walls} (psf)
0-15	0.85	18.50	12.58	9.06	21.64	12.98
15-20	0.9	19.58	13.32	9.06	22.38	13.43
20-25	0.94	20.45	13.91	9.06	22.97	13.78
25-30	0.98	21.32	14.50	9.06	23.56	14.14
30-40	1.04	22.63	15.39	9.06	24.45	14.67

P _{ww roof}	P _{lw roof}	P _{roof} (psf)	P _{roof} (psf)
5.44	10.88	16.31	9.79

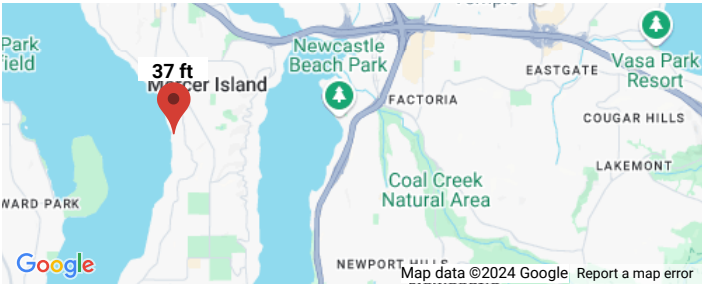


Project: Zimmer Residence
4661 Forest Ave SE
Yarrow Point, WA 98004

Date: 9/12/2024
Design: CRD

Search Information

Address:	4661 Forest Ave SE, Mercer Island, WA 98040, USA
Coordinates:	47.5619802, -122.230759
Elevation:	37 ft
Timestamp:	2024-09-12T18:40:42.801Z
Hazard Type:	Seismic
Reference Document:	ASCE7-16
Risk Category:	II
Site Class:	D



Basic Parameters

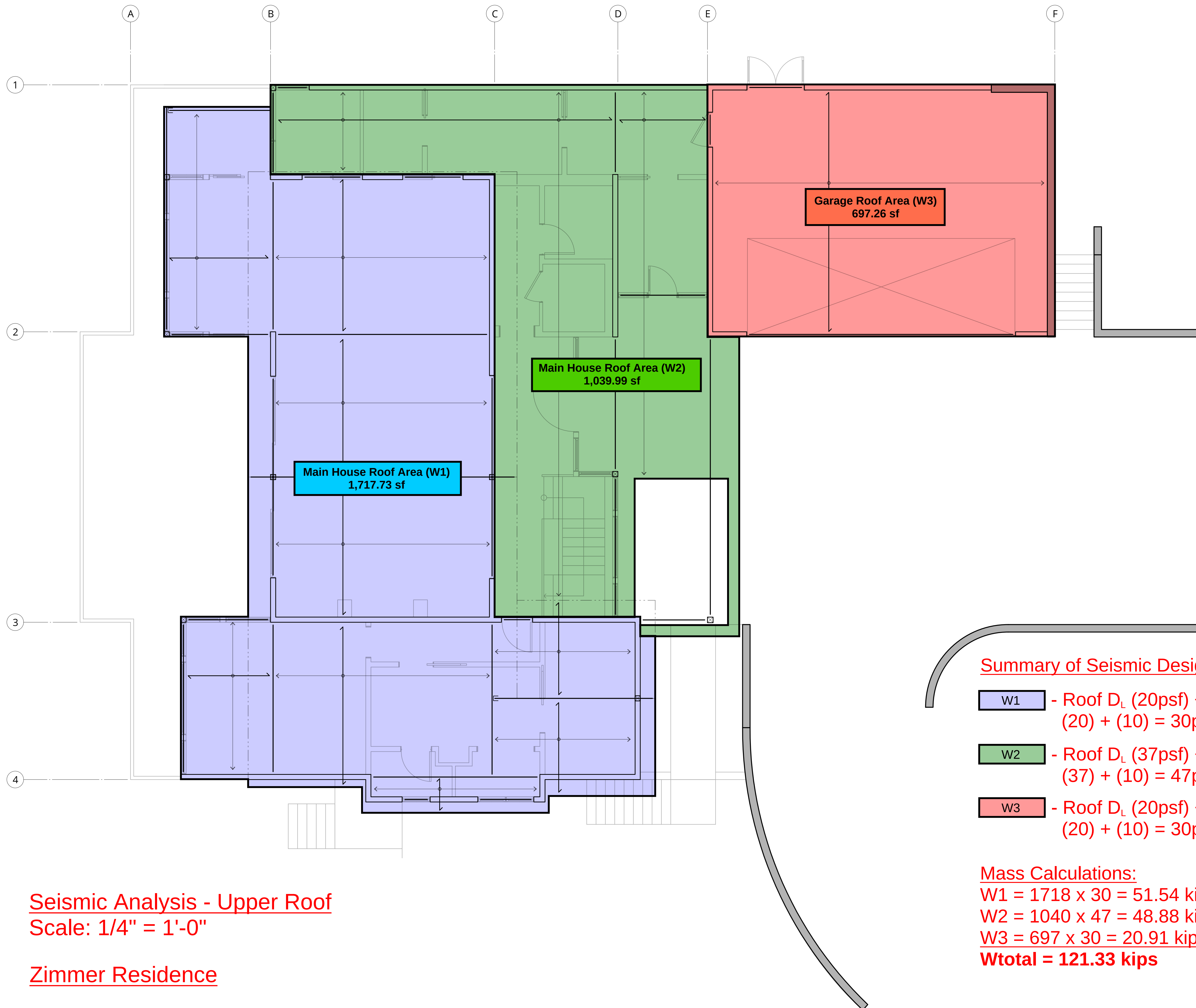
Name	Value	Description
S _S	1.441	MCE _R ground motion (period=0.2s)
S ₁	0.5	MCE _R ground motion (period=1.0s)
S _{MS}	1.441	Site-modified spectral acceleration value
S _{M1}	* null	Site-modified spectral acceleration value
S _{DS}	0.96	Numeric seismic design value at 0.2s SA
S _{D1}	* null	Numeric seismic design value at 1.0s SA

* See Section 11.4.8

▼Additional Information

Name	Value	Description
SDC	* null	Seismic design category
F _a	1	Site amplification factor at 0.2s
F _v	* null	Site amplification factor at 1.0s
CR _S	0.902	Coefficient of risk (0.2s)
CR ₁	0.898	Coefficient of risk (1.0s)
PGA	0.617	MCE _G peak ground acceleration
F _{PGA}	1.1	Site amplification factor at PGA
PGA _M	0.679	Site modified peak ground acceleration
T _L	6	Long-period transition period (s)
SsRT	1.441	Probabilistic risk-targeted ground motion (0.2s)
SsUH	1.597	Factored uniform-hazard spectral acceleration (2% probability of exceedance in 50 years)
SsD	3.919	Factored deterministic acceleration value (0.2s)
S1RT	0.5	Probabilistic risk-targeted ground motion (1.0s)
S1UH	0.557	Factored uniform-hazard spectral acceleration (2% probability of exceedance in 50 years)
S1D	1.54	Factored deterministic acceleration value (1.0s)
PGAd	1.322	Factored deterministic acceleration value (PGA)

* See Section 11.4.8



Seismic Analysis - Upper Roof
Scale: 1/4" = 1'-0"

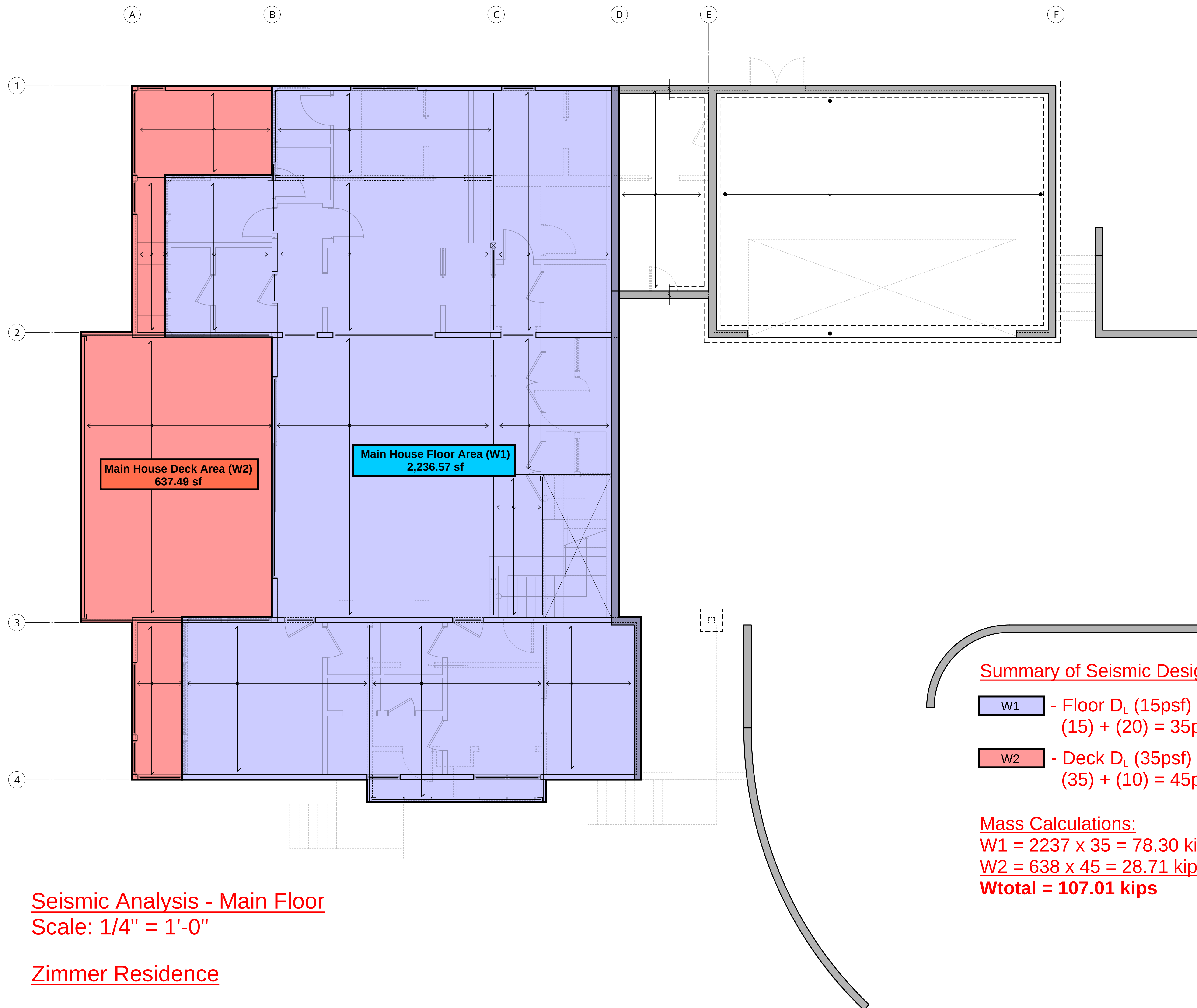
Zimmer Residence

Summary of Seismic Design Loads:

- W1** - Roof D_L (20psf) + Wall D_L (10psf)
(20) + (10) = 30psf
- W2** - Roof D_L (37psf) + Wall D_L (10psf)
(37) + (10) = 47psf
- W3** - Roof D_L (20psf) + Wall D_L (10psf)
(20) + (10) = 30psf

Mass Calculations:

W1 = 1718 x 30 = 51.54 kips
W2 = 1040 x 47 = 48.88 kips
W3 = 697 x 30 = 20.91 kips
Wtotal = 121.33 kips



Seismic Analysis - Main Floor
 Scale: 1/4" = 1'-0"
 Zimmer Residence

Summary of Seismic Design Loads:

- W1 - Floor D_L (15psf) + Wall D_L (20psf)
 (15) + (20) = 35psf
- W2 - Deck D_L (35psf) + Wall D_L (10psf)
 (35) + (10) = 45psf

Mass Calculations:

$W1 = 2237 \times 35 = 78.30 \text{ kips}$
 $W2 = 638 \times 45 = 28.71 \text{ kips}$
 $W_{total} = 107.01 \text{ kips}$

Seismic Analysis (ASCE 7-16)

Base Shear for House

$S_s =$	1.441	$F_a =$	1.00	$S_{ms} =$	1.44	$S_{Ds} =$	0.96
$S_1 =$	0.50	$F_v =$	1.80	$S_{m1} =$	0.9	$S_{D1} =$	0.60

Site Class =	D	
Mean Roof Height =	26.5	ft
T =	0.23	sec
R =	6.5	
I =	1.0	
rho =	1.0	
Cs =	0.148	
W =	207.43	K
Allowable Base Shear V =	21.47	K

House

Level	Wx (K)	hx (ft)	Wxhx	Cvx	Fx (K)
Roof	100.42	22.25	2234	0.68	14.5
Second	107.01	10	1070	0.32	7.0
Sum	207.43		3304	1.0	21.5

Seismic Analysis (ASCE 7-16)

Base Shear for Garage

$S_s =$	1.441	$F_a =$	1.00	$S_{ms} =$	1.44	$S_{Ds} =$	0.96
$S_1 =$	0.50	$F_v =$	1.80	$S_{m1} =$	0.9	$S_{D1} =$	0.60

Site Class =	D	
Mean Roof Height =	13.5	ft
T =	0.14	sec
R =	6.5	
I =	1.0	
rho =	1.0	
Cs =	0.148	
W =	20.91	K
Allowable Base Shear V =	2.16	K

House

Level	Wx (K)	hx (ft)	Wxhx	Cvx	Fx (K)
Roof	20.91	13.5	282	1.00	2.2
Sum	20.91		282	1.0	2.2

ATC

Hazards by Location

Search Information

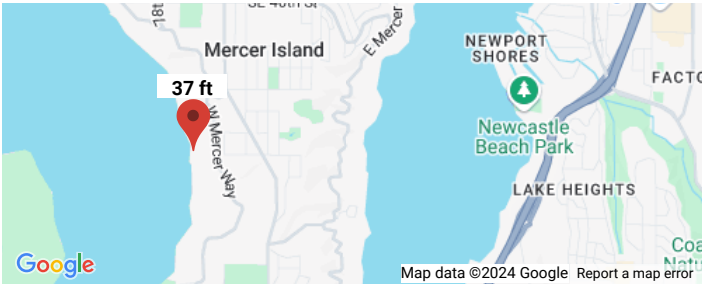
Address: 4661 Forest Ave SE, Mercer Island, WA 98040, USA

Coordinates: 47.5619802, -122.230759

Elevation: 37 ft

Timestamp: 2024-09-19T16:09:55.193Z

Hazard Type: Wind



ASCE 7-16

MRI 10-Year 67 mph

MRI 25-Year 73 mph

MRI 50-Year 78 mph

MRI 100-Year 83 mph

Risk Category I 92 mph

Risk Category II 97 mph

Risk Category III 104 mph

Risk Category IV 108 mph

ASCE 7-10

MRI 10-Year 72 mph

MRI 25-Year 79 mph

MRI 50-Year 85 mph

MRI 100-Year 91 mph

Risk Category I 100 mph

Risk Category II 110 mph

Risk Category III-IV 115 mph

ASCE 7-05

ASCE 7-05 Wind Speed 85 mph

The results indicated here DO NOT reflect any state or local amendments to the values or any delineation lines made during the building code adoption process. Users should confirm any output obtained from this tool with the local Authority Having Jurisdiction before proceeding with design.

Please note that the ATC Hazards by Location website will not be updated to support ASCE 7-22. [Find out why.](#)

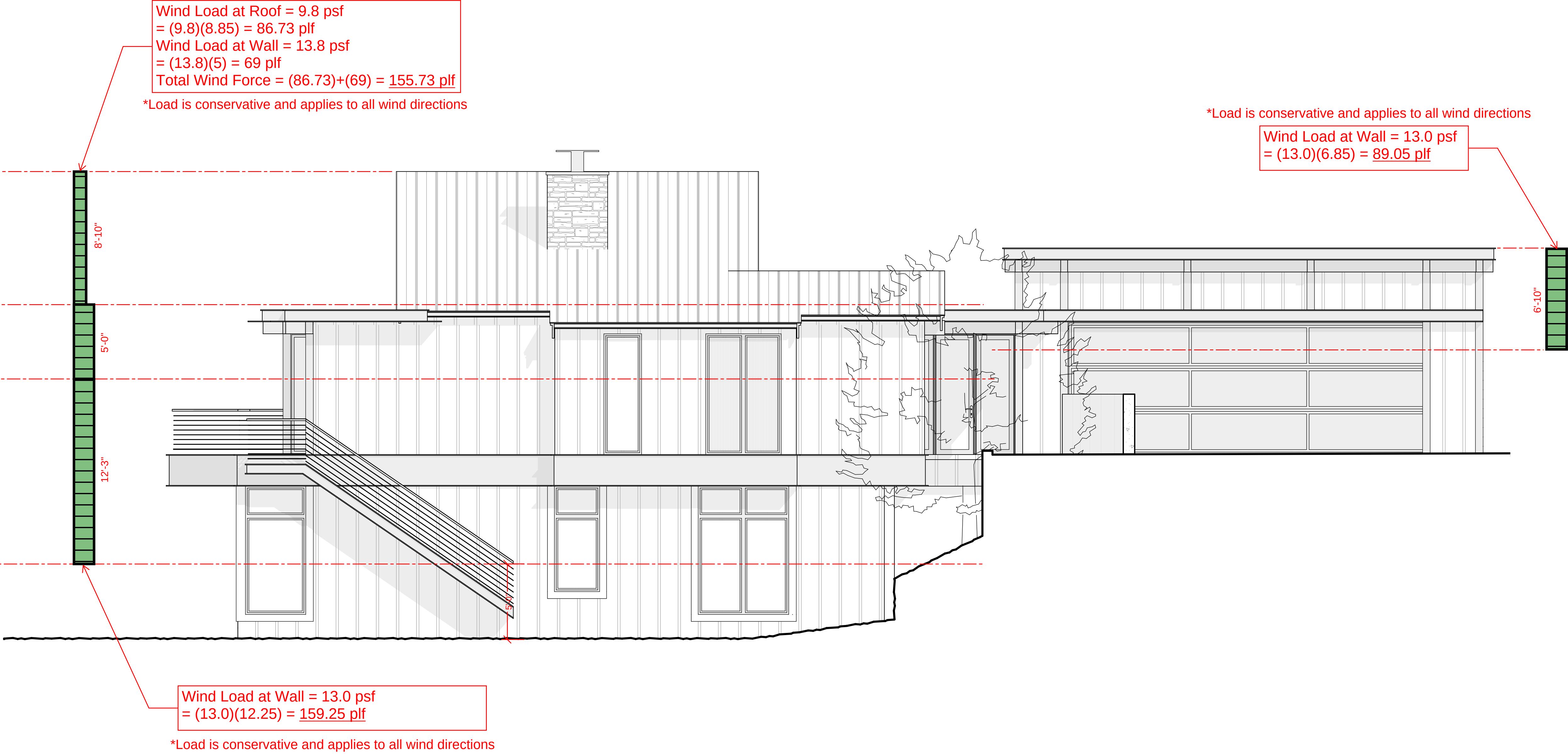
Disclaimer

Hazard loads are interpolated from data provided in ASCE 7 and rounded up to the nearest whole integer. Per ASCE 7, islands and coastal areas outside the last contour should use the last wind speed contour of the coastal area – in some cases, this website will extrapolate past the last wind speed contour and therefore, provide a wind speed that is slightly higher. NOTE: For queries near wind-borne debris region boundaries, the resulting determination is sensitive to rounding which may affect whether or not it is considered to be within a wind-borne debris region.

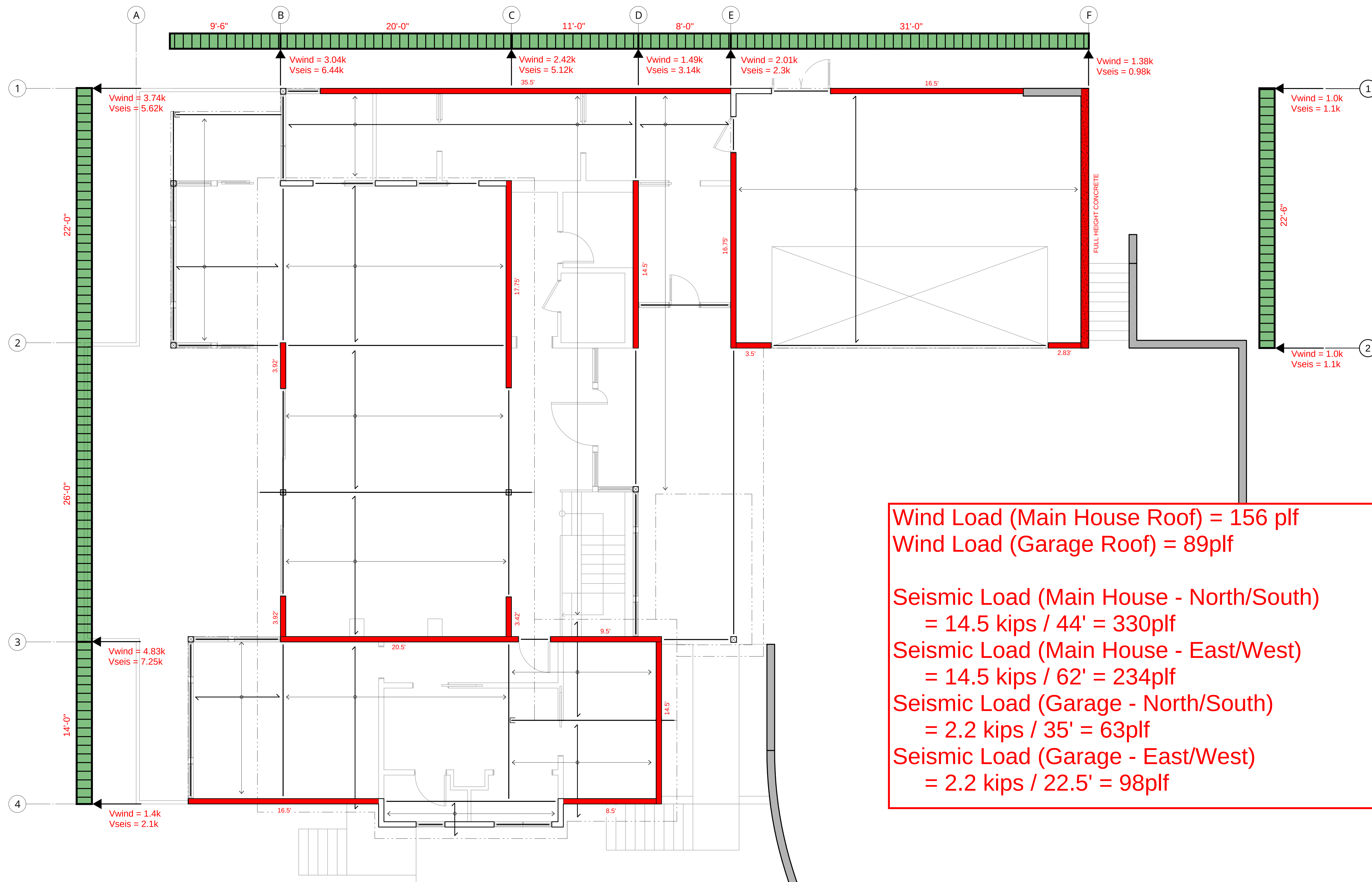
Mountainous terrain, gorges, ocean promontories, and special wind regions shall be examined for unusual wind conditions.

While the information presented on this website is believed to be correct, ATC and its sponsors and contributors assume no responsibility or liability for its accuracy. The material presented in the report should not be used or relied upon for any specific application without competent examination and verification of its accuracy, suitability and applicability by engineers or other licensed professionals. ATC does not intend that the use of this information replace the sound judgment of such competent professionals, having experience and knowledge in the field of practice, nor to substitute for the standard of care required of such professionals in interpreting and applying the results of the report provided by this website. Users of the information from this website assume all liability arising from such use. Use of the output of this website does not imply approval by the governing building code bodies responsible for building code approval and interpretation for the building site described by latitude/longitude location in the report.

Wind Analysis
Scale: 1/4" = 1'-0"



Zimmer Residence



Lateral Design and Shearwall Keyplan - Upper Roof/Main Floor Walls
 Scale: 1/4" = 1'-0"

Zimmer Residence

Roof Diaphragm/Main Floor Walls

North/South Direction

Grid	B	C	D
Vwind (kips)	3.04	2.42	1.49
Vseismic (kips)	6.44	5.12	3.14
Length of wall (ft)	8	21.17	31.25
v_wind (p/f)**	Moment Frame	167	48
v_siesmic (p/l)**		354	100
h (ft)		10	10
OTF_Wind (lbs)*		1671	477
OTF_Seismic (lbs)*		2419	1005
Length of shortest wall pier (ft)		3.42	14.5
Apect Ratio Reduction for Seismic Loads		2.92	0.69
Siesmic Penalty		0.68	1.0
Shearwall		W4	W6
Holdown		(2) CS16	H DU2

*OTF does not take into account dead load and weight of the wall uno

**v_siesmic/wind includes penalty

North/South Direction

Grid	E	F
Vwind (kips)	2.01	1.38
Vseismic (kips)	2.3	0.98
Length of wall (ft)	29	22.5
v_wind (p/f)**	69	Full Height Concrete
v_siesmic (p/l)**	79	
h (ft)	10	
OTF_Wind (lbs)*	693	
OTF_Seismic (lbs)*	793	
Length of shortest wall pier (ft)	16.75	
Apect Ratio Reduction for Seismic Loads	0.60	
Siesmic Penalty	1.0	
Shearwall	W6	
Holdown	H DU2	

*OTF does not take into account dead load and weight of the wall uno

**v_siesmic/wind includes penalty

Roof Diaphragm/Main Floor Walls

East/West Direction

Grid	1	3	4
Vwind (kips)	3.74	4.83	1.4
Vseismic (kips)	5.62	7.25	2.1
Length of wall (ft)	35.5	30	25
v_wind (p/f)**	105	161	56
v_siesmic (p/l)**	158	242	84
h (ft)	10	10	10
OTF_Wind (lbs)*	1054	1610	560
OTF_Seismic (lbs)*	1583	2417	840
Length of shortest wall pier (ft)	35.5	9.5	8.5
Apect Ratio Reduction for Seismic Loads	0.28	1.05	1.18
Siesmic Penalty	1.0	1.0	1.0
Shearwall	W6	W6	W6
Holdown	CS16 or HDU2	(2)CS16 or HDU4	CS16 or HDU2

***OTF does not take into account dead load and weight of the wall uno**

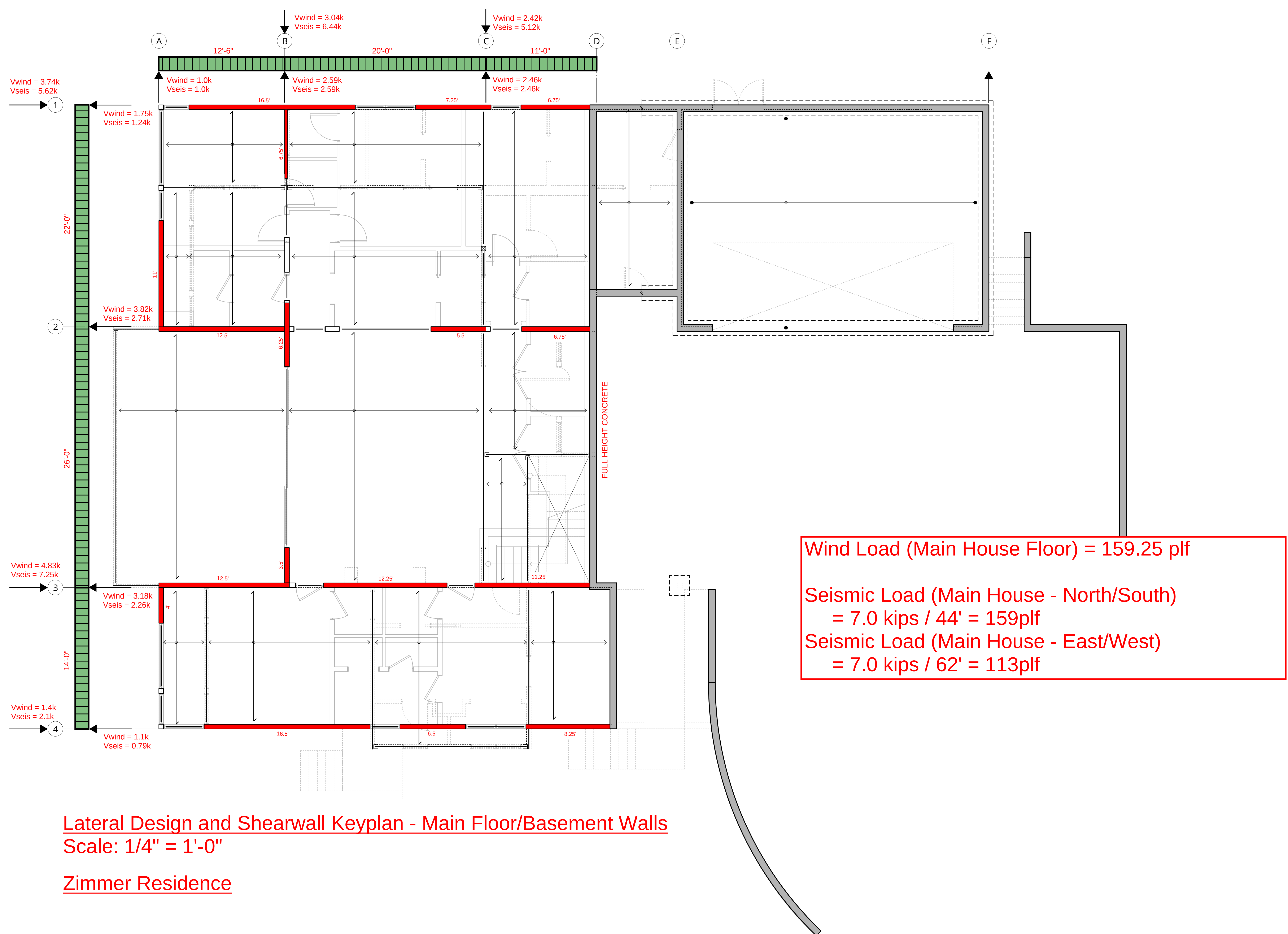
****v_siesmic/wind includes penalty**

East/West Direction

Grid	1 (Between E & F)	2 (Between E & F)
Vwind (kips)	1	1
Vseismic (kips)	1.1	1.1
Length of wall (ft)	16.5	3.5
v_wind (p/f)**	61	408
v_siesmic (p/l)**	67	449
h (ft)	10	10
OTF_Wind (lbs)*	606	4082
OTF_Seismic (lbs)*	667	3143
Length of shortest wall pier (ft)	16.5	3.5
Apect Ratio Reduction for Seismic Loads	0.61	2.86
Siesmic Penalty	1.0	0.70
Shearwall	W6	W3
Holdown	HDU2	HDU5

***OTF does not take into account dead load and weight of the wall uno**

****v_siesmic/wind includes penalty**



Main Floor Diaphragm/Basement \

North/South Direction

Grid	A	B
Vwind (kips)	1	5.63
Vseismic (kips)	1	9.03
Length of wall (ft)	15	16.5
v_wind (p/f)**	83	Moment Frame
v_siesmic (p/l)**	83	
h (ft)	10	
OTF_Wind (lbs)*	833	
OTF_Seismic (lbs)*	667	
Length of shortest wall pier (ft)	4	
Apect Ratio Reduction for Seismic Loads	2.50	
Siesmic Penalty	0.80	
Shearwall	W6	
Holdown	HDU2	

*OTF does not take into account dead load and weight of the wall uno

**v_siesmic/wind includes penalty

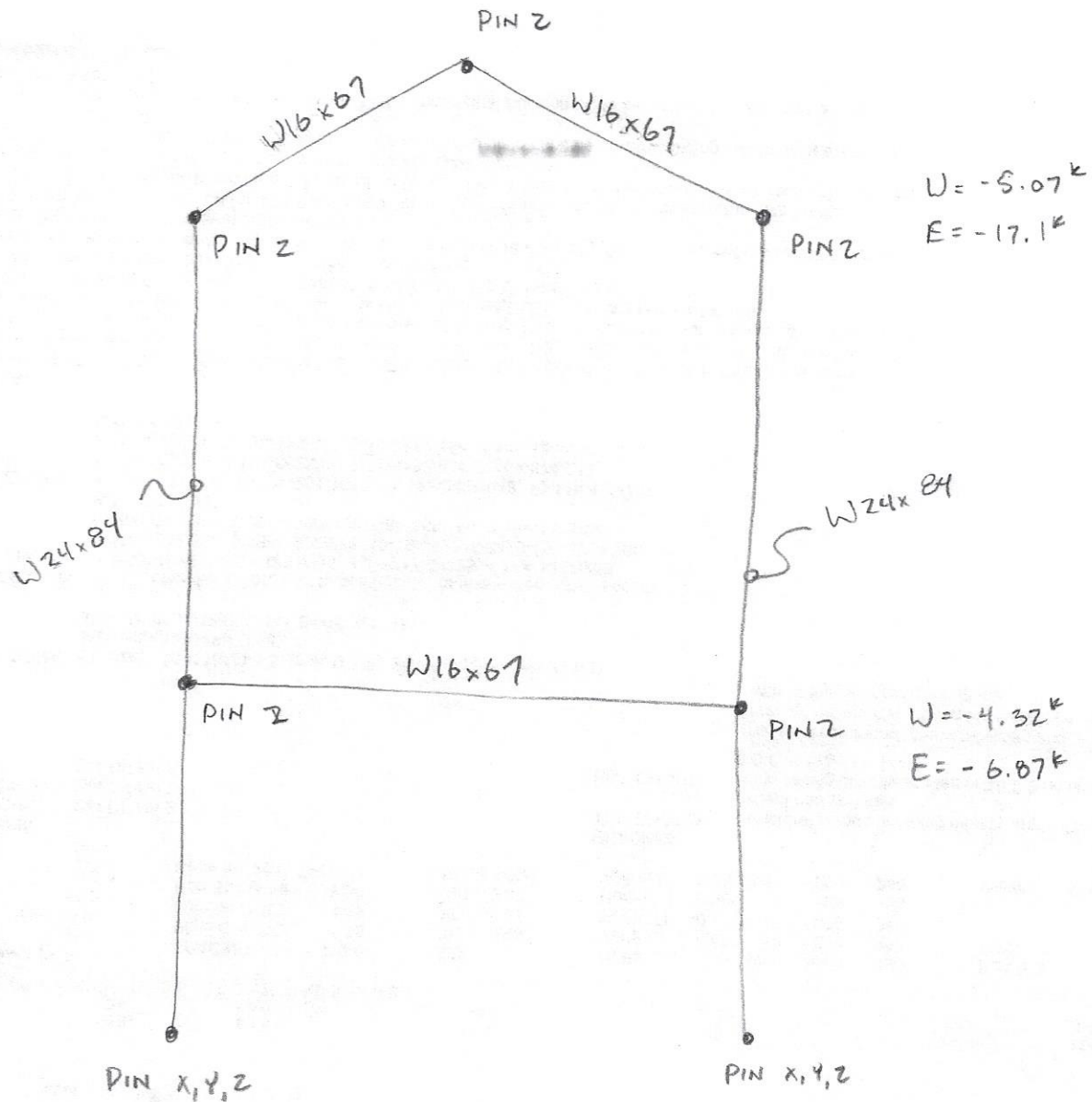
East/West Direction

Grid	1	2	3	4
Vwind (kips)	5.49	3.82	8.01	2.5
Vseismic (kips)	6.86	2.71	9.51	2.89
Length of wall (ft)	30.5	24.75	36	31.25
v_wind (p/f)**	180	154	223	80
v_siesmic (p/l)**	225	109	264	92
h (ft)	10	10	10	10
OTF_Wind (lbs)*	1800	1543	2225	800
OTF_Seismic (lbs)*	2249	1095	2642	925
Length of shortest wall pier (ft)	6.75	5.5	11.25	6.5
Apect Ratio Reduction for Seismic Loads	1.48	1.82	0.89	1.54
Siesmic Penalty	1.0	1.0	1.0	1.0
Shearwall	W6	W6	W6	W6
Holdown	HDU4	HDU2	HDU5	HDU2

*OTF does not take into account dead load and weight of the wall uno

**v_siesmic/wind includes penalty

MOMENT FRAME - INITIAL DRIFT CHECK



$$\Delta_2 (\text{ALLOWABLE DRIFT}) = 226.5'' (0.02) = 4.53'' \Rightarrow \delta_2 \leq \frac{4.53''}{3} = 1.51''$$

$$\Delta_1 (\text{ALLOWABLE DRIFT}) = 133.5'' (0.02) = 2.67'' \Rightarrow \delta_1 \leq \frac{0.742''}{3} = 0.89''$$



Simpson Strong-Tie Yield-Link Moment Connection Calculations

Project Name: Zimmer Residence

Frame Elevation ID: 1

Project Address:

Design Firm:

Design Engineer:

Designed for:

Design Date: 11/13/2024

Tool Version: 4.1.2

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

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	

Column Check (BLC Details):

	DCR	Limit
SCWL_DCR:	1.00	
DCR_PZ:	1.00	
Stiffener DCR:	1.00	
Column Flange / Net Section DCR:	1.00	

Beam and Link Check (BLC Details):

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

Use section: **AISC W Sections**

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

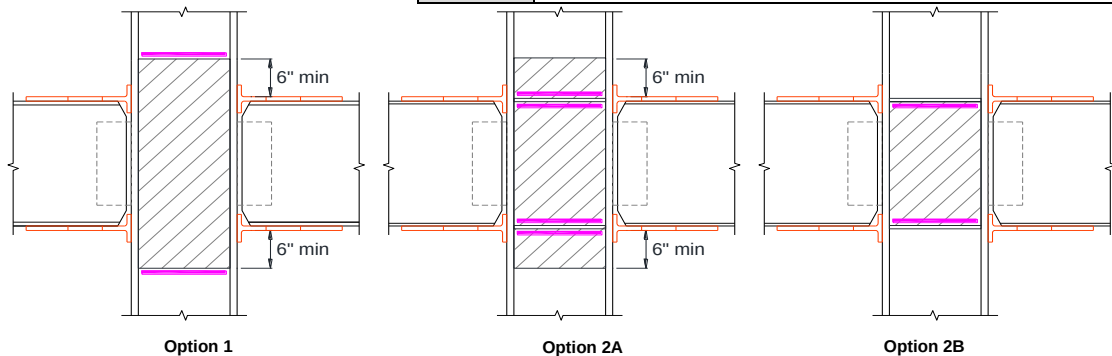
Flange (Yes/NO): **NO**

User Input for Connection Welding Preferences:

Doubler PL to col web/Cont plate Weld:

Option_2B

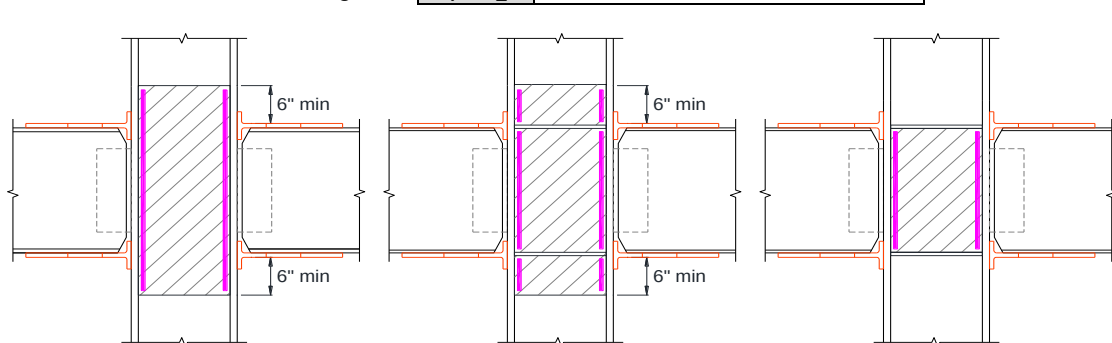
Doubler placed between continuity plates



Doubler PL to col flange weld:

Option_1

Fillet Weld



Use of Plug-weld for Doubler Plates? **NO**

Use **Pu** for Column PZ Check

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

Design Parameters	
f1=	0.5
f2=	0.7
ρ=	1.0
Ω=	2.5
S _{DS} =	0.961

Total LC= 36

ID	Load Combinations	Load Multipliers											Link Strength	Seismic Drift	Beam + Col. (P+M)	V_bm Gravity	Wind Defl.	Excel Tool ID
		DL	LL	LR	SL	RL	WL	NL	EL_D	EL								
SST_C1	DL	1	0	0	0	0	0	0	0	0	0	Load Case						
SST_C2	LL	0	1	0	0	0	0	0	0	0	0							
SST_C3	LR	0	0	1	0	0	0	0	0	0	0							
SST_C4	SL	0	0	0	1	0	0	0	0	0	0							
SST_C5	RL	0	0	0	0	1	0	0	0	0	0							
SST_C6	WL	0	0	0	0	0	1	0	0	0	0							
SST_C7	NL	0	0	0	0	0	0	1	0	0	0							
SST_C8	EL_D	0	0	0	0	0	0	0	1	0	0							
SST_C9	EL	0	0	0	0	0	0	0	0	1	1							
SST_LC01	1.4 DL + NL	1.4	0	0	0	0	0	1	0	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	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	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	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	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	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	0		x		x	x		
SST_LC08	(1.2 + 0.2*SDS)DL + f1*LL + f2*SL	1.3922	0.5	0.5	0.7	0.5	0	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	0		x		x			
SST_LC10	1.2 DL + 1.6 LR - 0.5 WL	1.2	0	1.6	0	0	-0.5	0	0	0	0	x		x				
SST_LC11	1.2 DL + 1.6 SL + 0.5 WL	1.2	0	0	1.6	0	0.5	0	0	0	0	x		x			11	
SST_LC12	1.2 DL + 1.6 SL - 0.5 WL	1.2	0	0	1.6	0	-0.5	0	0	0	0	x		x			12	
SST_LC13	1.2 DL + 1.6 RL + 0.5 WL	1.2	0	0	0	1.6	0.5	0	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	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	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	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	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	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	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	0	x		x				
SST_LC21	0.9 DL + 1.0 WL	0.9	0	0	0	0	1	0	0	0	0	x		x				
SST_LC22	0.9 DL - 1.0 WL	0.9	0	0	0	0	-1	0	0	0	0	x		x				
SST_LC23	1.0 DL + 0.5 LL + 0.5 LR + 0.45 WL	1.0	0.75	0.75	0	0	0.45	0	0	0	0					x	13	
SST_LC24	1.0 DL + 0.5 LL + 0.5 LR - 0.45 WL	1.0	0.75	0.75	0	0	-0.45	0	0	0	0					x	14	
SST_LC25	(1.2 + 0.2*SDS)DL + EL*rho + f1*LL + f2*SL	1.3922	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	1.3922	0.5	0.5	0.7	0	0	0	0	-1.0			x					6
SST_LC27	(0.9 - 0.2 SDS)DL + EL*rho	0.7078	0	0	0	0	0	0	0	1.0			x					
SST_LC28	(0.9 - 0.2 SDS)DL - EL*rho	0.7078	0	0	0	0	0	0	0	-1.0			x					
SST_LC29	(1.2 + 0.2*SDS)DL + EL_D + f1*LL + f2*SL	1.3922	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	1.3922	0.5	0.5	0.7	0	0	0	-1.0	0				x				2
SST_LC31	(0.9 - 0.2 SDS)DL + EL_D	0.7078	0	0	0	0	0	0	1.0	0				x				
SST_LC32	(0.9 - 0.2 SDS)DL - EL_D	0.7078	0	0	0	0	0	0	-1.0	0				x				
SST_LC33	(1.2 + 0.2 SDS)DL + OmegaEL + f1*LL + f2*SL	1.3922	0.5	0	0.7	0	0	0	0	2.5			Ω			x		
SST_LC34	(1.2 + 0.2 SDS)DL - OmegaEL + f1*LL + f2*SL	1.3922	0.5	0	0.7	0	0	0	0	-2.5					x			8
SST_LC35	(0.9 - 0.2 SDS)DL + OmegaEL	0.7078	0	0	0	0	0	0	0	2.5					x			
SST_LC36	(0.9 - 0.2 SDS)DL - OmegaEL	0.7078	0	0	0	0	0	0	0	-2.5				x				

Job Name: --> Please save an Excel File for EACH frame elevation
Frame Elevation ID:
File Path:
sub path:
file:

Typ. Roof Slab / Nailer Depth:
Typ. Floor Slab / Nailer Depth:

Number of Stories:
Typical Story Height:

DSA / OSHPD ?	NO
IBC	2021
R=	3.5
Cd=	3
Ie=	1
Omega=	2.5
Rho=	1
Adjust Allowable Drift $\Delta a/\rho$?	NO

Base=
Extension below Base=

Number of Bays:
Typical Bay Span:

Base=
Extension below Base=

Typ. Allowable Drift Limit:
Seismic Drift:
Wind Drift: Check for Load Combinations

Basic Gravity Loads:

wDL_roof=	0.027 klf	Typical uniform dead load on ROOF beams
wLL_roof=	0.000 klf	Typical uniform live load on ROOF beams
wDL_floor=	0.100 klf	Typical uniform dead load on FLOOR beams
wLL_floor=	0.200 klf	Typical uniform live load on FLOOR beams
S _{DS} =	0.961	Short period design spectral response acceleration parameter

OTHER Loads:

	Left End	Right End	
wSL_roof:	0.033	0.033	klf, Typical uniform/Trapezoidal SNOW load (SL) on ROOF beams
wRL_roof:	0.000	0.000	klf, Typical uniform/Trapezoidal RAIN load (RL) on ROOF beams

vW_roof=	0.000 klf	Typical uniform wind load (W) on ROOF (Vertical Load) [neg. value up]
f1=	0.5	Live Load factor, See IBC 2021, 1605.2
f2=	0.7	Snow Load factor, See IBC 2021, 1605.2

Frame Geometry																	
Elev ID	Grid ID	Story	Story Height (in.)	Left Beam ID	Left Beam Size	Left Beam Bay Span (in.)	Column size	Column Orientation	Base Offset (in.)	Fixed Base Col width-to-thickness DCR	Beam ID	Beam Size	Link Size	I_End Column ID	J_End Column ID	I_End Column Size	J_End Column Size
1	A	Story2	186.8	N/A	N/A	N/A	W24X84	Strong	N/A	N/A							
1	B	Story2	186.8	2001	W16X67	251.1	W24X84	Strong	N/A	N/A	2001	W16X67	YLMC6-3.5	1-A-Story2	1-B-Story2	W24X84	W24X84
1	A	Story1	147.2	N/A	N/A	N/A	W24X84	Strong	0.0	N/A							
1	B	Story1	147.2	1001	W16X67	251.1	W24X84	Strong	0.0	N/A	1001	W16X67	YLMC6-3.5	1-A-Story1	1-B-Story1	W24X84	W24X84

1. INITIAL YIELD-LINK SELECTION SUMMARY																		
Elev ID	Grid ID	Story	Assign Link at I_End	Assign Link at J_End	Mu-link (kip-in.)	Vgravity (kips)	Pu-sp (kips)	Initial tbr check	Initial bbr check	Initial bcf check	Initial Ly-link check	Link Strength DCR	PZ I-End Col. DCR	PZ J-End Col. DCR	K_rot (kip-in./rad)	phi * Mn (kip-in.)	Mcsp_link (kip-in.)	Beam Slope Condition (x" per foot)
1	A	Story2																
1	B	Story2	YES	YES	1327	4.5	22.8	N/A	0.76	0.83	0.65	0.66	N/A	N/A	6.63E+05	2014	3491	0.00
1	A	Story1																
1	B	Story1	YES	YES	1764	3.8	9.8	N/A	0.76	0.83	0.65	0.88	N/A	N/A	6.63E+05	2014	3491	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.)
1	A	Story2	3	W24X84																														
1	B	Story2	4	W24X84				2001	W16X67	251.1	2.5	125.6																3.1	125.6					
1	A	Story1	1	W24X84																														
1	B	Story1	2	W24X84				1001	W16X67	251.1																								

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
1	Story2	1-A-Story2	186.8	251.1	17.1	17.1	0.8	0.020 Hx	3	1	1	5.1	hx/250
1	Story1	1-A-Story1	147.2	251.1	6.9	6.9	0.3	0.020 Hx	3	1	1	4.3	hx/250

Column Vertical Point Loads (DL, LL/LR, SL)							Beam Uniform Load (Dead)										Beam Uniform Loads (Live/Roof Live)										Beam Uniform/Trapezoidal Loads (Snow)															
Elev ID	Grid ID	Story	Column ID	Column Size	Top Story?	Roof Beam?	Beam ID	Beam Size	Beam Span (in.)	W _{DL1} (kips/ft)	W _{DL1_x1} (in.)	W _{DL1_x2} (in.)	W _{DL2} (kips/ft)	W _{DL2_x1} (in.)	W _{DL2_x2} (in.)	W _{DL3} (kips/ft)	W _{DL3_x1} (in.)	W _{DL3_x2} (in.)	W _{LL1} (kips/ft)	W _{LL1_x1} (in.)	W _{LL1_x2} (in.)	W _{LL2} (kips/ft)	W _{LL2_x1} (in.)	W _{LL2_x2} (in.)	W _{LL3} (kips/ft)	W _{LL3_x1} (in.)	W _{LL3_x2} (in.)	W _{S1_start} (kips/ft)	W _{S1_end} (kips/ft)	W _{S1_x1} (in.)	W _{S1_x2} (in.)	W _{S2_start} (kips/ft)	W _{S2_end} (kips/ft)	W _{S2_x1} (in.)	W _{S2_x2} (in.)	W _{S3_start} (kips/ft)	W _{S3_end} (kips/ft)	W _{S3_x1} (in.)	W _{S3_x2} (in.)			
1	A	Story2	3	N/A	YES	YES																																				
1	B	Story2	4	2001	YES	YES	2001	W16X67	251.1	0.037	0.0	251.1																	0.033	0.033	0.0	251.1										
1	A	Story1	1	N/A	NO	NO																																				
1	B	Story1	2	1001	NO	NO	1001	W16X67	251.1	0.100	0.0	251.1							0.200	0.0	251.1								0.000	0.000	0.0	248.0										

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 18.2]	δ_{xe} (in.)	C_d	I_e	δ_x (in.)	Allowable Drift Limit	Allowable Drift (in.)	Drift DCR
1	A	Story2	1-A-Story2	186.8	17.10	3	1.954	1.008	3	1	3.023	0.020 Hx	3.74	0.809
1	A	Story1	1-A-Story1	147.2	6.87	2	0.946	0.946	3	1	2.838	0.020 Hx	2.94	0.964

					2a. Stability Check for P-delta Effects									2b. Wind deflection						
Elev ID	Grid ID	Story	Column Location ID	Story Height (in.)	P _i (kips)	P _x (kips)	F _x (kips)	V _x (kips)	hs _x (in.)	Stability Coeff, θ	β (max Link DCR)	θ_{max}	Stability θ , DCR	F _{wind} (kips)	Drift Node ID	Frame Wind Displacement (in.) [LC 13&14]	Drift (in.)	Allowable Drift Limit	Allowable Drift (in.)	Wind Drift DCR
1	A	Story2	1-A-Story2	186.8	1	1	17.10	17	187	0.0003	1	0.17	0.002	5.07	3	0.307	0.155	hx/250	0.75	0.207
1	A	Story1	1-A-Story1	147.2	1	2	6.87	24	147	0.0005	1	0.17	0.003	4.32	2	0.152	0.152	hx/250	0.59	0.259

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)
SST_C1	DL	1	0.1	6.2	0.0	4	-0.1	6.2	0.0
SST_C2	LL	1	0.3	2.1	0.0	4	-0.3	2.1	0.0
SST_C3	LR	1	0.0	0.0	0.0	4	0.0	0.0	0.0
SST_C4	SL	1	-0.1	2.4	0.0	4	0.1	2.4	0.0
SST_C5	RL	1	0.0	0.0	0.0	4	0.0	0.0	0.0
SST_C6	WL	1	-4.7	-8.8	0.0	4	-4.7	8.8	0.0
SST_C7	NL	1	-0.5	-1.1	0.0	4	-0.5	1.1	0.0
SST_C8	EL_D	1	-12.0	-25.5	0.0	4	-12.0	25.5	0.0
SST_C9	EL	1	-12.0	-25.5	0.0	4	-12.0	25.5	0.0
SST_LC01	1.4 DL + NL	1	-0.4	7.6	0.0	4	-0.7	9.9	0.0
SST_LC02	1.2 DL + 1.6 LL + 0.5 LR + NL	1	0.0	9.7	0.0	4	-1.1	12.0	0.0
SST_LC03	1.2 DL + 1.6 LL + 0.5 SL + NL	1	0.0	10.9	0.0	4	-1.0	13.2	0.0
SST_LC04	1.2 DL + 1.6 LL + 0.5 RL + NL	1	0.0	9.7	0.0	4	-1.1	12.0	0.0
SST_LC05	1.2 DL + 1.6 LR + f1 LL + NL	1	-0.3	7.4	0.0	4	-0.8	9.7	0.0
SST_LC06	1.2 DL + 1.6 SL + f1 LL + NL	1	-0.4	11.3	0.0	4	-0.7	13.5	0.0
SST_LC07	1.2 DL + 1.6 RL + f1 LL + NL	1	-0.3	7.4	0.0	4	-0.8	9.7	0.0
SST_LC08	(1.2 + 0.2*SDS)DL + f1*LL + f2*SL	1	0.2	11.4	0.0	4	-0.2	11.4	0.0
SST_LC09	1.2 DL + 1.6 LR + 0.5 WL	1	-2.2	3.1	0.0	4	-2.5	11.9	0.0
SST_LC10	1.2 DL + 1.6 LR - 0.5 WL	1	2.5	11.9	0.0	4	2.2	3.1	0.0
SST_LC11	1.2 DL + 1.6 SL + 0.5 WL	1	-2.3	7.0	0.0	4	-2.4	15.7	0.0
SST_LC12	1.2 DL + 1.6 SL - 0.5 WL	1	2.4	15.8	0.0	4	2.3	7.0	0.0
SST_LC13	1.2 DL + 1.6 RL + 0.5 WL	1	-2.2	3.1	0.0	4	-2.5	11.9	0.0
SST_LC14	1.2 DL + 1.6 RL - 0.5 WL	1	2.5	11.9	0.0	4	2.2	3.1	0.0
SST_LC15	1.2 DL + 1.0 WL + f1 LL + 0.5 LR	1	-4.5	-0.3	0.0	4	-4.9	17.3	0.0
SST_LC16	1.2 DL - 1.0 WL + f1 LL + 0.5 LR	1	4.9	17.3	0.0	4	4.5	-0.3	0.0
SST_LC17	1.2 DL + 1.0 WL + f1 LL + 0.5 SL	1	-4.5	1.0	0.0	4	-4.9	18.5	0.0
SST_LC18	1.2 DL - 1.0 WL + f1 LL + 0.5 SL	1	4.9	18.5	0.0	4	4.5	0.9	0.0
SST_LC19	1.2 DL + 1.0 WL + f1 LL + 0.5 RL	1	-4.5	-0.3	0.0	4	-4.9	17.3	0.0
SST_LC20	1.2 DL - 1.0 WL + f1 LL + 0.5 RL	1	4.9	17.3	0.0	4	4.5	-0.3	0.0
SST_LC21	0.9 DL + 1.0 WL	1	-4.6	-3.2	0.0	4	-4.8	14.4	0.0
SST_LC22	0.9 DL - 1.0 WL	1	4.8	14.4	0.0	4	4.6	-3.2	0.0
SST_LC23	1.0 DL + 0.5 LL + 0.5 LR + 0.45 WL	1	-1.8	3.8	0.0	4	-2.4	11.7	0.0
SST_LC24	1.0 DL + 0.5 LL + 0.5 LR - 0.45 WL	1	2.4	11.7	0.0	4	1.8	3.8	0.0
SST_LC25	(1.2 + 0.2*SDS)DL + EL*rho + f1*LL + f2*SL	1	-11.8	-14.1	0.0	4	-12.2	36.9	0.0
SST_LC26	(1.2 + 0.2 SDS)DL - EL*rho + f1*LL + f2* SL	1	12.2	36.9	0.0	4	11.8	-14.1	0.0
SST_LC27	(0.9 - 0.2 SDS) DL + EL*rho	1	-11.9	-21.1	0.0	4	-12.0	29.9	0.0
SST_LC28	(0.9 - 0.2 SDS) DL - EL*rho	1	12.0	29.9	0.0	4	11.9	-21.1	0.0
SST_LC29	(1.2 + 0.2*SDS)DL + EL_D+ f1*LL + f2*SL	1	-11.8	-14.1	0.0	4	-12.2	36.9	0.0
SST_LC30	(1.2 + 0.2 SDS)DL - EL_D+ f1*LL + f2* SL	1	12.2	36.9	0.0	4	11.8	-14.1	0.0
SST_LC31	(0.9 - 0.2 SDS) DL + EL_D	1	-11.9	-21.1	0.0	4	-12.0	29.9	0.0
SST_LC32	(0.9 - 0.2 SDS) DL - EL_D	1	12.0	29.9	0.0	4	11.9	-21.1	0.0
SST_LC33	(1.2 + 0.2 SDS)DL + OmegaEL + f1*LL + f2*SL	1	-29.7	-52.3	0.0	4	-30.2	75.1	0.0
SST_LC34	(1.2 + 0.2 SDS)DL - OmegaEL + f1*LL + f2*SL	1	30.2	75.1	0.0	4	29.7	-52.3	0.0
SST_LC35	(0.9 - 0.2 SDS) DL + OmegaEL	1	-29.9	-59.3	0.0	4	-30.0	68.1	0.0
SST_LC36	(0.9 - 0.2 SDS) DL - OmegaEL	1	30.0	68.1	0.0	4	29.9	-59.3	0.0

2B. BEAM DEFLECTION CHECK SUMMARY																	
Elev ID	Grid ID	Story	Beam ID	Beam Size	DL Max. Defl. (in.)	LL Max. Defl. (in.)	DL+LL Max. Defl. (in.)	SL Max. Defl. (in.)	WL Max. Defl. (in.)	LL Defl. Limit	DL+LL Defl. Limit	SL Defl. Limit	WL Defl. Limit	LL Defl. DCR	DL+LL Defl. DCR	SL Defl. DCR	WL Defl. DCR
1	B	Story2	2001	W16X67	0.022	0.000	0.022	0.021	0.018	L/360	L/240	L/360	L/360	0.001	0.021	0.030	0.025
1	B	Story1	1001	W16X67	0.011	0.014	0.024	0.003	0.024	L/360	L/240	L/360	L/360	0.019	0.023	0.004	0.035

3. BEAM AND YIELD-LINK CHECK SUMMARY															
Elev ID	Grid ID	Story	Beam ID	Beam Size	Link Size	M _{u-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-link} DCR	Beam b/t	Beam h/tw
1	A	Story2	2001	W16X67	YLMC6-3.5	1327	BRP6A	0.764	0.602	0.746	0.659	0.598	0.651	Compact	Compact
1	A	Story1	1001	W16X67	YLMC6-3.5	1764	BRP6A	0.764	0.602	0.746	0.876	0.795	0.651	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
1	A	Story2	2001	W16X67	251.1	YLMC6-3.5	22.82	4.46	36.20	5.6	3	2	7/8	A325-N	3/8	1	1/4	0.379	OK	0.981	0.682	0.464
1	A	Story1	1001	W16X67	251.1	YLMC6-3.5	9.80	3.84	35.58	5.2	3	2	7/8	A325-N	3/8	1	1/4	0.327	OK	0.865	0.527	0.456

5. COLUMN CHECK SUMMARY																				
Elev ID	Grid ID	Story	Column Location	Story Height (in.)	Column size	Pu Column (kips)	Bottom Stiffener Provided?	Doubler Plate Required?	Stiffener Required for AISC 360 J10?	Min. Stiffener Thickness (in.)	Provided Doubler Thickness (in.)	SCWB DCR Check	Column Net Section 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	LRFD Column Bracing Force (kips)
1	A	Story2	1-A-Story2	186.8	W24X84	32.9	YES	NO	NO	3/8	0	N/A	0.095	0.255	0.585	3/16	3/16	Compact	Compact	2.8875
1	B	Story2	1-B-Story2	186.8	W24X84	32.9	YES	NO	NO	3/8	0	N/A	0.095	0.255	0.585	3/16	3/16	Compact	Compact	2.8875
1	A	Story1	1-A-Story1	147.2	W24X84	75.1	YES	NO	NO	3/8	0	0.099	0.128	0.304	0.619	3/16	3/16	Compact	Compact	2.8875
1	B	Story1	1-B-Story1	147.2	W24X84	75.1	YES	NO	NO	3/8	0	0.099	0.128	0.304	0.619	3/16	3/16	Compact	Compact	2.8875

ID	Elev.	Grid	Story	Column	Left Shear PL		Right Shear PL		Stiff. PL	Doublers PL	1A. Left ST to col flange Weld			1B. Right 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 Weld		5. DP to col flange weld		
					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	Fillet Size, (in.)	# of sides	Fillet Size, (in.)	# of sides	Fillet Size, (in.)	# of sides	Fillet Size, (in.)	# of sides	Diameter, (in.)
3	1	A	Story2	W24X84	N/A	N/A	1	3/8	3/8	N/A	N/A	N/A	N/A	1/4	2	N/A	N/A	3/16	2	3/16	2	N/A	N/A	N/A	N/A	N/A	N/A
4	1	B	Story2	W24X84	1	3/8	N/A	N/A	3/8	N/A	1/4	2	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	N/A
1	1	A	Story1	W24X84	N/A	N/A	1	3/8	3/8	N/A	N/A	N/A	N/A	1/4	2	N/A	N/A	3/16	2	3/16	2	N/A	N/A	N/A	N/A	N/A	N/A
2	1	B	Story1	W24X84	1	3/8	N/A	N/A	3/8	N/A	1/4	2	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	N/A

7. Preliminary Beam Design Summary																					
Elev ID	Grid ID	Story	Beam ID	Beam Size	Link Size	Beam Bay Span (in.)	Left Column	Right Column	Lb (in.)	Beam Interm. Bracing	Mcap_link (k-in)	Pu_sp (kips)	Mu Omega (k-in)	Vu (kips)	Mcap_Link/Mu_Omega	Axial DCR	Flexural DCR	P+M DCR	V DCR	Adj. Flexure DCR	Total P+M DCR
1	A	Story2	2001	W16X67	YLMC6-3.5	251.1	W24X84	W24X84	220	None	3491	22.8	3511.5	31.06	0.99	0.046	0.603	0.626	0.161	0.599	0.622
1	A	Story1	1001	W16X67	YLMC6-3.5	251.1	W24X84	W24X84	220	None	3491	9.8	4780.1	40.93	0.73	0.020	0.819	0.829	0.212	0.598	0.608

			7B. Beam Design Per Analysis 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
1	A	Story2	2001	22.8	3511.5	31.06	491.5	5850.0	193.2	3491	0.597	0.046	0.603	0.99	0.161	0.626	0.622
1	A	Story1	1001	9.8	4780.1	40.93	491.5	5850.0	193.2	3491	0.597	0.020	0.819	0.73	0.212	0.829	0.608

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?	Column Orientation	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
3	1	A	Story2	1-A-Story2	186.8	W24X84	NO	Strong	1.83	32.9	22.8	3511	688	3511	0.058	0.399	0.428	0.067
4	1	B	Story2	1-B-Story2	186.8	W24X84	NO	Strong	1.83	32.9	22.8	3512	688	3512	0.058	0.399	0.428	0.067
1	1	A	Story1	1-A-Story1	147.2	W24X84	NO	Strong	1.61	75.1	30.2	4092	0	4092	0.105	0.465	0.518	0.089
2	1	B	Story1	1-B-Story1	147.2	W24X84	NO	Strong	1.61	75.1	30.2	4092	0	4092	0.105	0.465	0.518	0.089

8. AISC 360-16 Steel Column Design

Column ID: 1
LC: 8
Column Size= W24X84
Column Orientation= Strong
W-Section
Col Beam Bot Bracing? NO
L_b= 147 in

8.1. Loading

M_u= 4092 kip-in
M_{max}= 4092 kip-in
M_A= 1077 kip-in
P_u= 75.1 kip
M_{u_top}= 4092 kip-in
M_B= 2153.9 kip-in
V_u= 30.2 kip
M_{u_bot}= 0 kip-in
M_C= 3230.9 kip-in
TopGov

8.2. Material properties

F_y= 50 ksi
F_u= 65 ksi
b_f= 9.02 in
d= 24.1 in
k_{des}= 1.27 in
I_x= 2370.0 in⁴
I_y= 94.4 in⁴
J= 3.700 in⁴
h= 22.56 in
h_c= 22.76 in
h_p= 24.10 in
E= 29000 ksi
G= 11200 ksi
r_x= 9.790 in
r_y= 1.95 in
t_f=t_{fi}= 0.77 in
t_{fe}= 0.77 in
t_w= 0.47 in
C_w= 12800 in⁶
A_g= 24.700 in²
Z_x= 224.000 in³
Z_y= 32.600 in³
S_x= 196.000 in³
S_{xc}= 196.00 in³
S_{xt}= 196.00 in³
H_z= 1.0
S_y= 20.900 in³
=d-2*t_f
=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)
=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= 0.58
λ= 5.86
λ_p= 9.15
F_L= 35.00
λ_{r_case10}= 24.08
λ_{r_case11}= 20.78
λ_r= 24.08
Flange: Compact
=max(0.35,min(4/sqrt(h/t_w),0.76)
b_f/(2*t_f)
=0.38*sqrt(E/F_y)
=IF(S_{xt}/S_{xc}>=0.7,0.7*F_y,max(F_y*S_{xt}/S_{xc},0.5*F_y))
=1*sqrt(E/F_y)
=0.95*sqrt(K_c*E/F_L)
=IF(λ<λ_p,"Compact",IF(AND(λ<λ_r,λ>λ_p),"Non-Compact","Slender"))
For built-up sections

8.3.1.a Web compactness check flexure (Table B4.1b: Case 15):

λ= 48.00
λ_p= 90.55
λ_r= 137.27
Web: Compact
=h/t_w
=3.76*sqrt(E/F_y)
=5.70*sqrt(E/F_y)
=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
Φ_c= 0.9
Φ_v= 1
=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"))))

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= 5.86
λ_{r_case1}= 13.49
k_c= 0.58
λ_{r_case2}= 11.71
λ_r= 13.49
Flange: Nonslender
b_f/(2*t_f)
=0.56*sqrt(E/F_y)
From Section 3
=0.64*sqrt(K_c*E/F_y)

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

λ= 48.00
λ_r= 35.88
Web: Slender
=h/t_w
=1.49*sqrt(E/F_y)

8.5. Flexural Capacity

8.5.1. Yielding (F2.1)

$$M_{n_{F2.1}} = 9800 \text{ kip-in}$$

$$M_y = F_y * S_x \text{ (Col not braced at Bm bottom)}$$

Weak Axis Bending:

$$M_{n_{y_{F2.1}}} = 1630 \text{ kip-in}$$

$$= M_{py} = F_y * Z_y \text{ (F6-1)}$$

8.5.2. Lateral-torsional buckling (F2.2)

$$L_p = 82.65 \text{ in}$$

$$= 1.76 * r_y * \sqrt{E/F_y}$$

$$\text{(F2-5)}$$

$$r_{ts} = 2.37 \text{ in}$$

$$= \sqrt{\sqrt{I_y * C_w} / S_x}$$

$$\text{(F2-7)}$$

$$h_o = 23.330 \text{ in}$$

$$= d - t_f$$

Distance between centroids

$$c = 1$$

1 for doubly symmetric I-Shapes

$$\text{(EQ F2-8a)}$$

$$L_r = 243.50 \text{ in}$$

$$= 1.95 * r_{ts} * E / (0.7 * F_y) * \sqrt{(J * c) / (S_x * h_o)} * \sqrt{1 + \sqrt{1 + 6.76 * (0.7 * F_y / E * S_x * h_o / (J * c))^2}}$$

$$\text{(F2-6)}$$

$$C_{b_calculated} = 1.61$$

$$= 12.5 * M_{max} / (2.5 * M_{max} + 3 * M_A + 4 * M_B + 3 * M_C)$$

$$C_b = 1.61$$

$$= IF(C_b = C_{b_calculated}, C_{b_calculated}, C_b)$$

$$\text{(F1-1)}$$

$$F_{cr} = 133.0 \text{ ksi}$$

$$= C_b * \pi^2 * E / (L_b / r_{ts})^2 * \sqrt{1 + 0.078 * J_{col} * C_{col} / (S_x * h_o) * (L_b / r_{ts})^2}$$

$$\text{(F2-4)}$$

$$M_{n_{F2.2}} = 9800 \text{ kip-in}$$

$$= IF(L_b \leq L_p, M_p, IF(AND(L_p < L_b, L_b \leq L_r), MIN(C_b * (M_p - (M_p - 0.7 * F_y * S_x) * ((L_b - L_p) / (L_r - L_p))), M_p), MIN(F_{cr} * S_x, M_p)))$$

$$\text{(F2-2, 2-3)}$$

8.5.3. Lateral torsional buckling (F3.1)

$$M_{n_{F3.1}} = 9800$$

$$= M_{n_{F2.2}}$$

8.5.4. Compression flange local buckling (F3.2)

$$K_c = 0.58$$

$$= \min(\max(0.35, 4 / \sqrt{h/t_w}), 0.76)$$

$$\lambda = 5.857$$

$$= b_f / (2 * t_f)$$

$$\lambda_{pf} = 9.15$$

$$= 0.38 * \sqrt{E/F_y}$$

$$\lambda_{rf} = 24.1$$

$$= 1.0 * \sqrt{E/F_y}$$

$$M_{n_{F3.2}} = 9800$$

$$= IF(\text{Flange} = \text{"Non-Compact"}, M_p - (M_p - 0.7 * F_y * S_x) * (\lambda - \lambda_{pf}) / (\lambda_{rf} - \lambda_{pf}), IF(\text{Flange} = \text{"Slender"}, 0.9 * E * K_c * S_x / \lambda^2, M_p))$$

$$\text{(F3-1, 3-2)}$$

8.5.5. Compression flange yielding (F4.1)

$$h_c = 22.56 \text{ in}$$

$$= d - 2 * t_f$$

$$\lambda_{w_{F4}} = 48.00$$

$$= h_o / t_w$$

$$\lambda_{pw} = 90.55$$

$$= 3.76 * \sqrt{E/F_y}$$

$$\lambda_{rw} = 137.27$$

$$= 5.7 * \sqrt{E/F_y}$$

$$M_{yc} = 9800 \text{ kip-in}$$

$$= F_y * S_{xc}$$

$$R_{pc} = 1.00$$

$$= IF(h_c / t_w \leq \lambda_{pw}, M_p / M_{yc}, MIN(M_p / M_{yc} - (M_p / M_{yc} - 1) * (\lambda - \lambda_{pw}) / (\lambda_{rw} - \lambda_{pw}), M_p / M_{yc}))$$

$$M_{n_{F4.1}} = 9800$$

$$= R_{pc} * M_{yc}$$

$$\text{(F4-1)}$$

8.5.6. Lateral torsional buckling (F4.2)

$$a_{w_{F42}} = 1.53$$

$$= h_c * t_w / (b_f * t_f)$$

$$\text{(F4-12)}$$

$$r_{t_{F4}} = 2.32 \text{ in}$$

$$= b_f / \sqrt{12 * (1 + 1/6 * a_{w_{F42}})}$$

$$\text{(F4-11)}$$

$$L_{p_{F4}} = 61.59$$

$$= 1.1 * r_{t_{F4}} * \sqrt{E/F_y}$$

$$\text{(F4-7)}$$

$$L_{r_{F4}} = 239.04 \text{ in}$$

$$= 1.95 * r_{t_{F4}} * E / F_{L} * \sqrt{J / (S_{xc} * h_o)} + \sqrt{(J / S_{xc} * h_o)^2 + 6.76 * (F_L / E)^2}$$

$$\text{(F4-8)}$$

$$F_{cr_{F4}} = 128.68 \text{ ksi}$$

$$= C_b * \pi^2 * E / (L_b / r_{t_{F4}})^2 * \sqrt{1 + 0.078 * J / (S_{xc} * h_o) * (L_b / r_{t_{F4}})^2}$$

$$\text{(F4-5)}$$

$$M_{n_{F4.2}} = 9800 \text{ kip-in}$$

$$= IF(L_b \leq L_p, M_p, IF(L_b > L_r, F_{cr_{F4}} * S_{xc}, MIN(C_b * (R_{pc} * M_{yc} - (R_{pc} * M_{yc} - F_L * S_{xc}) * (L_b - L_p) / (L_r - L_p)), R_{pc} * M_{yc})))$$

$$\text{(F4-2, 4-3)}$$

8.5.7. Compression flange local buckling (F4.3)

$$\lambda_{f_{F43}} = 5.86$$

$$= b_f / (2 * t_f)$$

$$\lambda_{pf} = 9.15$$

From Section 3 above

$$\lambda_{rf} = 24.08$$

From Section 3 above

$$M_{n_{F13}} = 10449$$

$$= R_{pc} * M_{yc} - (R_{pc} * M_{yc} - F_L * S_{xc}) * (\lambda_{f_{F43}} - \lambda_{pf}) / (\lambda_{rf} - \lambda_{pf})$$

$$\text{(F4-13)}$$

$$M_{n_{F14}} = 86092$$

$$0.9 * E * K_c * S_{xc} / \lambda_{f_{F432}}^2$$

$$\text{(F4-14)}$$

$$M_{n_{F43}} = 9800 \text{ kip-in}$$

$$= IF(\text{Flange} = \text{"Compact"}, M_p, IF(\text{Flange} = \text{"Non-Compact"}, M_{n_{F13}}, M_{n_{F14}}))$$

8.5.8. Tension flange yielding (F4.4)

$\lambda_{F44} =$	48.00		$= h_c / t_w$	
$M_{yt} =$	9800	kip-in	$= F_y * S_{xt}$	(F4-16a)
$\lambda_{pw} =$	91		From Section 3 above	
$\lambda_{rw} =$	137		From Section 3 above	
$R_{pt_F4-16a} =$	1.00		$= M_p / M_{yt}$	(F4-16b)
$R_{pt_F4-16b} =$	1.00		$= M_p / M_{yt} * (M_p / M_{yt} - 1) * (\lambda_{F44} - \lambda_{pw}) / (\lambda_{rw} - \lambda_{pw})$	
$R_{pt} =$	1.00		$= IF(\lambda_{F44} \leq \lambda_{pw}, R_{pt_F4-16a}, R_{pt_F4-16b})$	
$M_{n_F4.4} =$	9800	kip-in	$= R_{pt} * M_{yt}$	(F4-15)

8.5.9. Compression flange yielding (F5.1)

$a_w =$	1.53		$= \min(10, a_{w_F42})$	
$r_{t_F5} =$	2.32	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)
$M_{n_F5.1} =$	9800	kip-in	$= R_{pg} * F_y * S_{xc}$	(F5-1)

8.5.10. Lateral-torsional buckling (F5.2)

$L_{p_F5} =$	61.6	in	$= 1.1 * r_{t_F5} * \sqrt{E / F_y}$	(F4-7)
$L_{r_F5} =$	210	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	$= IF(L_b \leq L_{p_F5}, "NA", IF(L_b > L_{r_F5}, F_{cr_F5.4}, F_{cr_F5.3}))$	
$M_{n_F5.2} =$	9800	kip-in	$= IF(L_b \leq L_{p_F5}, M_p, R_{pg} * F_{cr_LTB_F5} * S_{xc})$	(F5-2)

8.5.11. Compression flange local buckling (F5.3)

$\lambda_f =$	5.86		From Section 3 above	
$\lambda_{fp} =$	9.15		From Section 3 above	
$\lambda_{fr} =$	24.08		From Section 3 above	
$F_{cr_F5.8} =$	53	ksi	$= F_y - 0.3 * F_y * (\lambda_f - \lambda_{fp}) / (\lambda_{fr} - \lambda_{fp})$	(F5-8)
$F_{cr_F5.9} =$	439	ksi	$= 0.9 * E * k_c / (b_f / (2 * t_f))^2$	(F5-9)
$F_{cr_F5} =$	50	ksi	$= \text{if}(\text{Flange} = "Compact", 50, \text{if}(\text{Flange} = "Non-Compact", F_{cr_F5.8}, F_{cr_F5.9}))$	
$M_{n_F5.3} =$	9800	kip-in	$= R_{pg} * F_{cr_F5} * S_{xc}$	(F5-7)

Weak Axis Bending:

$F_{cr_F6.4} =$	583	ksi		(F6-4)
$M_{n_F6.2} =$	1828	k-in		(F6-2)
$M_{n_F6.4} =$	12191	k-in		(F6-3)

8.5.12. Tension flange yielding (F5.4)

$M_{n_F5.4} =$	9800	kip-in	$= M_p$	Both flanges are symmetric	(F5-10)
-----------------	------	--------	---------	----------------------------	---------

8.5.13. Flange hole reduction (F13.1)

$D_{HoleT} =$	0.000	in		
$A_{fg} =$	6.95	in ²	$= b_f * t_f$	
$A_{fn} =$	6.95	in ²	$= t_f * (b_f - 2 * D_{HoleT})$	
$Y_t =$	1.00		$= IF(F_u / F_y \leq 0.8, 1, 1.1)$	
$M_{n_F13.1} =$	12740	kip-in	$= F_u * A_{fn} / A_{fg} * S_x$	(F13-1)

Flexural Capacity Summary

$M_{n_F2} =$	9800	kip-in	$= \min(M_{n_F2.1}, M_{n_F2.2})$	Weak Axis Bending:
$M_{n_F3} =$	9800	kip-in	$= \min(M_{n_F3.1}, M_{n_F3.2})$	$M_{ny} =$ 1630 kip-in
$M_{n_F4} =$	9800	kip-in	$= \min(M_{n_F4.1}, M_{n_F4.2}, M_{n_F4.3}, M_{n_F4.4})$	
$M_{n_F5} =$	9800	kip-in	$= \min(M_{n_F5.1}, M_{n_F5.2}, M_{n_F5.3}, M_{n_F5.4})$	
$M_n =$	9800	kip-in	$= IF(B3 = "Beam", IF(\text{DesignSection} = "F2", \min(M_{n_F2}, M_{n_F13.1}), IF(\text{DesignSection} = "F3", \min(M_{n_F3}, M_{n_F13.1}), IF(\text{DesignSection} = "F4", \min(M_{n_F4}, M_{n_F13.1}), \min(M_{n_F5}, M_{n_F13.1}))))), M_{n_F4})$	
$\Phi M_n =$	8820	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
$L_b =$	147.2 in	$L_y =$	147 in	$L_z =$	147 in
$K_{LR} =$	75.49	$=\max(K_x * L_b / r_x, K_y * L_b / r_y)$			
$F_e =$	50.23 ksi	$=\pi^2 * E / (K_{LR})^2$			
$F_{cr_E3} =$	32.96 ksi	$=IF(K_{LR} \leq 4.71 * \text{SQRT}(E/F_y), F_y * 0.658^{(F_y/F_e)}, 0.877 * F_e)$			
$P_{n_E3} =$	814.18 kip	$=F_{cr_E3} * A_g$			

(E3-4)
(E3-2, 3-3)
(E3-1)

8.6.2. E.4 Torsional and flexural-torsional buckling: (Applies to singly symmetric shapes)

$F_{ey} =$	50.23 ksi	$=\pi^2 * E / (K_y * L_y / r_y)^2$	(E3-4)
$F_{ez} =$	85.42 ksi	$=\pi^2 * E * C_w / (K_z * L_z)^2 + G * J) / (1 / (I_x + I_y))$	(E4-2)
$F_{e_E4} =$	50.23 ksi	$=(F_{ey} + F_{ez}) / (2 * H_z) * (1 - \text{sqrt}(1 - 4 * F_{ey} * F_{ez} * H_z / (F_{ey} + F_{ez})^2))$	(E4-3)
$F_{cr_E4} =$	32.96 ksi	$=IF(K_{LR} \leq 4.71 * \text{SQRT}(E/F_y), F_y * 0.658^{(F_y/F_{e_E4})}, 0.877 * F_{e_E4})$	(E3-2, 3-3)
$P_{n_E4} =$	814.2 kip	$=A_g * F_{cr_E4}$	(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

$\lambda =$	5.86	From Section 3 above
$\lambda_f =$	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
$F_{el} =$	588.5 ksi	$=(c2 * \lambda_{rw} / \lambda_w)^2 * F_y$
$F_{cr} =$	33.0 ksi	$=\text{Min}(F_{cr_E3}, F_{cr_E4})$
$bf =$	9.02 in	=Full Flange Width from AISC Database
$b_{e_E7-3} =$	2.7 in	$=bf * (1 - c1 * \text{sqrt}(F_{el} / F_{cr})) * \text{sqrt}(F_{el} / F_{cr})$, but can't be more than bf
$be =$	9.02 in	$=If(\text{Flange} = "Nonslender", b_f, b_{e_E7-3})$
$A_{web} =$	10.81 in ²	$=A_g - 2 * t_f * bf$
$A_f =$	13.89 in ²	$=2 * t_f * be$
$A_{e1} =$	24.7 in ²	$=A_{web} + A_f$

(E7-3)

8.6.2b Calculation for effective web depth

Web= Slender

$\lambda_w =$	48.00	From Section 4.1 above
$\lambda_{rw} =$	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
$F_{el} =$	48.0 ksi	$=(c2 * \lambda_{rw} / \lambda_w)^2 * F_y$
$h =$	22.6 in	
$h_{e_E7-3} =$	21.3 in	$=h * (1 - c1 * \text{sqrt}(F_{el} / F_{cr})) * \text{sqrt}(F_{el} / F_{cr})$, but can't be more than h
$h_e =$	21.3 in	$=If(\text{Web} = "Nonslender", h, h_{e_E7-3})$
$A_{flange} =$	13.9 in ²	$=2 * bf * t_f$
$A_{web_radius} =$	10.8 in ²	$=A_g - A_{flange}$
$A_{web} =$	10.6 in ²	$=h * t_w$
$A_{radius} =$	0.2 in ²	$=A_{web_radius} - A_{web}$
$A_{e_web} =$	10.01 in ²	$=h_e * t_w$
$A_{e2} =$	24.1 in ²	$=A_{flange} + A_{radius} + A_{e_web}$
$A_e =$	24.1 in ²	$=\text{Min}(A_{e1}, A_{e2})$
$F_{cr} =$	33.0 ksi	
$P_{n_E7} =$	794.7 kips	
$P_n =$	794.7 kip	$=IF(\text{AxialDesignSection} = "E3", P_{n_E3}, P_{n_E7})$
$\Phi_c * P_n =$	715.2 kip	

(E7-3)

8.7. Demand per AISC 360-16 Appendix 8 (to include p-small delta)

$P_y =$	1235	kip	$= A_g * F_y$	
$P_r =$	75	kip	$= P_{u_omega}$	
$\alpha =$	1		1 for LRFD, 1.6 for ASD	
$C_m =$	1		conservative assumption	(A-8-5)
$P_{e1y} =$	31306	kip	$= \pi^2 * E * I_x / (L_b)^2$	
$B_1 =$	1.002		$= C_m / (1 - \alpha * P_r / P_{e1y})$	(A-8-3)

8.8 Axial + flexural interaction (Chapter H)

$M_r =$	4102.3	kip-in	$= M_u * B_1$	
$M_c =$	8820	kip-in	$= \Phi * M_n$	
$P_r =$	75.1	kip	$= P_u$	
$P_c =$	715.2	kip	$= \Phi_c * P_n$	
DCR_Axial=	0.105		$= P_r / P_c$	
DCR_Flexural=	0.465		$= M_r / M_c$	
DCR_P+M=	0.518		$= \text{IF}(P_r / P_c \geq 0.2, P_r / P_c + 8/9 * M_r / M_c, P_r / (2 * P_c) + M_r / M_c)$	(H1-1)

8.9. Shear capacity (Chapter G)

Hole _{Web} =	0	in		
$V_u =$	30.19	kips		
$A_w =$	11.33	in ²	$= \text{IF}(\text{ColOrient} = \text{"Strong"}, t_w * (d - \text{Hole}_{web}), 2 * (b_f * t_f))$	
$h/t_w =$	48			
$k_v =$	5.34		Webs without transverse stiffeners	
$2.24 \sqrt{E/F_y} =$	53.9			
$C_{v1_G2-2} =$	1			(G2-2)
$1.1 \sqrt{k_v E/F_y} =$	61.2			
$C_{v1_G2-3} =$	1.00			(G2-3)
$C_{v1_G2-4} =$	1.3		$= 1.1 * \sqrt{k_v * E/F_y} / (h/t_w)$	(G2-4)
$C_{v1_G2-3_4?} =$	1.0		$= \text{if}(h/t_w \leq 1.1 * \sqrt{k_v * E/F_y}, C_{v1_G2-3}, C_{v1_G2-4})$	
$C_{v1} =$	1.0		$= \text{if}(C_{v1_G2-2} = \text{"N/A"}, C_{v1_G2-3_4?})$	
$V_n =$	339.8	kip	$= 0.6 * F_y * A_w * C_{v1}$	(G2-1)
DCR_Shear=	0.089		$= V_u / (\Phi_v * V_n)$	

8.10. Design Summary:

		Demand-to-Capacity Ratios			
LC ID	Load Combination	P	M	P+M	V
LC 8	(1.2 + 0.2 SDS)DL -OmegaEL + f1*LL + f2*SL	0.105	0.465	0.518	0.089

8. AISC 360-16 Steel Column Design

Column ID: **2** Column Size= **W24X84** W-Section **L_b= 147** in
LC: **7** Column Orientation= **Strong** Col Beam Bot Bracing? **NO**

8.1. Loading

$M_u = 4092$ kip-in $P_u = 75.1$ kip $V_u = 30.2$ kip
 $M_{max} = 4092$ kip-in $M_{u_top} = 4092$ kip-in **TopGov** $M_{u_bot} = 0$ kip-in
 $M_A = 1077$ kip-in $M_B = 2153.9$ kip-in $M_C = 3230.9$ kip-in

8.2. Material properties

$F_y = 50$ ksi $E = 29000$ ksi $A_g = 24.700$ in²
 $F_u = 65$ ksi $G = 11200$ ksi $Z_x = 224.000$ in³
 $bf = 9.02$ in $r_x = 9.790$ in $Z_y = 32.600$ in³
 $d = 24.1$ in $r_y = 1.95$ in $S_x = 196.000$ in³
 $k_{des} = 1.27$ in $t_f = t_{fi} = 0.77$ in $S_{xc} = 196.00$ in³
 $I_x = 2370.0$ in⁴ $t_{fe} = 0.77$ in $S_{xt} = 196.00$ in³
 $I_y = 94.4$ in⁴ $t_w = 0.47$ in $H_z = 1.0$
 $J = 3.700$ in⁴ $C_w = 12800$ in⁶ $S_y = 20.900$ in³
 $h = 22.56$ in $= d - 2 \cdot t_f$
 $h_c = 22.76$ in $= 2 \cdot (d - t_{fi} - (b_f \cdot t_{fi}^2 / 2 + t_w \cdot (d - t_{fe} - t_{fi}) \cdot (d - t_{fe} + t_{fi}) / 2 + b_f \cdot t_{fe} \cdot (d - t_{fe} / 2)) / A_g)$
 $h_p = 24.10$ in $= 2 \cdot (d - (b_f \cdot (t_{fe} - t_{fi}) / 2 + t_w \cdot (d + t_{fi} - t_{fe})) / (2 \cdot t_w))$

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

$k_c = 0.58$ $= \max(0.35, \min(4 / \sqrt{h / t_w}, 0.76))$ For built-up sections
 $\lambda = 5.86$ $b_f / (2 \cdot t_f)$
 $\lambda_p = 9.15$ $= 0.38 \cdot \sqrt{E / F_y}$
 $F_L = 35.00$ $= \text{IF}(S_{xt} / S_{xc} > 0.7, 0.7 \cdot F_y, \max(F_y \cdot S_{xt} / S_{xc}, 0.5 \cdot F_y))$
 $\lambda_{r_case10} = 24.08$ $= 1 \cdot \sqrt{E / F_y}$ Case 10-W-Sections
 $\lambda_{r_case11} = 20.78$ $= 0.95 \cdot \sqrt{K_c \cdot E / F_L}$ Case 11 BU-Sections
 $\lambda_f = 24.08$
Flange: **Compact** $= \text{IF}(\lambda < \lambda_p, \text{"Compact"}, \text{IF}(\text{AND}(\lambda < \lambda_r, \lambda > \lambda_p), \text{"Non-Compact"}, \text{"Slender"}))$

8.3.1.a Web compactness check flexure (Table B4.1b: Case 15):

$\lambda = 48.00$ $= h / t_w$
 $\lambda_p = 90.55$ $= 3.76 \cdot \sqrt{E / F_y}$
 $\lambda_r = 137.27$ $= 5.70 \cdot \sqrt{E / F_y}$
Web: **Compact** $= \text{IF}(\lambda < \lambda_p, \text{"Compact"}, \text{IF}(\text{AND}(\lambda < \lambda_r, \lambda > \lambda_p), \text{"Non-Compact"}, \text{"Slender"}))$

Per Table F1.1 for Flexure

Section: **F2**
 $\Phi_b = 0.9$ $= \text{IF}(\text{AND}(\text{Flange} = \text{"Compact"}, \text{Web} = \text{"Compact"}), \text{"F2"}, \text{IF}(\text{AND}(\text{OR}(\text{Flange} = \text{"Non-Compact"}, \text{Flange} = \text{"Slender"}), \text{Web} = \text{"Compact"}), \text{"F3"}, \text{IF}(\text{AND}(\text{OR}(\text{Flange} = \text{"Compact"}, \text{Flange} = \text{"Non-Compact"}, \text{Flange} = \text{"Slender"}), \text{Web} = \text{"Slender"}), \text{"F5"}, \text{"F4"})))$
 $\Phi_c = 0.9$
 $\Phi_v = 1$

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

$\lambda = 5.86$ $b_f / (2 \cdot t_f)$
 $\lambda_{r_case1} = 13.49$ $= 0.56 \cdot \sqrt{E / F_y}$
 $k_c = 0.58$ From Section 3
 $\lambda_{r_case2} = 11.71$ $= 0.64 \cdot \sqrt{K_c \cdot E / F_y}$
 $\lambda_f = 13.49$
Flange: **Nonslender**

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

$\lambda = 48.00$ $= h / t_w$
 $\lambda_r = 35.88$ $= 1.49 \cdot \sqrt{E / F_y}$
Web: **Slender**

8.5. Flexural Capacity

8.5.1. Yielding (F2.1)

$$M_{n_{F2.1}} = 9800 \text{ kip-in}$$

$$M_y = F_y * S_x \text{ (Col not braced at Bm bottom)}$$

Weak Axis Bending:

$$M_{n_{y_{F2.1}}} = 1630 \text{ kip-in}$$

$$= M_{py} = F_y * Z_y \text{ (F6-1)}$$

8.5.2. Lateral-torsional buckling (F2.2)

$$L_p = 82.65 \text{ in}$$

$$= 1.76 * r_y * \sqrt{E/F_y}$$

$$\text{(F2-5)}$$

$$r_{ts} = 2.37 \text{ in}$$

$$= \sqrt{\sqrt{I_y * C_w} / S_x}$$

$$\text{(F2-7)}$$

$$h_o = 23.330 \text{ in}$$

$$= d - t_f$$

Distance between centroids

$$c = 1$$

1 for doubly symmetric I-Shapes

$$\text{(EQ F2-8a)}$$

$$L_r = 243.50 \text{ in}$$

$$= 1.95 * r_{ts} * E / (0.7 * F_y) * \sqrt{(J * c) / (S_x * h_o)} * \sqrt{1 + \sqrt{1 + 6.76 * (0.7 * F_y / E * S_x * h_o / (J * c))^2}}$$

$$\text{(F2-6)}$$

$$C_{b_calculated} = 1.61$$

$$= 12.5 * M_{max} / (2.5 * M_{max} + 3 * M_A + 4 * M_B + 3 * M_C)$$

$$C_b = 1.61$$

$$= IF(C_b = C_{b_calculated}, C_{b_calculated}, C_b)$$

$$\text{(F1-1)}$$

$$F_{cr} = 133.0 \text{ ksi}$$

$$= C_b * \pi^2 * E / (L_b / r_{ts})^2 * \sqrt{1 + 0.078 * J_{col} * C_{col} / (S_x * h_o) * (L_b / r_{ts})^2}$$

$$\text{(F2-4)}$$

$$M_{n_{F2.2}} = 9800 \text{ kip-in}$$

$$= IF(L_b \leq L_p, M_p, IF(AND(L_p < L_b, L_b \leq L_r), MIN(C_b * (M_p - (M_p - 0.7 * F_y * S_x) * ((L_b - L_p) / (L_r - L_p))), M_p), MIN(F_{cr} * S_x, M_p)))$$

$$\text{(F2-2, 2-3)}$$

8.5.3. Lateral torsional buckling (F3.1)

$$M_{n_{F3.1}} = 9800$$

$$= M_{n_{F2.2}}$$

8.5.4. Compression flange local buckling (F3.2)

$$K_c = 0.58$$

$$= \min(\max(0.35, 4 / \sqrt{h/t_w}), 0.76)$$

$$\lambda = 5.857$$

$$= b_f / (2 * t_f)$$

$$\lambda_{pf} = 9.15$$

$$= 0.38 * \sqrt{E/F_y}$$

$$\lambda_{rf} = 24.1$$

$$= 1.0 * \sqrt{E/F_y}$$

$$M_{n_{F3.2}} = 9800$$

$$= IF(\text{Flange} = \text{"Non-Compact"}, M_p - (M_p - 0.7 * F_y * S_x) * (\lambda - \lambda_{pf}) / (\lambda_{rf} - \lambda_{pf}), IF(\text{Flange} = \text{"Slender"}, 0.9 * E * K_c * S_x / \lambda^2, M_p))$$

$$\text{(F3-1, 3-2)}$$

8.5.5. Compression flange yielding (F4.1)

$$h_c = 22.56 \text{ in}$$

$$= d - 2 * t_f$$

$$\lambda_{w_{F4}} = 48.00$$

$$= h_o / t_w$$

$$\lambda_{pw} = 90.55$$

$$= 3.76 * \sqrt{E/F_y}$$

$$\lambda_{rw} = 137.27$$

$$= 5.7 * \sqrt{E/F_y}$$

$$M_{yc} = 9800 \text{ kip-in}$$

$$= F_y * S_{xc}$$

$$R_{pc} = 1.00$$

$$= IF(h_c / t_w \leq \lambda_{pw}, M_p / M_{yc}, MIN(M_p / M_{yc} - (M_p / M_{yc} - 1) * (\lambda - \lambda_{pw}) / (\lambda_{rw} - \lambda_{pw}), M_p / M_{yc}))$$

$$M_{n_{F4.1}} = 9800$$

$$= R_{pc} * M_{yc}$$

$$\text{(F4-1)}$$

8.5.6. Lateral torsional buckling (F4.2)

$$a_{w_{F42}} = 1.53$$

$$= h_c * t_w / (b_f * t_f)$$

$$\text{(F4-12)}$$

$$r_{t_{F4}} = 2.32 \text{ in}$$

$$= b_f / \sqrt{12 * (1 + 1/6 * a_{w_{F42}})}$$

$$\text{(F4-11)}$$

$$L_{p_{F4}} = 61.59$$

$$= 1.1 * r_{t_{F4}} * \sqrt{E/F_y}$$

$$\text{(F4-7)}$$

$$L_{r_{F4}} = 239.04 \text{ in}$$

$$= 1.95 * r_{t_{F4}} * E / F_{L} * \sqrt{J / (S_{xc} * h_o)} + \sqrt{J / (S_{xc} * h_o)^2 + 6.76 * (F_L / E)^2}$$

$$\text{(F4-8)}$$

$$F_{cr_{F4}} = 128.68 \text{ ksi}$$

$$= C_b * \pi^2 * E / (L_b / r_{t_{F4}})^2 * \sqrt{1 + 0.078 * J / (S_{xc} * h_o) * (L_b / r_{t_{F4}})^2}$$

$$\text{(F4-5)}$$

$$M_{n_{F4.2}} = 9800 \text{ kip-in}$$

$$= IF(L_b \leq L_p, M_p, IF(L_b > L_r, F_{cr_{F4}} * S_{xc}, MIN(C_b * (R_{pc} * M_{yc} - (R_{pc} * M_{yc} - F_L * S_{xc}) * (L_b - L_p) / (L_r - L_p)), R_{pc} * M_{yc})))$$

$$\text{(F4-2, 4-3)}$$

8.5.7. Compression flange local buckling (F4.3)

$$\lambda_{f_{F43}} = 5.86$$

$$= b_f / (2 * t_f)$$

$$\lambda_{pf} = 9.15$$

From Section 3 above

$$\lambda_{rf} = 24.08$$

From Section 3 above

$$M_{n_{F13}} = 10449$$

$$= R_{pc} * M_{yc} - (R_{pc} * M_{yc} - F_L * S_{xc}) * (\lambda_{f_{F43}} - \lambda_{pf}) / (\lambda_{rf} - \lambda_{pf})$$

$$\text{(F4-13)}$$

$$M_{n_{F14}} = 86092$$

$$0.9 * E * K_c * S_{xc} / \lambda_{f_{F432}}^2$$

$$\text{(F4-14)}$$

$$M_{n_{F43}} = 9800 \text{ kip-in}$$

$$= IF(\text{Flange} = \text{"Compact"}, M_p, IF(\text{Flange} = \text{"Non-Compact"}, M_{n_{F13}}, M_{n_{F14}}))$$

8.5.8. Tension flange yielding (F4.4)

$\lambda_{F44} =$	48.00		$=h_c/t_w$	
$M_{yt} =$	9800	kip-in	$=F_y * S_{xt}$	(F4-16a)
$\lambda_{pw} =$	91		From Section 3 above	
$\lambda_{rw} =$	137		From Section 3 above	
$R_{pt_F4-16a} =$	1.00		$=M_p/M_{yt}$	(F4-16b)
$R_{pt_F4-16b} =$	1.00		$=M_p/M_{yt} * (M_p/M_{yt} - 1) * (\lambda_{F44} - \lambda_{pw}) / (\lambda_{rw} - \lambda_{pw})$	
$R_{pt} =$	1.00		$=IF(\lambda_{F44} \leq \lambda_{pw}, R_{pt_F4-16a}, R_{pt_F4-16b})$	
$M_{n_F4.4} =$	9800	kip-in	$=R_{pt} * M_{yt}$	(F4-15)

8.5.9. Compression flange yielding (F5.1)

$a_w =$	1.53		$=\min(10, a_{w_F42})$	
$r_{t_F5} =$	2.32	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)
$M_{n_F5.1} =$	9800	kip-in	$=R_{pg} * F_y * S_{xc}$	(F5-1)

8.5.10. Lateral-torsional buckling (F5.2)

$L_{p_F5} =$	61.6	in	$=1.1 * r_{t_F5} * \sqrt{E/F_y}$	(F4-7)
$L_{r_F5} =$	210	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	$=IF(L_b \leq L_{p_F5}, "NA", IF(L_b > L_{r_F5}, F_{cr_F5.4}, F_{cr_F5.3}))$	
$M_{n_F5.2} =$	9800	kip-in	$=IF(L_b \leq L_{p_F5}, M_p, R_{pg} * F_{cr_LTB_F5} * S_{xc})$	(F5-2)

8.5.11. Compression flange local buckling (F5.3)

$\lambda_f =$	5.86		From Section 3 above					
$\lambda_{fp} =$	9.15		From Section 3 above					
$\lambda_{fr} =$	24.08		From Section 3 above		Weak Axis Bending:			
$F_{cr_F5.8} =$	53	ksi	$=F_y - 0.3 * F_y * (\lambda_f - \lambda_{fp}) / (\lambda_{fr} - \lambda_{fp})$	(F5-8)	$F_{cr_F6.4} =$	583	ksi	(F6-4)
$F_{cr_F5.9} =$	439	ksi	$=0.9 * E * k_c / (b_f / (2 * t_f))^2$	(F5-9)	$M_{n_F6.2} =$	1828	k-in	(F6-2)
$F_{cr_F5} =$	50	ksi	$=if(Flange="Compact", 50, if(Flange="Non-Compact", F_{cr_F5.8}, F_{cr_F5.9}))$					
$M_{n_F5.3} =$	9800	kip-in	$=R_{pg} * F_{cr_F5} * S_{xc}$	(F5-7)	$M_{n_F6.4} =$	12191	k-in	(F6-3)

8.5.12. Tension flange yielding (F5.4)

$M_{n_F5.4} =$	9800	kip-in	$=M_p$	Both flanges are symmetric	(F5-10)
-----------------	------	--------	--------	----------------------------	---------

8.5.13. Flange hole reduction (F13.1)

$D_{HoleT} =$	0.000	in		
$A_{fg} =$	6.95	in ²	$=b_f * t_f$	
$A_{fn} =$	6.95	in ²	$=t_f * (b_f - 2 * D_{HoleT})$	
$Y_t =$	1.00		$=IF(F_y/F_u \leq 0.8, 1, 1.1)$	
$M_{n_F13.1} =$	12740	kip-in	$=F_u * A_{fn} / A_{fg} * S_x$	(F13-1)

Flexural Capacity Summary

$M_{n_F2} =$	9800	kip-in	$=\min(M_{n_F2.1}, M_{n_F2.2})$	Weak Axis Bending:		
$M_{n_F3} =$	9800	kip-in	$=\min(M_{n_F3.1}, M_{n_F3.2})$	$M_{ny} =$	1630	kip-in
$M_{n_F4} =$	9800	kip-in	$=\min(M_{n_F4.1}, M_{n_F4.2}, M_{n_F4.3}, M_{n_F4.4})$			
$M_{n_F5} =$	9800	kip-in	$=\min(M_{n_F5.1}, M_{n_F5.2}, M_{n_F5.3}, M_{n_F5.4})$			
$M_n =$	9800	kip-in	$IF(B3="Beam", IF(DesignSection="F2", \min(M_{n_F2}, M_{n_F13.1}), IF(DesignSection="F3", \min(M_{n_F3}, M_{n_F13.1}), IF(DesignSection="F4", \min(M_{n_F4}, M_{n_F13.1}), \min(M_{n_F5}, M_{n_F13.1}))), M_{n_F4})$			
$\Phi M_n =$	8820	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)

$K_x = 1$	$K_y = 1$	$K_z = 1$
$L_b = 147.2$ in	$L_y = 147$ in	$L_z = 147$ in
$K_{LR} = 75.49$	$= \max(K_x * L_b / r_x, K_y * L_b / r_y)$	
$F_e = 50.23$ ksi	$= \pi^2 * E / (K_{LR})^2$ (E3-4)	
$F_{cr_E3} = 32.96$ ksi	$= \text{IF}(K_{LR} \leq 4.71 * \text{SQRT}(E/F_y), F_y * 0.658^{(F_y/F_e)}, 0.877 * F_e)$ (E3-2, 3-3)	
$P_{n_E3} = 814.18$ kip	$= F_{cr_E3} * A_g$ (E3-1)	

8.6.2. E.4 Torsional and flexural-torsional buckling: (Applies to singly symmetric shapes)

$F_{ey} = 50.23$ ksi	$= \pi^2 * E / (K_y * L_y / r_y)^2$	(E3-4)
$F_{ez} = 85.42$ ksi	$= \pi^2 * E * C_w / (K_z * L_z)^2 + G * J) / (1 / (I_x + I_y))$	(E4-2)
$F_{e_E4} = 50.23$ ksi	$= (F_{ey} + F_{ez}) / (2 * H_z) * (1 - \text{sqrt}(1 - 4 * F_{ey} * F_{ez} * H_z / (F_{ey} + F_{ez})^2))$	(E4-3)
$F_{cr_E4} = 32.96$ ksi	$= \text{IF}(K_{LR} \leq 4.71 * \text{SQRT}(E/F_y), F_y * 0.658^{(F_y/F_{e_E4})}, 0.877 * F_{e_E4})$	(E3-2, 3-3)
$P_{n_E4} = 814.2$ kip	$= A_g * F_{cr_E4}$	(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

$\lambda = 5.86$	From Section 3 above
$\lambda_f = 13.49$	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
$F_{el} = 588.5$ ksi	$= (c2 * \lambda_{rw} / \lambda_w)^2 * F_y$
$F_{cr} = 33.0$ ksi	$= \text{Min}(F_{cr_E3}, F_{cr_E4})$
$bf = 9.02$ in	= Full Flange Width from AISC Database
$b_{e_E7-3} = 2.7$ in	$= bf * (1 - c1 * \text{sqrt}(F_{el}/F_{cr})) * \text{sqrt}(F_{el}/F_{cr})$, but can't be more than bf
$be = 9.02$ in	$= \text{If}(\text{Flange} = "Nonslender", b_f, b_{e_E7-3})$
$A_{web} = 10.81$ in ²	$= A_g - 2 * t_f * bf$
$A_f = 13.89$ in ²	$= 2 * t_f * be$
$A_{e1} = 24.7$ in ²	$= A_{web} + A_f$

8.6.2b Calculation for effective web depth

Web= Slender

$\lambda_w = 48.00$	From Section 4.1 above
$\lambda_{rw} = 35.88$	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
$F_{el} = 48.0$ ksi	$= (c2 * \lambda_{rw} / \lambda_w)^2 * F_y$
$h = 22.6$ in	
$h_{e_E7-3} = 21.3$ in	$= h * (1 - c1 * \text{sqrt}(F_{el}/F_{cr})) * \text{sqrt}(F_{el}/F_{cr})$, but can't be more than h
$h_e = 21.3$ in	$= \text{If}(\text{Web} = "Nonslender", h, h_{e_E7-3})$
$A_{flange} = 13.9$ in ²	$= 2 * bf * t_f$
$A_{web_radius} = 10.8$ in ²	$= A_g - A_{flange}$
$A_{web} = 10.6$ in ²	$= h * t_w$
$A_{radius} = 0.2$ in ²	$= A_{web_radius} - A_{web}$
$A_{e_web} = 10.01$ in ²	$= h_e * t_w$
$A_{e2} = 24.1$ in ²	$= A_{flange} + A_{radius} + A_{e_web}$
$A_e = 24.1$ in ²	$= \text{Min}(A_{e1}, A_{e2})$
$F_{cr} = 33.0$ ksi	
$P_{n_E7} = 794.7$ kips	
$P_n = 794.7$ kip	$= \text{IF}(\text{AxialDesignSection} = "E3", P_{n_E3}, P_{n_E7})$
$\Phi_c * P_n = 715.2$ kip	

8.7. Demand per AISC 360-16 Appendix 8 (to include p-small delta)

$$\begin{aligned}
 P_y &= 1235 \text{ kip} & = A_g * F_y \\
 P_r &= 75 \text{ kip} & = P_{u_omega} \\
 \alpha &= 1 & 1 \text{ for LRFD, 1.6 for ASD} \\
 C_m &= 1 & \text{conservative assumption} \\
 P_{e1y} &= 31306 \text{ kip} & = \pi^2 * E * I_x / (L_b)^2 \\
 B_1 &= 1.002 & = C_m / (1 - \alpha * P_r / P_{e1y})
 \end{aligned}$$

(A-8-5)

(A-8-3)

8.8 Axial + flexural interaction (Chapter H)

$$\begin{aligned}
 M_r &= 4102.3 \text{ kip-in} & = M_u * B_1 \\
 M_c &= 8820 \text{ kip-in} & = \Phi * M_n \\
 P_r &= 75.1 \text{ kip} & = P_u \\
 P_c &= 715.2 \text{ kip} & = \Phi_c * P_n \\
 DCR_Axial &= 0.105 & = P_r / P_c \\
 DCR_Flexural &= 0.465 & = M_r / M_c \\
 DCR_P+M &= 0.518 & = \text{IF}(P_r / P_c \geq 0.2, P_r / P_c + 8/9 * M_r / M_c, P_r / (2 * P_c) + M_r / M_c)
 \end{aligned}$$

(H1-1)

8.9. Shear capacity (Chapter G)

$$\begin{aligned}
 \text{Hole}_{web} &= 0 \text{ in} \\
 V_u &= 30.19 \text{ kips} \\
 A_w &= 11.33 \text{ in}^2 & = \text{IF}(\text{ColOrient} = "Strong", t_w * (d - \text{Hole}_{web}), 2 * (b_f * t_f)) \\
 h/t_w &= 48 \\
 k_v &= 5.34 & \text{Webs without transverse stiffeners} \\
 2.24 \sqrt{E/F_y} &= 53.9 \\
 C_{v1_G2-2} &= 1 & (G2-2) \\
 1.1 \sqrt{k_v E/F_y} &= 61.2 \\
 C_{v1_G2-3} &= 1.00 & (G2-3) \\
 C_{v1_G2-4} &= 1.3 & = 1.1 * \sqrt{k_v * E/F_y} / (h/t_w) & (G2-4) \\
 C_{v1_G2-3_4?} &= 1.0 & = \text{if}(h/t_w \leq 1.1 * \sqrt{k_v * E/F_y}, C_{v1_G2-3}, C_{v1_G2-4}) \\
 C_{v1} &= 1.0 & = \text{if}(C_{v1_G2-2} = "N/A", C_{v1_G2-3_4?}) \\
 V_n &= 339.8 \text{ kip} & = 0.6 * F_y * A_w * C_{v1} & (G2-1) \\
 DCR_Shear &= 0.089 & = V_u / (\Phi_v * V_n)
 \end{aligned}$$

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	0.105	0.465	0.518	0.089

8. AISC 360-16 Steel Column Design

Column ID: **3**
LC: **8**
Column Size= **W24X84** W-Section **L_b= 187** in
Column Orientation= **Strong** Col Beam Bot Bracing? **NO**

8.1. Loading

$M_u = 3511$ kip-in $P_u = 32.9$ kip $V_u = 22.8$ kip
 $M_{max} = 3511$ kip-in $M_{u_top} = 3511$ kip-in **TopGov** $M_{u_bot} = 688$ kip-in
 $M_A = 417$ kip-in $M_B = 1522.4$ kip-in $M_C = 2627.4$ kip-in

8.2. Material properties

$F_y = 50$ ksi $E = 29000$ ksi $A_g = 24.700$ in²
 $F_u = 65$ ksi $G = 11200$ ksi $Z_x = 224.000$ in³
 $bf = 9.02$ in $r_x = 9.790$ in $Z_y = 32.600$ in³
 $d = 24.1$ in $r_y = 1.95$ in $S_x = 196.000$ in³
 $k_{des} = 1.27$ in $t_f = t_{fi} = 0.77$ in $S_{xc} = 196.00$ in³
 $I_x = 2370.0$ in⁴ $t_{fe} = 0.77$ in $S_{xt} = 196.00$ in³
 $I_y = 94.4$ in⁴ $t_w = 0.47$ in $H_z = 1.0$
 $J = 3.700$ in⁴ $C_w = 12800$ in⁶ $S_y = 20.900$ in³
 $h = 22.56$ in $= d - 2 \cdot t_f$
 $h_c = 22.76$ in $= 2 \cdot (d - t_{fi} - (b_f \cdot t_{fi}^2 / 2 + t_w \cdot (d - t_{fe} - t_{fi}) \cdot (d - t_{fe} + t_{fi}) / 2 + b_f \cdot t_{fe} \cdot (d - t_{fe} / 2)) / A_g)$
 $h_p = 24.10$ in $= 2 \cdot (d - (b_f \cdot (t_{fe} - t_{fi}) / 2 + t_w \cdot (d + t_{fi} - t_{fe})) / (2 \cdot t_w))$

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

$k_c = 0.58$ $= \max(0.35, \min(4 / \sqrt{h / t_w}, 0.76))$ For built-up sections
 $\lambda = 5.86$ $b_f / (2 \cdot t_f)$
 $\lambda_p = 9.15$ $= 0.38 \cdot \sqrt{E / F_y}$
 $F_L = 35.00$ $= \text{IF}(S_{xt} / S_{xc} > 0.7, 0.7 \cdot F_y, \max(F_y \cdot S_{xt} / S_{xc}, 0.5 \cdot F_y))$
 $\lambda_{r_case10} = 24.08$ $= 1 \cdot \sqrt{E / F_y}$ Case 10-W-Sections
 $\lambda_{r_case11} = 20.78$ $= 0.95 \cdot \sqrt{K_c \cdot E / F_L}$ Case 11 BU-Sections
 $\lambda_r = 24.08$
Flange: **Compact** $= \text{IF}(\lambda < \lambda_p, "Compact", \text{IF}(\text{AND}(\lambda < \lambda_r, \lambda > \lambda_p), "Non-Compact", "Slender"))$

8.3.1.a Web compactness check flexure (Table B4.1b: Case 15):

$\lambda = 48.00$ $= h / t_w$
 $\lambda_p = 90.55$ $= 3.76 \cdot \sqrt{E / F_y}$
 $\lambda_r = 137.27$ $= 5.70 \cdot \sqrt{E / F_y}$
Web: **Compact** $= \text{IF}(\lambda < \lambda_p, "Compact", \text{IF}(\text{AND}(\lambda < \lambda_r, \lambda > \lambda_p), "Non-Compact", "Slender"))$

Per Table F1.1 for Flexure

Section: **F2**
 $\Phi_b = 0.9$ $= \text{IF}(\text{AND}(\text{Flange} = "Compact", \text{Web} = "Compact"), "F2", \text{IF}(\text{AND}(\text{OR}(\text{Flange} = "Non-Compact", \text{Flange} = "Slender"), \text{Web} = "Compact"), "F3", \text{IF}(\text{AND}(\text{OR}(\text{Flange} = "Compact", \text{Flange} = "Non-Compact", \text{Flange} = "Slender"), \text{Web} = "Slender"), "F5", "F4"))))$
 $\Phi_c = 0.9$
 $\Phi_v = 1$

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

$\lambda = 5.86$ $b_f / (2 \cdot t_f)$
 $\lambda_{r_case1} = 13.49$ $= 0.56 \cdot \sqrt{E / F_y}$
 $k_c = 0.58$ From Section 3
 $\lambda_{r_case2} = 11.71$ $= 0.64 \cdot \sqrt{K_c \cdot E / F_y}$
 $\lambda_r = 13.49$
Flange: **Nonslender**

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

$\lambda = 48.00$ $= h / t_w$
 $\lambda_r = 35.88$ $= 1.49 \cdot \sqrt{E / F_y}$
Web: **Slender**

8.5. Flexural Capacity

8.5.1. Yielding (F2.1)

$$M_{n_{F2.1}} = 9800 \text{ kip-in}$$

$$M_y = F_y * S_x \text{ (Col not braced at } B_m \text{ bottom)}$$

Weak Axis Bending:

$$M_{n_{y_{F2.1}}} = 1630 \text{ kip-in}$$

$$= M_{py} = F_y * Z_y \text{ (F6-1)}$$

8.5.2. Lateral-torsional buckling (F2.2)

$$L_p = 82.65 \text{ in}$$

$$= 1.76 * r_y * \sqrt{E/F_y}$$

$$\text{(F2-5)}$$

$$r_{ts} = 2.37 \text{ in}$$

$$= \sqrt{\sqrt{I_y * C_w} / S_x}$$

$$\text{(F2-7)}$$

$$h_o = 23.330 \text{ in}$$

$$= d - t_f$$

Distance between centroids

$$c = 1$$

1 for doubly symmetric I-Shapes

$$\text{(EQ F2-8a)}$$

$$L_r = 243.50 \text{ in}$$

$$= 1.95 * r_{ts} * E / (0.7 * F_y) * \sqrt{(J * c) / (S_x * h_o)} * \sqrt{1 + \sqrt{1 + 6.76 * (0.7 * F_y / E * S_x * h_o / (J * c))^2}}$$

$$\text{(F2-6)}$$

$$C_{b_calculated} = 1.83$$

$$= 12.5 * M_{max} / (2.5 * M_{max} + 3 * M_A + 4 * M_B + 3 * M_C)$$

$$C_b = 1.83$$

$$= IF(C_b = C_{b_calculated}, C_{b_calculated}, C_b)$$

$$\text{(F1-1)}$$

$$F_{cr} = 99.3 \text{ ksi}$$

$$= C_b * \pi^2 * E / (L_b / r_{ts})^2 * \sqrt{1 + 0.078 * J_{col} * C_{col} / (S_x * h_o) * (L_b / r_{ts})^2}$$

$$\text{(F2-4)}$$

$$M_{n_{F2.2}} = 9800 \text{ kip-in}$$

$$= IF(L_b \leq L_p, M_p, IF(AND(L_p < L_b, L_b \leq L_r), MIN(C_b * (M_p - (M_p - 0.7 * F_y * S_x) * ((L_b - L_p) / (L_r - L_p))), M_p), MIN(F_{cr} * S_x, M_p)))$$

$$\text{(F2-2, 2-3)}$$

8.5.3. Lateral torsional buckling (F3.1)

$$M_{n_{F3.1}} = 9800$$

$$= M_{n_{F2.2}}$$

8.5.4. Compression flange local buckling (F3.2)

$$K_c = 0.58$$

$$= \min(\max(0.35, 4 / \sqrt{h/t_w}), 0.76)$$

$$\lambda = 5.857$$

$$= b_f / (2 * t_f)$$

$$\lambda_{pf} = 9.15$$

$$= 0.38 * \sqrt{E/F_y}$$

$$\lambda_{rf} = 24.1$$

$$= 1.0 * \sqrt{E/F_y}$$

$$M_{n_{F3.2}} = 9800$$

$$= IF(\text{Flange} = \text{"Non-Compact"}, M_p - (M_p - 0.7 * F_y * S_x) * (\lambda - \lambda_{pf}) / (\lambda_{rf} - \lambda_{pf}), IF(\text{Flange} = \text{"Slender"}, 0.9 * E * K_c * S_x / \lambda^2, M_p))$$

$$\text{(F3-1, 3-2)}$$

8.5.5. Compression flange yielding (F4.1)

$$h_c = 22.56 \text{ in}$$

$$= d - 2 * t_f$$

$$\lambda_{w_{F4}} = 48.00$$

$$= h_o / t_w$$

$$\lambda_{pw} = 90.55$$

$$= 3.76 * \sqrt{E/F_y}$$

$$\lambda_{rw} = 137.27$$

$$= 5.7 * \sqrt{E/F_y}$$

$$M_{yc} = 9800 \text{ kip-in}$$

$$= F_y * S_{xc}$$

$$R_{pc} = 1.00$$

$$= IF(h_c/t_w \leq \lambda_{pw}, M_p / M_{yc}, MIN(M_p / M_{yc} - (M_p / M_{yc} - 1) * (\lambda - \lambda_{pw}) / (\lambda_{rw} - \lambda_{pw}), M_p / M_{yc}))$$

$$M_{n_{F4.1}} = 9800$$

$$= R_{pc} * M_{yc}$$

$$\text{(F4-1)}$$

8.5.6. Lateral torsional buckling (F4.2)

$$a_{w_{F42}} = 1.53$$

$$= h_c * t_w / (b_f * t_f)$$

$$\text{(F4-12)}$$

$$r_{t_{F4}} = 2.32 \text{ in}$$

$$= b_f / \sqrt{12 * (1 + 1/6 * a_{w_{F42}})}$$

$$\text{(F4-11)}$$

$$L_{p_{F4}} = 61.59$$

$$= 1.1 * r_{t_{F4}} * \sqrt{E/F_y}$$

$$\text{(F4-7)}$$

$$L_{r_{F4}} = 239.04 \text{ in}$$

$$= 1.95 * r_{t_{F4}} * E / F_{L} * \sqrt{J / (S_{xc} * h_o)} + \sqrt{J / (S_{xc} * h_o)^2 + 6.76 * (F_L / E)^2}$$

$$\text{(F4-8)}$$

$$F_{cr_{F4}} = 96.18 \text{ ksi}$$

$$= C_b * \pi^2 * E / (L_b / r_{t_{F4}})^2 * \sqrt{1 + 0.078 * J / (S_{xc} * h_o) * (L_b / r_{t_{F4}})^2}$$

$$\text{(F4-5)}$$

$$M_{n_{F4.2}} = 9800 \text{ kip-in}$$

$$= IF(L_b \leq L_p, M_p, IF(L_b > L_r, F_{cr_{F4}} * S_{xc}, MIN(C_b * (R_{pc} * M_{yc} - (R_{pc} * M_{yc} - F_L * S_{xc}) * (L_b - L_p) / (L_r - L_p)), R_{pc} * M_{yc})))$$

$$\text{(F4-2, 4-3)}$$

8.5.7. Compression flange local buckling (F4.3)

$$\lambda_{f_{F43}} = 5.86$$

$$= b_f / (2 * t_f)$$

$$\lambda_{pf} = 9.15$$

From Section 3 above

$$\lambda_{rf} = 24.08$$

From Section 3 above

$$M_{n_{F13}} = 10449$$

$$= R_{pc} * M_{yc} - (R_{pc} * M_{yc} - F_L * S_{xc}) * (\lambda_{f_{F43}} - \lambda_{pf}) / (\lambda_{rf} - \lambda_{pf})$$

$$\text{(F4-13)}$$

$$M_{n_{F14}} = 86092$$

$$0.9 * E * K_c * S_{xc} / \lambda_{f_{F432}}^2$$

$$\text{(F4-14)}$$

$$M_{n_{F43}} = 9800 \text{ kip-in}$$

$$= IF(\text{Flange} = \text{"Compact"}, M_p, IF(\text{Flange} = \text{"Non-Compact"}, M_{n_{F13}}, M_{n_{F14}}))$$

8.5.8. Tension flange yielding (F4.4)

$\lambda_{F44} =$	48.00		$= h_c / t_w$	
$M_{yt} =$	9800	kip-in	$= F_y * S_{xt}$	(F4-16a)
$\lambda_{pw} =$	91		From Section 3 above	
$\lambda_{rw} =$	137		From Section 3 above	
$R_{pt_F4-16a} =$	1.00		$= M_p / M_{yt}$	(F4-16b)
$R_{pt_F4-16b} =$	1.00		$= M_p / M_{yt} * (M_p / M_{yt} - 1) * (\lambda_{F44} - \lambda_{pw}) / (\lambda_{rw} - \lambda_{pw})$	
$R_{pt} =$	1.00		$= IF(\lambda_{F44} \leq \lambda_{pw}, R_{pt_F4-16a}, R_{pt_F4-16b})$	
$M_{n_F4.4} =$	9800	kip-in	$= R_{pt} * M_{yt}$	(F4-15)

8.5.9. Compression flange yielding (F5.1)

$a_w =$	1.53		$= \min(10, a_{w_F42})$	
$r_{t_F5} =$	2.32	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)
$M_{n_F5.1} =$	9800	kip-in	$= R_{pg} * F_y * S_{xc}$	(F5-1)

8.5.10. Lateral-torsional buckling (F5.2)

$L_{p_F5} =$	61.6	in	$= 1.1 * r_{t_F5} * \sqrt{E / F_y}$	(F4-7)
$L_{r_F5} =$	210	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	$= IF(L_b \leq L_{p_F5}, "NA", IF(L_b > L_{r_F5}, F_{cr_F5.4}, F_{cr_F5.3}))$	
$M_{n_F5.2} =$	9800	kip-in	$= IF(L_b \leq L_{p_F5}, M_p, R_{pg} * F_{cr_LTB_F5} * S_{xc})$	(F5-2)

8.5.11. Compression flange local buckling (F5.3)

$\lambda_f =$	5.86		From Section 3 above					
$\lambda_{fp} =$	9.15		From Section 3 above					
$\lambda_{fr} =$	24.08		From Section 3 above		Weak Axis Bending:			
$F_{cr_F5.8} =$	53	ksi	$= F_y - 0.3 * F_y * (\lambda_f - \lambda_{fp}) / (\lambda_{fr} - \lambda_{fp})$	(F5-8)	$F_{cr_F6.4} =$	583	ksi	(F6-4)
$F_{cr_F5.9} =$	439	ksi	$= 0.9 * E * k_c / (b_f / (2 * t_f))^2$	(F5-9)	$M_{n_F6.2} =$	1828	k-in	(F6-2)
$F_{cr_F5} =$	50	ksi	$= \text{if}(\text{Flange} = "Compact", 50, \text{if}(\text{Flange} = "Non-Compact", F_{cr_F5.8}, F_{cr_F5.9}))$					
$M_{n_F5.3} =$	9800	kip-in	$= R_{pg} * F_{cr_F5} * S_{xc}$	(F5-7)	$M_{n_F6.4} =$	12191	k-in	(F6-3)

8.5.12. Tension flange yielding (F5.4)

$M_{n_F5.4} =$	9800	kip-in	$= M_p$	Both flanges are symmetric	(F5-10)
-----------------	------	--------	---------	----------------------------	---------

8.5.13. Flange hole reduction (F13.1)

$D_{HoleT} =$	0.000	in		
$A_{fg} =$	6.95	in ²	$= b_f * t_f$	
$A_{fn} =$	6.95	in ²	$= t_f * (b_f - 2 * D_{HoleT})$	
$Y_t =$	1.00		$= IF(F_y / F_u \leq 0.8, 1, 1.1)$	
$M_{n_F13.1} =$	12740	kip-in	$= F_u * A_{fn} / A_{fg} * S_x$	(F13-1)

Flexural Capacity Summary

$M_{n_F2} =$	9800	kip-in	$= \min(M_{n_F2.1}, M_{n_F2.2})$	Weak Axis Bending:		
$M_{n_F3} =$	9800	kip-in	$= \min(M_{n_F3.1}, M_{n_F3.2})$	$M_{ny} =$	1630	kip-in
$M_{n_F4} =$	9800	kip-in	$= \min(M_{n_F4.1}, M_{n_F4.2}, M_{n_F4.3}, M_{n_F4.4})$			
$M_{n_F5} =$	9800	kip-in	$= \min(M_{n_F5.1}, M_{n_F5.2}, M_{n_F5.3}, M_{n_F5.4})$			
$M_n =$	9800	kip-in	$= IF(B3 = "Beam", IF(\text{DesignSection} = "F2", \min(M_{n_F2}, M_{n_F13.1}), IF(\text{DesignSection} = "F3", \min(M_{n_F3}, M_{n_F13.1}), IF(\text{DesignSection} = "F4", \min(M_{n_F4}, M_{n_F13.1}), \min(M_{n_F5}, M_{n_F13.1}))))), M_{n_F4})$			
$\Phi M_n =$	8820	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)

$K_x = 1$	$K_y = 1$	$K_z = 1$
$L_b = 186.8$ in	$L_y = 187$ in	$L_z = 187$ in
$K_{LR} = 95.79$	$=\max(K_x * L_b / r_x, K_y * L_b / r_y)$	
$F_e = 31.19$ ksi	$=\pi^2 * E / (K_{LR})^2$ (E3-4)	
$F_{cr_E.3} = 25.56$ ksi	$=IF(K_{LR} \leq 4.71 * \text{SQRT}(E/F_y), F_y * 0.658^{(F_y/F_e)}, 0.877 * F_e)$ (E3-2, 3-3)	
$P_{n_E3} = 631.35$ kip	$=F_{cr_E.3} * A_g$ (E3-1)	

8.6.2. E.4 Torsional and flexural-torsional buckling: (Applies to singly symmetric shapes)

$F_{ey} = 31.19$ ksi	$=\pi^2 * E / (K_y * L_y / r_y)^2$ (E3-4)
$F_{ez} = 59.42$ ksi	$=\pi^2 * E * C_w / (K_z * L_z)^2 + G * J) / (1 / (I_x + I_y))$ (E4-2)
$F_{e_E.4} = 31.19$ ksi	$=(F_{ey} + F_{ez}) / (2 * H_z) * (1 - \text{sqrt}(1 - 4 * F_{ey} * F_{ez} * H_z / (F_{ey} + F_{ez})^2))$ (E4-3)
$F_{cr_E.4} = 25.56$ ksi	$=IF(K_{LR} \leq 4.71 * \text{SQRT}(E/F_y), F_y * 0.658^{(F_y/F_{e_E.4})}, 0.877 * F_{e_E.4})$ (E3-2, 3-3)
$P_{n_E.4} = 631.3$ 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

$\lambda = 5.86$	From Section 3 above
$\lambda_f = 13.49$	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
$F_{el} = 588.5$ ksi	$=(c2 * \lambda_{rw} / \lambda_w)^2 * F_y$
$F_{cr} = 25.6$ ksi	$=\text{Min}(F_{cr_E3}, F_{cr_E4})$
$bf = 9.02$ in	=Full Flange Width from AISC Database
$b_{e_E7-3} = -2.4$ in	$=bf * (1 - c1 * \text{sqrt}(F_{el} / F_{cr})) * \text{sqrt}(F_{el} / F_{cr})$, but can't be more than bf (E7-3)
$be = 9.02$ in	$=If(\text{Flange} = "Nonslender", b_f, b_{e_E7-3})$
$A_{web} = 10.81$ in ²	$=A_g - 2 * t_f * bf$
$A_f = 13.89$ in ²	$=2 * t_f * be$
$A_{e1} = 24.7$ in ²	$=A_{web} + A_f$

8.6.2b Calculation for effective web depth

Web= Slender

$\lambda_w = 48.00$	From Section 4.1 above
$\lambda_{rw} = 35.88$	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
$F_{el} = 48.0$ ksi	$=(c2 * \lambda_{rw} / \lambda_w)^2 * F_y$
$h = 22.6$ in	
$h_{e_E7-3} = 22.6$ in	$=h * (1 - c1 * \text{sqrt}(F_{el} / F_{cr})) * \text{sqrt}(F_{el} / F_{cr})$, but can't be more than h (E7-3)
$h_e = 22.6$ in	$=If(\text{Web} = "Nonslender", h, h_{e_E7-3})$
$A_{flange} = 13.9$ in ²	$=2 * bf * t_f$
$A_{web_radius} = 10.8$ in ²	$=A_g - A_{flange}$
$A_{web} = 10.6$ in ²	$=h * t_w$
$A_{radius} = 0.2$ in ²	$=A_{web_radius} - A_{Web}$
$A_{e_web} = 10.60$ in ²	$=h_e * t_w$
$A_{e2} = 24.7$ in ²	$=A_{flange} + A_{radius} + A_{e_web}$
$A_e = 24.7$ in ²	$=\text{Min}(A_{e1}, A_{e2})$
$F_{cr} = 25.6$ ksi	
$P_{n_E7} = 631.3$ kips	
$P_n = 631.3$ kip	$=IF(\text{AxialDesignSection} = "E3", P_{n_E3}, P_{n_E7})$
$\Phi_c * P_n = 568.2$ kip	

8.7. Demand per AISC 360-16 Appendix 8 (to include p-small delta)

$P_y =$	1235	kip	$= A_g * F_y$	
$P_r =$	33	kip	$= P_{u_omega}$	
$\alpha =$	1		1 for LRFD, 1.6 for ASD	
$C_m =$	1		conservative assumption	(A-8-5)
$P_{e1y} =$	19440	kip	$= \pi^2 * E * I_x / (L_b)^2$	
$B_1 =$	1.002		$= C_m / (1 - \alpha * P_r / P_{e1y})$	(A-8-3)

8.8 Axial + flexural interaction (Chapter H)

$M_r =$	3517.4	kip-in	$= M_u * B_1$	
$M_c =$	8820	kip-in	$= \Phi * M_n$	
$P_r =$	32.9	kip	$= P_u$	
$P_c =$	568.2	kip	$= \Phi_c * P_n$	
DCR_Axial=	0.058		$= P_r / P_c$	
DCR_Flexural=	0.399		$= M_r / M_c$	
DCR_P+M=	0.428		$= \text{IF}(P_r / P_c \geq 0.2, P_r / P_c + 8/9 * M_r / M_c, P_r / (2 * P_c) + M_r / M_c)$	(H1-1)

8.9. Shear capacity (Chapter G)

Hole _{Web} =	0	in		
$V_u =$	22.82	kips		
$A_w =$	11.33	in ²	$= \text{IF}(\text{ColOrient} = \text{"Strong"}, t_w * (d - \text{Hole}_{web}), 2 * (b_f * t_f))$	
$h/t_w =$	48			
$k_v =$	5.34		Webs without transverse stiffeners	
$2.24 \sqrt{E/F_y} =$	53.9			
$C_{v1_G2-2} =$	1			(G2-2)
$1.1 \sqrt{k_v E/F_y} =$	61.2			
$C_{v1_G2-3} =$	1.00			(G2-3)
$C_{v1_G2-4} =$	1.3		$= 1.1 * \sqrt{k_v * E/F_y} / (h/t_w)$	(G2-4)
$C_{v1_G2-3_4?} =$	1.0		$= \text{if}(h/t_w \leq 1.1 * \sqrt{k_v * E/F_y}, C_{v1_G2-3}, C_{v1_G2-4})$	
$C_{v1} =$	1.0		$= \text{if}(C_{v1_G2-2} = \text{"N/A"}, C_{v1_G2-3_4?})$	
$V_n =$	339.8	kip	$= 0.6 * F_y * A_w * C_{v1}$	(G2-1)
DCR_Shear=	0.067		$= V_u / (\Phi_v * V_n)$	

8.10. Design Summary:

		Demand-to-Capacity Ratios			
LC ID	Load Combination	P	M	P+M	V
LC 8	(1.2 + 0.2 SDS)DL -OmegaEL + f1*LL + f2*SL	0.058	0.399	0.428	0.067

8. AISC 360-16 Steel Column Design

Column ID: 4
LC: 7
Column Size= W24X84
Column Orientation= Strong
W-Section
Col Beam Bot Bracing? NO
L_b= 187 in

8.1. Loading

M_u= 3512 kip-in
M_{max}= 3512 kip-in
M_A= 417 kip-in
P_u= 32.9 kip
M_{u_top}= 3512 kip-in
M_B= 1522.4 kip-in
V_u= 22.8 kip
M_{u_bot}= 688 kip-in
M_C= 2627.5 kip-in
TopGov

8.2. Material properties

F_y= 50 ksi
F_u= 65 ksi
b_f= 9.02 in
d= 24.1 in
k_{des}= 1.27 in
I_x= 2370.0 in⁴
I_y= 94.4 in⁴
J= 3.700 in⁴
h= 22.56 in
h_c= 22.76 in
h_p= 24.10 in
E= 29000 ksi
G= 11200 ksi
r_x= 9.790 in
r_y= 1.95 in
t_f=t_{fi}= 0.77 in
t_{fe}= 0.77 in
t_w= 0.47 in
C_w= 12800 in⁶
A_g= 24.700 in²
Z_x= 224.000 in³
Z_y= 32.600 in³
S_x= 196.000 in³
S_{xc}= 196.00 in³
S_{xt}= 196.00 in³
H_z= 1.0
S_y= 20.900 in³
=d-2*t_f
=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)
=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= 0.58
λ= 5.86
λ_p= 9.15
F_L= 35.00
λ_{r_case10}= 24.08
λ_{r_case11}= 20.78
λ_r= 24.08
Flange: Compact
=max(0.35,min(4/sqrt(h/t_w),0.76)
b_f/(2*t_f)
=0.38*sqrt(E/F_y)
=IF(S_{xt}/S_{xc}>=0.7,0.7*F_y,max(F_y*S_{xt}/S_{xc},0.5*F_y))
=1*sqrt(E/F_y)
=0.95*sqrt(K_c*E/F_L)
=IF(λ<λ_p,"Compact",IF(AND(λ<λ_r,λ>λ_p),"Non-Compact","Slender"))

8.3.1.a Web compactness check flexure (Table B4.1b: Case 15):

λ= 48.00
λ_p= 90.55
λ_r= 137.27
Web: Compact
=h/t_w
=3.76*sqrt(E/F_y)
=5.70*sqrt(E/F_y)
=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
Φ_c= 0.9
Φ_v= 1
=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"))))

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= 5.86
λ_{r_case1}= 13.49
k_c= 0.58
λ_{r_case2}= 11.71
λ_r= 13.49
Flange: Nonslender
b_f/(2*t_f)
=0.56*sqrt(E/F_y)
From Section 3
=0.64*sqrt(K_c*E/F_y)

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

λ= 48.00
λ_r= 35.88
Web: Slender
=h/t_w
=1.49*sqrt(E/F_y)

8.5. Flexural Capacity

8.5.1. Yielding (F2.1)

$$M_{n_{F2.1}} = 9800 \text{ kip-in}$$

$$M_y = F_y * S_x \text{ (Col not braced at } B_m \text{ bottom)}$$

Weak Axis Bending:

$$M_{n_{y_{F2.1}}} = 1630 \text{ kip-in}$$

$$= M_{py} = F_y * Z_y \text{ (F6-1)}$$

8.5.2. Lateral-torsional buckling (F2.2)

$$L_p = 82.65 \text{ in} = 1.76 * r_y * \sqrt{E/F_y} \text{ (F2-5)}$$

$$r_{ts} = 2.37 \text{ in} = \sqrt{\sqrt{I_y * C_w} / S_x} \text{ (F2-7)}$$

$$h_o = 23.330 \text{ in} = d - t_f \text{ Distance between centroids}$$

$$c = 1 \text{ 1 for doubly symmetric I-Shapes (EQ F2-8a)}$$

$$L_r = 243.50 \text{ in} = 1.95 * r_{ts} * E / (0.7 * F_y) * \sqrt{(J * c) / (S_x * h_o)} * \sqrt{1 + \sqrt{1 + 6.76 * (0.7 * F_y / E * S_x * h_o / (J * c))^2}} \text{ (F2-6)}$$

$$C_{b_calculated} = 1.83 = 12.5 * M_{max} / (2.5 * M_{max} + 3 * M_A + 4 * M_B + 3 * M_C) \text{ (F1-1)}$$

$$C_b = 1.83 = IF(C_b = C_{b_calculated}, C_{b_calculated}, C_b) \text{ (F1-1)}$$

$$F_{cr} = 99.3 \text{ ksi} = C_b * \pi^2 * E / (L_b / r_{ts})^2 * \sqrt{1 + 0.078 * J_{col} * C_{col} / (S_x * h_o) * (L_b / r_{ts})^2} \text{ (F2-4)}$$

$$M_{n_{F2.2}} = 9800 \text{ kip-in} = IF(L_b \leq L_p, M_p, IF(AND(L_p < L_b, L_b \leq L_r), MIN(C_b * (M_p - (M_p - 0.7 * F_y * S_x) * ((L_b - L_p) / (L_r - L_p))), M_p), MIN(F_{cr} * S_x, M_p))) \text{ (F2-2, 2-3)}$$

8.5.3. Lateral torsional buckling (F3.1)

$$M_{n_{F3.1}} = 9800 = M_{n_{F2.2}}$$

8.5.4. Compression flange local buckling (F3.2)

$$K_c = 0.58 = \min(\max(0.35, 4 / \sqrt{h / t_w}), 0.76)$$

$$\lambda = 5.857 = b_f / (2 * t_f)$$

$$\lambda_{pf} = 9.15 = 0.38 * \sqrt{E / F_y}$$

$$\lambda_{rf} = 24.1 = 1.0 * \sqrt{E / F_y}$$

$$M_{n_{F3.2}} = 9800 = IF(Flange = "Non-Compact", M_p - (M_p - 0.7 * F_y * S_x) * (\lambda - \lambda_{pf}) / (\lambda_{rf} - \lambda_{pf}), IF(Flange = "Slender", 0.9 * E * K_c * S_x / \lambda^2, M_p)) \text{ (F3-1, 3-2)}$$

8.5.5. Compression flange yielding (F4.1)

$$h_c = 22.56 \text{ in} = d - 2 * t_f$$

$$\lambda_{w_{F4}} = 48.00 = h_o / t_w$$

$$\lambda_{pw} = 90.55 = 3.76 * \sqrt{E / F_y}$$

$$\lambda_{rw} = 137.27 = 5.7 * \sqrt{E / F_y}$$

$$M_{yc} = 9800 \text{ kip-in} = F_y * S_{xc}$$

$$R_{pc} = 1.00 = IF(h_c / t_w \leq \lambda_{pw}, M_p / M_{yc}, MIN(M_p / M_{yc} - (M_p / M_{yc} - 1) * (\lambda - \lambda_{pw}) / (\lambda_{rw} - \lambda_{pw}), M_p / M_{yc}))$$

$$M_{n_{F4.1}} = 9800 = R_{pc} * M_{yc} \text{ (F4-1)}$$

8.5.6. Lateral torsional buckling (F4.2)

$$a_{w_{F42}} = 1.53 = h_c * t_w / (b_f * t_f) \text{ (F4-12)}$$

$$r_{t_{F4}} = 2.32 \text{ in} = b_f / \sqrt{12 * (1 + 1/6 * a_{w_{F42}})} \text{ (F4-11)}$$

$$L_{p_{F4}} = 61.59 = 1.1 * r_{t_{F4}} * \sqrt{E / F_y} \text{ (F4-7)}$$

$$L_{r_{F4}} = 239.04 \text{ in} = 1.95 * r_{t_{F4}} * E / F_{L} * \sqrt{J / (S_{xc} * h_o)} + \sqrt{(J / S_{xc} * h_o)^2 + 6.76 * (F_L / E)^2} \text{ (F4-8)}$$

$$F_{cr_{F4}} = 96.18 \text{ ksi} = C_b * \pi^2 * E / (L_b / r_{t_{F4}})^2 * \sqrt{1 + 0.078 * J / (S_{xc} * h_o) * (L_b / r_{t_{F4}})^2} \text{ (F4-5)}$$

$$M_{n_{F4.2}} = 9800 \text{ kip-in} = IF(L_b \leq L_p, M_p, IF(L_b > L_r, F_{cr_{F4}} * S_{xc}, MIN(C_b * (R_{pc} * M_{yc} - (R_{pc} * M_{yc} - F_L * S_{xc}) * (L_b - L_p) / (L_r - L_p)), R_{pc} * M_{yc}))) \text{ (F4-2, 4-3)}$$

8.5.7. Compression flange local buckling (F4.3)

$$\lambda_{f_{F43}} = 5.86 = b_f / (2 * t_f)$$

$$\lambda_{pf} = 9.15 \text{ From Section 3 above}$$

$$\lambda_{rf} = 24.08 \text{ From Section 3 above}$$

$$M_{n_{F13}} = 10449 = R_{pc} * M_{yc} - (R_{pc} * M_{yc} - F_L * S_{xc}) * (\lambda_{f_{F43}} - \lambda_{pf}) / (\lambda_{rf} - \lambda_{pf}) \text{ (F4-13)}$$

$$M_{n_{F14}} = 86092 = 0.9 * E * K_c * S_{xc} / \lambda_{f_{F432}}^2 \text{ (F4-14)}$$

$$M_{n_{F43}} = 9800 \text{ kip-in} = IF(Flange = "Compact", M_p, IF(Flange = "Non-Compact", M_{n_{F13}}, M_{n_{F14}}))$$

8.5.8. Tension flange yielding (F4.4)

$\lambda_{F44} =$	48.00		$=h_c/t_w$	
$M_{yt} =$	9800	kip-in	$=F_y * S_{xt}$	(F4-16a)
$\lambda_{pw} =$	91		From Section 3 above	
$\lambda_{rw} =$	137		From Section 3 above	
$R_{pt_F4-16a} =$	1.00		$=M_p/M_{yt}$	(F4-16b)
$R_{pt_F4-16b} =$	1.00		$=M_p/M_{yt} * (M_p/M_{yt} - 1) * (\lambda_{F44} - \lambda_{pw}) / (\lambda_{rw} - \lambda_{pw})$	
$R_{pt} =$	1.00		$=IF(\lambda_{F44} \leq \lambda_{pw}, R_{pt_F4-16a}, R_{pt_F4-16b})$	
$M_{n_F4.4} =$	9800	kip-in	$=R_{pt} * M_{yt}$	(F4-15)

8.5.9. Compression flange yielding (F5.1)

$a_w =$	1.53		$=\min(10, a_{w_F42})$	
$r_{t_F5} =$	2.32	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)
$M_{n_F5.1} =$	9800	kip-in	$=R_{pg} * F_y * S_{xc}$	(F5-1)

8.5.10. Lateral-torsional buckling (F5.2)

$L_{p_F5} =$	61.6	in	$=1.1 * r_{t_F5} * \sqrt{E/F_y}$	(F4-7)
$L_{r_F5} =$	210	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	$=IF(L_b \leq L_{p_F5}, "NA", IF(L_b > L_{r_F5}, F_{cr_F5.4}, F_{cr_F5.3}))$	
$M_{n_F5.2} =$	9800	kip-in	$=IF(L_b \leq L_{p_F5}, M_p, R_{pg} * F_{cr_LTB_F5} * S_{xc})$	(F5-2)

8.5.11. Compression flange local buckling (F5.3)

$\lambda_f =$	5.86		From Section 3 above					
$\lambda_{fp} =$	9.15		From Section 3 above					
$\lambda_{fr} =$	24.08		From Section 3 above		Weak Axis Bending:			
$F_{cr_F5.8} =$	53	ksi	$=F_y - 0.3 * F_y * (\lambda_f - \lambda_{fp}) / (\lambda_{fr} - \lambda_{fp})$	(F5-8)	$F_{cr_F6.4} =$	583	ksi	(F6-4)
$F_{cr_F5.9} =$	439	ksi	$=0.9 * E * k_c / (b_f / (2 * t_f))^2$	(F5-9)	$M_{n_F6.2} =$	1828	k-in	(F6-2)
$F_{cr_F5} =$	50	ksi	$=if(Flange="Compact", 50, if(Flange="Non-Compact", F_{cr_F5.8}, F_{cr_F5.9}))$					
$M_{n_F5.3} =$	9800	kip-in	$=R_{pg} * F_{cr_F5} * S_{xc}$	(F5-7)	$M_{n_F6.4} =$	12191	k-in	(F6-3)

8.5.12. Tension flange yielding (F5.4)

$M_{n_F5.4} =$	9800	kip-in	$=M_p$	Both flanges are symmetric	(F5-10)
-----------------	------	--------	--------	----------------------------	---------

8.5.13. Flange hole reduction (F13.1)

$D_{HoleT} =$	0.000	in		
$A_{fg} =$	6.95	in ²	$=b_f * t_f$	
$A_{fn} =$	6.95	in ²	$=t_f * (b_f - 2 * D_{HoleT})$	
$Y_t =$	1.00		$=IF(F_y/F_u \leq 0.8, 1, 1.1)$	
$M_{n_F13.1} =$	12740	kip-in	$=F_u * A_{fn} / A_{fg} * S_x$	(F13-1)

Flexural Capacity Summary

$M_{n_F2} =$	9800	kip-in	$=\min(M_{n_F2.1}, M_{n_F2.2})$	Weak Axis Bending:		
$M_{n_F3} =$	9800	kip-in	$=\min(M_{n_F3.1}, M_{n_F3.2})$	$M_{ny} =$	1630	kip-in
$M_{n_F4} =$	9800	kip-in	$=\min(M_{n_F4.1}, M_{n_F4.2}, M_{n_F4.3}, M_{n_F4.4})$			
$M_{n_F5} =$	9800	kip-in	$=\min(M_{n_F5.1}, M_{n_F5.2}, M_{n_F5.3}, M_{n_F5.4})$			
$M_n =$	9800	kip-in	$IF(B3="Beam", IF(DesignSection="F2", \min(M_{n_F2}, M_{n_F13.1}), IF(DesignSection="F3", \min(M_{n_F3}, M_{n_F13.1}), IF(DesignSection="F4", \min(M_{n_F4}, M_{n_F13.1}), \min(M_{n_F5}, M_{n_F13.1}))))), M_{n_F4})$			
$\Phi M_n =$	8820	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)

$K_x = 1$	$K_y = 1$	$K_z = 1$
$L_b = 186.8$ in	$L_y = 187$ in	$L_z = 187$ in
$K_{LR} = 95.79$	$= \max(K_x * L_b / r_x, K_y * L_b / r_y)$	
$F_e = 31.19$ ksi	$= \pi^2 * E / (K_{LR})^2$ (E3-4)	
$F_{cr_E3} = 25.56$ ksi	$= \text{IF}(K_{LR} \leq 4.71 * \text{SQRT}(E/F_y), F_y * 0.658^{(F_y/F_e)}, 0.877 * F_e)$ (E3-2, 3-3)	
$P_{n_E3} = 631.35$ kip	$= F_{cr_E3} * A_g$ (E3-1)	

8.6.2. E.4 Torsional and flexural-torsional buckling: (Applies to singly symmetric shapes)

$F_{ey} = 31.19$ ksi	$= \pi^2 * E / (K_y * L_y / r_y)^2$	(E3-4)
$F_{ez} = 59.42$ ksi	$= \pi^2 * E * C_w / (K_z * L_z)^2 + G * J) / (1 / (I_x + I_y))$	(E4-2)
$F_{e_E4} = 31.19$ ksi	$= (F_{ey} + F_{ez}) / (2 * H_z) * (1 - \text{sqrt}(1 - 4 * F_{ey} * F_{ez} * H_z / (F_{ey} + F_{ez})^2))$	(E4-3)
$F_{cr_E4} = 25.56$ ksi	$= \text{IF}(K_{Lr} \leq 4.71 * \text{SQRT}(E/F_y), F_y * 0.658^{(F_y/F_{e_E4})}, 0.877 * F_{e_E4})$	(E3-2, 3-3)
$P_{n_E4} = 631.3$ kip	$= A_g * F_{cr_E4}$	(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

$\lambda = 5.86$	From Section 3 above
$\lambda_f = 13.49$	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
$F_{el} = 588.5$ ksi	$= (c2 * \lambda_{rw} / \lambda_w)^2 * F_y$
$F_{cr} = 25.6$ ksi	$= \text{Min}(F_{cr_E3}, F_{cr_E4})$
$bf = 9.02$ in	= Full Flange Width from AISC Database
$b_{e_E7-3} = -2.4$ in	$= bf * (1 - c1 * \text{sqrt}(F_{el}/F_{cr})) * \text{sqrt}(F_{el}/F_{cr})$, but can't be more than bf
$be = 9.02$ in	$= \text{If}(\text{Flange} = "Nonslender", b_f, b_{e_E7-3})$
$A_{web} = 10.81$ in ²	$= A_g - 2 * t_f * bf$
$A_f = 13.89$ in ²	$= 2 * t_f * be$
$A_{e1} = 24.7$ in ²	$= A_{web} + A_f$

8.6.2b Calculation for effective web depth

Web= Slender

$\lambda_w = 48.00$	From Section 4.1 above
$\lambda_{rw} = 35.88$	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
$F_{el} = 48.0$ ksi	$= (c2 * \lambda_{rw} / \lambda_w)^2 * F_y$
$h = 22.6$ in	
$h_{e_E7-3} = 22.6$ in	$= h * (1 - c1 * \text{sqrt}(F_{el}/F_{cr})) * \text{sqrt}(F_{el}/F_{cr})$, but can't be more than h
$h_e = 22.6$ in	$= \text{If}(\text{Web} = "Nonslender", h, h_{e_E7-3})$
$A_{flange} = 13.9$ in ²	$= 2 * bf * t_f$
$A_{web_radius} = 10.8$ in ²	$= A_g - A_{flange}$
$A_{web} = 10.6$ in ²	$= h * t_w$
$A_{radius} = 0.2$ in ²	$= A_{web_radius} - A_{Web}$
$A_{e_web} = 10.60$ in ²	$= h_e * t_w$
$A_{e2} = 24.7$ in ²	$= A_{flange} + A_{radius} + A_{e_web}$
$A_e = 24.7$ in ²	$= \text{Min}(A_{e1}, A_{e2})$
$F_{cr} = 25.6$ ksi	
$P_{n_E7} = 631.3$ kips	
$P_n = 631.3$ kip	$= \text{IF}(\text{AxialDesignSection} = "E3", P_{n_E3}, P_{n_E7})$
$\Phi_c * P_n = 568.2$ kip	

8.7. Demand per AISC 360-16 Appendix 8 (to include p-small delta)

$$\begin{aligned}
 P_y &= 1235 \text{ kip} & = A_g * F_y \\
 P_r &= 33 \text{ kip} & = P_{u_omega} \\
 \alpha &= 1 & 1 \text{ for LRFD, 1.6 for ASD} \\
 C_m &= 1 & \text{conservative assumption} \\
 P_{e1y} &= 19440 \text{ kip} & = \pi^2 * E * I_x / (L_b)^2 \\
 B_1 &= 1.002 & = C_m / (1 - \alpha * P_r / P_{e1y})
 \end{aligned}$$

(A-8-5)

(A-8-3)

8.8 Axial + flexural interaction (Chapter H)

$$\begin{aligned}
 M_r &= 3517.4 \text{ kip-in} & = M_u * B_1 \\
 M_c &= 8820 \text{ kip-in} & = \Phi * M_n \\
 P_r &= 32.9 \text{ kip} & = P_u \\
 P_c &= 568.2 \text{ kip} & = \Phi_c * P_n \\
 DCR_Axial &= 0.058 & = P_r / P_c \\
 DCR_Flexural &= 0.399 & = M_r / M_c \\
 DCR_P+M &= 0.428 & = \text{IF}(P_r / P_c \geq 0.2, P_r / P_c + 8/9 * M_r / M_c, P_r / (2 * P_c) + M_r / M_c)
 \end{aligned}$$

(H1-1)

8.9. Shear capacity (Chapter G)

$$\begin{aligned}
 \text{Hole}_{web} &= 0 \text{ in} \\
 V_u &= 22.82 \text{ kips} \\
 A_w &= 11.33 \text{ in}^2 & = \text{IF}(\text{ColOrient} = "Strong", t_w * (d - \text{Hole}_{web}), 2 * (b_f * t_f)) \\
 h/t_w &= 48 \\
 k_v &= 5.34 & \text{Webs without transverse stiffeners} \\
 2.24 \sqrt{E/F_y} &= 53.9 \\
 C_{v1_G2-2} &= 1 & (G2-2) \\
 1.1 \sqrt{k_v E/F_y} &= 61.2 \\
 C_{v1_G2-3} &= 1.00 & (G2-3) \\
 C_{v1_G2-4} &= 1.3 & = 1.1 * \sqrt{k_v * E/F_y} / (h/t_w) & (G2-4) \\
 C_{v1_G2-3_4?} &= 1.0 & = \text{if}(h/t_w \leq 1.1 * \sqrt{k_v * E/F_y}, C_{v1_G2-3}, C_{v1_G2-4}) \\
 C_{v1} &= 1.0 & = \text{if}(C_{v1_G2-2} = "N/A", C_{v1_G2-3_4?}) \\
 V_n &= 339.8 \text{ kip} & = 0.6 * F_y * A_w * C_{v1} & (G2-1) \\
 DCR_Shear &= 0.067 & = V_u / (\Phi_v * V_n)
 \end{aligned}$$

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	0.058	0.399	0.428	0.067

7. AISC 360-16 Steel Beam Design

Beam ID: **1001** Beam Size= **W16X67** $L_b = 220.0$ in $R = 3.5$
LC: **7** Beam Type= W-Section Link Size= **YLMC6-3.5**

7.1. Loading

$M_u = 4780$ kip-in $P_u = 9.80$ kip $V_u = 40.92$ kip
 $M_{max} = 4780$ kip-in $M_{cap_link} = 3491$ $M_{cap}/M_u = 0.730$ =1 for wind loads, $R=3$
 $M_A = 2269$ kip-in $M_B = 128$ kip-in $M_C = 2659$ kip-in
Beam Bracing= **None** LeftLink= **YES** RightLink= **YES**

7.2. Material properties

$F_y = 50$ ksi $E = 29000$ ksi $A_g = 19.6$ in²
 $F_u = 65$ ksi $G = 11200$ ksi $Z_x = 130$ in³
 $bf = 10.2$ in $r_x = 6.960$ in $Z_y = 35.5$ in³
 $d = 16.3$ in $r_y = 2.460$ in $S_x = 117.00$ in³
 $k_{des} = 1.07$ in $t_f = t_{fi} = 0.665$ in $S_{xc} = 117$ in³
 $I_x = 954$ in⁴ $t_{fe} = 0.665$ in $S_{xt} = 117$ in³
 $I_y = 119$ in⁴ $t_w = 0.395$ in $H_z = 1.0$
 $J = 2.39$ in⁴ $C_w = 7300.00$ in⁶ $L = 220$ in
 $h = 14.97$ in $= d - 2 \cdot t_{tf}$
 $h_c = 15.07$ in $= 2 \cdot (d - t_{fi} - (b_f \cdot t_{fi}^2 / 2 + t_w \cdot (d - t_{fe} - t_{fi}) \cdot (d - t_{fe} + t_{fi}) / 2 + b_f \cdot t_{fe} \cdot (d - t_{fe} / 2)) / A_g)$
 $h_p = 16.30$ in $= 2 \cdot (d - (b_f \cdot (t_{fe} - t_{fi}) / 2 + t_w \cdot (d + t_{fi} - t_{fe})) / (2 \cdot t_w))$

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

$k_c = 0.65$ $= \max(0.35, \min(4 / \sqrt{h/t_w}, 0.76))$ For built-up sections
 $\lambda = 7.67$ $b_f / (2 \cdot t_f)$
 $\lambda_p = 9.15$ $= 0.38 \cdot \sqrt{E/F_y}$
 $F_t = 35.00$ $= IF(S_{xt}/S_{xc} > 0.7, 0.7 \cdot F_y, \max(F_y \cdot S_{xt}/S_{xc}, 0.5 \cdot F_y))$
 $\lambda_{r_case10} = 24.08$ $= 1 \cdot \sqrt{E/F_y}$ Case 10-W-Sections
 $\lambda_{r_case11} = 22.04$ $= 0.95 \cdot \sqrt{K_c \cdot E/F_t}$ Case 11 BU-Sections
 $\lambda_r = 24.08$
Flange: **Compact** $= IF(\lambda < \lambda_p, "Compact", IF(AND(\lambda < \lambda_r, \lambda > \lambda_p), "Non-Compact", "Slender"))$

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

$\lambda = 37.90$ $= h/t_w$
 $\lambda_p = 90.55$ $= 3.76 \cdot \sqrt{E/F_y}$
 $\lambda_r = 137.27$ $= 5.70 \cdot \sqrt{E/F_y}$
Web: **Compact** $= IF(\lambda < \lambda_p, "Compact", IF(AND(\lambda < \lambda_r, \lambda > \lambda_p), "Non-Compact", "Slender"))$

Per Table F1.1 for Flexure

Section: **F2**
 $\Phi_b = 0.9$ $= IF(AND(Flange="Compact", Web="Compact"), "F2", IF(AND(OR(Flange="Non-Compact", Flange="Slender"), Web="Compact"), "F3", IF(AND(OR(Flange="Compact", Flange="Non-Compact", Flange="Slender"), Web="Slender"), "F5", "F4")))$
 $\Phi_c = 0.9$
 $\Phi_v = 1$

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

$\lambda = 7.67$ $b_f / (2 \cdot t_f)$
 $\lambda_{r_case1} = 13.49$ $= 0.56 \cdot \sqrt{E/F_y}$
 $k_c = 0.65$ From Section 3
 $\lambda_{r_case2} = 12.42$ $= 0.64 \cdot \sqrt{K_c \cdot E/F_y}$
 $\lambda_r = 13.49$
Flange: **Nonslender**

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

$\lambda = 37.90$ $= h/t_w$
 $\lambda_r = 35.88$ $= 1.49 \cdot \sqrt{E/F_y}$
Web: **Slender**

7.5. Flexural Capacity

7.5.1. Yielding (F2.1)

$$M_{n_F2.1} = 6500 \text{ kip-in} = M_p = F_y * Z_x \quad (F2-1)$$

7.5.2. Lateral-torsional buckling (F2.2)

$$\begin{aligned} L_b &= 220 \text{ in} && \text{Lateral torsional bracing length} \\ L_p &= 104.27 \text{ in} && = 1.76 * r_y * \sqrt{E/F_y} \quad (F2-5) \\ r_{ts} &= 2.82 \text{ in} && = \sqrt{\sqrt{I_y * C_w} / S_x} \quad (F2-7) \\ h_o &= 15.635 \text{ in} && = d - t_f \quad \text{Distance between centroids} \\ c &= 1 && 1 \text{ for doubly symmetric I-Shapes} \quad (EQ F2-8a) \\ L_r &= 312.8 \text{ in} && = 1.95 * r_{ts} * E / (0.7 * F_y) * \sqrt{(J * c) / (S_x * h_o)} * \sqrt{1 + \sqrt{1 + 6.76 * (0.7 * F_y / E * S_x * h_o / (J * c))^2}} \quad (F2-6) \end{aligned}$$

$$C_b = 2.19 = 12.5 * M_{max} / (2.5 * M_{max} + 3 * M_A + 4 * M_B + 3 * M_C) \quad (F1-1)$$

$$F_{cr} = 131.5 \text{ ksi} = C_b * \pi^2 * E / (L_b / r_{ts})^2 * \sqrt{1 + 0.078 * J_{col} * C_{col} / (S_x * h_o) * (L_b / r_{ts})^2} \quad (F2-4)$$

$$M_{n_F2.2} = 6500 \text{ kip-in} = \text{IF}(L_b \leq L_p, M_p, \text{IF}(\text{AND}(L_p < L_b, L_b \leq L_r), \text{MIN}(C_b * (M_p - (M_p - 0.7 * F_y * S_x) * ((L_b - L_p) / (L_r - L_p))), M_p), \text{MIN}(F_{cr} * S_x, M_p))) \quad (F2-2, 2-3)$$

7.5.3. Lateral torsional buckling (F3.1)

$$M_{n_F3.1} = 6500 = M_{n_F2.2}$$

7.5.4. Compression flange local buckling (F3.2)

$$k_c = 0.65 = \min(\max(0.35, 4 / \sqrt{h/t_w}), 0.76)$$

$$\lambda = 7.669 = b_f / (2 * t_f)$$

$$\lambda_{pf} = 9.15 = 0.38 * \sqrt{E/F_y}$$

$$\lambda_{rf} = 24.1 = 1.0 * \sqrt{E/F_y}$$

$$M_{n_F3.2} = 6500 = \text{IF}(\text{Flange} = \text{"Non-Compact"}, M_p - (M_p - 0.7 * S_x * F_y) * (\lambda - \lambda_{pf}) / (\lambda_{rf} - \lambda_{pf}), \text{IF}(\text{Flange} = \text{"Slender"}, 0.9 * E * k_c * S_x / \lambda^2, M_p)) \quad (F3-1, 3-2)$$

7.5.5. Compression flange yielding (F4.1)

$$h_c = 14.97 \text{ in} = d - 2 * t_f$$

$$\lambda_{w_F4} = 37.90 = h_c / t_w$$

$$\lambda_{pw} = 90.55 = 3.76 * \sqrt{E/F_y}$$

$$\lambda_{rw} = 137.27 = 5.7 * \sqrt{E/F_y}$$

$$M_{yc} = 5850 \text{ kip-in} = F_y * S_{xc}$$

$$R_{pc} = 1.11 = \text{IF}(h_c / t_w \leq \lambda_{pw}, M_p / M_{yc}, \text{MIN}(M_p / M_{yc} - (M_p / M_{yc} - 1) * (\lambda - \lambda_{pw}) / (\lambda_{rw} - \lambda_{pw}), M_p / M_{yc}))$$

$$M_{n_F4.1} = 6500 = R_{pc} * M_{yc} \quad (F4-1)$$

7.5.6. Lateral torsional buckling (F4.2)

$$a_{w_F4.2} = 0.87 = h_c * t_w / (b_f * t_f) \quad (F4-12)$$

$$r_{t_F4} = 2.75 \text{ in} = b_f / \sqrt{12 * (1 + 1/6 * a_{w_F4.2})} \quad (F4-11)$$

$$L_{p_F4} = 72.89 = 1.1 * r_{t_F4} * \sqrt{E/F_y} \quad (F4-7)$$

$$L_{r_F4} = 304.95 \text{ in} = 1.95 * r_{t_F4} * E / F_y * \sqrt{J / (S_{xc} * h_o)} + \sqrt{(J / S_{xc} * h_o)^2 + 6.76 * (F_y / E)^2} \quad (F4-8)$$

$$F_{cr_F4} = 126.16 \text{ ksi} = C_b * \pi^2 * E / (L_b / r_{t_F4})^2 * \sqrt{1 + 0.078 * J / (S_{xc} * h_o) * (L_b / r_{t_F4})^2} \quad (F4-5)$$

$$M_{n_F4.2} = 6500 \text{ kip-in} = \text{IF}(L_b \leq L_p, M_p, \text{IF}(L_b > L_r, F_{cr_F4} * S_{xc}, \text{MIN}(C_b * (R_{pc} * M_{yc} - (R_{pc} * M_{yc} - F_y * S_{xc}) * (L_b - L_p) / (L_r - L_p)), R_{pc} * M_{yc}))) \quad (F4-2, 4-3)$$

7.5.7. Compression flange local buckling (F4.3)

$$\lambda_{f_F4.3} = 7.67 = b_f / (2 * t_f)$$

$$\lambda_{pf} = 9.15 \text{ From Section 3 above}$$

$$\lambda_{rf} = 24.08 \text{ From Section 3 above}$$

$$M_{n_F13} = 6739 = R_{pc} * M_{yc} - (R_{pc} * M_{yc} - F_y * S_{xc}) * (\lambda_{f_F4.3} - \lambda_{pf}) / (\lambda_{rf} - \lambda_{pf}) \quad (F4-13)$$

$$M_{n_F14} = 33735 = 0.9 * E * k_c * S_{xc} / \lambda_{f_F4.3}^2 \quad (F4-14)$$

$$M_{n_F4.3} = 6500 \text{ kip-in} = \text{IF}(\text{Flange} = \text{"Compact"}, M_p, \text{IF}(\text{Flange} = \text{"Non-Compact"}, M_{n_F13}, M_{n_F14}))$$

7.5.8. Tension flange yielding (F4.4)

$\lambda_{F44} =$	37.90		$= h_c / t_w$	
$M_{yt} =$	5850	kip-in	$= F_y * S_{xt}$	(F4-16a)
$\lambda_{pw} =$	91		From Section 3 above	
$\lambda_{rw} =$	137		From Section 3 above	
$R_{pt_F4-16a} =$	1.11		$= M_p / M_{yt}$	(F4-16b)
$R_{pt_F4-16b} =$	1.24		$= M_p / M_{yt} - (M_p / M_{yt} - 1) * (\lambda_{F44} - \lambda_{pw}) / (\lambda_{rw} - \lambda_{pw})$	
$R_{pt} =$	1.11		$= IF(\lambda_{F44} < \lambda_{pw}, R_{pt_F4-16a}, R_{pt_F4-16b})$	
$M_{n_F4.4} =$	6500	kip-in	$= R_{pt} * M_{yt}$	(F4-15)

7.5.9. Compression flange yielding (F5.1)

$a_w =$	0.87		$= \min(10, a_{w_F42})$	
$r_{t_F5} =$	2.75	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)
$M_{n_F5.1} =$	5850	kip-in	$= R_{pg} * F_y * S_{xc}$	(F5-1)

7.5.10. Lateral-torsional buckling (F5.2)

$L_{p_F5} =$	72.9	in	$= 1.1 * r_{t_F5} * \sqrt{E / F_y}$	(F4-7)
$L_{r_F5} =$	249	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	$= IF(L_b < L_{p_F5}, "NA", IF(L_b > L_{r_F5}, F_{cr_F5.4}, F_{cr_F5.3}))$	
$M_{n_F5.2} =$	5850	kip-in	$= IF(L_b < L_{p_F5}, M_p, R_{pg} * F_{cr_LTB_F5} * S_{xc})$	(F5-2)

7.5.11. Compression flange local buckling (F5.3)

$\lambda_f =$	7.67		From Section 3 above	
$\lambda_{fp} =$	9.15		From Section 3 above	
$\lambda_{fr} =$	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)
$F_{cr_F5.9} =$	288	ksi	$= 0.9 * E * k_c / (b_f / (2 * t_f))^2$	(F5-9)
$F_{cr_F5} =$	50	ksi	$= IF(Flange = "Compact", 50, IF(Flange = "Non-Compact", F_{cr_F5.8}, F_{cr_F5.9}))$	
$M_{n_F5.3} =$	5850	kip-in	$= R_{pg} * F_{cr_F5} * S_{xc}$	(F5-7)

7.5.12. Tension flange yielding (F5.4)

$M_{n_F5.4} =$	6500	kip-in	$= M_p$ Both flanges are symmetric	(F5-10)
-----------------	------	--------	------------------------------------	---------

7.5.13. Flange hole reduction (F13.1)

$D_{HoleT} =$	0.000	in		
$A_{fg} =$	6.78	in ²	$= b_f * t_f$	
$A_{fn} =$	6.78	in ²	$= t_f * (b_f - 2 * D_{HoleT})$	
$Y_t =$	1.00		$= IF(F_y / F_u < 0.8, 1, 1.1)$	
$M_{n_F13.1} =$	7605	kip-in	$= F_u * A_{fn} / A_{fg} * S_x$	(F13-1)

Flexural Capacity Summary

$M_{n_F2} =$	6500	kip-in	$= \min(M_{n_F2.1}, M_{n_F2.2})$
$M_{n_F3} =$	6500	kip-in	$= \min(M_{n_F3.1}, M_{n_F3.2})$
$M_{n_F4} =$	6500	kip-in	$= \min(M_{n_F4.1}, M_{n_F4.2}, M_{n_F4.3}, M_{n_F4.4})$
$M_{n_F5} =$	5850	kip-in	$= \min(M_{n_F5.1}, M_{n_F5.2}, M_{n_F5.3}, M_{n_F5.4})$
$M_n =$	6500	kip-in	$= IF(B3 = "Beam", IF(DesignSection = "F2", \min(M_{n_F2}, M_{n_F13.1}), IF(DesignSection = "F3", \min(M_{n_F3}, M_{n_F13.1}), IF(DesignSection = "F4", \min(M_{n_F4}, M_{n_F13.1}), \min(M_{n_F5}, M_{n_F13.1}))))), M_{n_F4})$
$\Phi M_n =$	5850	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)

$K_x = 1$	$K_y = 1$	$K_z = 1$	
$L_x = 220$ in	$L_y = 220.0$ in	$L_z = 220.0$ in	
$K_{LR} = 89.43$	$= \max(K_x * L_x / r_x, K_y * L_y / r_y)$		
$F_e = 35.79$ ksi	$= \pi^2 * E / (K_{LR})^2$		(E3-4)
$F_{cr_E.3} = 27.86$ ksi	$= \text{IF}(K_{LR} \leq 4.71 * \text{SQRT}(E/F_y), F_y * 0.658^{(F_y/F_e)}, 0.877 * F_e)$		(E3-2, 3-3)
$P_{n_E3} = 546.08$ kip	$= F_{cr_E.3} * A_g$		(E3-1)

7.6.2. E.4 Torsional and flexural-torsional buckling: (Applies to singly symmetric shapes)

$F_{ey} = 35.79$ ksi	$= \pi^2 * E / (K_y * L_y / r_y)^2$	(E3-4)
$F_{ez} = 65.18$ ksi	$= \pi^2 * E * C_w / (K_z * L_z)^2 + G * J / (1 / (I_x + I_y))$	(E4-2)
$F_{e_E.4} = 35.79$ ksi	$= (F_{ey} + F_{ez}) / (2 * H_z) * (1 - \text{sqrt}(1 - 4 * F_{ey} * F_{ez} * H_z / (F_{ey} + F_{ez})^2))$	(E4-3)
$F_{cr_E.4} = 27.86$ ksi	$= \text{IF}(K_{Lr} \leq 4.71 * \text{SQRT}(E/F_y), F_y * 0.658^{(F_y/F_{e_E.4})}, 0.877 * F_{e_E.4})$	(E3-2, 3-3)
$P_{n_E.4} = 546.1$ 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		
$\lambda = 7.67$	From Section 3 above	
$\lambda_r = 13.49$	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	
$F_{el} = 343.3$ ksi	$= (c2 * \lambda_{rw} / \lambda_w)^2 * F_y$	
$F_{cr} = 27.9$ ksi	$= \text{Min}(F_{cr_E3}, F_{cr_E4})$	
$bf = 10.20$ in	= Full Flange Width from AISC Database	
$b_{e_E7-3} = 8.2$ in	$= bf * (1 - c1 * \text{sqrt}(F_{el}/F_{cr})) * \text{sqrt}(F_{el}/F_{cr})$, but can't be more than bf	(E7-3)
$be = 10.20$ in	$= \text{If}(\text{Flange} = \text{"Nonslender"}, bf, b_{e_E7-3})$	
$A_{web} = 6.03$ in ²	$= A_g - 2 * tf * bf$	
$A_f = 13.57$ in ²	$= 2 * tf * be$	
$A_{e1} = 19.6$ in ²	$= A_{web} + A_f$	

7.6.2b Calculation for effective web depth

Web= Slender		
$\lambda_w = 37.90$	From Section 4.1 above	
$\lambda_{rw} = 35.88$	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	
$F_{el} = 76.9$ ksi	$= (c2 * \lambda_{rw} / \lambda_w)^2 * F_y$	
$h = 15.0$ in		
$h_{e_E7-3} = 15.0$ in	$= h * (1 - c1 * \text{sqrt}(F_{el}/F_{cr})) * \text{sqrt}(F_{el}/F_{cr})$, but can't be more than h	(E7-3)
$h_e = 15.0$ in	$= \text{If}(\text{Web} = \text{"Nonslender"}, h, h_{e_E7-3})$	
$A_{flange} = 13.6$ in ²	$= 2 * bf * tf$	
$A_{web_radius} = 6.0$ in ²	$= A_g - A_{flange}$	
$A_{web} = 5.9$ in ²	$= h * t_w$	
$A_{radius} = 0.1$ in ²	$= A_{web_radius} - A_{web}$	
$A_{e_web} = 5.91$ in ²	$= h_e * t_w$	
$A_{e2} = 19.6$ in ²	$= A_{flange} + A_{radius} + A_{e_web}$	
$A_e = 19.6$ in ²	$= \text{Min}(A_{e1}, A_{e2})$	
$F_{cr} = 27.9$ ksi		
$P_{n_E7} = 546.1$ kips		
$P_n = 546.1$ kip	$= \text{IF}(\text{AxialDesignSection} = \text{"E3"}, P_{n_E3}, P_{n_E7})$	
$\Phi_c * P_n = 491.5$ kip		

7.7. Demand per AISC 360-16 Appendix 8 (to include p-small delta)

$$\begin{aligned}
 P_y &= 980 \text{ kip} & = A_g * F_y \\
 P_r &= 10 \text{ kip} & = P_{u_omega} \\
 \alpha &= 1 & \text{1 for LRFD, 1.6 for ASD} \\
 C_m &= 1 & \text{conservative assumption} \\
 P_{e1y} &= 5642 \text{ kip} & = \pi^2 * E * I_{xx} / (L_b)^2 \\
 B_1 &= 1.002 & = C_m / (1 - \alpha * P_r / P_{e1y})
 \end{aligned}$$

(A-8-5)

(A-8-3)

7.8. Axial + flexural interaction (Chapter H)

$$\begin{aligned}
 M_r &= 4788 \text{ kip-in} & = M_u * B_1 \\
 M_c &= 5850 \text{ kip-in} & = \Phi * M_n \\
 P_r &= 9.8 \text{ kip} & = P_u \\
 P_c &= 491.5 \text{ kip} & = \Phi_c * P_n \\
 DCR_Axial &= 0.020 & = P_r / P_c \\
 DCR_Flexural &= 0.819 & = M_r / M_c \quad \text{Adj_DCR_Flexural} = 0.598 \text{ For } R > 3 \\
 DCR_P+M &= 0.829 & = \text{IF}(P_r / P_c > 0.2, P_r / P_c + 8/9 * M_r / M_c, P_r / (2 * P_c) + M_r / M_c) \\
 \text{Adj } P+M &= 0.608 & = \text{IF}(P_r / P_c > 0.2, P_r / P_c + 8/9 * \text{Adj_DCR_Flexural}, P_r / (2 * P_c) + \text{Adj_DCR_Flexural})
 \end{aligned}$$

(H1-1)

7.9. Shear capacity (Chapter G)

$$\begin{aligned}
 V_u &= 40.92 \text{ kips} \\
 A_w &= 6.44 \text{ in}^2 & = t_w * (d - \text{Hole}_{web}) \\
 h/t_w &= 38 \\
 k_v &= 5.34 & \text{Webs without transverse stiffeners} \\
 2.24 \sqrt{E/F_y} &= 53.9 \\
 C_{v1_G2-2} &= 1 & \text{(G2-2)} \\
 1.1 \sqrt{k_v E/F_y} &= 61.2 \\
 C_{v1_G2-3} &= 1.00 & \text{(G2-3)} \\
 C_{v1_G2-4} &= 1.6 & = 1.1 * \sqrt{k_v * E/F_y} / (h/t_w) \text{ (G2-4)} \\
 C_{v1_G2-3_4?} &= 1.0 & = \text{if}(h/t_w \leq 1.1 * \sqrt{k_v * E/F_y}, C_{v1_G2-3}, C_{v1_G2-4}) \\
 C_{v1} &= 1.0 & = \text{if}(C_{v1_G2-2} = "N/A", C_{v1_G2-3_4?}) \\
 \phi_v V_n &= 193.2 \text{ kip} & = \phi_v * 0.6 * F_y * A_w * C_{v1} \text{ (G2-1)} \\
 DCR_Shear &= 0.212 & = V_u / (\Phi_v * V_n)
 \end{aligned}$$

7.10. Design Summary:

		Demand-to-Capacity Ratios					
LC ID	Load Combination	P	M	P+M	V	Adj M	Adj P+M
LC 7	(1.2 + 0.2 SDS)DL + OmegaEL + f1*LL + f2*SL	0.020	0.819	0.829	0.212	0.598	0.608

7. AISC 360-16 Steel Beam Design

Beam ID: **2001** Beam Size= **W16X67** $L_b = 220.0$ in $R = 3.5$
LC: **7** Beam Type= W-Section Link Size= **YLMC6-3.5**

7.1. Loading

$M_u = 3512$ kip-in $P_u = 22.82$ kip $V_u = 31.06$ kip
 $M_{max} = 3512$ kip-in $M_{cap_link} = 3491$ $M_{cap}/M_u = 0.994$ =1 for wind loads, $R=3$
 $M_A = 1676$ kip-in $M_B = 92$ kip-in $M_C = 1888$ kip-in
Beam Bracing= **None** LeftLink= **YES** RightLink= **YES**

7.2. Material properties

$F_y = 50$ ksi $E = 29000$ ksi $A_g = 19.6$ in²
 $F_u = 65$ ksi $G = 11200$ ksi $Z_x = 130$ in³
 $bf = 10.2$ in $r_x = 6.960$ in $Z_y = 35.5$ in³
 $d = 16.3$ in $r_y = 2.460$ in $S_x = 117.00$ in³
 $k_{des} = 1.07$ in $t_f = t_{fi} = 0.665$ in $S_{xc} = 117$ in³
 $I_x = 954$ in⁴ $t_{fe} = 0.665$ in $S_{xt} = 117$ in³
 $I_y = 119$ in⁴ $t_w = 0.395$ in $H_z = 1.0$
 $J = 2.39$ in⁴ $C_w = 7300.00$ in⁶ $L = 220$ in
 $h = 14.97$ in $= d - 2 * t_f$
 $h_c = 15.07$ in $= 2 * (d - t_{fi} - (b_f * t_{fi}^2 / 2 + t_w * (d - t_{fe} - t_{fi}) * (d - t_{fe} + t_{fi}) / 2 + b_f * t_{fe} * (d - t_{fe} / 2)) / A_g)$
 $h_p = 16.30$ 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)

$k_c = 0.65$ $= \max(0.35, \min(4 / \sqrt{h/t_w}, 0.76))$ For built-up sections
 $\lambda = 7.67$ $b_f / (2 * t_f)$
 $\lambda_p = 9.15$ $= 0.38 * \sqrt{E/F_y}$
 $F_L = 35.00$ $= IF(S_{xt}/S_{xc} > 0.7, 0.7 * F_y, \max(F_y * S_{xt}/S_{xc}, 0.5 * F_y))$
 $\lambda_{r_case10} = 24.08$ $= 1 * \sqrt{E/F_y}$ Case 10-W-Sections
 $\lambda_{r_case11} = 22.04$ $= 0.95 * \sqrt{K_c * E/F_L}$ Case 11 BU-Sections
 $\lambda_r = 24.08$
Flange: **Compact** $= IF(\lambda < \lambda_p, "Compact", IF(AND(\lambda < \lambda_r, \lambda > \lambda_p), "Non-Compact", "Slender"))$

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

$\lambda = 37.90$ $= h/t_w$
 $\lambda_p = 90.55$ $= 3.76 * \sqrt{E/F_y}$
 $\lambda_r = 137.27$ $= 5.70 * \sqrt{E/F_y}$
Web: **Compact** $= IF(\lambda < \lambda_p, "Compact", IF(AND(\lambda < \lambda_r, \lambda > \lambda_p), "Non-Compact", "Slender"))$

Per Table F1.1 for Flexure

Section: **F2**
 $\Phi_b = 0.9$ $= IF(AND(Flange="Compact", Web="Compact"), "F2", IF(AND(OR(Flange="Non-Compact", Flange="Slender"), Web="Compact"), "F3", IF(AND(OR(Flange="Compact", Flange="Non-Compact", Flange="Slender"), Web="Slender"), "F5", "F4")))$
 $\Phi_c = 0.9$
 $\Phi_v = 1$

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

$\lambda = 7.67$ $b_f / (2 * t_f)$
 $\lambda_{r_case1} = 13.49$ $= 0.56 * \sqrt{E/F_y}$
 $k_c = 0.65$ From Section 3
 $\lambda_{r_case2} = 12.42$ $= 0.64 * \sqrt{K_c * E/F_y}$
 $\lambda_r = 13.49$
Flange: **Nonslender**

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

$\lambda = 37.90$ $= h/t_w$
 $\lambda_r = 35.88$ $= 1.49 * \sqrt{E/F_y}$
Web: **Slender**

7.5. Flexural Capacity

7.5.1. Yielding (F2.1)

$$M_{n_F2.1} = 6500 \text{ kip-in} = M_p = F_y * Z_x \quad (F2-1)$$

7.5.2. Lateral-torsional buckling (F2.2)

$$L_b = 220 \text{ in} \quad \text{Lateral torsional bracing length}$$

$$L_p = 104.27 \text{ in} \quad = 1.76 * r_y * \sqrt{E/F_y} \quad (F2-5)$$

$$r_{ts} = 2.82 \text{ in} \quad = \sqrt{\sqrt{I_y * C_w} / S_x} \quad (F2-7)$$

$$h_o = 15.635 \text{ in} \quad = d - t_f \quad \text{Distance between centroids}$$

$$c = 1 \quad \text{1 for doubly symmetric I-Shapes} \quad (EQ F2-8a)$$

$$L_r = 312.8 \text{ in} \quad = 1.95 * r_{ts} * E / (0.7 * F_y) * \sqrt{(J * c) / (S_x * h_o)} * \sqrt{1 + \sqrt{1 + 6.76 * (0.7 * F_y / E * S_x * h_o / (J * c))^2}} \quad (F2-6)$$

$$C_b = 2.21 \quad = 12.5 * M_{max} / (2.5 * M_{max} + 3 * M_A + 4 * M_B + 3 * M_C) \quad (F1-1)$$

$$F_{cr} = 132.6 \text{ ksi} \quad = C_b * \pi^2 * E / (L_b / r_{ts})^2 * \sqrt{1 + 0.078 * J_{col} * C_{col} / (S_x * h_o) * (L_b / r_{ts})^2} \quad (F2-4)$$

$$M_{n_F2.2} = 6500 \text{ kip-in} \quad = IF(L_b \leq L_p, M_p, IF(AND(L_p < L_b, L_b \leq L_r), MIN(C_b * (M_p - (M_p - 0.7 * F_y * S_x) * ((L_b - L_p) / (L_r - L_p))), M_p), MIN(F_{cr} * S_x, M_p))) \quad (F2-2, 2-3)$$

7.5.3. Lateral torsional buckling (F3.1)

$$M_{n_F3.1} = 6500 \quad = M_{n_F2.2}$$

7.5.4. Compression flange local buckling (F3.2)

$$k_c = 0.65 \quad = \min(\max(0.35, 4 / \sqrt{h/t_w}), 0.76)$$

$$\lambda = 7.669 \quad = b_f / (2 * t_f)$$

$$\lambda_{pf} = 9.15 \quad = 0.38 * \sqrt{E/F_y}$$

$$\lambda_{rf} = 24.1 \quad = 1.0 * \sqrt{E/F_y}$$

$$M_{n_F3.2} = 6500 \quad = IF(Flange = "Non-Compact", M_p - (M_p - 0.7 * S_x * F_y) * (\lambda - \lambda_{pf}) / (\lambda_{rf} - \lambda_{pf}), IF(Flange = "Slender", 0.9 * E * k_c * S_x / \lambda^2, M_p)) \quad (F3-1, 3-2)$$

7.5.5. Compression flange yielding (F4.1)

$$h_c = 14.97 \text{ in} \quad = d - 2 * t_f$$

$$\lambda_{w_F4} = 37.90 \quad = h_c / t_w$$

$$\lambda_{pw} = 90.55 \quad = 3.76 * \sqrt{E/F_y}$$

$$\lambda_{rw} = 137.27 \quad = 5.7 * \sqrt{E/F_y}$$

$$M_{yc} = 5850 \text{ kip-in} \quad = F_y * S_{xc}$$

$$R_{pc} = 1.11 \quad = IF(h_c / t_w \leq \lambda_{pw}, M_p / M_{yc}, MIN(M_p / M_{yc} - (M_p / M_{yc} - 1) * (\lambda - \lambda_{pw}) / (\lambda_{rw} - \lambda_{pw}), M_p / M_{yc}))$$

$$M_{n_F4.1} = 6500 \quad = R_{pc} * M_{yc} \quad (F4-1)$$

7.5.6. Lateral torsional buckling (F4.2)

$$a_{w_F4.2} = 0.87 \quad = h_c * t_w / (b_f * t_f) \quad (F4-12)$$

$$r_{t_F4} = 2.75 \text{ in} \quad = b_f / \sqrt{12 * (1 + 1/6 * a_{w_F4.2})} \quad (F4-11)$$

$$L_{p_F4} = 72.89 \quad = 1.1 * r_{t_F4} * \sqrt{E/F_y} \quad (F4-7)$$

$$L_{r_F4} = 304.95 \text{ in} \quad = 1.95 * r_{t_F4} * E / F_y * \sqrt{J / (S_{xc} * h_o)} + \sqrt{(J / S_{xc} * h_o)^2 + 6.76 * (F_y / E)^2} \quad (F4-8)$$

$$F_{cr_F4} = 127.30 \text{ ksi} \quad = C_b * \pi^2 * E / (L_b / r_{t_F4})^2 * \sqrt{1 + 0.078 * J / (S_{xc} * h_o) * (L_b / r_{t_F4})^2} \quad (F4-5)$$

$$M_{n_F4.2} = 6500 \text{ kip-in} \quad = IF(L_b \leq L_p, M_p, IF(L_b > L_r, F_{cr_F4} * S_{xc}, MIN(C_b * (R_{pc} * M_{yc} - (R_{pc} * M_{yc} - F_y * S_{xc}) * (L_b - L_p) / (L_r - L_p)), R_{pc} * M_{yc}))) \quad (F4-2, 4-3)$$

7.5.7. Compression flange local buckling (F4.3)

$$\lambda_{f_F4.3} = 7.67 \quad = b_f / (2 * t_f)$$

$$\lambda_{pf} = 9.15 \quad \text{From Section 3 above}$$

$$\lambda_{rf} = 24.08 \quad \text{From Section 3 above}$$

$$M_{n_F13} = 6739 \quad = R_{pc} * M_{yc} - (R_{pc} * M_{yc} - F_y * S_{xc}) * (\lambda_{f_F4.3} - \lambda_{pf}) / (\lambda_{rf} - \lambda_{pf}) \quad (F4-13)$$

$$M_{n_F14} = 33735 \quad = 0.9 * E * k_c * S_{xc} / \lambda_{f_F4.3}^2 \quad (F4-14)$$

$$M_{n_F4.3} = 6500 \text{ kip-in} \quad = IF(Flange = "Compact", M_p, IF(Flange = "Non-Compact", M_{n_F13}, M_{n_F14}))$$

7.5.8. Tension flange yielding (F4.4)

$\lambda_{F44} =$	37.90		$= h_c / t_w$	
$M_{yt} =$	5850	kip-in	$= F_y * S_{xt}$	(F4-16a)
$\lambda_{pw} =$	91		From Section 3 above	
$\lambda_{rw} =$	137		From Section 3 above	
$R_{pt_F4-16a} =$	1.11		$= M_p / M_{yt}$	(F4-16b)
$R_{pt_F4-16b} =$	1.24		$= M_p / M_{yt} - (M_p / M_{yt} - 1) * (\lambda_{F44} - \lambda_{pw}) / (\lambda_{rw} - \lambda_{pw})$	
$R_{pt} =$	1.11		$= IF(\lambda_{F44} < \lambda_{pw}, R_{pt_F4-16a}, R_{pt_F4-16b})$	
$M_{n_F4.4} =$	6500	kip-in	$= R_{pt} * M_{yt}$	(F4-15)

7.5.9. Compression flange yielding (F5.1)

$a_w =$	0.87		$= \min(10, a_{w_F42})$	
$r_{t_F5} =$	2.75	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)
$M_{n_F5.1} =$	5850	kip-in	$= R_{pg} * F_y * S_{xc}$	(F5-1)

7.5.10. Lateral-torsional buckling (F5.2)

$L_{p_F5} =$	72.9	in	$= 1.1 * r_{t_F5} * \sqrt{E / F_y}$	(F4-7)
$L_{r_F5} =$	249	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	$= IF(L_b < L_{p_F5}, "NA", IF(L_b > L_{r_F5}, F_{cr_F5.4}, F_{cr_F5.3}))$	
$M_{n_F5.2} =$	5850	kip-in	$= IF(L_b < L_{p_F5}, M_p, R_{pg} * F_{cr_LTB_F5} * S_{xc})$	(F5-2)

7.5.11. Compression flange local buckling (F5.3)

$\lambda_f =$	7.67		From Section 3 above	
$\lambda_{fp} =$	9.15		From Section 3 above	
$\lambda_{fr} =$	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)
$F_{cr_F5.9} =$	288	ksi	$= 0.9 * E * k_c / (b_f / (2 * t_f))^2$	(F5-9)
$F_{cr_F5} =$	50	ksi	$= IF(Flange = "Compact", 50, IF(Flange = "Non-Compact", F_{cr_F5.8}, F_{cr_F5.9}))$	
$M_{n_F5.3} =$	5850	kip-in	$= R_{pg} * F_{cr_F5} * S_{xc}$	(F5-7)

7.5.12. Tension flange yielding (F5.4)

$M_{n_F5.4} =$	6500	kip-in	$= M_p$ Both flanges are symmetric	(F5-10)
-----------------	------	--------	------------------------------------	---------

7.5.13. Flange hole reduction (F13.1)

$D_{HoleT} =$	0.000	in		
$A_{fg} =$	6.78	in ²	$= b_f * t_f$	
$A_{fn} =$	6.78	in ²	$= t_f * (b_f - 2 * D_{HoleT})$	
$Y_t =$	1.00		$= IF(F_y / F_u < 0.8, 1, 1.1)$	
$M_{n_F13.1} =$	7605	kip-in	$= F_u * A_{fn} / A_{fg} * S_x$	(F13-1)

Flexural Capacity Summary

$M_{n_F2} =$	6500	kip-in	$= \min(M_{n_F2.1}, M_{n_F2.2})$
$M_{n_F3} =$	6500	kip-in	$= \min(M_{n_F3.1}, M_{n_F3.2})$
$M_{n_F4} =$	6500	kip-in	$= \min(M_{n_F4.1}, M_{n_F4.2}, M_{n_F4.3}, M_{n_F4.4})$
$M_{n_F5} =$	5850	kip-in	$= \min(M_{n_F5.1}, M_{n_F5.2}, M_{n_F5.3}, M_{n_F5.4})$
$M_n =$	6500	kip-in	$= IF(B3 = "Beam", IF(DesignSection = "F2", \min(M_{n_F2}, M_{n_F13.1}), IF(DesignSection = "F3", \min(M_{n_F3}, M_{n_F13.1}), IF(DesignSection = "F4", \min(M_{n_F4}, M_{n_F13.1}), \min(M_{n_F5}, M_{n_F13.1}))))), M_{n_F4})$
$\Phi M_n =$	5850	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)

$K_x = 1$	$K_y = 1$	$K_z = 1$	
$L_x = 220$ in	$L_y = 220.0$ in	$L_z = 220.0$ in	
$K_{LR} = 89.43$	$= \max(K_x * L_x / r_x, K_y * L_y / r_y)$		
$F_e = 35.79$ ksi	$= \pi^2 * E / (K_{LR})^2$		(E3-4)
$F_{cr_E.3} = 27.86$ ksi	$= \text{IF}(K_{LR} \leq 4.71 * \text{SQRT}(E/F_y), F_y * 0.658^{(F_y/F_e)}, 0.877 * F_e)$		(E3-2, 3-3)
$P_{n_E3} = 546.08$ kip	$= F_{cr_E.3} * A_g$		(E3-1)

7.6.2. E.4 Torsional and flexural-torsional buckling: (Applies to singly symmetric shapes)

$F_{ey} = 35.79$ ksi	$= \pi^2 * E / (K_y * L_y / r_y)^2$	(E3-4)
$F_{ez} = 65.18$ ksi	$= \pi^2 * E * C_w / (K_z * L_z)^2 + G * J / (1 / (I_x + I_y))$	(E4-2)
$F_{e_E.4} = 35.79$ ksi	$= (F_{ey} + F_{ez}) / (2 * H_z) * (1 - \text{sqrt}(1 - 4 * F_{ey} * F_{ez} * H_z / (F_{ey} + F_{ez})^2))$	(E4-3)
$F_{cr_E.4} = 27.86$ ksi	$= \text{IF}(K_{Lr} \leq 4.71 * \text{SQRT}(E/F_y), F_y * 0.658^{(F_y/F_{e_E.4})}, 0.877 * F_{e_E.4})$	(E3-2, 3-3)
$P_{n_E.4} = 546.1$ 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

$\lambda = 7.67$	From Section 3 above	
$\lambda_r = 13.49$	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	
$F_{el} = 343.3$ ksi	$= (c2 * \lambda_{rw} / \lambda_w)^2 * F_y$	
$F_{cr} = 27.9$ ksi	$= \text{Min}(F_{cr_E3}, F_{cr_E4})$	
$bf = 10.20$ in	= Full Flange Width from AISC Database	
$b_{e_E7-3} = 8.2$ in	$= bf * (1 - c1 * \text{sqrt}(F_{el}/F_{cr})) * \text{sqrt}(F_{el}/F_{cr})$, but can't be more than bf	(E7-3)
$be = 10.20$ in	= If (Flange="Nonslender", b_f , b_{e_E7-3})	
$A_{web} = 6.03$ in ²	$= A_g - 2 * t_f * bf$	
$A_f = 13.57$ in ²	$= 2 * t_f * be$	
$A_{e1} = 19.6$ in ²	$= A_{web} + A_f$	

7.6.2b Calculation for effective web depth

Web= Slender		
$\lambda_w = 37.90$	From Section 4.1 above	
$\lambda_{rw} = 35.88$	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	
$F_{el} = 76.9$ ksi	$= (c2 * \lambda_{rw} / \lambda_w)^2 * F_y$	
$h = 15.0$ in		
$h_{e_E7-3} = 15.0$ in	$= h * (1 - c1 * \text{sqrt}(F_{el}/F_{cr})) * \text{sqrt}(F_{el}/F_{cr})$, but can't be more than h	(E7-3)
$h_e = 15.0$ in	= If (Web="Nonslender", h , h_{e_E7-3})	
$A_{flange} = 13.6$ in ²	$= 2 * b_f * t_f$	
$A_{web_radius} = 6.0$ in ²	$= A_g - A_{flange}$	
$A_{web} = 5.9$ in ²	$= h * t_w$	
$A_{radius} = 0.1$ in ²	$= A_{web_radius} - A_{web}$	
$A_{e_web} = 5.91$ in ²	$= h_e * t_w$	
$A_{e2} = 19.6$ in ²	$= A_{flange} + A_{radius} + A_{e_web}$	
$A_e = 19.6$ in ²	$= \text{Min}(A_{e1}, A_{e2})$	
$F_{cr} = 27.9$ ksi		
$P_{n_E7} = 546.1$ kips		
$P_n = 546.1$ kip	$= \text{IF}(\text{AxialDesignSection}="E3", P_{n_E3}, P_{n_E7})$	
$\Phi_c * P_n = 491.5$ kip		

7.7. Demand per AISC 360-16 Appendix 8 (to include p-small delta)

$$\begin{aligned}
 P_u &= 980 \text{ kip} & = A_g * F_y \\
 P_r &= 23 \text{ kip} & = P_u_{\text{omega}} \\
 \alpha &= 1 & \text{1 for LRFD, 1.6 for ASD} \\
 C_m &= 1 & \text{conservative assumption} \\
 P_{e1y} &= 5642 \text{ kip} & = \pi^2 * E * I_x / (L_b)^2 \\
 B_1 &= 1.004 & = C_m / (1 - \alpha * P_r / P_{e1y})
 \end{aligned}$$

(A-8-5)

(A-8-3)

7.8. Axial + flexural interaction (Chapter H)

$$\begin{aligned}
 M_u &= 3526 \text{ kip-in} & = M_u * B_1 \\
 M_c &= 5850 \text{ kip-in} & = \Phi * M_n \\
 P_u &= 22.8 \text{ kip} & = P_u \\
 P_c &= 491.5 \text{ kip} & = \Phi_c * P_n \\
 \text{DCR}_{\text{Axial}} &= 0.046 & = P_r / P_c \\
 \text{DCR}_{\text{Flexural}} &= 0.603 & = M_r / M_c \\
 \text{DCR}_{P+M} &= 0.626 & = \text{IF}(P_r / P_c \geq 0.2, P_r / P_c + 8/9 * M_r / M_c, P_r / (2 * P_c + M_r / M_c)) \\
 \text{Adj } P+M &= 0.622 & = \text{IF}(P_r / P_c \geq 0.2, P_r / P_c + 8/9 * \text{Adj}_{\text{DCR}_{\text{Flexural}}}, P_r / (2 * P_c + \text{Adj}_{\text{DCR}_{\text{Flexural}}}))
 \end{aligned}$$

Adj_DCR_Flexural = 0.599 For R>3

(H1-1)

7.9. Shear capacity (Chapter G)

$$\begin{aligned}
 V_u &= 31.06 \text{ kips} \\
 A_w &= 6.44 \text{ in}^2 & = t_w * (d - \text{Hole}_{\text{web}}) \\
 h/t_w &= 38 \\
 k_v &= 5.34 & \text{Webs without transverse stiffeners} \\
 2.24 \sqrt{E/F_y} &= 53.9 \\
 C_{v1_G2-2} &= 1 \\
 1.1 \sqrt{k_v E/F_y} &= 61.2 \\
 C_{v1_G2-3} &= 1.00 \\
 C_{v1_G2-4} &= 1.6 & = 1.1 * \sqrt{k_v * E/F_y} / (h/t_w) \\
 C_{v1_G2-3_4?} &= 1.0 & = \text{if}(h/t_w \leq 1.1 * \sqrt{k_v * E/F_y}, C_{v1_G2-3}, C_{v1_G2-4}) \\
 C_{v1} &= 1.0 & = \text{if}(C_{v1_G2-2} = "N/A", C_{v1_G2-3_4?}) \\
 \phi_v V_n &= 193.2 \text{ kip} & = \phi_v * 0.6 * F_y * A_w * C_{v1} \\
 \text{DCR}_{\text{Shear}} &= 0.161 & = V_u / (\phi_v * V_n)
 \end{aligned}$$

(G2-2)

(G2-3)

(G2-4)

(G2-1)

7.10. Design Summary:

		Demand-to-Capacity Ratios					
LC ID	Load Combination	P	M	P+M	V	Adj M	Adj P+M
LC 7	(1.2 + 0.2 SDS)DL + OmegaEL + f1*LL + f2*SL	0.046	0.603	0.626	0.161	0.599	0.622

3.1 CURRENT MEMBER:

Beam Unique Name:	1001	Link ID:	YLMC6-3.5	R=	3.5
Beam Size:	W16X67	Mu_max:	1764 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 (L_{col_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})=	8.00	in
NY Length BeamSide (L_{bm_side})=	11.25	in	Central Neck Yield Width (b_{yield})=	3.50	in
L_{stem} =	26.25	in	NY Width BeamSide (b_{bm_side})=	8.00	in
Link Radius (r_{link})=	0.75	in	Yielding Area ($A_{stemYield}$)=	2.63	in ²

3.3 LINK STEM BOLTS: (From YL Database)

Num. Bolts (n_{bolt_linkBm})=	6.00		Gauge Along Width (g_{stem})=	5.00	in
Bolt Type (Bolt_Gr_linkBm)=	A490		Spacing Along Length (s_{stem})=	3.38	in
Stem Bolt Dia=	1.00	in	First Bolt distance to Neck (S_o)=	2.50	in
Min. Bolt length =	3.000	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})=	8.00	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})=	5.00	in
Bolt Dia (boltD_linkCol)=	1.00	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})=	8.525	in	$= (db + t_{stem})/2$
Link Extension (linkDelta)=	0.426	in	$= d_{arm} * r_{linkLimit}$
Lyield_req=	6.51	in	$= r_{linkLimit} * d_{arm} / 0.085 + 2 * r_{link}$ (EQ 12.9-4)
Lyield=	10.00	in	from previous value
Lyield_DCR=	0.65		$= L_{stem_yield_req} / L_{stemYield}$
Check=	OK		OK, if $L_{stem_DCR} < DCR_{allowed}$

Strength Check: Step 3

Mu=	1764	k-in
d=	16.3	in
d+t _{stem} =	17.05	in
P _{u_link} =	103.43	kips
P _{y_link} =	131.25	kips
Link stem DCR=	0.876	OK

Krot:

Moment Demand (LRDF combos)
Beam Depth from database
Link Axial demand, Mu/(d+t _{stem})
Link Yield Strength from YL database
$= P_{u_link} / (0.9 * P_{y_link})$

AISC 358, 12.9 Step 11:

K1=	28602	k/in
K2=	18939	k/in
K3=	7612.5	k/in
Keff=	4564	k/in
Mye=	2461.59375	k*in
Δ_y =	0.0316365	in
θ_y =	0.00371103	rad
Krot=	663319	k*in/rad

Slip-Critical Check:

Pu_slip=	103.43	kips
ϕ =	1.0	
μ =	0.30	
Du=	1.13	
hf=	1.00	
ns=	1.00	
Tb=	64.00	kips
n_bolt_linkBm=	6.000	
ϕRn slip=	130.2	kips
Link slip DCR=	0.795	OK

$$= P_{u_link}$$

Minimum Bolt Pretension per Table J3.1

$$= \phi * \mu * Du * hf * Tb * ns * n_{bolt_linkBm}$$

$$= P_{u_link} / \phi Rn_{slip}$$

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_cant=	6.500	in	=Lbrp - Lend - S _{BRP}
Lbrp=	12.750	in	from Database
Wbrp_min=	8.000	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=	204.75	kips	=if(R=3,Py_link, Pr_link)
Lelong=	0.426	in	Elongated distance at 0.05 rad
L1st_bolt=	3.75	in	Start of yielding region to 1st BRP bolt centerline
Lcant=	3.77	in	Lever arm from edge of yield link to edge of bolt hole + elongated dist at 0.05 rad
bn_brp=	6.25	in	Net width of BRP, without bolt holes
tbrp_min=	0.764	in	AISC 358, EQ 12.9-13
tbrp_use=	1.000	in	value previously defined
tbrp_DCR=	0.764		OK

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

Joint Rotation=	0.04	rad	
e _{0.04} =	0.04012	in/in	
g=	0.00752	in	gap between BRP and beam flange
ly=	0.123	in^4	
l _o =	1.071	in	effective buckling wave length
Qi=	5.75	kips	
N=	9.34		=LstemYield / l _o
N=	9.0		Round down to next integer
Ndesign=	5.0		N used for design (Contact points with BRP)
Tux=	28.8	kips	=Ndesign * Qi
Tux_bolt=	7.19	kips	=Tux/Nbolts
φ=	1.0		
Ledge=	6.500	in	1st bolt to edge of BRP
S_brpBolt_min=	5.25	in	=byield + 2*(0.125 + d_brp_bolt)
gbrp=	5.750	in	
b=	2.678	in	BRP bolt to face of web
c=	4.903	in	edge of beam flange to face of web
a=	2.225	in	center of BRP bolt to edge of beam flange
x=sqrt(b*c)=	3.623	in	
4*x=	14.49	in	1 row of BRP bolts:
2*x=	7.25	in	pe (in)= 14.49
S_brp_bolt=	2.500	in	2 rows of BRP bolts:
pe=	8.50	in	pe1 (in)= 8.50
b'=	2.303	in	pe2 (in)= 14.49
tf_min=	0.400	in	pe (in)= 8.50
tbtf=	0.665	in	3 rows of BRP bolts:
tbtf_DCR=	0.602		pe1 (in)= 8.50
			pe2 (in)= 3.06
			pe3 (in)= 8.50
			pe (in)= 6.68

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

I _x =	2.680	in ⁴	=byield^3 * tstem /12, strong axis moment of inertia of reduced link region
gap=	0.188	in	gap between link and spacer plate
Vuy/Spacer=	26.68	kips	
Vuy per bolt=	13.342	kips	
	Tx (k)	Vx (kip)	Ty (kips)
Bolt Forces	7.19	2.157	0
			Vy (kips)
			13.342

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.	Y-Dir.
	DCR	DCR
Tension	0.24	0.00
Shear	0.121	0.746
RnV_(V+T), kips	17.89	17.89
T+V	0.121	0.746

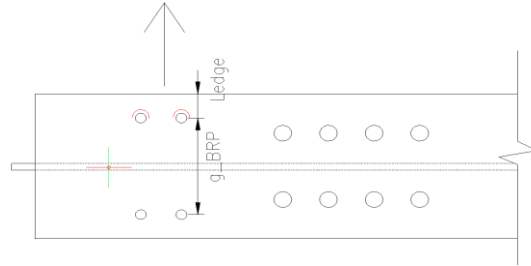
Max DCR= 0.746 OK

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

bbf=	10.20	in	= beam flange width
bbf_BRP_bolt=	7.750	in	=minimum flange width for BRP bolt spacing + min. bolt edge distance
bbf_stem_bolt=	7.500	in	=minimum flange width for stem bolt spacing + min. bolt edge distance
bbf_min=	7.75	in	=max (bbf_BRP_bolt, bbf_stem_bolt)
bf_DCR=	0.760		OK

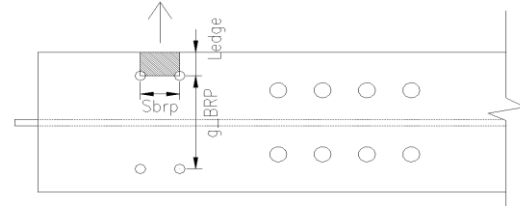
3.13 BRP BOLT BEAM FLANGE BEARING AND TEAROUT IN WEAK-AXIS DIRECTION (AISC 360, J3.10)

bbf=	10.2	in
g_brp=	5.75	in
d_brp=	0.750	in
Ledge=	2.225	in
Ledge_min=	1.00	in
Ledge>1 bolt Dia?	OK	
Ledge>=Ledge_min?	OK	
Vy=	13.34	kips/bolt
Rn_bearing=	77.805	kips
φRn_bearing=	58.35	kips
DCR_bearing=	0.229	OK
Lc=	2.225	in
Rn_tearout=	115.4	kips
φRn_tearout=	86.6	kips
DCR_tearout=	0.154	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=	2.225	in
tbf=	0.665	in
Vuy=	26.68	kips
Lgv=	4.45	in
Agv=	2.95925	in^2
Rn_yield=	88.8	kips
φRn_yield=	88.8	kips
DCR_bearing=	0.301	OK
Lnv=	3.638	in
Anv=	2.419	in^2
Rn_tearout=	94.3	kips
φRn_tearout=	70.75	kips
DCR_tearout=	0.377	OK



Lnt=	1.6875	in
Ant=	1.122	in^2
Rn_BS1=	167.28	kips
Rn_BS2=	161.72	kips
Rn_BS=	161.72	kips
φRn_BS=	121.29	kips
DCR_BS=	0.220	OK

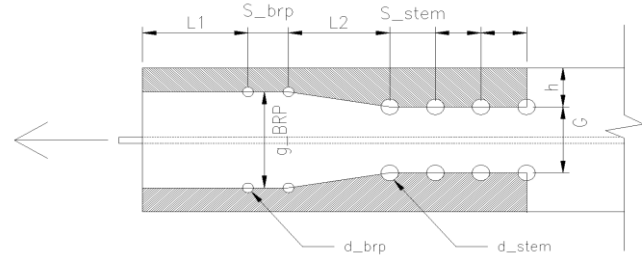
(Rn_BS1,Rn_BS2)

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

Pr_link=	204.75	kips	
tbf=	0.665	in	Beam Flange thickness
bolt_Dia=	1	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=	2.3125	in	
Fy_bm=	50	ksi	from Design_Limits
Fu_bm=	65	ksi	from Design_Limits
Rn_bearing=	622.4	kips	=2.4*bolt_Dia*tbf*Fu_bm*n_bolts
φRn_bearing=	466.83	kips	=0.75*Rn_bearing
DCR_bearing=	0.439	OK	
Rn_tearout=	719.7	kips	=1.2*Lc*tbf*Fu_bm*n_bolts
φRn_tearout=	539.8	kips	=0.75*Rn_tearout
DCR_tearout=	0.379	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=	5	in
gbrp=	5.75	in
d_brp=	0.8125	in
d_stem=	1.0625	in
bf=	10.2	in
h=	2.600	in
h_min=	1.25	in
h>1 bolt Dia?	OK	
h>=h_min?	OK	



Lgv=	22.00	in
Agv=	14.630	in^2
Rn_yield=	877.8	kips
φRn_yield=	658.4	kips
DCR_yield=	0.311	OK
Lnv=	17.72	in
Anv=	11.78	in^2
Rn_rupture=	459.5	kips
φRn_rupture=	344.7	kips
DCR_rupture=	0.594	OK

Lnt=	4.1375	in
Ant=	2.751	in^2
Rn_BS1=	638	kips
Rn_BS2=	617.7	kips
Rn_BS=	618	kips
φRn_BS=	463.3	kips
DCR_BS=	0.442	OK

(Rn_BS1,Rn_BS2)

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

Zbx=	130	in^3		Afg=	6.78	in^2
Zholes=	22.1	in^3	=2*Dholes*tbf*(d-tbf)	Afn=	5.37	in^2
Znet=	108	in^3	=Zbx - Zholes	Yt=	1	
Mpb_net=	5935	k-in	=Znet * Fy * Ry	Fu*Afn=	349	kips
Pr_link=	204.8	kips		Yt*Fu*Afn=	339	kips
d+ststem=	17.05	in		Sx=	117	in^3
Mpr_link=	3491	k-in		φ*Mn=	5850	k-in
DCR_NetSection=	0.588	=Mpr_link/Mpb_net		DCR_NetSection2=	0.597	=Mpr_link/φ*Mn

3.18 DESIGN SUMMARY:

	Design Sections							Max DCR	Check
	3.7	3.9	3.10	3.11	3.13 & 3.14	3.15 & 3.16	3.17		
t_BRP DCR	-	0.764	-	-	-	-	-	0.764	OK
Beam tf DCR	-	-	0.602	-	0.377	0.594	0.597	0.602	OK
BRP Bolt DCR	-	-	-	0.746	-	-	-	0.746	OK
Link Strength DCR	0.876	-	-	-	-	-	-	0.876	OK
Link Slip DCR	0.795	-	-	-	-	-	-	0.795	OK
Ly_yield DCR	0.651	-	-	-	-	-	-	0.651	OK
BRP Bolt Edge Dist>1d	-	-	-	-	OK	-	-	-	OK
Stem Bolt Edge Dist>1d	-	-	-	-	-	OK	-	-	OK
Overall Check=								OK	

3.1 CURRENT MEMBER:

Beam Unique Name:	2001	Link ID:	YLMC6-3.5	R=	3.5
Beam Size:	W16X67	Mu_max:	1327 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 (L_{col_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})=	8.00	in
NY Length BeamSide (L_{bm_side})=	11.25	in	Central Neck Yield Width (b_{yield})=	3.50	in
L_{stem} =	26.25	in	NY Width BeamSide (b_{bm_side})=	8.00	in
Link Radius (r_{link})=	0.75	in	Yielding Area ($A_{stemYield}$)=	2.63	in^2

3.3 LINK STEM BOLTS: (From YL Database)

Num. Bolts (n_{bolt_linkBm})=	6.00		Gauge Along Width (g_{stem})=	5.00	in
Bolt Type (Bolt_Gr_linkBm)=	A490		Spacing Along Length (s_{stem})=	3.38	in
Stem Bolt Dia=	1.00	in	First Bolt distance to Neck (S_o)=	2.50	in
Min. Bolt length =	3.000	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})=	8.00	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})=	5.00	in
Bolt Dia (boltD_linkCol)=	1.00	in			

3.6 LINK MATERIAL: (AISC 358, 12.8.2)

F_y_{link} =	50	ksi	R_y_{link} =	1.1	
F_u_{link} =	65	ksi	R_t_{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})=	8.525	in	$= (db + t_{stem})/2$
Link Extension (linkDelta)=	0.426	in	$= d_{arm} * r_{linkLimit}$
L_{yield_req} =	6.51	in	$= r_{linkLimit} * d_{arm} / 0.085 + 2 * r_{link}$ (EQ 12.9-4)
L_{yield} =	10.00	in	from previous value
L_{yield_DCR} =	0.65		$= L_{stem_yield_req} / L_{stemYield}$
Check=	OK		OK, if $L_{stem_DCR} < DCR_{allowed}$

Strength Check: Step 3

M_u =	1327	k-in
d =	16.3	in
$d+t_{stem}$ =	17.05	in
P_{u_link} =	77.84	kips
P_{y_link} =	131.25	kips
Link stem DCR=	0.659	OK

Krot:

Moment Demand (LRDF combos)
Beam Depth from database
Link Axial demand, $M_u/(d+t_{stem})$
Link Yield Strength from YL database
$= P_{u_link} / (0.9 * P_{y_link})$

AISC 358, 12.9 Step 11:

K1=	28602	k/in
K2=	18939	k/in
K3=	7612.5	k/in
K_{eff} =	4564	k/in
M_{ye} =	2461.59375	k*in
Δ_y =	0.0316365	in
θ_y =	0.00371103	rad
Krot=	663319	k*in/rad

Slip-Critical Check:

P_{u_slip} =	77.84	kips
ϕ =	1.0	
μ =	0.30	
D_u =	1.13	
h_f =	1.00	
n_s =	1.00	
T_b =	64.00	kips
n_{bolt_linkBm} =	6.000	
ϕR_n slip=	130.2	kips
Link slip DCR=	0.598	OK

$$= P_{u_link}$$

Minimum Bolt Pretension per Table J3.1

$$= \phi * \mu * D_u * h_f * T_b * n_s * n_{bolt_linkBm}$$

$$= P_{u_link} / \phi R_n_{slip}$$

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_cant=	6.500	in	=Lbrp - Lend - S _{BRP}
Lbrp=	12.750	in	from Database
Wbrp_min=	8.000	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=	204.75	kips	=if(R=3,Py_link, Pr_link)
Lelong=	0.426	in	Elongated distance at 0.05 rad
L1st_bolt=	3.75	in	Start of yielding region to 1st BRP bolt centerline
Lcant=	3.77	in	Lever arm from edge of yield link to edge of bolt hole + elongated dist at 0.05 rad
bn_brp=	6.25	in	Net width of BRP, without bolt holes
tbrp_min=	0.764	in	AISC 358, EQ 12.9-13
tbrp_use=	1.000	in	value previously defined
tbrp_DCR=	0.764		OK

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

Joint Rotation=	0.04	rad	
e _{0.04} =	0.04012	in/in	
g=	0.00752	in	gap between BRP and beam flange
ly=	0.123	in^4	
l _o =	1.071	in	effective buckling wave length
Qi=	5.75	kips	
N=	9.34		=LstemYield / l _o
N=	9.0		Round down to next integer
Ndesign=	5.0		N used for design (Contact points with BRP)
Tux=	28.8	kips	=Ndesign * Qi
Tux_bolt=	7.19	kips	=Tux/Nbolts
φ=	1.0		
Ledge=	6.500	in	1st bolt to edge of BRP
S_brpBolt_min=	5.25	in	=byield + 2*(0.125 + d_brp_bolt)
gbrp=	5.750	in	
b=	2.678	in	BRP bolt to face of web
c=	4.903	in	edge of beam flange to face of web
a=	2.225	in	center of BRP bolt to edge of beam flange
x=sqrt(b*c)=	3.623	in	
4*x=	14.49	in	1 row of BRP bolts:
2*x=	7.25	in	pe (in)= 14.49
S_brp_bolt=	2.500	in	2 rows of BRP bolts:
pe=	8.50	in	pe1 (in)= 8.50
b'=	2.303	in	pe2 (in)= 14.49
tf_min=	0.400	in	pe (in)= 8.50
tbtf=	0.665	in	3 rows of BRP bolts:
tbtf_DCR=	0.602		pe1 (in)= 8.50
			pe2 (in)= 3.06
			pe3 (in)= 8.50
			pe (in)= 6.68

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

I _x =	2.680	in ⁴	=byield^3 * tstem / 12, strong axis moment of inertia of reduced link region
gap=	0.188	in	gap between link and spacer plate
Vuy/Spacer=	26.68	kips	
Vuy per bolt=	13.342	kips	
	Tx (k)	Vx (kip)	Ty (kips)
Bolt Forces	7.19	2.157	0
			Vy (kips)
			13.342

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.	Y-Dir.
	DCR	DCR
Tension	0.24	0.00
Shear	0.121	0.746
RnV_(V+T), kips	17.89	17.89
T+V	0.121	0.746

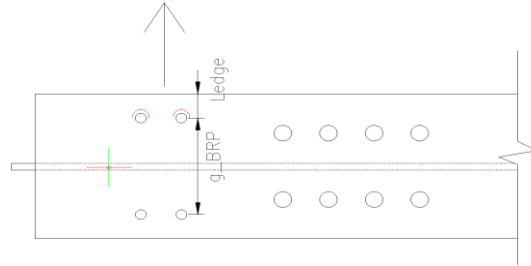
Max DCR= 0.746 OK

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

bbf=	10.20	in	= beam flange width
bbf_BRP_bolt=	7.750	in	=minimum flange width for BRP bolt spacing + min. bolt edge distance
bbf_stem_bolt=	7.500	in	=minimum flange width for stem bolt spacing + min. bolt edge distance
bbf_min=	7.75	in	=max (bbf_BRP_bolt, bbf_stem_bolt)
bf_DCR=	0.760		OK

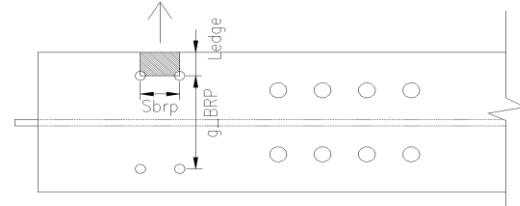
3.13 BRP BOLT BEAM FLANGE BEARING AND TEAROUT IN WEAK-AXIS DIRECTION (AISC 360, J3.10)

bbf=	10.2	in
g_brp=	5.75	in
d_brp=	0.750	in
Ledge=	2.225	in
Ledge_min=	1.00	in
Ledge>1 bolt Dia?	OK	
Ledge>=Ledge_min?	OK	
Vy=	13.34	kips/bolt
Rn_bearing=	77.805	kips
φRn_bearing=	58.35	kips
DCR_bearing=	0.229	OK
Lc=	2.225	in
Rn_tearout=	115.4	kips
φRn_tearout=	86.6	kips
DCR_tearout=	0.154	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=	2.225	in
tbf=	0.665	in
Vuy=	26.68	kips
Lgv=	4.45	in
Agv=	2.95925	in^2
Rn_yield=	88.8	kips
φRn_yield=	88.8	kips
DCR_bearing=	0.301	OK
Lnv=	3.638	in
Anv=	2.419	in^2
Rn_tearout=	94.3	kips
φRn_tearout=	70.75	kips
DCR_tearout=	0.377	OK



Lnt=	1.6875	in
Ant=	1.122	in^2
Rn_BS1=	167.28	kips
Rn_BS2=	161.72	kips
Rn_BS=	161.72	kips
φRn_BS=	121.29	kips
DCR_BS=	0.220	OK

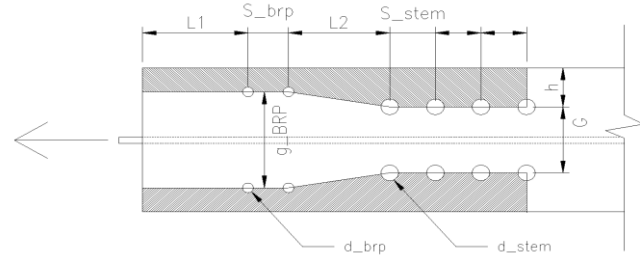
(Rn_BS1,Rn_BS2)

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

Pr_link=	204.75	kips	
tb_f=	0.665	in	Beam Flange thickness
bolt_Dia=	1	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=	2.3125	in	
Fy_bm=	50	ksi	from Design_Limits
Fu_bm=	65	ksi	from Design_Limits
Rn_bearing=	622.4	kips	$=2.4 \cdot \text{bolt_Dia} \cdot \text{tb_f} \cdot \text{Fu_bm} \cdot \text{n_bolts}$
$\phi Rn_bearing$ =	466.83	kips	$=0.75 \cdot Rn_bearing$
DCR_bearing=	0.439		OK
Rn_tearout=	719.7	kips	$=1.2 \cdot Lc \cdot \text{tb_f} \cdot \text{Fu_bm} \cdot \text{n_bolts}$
$\phi Rn_tearout$ =	539.8	kips	$=0.75 \cdot Rn_tearout$
DCR_tearout=	0.379		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=	5	in
gbrp=	5.75	in
d_brp=	0.8125	in
d_stem=	1.0625	in
bf=	10.2	in
h=	2.600	in
h_min=	1.25	in
h>1 bolt Dia?	OK	
h>=h_min?	OK	



Lgv=	22.00	in
Agv=	14.630	in ²
Rn_yield=	877.8	kips
ϕRn_yield =	658.4	kips
DCR_yield=	0.311	
Lnv=	17.72	in
Anv=	11.78	in ²
Rn_rupture=	459.5	kips
$\phi Rn_rupture$ =	344.7	kips
DCR_rupture=	0.594	

Lnt=	4.1375	in
Ant=	2.751	in ²
Rn_BS1=	638	kips
Rn_BS2=	617.7	kips
Rn_BS=	618	kips
ϕRn_BS =	463.3	kips
DCR_BS=	0.442	

(Rn_BS1,Rn_BS2)

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

Z _{bx} =	130	in ³		Afg=	6.78	in ²
Z _{holes} =	22.1	in ³	$=2 \cdot D_{holes} \cdot \text{tb_f} \cdot (d - \text{tb_f})$	Afn=	5.37	in ²
Z _{net} =	108	in ³	$=Z_{bx} - Z_{holes}$	Yt=	1	
M _{pb_net} =	5935	k-in	$=Z_{net} \cdot F_y \cdot R_v$	Fu*Afn=	349	kips
P _{r_link} =	204.8	kips		Yt*Fy*Afg=	339	kips
d+ _{tstem} =	17.05	in		Sx=	117	in ³
M _{pr_link} =	3491	k-in		$\phi \cdot Mn$ =	5850	k-in
DCR_NetSection=	0.588	$=M_{pr_link}/M_{pb_net}$		DCR_NetSection2=	0.597	$=M_{pr_link}/\phi \cdot Mn$

3.18 DESIGN SUMMARY:

	Design Sections							Max DCR	Check
	3.7	3.9	3.10	3.11	3.13 & 3.14	3.15 & 3.16	3.17		
t_BRP DCR	-	0.764	-	-	-	-	-	0.764	OK
Beam tf DCR	-	-	0.602	-	0.377	0.594	0.597	0.602	OK
BRP Bolt DCR	-	-	-	0.746	-	-	-	0.746	OK
Link Strength DCR	0.659	-	-	-	-	-	-	0.659	OK
Link Slip DCR	0.598	-	-	-	-	-	-	0.598	OK
Ly_yield DCR	0.651	-	-	-	-	-	-	0.651	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:	W24X84	Left Beam ID=	N/A	Right Beam ID=	1001
Pu (kips):	75.11	Left Beam size=	N/A	Right Beam size=	W16X67
Hcc t (in):	187	RightEndLink=	N/A	RightEndLink=	YES
Stiffener (Y/N):	YES	db_left (in)=	N/A	db_right (in)=	16.3
Ext. Stiffener depth=	Full Depth	Left Link Size=	N/A	Right Link Size=	YLMC6-3.5
Current Story=	Story1	Left Beam Lh (in):	N/A	Right Beam Lh (in):	251.10
Story Above=	Story2	Vu Gravity (kips):	0.00	Vu Gravity (kips):	3.84

5.2 LINK PROPERTIES: (From YL Database)

	Left Link	Right Link			
Thickness (t_stem) =	0.000	0.750	in	R=	3.5
Yield Width (b_yield)=	0.000	3.500	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	5.000	in	Pu_link=	103 kips
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
a=	0.000	3.500	in	Py_linkR=	131.25 kips
Pcap_Link=	0.0	103.4	kips	Pr_linkR=	204.75 kips
Mcap=	0	1764	k-in		

= if (R >= 6.5, Pr_link, use selection in "Design_Limits")
= Pcap_Link *(db+dstem)

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

	Left Side	Right Side			
Mcap=	0.00	1763.56	kip*in	Value previously defined	Column Size= AISC 360 Table B4.1b
Lh=	N/A	220.0	in		W24X84
Vbm_gravity=	0.00	3.84	kip		Col b/t= 5.86 Compact
Vu_bm=	0.00	19.88	kip	= n_sides* Mpr/ Lh + Vbm_gravity	Col h/tw= 45.9 Compact
Column Size=	W24X84	W24X84		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= 5.86 Hi.Ductile
Agc=	24.7	24.7	in^2	Looked up value	
Zcx=	224.00	224.00	in^3	Looked up value	Col h/tw= 45.9 Hi.Ductile
dc=	24.1	24.1	in	Looked up value	
tcf=	0.77	0.77	in	Looked up value	
Pu_colPositive=	75.11	75.11	kips	Max Axial Force of column per Overstrength combo	
Muv=	0.0	309.1	kips*in	= Vu_bm* (dc/2 + a)	
ΣMp_col=	21038	21038	kip*in	= Zcx* (Fyc - Pu_colPositive/ Agc) * if(top story,1 else 2)	
ΣMpb=	0	2073	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	0.099			
SCWB_DCR_total=	0.099	OK			

5.4 COLUMN NET SECTION CHECK

	Left Side	Right Side			
Mpc=	0	11200	kip*in	Zcx*Fyc	Ry= 1.1
Dflange=	0.000	1.000	in		Rt= 1.1
Dhole=	0.000	1.188	in	Dflange+3/16	Fu= 65 ksi
nBoltrow=	0	2		nlinkCol_bolts/2	Fy= 50 ksi
Znet=	0.000	198.727	in^3		
Znet_total=	198.727		in^3		Put = 53.58 kips
Mpc_net=	13743		kip*in	Znet*(Rt*Fu-Pu_tension/Anet)	Anet = 22.87 in^2
Pr_link=	0	204.75	kip	Ay_link*Fu_link*Rt_link	
Mpr_link=	0	1764	kip*in	Value previously defined	
DCR_ColNetSect=	0.128	OK			

5.5 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=	24.10	in	Value previously defined		
tcw=	0.47	in	Value previously defined		
Agc=	24.70	in^2	Value previously defined		
Pc=	1235	kips	= Fyc* Agc		
Pu=	75.11	kips	Value previously defined		
φRn_PZ=	305.83	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	10.56	kips	= ΣMpb/((Hcc_b+Hcc_t)/2), 0 for top story	
Pcap_Link=	0.00	103.43	kips	= Pu_link	
Ru=	0.00	92.87	kips		
DCR_PZ=	0.000	0.304		= Pr_link - Vu_c / φRn_PZ	
DCR_PZ_total=	0.304	OK		Ok, if DCR_PZ <= DCR_PZ_Limit	

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

Doubler Plate Required=	NO	<----If NO, Skip 5.4.1	USE Plug Weld?	NO
Fy_dpl=	50	ksi	Doubler Plate Yield Strength	
ϕRn_PZ_NODBLR =	305.83	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 capacity required	
ϕRn_PZ_NEW =	305.8	kips	= $\phi Rn_PZ_NODBLR + \phi Rn_PZ_req$	
t_dblr_strength=	0.000	in	Doubler Plate thickness	
t_dblr_use=	0.000	in	= ϕv_pz if ($Pu < 0.4 * Pc$, $0.6 * (Fy_col * dc * tcw + Fy_dpl * (dc - 2 * tcf) * t_dblr)$, $0.6 * (Fy_col * dc * tcw + Fy_dpl * (dc - 2 * tcf) * t_dblr) * (1.4 - Pu/Pc)$)	
ϕRn_PZ_DBLR =	305.83	kips		
	Left Side	Right Side		
DCR_PZ_NEW=	0.000	0.304		
DCR_PZ_total_NEW=	0.304	OK		
Max Doubler Thickness	1.250	in	Maximum thickness of doubler plate to not interfere with YL column bolts	
DCR Doubler Plate Thickness	3.000	NG	Use CJP Weld or choose another link size with wider flange	
DCR_PZ_Total=	3.000		=max of DCR_PZ (strength), DCR_PZ (DP_Thickness)	

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

Option 1: Doubler without Continuity plates

t_dblr=	0.000	in	doubler plate thickness
tcw=	0.47	in	column web thickness
min t=	0.000	in	=min of tDP and tcw, rounded up to nearest 1/16
willet=	0	in	From AISC 360 Table J2.4

Option 2A: Extended Doubler Plate

t_dblr=	0.000	in	Continuity plate to doubler per Section E3.6f.2(c)
willet=	0.0000	in	doubler plate thickness (c.3, available strength of doubler plate)
willet_use=	0.0000	in	=5/8*tDP (develop shear strength of the doubler Plate), rounded to nearest 1/16
# 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=	0.000	in	doubler plate thickness
willet_use=	0.0000	in	t_dblr, same thickness to develop doubler Plate capacity
# of sides=	1		

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

Doubler Plate Plug-Weld Used?	NO		
PlugWeld_Dia=	0.313	in	Weld Depth= 0.000 in

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

Option 1: Fillet weld to develop doubler plate shear capacity

t_dblr=	0.000	in	doubler plate thickness
Vn=	0.00	kips	$0.6 * Fy * Agv$ (per 1 in length)
willet=	0.0	/16	Vn/1.39, rounded up to 1/16"

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

5.6 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	16.3	in d beam
tbf=	N/A	0.665	in
d_stiff_cap=	N/A	3.5	in
d_stiffTop=	48.2	48.2	in Location of top stiffener to top of column cap, assume 2 dc if not at top story
d_stiffBot=	N/A	64.5	in Location of bottom stiffener to top of column cap = d + d_stiffTop

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

	Left Side	Right Side	
ϕWLY =	1	1	Value previously defined
dc=	24.1	24.1	in Looked up value
kdes=	1.27	1.27	in Link flange thickness
t_flange=	0.000	1.250	in Link stem thickness
t_stem=	0.000	0.750	in = 2 * t_flange + t_stem
l_b=	0	3.25	in = if (d_stiffTop <= dc, 0.5, 1)
Ct_top=	1	1	in = if (d_botTop <= dc, 0.5, 1)
Ct_bot=	1	1	in = $\phi WLY * Fy_col * tcw * (5 * kdes + l_b)$
$\phi Rn_WLYmoreD$ =	149.23	225.60	kips = $\phi WLY * Fy_col * tcw * (2.5 * kdes + l_b)$
$\phi Rn_WLYlessD$ =	74.61	150.99	kips = if (d_stiffTop > dc, fRn_WLYmoreD, fRn_WLYlessD)
ϕRn_WLY_top =	149.23	225.60	kips = if (d_stiffTop > dc, fRn_WLYmoreD, fRn_WLYlessD)
ϕRn_WLY_bot =	149.23	225.60	kips

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

$\phi WLC =$	0.75		
$E_s =$	29000	ksi	
$t_{cf} =$	0.77	in	Value previously defined
$tcw =$	0.47	in	Value previously defined
$\phi R_n \text{ WLC more}0.5D =$	243.7	kip	$= \phi WLC * 0.8 * tcw^2 * (1 + 3 * (L_b/dc) * (tcw/tcf)^{1.5}) * \text{Sqrt}(E_s * F_y \cdot col * tcf / tcw)$
AISC J10-5a =	121.85	kips	$= 0.5 * \phi R_n \text{ WLC more}0.5D$
AISC J10-5b =	118.67	kips	$= \phi WLC * 0.4 * tcw^2 * (1 + (4 * L_b/dc * 0.2) * (tcw/tcf)^{1.5}) * \text{Sqrt}(E_s * F_y \cdot col * tcf / tcw)$
$\phi R_n \text{ WLC less}0.5D =$	121.85	kips	$= \text{if } (L_b/dc \leq 0.2, \text{AISC J10-5a, AISC J10-5b})$
	Left Side	Right Side	
$\phi R_n \text{ WLC top} =$	243.69	243.7	kips
$\phi R_n \text{ WLC bot} =$	243.7	243.7	kips
			$= \text{if } (d_stiffTop > dc/2, \phi R_n \text{ WLC more}0.5D, \phi R_n \text{ WLC less}0.5D)$
			$= \text{if } (d_stiffBot > dc/2, \phi R_n \text{ WLC more}0.5D, \phi R_n \text{ WLC less}0.5D)$

5.9 COLUMN WEB COMPRESSION BUCKLING (AISC 360 J10.5):

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

$\phi WCB =$	0.9		
$h =$	21.56	in	$= dc - 2 * kdes$
$\phi R_n \text{ WCB} =$	125.3	kips	$\text{AISC J10-8, } = \phi WCB * 24 * tcw^3 * \text{sqrt}(E_s * F_y \cdot col) / h$
	Left Side	Right Side	
$\phi R_n \text{ WCB top} =$	62.63	125.25	kips
$\phi R_n \text{ WCB bot} =$	125.25	125.25	kips
			$= \text{if } (d_stiffTop < d, \phi R_n \text{ WCB} * 0.5, \phi R_n \text{ WCB})$
			$= \text{if } (d_stiffBot < d, \phi R_n \text{ WCB} * 0.5, \phi R_n \text{ WCB})$

5.10 SUMMARY OF LOCAL CAPACITIES:

	Left Side				At Bottom Stiffener		
At Top Stiffener							
$\phi R_n \text{ WLY top} =$	149.23	kips			$\phi R_n \text{ WLY bot} =$	149.23	kips
$\phi R_n \text{ WLC top} =$	243.69	kips			$\phi R_n \text{ WLC bot} =$	243.7	kips
$\phi R_n \text{ WCB top} =$	62.63	kips			$\phi R_n \text{ WCB bot} =$	125.25	kips
$\text{if } (SMF_Sides = 1, \phi R_n \text{ Stiff top} = \min(\phi R_n \text{ WLY top}, \phi R_n \text{ WLC top}, \phi R_n \text{ WCB top}), \phi R_n \text{ Stiff bot} = \min(\phi R_n \text{ WLY bot}, \phi R_n \text{ WLC bot}, \phi R_n \text{ WCB bot}))$							
$\phi R_n \text{ Stiff top} =$	149.23	kips			$\phi R_n \text{ Stiff bot} =$	149.23	kips
Top Stiffener Required =	No				Bottom Stiffener Required =	No	
Top Stiff Provided?:	YES				Bottom Stiff Provided?:	YES	
					Stiff Req'd =	NO	
	Right Side				At Bottom Stiffener		
At Top Stiffener							
$\phi R_n \text{ WLY top} =$	225.60	kips			$\phi R_n \text{ WLY bot} =$	225.60	kips
$\phi R_n \text{ WLC top} =$	243.69	kips			$\phi R_n \text{ WLC bot} =$	243.7	kips
$\phi R_n \text{ WCB top} =$	125.25	kips			$\phi R_n \text{ WCB bot} =$	125.25	kips
$\text{if } (SMF_Sides = 1, \phi R_n \text{ Stiff top} = \min(\phi R_n \text{ WLY top}, \phi R_n \text{ WLC top}, \phi R_n \text{ WCB top}), \phi R_n \text{ Stiff bot} = \min(\phi R_n \text{ WLY bot}, \phi R_n \text{ WLC bot}, \phi R_n \text{ WCB bot}))$							
$\phi R_n \text{ Stiff top} =$	225.60	kips			$\phi R_n \text{ Stiff bot} =$	225.60	kips
Top Stiffener Required =	No				Bottom Stiffener Required =	No	
Top Stiff Provided?:	YES				Bottom Stiff Provided?:	YES	
					Stiff Req'd =	NO	

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

Note: Assume compression flange is NOT restrained against rotation

	Left Side	Right Side		
$\phi WSB =$	0.85	0.85		
$H_{cc} =$	147.20	147.20	in	Value previously defined
$b_{cf} =$	9.02	9.02	in	Column flange width
$Sc_x =$	196.00	196.0	in^3	Looked up value
$L_b =$	N/A	139.1	in	$= H_{cc} - d/2$
$M_y =$	9800.0	9800.0	kips*in	$= Sc_x * F_{yc}$
$M_u =$	0	1764	kips*in	$= M_{pr}$
$C_r =$	960000	960000	ksi	$= \text{if } (M_u < M_y, 960000, 480000)$
$(h/tw)/(L_b/b_{cf}) =$	N/A	2.98		$= (h/tw)/(L_b/b_{cf})$
$\phi R_n \text{ WSB} =$	N/A	1479	kips	$\text{AISC J10-7, } = (\phi WSB * C_r * tcw^3 * t_{cf} / h^2) / (0.4 * C_r^3)$
WSB_check =	N/A	N/A		Not applicable
BotStiffener_Bracing_Req'd =	No	No		$= \text{if } (And(\phi R_n \text{ WSB} < Pr_link, \text{WSB_check} = \text{"Applicable"}), \text{"Yes"}, \text{"No"})$

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

	Left Side	Right Side		
$F_{su} \text{ top} =$	0.0	0.0	kips	$= \text{if } (Pr_link - \phi R_n \text{ Stiff top} < 0, 0, Pr_link - \phi R_n \text{ Stiff top})$
$F_{su} \text{ bot} =$	0.0	0.0	kips	$= \text{if } (Pr_link - \phi R_n \text{ Stiff bot} < 0, 0, Pr_link - \phi R_n \text{ Stiff bot})$
$F_{su} =$	0.0	0.0	kips	$= \max(F_{su} \text{ top}, F_{su} \text{ bot}) * SMF_Sides$

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

$\phi t =$	0.9			Value previously defined
$F_y \text{ stiff} =$	50	ksi		Value previously defined
$K_{det} =$	1.688	in		Looked up value
$K_1 =$	1.063	in		Looked up value
$tcw_{det}/2 =$	0.250	in		Looked up value
$t_{cf}(det) =$	0.750	in		Looked up value
	Left Side	Right Side		
$As \text{ min} =$	0.000	0.00	in^2	$= F_{su} / (t * F_y \text{ Stiff})$
$L_{clip} \text{ web min} =$	2.438	2.438	in	$= K_{det} - t_{cf} + 1.5$
$L_{clip} \text{ web} =$	2.438	2.438	in	$= \text{Ceiling}(L_{clip} \text{ web min}, 0.0625)$
$L_{clip} \text{ flange min} =$	0.813	0.813	in	$= K_1 - tcw_{det}/2$
$L_{clip} \text{ flange} =$	0.813	0.813	in	$= \text{Ceiling}(L_{clip} \text{ flange min}, 0.0625)$
$bstiff \text{ min} =$	2.7717	2.7717	in	$\text{AISC 360 J10.8, } = b_{cf}/3 - tcw/2$
$bstiff \text{ max} =$	4.275	4.275	in	$= (b_{cf} - tcw)/2$
$bstiff =$	4.2500	4.2500	in	$= \text{Floor}(bstiff \text{ max}, 0.125), \text{ Each side of column web}$
$tstiff \text{ USE?} =$	YES	YES	in	Input value
$t \text{ stiff tension} =$	0.000	0.000	in	$= As \text{ min} / (bstiff \text{ min} * L_{clip} \text{ flange}) / 2, SMF_Sides, OK, \text{ if } t \text{ stiff tension} \leq tstiff$
$tstiff \text{ min} =$	0.000	0.375	in	$= \max(t \text{ stem}/2, bstiff/16, \text{looked up value}), OK, \text{ if } tstiff \text{ min} \leq tstiff$
$tstiff \text{ max} =$	0.000	0.750	in	$= t \text{ stem}, OK, \text{ if } tstiff \text{ max} \geq tstiff$
$t \text{ stiff Use} =$	0.375	0.375		
$Lstiff \text{ Pdepth ini} =$	11.28		in	$= 0.5 * dc - t_f, \text{ For Single sided connections (partial depth)}$
$Lstiff \text{ Fdepth ini} =$	22.56		in	$= dc - 2 * t_{cf}, \text{ For double sided connections}$
$Lstiff \text{ ini} =$	22.56		in	$= \text{if } (and(SMF_Sides = 1, \text{Ext. Stiffener depth} = \text{"Partial"}), Lstiff \text{ Pdepth ini}, Lstiff \text{ Fdepth ini})$
$Lstiff =$	22.50		in	$= \text{Floor}(Lstiff \text{ ini}, 0.0625)$

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

	Left Side	Right Side		
ϕ_c =	0.9	0.9		
Kstiff=	0.75	0.75		
Astiff_fd=	5.84	5.84	in ²	= 2* bstiff* tstiff + 12* tcw ²
Lstiff_fd=	22.60	22.60	in ⁴	= 12* tcw ³ tcw ³ /12 + 2* (tstiff* bstiff ³ /3/ 12+ bstiff* tstiff* (bstiff/2 +tc/2) ²)
rstiff_fd=	1.97	1.97	in	= Sqrt (Lstiff_fd/ Astiff_fd)
Kstiff*Lstiff_fd/rstiff_fd=	8.58	8.58		= Kstiff* Lstiff_fd/ rstiff_fd
Fe=	3891	3891	ksi	= Pi ² * 29000/ (Kstiff* Lstiff_fd/ rstiff_fd) ²
Fcr=	49.73	49.73	ksi	= if (Kstiff* Lstiff_fd/ rstiff_fd <= 4.71* Sqrt (29000/ Fy_stiff), 0.658* (Fy_stiff/ Fc)* Fy_stiff,
Pn_stiff=	291.9	291.9	kips	= Kstiff* Lstiff_fd* rstiff_fd > 25, Fcr* Astiff_fd, Astiff_fd* Fy_stiff
ϕ_c *Pn_stiff=	262.7	262.7	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

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

ϕ_{fillet} =	0.75		
Fexx=	70	ksi	
tstiff=	0.375	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=	0.0000	0.0000	in
wfillet_flange=	0.1875	0.1875	in
wfillet_flange_DCR=	0.000	0.000	

= (0.5* Fsu* SMF_Sides)/ (ϕ_{fillet} 1.5* 0.6* Fexx* (bstiff - Lclip_flange-0.5)*2^{1/2})
=Max (wfillet_min, ceiling(wfillet_flange_min, 1/16"))
= wfillet_flange_min/ wfillet_flange

5.16 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
wfillet_min_web=	0.0000	0.0000	in
wfillet_web=	0.1875	0.1875	in
wfillet_web_DCR=	0.000	0.000	

Minimum fillet weld size per AISC 360 Table J2.4
= 0.5* Fsu* SMF_Sides/ (ϕ_{fillet} 0.6* Fexx* (Lstiff - Lclip_web-0.5)*2^{1/2})
=max(wfillet_min, ceiling(wfillet_web, 1/16"))

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

	Left Side	Right Side	
bcf=	9.0	9.0	in
bcf_LinkFlg_bolt=	0	7.5	in
bcf_DCR=	0.000	0.831	OK

=column flange width
=Link Sflange+2*Ledge_min

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

	Left Side	Right Side	
ϕ_b =	0.9	0.9	
tcf=	0.77	0.77	in
c=	0.000	5.750	in
g=	0.000	5.000	in
H_flange=	0.000	9.250	in
s=	0.000	3.358	in
ps0=	0.000	2.313	in
psi_tmp=	0.000	2.313	in
psi=	0.000	2.313	in
h1=	0	14.55	in
h0=	0	19.55	in
Yc_unstiffened=	0.00	137.05	in
tcf_req_unstiffened=	0.00	0.56	in
Yc_stiffened=	0.00	189.65	in
tcf_req_stiffened=	0.00	0.48	in
Stiffened?=	YES	YES	
tcf_min=	0.000	0.477	in
DCR_colFLB1=	N/A	0.619	

Value previously defined
= bolt_g_flange
= bolt_s_flange
Value previously defined
= 0.5* Sqrt (bcf* g)
= (g - tstiff)/2
= ps0
= if (psi_tmp > s, s, psi_tmp)
= db + t_stem - g/2
= db + t_stem + g/2
= bcf/2* (h1/s + h0/s) + 2/g* (h1* (s + 3*c/4) + h0* (s + c/4) + c²/4) + g/2
= Sqrt (1.1* Mcap/ (ϕ_b * Fyc* Yc_unstiffened))
= bcf/2* (h1* (1/s + 1/psi) + h0* (1/s + 1/ps0)) + 2/g* (h1* (s + psi) + h0* (s + ps0))
= Sqrt (1.1* Mcap/ (ϕ_b * Fyc* Yc_stiffened))
Bottom Stiffener Required per 17.3
= if (Stiffened? = YES, tcf_req_stiffened, tcf_req_unstiffened)
= tcf_min/ tcf

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

	Left Side	Right Side	
de=	1.25	1.25	in
pfi=	0.000	2.313	in
ps0=	0.000	2.313	in
Yp=	0.0	160.036	in
tcf_req=	0.000	0.519	in
DCR_colFLB2=	N/A	0.674	

centerline of top link bolt to top of column
= (g - tstiff)/ 2
Value previously defined
= bcf/2* (h1* (1/pfi + 1/ps0) + h0* (1/ps0 + 1/2/s)) + 2/g* (h1* (pfi + s) + h0* (d2 + ps0))
= Sqrt* (1.1* Mcap/ (ϕ_b * Fy* Yp))
= tcf_req/ tcf

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

	Left Side	Right Side	
Link Size=	N/A	YLMC6-3.5	
byield=	0.000	3.500	in
tstem=	0.000	0.750	in
Fy_link=	50	50	ksi
Ry=	1.1	1.1	
Pye_link=	0	144.375	kips
0.02 * Pye_link=	0.000	2.888	kips
Pbrace (ASD)=	0.000	2.021	kips

AISC 341, E3.4.c.1(b)
Bracing Force (LRFD)
= 0.7*Bracing Force (LRFD)

5.19 DESIGN SUMMARY:

DCR Summary:		Report:
SCWB_DCR_total=	0.099	0.099
DCR_PZ_total=	0.304	Overall PZ chk: NG
DCR_PZ_total_NEW=	3.000	NO 0.304
DCR_stiff_comp_DCR=	0.000	0.000 0.000
wfillet_flange_DCR=	0.000	0.000 0.000
wfillet_web_DCR=	0.000	0.000 0.000
DCR_colFLB1=	N/A	0.619 0.619
DCR_colFLB2=	N/A	0.674 0.674
DCR_colFLB=		0.619

Geometry/Weld Summary:		Report:
DP used?	NO	PlugWeld? NO
t_dblr_use=	0.000	0.000
Stiffener Req'd=	NO	NO
t_stiff_Use=	0.375	0.375
wfillet_flange=	0.188	0.188 0.188
wfillet_web=	0.188	0.188 0.188

5.1 CURRENT MEMBER:

Column ID:	2	Top Story=	Story2	Use Doubler PL? =	NO
Column size:	W24X84	Left Beam ID=	1001	Right Beam ID=	N/A
Pu (kips):	75.09	Left Beam size=	W16X67	Right Beam size=	N/A
Hcc t (in):	187	RightEndLink=	YES	RightEndLink=	N/A
Stiffener (Y/N):	YES	db_left (in)=	16.3	db_right (in)=	N/A
Ext. Stiffener depth=	Full Depth	Left Link Size=	YLMC6-3.5	Right Link Size=	N/A
Current Story=	Story1	Left Beam Lh (in):	251.10	Right Beam Lh (in):	N/A
Story Above=	Story2	Vu Gravity (kips):	3.84	Vu Gravity (kips):	0.00

5.2 LINK PROPERTIES: (From YL Database)

	Left Link	Right Link			
Thickness (t_stem) =	0.750	0.000	in	R=	3.5
Yield Width (b_yield)=	3.500	0.000	in	Fy_link=	50 ksi
Thickness (t_flange)	1.250	0.000	in	Fu_link=	65 ksi
Spacing Along Length (horiz) (bolt_s_flange)	5.000	0.000	in	Pu_link=	103 kips
Gauge Along Width (vertical) (bolt_g_flange)=	5.750	0.000	in	Py_linkL=	131.25 kips
Flange height (H_flange) =	9.250	0.000	in	Pr_linkL=	204.75 kips
a=	3.500	0.000	in	Py_linkR=	0 kips
Pcap_Link=	103.4	0.0	kips	Pr_linkR=	0 kips
Mcap=	1764	0	k-in		

= if (R >= 6.5, Pr_link, use selection in "Design_Limits")
= Pcap_Link *(db+dstem)

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

	Left Side	Right Side			
Mcap=	1763.56	0.00	kip*in	Value previously defined	AISC 360 Table B4.1b
Lh=	220.0	N/A	in		Column Size= W24X84
Vbm_gravity=	3.84	0.00	kip		Col b/t= 5.86 Compact
Vu_bm=	19.88	0.00	kip	= n_sides* Mpr/ Lh + Vbm_gravity	Col h/tw= 45.9 Compact
Column Size=	W24X84	W24X84		Value previously defined	
Fyc=	50	50	ksi	Value previously defined	AISC 341 Table D1.1
a=	3.50	0.00	in	Value previously defined	Col b/t= 5.86 Hi.Ductile
Agc=	24.7	24.7	in^2	Looked up value	
Zcx=	224.00	224.00	in^3	Looked up value	Col h/tw= 45.9 Hi.Ductile
dc=	24.1	24.1	in	Looked up value	
tcf=	0.77	0.77	in	Looked up value	
Pu_colPositive=	75.09	75.09	kips	Max Axial Force of column per Overstrength combo	
Muv=	309.1	0.0	kip*in	= Vu_bm* (dc/2 + a)	
ΣMp_col=	21038	21038	kip*in	= Zcx* (Fyc - Pu_colPositive/ Agc) * if(top story, 1 else 2)	
ΣMpb=	2073	0	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=	0.099	0.000			
SCWB_DCR_total=	0.099	OK			

5.4 COLUMN NET SECTION CHECK

	Left Side	Right Side			
Mpc=	11200	0	kip*in	Zcx*Fyc	Ry= 1.1
Dflange=	1.000	0.000	in		Rt= 1.1
Dhole=	1.188	0.000	in	Dflange+3/16	Fu= 65 ksi
nBoltrow=	2	0		nlinkCol_bolts/2	Fy= 50 ksi
Znet=	198.727	0.000	in^3		
Znet_total=	198.727		in^3		Put = 53.60 kips
Mpc_net=	13743		kip*in	Znet*(Rt*Fu-Pu_tension/Anet)	Anet = 22.87 in^2
Pr_link=	204.75	0	kip	Ay_link*Fu_link*Rt_link	
Mpr_link=	1764	0	kip*in	Value previously defined	
DCR_ColNetSect=	0.128	OK			

5.5 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=	24.10	in		Value previously defined	
tcw=	0.47	in		Value previously defined	
Agc=	24.70	in^2		Value previously defined	
Pc=	1235	kips		= Fyc* Agc	
Pu=	75.09	kips		Value previously defined	
φRn_PZ=	305.83	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=	10.56	0.00	kips	= ΣMpb/((Hcc_b+Hcc_t)/2), 0 for top story	
Pcap_Link=	103.43	0.00	kips	= Pu_link	
Ru=	92.87	0.00	kips		
DCR_PZ=	0.304	0.000		= Pr_link - Vu_c / φRn_PZ	
DCR_PZ_total=	0.304	OK		Ok, if DCR_PZ <= DCR_PZ_Limit	

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

Doubler Plate Required=	NO	<----If NO, Skip 5.4.1	USE Plug Weld?	NO
Fy_dpl=	50	ksi	Doubler Plate Yield Strength	
ϕR_n _PZ_NODBLR=	305.83	kips	= ϕv_{pz} if ($P_u < 0.4 \cdot P_c$, $0.6 \cdot F_y \cdot c_{ol} \cdot d_c \cdot t_{cw}$, $0.6 \cdot F_y \cdot c_{ol} \cdot d_c \cdot t_{cw} \cdot (1.4 - P_u/P_c)$)	
ϕR_n _PZ_req=	0.00	kips	Additional capacity required	
ϕR_n _PZ_NEW=	305.8	kips	= ϕR_n _PZ_NODBLR + ϕR_n _PZ_req	
t_dblr_strength=	0.000	in	Doubler Plate thickness	
t_dblr_use=	0.000	in	= ϕv_{pz} if ($P_u < 0.4 \cdot P_c$, $0.6 \cdot (F_y \cdot c_{ol} \cdot d_c \cdot t_{cw} + F_y \cdot d_{pl} \cdot (d_c - 2 \cdot t_{cf}) \cdot t_{dblr})$, $0.6 \cdot (F_y \cdot c_{ol} \cdot d_c \cdot t_{cw} + F_y \cdot d_{pl} \cdot (d_c - 2 \cdot t_{cf}) \cdot t_{dblr}) \cdot (1.4 - P_u/P_c)$)	
ϕR_n _PZ_DBLR=	305.83	kips		
	Left Side	Right Side		
DCR_PZ_NEW=	0.304	0.000		
DCR_PZ_total_NEW=	0.304	OK		
Max Doubler Thickness	1.250	in	Maximum thickness of doubler plate to not interfere with YL column bolts	
DCR Doubler Plate Thickness	3.000	NG	Use CJP Weld or choose another link size with wider flange	
DCR_PZ_Total=	3.000		=max of DCR_PZ (strength), DCR_PZ (DP_Thickness)	

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

Option 1: Doubler without Continuity plates

t_dblr=	0.000	in	doubler plate thickness
t_cw=	0.47	in	column web thickness
min t=	0.000	in	=min of tDP and t_cw, rounded up to nearest 1/16
willet=	0	in	From AISC 360 Table J2.4

Option 2A: Extended Doubler Plate

t_dblr=	0.000	in	Continuity plate to doubler per Section E3.6f.2(c)
willet=	0.0000	in	doubler plate thickness (c.3, available strength of doubler plate)
willet_use=	0.0000	in	=5/8*tDP (develop shear strength of the doubler Plate), rounded to nearest 1/16
# 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=	0.000	in	doubler plate thickness
willet_use=	0.0000	in	t_dblr, same thickness to develop doubler Plate capacity
# of sides=	1		

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

Doubler Plate Plug-Weld Used?	NO		
PlugWeld_Dia=	0.313	in	Weld Depth= 0.000 in

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

Option 1: Fillet weld to develop doubler plate shear capacity

t_dblr=	0.000	in	doubler plate thickness
Vn=	0.00	kips	$0.6 \cdot F_y \cdot A_{gv}$ (per 1 in length)
willet=	0.0	/16	Vn/1.39, rounded up to 1/16"

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

5.6 CHECK UNSTIFFENED COLUMN FLANGE AND WEB:

SMF_Sides=	1		(1 for singled sided SMF, 2 for double sided SMF)
	Left Side	Right Side	
dbm=	16.3	N/A	in d beam
tbf=	0.665	N/A	
d_stiff_cap=	3.5	N/A	
d_stiffTop=	48.2	48.2	in Location of top stiffener to top of column cap, assume 2 dc if not at top story
d_stiffBot=	64.5	N/A	in Location of bottom stiffener to top of column cap = d + d_stiffTop

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

	Left Side	Right Side	
ϕWLY =	1	1	
dc=	24.1	24.1	in Value previously defined
kdes=	1.27	1.27	in Looked up value
t_flange=	1.250	0.000	in Link flange thickness
t_stem=	0.750	0.000	in Link stem thickness
l_b=	3.25	0	in = 2* t_flange + t_stem
Ct_top=	1	1	in = if (d_stiffTop <= dc, 0.5, 1)
Ct_bot=	1	1	in = if (d_botTop <= dc, 0.5, 1)
ϕR_n WLYmoreD=	225.60	149.23	kips = $\phi WLY \cdot F_y \cdot c_{ol} \cdot t_{cw} \cdot (5 \cdot kdes + l_b)$
ϕR_n WLYlessD=	150.99	74.61	kips = $\phi WLY \cdot F_y \cdot c_{ol} \cdot t_{cw} \cdot (2.5 \cdot kdes + l_b)$
ϕR_n WLY_top=	225.60	149.23	kips = if (d_stiffTop > dc, fRn_WLYmoreD, fRn_WLYlessD)
ϕR_n WLY_bot=	225.60	149.23	kips = if (d_stiffTop > dc, fRn_WLYmoreD, fRn_WLYlessD)

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

$\phi WLC =$	0.75			
$E_s =$	29000	ksi		
$t_{cf} =$	0.77	in	Value previously defined	
$tcw =$	0.47	in	Value previously defined	
$\phi R_n \text{ WLC more}0.5D =$	243.7	kip	$= \phi WLC * 0.8 * tcw^2 * (1 + 3 * (l_b/dc) * (tcw/tcf)^{1.5}) * \text{Sqrt}(E_s * F_y_col * tcf / tcw)$	
AISC J10-5a =	121.85	kip	$= 0.5 * \phi R_n \text{ WLC more}0.5D$	
AISC J10-5b =	118.67	kip	$= \phi WLC * 0.4 * tcw^2 * (1 + (4 * l_b/dc * 0.2) * (tcw/tcf)^{1.5}) * \text{Sqrt}(E_s * F_y_col * tcf / tcw)$	
$\phi R_n \text{ WLC less}0.5D =$	121.85	kip	$= \text{if } (l_b/dc <= 0.2, \text{AISC J10-5a, AISC J10-5b})$	
	Left Side	Right Side		
$\phi R_n \text{ WLC top} =$	243.69	243.7	kip	$= \text{if } (d_stiffTop > dc/2, \phi R_n \text{ WLC more}0.5D, \phi R_n \text{ WLC less}0.5D)$
$\phi R_n \text{ WLC bot} =$	243.7	243.7	kip	$= \text{if } (d_stiffBot > dc/2, \phi R_n \text{ WLC more}0.5D, \phi R_n \text{ WLC less}0.5D)$

5.9 COLUMN WEB COMPRESSION BUCKLING (AISC 360 J10.5):

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

$\phi WCB =$	0.9								
$h =$	21.56	in	$= dc - 2 * kdes$						
$\phi R_n \text{ WCB} =$	125.3	kip	$\text{AISC J10-8, } = \phi WCB * 24 * tcw^3 * \text{sqrt}(Es^* Fy_col) / h$						
	<table><tr><th>Left Side</th><th>Right Side</th></tr><tr><td>125.25</td><td>62.63</td></tr><tr><td>125.25</td><td>125.25</td></tr></table>	Left Side	Right Side	125.25	62.63	125.25	125.25		
Left Side	Right Side								
125.25	62.63								
125.25	125.25								
$\phi R_n \text{ WCB top} =$	125.25	62.63	kip	$= \text{if } (d_stiffTop < d, \phi R_n_WCB * 0.5, \phi R_n_WCB)$					
$\phi R_n \text{ WCB bot} =$	125.25	125.25	kip	$= \text{if } (d_botTop < d, \phi R_n_WCB * 0.5, \phi R_n_WCB)$					

5.10 SUMMARY OF LOCAL CAPACITIES:

	Left Side				At Bottom Stiffener		
At Top Stiffener							
$\phi R_n \text{ WLY top} =$	225.60	kip			$\phi R_n \text{ WLY bot} =$	225.60	kip
$\phi R_n \text{ WLC top} =$	243.69	kip			$\phi R_n \text{ WLC bot} =$	243.7	kip
$\phi R_n \text{ WCB top} =$	125.25	kip			$\phi R_n \text{ WCB bot} =$	125.25	kip
$\text{if } (SMF_Sides = 1, \phi R_n \text{ Stiff top} = \min(\phi R_n \text{ WLY top}, \phi R_n \text{ WLC top}, \phi R_n \text{ WCB top}))$							
$\text{if } (SMF_Sides = 1, \phi R_n \text{ Stiff bot} = \min(\phi R_n \text{ WLY bot}, \phi R_n \text{ WLC bot}, \phi R_n \text{ WCB bot}))$							
$\phi R_n \text{ Stiff top} =$	225.60	kip			$\phi R_n \text{ Stiff bot} =$	225.60	kip
Top Stiffener Required =	No				Bottom Stiffener Required =	No	
Top Stiff Provided?:	YES				Bottom Stiff Provided?:	YES	
					Stiff Req'd =	NO	
	Right Side				At Bottom Stiffener		
At Top Stiffener							
$\phi R_n \text{ WLY top} =$	149.23	kip			$\phi R_n \text{ WLY bot} =$	149.23	kip
$\phi R_n \text{ WLC top} =$	243.69	kip			$\phi R_n \text{ WLC bot} =$	243.7	kip
$\phi R_n \text{ WCB top} =$	62.63	kip			$\phi R_n \text{ WCB bot} =$	125.25	kip
$\text{if } (SMF_Sides = 1, \phi R_n \text{ Stiff top} = \min(\phi R_n \text{ WLY top}, \phi R_n \text{ WLC top}, \phi R_n \text{ WCB top}))$							
$\text{if } (SMF_Sides = 1, \phi R_n \text{ Stiff bot} = \min(\phi R_n \text{ WLY bot}, \phi R_n \text{ WLC bot}, \phi R_n \text{ WCB bot}))$							
$\phi R_n \text{ Stiff top} =$	149.23	kip			$\phi R_n \text{ Stiff bot} =$	149.23	kip
Top Stiffener Required =	No				Bottom Stiffener Required =	No	
Top Stiff Provided?:	YES				Bottom Stiff Provided?:	YES	
					Stiff Req'd =	NO	

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

Note: Assume compression flange is NOT restrained against rotation

	Left Side	Right Side		
$\phi WSB =$	0.85	0.85		
$H_{cc} =$	147.20	147.20	in	Value previously defined
$b_{cf} =$	9.02	9.02	in	Column flange width
$Sc_x =$	196.00	196.0	in^3	Looked up value
$L_b =$	139.1	N/A	in	$= H_{cc} - d/2$
$M_y =$	9800.0	9800.0	kip*in	$= Sc_x * F_{yc}$
$M_u =$	1764	0	kip*in	$= M_{pr}$
$C_r =$	960000	960000	ksi	$= \text{if } (M_u < M_y, 960000, 480000)$
$(h/tw)/(L_b/b_{cf}) =$	2.98	N/A		$= (h/tw)/(L_b/b_{cf})$
$\phi R_n \text{ WSB} =$	1479	N/A	kip	$\text{AISC J10-7, } = (\phi WSB * C_r * tcw^3 * t_{cf} / h^2) / (0.4 * C_r^3)$
WSB check =	N/A	N/A		Not applicable
BotStiffener_Bracing_Req'd =	No	No		$= \text{if } (And(\phi R_n \text{ WSB} < Pr_link, \text{WSB_check} = \text{"Applicable"}), \text{"Yes"}, \text{"No"})$

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

	Left Side	Right Side		
$F_{su} \text{ top} =$	0.0	0.0	kip	$= \text{if } (Pr_link - \phi R_n \text{ Stiff_top} < 0, 0, Pr_link - \phi R_n \text{ Stiff_top})$
$F_{su} \text{ bot} =$	0.0	0.0	kip	$= \text{if } (Pr_link - \phi R_n \text{ Stiff_bot} < 0, 0, Pr_link - \phi R_n \text{ Stiff_bot})$
$F_{su} =$	0.0	0.0	kip	$= \max(F_{su_top}, F_{su_bot}) * SMF_Sides$

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

$\phi t =$	0.9			Value previously defined
$F_y \text{ stiff} =$	50	ksi		Value previously defined
$K_{det} =$	1.688	in		Looked up value
$K_1 =$	1.063	in		Looked up value
$tcw_{det}/2 =$	0.250	in		Looked up value
$t_{cf}(det) =$	0.750	in		Looked up value
	Left Side	Right Side		
$As \text{ min} =$	0.000	0.00	in^2	$= F_{su} / (f_t * F_y \text{ Stiff})$
$L_{clip \text{ web min}} =$	2.438	2.438	in	$= K_{det} - t_{cf} + 1.5$
$L_{clip \text{ web}} =$	2.438	2.438	in	$= \text{Ceiling}(L_{clip \text{ web min}}, 0.0625)$
$L_{clip \text{ flange min}} =$	0.813	0.813	in	$= K_1 - tcw_{det}/2$
$L_{clip \text{ flange}} =$	0.813	0.813	in	$= \text{Ceiling}(L_{clip \text{ flange min}}, 0.0625)$
$bstiff \text{ min} =$	2.7717	2.7717	in	$\text{AISC 360 J10.8, } = b_{cf}/3 - tcw/2$
$bstiff \text{ max} =$	4.275	4.275	in	$= (b_{cf} - tcw)/2$
$bstiff =$	4.2500	4.2500	in	$= \text{Floor}(bstiff \text{ max}, 0.125)$, Each side of column web
$tstiff \text{ USE?} =$	YES	YES	in	Input value
$t \text{ stiff tension} =$	0.000	0.000	in	$= As \text{ min} / (bstiff \text{ min} * L_{clip \text{ flange}}) / 2$, SMF_Sides, OK, if $t \text{ stiff tension} \leq tstiff$
$tstiff \text{ min} =$	0.375	0.000	in	$= \max(t \text{ stem}/2, bstiff/16, \text{looked up value})$, OK, if $tstiff \text{ min} \leq tstiff$
$tstiff \text{ max} =$	0.750	0.000	in	$= t \text{ stem}$, OK, if $tstiff \text{ max} \geq tstiff$
$t \text{ stiff Use} =$	0.375	0.375		
$Lstiff \text{ Pdepth ini} =$	11.28		in	$= 0.5 * dc - t_f$, For Single sided connections (partial depth)
$Lstiff \text{ Fdepth ini} =$	22.56		in	$= dc - 2 * t_{cf}$, For double sided connections
$Lstiff \text{ ini} =$	22.56		in	$= \text{if } (and(SMF_Sides = 1, \text{Ext. Stiffener depth} = \text{"Partial"}), Lstiff \text{ Pdepth ini}, Lstiff \text{ Fdepth ini})$
$Lstiff =$	22.50		in	$= \text{Floor}(Lstiff \text{ ini}, 0.0625)$

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

	Left Side	Right Side		
ϕ_c =	0.9	0.9		
Kstiff=	0.75	0.75		
Astiff_fd=	5.84	5.84	in ²	= 2* bstiff* tstiff + 12* tcw ²
Lstiff_fd=	22.60	22.60	in ⁴	= 12* tcw* tcw ³ /12 + 2* (tstiff* bstiff ³ /3/ 12+ bstiff* tstiff* (bstiff/2 +tc/2) ²)
rstiff_fd=	1.97	1.97	in	= Sqrt (Lstiff_fd/ Astiff_fd)
Kstiff*Lstiff_fd/rstiff_fd=	8.58	8.58		= Kstiff* Lstiff_fd/ rstiff_fd
Fe=	3891	3891	ksi	= Pi ² * 29000/ (Kstiff* Lstiff_fd/ rstiff_fd) ²
Fcr=	49.73	49.73	ksi	= if (Kstiff* Lstiff_fd/ rstiff_fd <= 4.71* Sqrt (29000/ Fy_stiff), 0.658* (Fy_stiff/ Fc)* Fy_stiff,
Pn_stiff=	291.9	291.9	kips	= Kstiff* Lstiff_fd* rstiff_fd > 25, Fcr* Astiff_fd, Astiff_fd* Fy_stiff
ϕ_c *Pn_stiff=	262.7	262.7	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

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

ϕ_{fillet} =	0.75		
Fexx=	70	ksi	
tstiff=	0.375	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=	0.0000	0.0000	in
wfillet_flange=	0.1875	0.1875	in
wfillet_flange_DCR=	0.000	0.000	

= (0.5* Fsu* SMF_Sides)/ (ϕ_{fillet} * 1.5* 0.6* Fexx* (bstiff - Lclip_flange-0.5)*2²/(1/2))
=Max (wfillet_min, ceiling(wfillet_flange_min, 1/16"))
= wfillet_flange_min/ wfillet_flange

5.16 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
wfillet_min_web=	0.0000	0.0000	in
wfillet_web=	0.1875	0.1875	in
wfillet_web_DCR=	0.000	0.000	

Minimum fillet weld size per AISC 360 Table J2.4
= 0.5* Fsu* SMF_Sides/ (ffillet* 0.6* Fexx* (Lstiff - Lclip_web-0.5)*2²/(1/2))
=max(wfillet_min, ceiling(wfillet_web,1/16))

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

	Left Side	Right Side	
bcf=	9.0	9.0	in
bcf_LinkFlg_bolt=	7.5	0	in
bcf_DCR=	0.831	0.000	OK

=column flange width
=Link Sflange+2*Ledge_min

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

	Left Side	Right Side	
ϕ_b =	0.9	0.9	
tcf=	0.77	0.77	in
c=	5.750	0.000	in
g=	5.000	0.000	in
H_flange=	9.250	0.000	in
s=	3.358	0.000	in
ps0=	2.313	0.000	in
psi_tmp=	2.313	0.000	in
psi=	2.313	0.000	in
h1=	14.55	0	in
h0=	19.55	0	in
Yc_unstiffened=	137.05	0.00	in
tcf_req_unstiffened=	0.56	0.00	in
Yc_stiffened=	189.65	0.00	in
tcf_req_stiffened=	0.48	0.00	in
Stiffened?=	YES	YES	
tcf_min=	0.477	0.000	in
DCR_colFLB1=	0.619	0.000	

Value previously defined
= bolt_g_flange
= bolt_s_flange
Value previously defined
= 0.5* Sqrt (bcf* g)
= (g - tstiff)/2
= ps0
= if (psi_tmp > s, s, psi_tmp)
= db + t_stem - g/2
= db + t_stem + g/2
= bcf/2* (h1/s + h0/s) + 2/g* (h1* (s + 3*c/4) + h0* (s + c/4) + c²/4) + g/2
= Sqrt (1.1* Mcap/ (ϕ_b * Fyc* Yc_unstiffened))
= bcf/2* (h1* (1/s + 1/psi) + h0* (1/s + 1/ps0)) + 2/g* (h1* (s + psi) + h0* (s + ps0))
= Sqrt (1.1* Mcap/ (fb* Fyc* Yc_stiffened))
Bottom Stiffener Required per 17.3
= if (Stiffened? = YES, tcf_req_stiffened, tcf_req_unstiffened)
= tcf_min/ tcf

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

	Left Side	Right Side	
de=	1.25	1.25	in
pfi=	2.313	0.000	in
ps0=	2.313	0.000	in
Yp=	160.0	0.000	in
tcf_req=	0.519	0.000	in
DCR_colFLB2=	0.674	0.000	

centerline of top link bolt to top of column
= (g - tstiff)/ 2
Value previously defined
= bcf/2* (h1* (1/pfi + 1/ps0) + h0* (1/ps0 + 1/2/s)) + 2/g* (h1* (pfi + s) + h0* (d2 + ps0))
= Sqrt* (1.1* Mcap/ (fb* Fy* Yp))
= tcf_req/ tcf

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

	Left Side	Right Side	
Link Size=	YLMC6-3.5	N/A	
byield=	3.500	0.000	in
tstem=	0.750	0.000	in
Fy_link=	50	50	ksi
Ry=	1.1	1.1	
Pye_link=	144.375	0	kips
0.02 * Pye_link=	2.888	0.000	kips
Pbrace (ASD)=	2.021	0.000	kips

AISC 341, E3.4.c.1(b)
Bracing Force (LRFD)
=0.7*Bracing Force (LRFD)

5.19 DESIGN SUMMARY:

DCR Summary:		Report:
SCWB_DCR_total=	0.099	0.099
DCR_PZ_total=	0.304	Overall PZ chk: NG
DCR_PZ_total_NEW=	3.000	NO 0.304
DCR_stiff_comp_DCR=	0.000	0.000 0.000
wfillet_flange_DCR=	0.000	0.000 0.000
wfillet_web_DCR=	0.000	0.000 0.000
DCR_colFLB1=	0.619	0.000 0.619
DCR_colFLB2=	0.674	0.000 0.674
DCR_colFLB=		0.619

Geometry/Weld Summary:		Report:
DP used?	NO	PlugWeld? NO
t_dblr_use=	0.000	0.000
Stiffener Req'd=	NO	NO
t_stiff_Use=	0.375	0.375 0.375
wfillet_flange=	0.188	0.188 0.188
wfillet_web=	0.188	0.188 0.188

5.1 CURRENT MEMBER:

Column ID:	3	Top Story=	Story2	Use Doubler PL? =	NO
Column size:	W24X84	Left Beam ID=	N/A	Right Beam ID=	2001
Pu (kips):	32.85	Left Beam size=	N/A	Right Beam size=	W16X67
Hcc t (in):	N/A	RightEndLink=	N/A	RightEndLink=	YES
Stiffener (Y/N):	YES	db_left (in)=	N/A	db_right (in)=	16.3
Ext. Stiffener depth=	Full Depth	Left Link Size=	N/A	Right Link Size=	YLMC6-3.5
Current Story=	Story2	Left Beam Lh (in):	N/A	Right Beam Lh (in):	251.10
Story Above=	Story3	Vu Gravity (kips):	0.00	Vu Gravity (kips):	4.46

5.2 LINK PROPERTIES: (From YL Database)

	Left Link	Right Link			
Thickness (t_stem) =	0.000	0.750	in	R=	3.5
Yield Width (b_yield)=	0.000	3.500	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	5.000	in	Pu_link=	78 kips
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
a=	0.000	3.500	in	Py_linkR=	131.25 kips
Pcap_Link=	0.0	77.8	kips	Pr_linkR=	204.75 kips
Mcap=	0	1327	k-in		

= if (R >= 6.5, Pr_link, use selection in "Design_Limits")
= Pcap_Link *(db+dstem)

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

	Left Side	Right Side			
Mcap=	0.00	1327.18	kip*in	Value previously defined	Column Size= AISC 360 Table B4.1b
Lh=	N/A	220.0	in		W24X84
Vbm_gravity=	0.00	4.46	kip		Col b/t= 5.86 Compact
Vu_bm=	0.00	16.53	kip	= n_sides* Mpr/ Lh + Vbm_gravity	Col h/tw= 45.9 Compact
Column Size=	W24X84	W24X84		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= 5.86 Hi.Ductile
Agc=	24.7	24.7	in^2	Looked up value	
Zcx=	224.00	224.00	in^3	Looked up value	Col h/tw= 45.9 Hi.Ductile
dc=	24.1	24.1	in	Looked up value	
tcf=	0.77	0.77	in	Looked up value	
Pu_colPositive=	32.85	32.85	kips	Max Axial Force of column per Overstrength combo	
Muv=	0.0	257.0	kips*in	= Vu_bm* (dc/2 + a)	
ΣMp_col=	10902	10902	kip*in	= Zcx* (Fyc - Pu_colPositive/ Agc) * if(top story,1 else 2)	
ΣMpb=	0	1584	kip*in	= Mpr + Muv	
Pu<0.3*Pc? =	YES	YES		AISC 341-16 E3.4a Exception (a)	
Top Story? =	YES	YES		AISC 341-16 E3.4a Exception (a) (1)	SCWB not applicable at top story
SCWB_DCR=	0.000	0.145			
SCWB_DCR_total=	N/A	N/A			

5.4 COLUMN NET SECTION CHECK

	Left Side	Right Side			
Mpc=	0	11200	kip*in	Zcx*Fyc	Ry= 1.1
Dflange=	0.000	1.000	in		Rt= 1.1
Dhole=	0.000	1.188	in	Dflange+3/16	Fu= 65 ksi
nBoltrow=	0	2		nlinkCol_bolts/2	Fy= 50 ksi
Znet=	0.000	198.727	in^3		
Znet_total=	198.727		in^3		Put = 22.13 kips
Mpc_net=	14017		kip*in	Znet*(Rt*Fu-Pu_tension/Anet)	Anet = 22.87 in^2
Pr_link=	0	204.75	kip	Ay_link*Fu_link*Rt_link	
Mpr_link=	0	1327	kip*in	Value previously defined	
DCR_ColNetSect=	0.095	OK			

5.5 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=	24.10	in	Value previously defined		
tcw=	0.47	in	Value previously defined		
Agc=	24.70	in^2	Value previously defined		
Pc=	1235	kips	= Fyc* Agc		
Pu=	32.85	kips	Value previously defined		
φRn_PZ=	305.83	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	0.00	kips	= ΣMpb/((Hcc_b+Hcc_t)/2), 0 for top story	
Pcap_Link=	0.00	77.84	kips	= Pu_link	
Ru=	0.00	77.84	kips		
DCR_PZ=	0.000	0.255		= Pr_link - Vu_c / φRn_PZ	
DCR_PZ_total=	0.255	OK		Ok, if DCR_PZ <= DCR_PZ_Limit	

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

Doubler Plate Required=	NO	<----If NO, Skip 5.4.1	USE Plug Weld?	NO
Fy_dpl=	50	ksi	Doubler Plate Yield Strength	
$\phi Rn_PZ_NODBLR=$	305.83	kips	$= \phi v_pz \text{ if } (Pu < 0.4 \cdot Pc, 0.6 \cdot Fy_col \cdot dc \cdot tcw, 0.6 \cdot Fy_col \cdot dc \cdot tcw \cdot (1.4 - Pu/Pc))$	
$\phi Rn_PZ_req=$	0.00	kips	Additional capacity required	
$\phi Rn_PZ_NEW=$	305.8	kips	$= \phi Rn_PZ_NODBLR + \phi Rn_PZ_req$	
t_dblr_strength=	0.000	in	Doubler Plate thickness	
t_dblr_use=	0.000	in	$= \phi v_pz \text{ if } (Pu < 0.4 \cdot Pc, 0.6 \cdot (Fy_col \cdot dc \cdot tcw + Fy_dpl \cdot (dc - 2 \cdot tcf) \cdot t_dblr),$	
$\phi Rn_PZ_DBLR=$	305.83	kips	$0.6 \cdot (Fy_col \cdot dc \cdot tcw + Fy_dpl \cdot (dc - 2 \cdot tcf) \cdot t_dblr) \cdot (1.4 - Pu/Pc))$	
	Left Side	Right Side		
DCR_PZ_NEW=	0.000	0.255		
DCR_PZ_total_NEW=	0.255	OK		
Max Doubler Thickness	1.250	in	Maximum thickness of doubler plate to not interfere with YL column bolts	
DCR Doubler Plate Thickness	3.000	NG	Use CJP Weld or choose another link size with wider flange	
DCR_PZ_Total=	3.000		=max of DCR_PZ (strength), DCR_PZ (DP_Thickness)	

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

Option 1: Doubler without Continuity plates

t_dblr=	0.000	in	doubler plate thickness
tcw=	0.47	in	column web thickness
min t=	0.000	in	=min of tDP and tcw, rounded up to nearest 1/16
willet=	0	in	From AISC 360 Table J2.4

Option 2A: Extended Doubler Plate

t_dblr=	0.000	in	Continuity plate to doubler per Section E3.6f.2(c)
willet=	0.0000	in	doubler plate thickness (c.3, available strength of doubler plate)
willet_use=	0.0000	in	=5/8*tDP (develop shear strength of the doubler Plate), rounded to nearest 1/16
# 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=	0.000	in	doubler plate thickness
willet_use=	0.0000	in	t_dblr, same thickness to develop doubler Plate capacity
# of sides=	1		

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

Doubler Plate Plug-Weld Used?	NO		
PlugWeld_Dia=	0.313	in	Weld Depth= 0.000 in

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

Option 1: Fillet weld to develop doubler plate shear capacity

t_dblr=	0.000	in	doubler plate thickness
Vn=	0.00	kips	$0.6 \cdot Fy \cdot Agv$ (per 1 in length)
willet=	0.0	/16	Vn/1.39, rounded up to 1/16"

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

5.6 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	16.3	in d beam
tbf=	N/A	0.665	
d_stiff_cap=	N/A	3.5	
d_stiffTop=	N/A	3.5	in Location of top stiffener to top of column cap, assume 2 dc if not at top story
d_stiffBot=	N/A	19.8	in Location of bottom stiffener to top of column cap = d + d_stiffTop

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

	Left Side	Right Side	
$\phi WLY=$	1	1	
dc=	24.1	24.1	in Value previously defined
kdes=	1.27	1.27	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 $= 2 \cdot t_flange + t_stem$
Ct_top=	1	0.5	in $= \text{if } (d_stiffTop \leq dc, 0.5, 1)$
Ct_bot=	1	0.5	in $= \text{if } (d_botTop \leq dc, 0.5, 1)$
$\phi Rn_WLYmoreD=$	149.23	225.60	kips $= \phi WLY \cdot Fy_col \cdot tcw \cdot (5 \cdot kdes + l_b)$
$\phi Rn_WLYlessD=$	74.61	150.99	kips $= \phi WLY \cdot Fy_col \cdot tcw \cdot (2.5 \cdot kdes + l_b)$
$\phi Rn_WLY_top=$	149.23	150.99	kips $= \text{if } (d_stiffTop > dc, \phi Rn_WLYmoreD, \phi Rn_WLYlessD)$
$\phi Rn_WLY_bot=$	149.23	150.99	kips $= \text{if } (d_stiffTop > dc, \phi Rn_WLYmoreD, \phi Rn_WLYlessD)$

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

$\phi WLC =$	0.75		
$E_s =$	29000	ksi	
$t_{cf} =$	0.77	in	Value previously defined
$tcw =$	0.47	in	Value previously defined
$\phi R_n \text{ WLC more } 0.5D =$	243.7	kip	$= \phi WLC * 0.8 * tcw^2 * (1 + 3 * (L_b/dc) * (tcw/tcf)^{1.5}) * \text{Sqrt}(E_s * F_y \cdot col * tcf / tcw)$
AISC J10-5a =	121.85	kip	$= 0.5 * \phi R_n \text{ WLC more } 0.5D$
AISC J10-5b =	118.67	kip	$= \phi WLC * 0.4 * tcw^2 * (1 + (4 * L_b/dc - 0.2) * (tcw/tcf)^{1.5}) * \text{Sqrt}(E_s * F_y \cdot col * tcf / tcw)$
$\phi R_n \text{ WLC less } 0.5D =$	121.85	kip	$= \text{if } (L_b/dc \leq 0.2, \text{AISC J10-5a, AISC J10-5b})$
	Left Side	Right Side	
$\phi R_n \text{ WLC top} =$	243.69	121.8	kip
$\phi R_n \text{ WLC bot} =$	243.7	243.7	kip
			$= \text{if } (d_stiffTop > dc/2, \phi R_n \text{ WLC more } 0.5D, \phi R_n \text{ WLC less } 0.5D)$
			$= \text{if } (d_stiffBot > dc/2, \phi R_n \text{ WLC more } 0.5D, \phi R_n \text{ WLC less } 0.5D)$

5.9 COLUMN WEB COMPRESSION BUCKLING (AISC 360 J10.5):

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

$\phi WCB =$	0.9		
$h =$	21.56	in	$= dc - 2 * k_{des}$
$\phi R_n \text{ WCB} =$	125.3	kip	$\text{AISC J10-8, } = \phi WCB * 24 * tcw^3 * \text{sqrt}(E_s * F_y \cdot col) / h$
	Left Side	Right Side	
$\phi R_n \text{ WCB top} =$	125.25	62.63	kip
$\phi R_n \text{ WCB bot} =$	125.25	125.25	kip
			$= \text{if } (d_stiffTop < d, \phi R_n \text{ WCB} * 0.5, \phi R_n \text{ WCB})$
			$= \text{if } (d_stiffBot < d, \phi R_n \text{ WCB} * 0.5, \phi R_n \text{ WCB})$

5.10 SUMMARY OF LOCAL CAPACITIES:

	Left Side				At Bottom Stiffener		
At Top Stiffener							
$\phi R_n \text{ WLY top} =$	149.23	kip			$\phi R_n \text{ WLY bot} =$	149.23	kip
$\phi R_n \text{ WLC top} =$	243.69	kip			$\phi R_n \text{ WLC bot} =$	243.7	kip
$\phi R_n \text{ WCB top} =$	125.25	kip			$\phi R_n \text{ WCB bot} =$	125.25	kip
$\text{if } (SMF_Sides = 1, \phi R_n \text{ Stiff top} = \min(\phi R_n \text{ WLY top}, \phi R_n \text{ WLC top}, \phi R_n \text{ WCB top}))$							
$\text{if } (SMF_Sides = 1, \phi R_n \text{ Stiff bot} = \min(\phi R_n \text{ WLY bot}, \phi R_n \text{ WLC bot}, \phi R_n \text{ WCB bot}))$							
$\min(\phi R_n \text{ WLY top}, \phi R_n \text{ WLC top}, \phi R_n \text{ WCB top})$							
$\min(\phi R_n \text{ WLY bot}, \phi R_n \text{ WLC bot}, \phi R_n \text{ WCB bot})$							
$\phi R_n \text{ Stiff top} =$	149.23	kip			$\phi R_n \text{ Stiff bot} =$	149.23	kip
Top Stiffener Required =	No				Bottom Stiffener Required =	No	
Top Stiff Provided? =	YES				Bottom Stiff Provided? =	YES	
							Stiff Req'd = NO
	Right Side				At Bottom Stiffener		
At Top Stiffener							
$\phi R_n \text{ WLY top} =$	150.99	kip			$\phi R_n \text{ WLY bot} =$	150.99	kip
$\phi R_n \text{ WLC top} =$	121.85	kip			$\phi R_n \text{ WLC bot} =$	243.7	kip
$\phi R_n \text{ WCB top} =$	62.63	kip			$\phi R_n \text{ WCB bot} =$	125.25	kip
$\text{if } (SMF_Sides = 1, \phi R_n \text{ Stiff top} = \min(\phi R_n \text{ WLY top}, \phi R_n \text{ WLC top}, \phi R_n \text{ WCB top}))$							
$\text{if } (SMF_Sides = 1, \phi R_n \text{ Stiff bot} = \min(\phi R_n \text{ WLY bot}, \phi R_n \text{ WLC bot}, \phi R_n \text{ WCB bot}))$							
$\min(\phi R_n \text{ WLY top}, \phi R_n \text{ WLC top}, \phi R_n \text{ WCB top})$							
$\min(\phi R_n \text{ WLY bot}, \phi R_n \text{ WLC bot}, \phi R_n \text{ WCB bot})$							
$\phi R_n \text{ Stiff top} =$	121.85	kip			$\phi R_n \text{ Stiff bot} =$	150.99	kip
Top Stiffener Required =	No				Bottom Stiffener Required =	No	
Top Stiff Provided? =	YES				Bottom Stiff Provided? =	YES	
							Stiff Req'd = NO

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

Note: Assume compression flange is NOT restrained against rotation

	Left Side	Right Side		
$\phi WSB =$	0.85	0.85		
$H_{cc} =$	186.80	186.80	in	Value previously defined
$b_{cf} =$	9.02	9.02	in	Column flange width
$Sc_x =$	196.00	196.0	in^3	Looked up value
$L_b =$	N/A	178.7	in	$= H_{cc} - d/2$
$M_y =$	9800.0	9800.0	kip*in	$= Sc_x * F_{yc}$
$M_u =$	0	1327	kip*in	$= M_{pr}$
$C_r =$	960000	960000	ksi	$= \text{if } (M_u < M_y, 960000, 480000)$
$(h/tw)/(L_b/b_{cf}) =$	N/A	2.32		$= (h/tw)/(L_b/b_{cf})$
$\phi R_n \text{ WSB} =$	N/A	697	kip	$\text{AISC J10-7, } = (\phi WSB * C_r * tcw^3 * t_{cf} / h^2) / (0.4 * C_r^3)$
WSB_check =	N/A	N/A		Not applicable
BotStiffener_Bracing_Req'd =	No	No		$= \text{if } (And(\phi R_n \text{ WSB} < Pr_link, \text{WSB_check} = \text{"Applicable"}), \text{"Yes"}, \text{"No"})$

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

	Left Side	Right Side		
$F_{su} \text{ top} =$	0.0	0.0	kip	$= \text{if } (Pr_link - \phi R_n \text{ Stiff_top} < 0, 0, Pr_link - \phi R_n \text{ Stiff_top})$
$F_{su} \text{ bot} =$	0.0	0.0	kip	$= \text{if } (Pr_link - \phi R_n \text{ Stiff_bot} < 0, 0, Pr_link - \phi R_n \text{ Stiff_bot})$
$F_{su} =$	0.0	0.0	kip	$= \max(F_{su_top}, F_{su_bot}) * SMF_Sides$

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

$\phi t =$	0.9			Value previously defined
$F_y \text{ stiff} =$	50	ksi		Value previously defined
$K_{det} =$	1.688	in		Looked up value
$K_1 =$	1.063	in		Looked up value
$tcw_{det}/2 =$	0.250	in		Looked up value
$t_{cf}(det) =$	0.750	in		Looked up value
	Left Side	Right Side		
$As \text{ min} =$	0.000	0.00	in^2	$= F_{su} / (t * F_y \text{ Stiff})$
$L_{clip \text{ web min}} =$	2.438	2.438	in	$= K_{det} - t_{cf} + 1.5$
$L_{clip \text{ web}} =$	2.438	2.438	in	$= \text{Ceiling}(L_{clip_web_min}, 0.0625)$
$L_{clip \text{ flange min}} =$	0.813	0.813	in	$= K_1 - tcw_{det}/2$
$L_{clip \text{ flange}} =$	0.813	0.813	in	$= \text{Ceiling}(L_{clip_flange_min}, 0.0625)$
$bstiff_min =$	2.7717	2.7717	in	$\text{AISC 360 J10.8, } = b_{cf}/3 - tcw/2$
$bstiff_max =$	4.275	4.275	in	$= (b_{cf} - tcw)/2$
$bstiff =$	4.2500	4.2500	in	$= \text{Floor}(bstiff_max, 0.125)$, Each side of column web
$tstiff_USE? =$	YES	YES	in	Input value
$t_stiff \text{ tension} =$	0.000	0.000	in	$= As_min / (bstiff_L_{clip_flange}) / 2$, SMF_Sides, OK, if $t_stiff \text{ tension} \leq tstiff$
$tstiff_min =$	0.000	0.375	in	$= \max(t_stem/2, bstiff/16, \text{looked up value})$, OK, if $tstiff_min \leq tstiff$
$tstiff_max =$	0.000	0.750	in	$= t_stem$, OK, if $tstiff_max \geq tstiff$
$t_stiff \text{ Use} =$	0.375	0.375		
$Lstiff \text{ Pdepth ini} =$	11.28		in	$= 0.5 * dc - t_f$, For Single sided connections (partial depth)
$Lstiff \text{ Fdepth ini} =$	22.56		in	$= dc - 2 * t_{cf}$, For double sided connections
$Lstiff_ini =$	22.56		in	$= \text{if } (and(SMF_Sides = 1, \text{Ext. Stiffener depth} = \text{"Partial"}), Lstiff_Pdepth_ini, Lstiff_Fdepth_ini)$
$Lstiff =$	22.50		in	$= \text{Floor}(Lstiff_ini, 0.0625)$

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

	Left Side	Right Side		
ϕ_c =	0.9	0.9		
Kstiff=	0.75	0.75		
Astiff_fd=	5.84	5.84	in ²	= 2* bstiff* tstiff + 12* tcw ²
Lstiff_fd=	22.60	22.60	in ⁴	= 12* tcw ³ tcw ³ /12 + 2* (tstiff* bstiff ³ /3/ 12+ bstiff* tstiff* (bstiff/2 +tc/2) ²)
rstiff_fd=	1.97	1.97	in	= Sqrt (Lstiff_fd/ Astiff_fd)
Kstiff*Lstiff_fd/rstiff_fd=	8.58	8.58		= Kstiff* Lstiff_fd/ rstiff_fd
Fe=	3891	3891	ksi	= Pi ² * 29000/ (Kstiff* Lstiff_fd/ rstiff_fd) ²
Fcr=	49.73	49.73	ksi	= if (Kstiff* Lstiff_fd/ rstiff_fd <= 4.71* Sqrt (29000/ Fy_stiff), 0.658* (Fy_stiff/ Fc)* Fy_stiff,
Pn_stiff=	291.9	291.9	kips	= Kstiff* Lstiff_fd* rstiff_fd > 25, Fcr* Astiff_fd, Astiff_fd* Fy_stiff
ϕ_c *Pn_stiff=	262.7	262.7	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

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

ϕ_{fillet} =	0.75		
Fexx=	70	ksi	
tstiff=	0.375	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=	0.0000	0.0000	in
wfillet_flange=	0.1875	0.1875	in
wfillet_flange_DCR=	0.000	0.000	

= (0.5* Fsu* SMF_Sides)/ (ϕ_{fillet} 1.5* 0.6* Fexx* (bstiff - Lclip_flange-0.5)*2^{1/2})
=Max (wfillet_min, ceiling(wfillet_flange_min, 1/16"))
= wfillet_flange_min/ wfillet_flange

5.16 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
wfillet_min_web=	0.0000	0.0000	in
wfillet_web=	0.1875	0.1875	in
wfillet_web_DCR=	0.000	0.000	

Minimum fillet weld size per AISC 360 Table J2.4
= 0.5* Fsu* SMF_Sides/ (ϕ_{fillet} 0.6* Fexx* (Lstiff - Lclip_web-0.5)*2^{1/2})
=max(wfillet_min, ceiling(wfillet_web, 1/16"))

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

	Left Side	Right Side	
bcf=	9.0	9.0	in
bcf_LinkFlg_bolt=	0	7.5	in
bcf_DCR=	0.000	0.831	OK

=column flange width
=Link Sflange+2*Ledge_min

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

	Left Side	Right Side	
ϕ_b =	0.9	0.9	
tcf=	0.77	0.77	in
c=	0.000	5.750	in
g=	0.000	5.000	in
H_flange=	0.000	9.250	in
s=	0.000	3.358	in
ps0=	0.000	2.313	in
psi_tmp=	0.000	2.313	in
psi=	0.000	2.313	in
h1=	0	14.55	in
h0=	0	19.55	in
Yc_unstiffened=	0.00	137.05	in
tcf_req_unstiffened=	0.00	0.49	in
Yc_stiffened=	0.00	189.65	in
tcf_req_stiffened=	0.00	0.41	in
Stiffened?=	YES	YES	
tcf_min=	0.000	0.414	in
DCR_colFLB1=	N/A	0.537	

Value previously defined
= bolt_g_flange
= bolt_s_flange
Value previously defined
= 0.5* Sqrt (bcf* g)
= (g - tstiff)/2
= ps0
= if (psi_tmp > s, s, psi_tmp)
= db + t_stem - g/2
= db + t_stem + g/2
= bcf/2* (h1/s + h0/s) + 2/g* (h1* (s + 3*c/4) + h0* (s + c/4) + c²/4) + g/2
= Sqrt (1.1* Mcap/ (ϕ_b * Fyc* Yc_unstiffened))
= bcf/2* (h1* (1/s + 1/psi) + h0* (1/s + 1/psi)) + 2/g* (h1* (s + psi) + h0* (s + psi))
= Sqrt (1.1* Mcap/ (ϕ_b * Fyc* Yc_stiffened))
Bottom Stiffener Required per 17.3
= if (Stiffened? = YES, tcf_req_stiffened, tcf_req_unstiffened)
= tcf_min/ tcf

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

	Left Side	Right Side	
de=	1.25	1.25	in
pfi=	0.000	2.313	in
ps0=	0.000	2.313	in
Yp=	0.0	160.036	in
tcf_req=	0.000	0.450	in
DCR_colFLB2=	N/A	0.585	

centerline of top link bolt to top of column
= (g - tstiff)/ 2
Value previously defined
= bcf/2* (h1* (1/pfi + 1/ps0) + h0* (1/ps0 + 1/2/s)) + 2/g* (h1* (pfi + s) + h0* (d2 + ps0))
= Sqrt* (1.1* Mcap/ (ϕ_b * Fy* Yp))
= tcf_req/ tcf

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

	Left Side	Right Side	
Link Size=	N/A	YLMC6-3.5	
byield=	0.000	3.500	in
tstem=	0.000	0.750	in
Fy_link=	50	50	ksi
Ry=	1.1	1.1	
Pye_link=	0	144.375	kips
0.02 * Pye_link=	0.000	2.888	kips
Pbrace (ASD)=	0.000	2.021	kips

AISC 341, E3.4.c.1(b)
Bracing Force (LRFD)
= 0.7*Bracing Force (LRFD)

5.19 DESIGN SUMMARY:

DCR Summary:		Report:
SCWB_DCR_total=	N/A	N/A
DCR_PZ_total=	0.255	Overall PZ chk: NG
DCR_PZ_total_NEW=	3.000	NO 0.255
DCR_stiff_comp_DCR=	0.000	0.000 0.000
wfillet_flange_DCR=	0.000	0.000 0.000
wfillet_web_DCR=	0.000	0.000 0.000
DCR_colFLB1=	N/A	0.537 0.537
DCR_colFLB2=	N/A	0.585 0.585
DCR_colFLB=		0.585

Geometry/Weld Summary:		Report:
DP used?	NO	PlugWeld? NO
t_dblr_use=	0.000	0.000
Stiffener Req'd=	NO	NO
t_stiff_Use=	0.375	0.375
wfillet_flange=	0.188	0.188 0.188
wfillet_web=	0.188	0.188 0.188

5.1 CURRENT MEMBER:

Column ID:	4	Top Story=	Story2	Use Doubler PL? =	NO
Column size:	W24X84	Left Beam ID=	2001	Right Beam ID=	N/A
Pu (kips):	32.85	Left Beam size=	W16X67	Right Beam size=	N/A
Hcc t (in):	N/A	RightEndLink=	YES	RightEndLink=	N/A
Stiffener (Y/N):	YES	db_left (in)=	16.3	db_right (in)=	N/A
Ext. Stiffener depth=	Full Depth	Left Link Size=	YLMC6-3.5	Right Link Size=	N/A
Current Story=	Story2	Left Beam Lh (in):	251.10	Right Beam Lh (in):	N/A
Story Above=	Story3	Vu Gravity (kips):	4.46	Vu Gravity (kips):	0.00

5.2 LINK PROPERTIES: (From YL Database)

	Left Link	Right Link			
Thickness (t_stem) =	0.750	0.000	in	R=	3.5
Yield Width (b_yield)=	3.500	0.000	in	Fy_link=	50 ksi
Thickness (t_flange)	1.250	0.000	in	Fu_link=	65 ksi
Spacing Along Length (horiz) (bolt_s_flange)	5.000	0.000	in	Pu_link=	78 kips
Gauge Along Width (vertical) (bolt_g_flange)=	5.750	0.000	in	Py_linkL=	131.25 kips
Flange height (H_flange) =	9.250	0.000	in	Pr_linkL=	204.75 kips
a=	3.500	0.000	in	Py_linkR=	0 kips
Pcap_Link=	77.8	0.0	kips	Pr_linkR=	0 kips
Mcap=	1327	0	k-in		

= if (R >= 6.5, Pr_link, use selection in "Design_Limits")
= Pcap_Link *(db+dstem)

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

	Left Side	Right Side			
Mcap=	1327.18	0.00	kip*in	Value previously defined	AISC 360 Table B4.1b
Lh=	220.0	N/A	in	Column Size=	W24X84
Vbm_gravity=	4.46	0.00	kip	Col b/t=	5.86 Compact
Vu_bm=	16.53	0.00	kip	Col h/tw=	45.9 Compact
Column Size=	W24X84	W24X84			
Fyc=	50	50	ksi		
a=	3.50	0.00	in		
Agc=	24.7	24.7	in^2		
Zcx=	224.00	224.00	in^3		
dc=	24.1	24.1	in		
tcf=	0.77	0.77	in		
Pu_colPositive=	32.85	32.85	kips		
Muv=	257.0	0.0	kip*in		
ΣMp_col=	10902	10902	kip*in		
ΣMpb=	1584	0	kip*in		
Pu<0.3*Pc? =	YES	YES			
Top Story? =	YES	YES			
SCWB_DCR=	0.145	0.000			
SCWB_DCR_total=	N/A	N/A			

= n_sides* Mpr / Lh + Vbm_gravity
Value previously defined
Value previously defined
Value previously defined
Looked up value
Looked up value
Looked up value
Max Axial Force of column per Overstrength combo
= Vu_bm* (dc/2 + a)
= Zcx* (Fyc - Pu_colPositive/ Agc) * if(top story, 1 else 2)
= Mpr + Muv
AISC 341-16 E3.4a Exception (a)
AISC 341-16 E3.4a Exception (a) (1)

SCWB not applicable at top story

5.4 COLUMN NET SECTION CHECK

	Left Side	Right Side			
Mpc=	11200	0	kip*in	Zcx*Fyc	Ry= 1.1
Dflange=	1.000	0.000	in		Rt= 1.1
Dhole=	1.188	0.000	in	Dflange+3/16	Fu= 65 ksi
nBoltrow=	2	0		nlinkCol_bolts/2	Fy= 50 ksi
Znet=	198.727	0.000	in^3		
Znet_total=	198.727		in^3		
Mpc_net=	14017		kip*in	Znet*(Rt*Fu-Pu_tension/Anet)	Put = 22.13 kips
Pr_link=	204.75	0	kip	Ay_link*Fu_link*Rt_link	Anet = 22.87 in^2
Mpr_link=	1327	0	kip*in	Value previously defined	
DCR_ColNetSect=	0.095	OK			

5.5 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=	24.10	in		Value previously defined	
tcw=	0.47	in		Value previously defined	
Agc=	24.70	in^2		Value previously defined	
Pc=	1235	kips		= Fyc* Agc	
Pu=	32.85	kips		Value previously defined	
φRn_PZ=	305.83	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	0.00	kips	= Σ Mpb/((Hcc_b+Hcc_t)/2), 0 for top story	
Pcap_Link=	77.84	0.00	kips	= Pu_link	
Ru=	77.84	0.00	kips		
DCR_PZ=	0.255	0.000		= Pr_link - Vu_c / φRn_PZ	
DCR_PZ_total=	0.255	OK		Ok, if DCR_PZ <= DCR_PZ_Limit	

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

Doubler Plate Required=	NO	<----If NO, Skip 5.4.1	USE Plug Weld?	NO
Fy_dpl=	50	ksi	Doubler Plate Yield Strength	
ϕRn_PZ_NODBLR =	305.83	kips	= ϕv_pz if ($Pu < 0.4 \cdot Pc$, $0.6 \cdot Fy_col \cdot dc \cdot tcw$, $0.6 \cdot Fy_col \cdot dc \cdot tcw \cdot (1.4 - Pu/Pc)$)	
ϕRn_PZ_req =	0.00	kips	Additional capacity required	
ϕRn_PZ_NEW =	305.8	kips	= $\phi Rn_PZ_NODBLR + \phi Rn_PZ_req$	
t_dblr_strength=	0.000	in	Doubler Plate thickness	
t_dblr_use=	0.000	in	= ϕv_pz if ($Pu < 0.4 \cdot Pc$, $0.6 \cdot (Fy_col \cdot dc \cdot tcw + Fy_dpl \cdot (dc - 2 \cdot tcf) \cdot t_dblr)$, $0.6 \cdot (Fy_col \cdot dc \cdot tcw + Fy_dpl \cdot (dc - 2 \cdot tcf) \cdot t_dblr) \cdot (1.4 - Pu/Pc)$)	
ϕRn_PZ_DBLR =	305.83	kips		
	Left Side	Right Side		
DCR_PZ_NEW=	0.255	0.000		
DCR_PZ_total_NEW=	0.255	OK		
Max Doubler Thickness	1.250	in	Maximum thickness of doubler plate to not interfere with YL column bolts	
DCR Doubler Plate Thickness	3.000	NG	Use CJP Weld or choose another link size with wider flange	
DCR_PZ_Total=	3.000		=max of DCR_PZ (strength), DCR_PZ (DP_Thickness)	

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

Option 1: Doubler without Continuity plates

t_dblr=	0.000	in	doubler plate thickness
tcw=	0.47	in	column web thickness
min t=	0.000	in	=min of tDP and tcw, rounded up to nearest 1/16
willet=	0	in	From AISC 360 Table J2.4

Option 2A: Extended Doubler Plate

t_dblr=	0.000	in	Continuity plate to doubler per Section E3.6f.2(c)
willet=	0.0000	in	doubler plate thickness (c.3, available strength of doubler plate)
willet_use=	0.0000	in	=5/8*tDP (develop shear strength of the doubler Plate), rounded to nearest 1/16
# 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=	0.000	in	doubler plate thickness
willet_use=	0.0000	in	t_dblr, same thickness to develop doubler Plate capacity
# of sides=	1		

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

Doubler Plate Plug-Weld Used?	NO	
PlugWeld_Dia=	0.313	in
Weld Depth=	0.000	in

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

Option 1: Fillet weld to develop doubler plate shear capacity

t_dblr=	0.000	in	doubler plate thickness
Vn=	0.00	kips	$0.6 \cdot Fy \cdot Agv$ (per 1 in length)
willet=	0.0	/16	Vn/1.39, rounded up to 1/16"

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

5.6 CHECK UNSTIFFENED COLUMN FLANGE AND WEB:

SMF_Sides=	1		(1 for singled sided SMF, 2 for double sided SMF)
	Left Side	Right Side	
dbm=	16.3	N/A	in d beam
tbf=	0.665	N/A	
d_stiff_cap=	3.5	N/A	
d_stiffTop=	3.5	N/A	in Location of top stiffener to top of column cap, assume 2 dc if not at top story
d_stiffBot=	19.8	N/A	in Location of bottom stiffener to top of column cap = d + d_stiffTop

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

	Left Side	Right Side	
ϕWLY =	1	1	
dc=	24.1	24.1	in Value previously defined
kdes=	1.27	1.27	in Looked up value
t_flange=	1.250	0.000	in Link flange thickness
t_stem=	0.750	0.000	in Link stem thickness
l_b=	3.25	0	in = $2 \cdot t_flange + t_stem$
Ct_top=	0.5	1	in = if ($d_stiffTop \leq dc$, 0.5, 1)
Ct_bot=	0.5	1	in = if ($d_botTop \leq dc$, 0.5, 1)
$\phi Rn_WLYmoreD$ =	225.60	149.23	kips = $\phi WLY \cdot Fy_col \cdot tcw \cdot (5 \cdot kdes + l_b)$
$\phi Rn_WLYlessD$ =	150.99	74.61	kips = $\phi WLY \cdot Fy_col \cdot tcw \cdot (2.5 \cdot kdes + l_b)$
ϕRn_WLY_top =	150.99	149.23	kips = if ($d_stiffTop > dc$, $\phi Rn_WLYmoreD$, $\phi Rn_WLYlessD$)
ϕRn_WLY_bot =	150.99	149.23	kips = if ($d_stiffTop > dc$, $\phi Rn_WLYmoreD$, $\phi Rn_WLYlessD$)

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

$\phi WLC =$	0.75		
$E_s =$	29000	ksi	
$t_{cf} =$	0.77	in	Value previously defined
$tcw =$	0.47	in	Value previously defined
$\phi R_n \text{ WLC more}0.5D =$	243.7	kip	$= \phi WLC * 0.8 * tcw^2 * (1 + 3 * (L_b/dc) * (tcw/tcf)^{1.5}) * \text{Sqrt}(E_s * F_y \text{ col} * tcf / tcw)$
AISC J10-5a =	121.85	kip	$= 0.5 * \phi R_n \text{ WLC more}0.5D$
AISC J10-5b =	118.67	kip	$= \phi WLC * 0.4 * tcw^2 * (1 + (4 * L_b/dc - 0.2) * (tcw/tcf)^{1.5}) * \text{Sqrt}(E_s * F_y \text{ col} * tcf / tcw)$
$\phi R_n \text{ WLC less}0.5D =$	121.85	kip	$= \text{if } (L_b/dc \leq 0.2, \text{AISC J10-5a, AISC J10-5b})$
	Left Side	Right Side	
$\phi R_n \text{ WLC top} =$	121.85	243.7	kip
$\phi R_n \text{ WLC bot} =$	243.7	243.7	kip
			$= \text{if } (d_{stiffTop} > dc/2, \phi R_n \text{ WLC more}0.5D, \phi R_n \text{ WLC less}0.5D)$
			$= \text{if } (d_{stiffBot} > dc/2, \phi R_n \text{ WLC more}0.5D, \phi R_n \text{ WLC less}0.5D)$

5.9 COLUMN WEB COMPRESSION BUCKLING (AISC 360 J10.5):

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

$\phi WCB =$	0.9		
$h =$	21.56	in	$= dc - 2 * k_{des}$
$\phi R_n \text{ WCB} =$	125.3	kip	$\text{AISC J10-8, } = \phi WCB * 24 * tcw^3 * \text{sqrt}(E_s * F_y \text{ col}) / h$
	Left Side	Right Side	
$\phi R_n \text{ WCB top} =$	62.63	125.25	kip
$\phi R_n \text{ WCB bot} =$	125.25	125.25	kip
			$= \text{if } (d_{stiffTop} < d, \phi R_n \text{ WCB} * 0.5, \phi R_n \text{ WCB})$
			$= \text{if } (d_{stiffBot} < d, \phi R_n \text{ WCB} * 0.5, \phi R_n \text{ WCB})$

5.10 SUMMARY OF LOCAL CAPACITIES:

	Left Side				At Bottom Stiffener		
At Top Stiffener							
$\phi R_n \text{ WLY top} =$	150.99	kip			$\phi R_n \text{ WLY bot} =$	150.99	kip
$\phi R_n \text{ WLC top} =$	121.85	kip			$\phi R_n \text{ WLC bot} =$	243.7	kip
$\phi R_n \text{ WCB top} =$	62.63	kip			$\phi R_n \text{ WCB bot} =$	125.25	kip
$\text{if } (SMF_Sides = 1, \phi R_n \text{ Stiff top} = \min(\phi R_n \text{ WLY top}, \phi R_n \text{ WLC top}, \phi R_n \text{ WCB top}), \phi R_n \text{ Stiff bot} = \min(\phi R_n \text{ WLY bot}, \phi R_n \text{ WLC bot}, \phi R_n \text{ WCB bot}))$							
$\phi R_n \text{ Stiff top} =$	121.85	kip			$\phi R_n \text{ Stiff bot} =$	150.99	kip
Top Stiffener Required =	No				Bottom Stiffener Required =	No	
Top Stiff Provided?:	YES				Bottom Stiff Provided?:	YES	
							Stiff Req'd= NO
	Right Side				At Bottom Stiffener		
At Top Stiffener							
$\phi R_n \text{ WLY top} =$	149.23	kip			$\phi R_n \text{ WLY bot} =$	149.23	kip
$\phi R_n \text{ WLC top} =$	243.69	kip			$\phi R_n \text{ WLC bot} =$	243.7	kip
$\phi R_n \text{ WCB top} =$	125.25	kip			$\phi R_n \text{ WCB bot} =$	125.25	kip
$\text{if } (SMF_Sides = 1, \phi R_n \text{ Stiff top} = \min(\phi R_n \text{ WLY top}, \phi R_n \text{ WLC top}, \phi R_n \text{ WCB top}), \phi R_n \text{ Stiff bot} = \min(\phi R_n \text{ WLY bot}, \phi R_n \text{ WLC bot}, \phi R_n \text{ WCB bot}))$							
$\phi R_n \text{ Stiff top} =$	149.23	kip			$\phi R_n \text{ Stiff bot} =$	149.23	kip
Top Stiffener Required =	No				Bottom Stiffener Required =	No	
Top Stiff Provided?:	YES				Bottom Stiff Provided?:	YES	
							Stiff Req'd= NO

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

Note: Assume compression flange is NOT restrained against rotation

	Left Side	Right Side		
$\phi WSB =$	0.85	0.85		
$H_{cc} =$	186.80	186.80	in	Value previously defined
$b_{cf} =$	9.02	9.02	in	Column flange width
$Sc_x =$	196.00	196.0	in^3	Looked up value
$L_b =$	178.7	N/A	in	$= H_{cc} - d/2$
$M_y =$	9800.0	9800.0	kip*in	$= Sc_x * F_{yc}$
$M_u =$	1327	0	kip*in	$= M_{pr}$
$C_r =$	960000	960000	ksi	$= \text{if } (M_u < M_y, 960000, 480000)$
$(h/tw)/(L_b/b_{cf}) =$	2.32	N/A		$= (h/tw)/(L_b/b_{cf})$
$\phi R_n \text{ WSB} =$	697	N/A	kip	$\text{AISC J10-7, } = (\phi WSB * C_r * tcw^3 * t_{cf} / h^2) / (0.4 * C_r^3)$
WSB check =	N/A	N/A		Not applicable
BotStiffener_Bracing_Req'd =	No	No		$= \text{if } (And(\phi R_n \text{ WSB} < Pr_link, \text{WSB_check} = \text{"Applicable"}), \text{"Yes"}, \text{"No"})$

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

	Left Side	Right Side		
$F_{su} \text{ top} =$	0.0	0.0	kip	$= \text{if } (Pr_link - \phi R_n \text{ Stiff top} < 0, 0, Pr_link - \phi R_n \text{ Stiff top})$
$F_{su} \text{ bot} =$	0.0	0.0	kip	$= \text{if } (Pr_link - \phi R_n \text{ Stiff bot} < 0, 0, Pr_link - \phi R_n \text{ Stiff bot})$
$F_{su} =$	0.0	0.0	kip	$= \max(F_{su} \text{ top}, F_{su} \text{ bot}) * SMF_Sides$

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

$\phi t =$	0.9			Value previously defined
$F_y \text{ stiff} =$	50	ksi		Value previously defined
$K_{det} =$	1.688	in		Looked up value
$K_1 =$	1.063	in		Looked up value
$tcw_{det}/2 =$	0.250	in		Looked up value
$t_{cf}(det) =$	0.750	in		Looked up value
	Left Side	Right Side		
$As \text{ min} =$	0.000	0.00	in^2	$= F_{su} / (t * F_y \text{ Stiff})$
$L_{clip} \text{ web min} =$	2.438	2.438	in	$= K_{det} - t_{cf} + 1.5$
$L_{clip} \text{ web} =$	2.438	2.438	in	$= \text{Ceiling}(L_{clip} \text{ web min}, 0.0625)$
$L_{clip} \text{ flange min} =$	0.813	0.813	in	$= K_1 - tcw_{det}/2$
$L_{clip} \text{ flange} =$	0.813	0.813	in	$= \text{Ceiling}(L_{clip} \text{ flange min}, 0.0625)$
$bstiff \text{ min} =$	2.7717	2.7717	in	$\text{AISC 360 J10.8, } = b_{cf}/3 - tcw/2$
$bstiff \text{ max} =$	4.275	4.275	in	$= (b_{cf} - tcw)/2$
$bstiff =$	4.2500	4.2500	in	$= \text{Floor}(bstiff \text{ max}, 0.125), \text{ Each side of column web}$
$tstiff \text{ USE?} =$	YES	YES	in	Input value
$t \text{ stiff tension} =$	0.000	0.000	in	$= As \text{ min} / (bstiff \text{ min} * L_{clip} \text{ flange}) / 2, SMF_Sides, OK, \text{ if } t \text{ stiff tension} \leq tstiff$
$tstiff \text{ min} =$	0.375	0.000	in	$= \max(t \text{ stem}/2, bstiff/16, \text{looked up value}), OK, \text{ if } tstiff \text{ min} \leq tstiff$
$tstiff \text{ max} =$	0.750	0.000	in	$= t \text{ stem}, OK, \text{ if } tstiff \text{ max} \geq tstiff$
$t \text{ stiff Use} =$	0.375	0.375		
$Lstiff \text{ Pdepth ini} =$	11.28		in	$= 0.5 * dc - t_f, \text{ For Single sided connections (partial depth)}$
$Lstiff \text{ Fdepth ini} =$	22.56		in	$= dc - 2 * t_{cf}, \text{ For double sided connections}$
$Lstiff \text{ ini} =$	22.56		in	$= \text{if } (and(SMF_Sides = 1, \text{Ext. Stiffener depth} = \text{"Partial"}), Lstiff \text{ Pdepth ini}, Lstiff \text{ Fdepth ini})$
$Lstiff =$	22.50		in	$= \text{Floor}(Lstiff \text{ ini}, 0.0625)$

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

	Left Side	Right Side		
ϕ_c =	0.9	0.9		
Kstiff=	0.75	0.75		
Astiff_fd=	5.84	5.84	in ²	= 2* bstiff* tstiff + 12* tcw ²
Lstiff_fd=	22.60	22.60	in ⁴	= 12* tcw ³ / 3 + 2* (bstiff* bstiff ³ / 12 + bstiff* tstiff* (bstiff/2 + tc/2) ²)
rstiff_fd=	1.97	1.97	in	= Sqrt (Lstiff_fd/ Astiff_fd)
Kstiff*Lstiff_fd/rstiff_fd=	8.58	8.58		= Kstiff* Lstiff_fd/ rstiff_fd
Fe=	3891	3891	ksi	= Pi ² * 29000/ (Kstiff* Lstiff_fd/ rstiff_fd) ²
Fcr=	49.73	49.73	ksi	= if (Kstiff* Lstiff_fd/ rstiff_fd <= 4.71* Sqrt (29000/ Fy_stiff), 0.658* (Fy_stiff/ Fc)* Fy_stiff,
Pn_stiff=	291.9	291.9	kips	= Kstiff* Lstiff_fd* rstiff_fd > 25, Fcr* Astiff_fd, Astiff_fd* Fy_stiff
ϕ_c *Pn_stiff=	262.7	262.7	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

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

ϕ_{fillet} =	0.75		
Fexx=	70	ksi	
tstiff=	0.375	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=	0.0000	0.0000	in
wfillet_flange=	0.1875	0.1875	in
wfillet_flange_DCR=	0.000	0.000	

= (0.5* Fsu* SMF_Sides)/ (ϕ_{fillet} 1.5* 0.6* Fexx* (bstiff - Lclip_flange-0.5)*2^{1/2})
=Max (wfillet_min, ceiling(wfillet_flange_min, 1/16"))
= wfillet_flange_min/ wfillet_flange

5.16 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
wfillet_min_web=	0.0000	0.0000	in
wfillet_web=	0.1875	0.1875	in
wfillet_web_DCR=	0.000	0.000	

Minimum fillet weld size per AISC 360 Table J2.4
= 0.5* Fsu* SMF_Sides/ (ϕ_{fillet} 0.6* Fexx* (Lstiff - Lclip_web-0.5)*2^{1/2})
= max(wfillet_min, ceiling(wfillet_web, 1/16"))

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

	Left Side	Right Side	
bcf=	9.0	9.0	in
bcf_LinkFlg_bolt=	7.5	0	in
bcf_DCR=	0.831	0.000	OK

=column flange width
=Link Sflange+2*Ledge_min

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

	Left Side	Right Side	
ϕ_b =	0.9	0.9	
tcf=	0.77	0.77	in
c=	5.750	0.000	in
g=	5.000	0.000	in
H_flange=	9.250	0.000	in
s=	3.358	0.000	in
ps0=	2.313	0.000	in
psi_tmp=	2.313	0.000	in
psi=	2.313	0.000	in
h1=	14.55	0	in
h0=	19.55	0	in
Yc_unstiffened=	137.05	0.00	in
tcf_req_unstiffened=	0.49	0.00	in
Yc_stiffened=	189.65	0.00	in
tcf_req_stiffened=	0.41	0.00	in
Stiffened?=	YES	YES	
tcf_min=	0.414	0.000	in
DCR_colFLB1=	0.537	0.000	

Value previously defined
= bolt_g_flange
= bolt_s_flange
Value previously defined
= 0.5* Sqrt (bcf* g)
= (g - tstiff)/2
= ps0
= if (psi_tmp > s, s, psi_tmp)
= db + t_stem - g/2
= db + t_stem + g/2
= bcf/2* (h1/s + h0/s) + 2/g* (h1* (s + 3*c/4) + h0* (s + c/4) + c²/4) + g/2
= Sqrt (1.1* Mcap/ (ϕ_b * Fyc* Yc_unstiffened))
= bcf/2* (h1* (1/s + 1/psi) + h0* (1/s + 1/ps0)) + 2/g* (h1* (s + psi) + h0* (s + ps0))
= Sqrt (1.1* Mcap/ (ϕ_b * Fyc* Yc_stiffened))
Bottom Stiffener Required per 17.3
= if (Stiffened? = YES, tcf_req_stiffened, tcf_req_unstiffened)
= tcf_min/ tcf

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

	Left Side	Right Side	
de=	1.25	1.25	in
pfi=	2.313	0.000	in
ps0=	2.313	0.000	in
Yp=	160.0	0.000	in
tcf_req=	0.450	0.000	in
DCR_colFLB2=	0.585	0.000	

centerline of top link bolt to top of column
= (g - tstiff)/ 2
Value previously defined
= bcf/2* (h1* (1/pfi + 1/ps0) + h0* (1/ps0 + 1/2/s)) + 2/g* (h1* (pfi + s) + h0* (d2 + ps0))
= Sqrt* (1.1* Mcap/ (ϕ_b * Fy* Yp))
= tcf_req/ tcf

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

	Left Side	Right Side	
Link Size=	YLMC6-3.5	N/A	
byield=	3.500	0.000	in
tstem=	0.750	0.000	in
Fy_link=	50	50	ksi
Ry=	1.1	1.1	
Pye_link=	144.375	0	kips
0.02 * Pye_link=	2.888	0.000	kips
Pbrace (ASD)=	2.021	0.000	kips

AISC 341, E3.4c.1(b)
Bracing Force (LRFD)
= 0.7*Bracing Force (LRFD)

5.19 DESIGN SUMMARY:

DCR Summary:		Report:
SCWB_DCR_total=	N/A	N/A
DCR_PZ_total=	0.255	Overall PZ chk: NG
DCR_PZ_total_NEW=	3.000	NO 0.255
DCR_stiff_comp_DCR=	0.000	0.000 0.000
wfillet_flange_DCR=	0.000	0.000 0.000
wfillet_web_DCR=	0.000	0.000 0.000
DCR_colFLB1=	0.537	0.000 0.537
DCR_colFLB2=	0.585	0.000 0.585
DCR_colFLB=		0.585

Geometry/Weld Summary:		Report:
DP used?	NO	PlugWeld? NO
t dblr use=	0.000	0.000
Stiffener Req'd=	NO	NO
t stiff Use=	0.375	0.375 0.375
wfillet_flange=	0.188	0.188 0.188
wfillet_web=	0.188	0.188 0.188

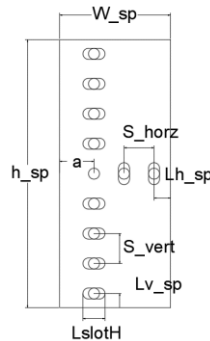
4.1 CURRENT MEMBER:

Beam Unique Name:	1001	Shear Plate Info:		
Beam Size:	W16X67	Thickness (tsp) (in):	0.38	in
Axial Pu (kips):	9.80	Bolt Dia (db_sp):	0.875	in
Shear V_Grav. SST_LC08 (kips):	3.84	No. of SP (n_sp):	1	
Shear Vu_Grav. SST_LC01-07(kips):	5.16	Bolt Type:	A325-N	
Link:	YLMC6-3.5	n_Vbolts:	3	
tstem (in):	0.75	n_Hbolts:	2	
		R=	3.5	
		Fy_bm=	50	ksi
		Fu_bm=	65	ksi
		Left Col=	W24X84	
		Right Col=	W24X84	
		Left_Conn=	YES	
		Right_Conn=	YES	
		no. of Conn =	2	

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

Beam Size=	W16X67	User Input Beam size	
db=	16.30	in	From AISC Database
tbf=	0.67	in	From AISC Database
tbw=	0.40	in	From AISC Database
Fy_sp=	50	ksi	User Input/Selection
Fu_sp=	65	ksi	User Input/Selection
Axial Load (Pu_sp)=	9.80	kips	Maximum Beam Axial Force from Overstrength Load Combinations
Lcc=	251	in	Column-to-column Centerline dimension. User Input value
Lh=	220.00	in	=Lcc - dc - 2*a
Pcap_link=	204.75	kips	=If (R=3, Py_link, Pr_link)
Mcap=	3491	kip-in	=Pcap_link*(db + tstem)
Vcap_link=	31.74	kips	=Mcap*(No. of Connections) / Lh
Vertical Load (Vu_bm)=	35.58	kips	= Vcap_link + Vbm_gravity (EQ 12.9-34)
No. of vertical Bolts (n_Vbolts)=	3.000		Looked up value
No. of horizontal Bolts (n_Hbolts)=	2.000		User Input
Vu_bolt=	12.83	kips	= Max(Sqrt ((Pu_sp/ n_Hbolt)^2 + (Vu_bm/ n_Vbolt)^2), (EQ 12.9-35)
φbolt=	0.75		Vu_grav.SSTLC01-07/n_Vbolts)
Bolt Type (Bolt_Gr_shearTab)=	A325-N		
Bolt Dia (db_sp)=	7/8	in	
Frv_A325N=	54.00	ksi	
Anb_sp=	0.601	in ²	= Pi*(db_sp/2)^2
Rn_stBolt_shear=	32.47	kips	= Frv_A325N* Anb_sb
φRn_stBolt_Shear=	24.35	kips	= Rn_stBolt_shear* φbolt * n_sp
DCR_shearTab_Bolt=	0.527		= 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=	0.375	in	[default to try and match beam web thickness]
db=	16.30	in	Value previously defined
Bolt Spacing (S_min)=	2.000	in	= 2.0*d_b_sp-hole (clear distance not less than d_b_sp-hole), see section J3.3
S_max=	2.650	in	
Bolt Spacing (Svert)=	2.625	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=	7.55	in	= db + tstem - h_flange - (2*h_clear)
Plate Depth (h_sp)=	7.50	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=	1.368	in	= db_sp + 1/8 + (2*0.07* Svert*(n_Vbolts - 1)/2)
LslotH = LslotV=	1.375	in	OK if Lslot_min/ Lslot <= 1.03
Shear Plate Geometry Check=	OK		OK if And (h_sp=OK, Lh_sp = OK, Lslot = OK)

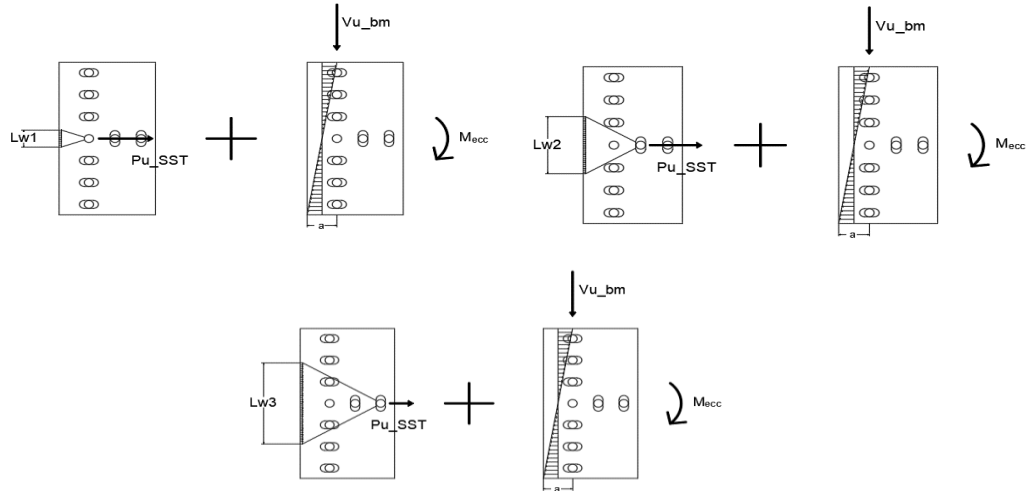
4.4 SHEAR PLATE YIELDING (VERTICAL): (AISC 358 Chapter 12, Step 15.3)

$\phi_{yield} =$	1			
$d_{Hole_sp} =$	1.00	in	$= db_sp + 1/8$	
$Asp_Agv =$	2.813	in ²	$= tsp * h_sp$	
$\phi V_{y_sp} =$	84.38	kips	$= \phi_{yield} * 0.6 * Asp_Agv * F_{y_sp} * n_sp$	
$DCR_sp_{Yield} =$	0.422		$= V_{u_bm} / \phi V_{y_sp}$	
Check=	OK		$OK \text{ if } DCR_sp_{Yield} \leq DCR_sp_{Yield_allowed}$	

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

$\phi_{rupture} =$	0.75			
$Asp_nv =$	1.69	in ²	$= hsp * tsp - d_{Hole_sp} * n_Vbolts * tsp$	
$\phi V_{rupture_sp} =$	49.36	kips	$= \phi_{rupture} * 0.6 * F_{u_shearTab} * Asp_nv * n_sp$	
$DCR_sp_{Rupture} =$	0.721		$= V_{u_bm} / \phi V_{rupture_sp}$	
Check=	OK		$OK \text{ if } DCR_sp_{Rupture} \leq DCR_sp_{Rupture_allowed}$	

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



$\phi_b =$	0.90			
$a =$	3.50	in	<i>Value previously defined</i>	
$V_{uy} =$	35.58	kips	$= V_{u_bm}$	
$Mecc =$	124.53	kip-in	$= V_{uy} * a$	
$\theta_{whitmore} =$	30	deg		
$L_{whitmore1} =$	4.916	in	$= \tan(\theta_{whitmore}) * a^2 + db_sp$	
$A_{whitmore1} =$	1.844	in ²	$= L_{whitmore1} * tsp * n_sp$	
$L_{whitmore2} =$	7.500	in	$= \min(\tan(\theta_{whitmore}) * (a + Shorz)^2 + db_sp, hsp)$	
$A_{whitmore2} =$	2.813	in ²	$= L_{whitmore2} * tsp * n_sp$	
$L_{whitmore3} =$	7.500	in	$= \min(\tan(\theta_{whitmore}) * (a + 2 * Shorz)^2 + db_sp, hsp)$	
$A_{whitmore3} =$	2.813	in ²	$= L_{whitmore3} * tsp * n_sp$	
$L_{whitmore4} =$	7.500	in	$= \min(\tan(\theta_{whitmore}) * (a + 3 * Shorz)^2 + db_sp, hsp)$	
$A_{whitmore4} =$	2.813	in ²	$= L_{whitmore4} * tsp * n_sp$	
$S_{sp} =$	3.516	in ³	$= (tsp * hsp^2 * n_sp) / 6$	
$I_{sp} =$	13.184	in ⁴	$= (tsp * hsp^3 * n_sp) / 12$	
$f_{b1} =$	35.423	ksi	$= Mecc / S_{sp}$	
$A_{whitmore} =$	2.813	in ²	$= IF(n_Hbolts=1, A_{whitmore1}, IF(n_Hbolts=2, A_{whitmore2}, IF(n_Hbolts=3, A_{whitmore3}, A_{whitmore4})))$	
$L_{whitmore} =$	7.500	in	$= IF(n_Hbolts=1, L_{whitmore1}, IF(n_Hbolts=2, L_{whitmore2}, IF(n_Hbolts=3, L_{whitmore3}, L_{whitmore4})))$	
$y_b =$	3.750	in	$= L_{whitmore} / 2$	
$f_{b2} =$	35.423	ksi	$= Mecc * y_b / I_{sp}$	
$f_{a2} =$	3.484	ksi	$= P_{u_sp} / A_{whitmore}$	
$f_{tot2} =$	38.907	ksi	$= f_{a2} + f_{b2}$	
$f_{max_sp} =$	38.907	ksi	$= \max(f_{b1}, f_{tot2})$	
$\phi_b * F_{y_sp} =$	45.000	ksi	$= \phi_b * F_{y_sp}$	
$DCR_sp =$	0.865		$= f_{max_sp} / (\phi_b * F_{y_sp})$	
Check=	OK		$= OK \text{ if } DCR_sp \leq DCR_sp_allowed$	

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

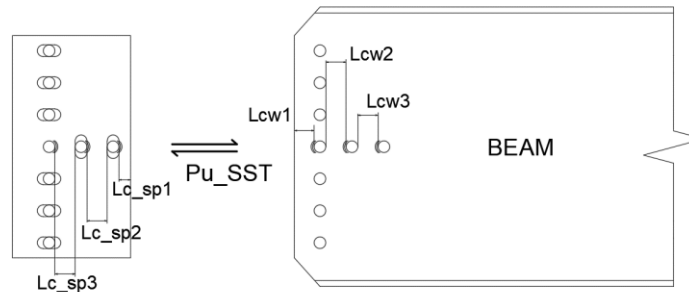
ϕ =	0.75		Value previously defined
tsp=	0.375	in	Value previously defined
tw_sp_min=	0.234	in	= $5/8 \cdot tsp$ (Fillet size required to develop plate capacity as per AISC Steel Manual 14th edition)
tw_sp=	0.250	in	= $CEILING(tw_sp_min, 1/16)$
Check=	OK		= OK if $DCR_tw_spWeld \leq DCR_tw_spWeld_allowed$
Vu_bm=	35.581	kips	Value previously defined
Lweld=	7.000	in	= Length per side assuming 1/4" hold off top and bottom
F _{EXX} =	70.00	ksi	= Filler metal classification strength
F _{nw} =	42.000	ksi	= $0.6 \cdot F_{EXX}$
Awe=	2.475	in ²	= $0.707 \cdot (2 \cdot Lweld) \cdot tw_sp$
ϕR_n =	77.947	kips	= $\phi \cdot F_{nw} \cdot Awe$
DCR_spWeld=	0.456		= Vu_bm / ϕR_n
Check=	OK		= OK if $DCR_spWeld \leq DCR_spWeld_allowed$

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

tsp=	0.375	in	Same thickness for both shear plates
ϕ_{PJP} =	0.75		
nsides=	1		
t _{eff_PJP} =	0.313	in	= $3/4 \cdot tsp$
Vu_PJP=	17.791	kips	= Vu_bm / 2
Aw=	2.188	in ²	= t _{eff_PJP} * Lweld * nsides
ϕR_n =	68.906	kips	= $\phi_{PJP} \cdot 0.6 \cdot F_{EXX} \cdot Aw$
DCR_PJPWeld1=	0.258		= Vu_PJP / ϕR_n
PJP weld P+M Check:			
ϕ =	0.8		
A _{PJP} =	2.344	in	= t _{eff_PJP} * h _{sp}
I _{PJP} =	10.986	in ⁴	= t _{eff_PJP} * h _{sp} ³ / 12
f _{PJP} =	23.344	ksi	= $0.5 \cdot Pu_sp / A_{PJP} + 0.5 \cdot Mecc \cdot yb / I_{PJP}$
f _{PJP_allowed} =	33.600	ksi	= $\phi \cdot 0.6 \cdot F_{EXX}$
DCR_PJPWeld2=	0.695		= Vu_PJP / ϕR_n
DCR_PJPWeld=	0.695		= max(DCR_PJPWeld1, DCR_PJPWeld2)
Check=	OK		= OK if $DCR_PJPWeld \leq DCR_PJPWeld_allowed$

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

CASE 1: HORIZONTAL REACTIONS



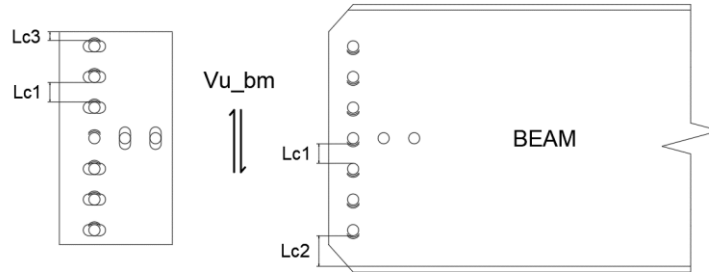
BEAM WEB:

ϕ_{bolt} =	0.75		Value previously defined
t _{bw} =	0.40	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 _{Hbolt_SST} = 2, 0, Lcw2)
Lcw4=	0.000	in	= if (n _{Hbolt_SST} = 3, 0, Lcw3)
Lc _{bmWeb} =	3.094	in	= Lcw1 + Lcw2 + Lcw3 + Lcw4
ϕR_n _{beamWeb1} =	71.49	kips	= $\phi_{bolt} \cdot 1.2 \cdot Lc_bmWeb \cdot t_{bw} \cdot Fu_bm$
ϕR_n _{beamWeb2} =	80.88	kips	= f _{bolt} * 2.4 * db _{sp} * t _{bw} * Fu _{bm} * n _{Hbolts}
ϕR_n _{beamWeb} =	71.49	kips	= min(ϕR_n _{beamWeb1} , ϕR_n _{beamWeb2})
DCR _{bmWebX} =	0.137		= Pu _{sp} / ϕR_n _{beamWeb}
Check=	OK		= OK if $DCR_bmWebX \leq DCR_bmWebX_allowed$

SHEAR PLATE:

tsp=	3/8	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}
ϕR_n _{sp1} =	63.18	kips	= IF(n _{Hbolt_SST} =1, $\phi_{bolt} \cdot (1.2 \cdot Lc_sp1 \cdot tsp \cdot Fu_sp \cdot n_sp)$, $\phi_{bolt} \cdot (1.2 \cdot Lc_sp2 + 1.0 \cdot (Lc_sp1 + Lc_sp3 + Lc_sp4)) \cdot tsp \cdot Fu_sp \cdot n_sp$)
ϕR_n _{sp2} =	70.38	kips	= $\phi_{bolt} \cdot (2.4 \cdot 1 + 2.0 \cdot (n_Hbolts - 1)) \cdot db_sp \cdot tsp \cdot Fu_sp \cdot n_sp$
ϕR_n _{sp} =	63.18	kips	= min(ϕR_n _{sp1} , ϕR_n _{sp2})
DCR _{spX} =	0.155		= Pu _{sp} / ϕR_n _{sp}
Check=	OK		= OK if $DCR_spX \leq DCR_spX_allowed$

CASE 2: VERTICAL REACTIONS



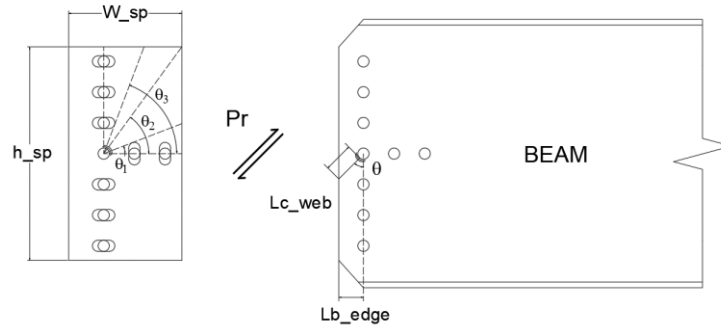
BEAM WEB:

ϕ bolt=	0.75		Value previously defined
Lc1=	1.688	in	= Svert - (db_sp + 1/16)
Lc2=	4.860	in	= (db - (n_Vbolts - 1))*Svert - 2tbw*0.5
Lcb_Vert=	8.235	in	= Lc2 + (n_Vbolts - 1)*Lc1
ϕ *Rn_bmWebY1=	190.29	kips	= ϕ bolt* 1.2* Lcb_Vert* tbw* Fu_bm
ϕ *Rn_bmWebY2=	121.31	kips	= ϕ bolt* 2.4* (db_sp* n_bolts_SST)* tbw* Fu_bm
ϕ *Rn_bmWebY=	121.31	kips	= min (ϕ *Rn_bmWebY1, ϕ *Rn_bmWebY2)
DCR_bmWebY=	0.293		= 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=	4.031	in	= Lc3 + (n_Vbolts - 1)*Lc1
ϕ *Rn_spY1=	79.87	kips	= ϕ bolt* (1.2*Lc1 + 1.0*(Lc3 + (n_Vbolts - 2))*Lc1)*tsp* Fu_sp *n_sp
ϕ *Rn_spY2=	102.38	kips	= ϕ bolt* (2.4*1+2.0*(n_bolts_SST-1))* db_sp* tsp* Fu_sp *n_sp
ϕ *Rn_spY=	79.87	kips	= min (ϕ *Rn_spY1, ϕ *Rn_spY2)
DCR_spY=	0.446		= Vu_bm / (ϕ * Rn_spY)
Check=	OK		= OK if DCR_spY <= DCR_spY_allowed

CASE 3: COMBINED AXIAL AND VERTICAL REACTIONS



Px=	4.90	kips	= Pu_sp/ n_Hbolts
Py=	11.86	kips	= Vu_bm/ n_Vbolts
Resultant (Pr)=	12.83	kips	= Sqrt (Px^2 + Py^2)
theta=	1.179	radians	= min(Atan (Py/ Px), 1.571))

BEAM WEB:

Method 1: T and V Circular Interaction

$$DCR_{bmWeb\theta Bearing1} = 0.105 = DCR_{bmWebX}^2 + DCR_{bmWebY}^2$$

Method 2: Bearing in a Diagonal Line

Lvg_web =	4.583	in	= Lb_edge/ (cos theta)
Lc_web =	4.083	in	= Lvg_web - dHole_sp/2
phi*Rn_bmWeb01=	94.358	kips	= phi*bolt* 1.2* Lc_web* tbw* Fu_bm
phi*Rn_bmWeb02=	80.876	kips	= fbolt* 2.4* db_sp* tbw* Fu_bm* n_Hbolts
phi*Rn_bmWeb0=	80.88	kips	= min (phi* Rn_bmWeb 01, phi* Rn_bmWeb 02)
DCR_kipWeb0Bearing2 =	0.159		= Pr / phi*Rn_bmWeb 0
DCR_bmWeb0Bearing =	0.159		= Min (DCR_bmWeb 0Bearing1, DCR_bmWeb 0Bearing2)
	OK		= OK if DCR_bmWeb 0 <= DCR_bmWeb 0_allowed

SHEAR PLATE:

Method 1: T and V Circular Interaction

$$DCR_{sp0Bearing1} = 0.223 = DCR_{spX}^2 + DCR_{spY}^2$$

Method 2: Bearing in a Diagonal Line

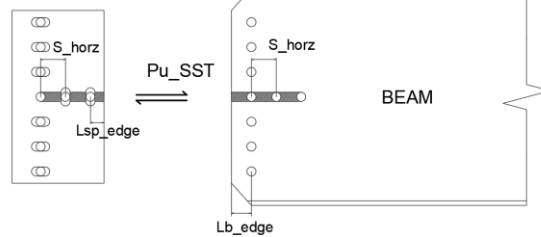
01=	0.2450	radians	= arctan((0.5*LslotH)/Shorz)
02=	0.6947	radians	= arctan((0.5*h_sp)/(w_sp-a))
0'=	0.2562	radians	= arctan((0.5*LslotV)/Svert)
03=	1.3146	radians	= (Pi()*0.5)- 0'
0=	1.1790	radians	Value previously defined

	CASE 1	CASE 2	CASE 3	CASE 4	
min. 0=	0	01	02	03	radians
max 0=	01	02	03	1.571	radians
Check=	NO	NO	YES	NO	
La_sp1=	-	-	4.058	-	in
La_sp=	4.058				in
phi*Rn_sp01=	89.016	kips			= phi*bolt* 1.2* La_sp* tsp* Fu_sp * n_sp
phi*Rn_sp02=	76.781	kips			= fbolt* 2.4* db_sp* tsp* Fu_sp* n_Hbolts * n_sp
phi*Rn_sp0=	76.78	kips			= min (phi* Rn_sp 01, phi* Rn_sp 02)
DCR_sp0bearing2=	0.167				= Pr / (phi* Rn_sp 0)
DCR_sp0bearing=	0.223				= max(DCR_sp 0Bearing1, DCR_sp 0bearing2)
Check=	OK				OK if DCR_sp 0bearing <= DCR_sp 0bearing_allowed

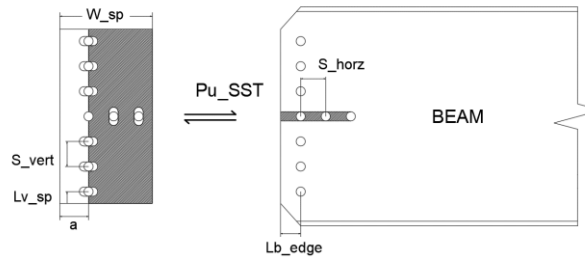
4.9 BEAM WEB AND SHEAR TAB BLOCKSHEAR CHECK: (AISC 358 Chapter 12, Step 15.5)

CASE 1: HORIZONTAL REACTIONS

CASE 1



CASE 2



BEAM WEB:

ϕ blockshear=	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=	3.555	in ²	$= Lh_bmWeb * tbw$
Ant_bmWebHorz=	0.000	in ²	$= 0$
Anv_bmWebHorz=	3.555	in ²	$= Agv_bmWebHorz$
Rn_bmWebHorz1=	138.65	kips	$= 0.6 * Fu_bmWeb * Anv_bmWebHorz + Ubs * Fu_bmWeb * Ant_bmWebHorz$
Rn_bmWebHorz2=	106.65	kips	$= 0.6 * Fy_bmWeb * Agv_bmWebHorz + Ubs * Fu_bmWeb * Ant_bmWebHorz$
Rn_bmWebHorz=	79.99	kips	$= \phi blockshear * MIN (Rn_bmWebHorz1, Rn_bmWebHorz2)$
DCR_bmWebHorz_BS=	0.123		$= Pu_sp / Rn_bmWebHorz$
Check=	OK		$= OK \text{ if } DCR_bmWebHorz_BS \leq DCR_bmWebHorz_BS_allowed$

SHEAR PLATE:

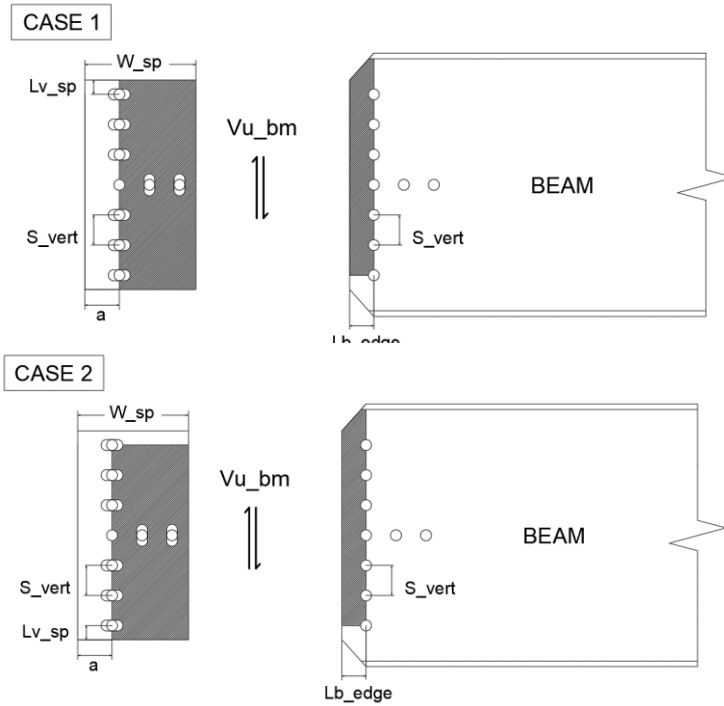
CASE 1:

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=	3.375	in ²	$= Lh_sp1 * tsp * n_sp$
Ant_spHorz1=	0.000	in ²	$= 0$
Anv_spHorz1=	3.375	in ²	$= Agv_spHorz1$
Rn_spHorz1=	131.63	kips	$= 0.6 * Fu_sp * Anv_spHorz1 + Ubs * Fu_sp * Ant_spHorz1$
Rn_spHorz2=	101.25	kips	$= 0.6 * Fy_sp * Agv_spHorz1 + Ubs * Fu_sp * Ant_spHorz1$
$\phi Rn_spHorz1=$	75.94	kips	$= \phi blockshear * min (Rn_spHorz1, Rn_spbHorz2)$
DCR_spHorz_BS1=	0.129		$= Pu_sp / Rn_spHorz1$

CASE 2:

Lh_sp2=	4.500	in	$= h_sp - (n_Vbolts * dHole_sp)$
Agv_spHorz2=	0.000	in ²	$= 0$
Ant_spHorz2=	1.688	in ³	$= Lh_sp2 * tsp * n_sp$
Anv_spHorz2=	0.000	in ²	$= 0$
Rn_spHorz3=	109.69	kips	$= 0.6 * Fu_sp * Anv_spHorz2 + Ubs * Fu_sp * Ant_spHorz2$
Rn_spHorz4=	109.69	kips	$= 0.6 * Fy_sp * Agv_spHorz2 + Ubs * Fu_sp * Ant_spHorz2$
$\phi Rn_spHorz2=$	82.27	kips	$= \phi blockshear * min (Rn_spHorz3, Rn_spbHorz4)$
DCR_spHorz_BS2=	0.119		$= Pu_sp / Rn_spHorz2$
DCR_spHorz_BS=	0.129		$= MAX (DCR_spHorz_BS1, DCR_spHorz_BS2)$
Check=	OK		$= OK \text{ if } DCR_spHorz_BS \leq DCR_spHorz_BS_allowed$

CASE 2: VERTICAL REACTIONS



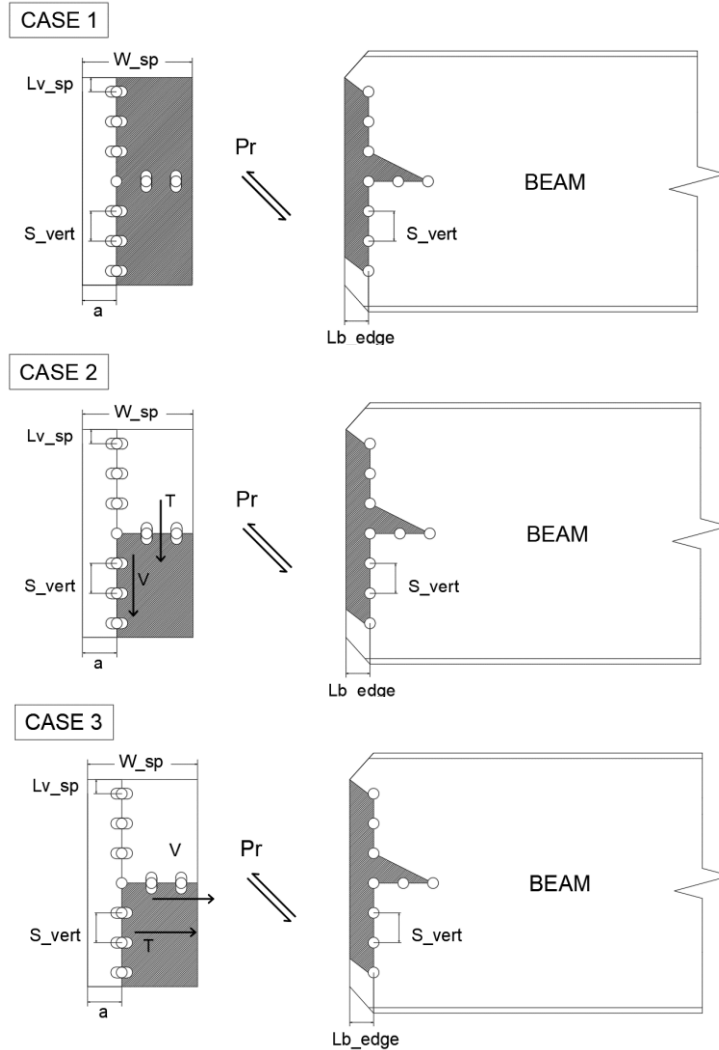
BEAM WEB:

S_vert=	2.63	in	value previously defined
Lb_edge=	1.75	in	value previously defined
h_bmWeb=	10.110	in	$= (d - (n_Vbolt - 1) * Svert - 2tb_f) * 0.5 + 2 * (n_Vbolt - 1) * Svert$
Agv_bmWeb1=	3.993	in^2	$= tbw * h_bmWeb$
Anv_bmWeb1=	3.006	in^2	$= tbw * (h_bmWeb - (n_Vbolt - 0.5) * dHole_sp)$
Ant_bmWeb1=	0.494	in^2	$= (Lb_edge - dHole_sp/2) * tbw$
Rn_bmWebVert1=	149.326	kips	$= 0.6 * Fu_bmWeb * Anv_bmWeb1 + Ubs * Fu_bmWeb * Ant_bmWeb1$
Rn_bmWebVert2=	151.897	kips	$= 0.6 * Fy_bmWeb * Agv_bmWeb1 + Ubs * Fu_bmWeb * Ant_bmWeb1$
φRn_bmWebVert=	111.99	kips	$= \phi blockshear * \min(Rn_bmWebVert1, Rn_bmWebVert2)$
DCR_bmWebVert_BS=	0.318		$= Vu_bm / \phi Rn_bmWebVert$
Check=	OK		$= OK \text{ if } DCR_bmWebVert_BS \leq DCR_bmWebVert_BS_allowed$

SHEAR PLATE:

CASE 1:			
hsp=	7.500	in	value previously defined
Agv_sp1=	2.813	in^2	$= hsp * tsp * n_sp$
Anv_sp1=	1.688	in^2	$= tsp * (hsp - (n_Vbolt) * dHole_sp) * n_sp$
Ant_sp1=	0	in	$= 0$
Rn_spVert1=	65.81	kips	$= 0.6 * Fu_sp * Anv_sp1 + Ubs * Fu_sp * Ant_sp1$
Rn_spVert2=	84.38	kips	$= 0.6 * Fy_sp * Agv_sp1 + Ubs * Fu_sp * Ant_sp1$
φRn_spVert=	49.36	kips	$= \phi blockshear * \min(Rn_spVert1, Rn_spVert2)$
DCR_spVert_BS1=	0.721		$= Vu_bm / \phi Rn_spVert$
CASE 2:			
Lv_sp=	1.125	in	value previously defined
hsp=	7.500	in	value previously defined
Lslot=	1.375	in	value previously defined
Agv_sp1=	2.203	in^2	$= (hsp - Lv_sp - (0.5 * dHole_sp)) * tsp * n_sp$
Anv_sp1=	1.875	in^2	$= tsp * (hsp - (n_Vbolt - 0.5) * dHole_sp) * n_sp$
Ant_sp1=	1.430	in^2	$= (W_sp - a - (0.5 * LslotH)) * tsp * n_sp$
Rn_spVert1=	166.055	kips	$= 0.6 * Fu_sp * Anv_sp1 + Ubs * Fu_sp * Ant_sp1$
Rn_spVert2=	159.023	kips	$= 0.6 * Fy_sp * Agv_sp1 + Ubs * Fu_sp * Ant_sp1$
φRn_spVert=	119.27	kips	$= \phi blockshear * \min(Rn_spVert1, Rn_spVert2)$
DCR_spVert_BS2=	0.298		$= Vu_bm / \phi Rn_spVert$
DCR_spVert_BS=	0.721		$= \max(DCR_spVert_BS1, DCR_spVert_BS2)$
Check=	OK		$= OK \text{ if } DCR_spVert_BS \leq DCR_spVert_BS_allowed$

CASE 3: COMBINED AXIAL AND VERTICAL REACTIONS



BEAM WEB:

Method 1: T and V Circular Interaction

$$DCR_{bmWeb\theta_BS1} = 0.116$$

$$= DCR_{bmWebHorz_BS^2} + DCR_{bmWebVert_BS^2}$$

Method 2: Failure along Hole Edges

Pr_BS=	36.91	kips
θ =	1.179	radians
Lb_edge=	1.750	in
Lbw_diag=	10.05	in
Lvg_web=	10.05	in
Lvn_web=	7.05	in
Ltg_web=	2.63	in
Ltn_web=	1.63	in
Agv_web=	3.97	in ²
Anv_web=	2.79	in ²
Agt_web=	1.04	in ²
Ant_web=	0.64	in ²
Rn_bmWeb_BS1=	150.35	kips
Rn_bmWeb_BS2=	160.83	kips
ϕRn_{bmWeb_BS}	112.77	kips
DCR_bmWeb θ_BS2 =	0.327	

$$= \sqrt{((P_u_{sp})^2 + (V_u_{bm})^2)}$$

value previously defined

value previously defined

$$\min(\sqrt{(S_{vert}^2 + S_{horz}^2)} + Lb_edge, (Lb_edge + S_{horz})/\cos \theta)$$

$$= 2 * Lb_edge + (n_Hbolt_SST - 1) * S_{horz} + \sqrt{(S_{vert}^2 + ((n_Hbolt_SST - 1) * S_{horz})^2)}$$

$$= Lvg_web - 2 * dHole_sp - (n_Hbolt_SST - 1) * dHole_sp$$

$$= (n_Vbolt - 2) * S_{vert}$$

$$= Ltg_web - (n_Vbolt - 2) * dHole_sp$$

$$= Lvg_web * tbw$$

$$= Lvn_web * tbw$$

$$= Ltg_web * tbw$$

$$= Ltn_web * tbw$$

$$= 0.6 * F_u_{bmWeb} * Agv_web + Ubs * F_u_{bmWeb} * Ant_web$$

$$= 0.6 * F_y_{bmWeb} * Agv_web + Ubs * F_u_{bmWeb} * Ant_web$$

$$= \phi_{blockshear} \min(Rn_{bmWeb_BS1}, Rn_{bmWeb_BS2})$$

$$= Pr_BS / \phi Rn_{bmWeb_BS}$$

$$DCR_{bmWeb\theta_BS} = 0.327$$

$$= \max(DCR_{bmWeb\theta_BS1}, DCR_{bmWeb\theta_BS2})$$

Check= OK

= OK if $DCR_{bmWeb\theta_BS} \leq DCR_{bmWeb\theta_BS_allowed}$

SHEAR PLATE:

Method 1: T and V Circular Interaction

DCR_spθ_BS1 = 0.536

$$= DCR_spVert_BS^2 + DCR_spHorz_BS^2$$

Method 2: Failure along Hole Edges

CASE 1:

Lvg_sp1=	7.50	in	= h_sp
Lvn_sp1=	4.50	in	= Lvg_sp1 - (n_Vbolts*dHole_sp)
Ltn_sp1=	0.00	in	= 0
Agv_sp1=	2.813	in	= Lvg_sp* tsp *n_sp
Anv_sp1=	1.688	in	= Lvn_sp* tsp *n_sp
Ant_sp1=	0.00	in	= Ltn_sp* tsp *n_sp
Rn_sp_BS1=	65.81	kips	= 0.6* Fu_sp* Anv_sp1 + Ubs* Fu_sp* Ant_sp1
Rn_sp_BS2=	84.38	kips	= 0.6* Fy_sp* Agv_sp1 + Ubs* Fu_sp* Ant_sp1
φRn_sp_BS1=	49.36	kips	= φblockshear* min (Rn_sp_BS1, Rn_sp_BS2)
DCR_spθ_BS2=	0.748		= Pr_BS / φRn_sp_BS1

CASE 2:

Pv=	23.7		= ceiling(n_vBolts/2)/n_vBolts*Vu_bm
Pr_BS1=	25.7		= SQRT(Pv^2 + Pu_sp^2)
Lvg_sp2=	3.75	in	= h_sp/2
Lvn_sp2=	2.25	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=	1.406	in	= Lvg_sp* tsp *n_sp
Anv_sp2=	0.844	in	= Lvn_sp* tsp *n_sp
Ant_sp2=	1.125	in	= Ltn_sp* tsp *n_sp
Rn_sp_BS3=	106.03	kips	= 0.6* Fu_sp* Anv_sp2 + Ubs* Fu_sp* Ant_sp2
Rn_sp_BS4=	115.31	kips	= 0.6* Fy_sp* Agv_sp2 + Ubs* Fu_sp* Ant_sp2
φRn_sp_BS2=	79.52	kips	= φblockshear* min (Rn_sp_BS3, Rn_sp_BS4)
DCR_spθ_BS3=	0.323		= 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=	2.25	in	= Lvg_sp2 - (((n_Vbolt-1)/2)*dHole_sp) - (0.5*dHole_sp)
Agv_sp3=	1.688	in	= Lvg_sp* tsp *n_sp
Anv_sp3=	1.125	in	= Lvn_sp* tsp *n_sp
Ant_sp3=	0.844	in	= Ltn_sp* tsp *n_sp
Rn_sp_BS5=	98.72	kips	= 0.6* Fu_sp* Anv_sp3 + Ubs* Fu_sp* Ant_sp3
Rn_sp_BS6=	105.47	kips	= 0.6* Fy_sp* Agv_sp3 + Ubs* Fu_sp* Ant_sp3
φRn_sp_BS3=	74.04	kips	= φblockshear* min (Rn_sp_BS5, Rn_sp_BS6)
DCR_spθ_BS4=	0.347		= Pr_BS1 / φRn_sp_BS3

DCR_spθ_BS=	0.748		= 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	-	-	-	-	0.293	0.327	0.327
Shear Plate	-	0.422	0.721	0.865	0.446	0.748	0.865
Shear Plate Bolt	0.527	-	-	-	-	-	0.527
SP Fillet Weld	0.456	-	-	-	-	-	0.456

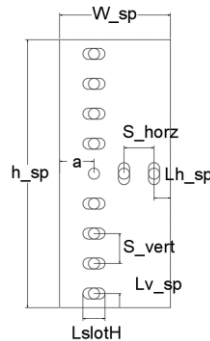
4.1 CURRENT MEMBER:

Beam Unique Name:	2001	Shear Plate Info:		
Beam Size:	W16X67	Thickness (tsp) (in):	0.38	in
Axial Pu (kips):	22.82	Bolt Dia (db_sp):	0.875	in
Shear V_Grav. SST_LC08 (kips):	4.46	No. of SP (n_sp):	1	
Shear Vu_Grav. SST_LC01-07(kips):	5.59	Bolt Type:	A325-N	
Link:	YLMC6-3.5	n_Vbolts:	3	
tstem (in):	0.75	n_Hbolts:	2	
		R=	3.5	
		Fy_bm=	50	ksi
		Fu_bm=	65	ksi
		Left Col=	W24X84	
		Right Col=	W24X84	
		Left_Conn=	YES	
		Right_Conn=	YES	
		no. of Conn =	2	

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

Beam Size=	W16X67	User Input Beam size	
db=	16.30	in	From AISC Database
tbf=	0.67	in	From AISC Database
tbfw=	0.40	in	From AISC Database
Fy_sp=	50	ksi	User Input/Selection
Fu_sp=	65	ksi	User Input/Selection
Axial Load (Pu_sp)=	22.82	kips	Maximum Beam Axial Force from Overstrength Load Combinations
Lcc=	251	in	Column-to-column Centerline dimension. User Input value
Lh=	220.00	in	=Lcc - dc - 2*a
Pcap_link=	204.75	kips	=If (R=3, Py_link, Pr_link)
Mcap=	3491	kip-in	=Pcap_link*(db + tstem)
Vcap_link=	31.74	kips	=Mcap*(No. of Connections) / Lh
Vertical Load (Vu_bm)=	36.20	kips	= Vcap_link + Vbm_gravity (EQ 12.9-34)
No. of vertical Bolts (n_Vbolts)=	3.000		Looked up value
No. of horizontal Bolts (n_Hbolts)=	2.000		User Input
Vu_bolt=	16.61	kips	= Max(Sqrt ((Pu_sp/ n_Hbolt)^2 + (Vu_bm/ n_Vbolt)^2), (EQ 12.9-35)
φbolt=	0.75		Vu_grav.SSTLC01-07/n_Vbolts)
Bolt Type (Bolt_Gr_shearTab)=	A325-N		
Bolt Dia (db_sp)=	7/8	in	
Frv_A325N=	54.00	ksi	
Anb_sp=	0.601	in ²	= Pi*(db_sp/2)^2
Rn_stBolt_shear=	32.47	kips	= Frv_A325N* Anb_sb
φRn_stBolt_Shear=	24.35	kips	= Rn_stBolt_shear* φbolt * n_sp
DCR_shearTab_Bolt=	0.682		= 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=	0.375	in	[default to try and match beam web thickness]
db=	16.30	in	Value previously defined
Bolt Spacing (S_min)=	2.000	in	= 2.0*d_b_sp-hole (clear distance not less than d_b_sp-hole), see section J3.3
S_max=	2.650	in	
Bolt Spacing (Svert)=	2.625	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=	7.55	in	= db + tstem - h_flange - (2*h_clear)
Plate Depth (h_sp)=	7.50	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=	1.368	in	= db_sp + 1/8 + (2*0.07* Svert*(n_Vbolts - 1)/2)
LslotH = LslotV=	1.375	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)

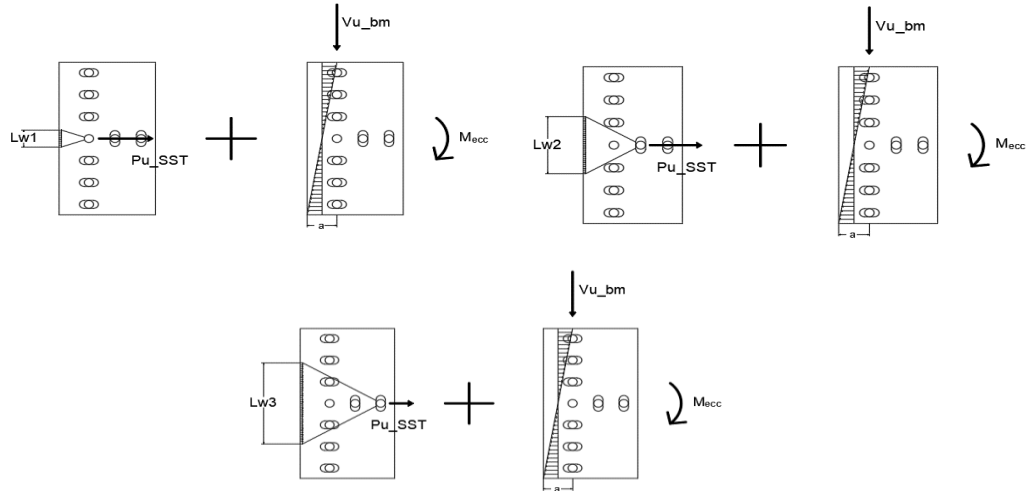
4.4 SHEAR PLATE YIELDING (VERTICAL): (AISC 358 Chapter 12, Step 15.3)

$\phi_{yield} =$	1		$= db_sp + 1/8$
$dHole_sp =$	1.00	in	$= tsp * h_sp$
$Asp_Agv =$	2.813	in ²	$= \phi_{yield} * 0.6 * Asp_Agv * Fy_sp * n_sp$
$\phi Vy_sp =$	84.38	kips	$= Vu_bm / \phi Vy_sp$
$DCR_spYield =$	0.429		$OK \text{ if } DCR_spYield \leq DCR_spYield_allowed$
Check=	OK		

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

$\phi_{rupture} =$	0.75		$= hsp * tsp - dHole_sp * n_Vbolts * tsp$
$Asp_nv =$	1.69	in ²	$= \phi_{rupture} * 0.6 * Fu_shearTab * Asp_nv * n_sp$
$\phi V_{rupture_sp} =$	49.36	kips	$= Vu_bm / \phi V_{rupture_sp}$
$DCR_spRupture =$	0.733		$OK \text{ if } DCR_spRupture \leq DCR_spRupture_allowed$
Check=	OK		

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



$\phi b =$	0.90		Value previously defined
$a =$	3.50	in	$= Vu_bm$
$Vuy =$	36.20	kips	$= Vuy * a$
$Mecc =$	126.70	kip-in	
$\theta_{whitmore} =$	30	deg	
$L_{whitmore1} =$	4.916	in	$= \tan(\theta_{whitmore}) * a^2 + db_sp$
$A_{whitmore1} =$	1.844	in ²	$= L_{whitmore1} * tsp * n_sp$
$L_{whitmore2} =$	7.500	in	$= \min(\tan(\theta_{whitmore}) * (a + Shorz)^2 + db_sp, hsp)$
$A_{whitmore2} =$	2.813	in ²	$= L_{whitmore2} * tsp * n_sp$
$L_{whitmore3} =$	7.500	in	$= \min(\tan(\theta_{whitmore}) * (a + 2 * Shorz)^2 + db_sp, hsp)$
$A_{whitmore3} =$	2.813	in ²	$= L_{whitmore3} * tsp * n_sp$
$L_{whitmore4} =$	7.500	in	$= \min(\tan(\theta_{whitmore}) * (a + 3 * Shorz)^2 + db_sp, hsp)$
$A_{whitmore4} =$	2.813	in ²	$= L_{whitmore4} * tsp * n_sp$
$Ssp =$	3.516	in ³	$= (tsp * hsp^2 * n_sp) / 6$
$Isp =$	13.184	in ⁴	$= (tsp * hsp^3 * n_sp) / 12$
$fb1 =$	36.039	ksi	$= Mecc / Ssp$
$A_{whitmore} =$	2.813	in ²	$= IF(n_Hbolts=1, A_{whitmore1}, IF(n_Hbolts=2, A_{whitmore2}, IF(n_Hbolts=3, A_{whitmore3}, A_{whitmore4})))$
$L_{whitmore} =$	7.500	in	$= IF(n_Hbolts=1, L_{whitmore1}, IF(n_Hbolts=2, L_{whitmore2}, IF(n_Hbolts=3, L_{whitmore3}, L_{whitmore4})))$
$y_b =$	3.750	in	$= L_{whitmore} / 2$
$fb2 =$	36.039	ksi	$= Mecc * y_b / Isp$
$fa2 =$	8.112	ksi	$= Pu_sp / A_{whitmore}$
$ftot2 =$	44.151	ksi	$= fa2 + fb2$
$fmax_sp =$	44.151	ksi	$= \max(fb1, ftot2)$
$\phi b * Fy_sp =$	45.000	ksi	$= \phi b * Fy_sp$
$DCR_sp =$	0.981		$= fmax_sp / (\phi b * Fy_sp)$
Check=	OK		$= OK \text{ if } DCR_sp \leq DCR_sp_allowed$

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

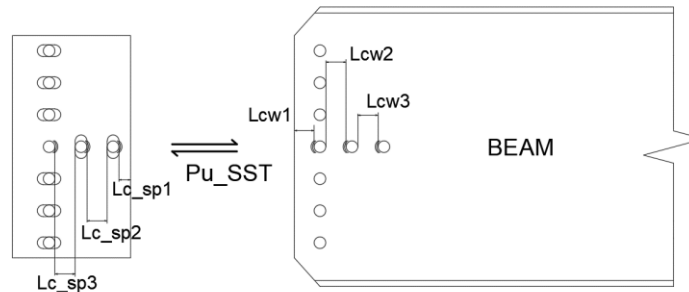
ϕ =	0.75		Value previously defined
tsp=	0.375	in	Value previously defined
tw_sp_min=	0.234	in	= 5/8*tsp (Fillet size required to develop plate capacity as per AISC Steel Manual 14th edition)
tw_sp=	0.250	in	= CEILING(tw_sp_min, 1/16)
Check=	OK		= OK if DCR_tw_spWeld <= DCR_tw_spWeld_allowed
Vu_bm=	36.199	kips	Value previously defined
Lweld=	7.000	in	= Length per side assuming 1/4" hold off top and bottom
F _{EXX} =	70.00	ksi	= Filler metal classification strength
F _{nw} =	42.000	ksi	= 0.6*F _{EXX}
A _{we} =	2.475	in ²	= 0.707*(2*Lweld)*tw_sp
ϕ Rn=	77.947	kips	= ϕ *F _{nw} *A _{we}
DCR_spWeld=	0.464		= Vu_bm / ϕ Rn
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=	0.375	in	Same thickness for both shear plates
ϕ PJP=	0.75		
nsides=	1		
t _{eff_PJP} =	0.313	in	= 3/4*tsp
Vu_PJP=	18.100	kips	= Vu_bm / 2
A _w =	2.188	in ²	= t _{eff_PJP} * Lweld * nsides
ϕ Rn=	68.906	kips	= ϕ PJP * 0.6 * F _{EXX} * A _w
DCR_PJPWeld1=	0.263		= Vu_PJP / ϕ Rn
PJP weld P+M Check:			
ϕ =	0.8		
A _{PJP} =	2.344	in	= t _{eff_PJP} * h _{sp}
I _{PJP} =	10.986	in ⁴	= t _{eff_PJP} * h _{sp} ³ / 12
f _{PJP} =	26.490	ksi	= 0.5 * Pu _{SST} / A _{PJP} + 0.5 * Mecc * yb / I _{PJP}
f _{PJP_allowed} =	33.600	ksi	= ϕ *0.6*F _{EXX}
DCR_PJPWeld2=	0.788		= Vu_PJP / ϕ Rn
DCR_PJPWeld=	0.788		= 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



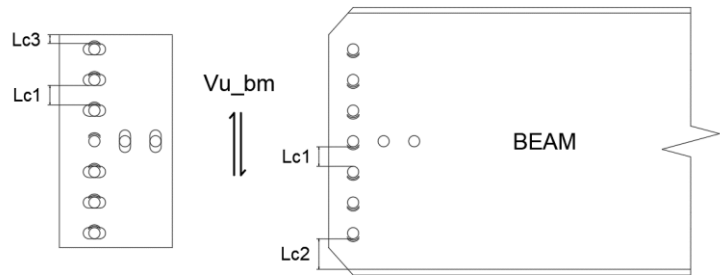
BEAM WEB:

ϕ bolt=	0.75		Value previously defined
t _{bw} =	0.40	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 _{Hbolt_SST} = 2, 0, Lcw2)
Lcw4=	0.000	in	= if (n _{Hbolt_SST} = 3, 0, Lcw3)
Lc _{bmWeb} =	3.094	in	= Lcw1 + Lcw2 + Lcw3 + Lcw4
ϕ Rn _{beamWeb1} =	71.49	kips	= ϕ bolt* 1.2* Lc _{bmWeb} * t _{bw} * Fu _{bm}
ϕ Rn _{beamWeb2} =	80.88	kips	= fbolt* 2.4* db _{sp} * t _{bw} * Fu _{bm} * n _{Hbolts}
ϕ Rn _{beamWeb} =	71.49	kips	= min (ϕ Rn _{beamWeb1} , ϕ Rn _{beamWeb2})
DCR _{bmWebX} =	0.319		= Pu _{sp} / ϕ Rn _{beamWeb}
Check=	OK		= OK if DCR _{bmWebX} <= DCR _{bmWebX_allowed}

SHEAR PLATE:

tsp=	3/8	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}
ϕ Rn _{sp1} =	63.18	kips	= IF(n _{Hbolt_SST} =1, ϕ bolt*(1.2*Lc _{sp1} *tsp*Fu _{sp} *n _{sp}), ϕ bolt*(1.2*Lc _{sp2} +1.0*(Lc _{sp1} +Lc _{sp3} +Lc _{sp4}))*tsp*Fu _{sp} *n _{sp})
ϕ Rn _{sp2} =	70.38	kips	= ϕ bolt*(2.4*1+2.0*(n _{Hbolts} -1))*db _{sp} *tsp*Fu _{sp} *n _{sp}
ϕ Rn _{sp} =	63.18	kips	= min (ϕ Rn _{sp1} , ϕ Rn _{sp2})
DCR _{spX} =	0.361		= Pu _{sp} / ϕ Rn _{sp}
Check=	OK		= OK if DCR _{spX} <= DCR _{spX_allowed}

CASE 2: VERTICAL REACTIONS



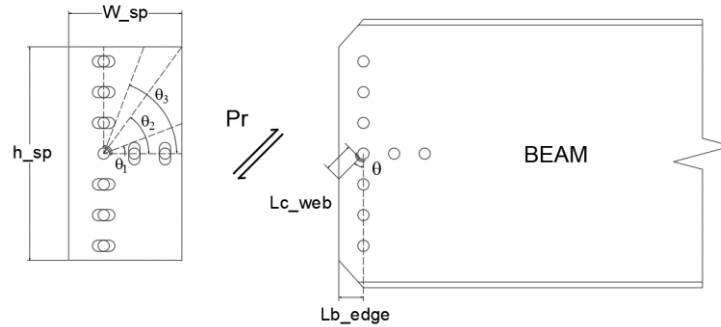
BEAM WEB:

ϕ bolt=	0.75		Value previously defined
Lc1=	1.688	in	= Svert - (db_sp + 1/16)
Lc2=	4.860	in	= (db - (n_Vbolts - 1))*Svert - 2tbw*0.5
Lcb_Vert=	8.235	in	= Lc2 + (n_Vbolts - 1)*Lc1
ϕ *Rn_bmWebY1=	190.29	kips	= ϕ bolt* 1.2* Lcb_Vert* tbw* Fu_bm
ϕ *Rn_bmWebY2=	121.31	kips	= ϕ bolt* 2.4* (db_sp* n_bolts_SST)* tbw* Fu_bm
ϕ *Rn_bmWebY=	121.31	kips	= min (ϕ *Rn_bmWebY1, ϕ *Rn_bmWebY2)
DCR_bmWebY=	0.298		= 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=	4.031	in	= Lc3 + (n_Vbolts - 1)*Lc1
ϕ *Rn_spY1=	79.87	kips	= ϕ bolt* (1.2*Lc1 + 1.0*(Lc3 + (n_Vbolts - 2))*Lc1)*tsp* Fu_sp *n_sp
ϕ *Rn_spY2=	102.38	kips	= ϕ bolt* (2.4*1+2.0*(n_bolts_SST-1))* db_sp* tsp* Fu_sp *n_sp
ϕ *Rn_spY=	79.87	kips	= min (ϕ *Rn_spY1, ϕ *Rn_spY2)
DCR_spY=	0.453		= Vu_bm / (ϕ * Rn_spY)
Check=	OK		= OK if DCR_spY <= DCR_spY_allowed

CASE 3: COMBINED AXIAL AND VERTICAL REACTIONS



Px=	11.41	kips	= Pu_sp/ n_Hbolts
Py=	12.07	kips	= Vu_bm/ n_Vbolts
Resultant (Pr)=	16.61	kips	= Sqrt (Px^2 + Py^2)
theta=	0.813	radians	= min(Atan (Py/ Px), 1.571))

BEAM WEB:

Method 1: T and V Circular Interaction

$$DCR_{bmWeb\theta Bearing1} = 0.191 = DCR_{bmWebX}^2 + DCR_{bmWebY}^2$$

Method 2: Bearing in a Diagonal Line

Lvg_web =	2.547	in	= Lb_edge/ (cos theta)
Lc_web =	2.047	in	= Lvg_web - dHole_sp/2
phi*Rn_bmWeb01=	47.309	kips	= phi*bolt* 1.2* Lc_web* tbw* Fu_bm
phi*Rn_bmWeb02=	80.876	kips	= fbolt* 2.4* db_sp* tbw* Fu_bm* n_Hbolts
phi*Rn_bmWeb0=	47.31	kips	= min (phi* Rn_bmWeb 01, phi* Rn_bmWeb 02)
DCR_kipWeb0Bearing2 =	0.351		= Pr / phi*Rn_bmWeb 0
DCR_bmWeb0Bearing =	0.351		= Min (DCR_bmWeb 0Bearing1, DCR_bmWeb 0Bearing2)
	OK		= OK if DCR_bmWeb 0 <= DCR_bmWeb 0_allowed

SHEAR PLATE:

Method 1: T and V Circular Interaction

$$DCR_{sp0Bearing1} = 0.336 = DCR_{spX}^2 + DCR_{spY}^2$$

Method 2: Bearing in a Diagonal Line

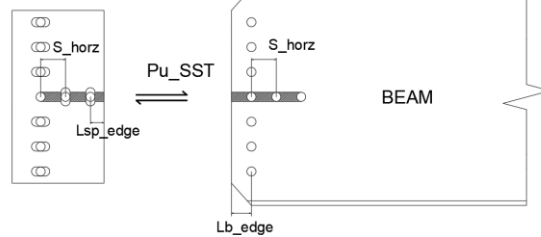
01=	0.2450	radians	= arctan((0.5*LslotH)/Shorz)
02=	0.6947	radians	= arctan((0.5*h_sp)/(w_sp-a))
0'=	0.2562	radians	= arctan((0.5*LslotV)/Svert)
03=	1.3146	radians	= (Pi()*0.5)- 0'
0=	0.8135	radians	Value previously defined

	CASE 1	CASE 2	CASE 3	CASE 4	
min. 0=	0	01	02	03	radians
max 0=	01	02	03	1.571	radians
Check=	NO	NO	YES	NO	
La_sp1=	-	-	5.162	-	in
La_sp=	5.162				in
phi*Rn_sp01=	113.232	kips			= phi*bolt* 1.2* La_sp* tsp* Fu_sp * n_sp
phi*Rn_sp02=	76.781	kips			= fbolt* 2.4* db_sp* tsp* Fu_sp* n_Hbolts * n_sp
phi*Rn_sp0=	76.78	kips			= min (phi* Rn_sp 01, phi* Rn_sp 02)
DCR_sp0bearing2=	0.216				= Pr / (phi* Rn_sp 0)
DCR_sp0bearing=	0.336				= max(DCR_sp 0Bearing1, DCR_sp 0bearing2)
Check=	OK				OK if DCR_sp 0bearing <= DCR_sp 0bearing_allowed

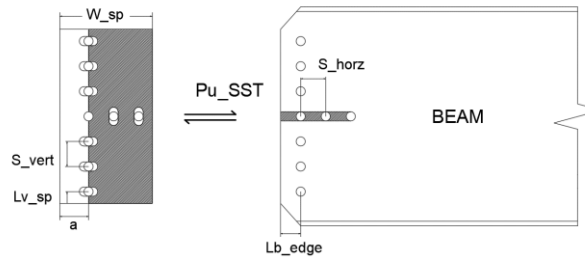
4.9 BEAM WEB AND SHEAR TAB BLOCKSHEAR CHECK: (AISC 358 Chapter 12, Step 15.5)

CASE 1: HORIZONTAL REACTIONS

CASE 1



CASE 2



BEAM WEB:

ϕ blockshear=	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=	3.555	in ²	$= Lh_bmWeb * tbw$
Ant_bmWebHorz=	0.000	in ²	$= 0$
Anv_bmWebHorz=	3.555	in ²	$= Agv_bmWebHorz$
Rn_bmWebHorz1=	138.65	kips	$= 0.6 * Fu_bmWeb * Anv_bmWebHorz + Ubs * Fu_bmWeb * Ant_bmWebHorz$
Rn_bmWebHorz2=	106.65	kips	$= 0.6 * Fy_bmWeb * Agv_bmWebHorz + Ubs * Fu_bmWeb * Ant_bmWebHorz$
Rn_bmWebHorz=	79.99	kips	$= \phi blockshear * MIN (Rn_bmWebHorz1, Rn_bmWebHorz2)$
DCR_bmWebHorz_BS=	0.285		$= Pu_sp / Rn_bmWebHorz$
Check=	OK		$= OK \text{ if } DCR_bmWebHorz_BS \leq DCR_bmWebHorz_BS_allowed$

SHEAR PLATE:

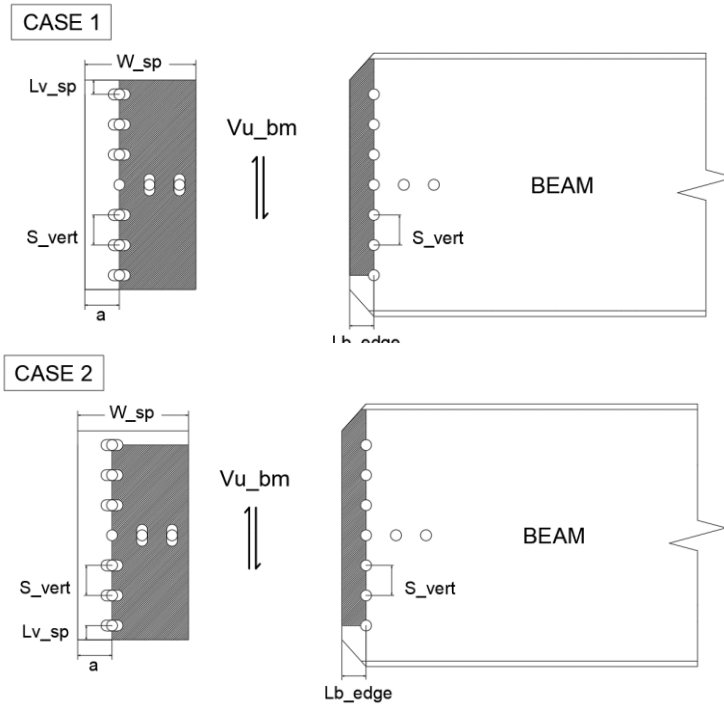
CASE 1:

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=	3.375	in ²	$= Lh_sp1 * tsp * n_sp$
Ant_spHorz1=	0.000	in ²	$= 0$
Anv_spHorz1=	3.375	in ²	$= Agv_spHorz1$
Rn_spHorz1=	131.63	kips	$= 0.6 * Fu_sp * Anv_spHorz1 + Ubs * Fu_sp * Ant_spHorz1$
Rn_spHorz2=	101.25	kips	$= 0.6 * Fy_sp * Agv_spHorz1 + Ubs * Fu_sp * Ant_spHorz1$
$\phi Rn_spHorz1=$	75.94	kips	$= \phi blockshear * min (Rn_spHorz1, Rn_spbHorz2)$
DCR_spHorz_BS1=	0.300		$= Pu_sp / Rn_spHorz1$

CASE 2:

Lh_sp2=	4.500	in	$= h_sp - (n_Vbolts * dHole_sp)$
Agv_spHorz2=	0.000	in ²	$= 0$
Ant_spHorz2=	1.688	in ³	$= Lh_sp2 * tsp * n_sp$
Anv_spHorz2=	0.000	in ²	$= 0$
Rn_spHorz3=	109.69	kips	$= 0.6 * Fu_sp * Anv_spHorz2 + Ubs * Fu_sp * Ant_spHorz2$
Rn_spHorz4=	109.69	kips	$= 0.6 * Fy_sp * Agv_spHorz2 + Ubs * Fu_sp * Ant_spHorz2$
$\phi Rn_spHorz2=$	82.27	kips	$= \phi blockshear * min (Rn_spHorz3, Rn_spbHorz4)$
DCR_spHorz_BS2=	0.277		$= Pu_sp / Rn_spHorz2$
DCR_spHorz_BS=	0.300		$= MAX (DCR_spHorz_BS1, DCR_spHorz_BS2)$
Check=	OK		$= OK \text{ if } DCR_spHorz_BS \leq DCR_spHorz_BS_allowed$

CASE 2: VERTICAL REACTIONS



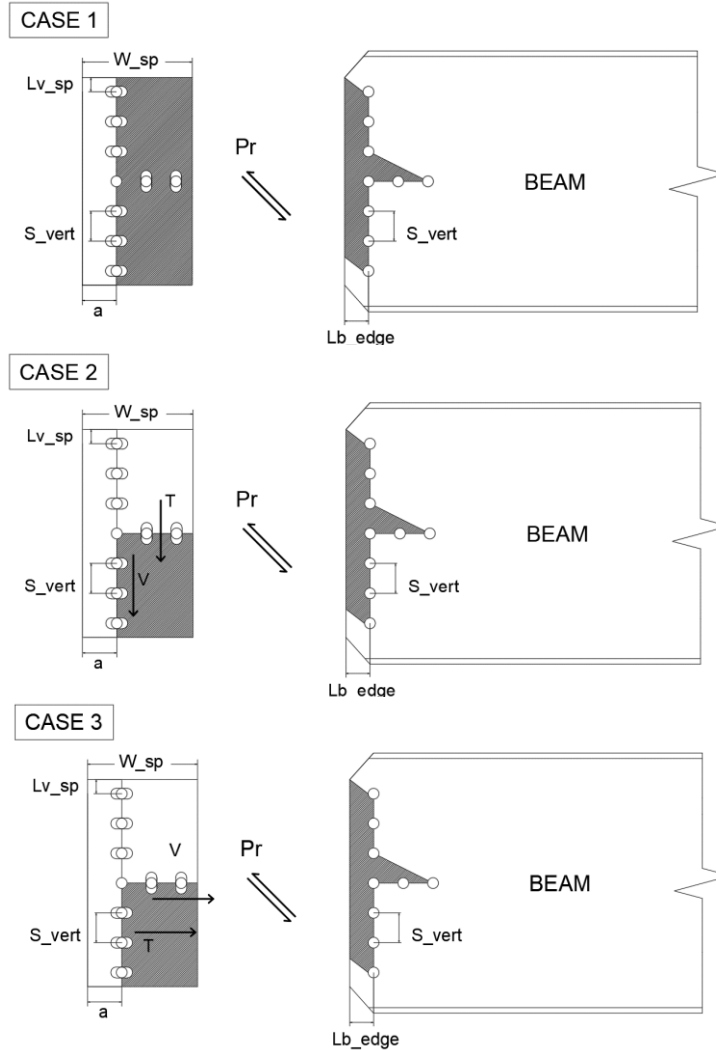
BEAM WEB:

S_vert=	2.63	in	value previously defined
Lb_edge=	1.75	in	value previously defined
h_bmWeb=	10.110	in	$= (d - (n_Vbolt - 1) * Svert - 2 * tb) * 0.5 + 2 * (n_Vbolt - 1) * Svert$
Agv_bmWeb1=	3.993	in^2	$= tbw * h_bmWeb$
Anv_bmWeb1=	3.006	in^2	$= tbw * (h_bmWeb - (n_Vbolt - 0.5) * dHole_sp)$
Ant_bmWeb1=	0.494	in^2	$= (Lb_edge - dHole_sp/2) * tbw$
Rn_bmWebVert1=	149.326	kips	$= 0.6 * Fu_bmWeb * Anv_bmWeb1 + Ubs * Fu_bmWeb * Ant_bmWeb1$
Rn_bmWebVert2=	151.897	kips	$= 0.6 * Fy_bmWeb * Agv_bmWeb1 + Ubs * Fu_bmWeb * Ant_bmWeb1$
φRn_bmWebVert=	111.99	kips	$= \phi blockshear * \min(Rn_bmWebVert1, Rn_bmWebVert2)$
DCR_bmWebVert_BS=	0.323		$= Vu_bm / \phi Rn_bmWebVert$
Check=	OK		$= OK \text{ if } DCR_bmWebVert_BS \leq DCR_bmWebVert_BS_allowed$

SHEAR PLATE:

CASE 1:			
hsp=	7.500	in	value previously defined
Agv_sp1=	2.813	in^2	$= hsp * tsp * n_sp$
Anv_sp1=	1.688	in^2	$= tsp * (hsp - (n_Vbolt) * dHole_sp) * n_sp$
Ant_sp1=	0	in	$= 0$
Rn_spVert1=	65.81	kips	$= 0.6 * Fu_sp * Anv_sp1 + Ubs * Fu_sp * Ant_sp1$
Rn_spVert2=	84.38	kips	$= 0.6 * Fy_sp * Agv_sp1 + Ubs * Fu_sp * Ant_sp1$
φRn_spVert=	49.36	kips	$= \phi blockshear * \min(Rn_spVert1, Rn_spVert2)$
DCR_spVert_BS1=	0.733		$= Vu_bm / \phi Rn_spVert$
CASE 2:			
Lv_sp=	1.125	in	value previously defined
hsp=	7.500	in	value previously defined
Lslot=	1.375	in	value previously defined
Agv_sp1=	2.203	in^2	$= (hsp - Lv_sp - (0.5 * dHole_sp)) * tsp * n_sp$
Anv_sp1=	1.875	in^2	$= tsp * (hsp - (n_Vbolt - 0.5) * dHole_sp) * n_sp$
Ant_sp1=	1.430	in^2	$= (W_sp - a - (0.5 * Lslot)) * tsp * n_sp$
Rn_spVert1=	166.055	kips	$= 0.6 * Fu_sp * Anv_sp1 + Ubs * Fu_sp * Ant_sp1$
Rn_spVert2=	159.023	kips	$= 0.6 * Fy_sp * Agv_sp1 + Ubs * Fu_sp * Ant_sp1$
φRn_spVert=	119.27	kips	$= \phi blockshear * \min(Rn_spVert1, Rn_spVert2)$
DCR_spVert_BS2=	0.304		$= Vu_bm / \phi Rn_spVert$
DCR_spVert_BS=	0.733		$= \max(DCR_spVert_BS1, DCR_spVert_BS2)$
Check=	OK		$= OK \text{ if } DCR_spVert_BS \leq DCR_spVert_BS_allowed$

CASE 3: COMBINED AXIAL AND VERTICAL REACTIONS



BEAM WEB:

Method 1: T and V Circular Interaction
DCR_bmWeb θ _BS1 = 0.186

$$= DCR_bmWebHorz_BS^2 + DCR_bmWebVert_BS^2$$

Method 2: Failure along Hole Edges

Pr_BS = 42.79 kips
 θ = 0.813 radians
 Lb_edge = 1.750 in
 Lbw_diag = 10.05 in
 Lvg_web = 10.05 in
 Lvn_web = 7.05 in
 Ltg_web = 2.63 in
 Ltn_web = 1.63 in
 Agv_web = 3.97 in²
 Anv_web = 2.79 in²
 Agt_web = 1.04 in²
 Ant_web = 0.64 in²
 Rn_bmWeb_BS1 = 150.35 kips
 Rn_bmWeb_BS2 = 160.83 kips
 ϕ Rn_bmWeb_BS = 112.77 kips
 DCR_bmWeb θ _BS2 = 0.379

$$= \sqrt{((P_u_sp)^2 + (V_u_bm)^2)}$$

value previously defined
 value previously defined
 $\min(\sqrt{S_{vert}^2 + S_{horz}^2}) + L_{b_edge}, (L_{b_edge} + S_{horz})/\cos \theta$
 $= 2 * L_{b_edge} + (n_Hbolt_SST - 1) * S_{horz} + \sqrt{S_{vert}^2 + ((n_Hbolt_SST - 1) * S_{horz})^2}$
 $= L_{vg_web} - 2 * d_{Hole_sp} - (n_Hbolt_SST - 1) * d_{Hole_sp}$
 $= (n_Vbolt - 2) * S_{vert}$
 $= L_{tg_web} - (n_Vbolt - 2) * d_{Hole_sp}$
 $= L_{vg_web} * tbw$
 $= L_{vn_web} * tbw$
 $= L_{tg_web} * tbw$
 $= L_{tn_web} * tbw$
 $= 0.6 * F_u_bmWeb * Agv_web + Ubs * F_u_bmWeb * Ant_web$
 $= 0.6 * F_y_bmWeb * Agv_web + Ubs * F_u_bmWeb * Ant_web$
 $= \phi_{blockshear} \min(Rn_bmWeb_BS1, Rn_bmWeb_BS2)$
 $= Pr_BS / \phi Rn_bmWeb_BS$

DCR_bmWeb_BS = 0.379
 Check = OK

$$= \max(DCR_bmWeb \theta_BS1, DCR_bmWeb \theta_BS2)$$

= OK if DCR_bmWeb θ _BS <= DCR_bmWeb θ _BS_allowed

SHEAR PLATE:

Method 1: T and V Circular Interaction

$$DCR_sp\theta_BS1 = 0.628$$

$$= DCR_spVert_BS^2 + DCR_spHorz_BS^2$$

Method 2: Failure along Hole Edges

CASE 1:

Lvg_sp1=	7.50	in	= h_sp
Lvn_sp1=	4.50	in	= Lvg_sp1 - (n_vbolts*dHole_sp)
Ltn_sp1=	0.00	in	= 0
Agv_sp1=	2.813	in	= Lvg_sp*tsp*n_sp
Anv_sp1=	1.688	in	= Lvn_sp*tsp*n_sp
Ant_sp1=	0.00	in	= Ltn_sp*tsp*n_sp
Rn_sp_BS1=	65.81	kips	= 0.6*Fu_sp*Anv_sp1 + Ubs*Fu_sp*Ant_sp1
Rn_sp_BS2=	84.38	kips	= 0.6*Fy_sp*Agv_sp1 + Ubs*Fu_sp*Ant_sp1
φRn_sp_BS1=	49.36	kips	= φblockshear*min(Rn_sp_BS1, Rn_sp_BS2)
DCR_spθ_BS2=	0.867		= Pr_BS / φRn_sp_BS1

CASE 2:

Pv=	24.1		= ceiling(n_vBolts/2)/n_vBolts*Vu_bm
Pr_BS1=	33.2		= SQRT(Pv^2 + Pu_sp^2)
Lvg_sp2=	3.75	in	= h_sp/2
Lvn_sp2=	2.25	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=	1.406	in	= Lvg_sp*tsp*n_sp
Anv_sp2=	0.844	in	= Lvn_sp*tsp*n_sp
Ant_sp2=	1.125	in	= Ltn_sp*tsp*n_sp
Rn_sp_BS3=	106.03	kips	= 0.6*Fu_sp*Anv_sp2 + Ubs*Fu_sp*Ant_sp2
Rn_sp_BS4=	115.31	kips	= 0.6*Fy_sp*Agv_sp2 + Ubs*Fu_sp*Ant_sp2
φRn_sp_BS2=	79.52	kips	= φblockshear*min(Rn_sp_BS3, Rn_sp_BS4)
DCR_spθ_BS3=	0.418		= 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=	2.25	in	= Lvg_sp2 - (((n_vbolt-1)/2)*dHole_sp) - (0.5*dHole_sp)
Agv_sp3=	1.688	in	= Lvg_sp*tsp*n_sp
Anv_sp3=	1.125	in	= Lvn_sp*tsp*n_sp
Ant_sp3=	0.844	in	= Ltn_sp*tsp*n_sp
Rn_sp_BS5=	98.72	kips	= 0.6*Fu_sp*Anv_sp3 + Ubs*Fu_sp*Ant_sp3
Rn_sp_BS6=	105.47	kips	= 0.6*Fy_sp*Agv_sp3 + Ubs*Fu_sp*Ant_sp3
φRn_sp_BS3=	74.04	kips	= φblockshear*min(Rn_sp_BS5, Rn_sp_BS6)
DCR_spθ_BS4=	0.449		= Pr_BS1 / φRn_sp_BS3

DCR_spθ_BS=	0.867		= max(DCR_spθ_BS1, DCR_spθ_BS2, DCR_spθ_BS3, DCR_spθ_BS4)
Check=	OK		= OK, if DCR_spθ_BS <= DCR_spθ_BS_allowed

DCR Summary:

Component	Shear	Yielding	Rupture	P+M	Bearing	BS	Max
Beam Web	-	-	-	-	0.351	0.379	0.379
Shear Plate	-	0.429	0.733	0.981	0.453	0.867	0.981
Shear Plate Bolt	0.682	-	-	-	-	-	0.682
SP Fillet Weld	0.464	-	-	-	-	-	0.464

Design Summary: Base Plate_G 1

Base Plate Design:

Thickness = 1 in
Fy = 36 Ksi

Anchorage Design:

Diameter = 1.25 in
Type = Headed Bolt
Embed = 12 in
Fy = 36 Ksi
Fu = 58 Ksi

Concrete Design:

F'c = 4 Ksi
Cracking = Yes

Steel Specification:

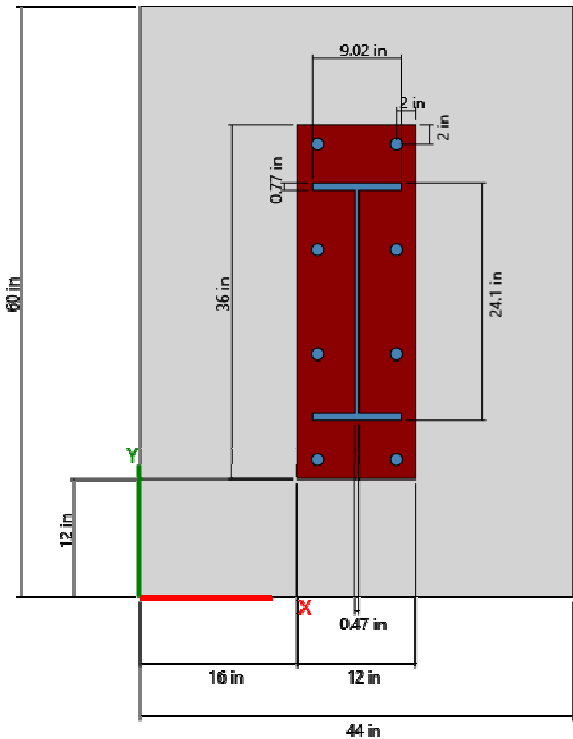
AISC 360-22

Concrete Specification:

ACI 318-19

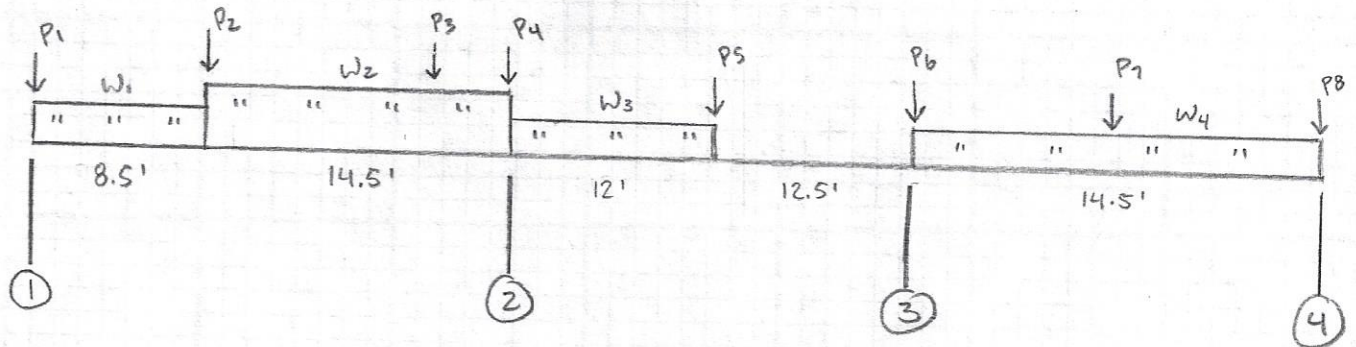
Analysis Method:

AISC Design Guide One



Limit State	Load Key	Demand	Capacity	Unity Value
Concrete Bearing	16	0.17865 Ksi	2.21 Ksi	0.081
Plate Bending	35	0.49325 K-ft	0.675 K-ft	0.731
AnchorageTension	35			
Steel Strength		62.408 K	292.25 K	0.214
Concrete Breakout		62.408 K	>1.0E20	0.000
Concrete Pullout		62.408 K	429.5 K	0.145
Concrete Blowout		N.A	N.A	N.A.
Anchorage Shear	16			
Steel Strength		-30.88 K	32.372 K	0.954
Concrete Breakout		30.88 K	32.434 K	0.952
Concrete Pryout		30.88 K	181.51 K	0.170
Anchorage Interaction	16	-	-	0.954
Anchorage Detailing	-	-	-	OK
Plate Detailing	-	-	-	OK

ZIMMER AUTOCOURT RETAINING WALL - GRID D



$$L_{TOTAL} = 61'-0"$$

LOADS

$$W_{1D} = (24/12)(15) = 30 \text{ pcf}$$

$$W_{1L} = (24/12)(40) = 80 \text{ pcf}$$

$$W_{2D} = (19/2)(32) + (24/12)(15) + 100 = 434 \text{ pcf}$$

$$W_{2S} = (19/2)(25) = 238 \text{ pcf}$$

$$W_{2L} = (24/12)(40) = 80 \text{ pcf}$$

$$W_{3D} = (12/12)(15) = 15 \text{ pcf}$$

$$W_{3L} = (12/12)(40) = 40 \text{ pcf}$$

$$W_{4D} = (16/12)(20) + (24/12)(15) + 100 = 157 \text{ pcf}$$

$$W_{4S} = (16/12)(25) = 33 \text{ pcf}$$

$$W_{4L} = (24/12)(40) = 80 \text{ pcf}$$

$$P_{1D} = 1140 \text{ \#}$$

$$P_{1S} = 890 \text{ \#}$$

$$P_{2D} = 1140 \text{ \#}$$

$$P_{2S} = 890 \text{ \#}$$

$$P_{2W} = 477 \text{ \#}$$

$$P_{2E} = 1005 \text{ \#}$$

$$P_{3D} = 750 \text{ \#}$$

$$P_{3S} = 580 \text{ \#}$$

$$P_{4D} = 1825 \text{ \#}$$

$$P_{4S} = 1425 \text{ \#}$$

$$P_{5D} = 3650 \text{ \#}$$

$$P_{5S} = 2850 \text{ \#}$$

$$P_{6D} = 100 \text{ \#}$$

$$P_{6S} = 75 \text{ \#}$$

$$P_{6W} = 2087 \text{ \#}$$

$$P_{6E} = 3422 \text{ \#}$$

$$P_{7D} = 1085 \text{ \#}$$

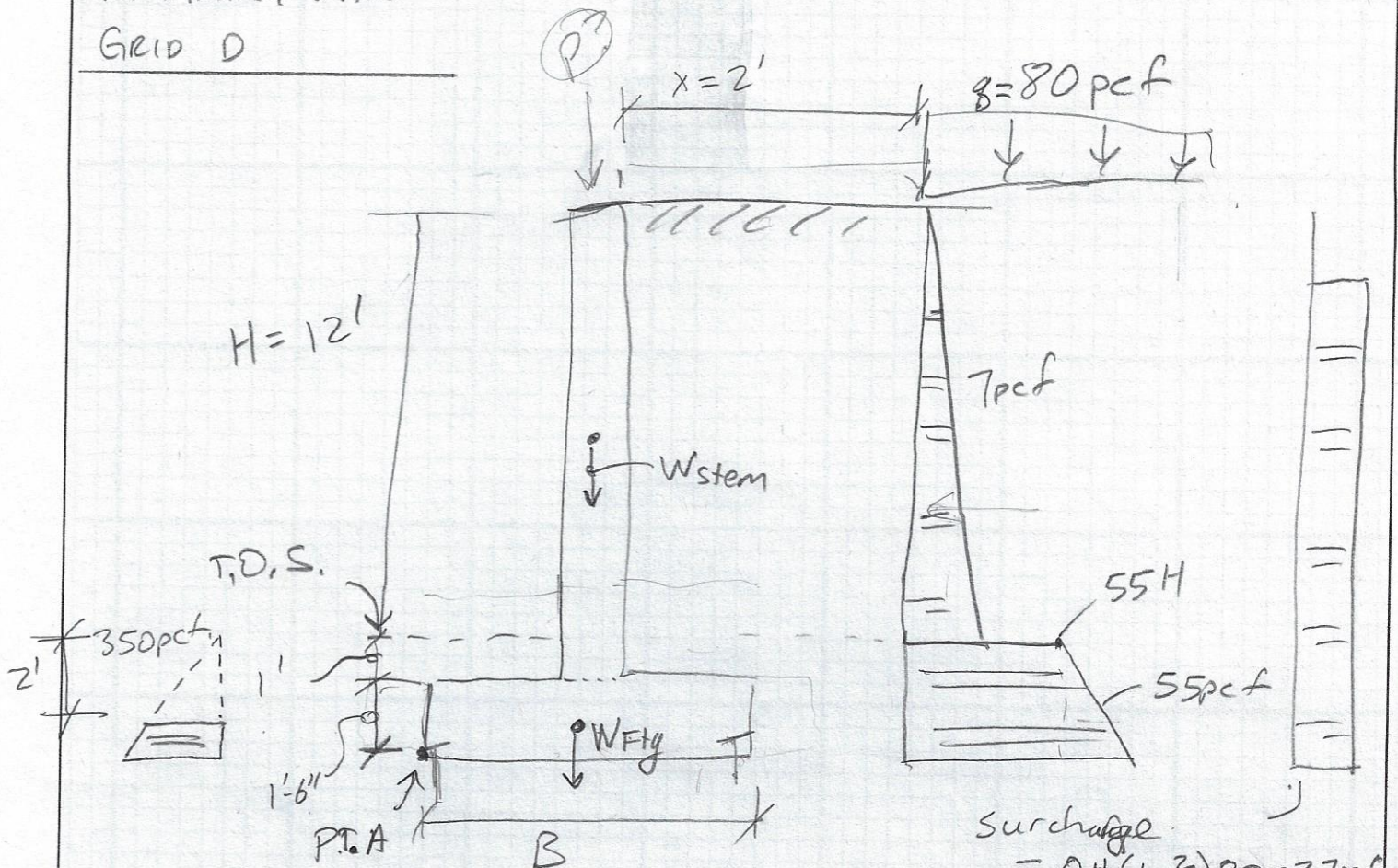
$$P_{7S} = 1355 \text{ \#}$$

$$P_{8W} = 1037 \text{ \#}$$

$$P_{8E} = 1845 \text{ \#}$$

RETAINING WALL

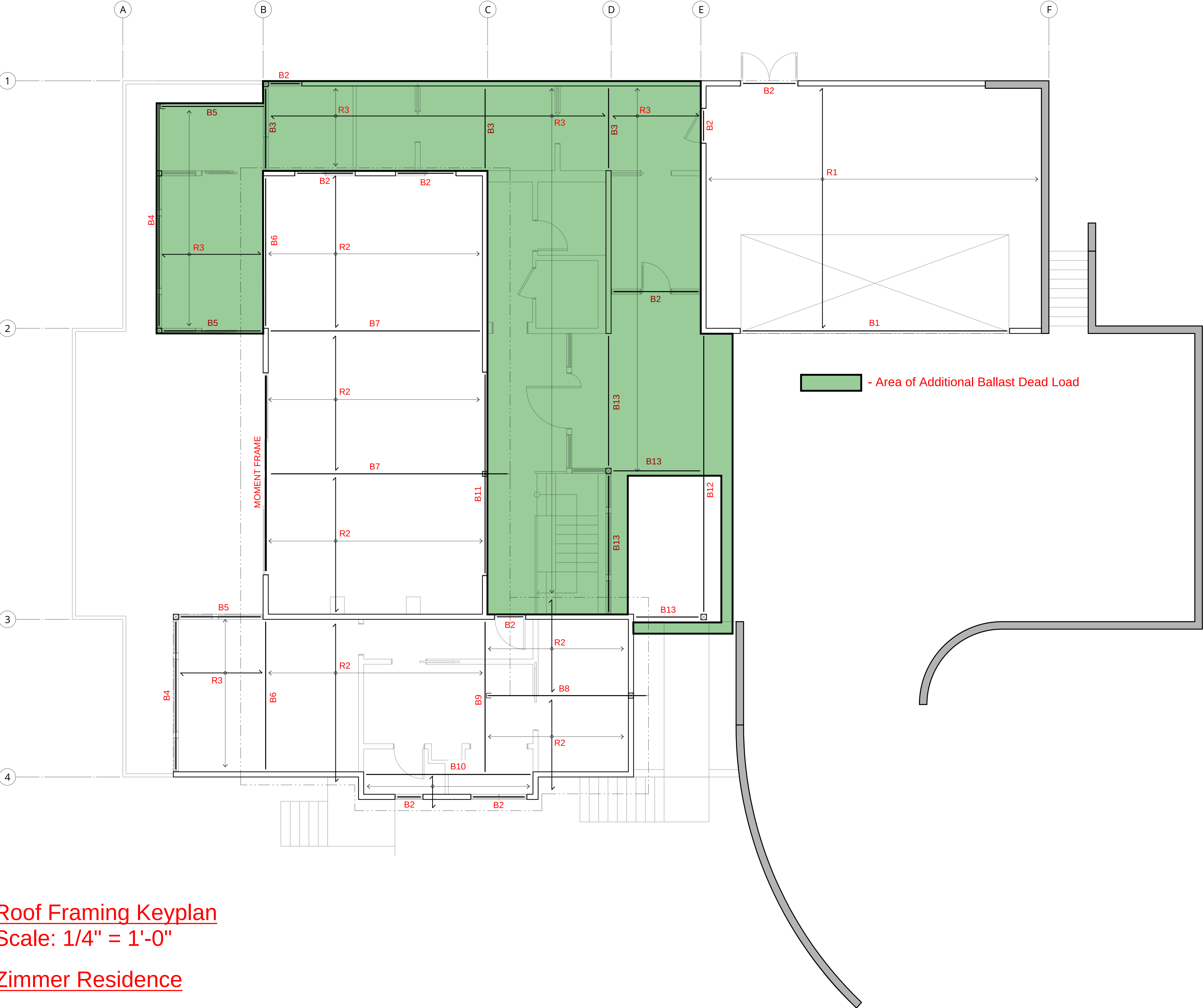
GRID D



$$O + M = 27\left(\frac{14.5^2}{2}\right) + 7\left(\frac{12^2}{2}\right)\left(2.5 + \frac{12}{3}\right) + 55(12)\left(\frac{2.5^2}{2}\right) + 55\left(\frac{2.5^3}{6}\right) = 8320 \text{ #-'/ft}$$

$$W_{\text{stem}} = 13'(100 \text{ pcf}) = 1300 \text{ #/ft}$$

$$W_{\text{fly}} = 225 \text{ pcf}(13)$$



Roof Framing Keyplan
Scale: 1/4" = 1'-0"
Zimmer Residence

	b (in)	d (in)	Sx (in ³)	Ix (in ⁴)
2x4	1.5	3.5	3.06	5.36
2x6	1.5	5.5	7.56	20.80
2x8	1.5	7.25	13.14	47.63
2x10	1.5	9.25	21.39	98.93
2x12	1.5	11.25	31.64	177.98
2x14	1.5	13.25	43.89	290.78
3x4	2.5	3.5	5.10	8.93
3x6	2.5	5.5	12.60	34.66
3x8	2.5	7.25	21.90	79.39
3x10	2.5	9.25	35.65	164.89
3x12	2.5	11.25	52.73	296.63
3x14	2.5	13.25	73.15	484.63
4x4	3.5	3.5	7.15	12.51
4x6	3.5	5.5	17.65	48.53
4x8	3.5	7.25	30.66	111.15
4x10	3.5	9.25	49.91	230.84
4x12	3.5	11.25	73.83	415.28
4x14	3.5	13.25	102.41	678.48
6x6	5.5	5.5	27.73	76.26
6x8	5.5	7.5	51.56	193.36
6x10	5.5	9.5	82.73	392.96
6x12	5.5	11.5	121.23	697.07
6x14	5.5	13.5	167.06	1127.67
6x16	5.5	15.5	220.23	1706.78

Hem-Fir No. 2			
M(#-ft)	Cd=1.0	Cd=1.15	Cd=1.6
(2)2x4	651	748	1,041
(2)2x6	1,393	1,602	2,228
(2)2x8	2,234	2,569	3,574
(2)2x10	3,333	3,833	5,333
(2)2x12	4,482	5,155	7,172
(2)2x14	5,596	6,435	8,954
DF-L No. 2			
3x4	574	660	919
3x6	1,229	1,413	1,966
3x8	1,971	2,267	3,154
3x10	2,941	3,382	4,706
3x12	3,955	4,548	6,328
3x14	4,938	5,678	7,900
DF-L No. 2			
4x4	804	924	1,286
4x6	1,720	1,979	2,753
4x8	2,989	3,438	4,783
4x10	4,492	5,166	7,187
4x12	6,091	7,004	9,745
4x14	7,681	8,833	12,289
DF-L No. 1			
6x6	3,120	3,587	4,991
6x8	5,801	6,671	9,281
6x10	9,307	10,703	14,891
6x12	13,638	15,684	21,821
6x14	18,550	21,333	29,680
6x16	24,081	27,693	38,530

TABLE 5

DOUGLAS-FIR GLUED LAMINATED BEAM SECTION PROPERTIES AND CAPACITIES $F_b = 2,400$ psi, $E = 1,800,000$ psi, $F_v = 265$ psi**3-1/8-INCH WIDTH**

Depth (in.)	6	7-1/2	9	10-1/2	12	13-1/2	15	16-1/2	18	19-1/2	21	22-1/2	24	25-1/2	27
Beam Weight (lb/ft)	4.6	5.7	6.8	8.0	9.1	10.3	11.4	12.5	13.7	14.8	16.0	17.1	18.2	19.4	20.5
A (in. ²)	18.8	23.4	28.1	32.8	37.5	42.2	46.9	51.6	56.3	60.9	65.6	70.3	75.0	79.7	84.4
S (in. ³)	19	29	42	57	75	95	117	142	169	198	230	264	300	339	380
I (in. ⁴)	56	110	190	301	450	641	879	1170	1519	1931	2412	2966	3600	4318	5126
EI (10 ⁶ lb-in. ²)	101	198	342	543	810	1153	1582	2106	2734	3476	4341	5339	6480	7773	9226
Moment Capacity (lb-ft)	3750	5859	8438	11484	15000	18984	23438	28359	33750	39609	45938	52734	60000	67734	75938
Shear Capacity (lb)	3313	4141	4969	5797	6625	7453	8281	9109	9938	10766	11594	12422	13250	14078	14906

3-1/2-INCH WIDTH

Depth (in.)	6	7-1/2	9	10-1/2	12	13-1/2	14	15	16	16-1/2	18	19-1/2	21	22-1/2	24
Beam Weight (lb/ft)	5.1	6.4	7.7	8.9	10.2	11.5	11.9	12.8	136	14.0	15.3	16.6	17.9	19.1	20.4
A (in. ²)	21.0	26.3	31.5	36.8	42.0	47.3	49.0	52.5	560	57.8	63.0	68.3	73.5	78.8	84.0
S (in. ³)	21	33	47	64	84	106	114	131	149	159	189	222	257	295	336
I (in. ⁴)	63	123	213	338	504	718	800	984	1195	1310	1701	2163	2701	3322	4032
EI (10 ⁶ lb-in. ²)	113	221	383	608	907	1292	1441	1772	2150	2358	3062	3893	4862	5980	7258
Moment Capacity (lb-ft)	4200	6563	9450	12863	16800	21263	22867	26250	29867	31763	37800	44363	51450	59063	67200
Shear Capacity (lb)	3710	4638	5565	6493	7420	8348	8657	9275	9893	10203	11130	12058	12985	13913	14840

5-1/8-INCH WIDTH

Depth (in.)	12	13-1/2	15	16-1/2	18	19-1/2	21	22-1/2	24	25-1/2	27	28-1/2	30	31-1/2	33
Beam Weight (lb/ft)	14.9	16.8	18.7	20.6	22.4	24.3	26.2	28.0	29.9	31.8	33.6	35.5	37.4	39.2	41.1
A (in. ²)	61.5	69.2	76.9	84.6	92.3	99.9	107.6	115.3	123.0	130.7	138.4	146.1	153.8	161.4	169.1
S (in. ³)	123	156	192	233	277	325	377	432	492	555	623	694	769	848	930
I (in. ⁴)	738	1051	1441	1919	2491	3167	3955	4865	5904	7082	8406	9887	11531	13349	15348
EI (10 ⁶ lb-in. ²)	1328	1891	2595	3453	4483	5700	7119	8757	10627	12747	15131	17796	20756	24028	27627
Moment Capacity (lb-ft)	24600	31134	38438	46509	55350	64959	75338	86484	98400	111084	124538	138759	153750	169509	186038
Shear Capacity (lb)	10865	12223	13581	14939	16298	17656	19014	20372	21730	23088	24446	25804	27163	28521	29879

5-1/2-INCH WIDTH

Depth (in.)	12	13-1/2	14	15	16	16-1/2	18	19-1/2	21	22-1/2	24	25-1/2	27	28-1/2	30
Beam Weight (lb/ft)	16.0	18.0	18.7	20.1	21.4	22.1	24.1	26.1	28.1	30.1	32.1	34.1	36.1	38.1	40.1
A (in. ²)	66.0	74.3	77.0	82.5	88.0	90.8	99.0	107.3	115.5	123.8	132.0	140.3	148.5	156.8	165.0
S (in. ³)	132	167	180	206	235	250	297	349	404	464	528	596	668	745	825
I (in. ⁴)	792	1128	1258	1547	1877	2059	2673	3398	4245	5221	6336	7600	9021	10610	12375
EI (10 ⁶ lb-in. ²)	1426	2030	2264	2784	3379	3706	4811	6117	7640	9397	11405	13680	16238	19098	22275
Moment Capacity (lb-ft)	26400	33413	35933	41250	46933	49913	59400	69713	80850	92813	105600	119213	133650	148913	165000
Shear Capacity (lb)	11660	13118	13603	14575	15547	16033	17490	18948	20405	21863	23320	24778	26235	27693	29150

6-3/4-INCH WIDTH

Depth (in.)	18	19-1/2	21	22-1/2	24	25-1/2	27	28-1/2	30	31-1/2	33	34-1/2	36	37-1/2	39
Beam Weight (lb/ft)	29.5	32.0	34.5	36.9	39.4	41.8	44.3	46.8	49.2	51.7	54.1	56.6	59.1	61.5	64.0
A (in. ²)	121.5	131.6	141.8	151.9	162.0	172.1	182.3	192.4	202.5	212.6	222.8	232.9	243.0	253.1	263.3
S (in. ³)	365	428	496	570	648	732	820	914	1013	1116	1225	1339	1458	1582	1711
I (in. ⁴)	3281	4171	5209	6407	7776	9327	11072	13021	15188	17581	20215	23098	26244	29663	33367
EI (10 ⁶ lb-in. ²)	5905	7508	9377	11533	13997	16789	19929	23438	27338	31647	36386	41577	47239	53394	60060
Moment Capacity (lb-ft)	72900	85556	99225	113906	129600	146306	164025	182756	202500	223256	245025	267806	291600	316406	342225
Shear Capacity (lb)	21465	23254	25043	26831	28620	30409	32198	33986	35775	37564	39353	41141	42930	44719	46508

8-3/4-INCH WIDTH

Depth (in.)	24	25-1/2	27	28-1/2	30	31-1/2	33	34-1/2	36	37-1/2	39	40-1/2	42	43-1/2	45
Beam Weight (lb/ft)	51.0	54.2	57.4	60.6	63.8	67.0	70.2	73.4	76.6	79.8	82.9	86.1	89.3	92.5	95.7
A (in. ²)	210.0	223.1	236.3	249.4	262.5	275.6	288.8	301.9	315.0	328.1	341.3	354.4	367.5	380.6	393.8
S (in. ³)	840	948	1063	1185	1313	1447	1588	1736	1890	2051	2218	2392	2573	2760	2953
I (in. ⁴)	10080	12091	14352	16880	19688	22791	26204	29942	34020	38452	43253	48439	54023	60020	66445
EI (10 ⁶ lb-in. ²)	18144	21763	25834	30383	35438	41023	47167	53896	61236	69214	77856	87190	97241	108036	119602
Moment Capacity (lb-ft)	168000	189656	212625	236906	262500	289406	317625	347156	378000	410156	443625	478406	514500	551906	590625
Shear Capacity (lb)	37100	39419	41738	44056	46375	48694	51013	53331	55650	57969	60288	62606	64925	67244	69563

Footnotes:

(1) Beam weight is based on density of 35 pcf.

(2) Moment capacity must be adjusted for volume effect: $C_v = \left(\frac{12}{d}\right)^{1/30} \times \left(\frac{5.125}{b}\right)^{1/30} \times \left(\frac{21}{L}\right)^{1/30} \leq 1.0$, where d = beam depth (in.), b = beam width (in.), and L = beam length (ft).

(3) Moment and shear capacities are based on a normal (10 years) duration of load and should be adjusted for the design duration of load per the applicable building code.

DESIGN PROPERTIES

Allowable Design Properties⁽¹⁾ (100% Load Duration)

Grade	Width	Design Property	Depth									
			4¾"	5½"	5½" Plank Orientation	7¼"	8¾"	9½"	11¾"	14"	16"	18"
TimberStrand® LSL												
1.3E	3½"	Moment (ft-lbs)	1,735	2,685	1,780	4,550	6,335					
		Shear (lbs)	4,340	5,455	1,925	7,190	8,555					
		Moment of Inertia (in ⁴)	24	49	20	111	187					
		Weight (plf)	4.5	5.6	5.6	7.4	8.8					
1.55E	1¾"	Moment (ft-lbs)						5,210	7,975	10,920	14,090	
		Shear (lbs)						3,435	4,295	5,065	5,785	
		Moment of Inertia (in ⁴)						125	244	400	597	
		Weight (plf)						5.2	6.5	7.7	8.8	
	3½"	Moment (ft-lbs)						10,420	15,955	21,840	28,180	
		Shear (lbs)						6,870	8,590	10,125	11,575	
		Moment of Inertia (in ⁴)						250	488	800	1,195	
		Weight (plf)						10.4	13	15.3	17.5	
Microllam® LVL												
2.0E	1¾"	Moment (ft-lbs)		2,125		3,555		3,885	3,925	12,180	15,565	19,375
		Shear (lbs)		1,830		2,410		3,160	3,950	4,665	5,320	5,985
		Moment of Inertia (in ⁴)		24		56		125	244	400	597	851
		Weight (plf)		2.8		3.7		4.8	6	7.1	8.2	9.2
Parallam® PSL												
2.2E	3½"	Moment (ft-lbs)						13,055	19,900	27,160	34,955	43,665
		Shear (lbs)						6,430	8,035	9,475	10,825	12,180
		Moment of Inertia (in ⁴)						250	488	800	1,195	1,701
		Weight (plf)						10.4	13.0	15.3	17.5	19.7
	5¼"	Moment (ft-lbs)						19,585	29,855	40,740	52,430	65,495
		Shear (lbs)						9,645	12,055	14,240	16,240	18,270
		Moment of Inertia (in ⁴)						375	733	1,201	1,792	2,552
		Weight (plf)						15.6	19.5	23.0	26.3	29.5
	7"	Moment (ft-lbs)						26,115	39,805	54,325	69,905	87,325
		Shear (lbs)						12,855	16,070	18,945	21,655	24,360
		Moment of Inertia (in ⁴)						500	977	1,601	2,389	3,402
		Weight (plf)						20.8	26.0	30.6	35.0	39.4

(1) For product in beam orientation, unless otherwise noted.

DESIGN PROPERTIES

Design Stresses⁽¹⁾ (100% Load Duration)

Grade	Orientation	G Shear Modulus of Elasticity (psi)	E Modulus of Elasticity (psi)	E _{min} Adjusted Modulus of Elasticity ⁽²⁾ (psi)	F _b Flexural Stress ⁽³⁾ (psi)	F _t Tension Stress ⁽⁴⁾ (psi)	F _{c⊥} Compression Perpendicular to Grain ⁽⁵⁾ (psi)	F _c Compression Parallel to Grain (psi)	F _v Horizontal Shear Parallel to Grain (psi)	SG Equivalent Specific Gravity ⁽⁶⁾
TimberStrand® LSL										
1.3E	Beam/Column	81,250	1.3 x 10 ⁶	660,750	1,700	1,075	710	1,835	425	0.50 ⁽⁷⁾
	Plank	81,250	1.3 x 10 ⁶	660,750	1,900 ⁽⁸⁾	1,075	635 ⁽⁹⁾	1,835	150	0.50 ⁽⁷⁾
1.55E	Beam	96,875	1.55 x 10 ⁶	787,815	2,325	1,070 ⁽¹⁰⁾	900	2,170	310 ⁽¹⁰⁾	0.50 ⁽⁷⁾
Microllam® LVL										
2.0E	Beam	125,000	2.0 x 10 ⁶	1,016,535	2,600	1,555	750	2,510	285	0.50
Parallam® PSL										
1.8E	Column	112,500	1.8 x 10 ⁶	914,880	2,400 ⁽¹¹⁾	1,755	545 ⁽¹¹⁾	2,500	190 ⁽¹¹⁾	0.50
2.0E	Beam	125,000	2.0 x 10 ⁶	1,016,535	2,900	2,025	625	2,900 ⁽¹²⁾	290	0.50

(1) Unless otherwise noted, adjustment to the design stresses for duration of load are permitted in accordance with the applicable code.

(2) Reference modulus of elasticity for beam and column stability calculations, per NDS®.

(3) For 12" depth. For other depths, multiply F_b by the appropriate factor as follows:

- For TimberStrand® LSL, multiply by $\left[\frac{12}{d}\right]^{0.092}$
- For Microllam® LVL, multiply by $\left[\frac{12}{d}\right]^{0.136}$
- For Parallam® PSL, multiply by $\left[\frac{12}{d}\right]^{0.111}$

(4) F_t has been adjusted to reflect the volume effects for most standard applications.

(5) F_{c⊥} may not be increased for duration of load.

(6) For lateral connection design only.

(7) Specific gravity of 0.58 may be used for bolts installed perpendicular to face and loaded perpendicular to grain.

(8) Values are for thickness up to 3½".

(9) For members less than 1¾" thick and in plank orientation, use F_{c⊥} of 670 psi. NDS® bearing area factor C_b = 1.0.

(10) Value accounts for large hole capabilities. See **Allowable Holes** on page 26.

(11) Value shown is for plank orientation.

(12) For column applications, use F_{c||} of 500 psi. Alternatively, refer to ESR-1387, Table 1, footnote 15.

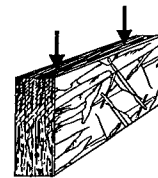
General Assumptions for Trus Joist® Beams

- Lateral support is required at bearing and along the span at 24" on-center, maximum.
- Bearing lengths are based on each product's bearing stress for applicable grade and orientation.
- All members 7¼" and less in depth are restricted to a maximum deflection of ⅛".
- Beams that are 1¾" x 16" and deeper require multiple plies.
- No camber.
- Beams and columns must remain straight to within 5L/4608 (in.) of true alignment. L is the unrestrained length of the member in feet.

For applications not covered in this brochure, contact your Weyerhaeuser representative.

See pages 28 and 29 for multiple-member beam connections.

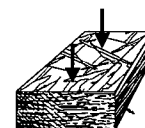
Beam Orientation



Column Orientation



Plank Orientation



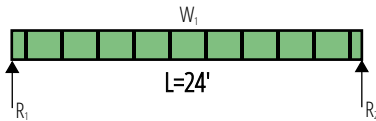
*TimberStrand® LSL, Microllam® LVL, and
untreated Parallam® PSL are intended
for dry-use applications*

Upper Roof Framing Design

$$D_L = 20\text{psf}, D_{L@BALLAST} = 32\text{psf}, S_L = 25\text{psf}$$

B1

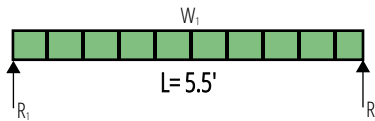
7x16 PSL



$$\begin{aligned} W_1 &= (21.5/2)(45) = 484 \text{ plf} \\ R_1 &= R_2 = 5810\# \\ M &= 34.8 \text{ K-ft} \\ f_b &= 1400 \text{ psi} \\ f_v &= 78 \text{ psi} \\ \Delta_{TL} &= 0.76" = \ell/379 \end{aligned}$$

B2

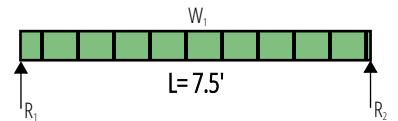
(2) 2x8



$$\begin{aligned} W_1 &= (21.5/2)(45) = 484 \text{ plf} \\ R_1 &= R_2 = 1330\# \\ M &= 1.83 \text{ K-ft} \\ f_b &= 836 \text{ psi} \\ f_v &= 92 \text{ psi} \\ \Delta_{TL} &= 0.07" = \ell/942 \end{aligned}$$

B3

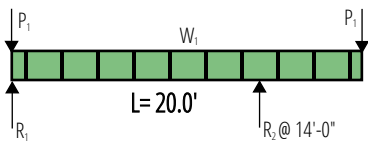
4x12



$$\begin{aligned} W_1 &= (19/2)(57) = 542 \text{ plf} \\ R_1 &= R_2 = 2030\# \\ M &= 3.81 \text{ K-ft} \\ f_b &= 620 \text{ psi} \\ f_v &= 77 \text{ psi} \\ \Delta_{TL} &= 0.06" = \ell/1500 \end{aligned}$$

B4

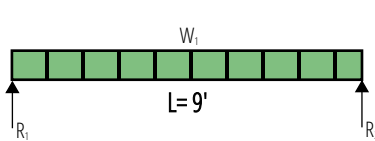
6x12



$$\begin{aligned} W_1 &= (9/2)(57) = 257 \text{ plf} \\ P &= 342\# \\ R_1 &= 1660\# \quad R_2 = 4160\# \\ M &= 6.68 \text{ K-ft} \\ f_b &= 661 \text{ psi} \\ f_v &= 99 \text{ psi} \\ \Delta_{TL} &= 0.12" = \ell/600 \end{aligned}$$

B5

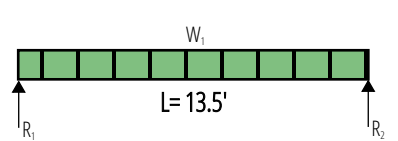
2x12



$$\begin{aligned} W_1 &= (16/12)(57) = 76 \text{ plf} \\ R_1 &= R_2 = 342\# \\ M &= 0.77 \text{ K-ft} \\ f_b &= 292 \text{ psi} \\ f_v &= 30 \text{ psi} \\ \Delta_{TL} &= 0.04" = \ell/1500 \end{aligned}$$

B6

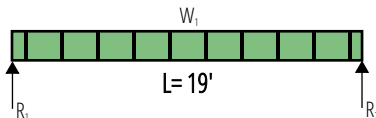
6x12



$$\begin{aligned} W_1 &= (10/2)(57) = 285 \text{ plf} \\ R_1 &= R_2 = 1920\# \\ M &= 6.49 \text{ K-ft} \\ f_b &= 643 \text{ psi} \\ f_v &= 45 \text{ psi} \\ \Delta_{TL} &= 0.19" = \ell/852 \end{aligned}$$

B7

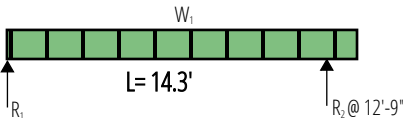
5 1/2" x 16 1/2" GLB



$$\begin{aligned} W_1 &= (26.5/2)(45) = 596 \text{ plf} \\ R_1 &= R_2 = 5660\# \\ M &= 26.9 \text{ K-ft} \\ f_b &= 1290 \text{ psi} \\ f_v &= 94 \text{ psi} \\ \Delta_{TL} &= 0.47" = \ell/485 \end{aligned}$$

B8

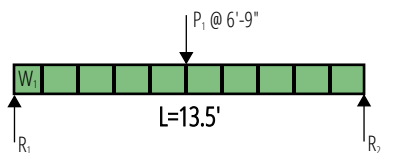
6x12



$$\begin{aligned} W_1 &= (13.5/2)(45) = 304 \text{ plf} \\ R_1 &= 1910\# \quad R_2 = 2440\# \\ M &= 6.00 \text{ K-ft} \\ f_b &= 975 \text{ psi} \leq F'_b = 1035 \text{ psi} \\ f_v &= 77 \text{ psi} \\ \Delta_{TL} &= 0.26" = \ell/660 \end{aligned}$$

B9

6x12



$$\begin{aligned} W_1 &= (16/12)(45) = 60 \text{ plf} \\ P &= 1910\# \\ R_1 &= R_2 = 1360\# \\ M &= 7.81 \text{ K-ft} \\ f_b &= 773 \text{ psi} \\ f_v &= 32 \text{ psi} \\ \Delta_{TL} &= 0.19" = \ell/852 \end{aligned}$$

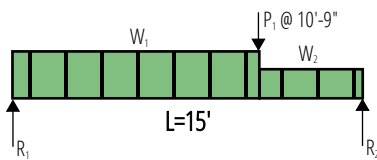
Zimmer Residence

Upper Roof Framing Design (Con't)

$$D_L = 20\text{psf}, D_{L@BALLAST} = 32\text{psf}, S_L = 25\text{psf}$$

B10

6x12



$$W_1 = (15.5/2)(45) = 349 \text{ plf}$$

$$W_2 = (8.75/2)(45) = 197 \text{ plf}$$

$$P = 1360\#$$

$$R_1 = 2910\# \quad R_2 = 3040\#$$

$$M = 12.1 \text{ K-ft}$$

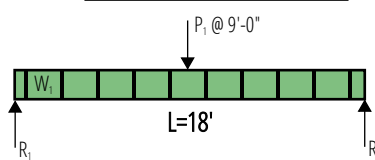
$$f_b = 1200 \text{ psi}$$

$$f_v = 72 \text{ psi}$$

$$\Delta_{it} = 0.44" = \ell/409$$

B11

5 1/2" x 18" GLB



$$W_1 = (16/12)(45) = 60 \text{ plf}$$

$$P = 5660\#$$

$$R_1 = R_2 = 3370\#$$

$$M = 27.9 \text{ K-ft}$$

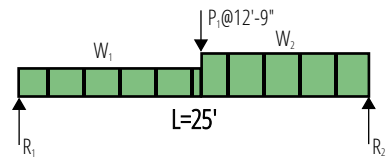
$$f_b = 1130 \text{ psi}$$

$$f_v = 51 \text{ psi}$$

$$\Delta_{it} = 0.28" = \ell/771$$

B12

5 1/2" x 16 1/2" GLB



$$W_1 = (16/12)(57) = 76 \text{ plf}$$

$$W_2 = (11/2)(57) = 314 \text{ plf}$$

$$P_1 = 304\#$$

$$R_1 = 1810\# \quad R_2 = 3310\#$$

$$M = 17.4 \text{ K-ft}$$

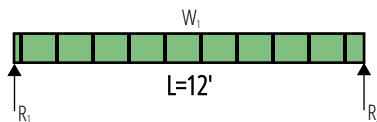
$$f_b = 837 \text{ psi}$$

$$f_v = 55 \text{ psi}$$

$$\Delta_{it} = 0.5" = \ell/600$$

B13

6x10



$$W_1 = (19/2)(57) = 542 \text{ plf}$$

$$R_1 = R_2 = 3250\#$$

$$M = 9.76 \text{ K-ft}$$

$$f_b = 1420 \text{ psi} \leq 1552 \text{ psi}$$

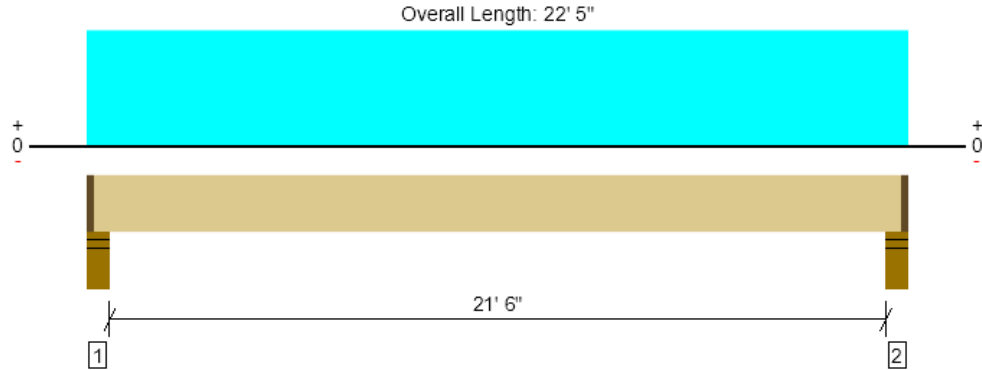
$$f_v = 93 \text{ psi}$$

$$\Delta_{it} = 0.4" = \ell/360$$

Zimmer Residence

Roof Level, Rafter R1

1 piece(s) 1 3/4" x 14" 2.0E Microllam® LVL @ 16" OC



Drawing is Conceptual. All locations are measured from the outside face of left support (or left cantilever end). All dimensions are horizontal (typ.).

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	664 @ 4' 1/2"	4102 (3.75")	Passed (16%)	--	1.0 D + 1.0 S (All Spans)
Shear (lbs)	575 @ 1' 7 1/2"	5353	Passed (11%)	1.15	1.0 D + 1.0 S (All Spans)
Moment (Ft-lbs)	3521 @ 11' 2 1/2"	14506	Passed (24%)	1.15	1.0 D + 1.0 S (All Spans)
Live Load Defl. (in)	0.216 @ 11' 2 1/2"	1.083	Passed (L/999+)	--	1.0 D + 1.0 S (All Spans)
Total Load Defl. (in)	0.388 @ 11' 2 1/2"	1.444	Passed (L/670)	--	1.0 D + 1.0 S (All Spans)

- Deflection criteria: LL (L/240) and TL (L/180).
- Allowed moment does not reflect the adjustment for the beam stability factor.
- A 4% increase in the moment capacity has been added to account for repetitive member usage.

Member Length : 22' 1 1/2"
System : Roof
Member Type : Joist
Building Use : Residential
Building Code : IBC 2021
Design Methodology : ASD
Member Pitch : 0/12

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Snow	Factored	
1 - Stud wall - DF	5.50"	3.75"	1.50"	299	374	673	1 3/4" Rim Board
2 - Stud wall - DF	5.50"	3.75"	1.50"	299	374	673	1 3/4" Rim Board

- Rim Board is assumed to carry all loads applied directly above it, bypassing the member being designed.

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	15' 8" o/c	
Bottom Edge (Lu)	22' 2" o/c	

- Maximum allowable bracing intervals based on applied load.

Vertical Load	Location (Side)	Spacing	Dead (0.90)	Snow (1.15)	Comments
1 - Uniform (PSF)	0 to 22' 5"	16"	20.0	25.0	Default Load

Weyerhaeuser Notes

Weyerhaeuser warrants that the sizing of its products will be in accordance with Weyerhaeuser product design criteria and published design values. Weyerhaeuser expressly disclaims any other warranties related to the software. Use of this software is not intended to circumvent the need for a design professional as determined by the authority having jurisdiction. The designer of record, builder or framer is responsible to assure that this calculation is compatible with the overall project. Accessories (Rim Board, Blocking Panels and Squash Blocks) are not designed by this software. Products manufactured at Weyerhaeuser facilities are third-party certified to sustainable forestry standards. Weyerhaeuser Engineered Lumber Products have been evaluated by ICC-ES under evaluation reports ESR-1153 and ESR-1387 and/or tested in accordance with applicable ASTM standards. For current code evaluation reports, Weyerhaeuser product literature and installation details refer to www.weyerhaeuser.com/woodproducts/document-library.

The product application, input design loads, dimensions and support information have been provided by ForteWEB Software Operator

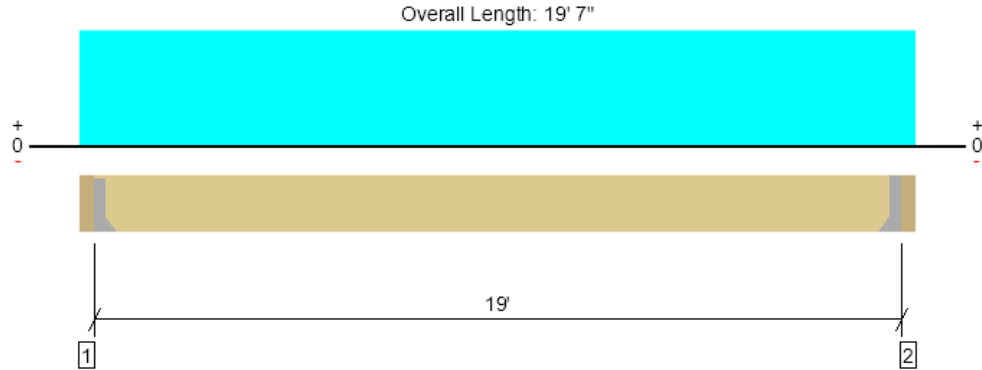
ForteWEB Software Operator	Job Notes
Craig Donison Buker Engineering (206) 258-6333 craig@bukerengineering.com	



Zimmer Residence - Rafter R2									
L =	13 ft	9 in				Lumber Type =	Hem-Fir #2		
w_{DL} =	20	psf				F_b =	850	psi	
w_{LL}	25	psf				F_v =	150	psi	
Spacing =	16	in o.c.				E =	1,300,000	psi	
Joist Size	2x12					C_D =	1		
S =	31.64	in ³				C_r =	1.15		
I =	177.98	in ⁴				C_F =	1		
A =	16.88	in ²				incised	no		
M =	1418	#-ft							
R1 = R2 =	413	#							
f_b =	538	psi				E' =	1300000	psi	
f_v =	36.7	psi				F_b' =	978	psi	OK
Δ_{DL} =	0.093	in	=			F_v' =	150	psi	OK
Δ_{LL} =	0.116	in	=			L/	1780		
Δ_{TL} =	0.209	in	=			L/	1424		
						L/	791		

Roof Level, Rafter R3

1 piece(s) 1 3/4" x 11 7/8" 2.0E Microllam® LVL @ 16" OC



Drawing is Conceptual. All locations are measured from the outside face of left support (or left cantilever end). All dimensions are horizontal (typ.).

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	722 @ 3 1/2"	1969 (1.50")	Passed (37%)	--	1.0 D + 1.0 S (All Spans)
Shear (lbs)	647 @ 1' 3 3/8"	4541	Passed (14%)	1.15	1.0 D + 1.0 S (All Spans)
Moment (Ft-lbs)	3429 @ 9' 9 1/2"	10673	Passed (32%)	1.15	1.0 D + 1.0 S (All Spans)
Live Load Defl. (in)	0.208 @ 9' 9 1/2"	0.950	Passed (L/999+)	--	1.0 D + 1.0 S (All Spans)
Total Load Defl. (in)	0.475 @ 9' 9 1/2"	1.267	Passed (L/480)	--	1.0 D + 1.0 S (All Spans)

- Deflection criteria: LL (L/240) and TL (L/180).
- Allowed moment does not reflect the adjustment for the beam stability factor.
- A 4% increase in the moment capacity has been added to account for repetitive member usage.

Member Length : 19'
System : Roof
Member Type : Joist
Building Use : Residential
Building Code : IBC 2021
Design Methodology : ASD
Member Pitch : 0/12

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Snow	Factored	
1 - Hanger on 11 7/8" DF beam	3.50"	Hanger ¹	1.50"	418	326	744	See note ¹
2 - Hanger on 11 7/8" DF beam	3.50"	Hanger ¹	1.50"	418	326	744	See note ¹

- At hanger supports, the Total Bearing dimension is equal to the width of the material that is supporting the hanger
- ¹ See Connector grid below for additional information and/or requirements.

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	13' 11" o/c	
Bottom Edge (Lu)	19' o/c	

- Maximum allowable bracing intervals based on applied load.

Connector: Simpson Strong-Tie

Support	Model	Seat Length	Top Fasteners	Face Fasteners	Member Fasteners	Accessories
1 - Face Mount Hanger	IUS1.81/9.5	2.00"	N/A	8-10dx1.5	2-10dx1.5	
2 - Face Mount Hanger	IUS1.81/9.5	2.00"	N/A	8-10dx1.5	2-10dx1.5	

- Refer to manufacturer notes and instructions for proper installation and use of all connectors.

Vertical Load	Location (Side)	Spacing	Dead (0.90)	Snow (1.15)	Comments
1 - Uniform (PSF)	0 to 19' 7"	16"	32.0	25.0	Default Load

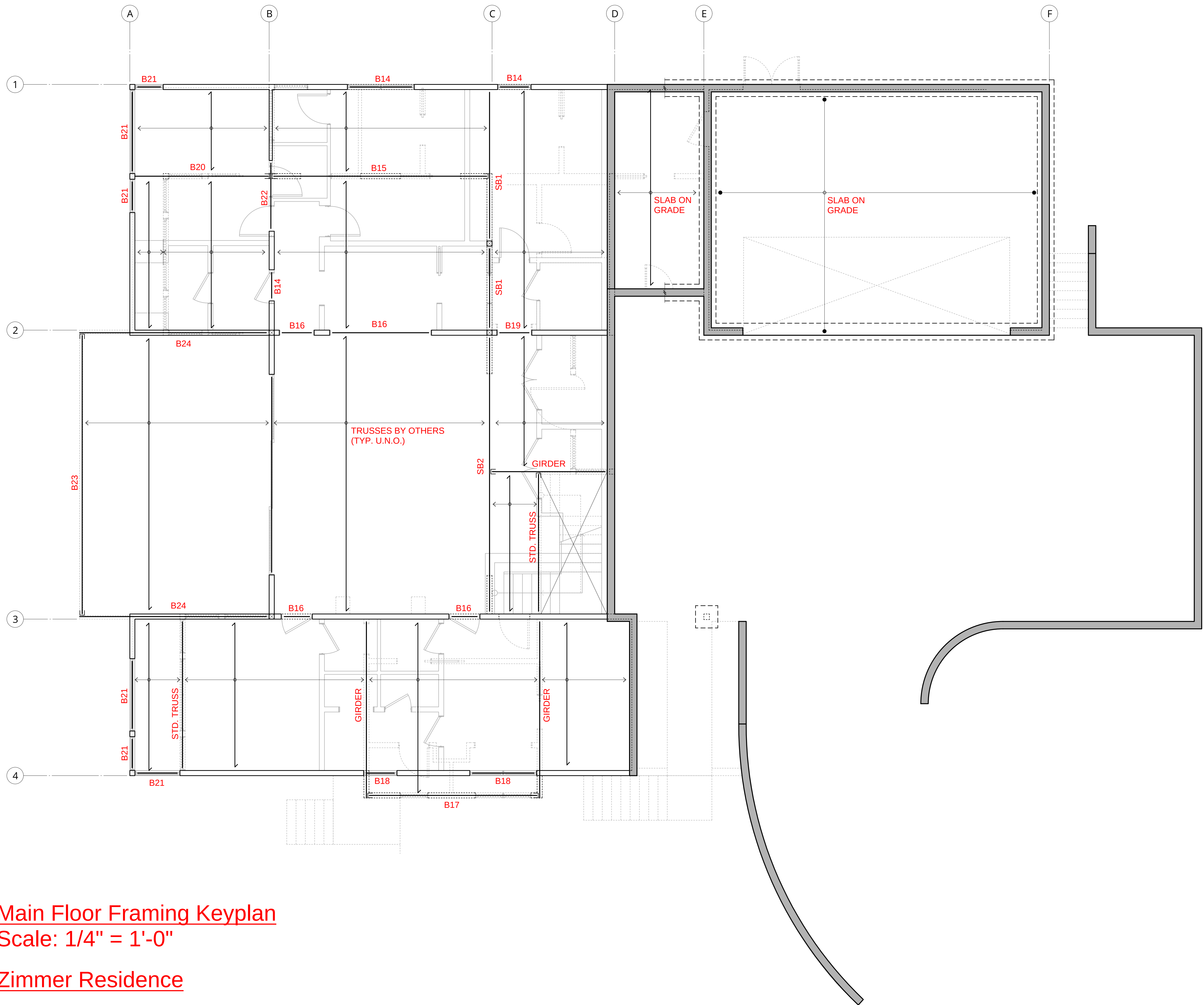
Weyerhaeuser Notes

Weyerhaeuser warrants that the sizing of its products will be in accordance with Weyerhaeuser product design criteria and published design values. Weyerhaeuser expressly disclaims any other warranties related to the software. Use of this software is not intended to circumvent the need for a design professional as determined by the authority having jurisdiction. The designer of record, builder or framer is responsible to assure that this calculation is compatible with the overall project. Accessories (Rim Board, Blocking Panels and Squash Blocks) are not designed by this software. Products manufactured at Weyerhaeuser facilities are third-party certified to sustainable forestry standards. Weyerhaeuser Engineered Lumber Products have been evaluated by ICC-ES under evaluation reports ESR-1153 and ESR-1387 and/or tested in accordance with applicable ASTM standards. For current code evaluation reports, Weyerhaeuser product literature and installation details refer to www.weyerhaeuser.com/woodproducts/document-library.

The product application, input design loads, dimensions and support information have been provided by ForteWEB Software Operator

ForteWEB Software Operator	Job Notes
Craig Donison Buker Engineering (206) 258-6333 craig@bukerengineering.com	





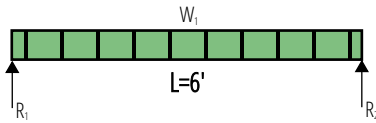
Main Floor Framing Keyplan
Scale: 1/4" = 1'-0"
Zimmer Residence

Main Floor Framing Design

$$D_L = 20\text{psf}, D_{L@DECK} = 35\text{psf}, S_L = 25\text{psf}, L_L = 40\text{psf}, L_{L@Deck} = 60\text{psf}$$

B14

(2) 2x8



$$W_1 = (10/2)(55) + (16/12)(57) + 100 = 382 \text{ plf}$$

$$R_1 = R_2 = 1150^\#$$

$$M = 1.72 \text{ K-ft}$$

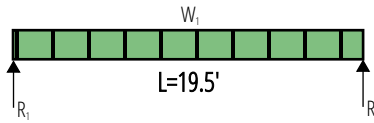
$$f_b = 785 \text{ psi}$$

$$f_v = 80 \text{ psi}$$

$$\Delta_{TL} = 0.09" = \ell/800$$

B15

5 1/2" x 21" GLB



$$W_1 = (21.5/2)(55) + (15/2)(45) + 100 = 1029 \text{ plf}$$

$$R_1 = R_2 = 10000^\#$$

$$M = 48.9 \text{ K-ft}$$

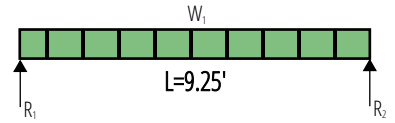
$$f_b = 1390 \text{ psi}$$

$$f_v = 130 \text{ psi}$$

$$\Delta_{TL} = 0.41" = \ell/570$$

B16

6x12



$$W_1 = (39/2)(55) + 100 = 1172 \text{ plf}$$

$$R_1 = R_2 = 5420^\#$$

$$M = 12.5 \text{ K-ft}$$

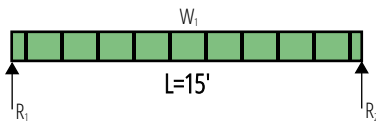
$$f_b = 1240 \text{ psi}$$

$$f_v = 129 \text{ psi}$$

$$\Delta_{TL} = 0.17" = \ell/652$$

B17

6x10



$$W_1 = (2/2)(55) + (2/2)(45) + 100 = 200 \text{ plf}$$

$$R_1 = R_2 = 1500^\#$$

$$M = 5.62 \text{ K-ft}$$

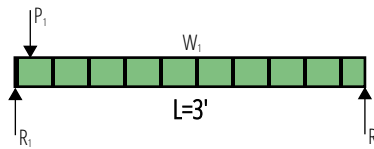
$$f_b = 816 \text{ psi}$$

$$f_v = 43 \text{ psi}$$

$$\Delta_{TL} = 0.36" = \ell/500$$

B18

6x8



$$W_1 = (16/2)(55) = 440 \text{ plf}$$

$$P_1 = 1580^\#$$

$$R_1 = 3410^\# \quad R_2 = 1210^\#$$

$$M = 1.65 \text{ K-ft}$$

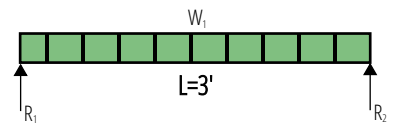
$$f_b = 384 \text{ psi}$$

$$f_v = 124 \text{ psi}$$

$$\Delta_{TL} = 0.01" = \ell/3600$$

B19

(2) 2x8



$$W_1 = (34/2)(55) = 935 \text{ plf}$$

$$R_1 = R_2 = 1400^\#$$

$$M = 1.05 \text{ K-ft}$$

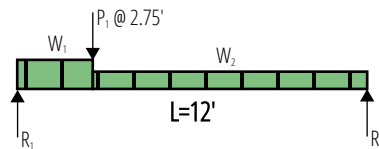
$$f_b = 480 \text{ psi}$$

$$f_v = 96 \text{ psi}$$

$$\Delta_{TL} = 0.02" = \ell/1800$$

B20

5 1/2" x 15" GLB



$$W_1 = (21.5/2)(100) = 1075 \text{ plf}$$

$$W_2 = (7.5/2)(100) + (13.5/2)(55) + 100 = 850 \text{ plf}$$

$$P_1 = 4160^\#$$

$$R_1 = 8850^\# \quad R_2 = 6120^\#$$

$$M = 22.1 \text{ K-ft}$$

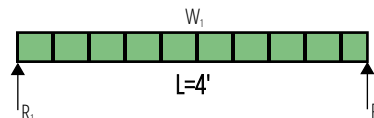
$$f_b = 1280 \text{ psi}$$

$$f_v = 160 \text{ psi}$$

$$\Delta_{TL} = 0.21" = \ell/685$$

B21

6x10



$$W_1 = (13.5/2)(100) = 675 \text{ plf}$$

$$R_1 = R_2 = 1350^\#$$

$$M = 1.35 \text{ K-ft}$$

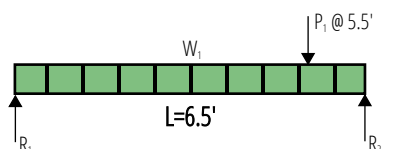
$$f_b = 616 \text{ psi}$$

$$f_v = 93 \text{ psi}$$

$$\Delta_{TL} = 0.04" = \ell/1200$$

B22

5 1/2" x 21" GLB



$$W_1 = (16/12)(100) = 133 \text{ plf}$$

$$P_1 = 6120^\# + 10000^\# = 16200^\#$$

$$R_1 = 2920^\# \quad R_2 = 14100^\#$$

$$M = 14.1 \text{ K-ft}$$

$$f_b = 418 \text{ psi}$$

$$f_v = 183 \text{ psi}$$

$$\Delta_{TL} = 0.01" = \ell/7800$$

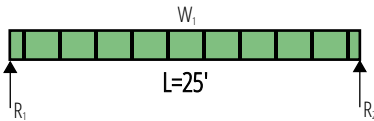
Zimmer Residence

Main Floor Framing Design (Con't)

$$D_L = 20\text{psf}, S_L = 25\text{psf}, L_L = 40\text{psf}, L_{L@Deck} = 60\text{psf}$$

B23

3 1/2" x 18" GLB



$$W_1 = (16/12)(100) = 133 \text{ plf}$$

$$R_1 = R_2 = 1660^\#$$

$$M = 10.4 \text{ K-ft}$$

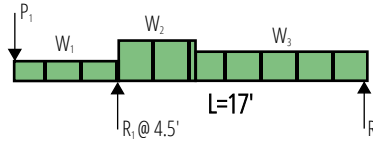
$$f_b = 660 \text{ psi}$$

$$f_v = 40 \text{ psi}$$

$$\Delta_n = 0.38" = \ell/789$$

B24

5 1/2" x 18" GLB



$$W_1 = (25/2)(100) = 1250 \text{ plf}$$

$$W_2 = (39/2)(100) + 100 = 1950 \text{ plf}$$

$$W_3 = (25/2)(100) + (14/2)(55) + 100 = 1735 \text{ plf}$$

$$P_1 = 1660^\#$$

$$R_1 = 20300^\# \quad R_2 = 9310^\#$$

$$M = 25.0 \text{ K-ft}$$

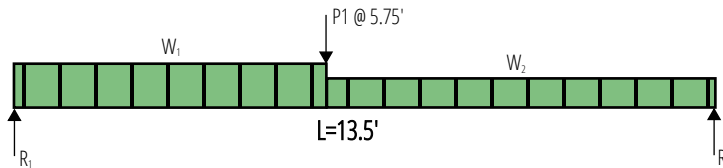
$$f_b = 1010 \text{ psi}$$

$$f_v = 192 \text{ psi}$$

$$\Delta_n = 0.13" = \ell/1153$$

SB1

5 1/2" x 21" GLB



$$W_{10} = (10.5/2)(37) + (16/12)(20) + (24/12)(15) + 100 = 351 \text{ plf}$$

$$W_{11} = (24/12)(40) = 80 \text{ plf}$$

$$W_{15} = (3/2)(25) = 38 \text{ plf}$$

$$W_{20} = (24/12)(15) = 30 \text{ plf}$$

$$W_{21} = (24/12)(40) = 80 \text{ plf}$$

$$P_{10} = 1139^\# + 4010^\# = 5149^\#$$

$$P_{11} = 4190^\#$$

$$P_{15} = 891^\# + 1830^\# = 2721^\#$$

$$P_{1W} = +/- 2785^\#$$

$$P_{1E} = +/- 3455^\#$$

RESULTS

$$M_{MAX} = 73.4 \text{ K-ft}$$

$$R_{MIN} = -2.56 \text{ K}$$

$$R_{MAX} = 14.43 \text{ K}$$

$$\Delta_{MAX} = 0.25" = \ell/648$$

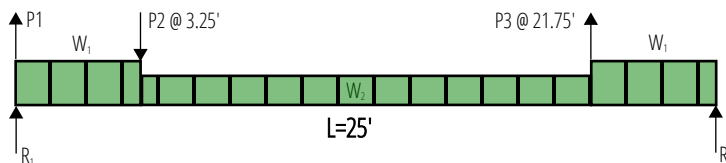
BEAM CAPACITIES

$$M = 80.85 \text{ K-ft}$$

$$R_{MAX} = 20.4 \text{ K}$$

SB2

5 1/2" x 21" GLB



$$W_{10} = (11.5/2)(32) + (16/12)(20) + (24/12)(15) + 100 = 525 \text{ plf}$$

$$W_{11} = (24/12)(40) = 80 \text{ plf}$$

$$W_{15} = (13/2)(25) = 163 \text{ plf}$$

$$W_{20} = (24/12)(15) = 30 \text{ plf}$$

$$W_{21} = (24/12)(40) = 80 \text{ plf}$$

$$P_{1W} = +/- 2785^\#$$

$$P_{1E} = +/- 3455^\#$$

$$P_{20} = 1500^\#$$

$$P_{25} = 1870^\#$$

$$P_{2W} = +/- 2785^\#$$

$$P_{2E} = +/- 3455^\#$$

$$P_{30} = 1500^\#$$

$$P_{35} = 1870^\#$$

$$P_{3W} = +/- 2785^\#$$

$$P_{3E} = +/- 3455^\#$$

RESULTS

$$M_{MAX} = 30.84 \text{ K-ft}$$

$$R_{MIN} = -3.59 \text{ K}$$

$$R_{MAX} = 10.77 \text{ K}$$

$$\Delta_{MAX} = 0.38" = \ell/789$$

BEAM CAPACITIES

$$M = 80.85 \text{ K-ft}$$

$$R_{MAX} = 20.4 \text{ K}$$

Zimmer Residence

Members

Name ↑	Node 1	Node 2	Shape	Material	End Connection	Length ft	Weight K
SB1	N001	N002	GL 5.5x21	Softwood Glulam 24F-E4 Douglas Fir (24F-1.8E)	Rigid Connect	13.5000	0.3556

Member Loads, Uniform

Member	Service Case	Direction	Magnitude	Full Length?	Start Offset ft	End Offset ft	Projected?	Predefined Load
SB1	D	Force Y	-0.3510 K/ft	No	0.0000	5.7500	No	N.A.
SB1	D	Shear y	-0.0300 K/ft	No	5.7500	13.5000	No	N.A.
SB1	L	Shear y	-0.0800 K/ft	Yes	0.0000	13.5000	No	N.A.
SB1	S	Shear y	-0.0380 K/ft	No	0.0000	5.7500	No	N.A.

Member Loads, Concentrated

Member	Service Case	Direction	Magnitude	Offset ft
SB1	D	Force Y	-5.1490 K	5.7500
SB1	E+X	Force Y	-3.4550 K	5.7500
SB1	E-X	Force Y	3.4550 K	5.7500
SB1	L	Force Y	-4.1900 K	5.7500
SB1	S	Force Y	-2.7210 K	5.7500
SB1	W+X	Force Y	-2.8750 K	5.7500
SB1	W-X	Force Y	2.7850 K	5.7500

Result Cases

Name	ID ↓	Design Checks	Result Type	Seismic Type	Effective Equation
D	1	No Design	Static	N.A.	D
L	2	No Design	Static	N.A.	L
S	3	No Design	Static	N.A.	S
W+X	4	No Design	Static	N.A.	W+X
W-X	5	No Design	Static	N.A.	W-X
E+X	6	No Design	Static	Normal	E+X
E-X	7	No Design	Static	Normal	E-X
D+L	8	Deflections	Static	N.A.	D + L
D+0.75(L+W) »+X	9	Deflections	Static	N.A.	D + 0.75L + 0.75W+X
D+0.75(L+W) »-X	10	Deflections	Static	N.A.	D + 0.75L + 0.75W-X
D+S	11	Deflections	Static	N.A.	D + S
D+Lr	12	Deflections	Static	N.A.	D
Live	13	Deflections	Static	N.A.	L
Snow	14	Deflections	Static	N.A.	S
Wind »+X	15	Deflections	Static	N.A.	W+X
Wind »-X	16	Deflections	Static	N.A.	W-X
Seismic »+X	17	Deflections	Static	Normal	E+X

Result Cases (continued)

Name	ID ↓	Design Checks	Result Type	Seismic Type	Effective Equation
Seismic »-X	18	Deflections	Static	Normal	E-X
1. D	19	Allowable (ASD)	Static	N.A.	D
2. D+L	20	Allowable (ASD)	Static	N.A.	D + L
3. D+S	21	Allowable (ASD)	Static	N.A.	D + S
4. D+0.75(L+Lr)	22	Allowable (ASD)	Static	N.A.	D + 0.75L
4. D+0.75(L+S)	23	Allowable (ASD)	Static	N.A.	D + 0.75L + 0.75S
5. D+0.6W »+X	24	Allowable (ASD)	Static	N.A.	D + 0.6W+X
5. D+0.6W »-X	25	Allowable (ASD)	Static	N.A.	D + 0.6W-X
6. D+0.75 (L+0.6W+Lr) »+X	26	Allowable (ASD)	Static	N.A.	D + 0.75L + 0.45W+X
6. D+0.75 (L+0.6W+Lr) »-X	27	Allowable (ASD)	Static	N.A.	D + 0.75L + 0.45W-X
6. D+0.75(L+0.6W+S) »+X	28	Allowable (ASD)	Static	N.A.	D + 0.75L + 0.75S + 0.45W+X
6. D+0.75(L+0.6W+S) »-X	29	Allowable (ASD)	Static	N.A.	D + 0.75L + 0.75S + 0.45W-X
7. 0.6D+0.6W »+X	30	Allowable (ASD)	Static	N.A.	0.6D + 0.6W+X
7. 0.6D+0.6W »-X	31	Allowable (ASD)	Static	N.A.	0.6D + 0.6W-X
7. 0.6D+0.7Di+0.7Wi	32	Allowable (ASD)	Static	N.A.	0.6D
8. D+0.7E »+X	33	Allowable (ASD)	Static	Normal	1.13454D + 0.7E+X
8. D+0.7E »+X:Ω	34	Allowable (ASD)	Static	Over-Strength	1.13454D + 2.1E+X
8. D+0.7E »-X	35	Allowable (ASD)	Static	Normal	1.13454D + 0.7E-X
8. D+0.7E »-X:Ω	36	Allowable (ASD)	Static	Over-Strength	1.13454D + 2.1E-X
9. D+0.75(0.7E+L+S) »+X	37	Allowable (ASD)	Static	Normal	1.100905D + 0.75L + 0.75S + 0.525E+X
9. D+0.75(0.7E+L+S) »+X:Ω	38	Allowable (ASD)	Static	Over-Strength	1.100905D + 0.75L + 0.75S + 1.575E+X
9. D+0.75(0.7E+L+S) »-X	39	Allowable (ASD)	Static	Normal	1.100905D + 0.75L + 0.75S + 0.525E-X
9. D+0.75(0.7E+L+S) »-X:Ω	40	Allowable (ASD)	Static	Over-Strength	1.100905D + 0.75L + 0.75S + 1.575E-X
10. 0.6D+0.7E »+X	41	Allowable (ASD)	Static	Normal	0.46546D + 0.7E+X
10. 0.6D+0.7E »+X:Ω	42	Allowable (ASD)	Static	Over-Strength	0.46546D + 2.1E+X
10. 0.6D+0.7E »-X	43	Allowable (ASD)	Static	Normal	0.46546D + 0.7E-X
10. 0.6D+0.7E »-X:Ω	44	Allowable (ASD)	Static	Over-Strength	0.46546D + 2.1E-X

Node Results

(extreme rows: max and min)

Node	Result Case	DX in	DY in	DZ in	FX K	FY K	FZ K	MX K-ft	MY K-ft	MZ K-ft
N001	9. D+0.75(0.7E+L+S) »+X:Ω	0.0000	0.0000	0.0000	0.0000	11.9056	0.0000	0.0000	0.0000	0.0000
N001	E-X	0.0000	0.0000	0.0000	0.0000	-1.9834	0.0000	0.0000	0.0000	0.0000
N001	Seismic »-X	0.0000	0.0000	0.0000	0.0000	-1.9834	0.0000	0.0000	0.0000	0.0000

Member Forces

Member	Fx Min K	Fx Max K	Vy K	Mz Min K-ft	Mz Max K-ft
SB1	0.0000 (44)	0.0000 (44)	11.9056 (38)	-13.9996 (44)	60.0139 (38)

Member Displacements

(extreme rows: max and min)

Member	Dy Min in	Dy Max in	Dz Min in	Dz Max in
SB1	-0.2098 (38)	0.0465 (44)	0.0000 (44)	0.0000 (44)

Wood_Beam X_G 1: Results

Configuration Has Bolt Holes: False Multi-piece Lams: False	Axial Manual Kz: False Kz Sidesway?: False Manual Ky: False Ky Sidesway?: False	Beam Beam Type: Simply Supported Loading Pattern: Point load at center w/o support Tension Flange On Bottom?: False Override Lbi?: False End Notch Type: No Notches Bearing length at start: 4 in Bearing length at end: 4 in Check bearing at start?: True Check bearing at end?: True Bearing force direction: Vy Shear @ d from start?: False Shear @ d from end?: False
Deflections - Strong (dy) Limit Type: Member Span Ratio L only: 480 W or S only: 360 D + L: 360 Other: 240	Deflections - Weak (dz) Limit Type: None	C-Factors Moisture Condition: Dry Service Condition Temp. Range: T <= 100F Wet Service Factor, CM: Calculate Temperature Factor, Ct: Calculate Incising Factor, Ci: Ignore Repetitive Member Factor, Cr: Ignore Buckling Stiffness Factor, CT: Ignore Bearing Area Factor, Cb: Ignore Slenderness Limit Increase: False Curvature Factor, Cc: 1 Pole/Pile Treatment: Air Dried Pole/Pile Group Count: 1
Overrides Override CF: False Override CI: False Override CL: False Override CP: False Override CV: False Override Ccs: False Override Cct: False Override Cfu: False Override Cls: False Override Cvr: False	Material Overrides Override Ft (Poles & Piles): False Override Fcp: False Neglect Size Constraints?: False	
Wood Material: Softwood Glulam 24F-E4 Douglas Fir (24F-1.8E) Specification: NDS 2018 ASD Overstrength?: True Live Load Reduction: None Disable Checks?: False Check Level: Each Limit State	Bracing Lateral Top (+y): Continuous Lateral Bottom (-y): Unbraced Strong (z): Unbraced	Size Constraints Limit Depth?: False Limit Width?: False

Wood_Beam X_G 1: Strong Deflection Check

(extreme rows: max)

Member	Section	Offset ft	Result Case	Demand dy in	Capacity dy in	Code Reference	Unity Check	Details
SB1	GL 5.5x21	6.4125	D+L	-0.1306	0.4500	IBC 1604.3.1	0.2902	CM(E) = 1, Ct = 1, L/Δ = 1240.6

Wood_Beam X_G 1: Strong Flexure Check

(extreme rows: max)

Member	Section	Offset ft	Result Case	Demand fbz Ksi	Capacity Fbz Ksi	Code Reference	Unity Check	Details
SB1	GL 5.5x21	5.7375	9. D+0.75(0.7E+L+S) »+X:Ω	1.7815	3.7683	3.3-1	0.4728	Lb = 0 ft, Mz = 60.014 K-ft, E'min = 900 Ksi, CM(E) = 1, Ct = 1, CM(Fb) = 1, Cfu = 1, Cc = 1, CI = 1, CV = 0.98134, CD = 1.6

Wood_Beam X_G 1: Strong Shear Check

(extreme rows: max)

Member	Section	Offset ft	Result Case	Demand fvy Ksi	Capacity Fvy Ksi	Code Reference	Unity Check	Details
SB1	GL 5.5x21	0.3333	2. D+L	0.0985	0.2650	3.4-1	0.3716	V = 7.5818 K, CM = 1, Ct = 1, Cvr = 1, CD = 1

Wood_Beam X_G 1: Wood Bearing Check

(extreme rows: max)

Member	Section	Offset ft	Result Case	Demand fcp Ksi	Capacity Fcp Ksi	Code Reference	Unity Check	Details
SB1	GL 5.5x21	0.0000	9. D+0.75(0.7E+L+S) »+X:Ω	0.5412	0.6500	3.10.2	0.8326	Bearing Area = 22 in ² , Bearing Length = 4 in, Bearing Width = 5.5 in, CM = 1, Ct = 1, Cb = 1

Members

Name ↑	Node 1	Node 2	Shape	Material	End Connection	Length ft	Weight K
SB2	N001	N002	GL 5.5x21	Softwood Glulam 24F-E4 Douglas Fir (24F-1.8E)	Rigid Connect	25.0000	0.6586

Member Loads, Uniform

Member	Service Case	Direction	Magnitude	Full Length?	Start Offset ft	End Offset ft	Projected?	Predefined Load
SB2	D	Force Y	-0.5250 K/ft	No	0.0000	3.2500	No	N.A.
SB2	D	Force Y	-0.0300 K/ft	No	3.2500	21.7500	No	N.A.
SB2	D	Shear y	-0.5250 K/ft	No	21.7500	25.0000	No	N.A.
SB2	L	Shear y	-0.0800 K/ft	Yes	0.0000	25.0000	No	N.A.
SB2	S	Force Y	-0.1630 K/ft	No	21.7500	25.0000	No	N.A.
SB2	S	Shear y	-0.1630 K/ft	No	0.0000	3.2500	No	N.A.

Member Loads, Concentrated

Member	Service Case	Direction	Magnitude	Offset ft
SB2	D	Force Y	-1.5000 K	3.2500
SB2	D	Force Y	-1.5000 K	21.7500
SB2	E+X	Force Y	-3.4550 K	0.0000
SB2	E+X	Force Y	-3.4550 K	21.7500
SB2	E+X	Force Y	3.4550 K	3.2500
SB2	E-X	Force Y	-3.4550 K	3.2500
SB2	E-X	Force Y	3.4550 K	0.0000
SB2	E-X	Force Y	3.4550 K	21.7500
SB2	S	Force Y	-1.8700 K	3.2500
SB2	S	Force Y	-1.8700 K	21.7500
SB2	W+X	Force Y	-2.8750 K	0.0000
SB2	W+X	Force Y	-2.8750 K	21.7500
SB2	W+X	Force Y	2.8750 K	3.2500
SB2	W-X	Force Y	-2.8750 K	3.2500
SB2	W-X	Force Y	2.8750 K	0.0000
SB2	W-X	Force Y	2.8750 K	21.7500

Result Cases

Name	ID ↓	Design Checks	Result Type	Seismic Type	Effective Equation
D	1	No Design	Static	N.A.	D
L	2	No Design	Static	N.A.	L
S	3	No Design	Static	N.A.	S
W+X	4	No Design	Static	N.A.	W+X
W-X	5	No Design	Static	N.A.	W-X
E+X	6	No Design	Static	Normal	E+X

Result Cases (continued)

Name	ID ↓	Design Checks	Result Type	Seismic Type	Effective Equation
E-X	7	No Design	Static	Normal	E-X
D+L	8	Deflections	Static	N.A.	D + L
D+0.75(L+W) »+X	9	Deflections	Static	N.A.	D + 0.75L + 0.75W+X
D+0.75(L+W) »-X	10	Deflections	Static	N.A.	D + 0.75L + 0.75W-X
D+S	11	Deflections	Static	N.A.	D + S
D+Lr	12	Deflections	Static	N.A.	D
Live	13	Deflections	Static	N.A.	L
Snow	14	Deflections	Static	N.A.	S
Wind »+X	15	Deflections	Static	N.A.	W+X
Wind »-X	16	Deflections	Static	N.A.	W-X
Seismic »+X	17	Deflections	Static	Normal	E+X
Seismic »-X	18	Deflections	Static	Normal	E-X
1. D	19	Allowable (ASD)	Static	N.A.	D
2. D+L	20	Allowable (ASD)	Static	N.A.	D + L
3. D+S	21	Allowable (ASD)	Static	N.A.	D + S
4. D+0.75(L+Lr)	22	Allowable (ASD)	Static	N.A.	D + 0.75L
4. D+0.75(L+S)	23	Allowable (ASD)	Static	N.A.	D + 0.75L + 0.75S
5. D+0.6W »+X	24	Allowable (ASD)	Static	N.A.	D + 0.6W+X
5. D+0.6W »-X	25	Allowable (ASD)	Static	N.A.	D + 0.6W-X
6. D+0.75 (L+0.6W+Lr) »+X	26	Allowable (ASD)	Static	N.A.	D + 0.75L + 0.45W+X
6. D+0.75 (L+0.6W+Lr) »-X	27	Allowable (ASD)	Static	N.A.	D + 0.75L + 0.45W-X
6. D+0.75(L+0.6W+S) »+X	28	Allowable (ASD)	Static	N.A.	D + 0.75L + 0.75S + 0.45W+X
6. D+0.75(L+0.6W+S) »-X	29	Allowable (ASD)	Static	N.A.	D + 0.75L + 0.75S + 0.45W-X
7. 0.6D+0.6W »+X	30	Allowable (ASD)	Static	N.A.	0.6D + 0.6W+X
7. 0.6D+0.6W »-X	31	Allowable (ASD)	Static	N.A.	0.6D + 0.6W-X
7. 0.6D+0.7Di+0.7Wi	32	Allowable (ASD)	Static	N.A.	0.6D
8. D+0.7E »+X	33	Allowable (ASD)	Static	Normal	1.13454D + 0.7E+X
8. D+0.7E »+X:Ω	34	Allowable (ASD)	Static	Over-Strength	1.13454D + 2.1E+X
8. D+0.7E »-X	35	Allowable (ASD)	Static	Normal	1.13454D + 0.7E-X
8. D+0.7E »-X:Ω	36	Allowable (ASD)	Static	Over-Strength	1.13454D + 2.1E-X
9. D+0.75(0.7E+L+S) »+X	37	Allowable (ASD)	Static	Normal	1.100905D + 0.75L + 0.75S + 0.525E+X
9. D+0.75(0.7E+L+S) »+X:Ω	38	Allowable (ASD)	Static	Over-Strength	1.100905D + 0.75L + 0.75S + 1.575E+X
9. D+0.75(0.7E+L+S) »-X	39	Allowable (ASD)	Static	Normal	1.100905D + 0.75L + 0.75S + 0.525E-X
9. D+0.75(0.7E+L+S) »-X:Ω	40	Allowable (ASD)	Static	Over-Strength	1.100905D + 0.75L + 0.75S + 1.575E-X
10. 0.6D+0.7E »+X	41	Allowable (ASD)	Static	Normal	0.46546D + 0.7E+X
10. 0.6D+0.7E »+X:Ω	42	Allowable (ASD)	Static	Over-Strength	0.46546D + 2.1E+X
10. 0.6D+0.7E »-X	43	Allowable (ASD)	Static	Normal	0.46546D + 0.7E-X

Result Cases (continued)

Name	ID	Design Checks	Result Type	Seismic Type	Effective Equation
10. 0.6D+0.7E »-X:Ω	44	Allowable (ASD)	Static	Over-Strength	0.46546D + 2.1E-X

Node Results

(extreme rows: max and min)

Node	Result Case	DX in	DY in	DZ in	FX K	FY K	FZ K	MX K-ft	MY K-ft	MZ K-ft
N002	9. D+0.75(0.7E+L+S) »+X:Ω	0.0000	0.0000	0.0000	0.0000	10.7744	0.0000	0.0000	0.0000	0.0000
N002	10. 0.6D+0.7E »-X:Ω	0.0000	0.0000	0.0000	0.0000	-3.5943	0.0000	0.0000	0.0000	0.0000

Member Forces

Member	Fx Min K	Fx Max K	Vy K	Mz Min K-ft	Mz Max K-ft
SB2	0.0000 (44)	0.0000 (44)	-10.7744 (38)	-13.0367 (44)	30.8487 (38)

Member Displacements

(extreme rows: max and min)

Member	Dy Min in	Dy Max in	Dz Min in	Dz Max in
SB2	-0.3790 (40)	0.0229 (18)	0.0000 (44)	0.0000 (44)

Wood_Beam X_G 1: Results

Configuration Has Bolt Holes: False Multi-piece Lams: False	Axial Manual Kz: False Kz Sidesway?: False Manual Ky: False Ky Sidesway?: False	Beam Beam Type: Simply Supported Loading Pattern: Uniform Tension Flange On Bottom?: False Override Lbi?: False End Notch Type: No Notches Bearing length at start: 5.5 in Bearing length at end: 5.5 in Check bearing at start?: True Check bearing at end?: True Bearing force direction: Vy Shear @ d from start?: False Shear @ d from end?: False
Deflections - Strong (dy) Limit Type: Member Span Ratio L only: 480 W or S only: 360 D + L: 360 Other: 240	Deflections - Weak (dz) Limit Type: None	C-Factors Moisture Condition: Dry Service Condition Temp. Range: T <= 100F Wet Service Factor, CM: Calculate Temperature Factor, Ct: Calculate Incising Factor, Ci: Ignore Repetitive Member Factor, Cr: Ignore Buckling Stiffness Factor, CT: Ignore Bearing Area Factor, Cb: Ignore Slenderness Limit Increase: False Curvature Factor, Cc: 1 Pole/Pile Treatment: Air Dried Pole/Pile Group Count: 1

Wood_Beam X_G 1: Results (continued)

Overrides Override CF: False Override CI: False Override CL: False Override CP: False Override CV: False Override Ccs: False Override Cct: False Override Cfu: False Override Cls: False Override Cvr: False	Material Overrides Override Ft (Poles & Piles): False Override Fcp: False Neglect Size Constraints?: False	
Wood Material: Softwood Glulam 24F-E4 Douglas Fir (24F-1.8E) Specification: NDS 2018 ASD Overstrength?: True Live Load Reduction: None Disable Checks?: False Check Level: Each Limit State	Bracing Lateral Top (+y): Continuous Lateral Bottom (-y): Unbraced Strong (z): Unbraced	Size Constraints Limit Depth?: False Limit Width?: False

Wood_Beam X_G 1: Strong Deflection Check

(extreme rows: max)

Member	Section	Offset ft	Result Case	Demand dy in	Capacity dy in	Code Reference	Unity Check	Details
SB2	GL 5.5x21	12.5000	D+L	-0.2867	0.8333	IBC 1604.3.1	0.3441	CM(E) = 1, Ct = 1, L/Δ = 1046.4

Wood_Beam X_G 1: Strong Flexure Check

(extreme rows: max)

Member	Section	Offset ft	Result Case	Demand fbz Ksi	Capacity Fbz Ksi	Code Reference	Unity Check	Details
SB2	GL 5.5x21	21.7500	9. D+0.75(0.7E+L+S) »+X:Ω	0.9157	3.5431	3.3-1	0.2585	Lb = 0 ft, Mz = 30.849 K-ft, E'min = 900 Ksi, CM(E) = 1, Ct = 1, CM(Fb) = 1, Cfu = 1, Cc = 1, CI = 1, CV = 0.92269, CD = 1.6

Wood_Beam X_G 1: Strong Shear Check

(extreme rows: max)

Member	Section	Offset ft	Result Case	Demand fvy Ksi	Capacity Fvy Ksi	Code Reference	Unity Check	Details
SB2	GL 5.5x21	24.5417	9. D+0.75(0.7E+L+S) »+X:Ω	-0.1352	0.4240	3.4-1	0.3189	V = -10.413 K, CM = 1, Ct = 1, Cvr = 1, CD = 1.6

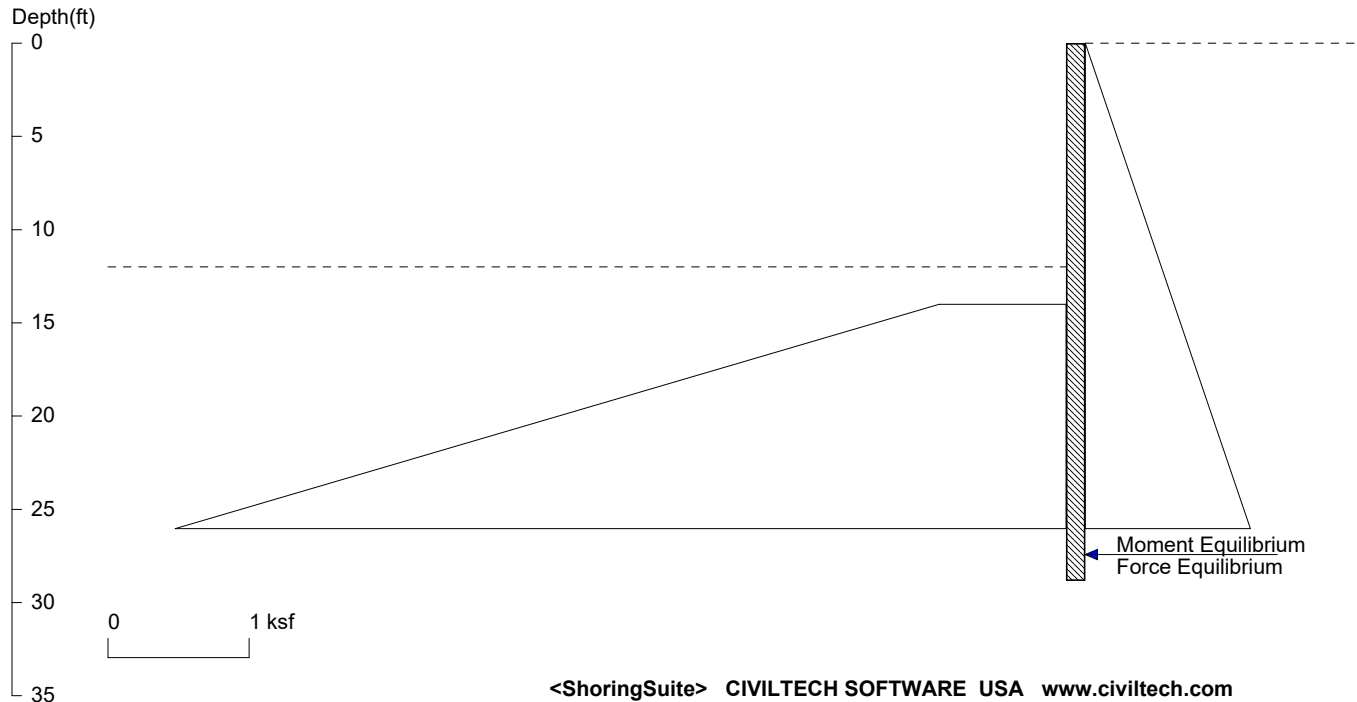
Wood_Beam X_G 1: Wood Bearing Check

(extreme rows: max)

Member	Section	Offset ft	Result Case	Demand fcp Ksi	Capacity Fcp Ksi	Code Reference	Unity Check	Details
SB2	GL 5.5x21	25.0000	9. D+0.75(0.7E+L+S) »+X:Ω	0.3562	0.6500	3.10.2	0.5480	Bearing Area = 30.25 in^2, Bearing Length = 5.5 in, Bearing Width = 5.5 in, CM = 1, Ct = 1, Cb = 1

Zimmer Res

12' Cantilever - Auto Court Normal Condition



Licensed to 4324324234 3424343

Date: 11/13/2024

File: H:\Projects\2024\Zimmer Residence\Calculations\Shoring\Zimmer - Autocourt 12 Cantilever - Normal.sh8

Wall Height=12.0 Pile Diameter=2.5 Pile Spacing=8.0 Wall Type: 2. Soldier Pile, Drilled

PILE LENGTH: Min. Embedment=16.83 Min. Pile Length=28.83

MOMENT IN PILE: Max. Moment=256.99 per Pile Spacing=8.0 at Depth=19.52

PILE SELECTION:

Request Min. Section Modulus = 129.8 in³/pile=2126.91 cm³/pile, F_y= 36 ksi = 248 MPa, F_b/F_y=0.66

W14X90 has Section Modulus = 143.0 in³/pile=2343.34 cm³/pile. It is greater than Min. Requirements!

Top Deflection = 1.05(in) based on E (ksi)=29000.00 and I (in⁴)/pile=999.0

DRIVING PRESSURES (ACTIVE, WATER, & SURCHARGE):

Z1	P1	Z2	P2	Slope
0	0	34	1.530	.045

PASSIVE PRESSURES: Pressures below will be divided by a Factor of Safety =1.5

Z1	P1	Z2	P2	Slope
14	.9	34	9.900	.45

ACTIVE SPACING:

No.	Z depth	Spacing
1	0.00	8.00
2	12.00	2.50

PASSIVE SPACING:

No.	Z depth	Spacing
1	12.00	5.00

UNITS: Width,Spacing,Diameter,Length,and Depth - ft; Force - kip; Moment - kip-ft
Friction,Bearing,and Pressure - ksf; Pres. Slope - kip/ft³; Deflection - in

Depth (ft)

0

5

10

15

20

25

30

35

0 1 ksf

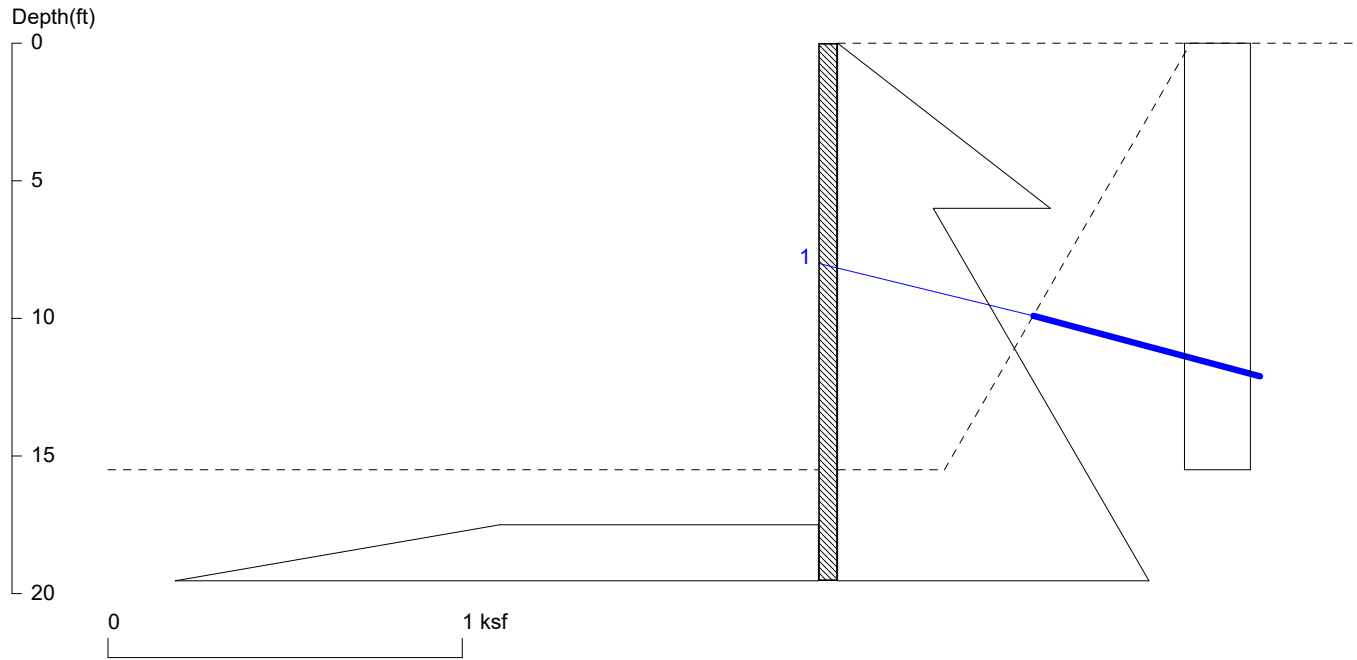
Moment Equilibrium

Force Equilibrium

<ShoringSuite> CIVIL TECH SOFTWARE, USA www.civiltech.com

Zimmer Res

9.5' - Auto Court Dynamic Slide Condition - Seismic



<ShoringSuite> CIVILTECH SOFTWARE USA www.civiltech.com

Licensed to 4324324234 3424343

Date: 11/10/2024

File: H:\Projects\2024\Zimmer Residence\Calculations\Zimmer - Autocourt 15_5 Impact_Seismic.sh8

Wall Height=15.5

Pile Diameter=2.5

Pile Spacing=6.5

Wall Type: 2. Soldier Pile, Drilled

PILE LENGTH: Min. Embedment=4.04 (5~10ft is recommended!!!!) Min. Pile Length=19.54

MOMENT IN PILE: Max. Moment=89.37 per Pile Spacing=6.5 at Depth=8.00

PILE SELECTION:

Request Min. Section Modulus = 45.1 in³/pile=739.65 cm³/pile, $F_y = 36 \text{ ksi} = 248 \text{ MPa}$, $F_b/F_y = 0.66$

W12X53 has Section Modulus = 70.6 in³/pile=1156.92 cm³/pile. It is greater than Min. Requirements!

Top Deflection = 0.30(in) based on $E \text{ (ksi)} = 29000.00$ and $I \text{ (in}^4\text{)}/\text{pile} = 425.0$

BRACE FORCE: Strut, Tieback, Plate Anchor, Deadman, Sheet Pile as Anchor

No. & Type	Depth	Angle	Space	Total F.	Horiz. F.	Vert. F.	L_free	Fixed Length
1. Tieback	8.0	15.0	6.3	56.7	54.8	14.7	7.4	28.4

UNITS: Width,Diameter,Spacing,Length,Depth,and Height - ft; Force - kip; Bond Strength and Pressure - ksf

DRIVING PRESSURES (ACTIVE, WATER, & SURCHARGE):

Z1	P1	Z2	P2	Slope
0	0	6	0.600	.1
6	.27	37.5	1.688	.045
*	Seismic	Load	= 12H	
0	.186	15.5	0.186	0.000000

PASSIVE PRESSURES: Pressures below will be divided by a Factor of Safety =1.15

Z1	P1	Z2	P2	Slope
17.5	.9	37.5	9.900	.45

ACTIVE SPACING:

No.	Z depth	Spacing
1	0.00	6.50
2	15.50	2.50

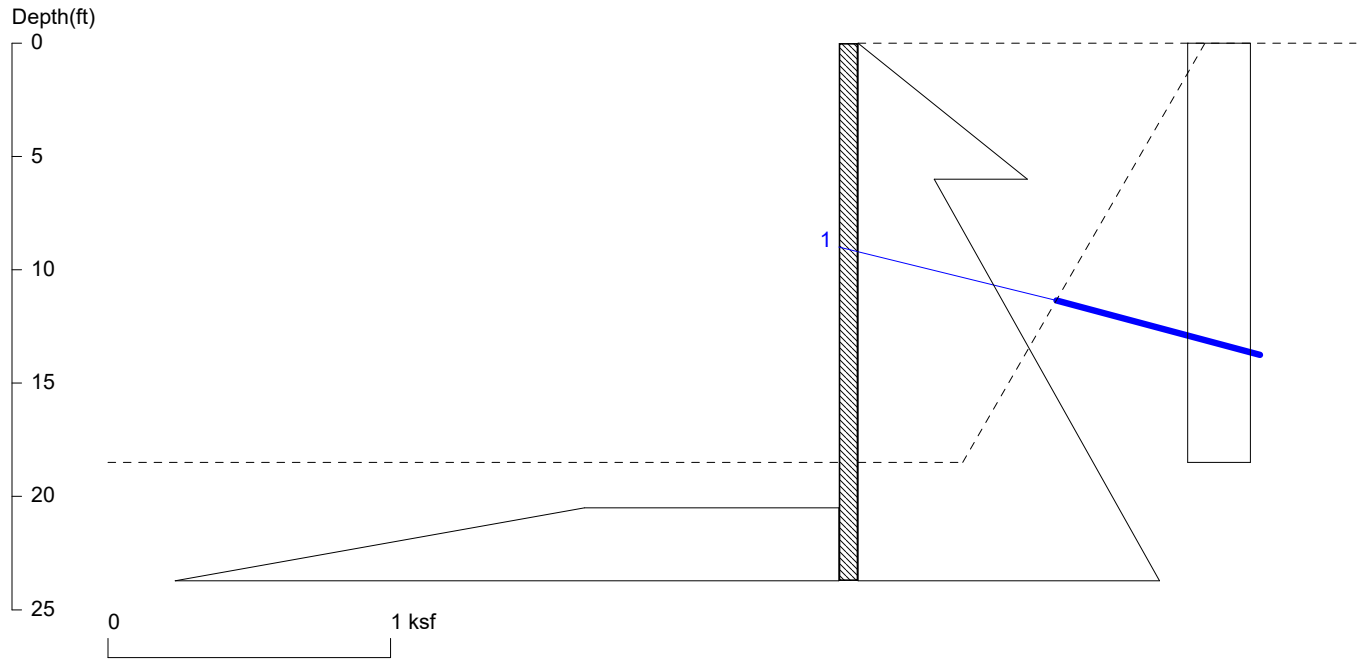
PASSIVE SPACING:

No.	Z depth	Spacing
1	15.50	5.00

UNITS: Width,Spacing,Diameter,Length,and Depth - ft; Force - kip; Moment - kip-ft
Friction,Bearing,and Pressure - ksf; Pres. Slope - kip/ft³; Deflection - in

Zimmer Res

12.5' - Auto Court Dynamic Slide Condition - Seism



<ShoringSuite> CIVILTECH SOFTWARE USA www.civiltech.com

Licensed to 4324324234 3424343

Date: 11/13/2024

File: H:\Projects\2024\Zimmer Residence\Calculations\Shoring\Zimmer - Autocourt 18_5 Impact_Seismic.sh8

Wall Height=18.5

Pile Diameter=2.5

Pile Spacing=7.0

Wall Type: 2. Soldier Pile, Drilled

PILE LENGTH: Min. Embedment=5.22 Min. Pile Length=23.72

MOMENT IN PILE: Max. Moment=135.84 per Pile Spacing=7.0 at Depth=9.00

PILE SELECTION:

Request Min. Section Modulus = 68.6 in³/pile=1124.22 cm³/pile, F_y= 36 ksi = 248 MPa, F_b/F_y=0.66

W12X53 has Section Modulus = 70.6 in³/pile=1156.92 cm³/pile. It is greater than Min. Requirements!

Top Deflection = 0.43(in) based on E (ksi)=29000.00 and I (in⁴)/pile=425.0

BRACE FORCE: Strut, Tieback, Plate Anchor, Deadman, Sheet Pile as Anchor

No. & Type	Depth	Angle	Space	Total F.	Horiz. F.	Vert. F.	L _{free}	Fixed Length
1. Tieback	9.0	15.0	6.3	74.2	71.6	19.2	9.1	37.2

UNITS: Width,Diameter,Spacing,Length,Depth,and Height - ft; Force - kip; Bond Strength and Pressure - ksf

DRIVING PRESSURES (ACTIVE, WATER, & SURCHARGE):

Z1	P1	Z2	P2	Slope
0	0	6	0.600	.1
6	.27	40.5	1.823	.045
*	Seism	Load	= 12H	
0	.222	18.5	0.222	0.000000

PASSIVE PRESSURES: Pressures below will be divided by a Factor of Safety =1.15

Z1	P1	Z2	P2	Slope
20.5	.9	40.5	9.900	.45

ACTIVE SPACING:

No.	Z depth	Spacing
1	0.00	7.00
2	18.50	2.50

PASSIVE SPACING:

No.	Z depth	Spacing
1	18.50	5.00

UNITS: Width,Spacing,Diameter,Length,and Depth - ft; Force - kip; Moment - kip-ft
Friction,Bearing,and Pressure - ksf; Pres. Slope - kip/ft³; Deflection - in