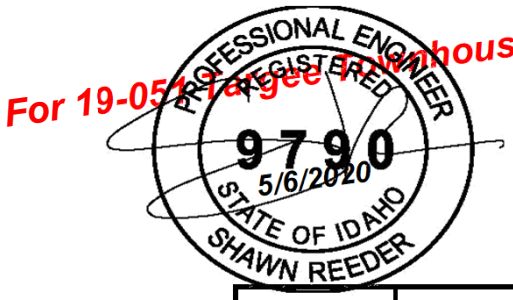
Lateral and Knee Brace Calculations

For

19-051 Woodlawn Townhomes

Boise, Idaho

2020-09655



For 19-051 Targee Townhousomes only

Expiration:	*5/6/2021*

Design Criteria

Governing Code: *2018 IBC*

Snow Criteria

Roof Load (P_f)	25 psf	
Ground Load (P_g)	25 psf	
Exposure Factor (C_e)	1.0	Partially
Thermal Factor (C_t)	1.0	Typical
Importance (I_s)	1.0	

Wind Criteria

Wind Speed (V_3)	115 mph	
Wind Exposure	B	Urban / wooded
Wind Importance (I_w)	1.0	
Building Category	II	

Seismic Criteria

Site Class	D	Stiff Soil	
Ss	0.31	Fa	1.56
S1	0.10	Fv	2.38
S_{DS}	0.32	S_{D1}	0.17
Risk Category	II	Other	
Seismic Importance (I_E)	1.0		
Seismic Design Category (SDC)	C		

Wall Material	Design Base Shear	Seismic Response Coefficient, R	
OSB	.05Wp	6.5	Typ @ Ext
GYP	.16Wp	2	Typ @ Int

Live Loads

Typ Residential	40 psf
Garage (P.V.)	40 psf
Sleeping Areas's	30 psf

Soil Bearing

Typical	1500 psf

Roof Dead Loads:

Deck	*1.5*
Insulation	*2.0*
Roofing	*3.0*
Joist	*2.5*
Ceiling	*3.0*
Misc	*4.5*
TOTAL	**17 psf**

Floor Dead Loads:

Deck	2.5
Joist	2.0
Ceiling	2.0
Flooring	2.5
Misc	3.0
TOTAL	**12 psf**

Exterior Wall Dead Loads:

Studs	*2.0*
Siding	*2.5*
Insulation	*0.5*
Gyp. Board	*2.5*
Sheating	*1.5*
Misc	*3.0*
TOTAL	**12 psf**

Interior Wall Dead Loads:

Studs	*2.0*
Gyp. Board	*2.5*
-	-
-	-
-	-
Misc	*3.0*
TOTAL	**8 psf**

OSB Seismic Loading Analysis

S_s =	*0.305*	C_T =	*0.020*	
S_1 =	*0.104*	h_n =	*28.25*	ft
F_a =	*1.6*			
F_v =	*2.4*			
R =	*6.5*			
I_E =	*1.0*			

$S_{MS} = F_a S_s$ =	0.4746	
$S_{M1} = F_v S_1$ =	0.2479	
		Seismic Design Category
$S_{DS} = 2/3\ S_{MS}$ =	0.3164	*B*
$S_{D1} = 2/3\ S_{M1}$ =	0.1653	*C*
$C_s = 1.2*S_{DS}/(R/I_E)$ =	0.0487	**Controls**
$T_a = C_T h_n^{3/4}$ =	0.2451	
$C_s < S_{D1}/[(R/I_E)T]$ =	0.1038	
$C_s > 0.044 S_{DS} I_E$ =	0.0139	
$C_s > 0.5 S_1/(R/I_E)$ =	0.0080	
$V = C_s W$ =	**0.0487** W	
0.7*V =	**0.0341** W	

OSB Seismic Component Loading

w_p =	*1*	psf	weight of element
V_{px} =	*0*	plf	Portion of seismic shear load at the level of the diaphragm, required to be transferred to the components of the vertical seismic-force-resisting system beacause of the offsets or changes in the stiffness of the vertical components above of below the diaphragm.
w_w =	*1*	psf	weight of wall
L_b =	*102*	ft	length of the building

NOTE: Use 1 for unit weight to achieve an answer per element unit weight

Connections

$F_p = 0.133\ S_{DS}\ w_p$ =	**0.04**	psf		
or				
$F_p = 0.05\ w_p$ =	**0.05**	psf	**Controls**	

Diaphragm

$F_p = 0.2\ I_E\ S_{DS}\ w_p + V_{px}$ =	**0.06**	psf		
$F_{p,max} = 0.4\ I_E\ S_{DS}\ w_p + V_{px}$ =	**0.13**	psf		
Bearing Walls & Shear Walls				
Out of Plane Forces				
$F_p = 0.40\ I_E\ S_{DS}\ w_w$ =	**0.13**	psf	**Controls**	*12.11.1*
$F_p = 0.10\ w_w$ =	**0.10**	psf		*12.11.1*

Anchorage

$F_p = 0.40\ I_E\ S_{DS}\ w_w\ k_a$ =	**0.3**	psf		*12.11-1*
$F_p = 0.2\ I_E\ k_a\ w_w$ =	**0.4040**	psf	**Controls**	
$k_a = 1.0 + L_b / 100$ =	**2.0200**			*12.11-2*

Note: *12.11.2.2.2* The strength design forces for steel elements of the structural wall anchorage system, with exception of anchor bolts and reinforcing steel, shall be increased by 1.4 times the forces otherwise noted above.

GYP Seismic Loading Analysis

S_s =	*0.305*	C_T =	*0.020*	
S_1 =	*0.104*	h_n =	*28.25*	ft
F_a =	*1.6*			
F_v =	*2.4*			
R =	*2.0*			
I_E =	*1.0*			

$S_{MS} = F_a S_s$ =	0.4746	
$S_{M1} = F_v S_1$ =	0.2479	
		Seismic Design Category
$S_{DS} = 2/3\ S_{MS}$ =	0.3164	*B*
$S_{D1} = 2/3\ S_{M1}$ =	0.1653	*C*
$C_s = 1.2*S_{DS}/(R/I_E)$ =	0.1582	**Controls**
$T_a = C_T h_n^{3/4}$ =	0.2451	
$C_s < S_{D1}/[(R/I_E)T]$ =	0.3372	
$C_s > 0.044 S_{DS} I_E$ =	0.0139	
$C_s > 0.5 S_1/(R/I_E)$ =	0.0260	
$V = C_s W$ =	**0.1582** W	
0.7*V =	**0.1107** W	

GYP Seismic Component Loading

w_p =	*1*	psf	weight of element
V_{px} =	*0*	plf	Portion of seismic shear load at the level of the diaphragm, required to be transferred to the components of the vertical seismic-force-resisting system beacause of the offsets or changes in the stiffness of the vertical components above of below the diaphragm.
w_w =	*1*	psf	weight of wall
L_b =	*102*	ft	length of the building

NOTE: Use 1 for unit weight to achieve an answer per element unit weight

Connections

$F_p = 0.133\ S_{DS}\ w_p$ =	**0.04**	psf		
or				
$F_p = 0.05\ w_p$ =	**0.05**	psf		

Diaphragm

$F_p = 0.2\ I_E\ S_{DS}\ w_p + V_{px}$ =	**0.06**	psf		

Bearing Walls & Shear Walls

Out of Plane Forces

$F_p = 0.40\ I_E\ S_{DS}\ w_w$ =	**0.13**	psf	**Controls**	*12.11.1*
$F_p = 0.10\ w_w$ =	**0.10**	psf		*12.11.1*

Anchorage

$F_p = 0.40\ I_E\ S_{DS}\ w_w\ k_a$ =	**0.3**	psf		*12.11-1*
$F_p = 0.2\ I_E\ k_a\ w_w$ =	**0.4040**	psf	**Controls**	
$k_a = 1.0 + L_b / 100$ =	**2.0200**			*12.11-2*

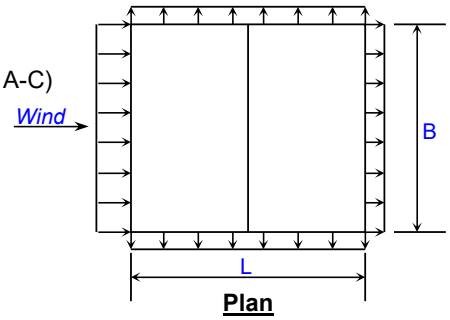
Note: *12.11.2.2.2* The strength design forces for steel elements of the structural wall anchorage system, with exception of anchor bolts and reinforcing steel, shall be increased by 1.4 times the forces otherwise noted above.

WIND LOADING ANALYSIS - Main Wind-Force Resisting System

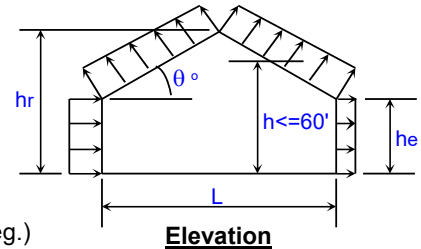
Per ASCE 7-16 Code for Enclosed or Partially Enclosed Buildings
Using Method 2: Analytical Procedure (Section 27 & 28) for Low-Rise Buildings

Input Data:

Wind Speed, V =	115	mph (Wind Map, Figure 26.5-1A-C)
Bldg. Classification =	II	(Table 1.5-1 Risk Category)
Exposure Category =	B	(Sect. 26.7)
Ridge Height, hr =	28.25	ft. (hr >= he)
Eave Height, he =	20.75	ft. (he <= hr)
Building Width =	46.00	ft. (Normal to Building Ridge)
Building Length =	102.00	ft. (Parallel to Building Ridge)
Roof Type =	Gable	(Gable or Monoslope)
Topo. Factor, Kzt =	1.00	(Sect. 26.8 & Figure 26.8-1)
Direct. Factor, Kd =	0.85	(Table 26.6)
Enclosed? (Y/N)	Y	(Sect. 26.2 & Table 26.11-1)
Hurricane Region?	N	



Plan



Elevation

Resulting Parameters and Coefficients:

Roof Angle, θ =	18.06	deg.
Mean Roof Ht., h =	24.50	ft. (h = (hr+he)/2, for angle >10 deg.)

Check Criteria for a Low-Rise Building:
1. Is h <= 60' ? Yes, O.K. 2. Is h <= Lesser of L or B? Yes, O.K.

External Pressure Coeff's., GCpf (Fig. 28.4-1):
(For values, see following wind load tabulations.)
Positive & Negative Internal Pressure Coefficients, GCpi (Table 26.11-1):

+GCpi Coef. =	0.18	(positive internal pressure)
-GCpi Coef. =	-0.18	(negative internal pressure)

If h < 15 then: Kh = 2.01*(15/zg)^(2/α) (Table 28.3-1)
If h >= 15 then: Kh = 2.01*(z/zg)^(2/α) (Table 28.3-1)

α =	7.00	(Table 26.9-1)
zg =	1200	(Table 26.9-1)
Kh =	0.70	(Kh = Kz evaluated at z = h)

Velocity Pressure: qz = 0.00256*Kz*Kzt*Kd*V^2 (Sect. 28.3.2, Eq. 28.3-1)
qh = 20.16 psf qh = 0.00256*Kh*Kzt*Kd*V^2 (qz evaluated at z = h)
Design Net External Wind Pressures (Sect. 28.4.1):
p = qh*[(GCpf) - (+/-GCpi)] (psf, Eq. 28.4-1)

Wall and Roof End Zone Widths 'a' and '2*a' (Fig. 28.4-1):

a =	4.60	ft.
2*a =	9.20	ft.

MWFRS Wind Load for Load Case A				MWFRS Wind Load for Load Case B			
Surface	GCpf	p = Net Pressures (psf)		Surface	*GCpf	p = Net Pressures (psf)	
		(w/ +GCpi)	(w/ -GCpi)			(w/ +GCpi)	(w/ -GCpi)
Zone 1	0.51	6.72	13.98	Zone 1	0.40	4.44	11.69
Zone 2	-0.69	-17.54	-10.28	Zone 2	-0.69	-17.54	-10.28
Zone 3	-0.47	-13.02	-5.76	Zone 3	-0.37	-11.09	-3.83
Zone 4	-0.41	-11.93	-4.68	Zone 4	-0.29	-9.48	-2.22
Zone 5	---	---	---	Zone 5	-0.45	-12.70	-5.44
Zone 6	---	---	---	Zone 6	-0.45	-12.70	-5.44
Zone 1E	0.78	12.00	19.26	Zone 1E	0.61	8.67	15.93
Zone 2E	-1.07	-25.20	-17.94	Zone 2E	-1.07	-25.20	-17.94
Zone 3E	-0.67	-17.12	-9.87	Zone 3E	-0.53	-14.31	-7.06
Zone 4E	-0.61	-15.98	-8.73	Zone 4E	-0.43	-12.30	-5.04
Zone 5E	---	---	---	Zone 5E	0.61	8.67	15.93
Zone 6E	---	---	---	Zone 6E	-0.43	-12.30	-5.04

*Note: Use roof angle θ = 0 degrees for Longitudinal Direction.

For Case A when GCpf is neg. in Zones 2/2E:
Zones 2/2E dist. = 23.00 ft.

For Case B when GCpf is neg. in Zones 2/2E:
Zones 2/2E dist. = 51.00 ft.

Remainder of roof Zones 2/2E extending to ridge line shall use roof Zones 3/3E pressure coefficients.

MWFRS Wind Load for Load Case A, Torsional Case				MWFRS Wind Load for Case B, Torsional Case			
Surface	GCpf	p = Net Pressure (psf)		Surface	GCpf	p = Net Pressure (psf)	
		(w/ +GCpi)	(w/ -GCpi)			(w/ +GCpi)	(w/ -GCpi)
Zone 1T	---	1.68	3.49	Zone 1T	---	1.11	2.92
Zone 2T	---	-4.39	-2.57	Zone 2T	---	-4.39	-2.57
Zone 3T	---	-3.25	-1.44	Zone 3T	---	-2.77	-0.96
Zone 4T	---	-2.98	-1.17	Zone 4T	---	-2.37	-0.55
Zone 5T	---	---	---	Zone 5T	---	-3.18	-1.36
Zone 6T	---	---	---	Zone 6T	---	-3.18	-1.36

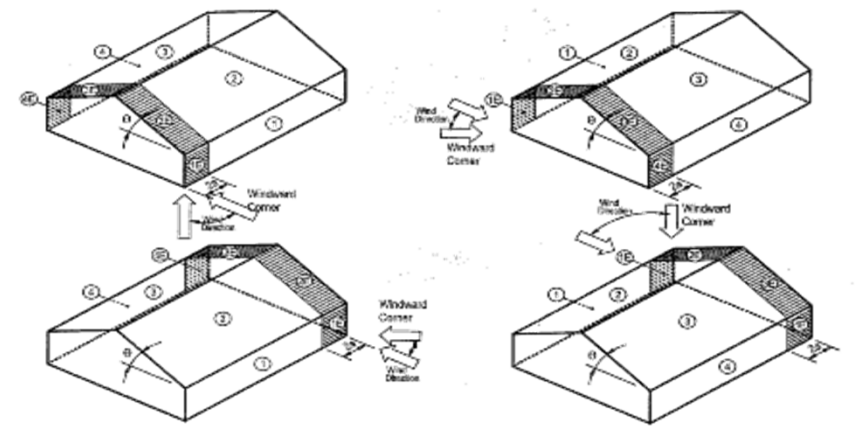
Notes: 1. For Load Case A (Transverse), Load Case B (Longitudinal), and Torsional Cases:

Zone 1 is windward wall for interior zone.	Zone 1E is windward wall for end zone.
Zone 2 is windward roof for interior zone.	Zone 2E is windward roof for end zone.
Zone 3 is leeward roof for interior zone.	Zone 3E is leeward roof for end zone.
Zone 4 is leeward wall for interior zone.	Zone 4E is leeward wall for end zone.
Zones 5 and 6 are sidewalls.	Zone 5E & 6E is sidewalls for end zone.
Zone 1T is windward wall for torsional case	Zone 2T is windward roof for torsional case.
Zone 3T is leeward roof for torsional case	Zone 4T is leeward wall for torsional case.

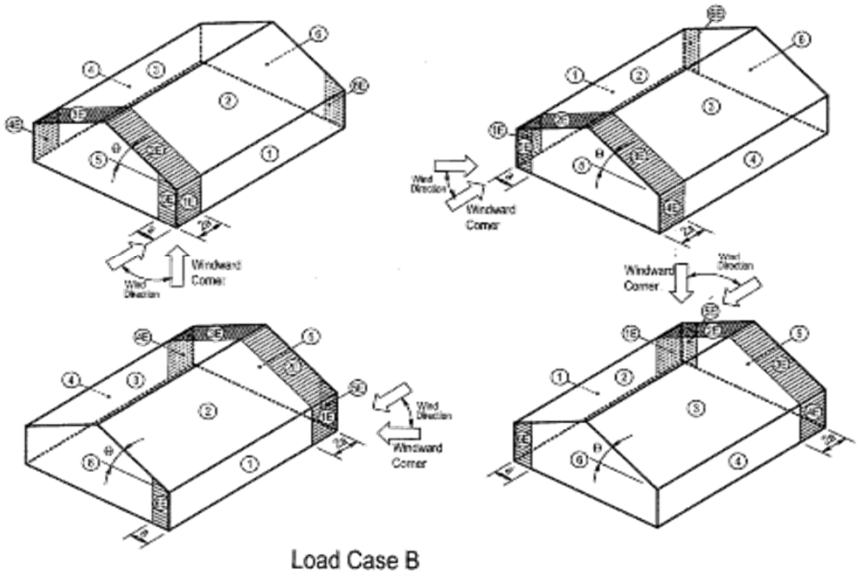
Zones 5T and 6T are sidewalls for torsional case.

2. (+) and (-) signs signify wind pressures acting toward & away from respective surfaces.
3. Building must be designed for all wind directions using the 8 load cases shown below. The load cases are applied to each building corner in turn as the reference corner.
4. Wind loads for torsional cases are 25% of respective transverse or longitudinal zone load values. Torsional loading shall apply to all 8 basic load cases applied at each reference corner.
   Exception: One-story buildings with "h" <= 30', buildings <= 2 stories framed with light frame construction, and buildings <=2 stories designed with flexible diaphragms need not be designed for torsional load cases.
5. Per Code Section 28.4.4, the minimum wind load for MWFRS shall not be less than 16 psf.

Low-Rise Buildings h <= 60'

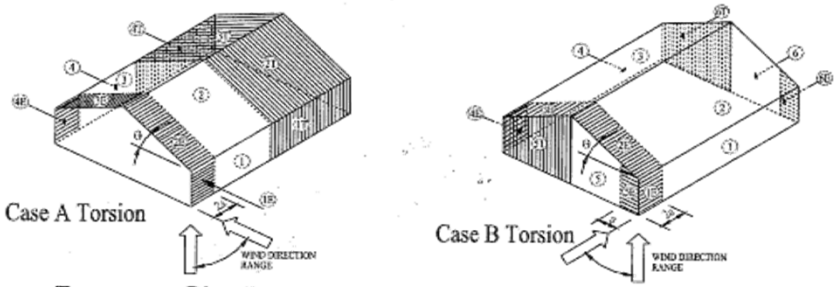


Load Case A



Load Case B

Basic Load Cases



Transverse Direction

Longitudinal Direction

Torsional Load Cases

Wind Shear Force Calculations

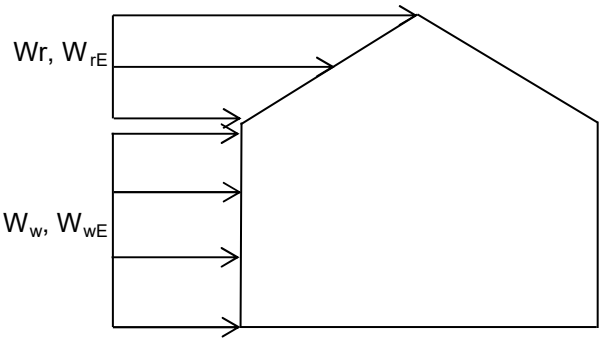
From 'ASCE 7-16 Wind Loading Analysis':

LOAD CASE 'A'	
a = 4.60 feet	2a = 9.20 feet
Z1 = 6.72 psf	Z1E = 12.00 psf
Z2 = -17.54 psf	Z2E = -25.20 psf
Z3 = -13.02 psf	Z3E = -17.12 psf
Z4 = -11.93 psf	Z4E = -15.98 psf

LOAD CASE 'B'	
a = 4.60 psf	2a = 9.20 feet
Z1 = 4.44 psf	Z1E = 8.67 psf
Z2 = -17.54 psf	Z2E = -25.20 psf
Z3 = -11.09 psf	Z3E = -14.31 psf
Z4 = -9.48 psf	Z4E = -12.30 psf

'A' FACTORED LOADS	
$0.6*W_r = (Z_2 + Z_3) * 0.6 =$	**2.7 psf**
$0.6*W_{rE} = (Z_{2E} + Z_{3E}) * 0.6 =$	**4.8 psf**
$0.6*W_w = (Z_1 + Z_4) * 0.6 =$	**11.2 psf**
$0.6*W_{wE} = (Z_{1E} + Z_{4E}) * 0.6 =$	**16.8 psf**

'B' FACTORED LOADS	
$0.6*W_r = (Z_2 + Z_3) * 0.6 =$	**3.9 psf**
$0.6*W_{rE} = (Z_{2E} + Z_{3E}) * 0.6 =$	**6.5 psf**
$0.6*W_w = (Z_1 + Z_4) * 0.6 =$	**8.3 psf**
$0.6*W_{wE} = (Z_{1E} + Z_{4E}) * 0.6 =$	**12.6 psf**



Wall Line	Wind Force (psf)	Wall ht (ft)	Upr. Flr Wall ht (ft)	wall line dist. (ft)	+	Wind Force (psf)	Wall ht (ft)	Wr, We truss trib (ft)	wall line dist (ft)	+	Shear, Upper (#)	=	**Wind Force (kips)**
1-2	12.39	9	0	43.00	+	9.60	0	7.50	43	+		=	**2.75**
2-2	12.39	9	0	43.00	+	9.60	0	7.50	43	+		=	**2.75**
A-2	12.71	9	0	34.00	+	9.60	0	7.50	34	+		=	**2.20**
B-2	11.95	9	0	68.00	+	9.60	0	7.50	68	+		=	**4.28**
C-2	11.95	9	0	68.00	+	9.60	0	7.50	68	+		=	**4.28**
D-2	12.71	9	0	34.00	+	9.60	0	7.50	34	+		=	**2.20**
1-1	12.39	10	6	43.00	+	0.00	0	0.00	0	+	2.75	=	**5.68**
2-1	12.39	10	6	43.00	+	0.00	0	0.00	0	+	2.75	=	**5.68**
A-1	12.71	10	6	34.00	+	0.00	0	0.00	0	+	2.20	=	**4.57**
B-1	11.95	10	6	68.00	+	0.00	0	0.00	0	+	4.28	=	**8.74**
C-1	11.97	10	6	66.00	+	0.00	0	0.00	0	+	4.28	=	**8.62**
D-1	12.80	10	6	32.00	+	0.00	0	0.00	0	+	2.20	=	**4.45**

Seismic Shear Force Calculations

From 'ASCE7-16 Seismic Loading Analysis':

Wall Line	Roof (psf)	Area W (ft)	Area L (ft)	+	Floor (psf)	Area W (ft)	Area L (ft)	+	Wall Type	Wall (psf)	Wall Height (ft)	Perp Wall length (ft)	*C_s	+	Shear upper (kips)	=	**Shear Force (kips)**	**Lateral Control**
1-2	17	43	102	+	18	0	0	+	OSB	12.0	9	43	.03Wp	+		=	**1.43**	**Wind**
2-2	17	43	102	+	18	0	0	+	OSB	12.0	9	43	.03Wp	+		=	**1.43**	**Wind**
A-2	17	34	43	+	18	0	0	+	OSB	12.0	9	34	.03Wp	+		=	**0.55**	**Wind**
B-2	17	68	43	+	18	0	0	+	GYP	8.0	9	68	.11Wp	+		=	**3.29**	**Wind**
C-2	17	68	43	+	18	0	0	+	GYP	8.0	9	68	.11Wp	+		=	**3.29**	**Wind**
D-2	17	34	43	+	18	0	0	+	OSB	12.0	9	34	.03Wp	+		=	**0.55**	**Wind**
1-1	17	0	0	+	18	43	100	+	OSB	12.0	10	0	.03Wp	+	1.43	=	**2.75**	**Wind**
2-1	17	0	0	+	18	43	100	+	OSB	12.0	10	0	.03Wp	+	1.43	=	**2.75**	**Wind**
A-1	17	0	0	+	18	34	42	+	OSB	12.0	10	0	.03Wp	+	0.55	=	**0.99**	**Wind**
B-1	17	0	0	+	18	68	42	+	GYP	8.0	10	0	.11Wp	+	3.29	=	**6.14**	**Wind**
C-1	17	0	0	+	18	66	43	+	GYP	8.0	10	0	.11Wp	+	3.29	=	**6.12**	**Wind**
D-1	17	0	0	+	18	32	43	+	OSB	12.0	10	0	.03Wp	+	0.55	=	**0.97**	**Wind**

Description: *1-2 Shear Wall*

Perforated Shear Wall Calculation Sheet: This spreadsheet is made in conformance to the IBC Chapters 2305-2308 and AFPA's "SDPWS: Lateral Force Resisting Systems".

Shear Wall Forces

				Shearwall segments
	81.33	ft	Total length of wall	8
L =	81.33	ft	Total length of shear wall	8
L_w =	24.00	ft	Total length of full height segments	8
H =	9.00	ft	height of shear wall	
H' =	6.00	ft	Maximum opening height	
V_1 =	2747	lbs	Total Wind force at top of wall	
$w_{DL\ self}$ =	108	plf	Self weight	
$w_{DL\ above}$ =	68.00	plf	Applied dead load	
	7/16	in	Prefered OSB thickness	
	1/2	in	Prefered Gyp thickness	
	N	y/n	Wall Connected to Concrete	
	Y	y/n	Wall Connected to Truss or Joist	
	Y	y/n	Wall Connected to Gable / Drag Truss or Rim	

Unit Base Shear

$\%_{fh}$ = L_w/L =	0.295		Percent of full height segments
$\%_{oh}$ = H'/H =	0.667		Percent of maximum opening height
SCAF =	0.59		Shear capacity adjustment factors (NDS SDPWS Table)
v_{base} = V_1/L_w =	114	plf	Unit base shear
v_{req} = v_{base}/SCAF	**195**	plf	Effective unit base shear
OTM =	**42,145**	lb ft	Overturning moment of total length of wall

Shear wall adjustment factor

RM =	**582,082**	lb ft	Resisting moment of total length of wall
r=	0.3857		
C_O=	0.5865		
	34 plf		Blocking Unit Shear
	195.12		Force Calculated

T = **Not Req'd** lbs

OSB Wall Sheathing attachment

Min Shear Wall Segment: *2.57 ft*

Provide: 7/16" OSB W/ 8d Nails @ 6" O.C. Va= 336

OR: 7/16" OSB W/ 1½ 16 Gage Staples @ 3" O.C. Va=

W1

Blocking / Nailing Framing Attachment

"No Blocking Required"

"Typ. Gable / Drag Truss or Rim Nailing"

Description: 2-2 Shear Wall (5 PANELS)

Perforated Shear Wall Calculation Sheet: This spreadsheet is made in conformance to the IBC Chapters 2305-2308 and AFPA's "SDPWS: Lateral Force Resisting Systems".

Shear Wall Forces

				Shearwall segments
	102.00	ft	Total length of wall	4
L =	4.00	ft	Total length of shear wall	
L_w =	4.00	ft	Total length of full height segments	
H =	9.00	ft	height of shear wall	
H' =	9.00	ft	Maximum opening height	
V_1 =	549	lbs	Total Wind force at top of wall	
$w_{DL\ self}$ =	108	plf	Self weight	
$w_{DL\ above}$ =	85.00	plf	Applied dead load	
	7/16	in	Prefered OSB thickness	
	1/2	in	Prefered Gyp thickness	
	N	y/n	Wall Connected to Concrete	
	Y	y/n	Wall Connected to Truss or Joist	
	Y	y/n	Wall Connected to Gable / Drag Truss or Rim	

Unit Base Shear

$\%_{fh}$ = L_w/L =	1.000		Percent of full height segments
$\%_{oh}$ = H'/H =	1.000		Percent of maximum opening height
SCAF =	1.00		Shear capacity adjustment factors (NDS SDPWS Table)
v_{base} = V_1/L_w =	137	plf	Unit base shear
v_{req} = v_{base}/SCAF	137	plf	Effective unit base shear
OTM =	4,944	lb ft	Overturning moment of total length of wall

Shear wall adjustment factor

RM =	1,544	lb ft	Resisting moment of total length of wall
r=	1.0000		
C_O=	1.0000		
	27 plf		Blocking Unit Shear
	137.33		Force Calculated

			Holdown	Ta	Type
T =	1004	lbs	Simpson MSTC28	1155	Strap
		OR:	Simpson DTT2Z	1155	Holdown

OSB Wall Sheathing attachment

Min Shear Wall Segment: *2.57 ft*

Provide: 7/16" OSB W/ 8d Nails @ 6" O.C. Va= 336

OR: 7/16" OSB W/ 1½ 16 Gage Staples @ 3" O.C. Va=

W1

Blocking / Nailing Framing Attachment

"No Blocking Required"

"Typ. Gable / Drag Truss or Rim Nailing"

Description: *A-2 Shear Wall*

Perforated Shear Wall Calculation Sheet: This spreadsheet is made in conformance to the IBC Chapters 2305-2308 and AFPA's "SDPWS: Lateral Force Resisting Systems".

Shear Wall Forces

				Shearwall segments
	16.00	ft	Total length of wall	4.2
L =	16.00	ft	Total length of shear wall	3.7
L_w =	7.90	ft	Total length of full height segments	
H =	9.00	ft	height of shear wall	
H' =	6.00	ft	Maximum opening height	
V_1 =	1405	lbs	Total Wind force at top of wall (Windward or Leeward only)	
$w_{DL\ self}$ =	108	plf	Self weight	
$w_{DL\ above}$ =	204.00	plf	Applied dead load	
	7/16	in	Prefered OSB thickness	
	1/2	in	Prefered Gyp thickness	
	N	y/n	Wall Connected to Concrete	
	Y	y/n	Wall Connected to Truss or Joist	
	N	y/n	Wall Connected to Gable / Drag Truss or Rim	

Unit Base Shear

$\%_{fh}$ = L_w/L =	0.494		Percent of full height segments
$\%_{oh}$ = H'/H =	0.667		Percent of maximum opening height
SCAF =	0.66		Shear capacity adjustment factors (NDS SDPWS Table)
v_{base} = V_1/L_w =	178	plf	Unit base shear
v_{req} = v_{base}/SCAF	**268**	plf	Effective unit base shear
OTM =	**19,053**	lb ft	Overturning moment of total length of wall

Shear wall adjustment factor

RM =	**39,936**	lb ft	Resisting moment of total length of wall
r=	0.5940		
C_O=	0.6639		
	88 plf		Blocking Unit Shear
	267.98		Force Calculated

T = **Not Req'd** lbs

OSB Wall Sheathing attachment

Min Shear Wall Segment: *2.57 ft*

Provide: 7/16" OSB W/ 8d Nails @ 6" O.C. Va= 336

OR: 7/16" OSB W/ 1½ 16 Gage Staples @ 3" O.C. Va=

W1

Blocking / Nailing Framing Attachment

"No Blocking Required"

Description: *A-2 (2) Shear Wall*

Perforated Shear Wall Calculation Sheet: This spreadsheet is made in conformance to the IBC Chapters 2305-2308 and AFPA's "SDPWS: Lateral Force Resisting Systems".

Shear Wall Forces

				Shearwall segments
	23.00	ft	Total length of wall	4.25
L =	4.25	ft	Total length of shear wall	
L_w =	4.25	ft	Total length of full height segments	
H =	9.00	ft	height of shear wall	
H' =	6.00	ft	Maximum opening height	
V_1 =	1405	lbs	Total Wind force at top of wall (Windward or Leeward only)	
$w_{DL\ self}$ =	108	plf	Self weight	
$w_{DL\ above}$ =	204.00	plf	Applied dead load	
	7/16	in	Prefered OSB thickness	
	1/2	in	Prefered Gyp thickness	
	N	y/n	Wall Connected to Concrete	
	Y	y/n	Wall Connected to Truss or Joist	
	N	y/n	Wall Connected to Gable / Drag Truss or Rim	

Unit Base Shear

$\%_{fh}$ = L_w/L =	1.000		Percent of full height segments
$\%_{oh}$ = H'/H =	0.667		Percent of maximum opening height
SCAF =	1.00		Shear capacity adjustment factors (NDS SDPWS Table)
v_{base} = V_1/L_w =	331	plf	Unit base shear
v_{req} = v_{base}/SCAF	**331**	plf	Effective unit base shear
OTM =	**12,649**	lb ft	Overturning moment of total length of wall

Shear wall adjustment factor

RM =	**2,818**	lb ft	Resisting moment of total length of wall
r=	1.0000		
C_O=	1.0000		
	61 plf		Blocking Unit Shear
	330.70		Force Calculated

			Holdown	Ta	Type
T =	**2579**	lbs	**Simpson MSTC40**	**2640**	**Strap**
		OR:	**Simpson HDU2**	**2640**	**Holdown**

OSB Wall Sheathing attachment

Min Shear Wall Segment: *2.57 ft*

Provide: 7/16" OSB W/ 8d Nails @ 6" O.C. Va= 336

OR: 7/16" OSB W/ 1½ 16 Gage Staples @ 3" O.C. Va=

W1

Blocking / Nailing Framing Attachment

"No Blocking Required"

Description: *B-2 Shear Wall*

Perforated Shear Wall Calculation Sheet: This spreadsheet is made in conformance to the IBC Chapters 2305-2308 and AFPA's "SDPWS: Lateral Force Resisting Systems".

Shear Wall Forces

				Shearwall segments
	22.70	ft	Total length of wall	22
L =	22.70	ft	Total length of shear wall	
L_w =	22.00	ft	Total length of full height segments	
H =	9.00	ft	height of shear wall	
H' =	0.00	ft	Maximum opening height	
V_1 =	4276	lbs	Total Wind force at top of wall	
$w_{DL\ self}$ =	108	plf	Self weight	
$w_{DL\ above}$ =	68.00	plf	Applied dead load	
	7/16	in	Prefered OSB thickness	
	1/2	in	Prefered Gyp thickness	
	N	y/n	Wall Connected to Concrete	
	N	y/n	Wall Connected to Truss or Joist	
	Y	y/n	Wall Connected to Gable / Drag Truss or Rim	

Unit Base Shear

$\%_{fh}$ = L_w/L =	0.969		Percent of full height segments
$\%_{oh}$ = H'/H =	0.000		Percent of maximum opening height
SCAF =	1.00		Shear capacity adjustment factors (NDS SDPWS Table)
v_{base} = V_1/L_w =	194	plf	Unit base shear
v_{req} = v_{base}/SCAF	**194**	plf	Effective unit base shear
OTM =	**38,485**	lb ft	Overturning moment of total length of wall

Shear wall adjustment factor

RM =	**45,346**	lb ft	Resisting moment of total length of wall
r=	1.0000		
C_O=	1.0318		
	188 plf		Blocking Unit Shear
	194.37		Force Calculated

			Holdown	Ta	Type
T =	**413**	lbs	**Intersecting wall**	**500**	**Misc**

Gyp Board Wall Sheathing attachment

Min Shear Wall Segment: *4.50 ft*

Provide: 1/2" GYP Board W/ 5d Cooler Nails or Screws, Unblocked @ 6' O.C. (BOTH SIDES) Va= 200 **WA**

Blocking / Nailing Framing Attachment

Nail Gable / Drag Truss or Rim to Top Plate W/ 10d's @ 6" O.C. to Top Plate

Description: *C-2 Shear Wall*

Perforated Shear Wall Calculation Sheet: This spreadsheet is made in conformance to the IBC Chapters 2305-2308 and AFPA's "SDPWS: Lateral Force Resisting Systems".

Shear Wall Forces

				Shearwall segments
	22.70	ft	Total length of wall	22
L =	22.70	ft	Total length of shear wall	
L_w =	22.00	ft	Total length of full height segments	
H =	9.00	ft	height of shear wall	
H' =	0.00	ft	Maximum opening height	
V_1 =	4276	lbs	Total Wind force at top of wall	
$w_{DL\ self}$ =	108	plf	Self weight	
$w_{DL\ above}$ =	68.00	plf	Applied dead load	
	7/16	in	Prefered OSB thickness	
	1/2	in	Prefered Gyp thickness	
	N	y/n	Wall Connected to Concrete	
	N	y/n	Wall Connected to Truss or Joist	
	Y	y/n	Wall Connected to Gable / Drag Truss or Rim	

Unit Base Shear

$\%_{fh}$ = L_w/L =	0.969		Percent of full height segments
$\%_{oh}$ = H'/H =	0.000		Percent of maximum opening height
SCAF =	1.00		Shear capacity adjustment factors (NDS SDPWS Table)
v_{base} = V_1/L_w =	194	plf	Unit base shear
v_{req} = v_{base}/SCAF	**194**	plf	Effective unit base shear
OTM =	**38,485**	lb ft	Overturning moment of total length of wall

Shear wall adjustment factor

RM =	**45,346**	lb ft	Resisting moment of total length of wall
r=	1.0000		
C_O=	1.0318		
	188 plf		Blocking Unit Shear
	194.37		Force Calculated

			Holdown	Ta	Type
T =	**413**	lbs	**Intersecting wall**	**500**	**Misc**

Gyp Board Wall Sheathing attachment

Min Shear Wall Segment: *4.50 ft*

Provide: 1/2" GYP Board W/ 5d Cooler Nails or Screws, Unblocked @ 6' O.C. (BOTH SIDES) Va= 200 **WA**

Blocking / Nailing Framing Attachment

Nail Gable / Drag Truss or Rim to Top Plate W/ 10d's @ 6" O.C. to Top Plate

Description: *D-2 Shear Wall*

Perforated Shear Wall Calculation Sheet: This spreadsheet is made in conformance to the IBC Chapters 2305-2308 and AFPA's "SDPWS: Lateral Force Resisting Systems".

Shear Wall Forces

				Shearwall segments
	16.00	ft	Total length of wall	4.2
L =	16.00	ft	Total length of shear wall	3.7
L_w =	7.90	ft	Total length of full height segments	
H =	9.00	ft	height of shear wall	
H' =	6.00	ft	Maximum opening height	
V_1 =	1405	lbs	Total Wind force at top of wall (Windward or Leeward only)	
$w_{DL\ self}$ =	108	plf	Self weight	
$w_{DL\ above}$ =	204.00	plf	Applied dead load	
	7/16	in	Prefered OSB thickness	
	1/2	in	Prefered Gyp thickness	
	N	y/n	Wall Connected to Concrete	
	Y	y/n	Wall Connected to Truss or Joist	
	Y	y/n	Wall Connected to Gable / Drag Truss or Rim	

Unit Base Shear

$\%_{fh}$ = L_w/L =	0.494		Percent of full height segments
$\%_{oh}$ = H'/H =	0.667		Percent of maximum opening height
SCAF =	0.66		Shear capacity adjustment factors (NDS SDPWS Table)
v_{base} = V_1/L_w =	178	plf	Unit base shear
v_{req} = v_{base}/SCAF	**268**	plf	Effective unit base shear
OTM =	**19,053**	lb ft	Overturning moment of total length of wall

Shear wall adjustment factor

RM =	**39,936**	lb ft	Resisting moment of total length of wall
r=	0.5940		
C_O=	0.6639		
	88 plf		Blocking Unit Shear
	267.98		Force Calculated

T = **Not Req'd** lbs

OSB Wall Sheathing attachment

Min Shear Wall Segment: *2.57 ft*

Provide: 7/16" OSB W/ 8d Nails @ 6" O.C. Va= 336

OR: 7/16" OSB W/ 1½ 16 Gage Staples @ 3" O.C. Va=

W1

Blocking / Nailing Framing Attachment

"No Blocking Required"

"Typ. Gable / Drag Truss or Rim Nailing"

Description: *D-2 (2) Shear Wall*

Perforated Shear Wall Calculation Sheet: This spreadsheet is made in conformance to the IBC Chapters 2305-2308 and AFPA's "SDPWS: Lateral Force Resisting Systems".

Shear Wall Forces

				Shearwall segments
	23.00	ft	Total length of wall	4.25
L =	23.00	ft	Total length of shear wall	3.25
L_w =	10.08	ft	Total length of full height segments	2.58
H =	9.00	ft	height of shear wall	
H' =	3.50	ft	Maximum opening height	
V_1 =	1405	lbs	Total Wind force at top of wall (Windward or Leeward only)	
$w_{DL\ self}$ =	108	plf	Self weight	
$w_{DL\ above}$ =	204.00	plf	Applied dead load	
	7/16	in	Prefered OSB thickness	
	1/2	in	Prefered Gyp thickness	
	N	y/n	Wall Connected to Concrete	
	Y	y/n	Wall Connected to Truss or Joist	
	N	y/n	Wall Connected to Gable / Drag Truss or Rim	

Unit Base Shear

$\%_{fh}$ = L_w/L =	0.438		Percent of full height segments
$\%_{oh}$ = H'/H =	0.389		Percent of maximum opening height
SCAF =	0.91		Shear capacity adjustment factors (NDS SDPWS Table)
v_{base} = V_1/L_w =	139	plf	Unit base shear
v_{req} = v_{base}/SCAF	152	plf	Effective unit base shear
OTM =	13,834	lb ft	Overturning moment of total length of wall

Shear wall adjustment factor

RM =	82,524	lb ft	Resisting moment of total length of wall
r=	0.6674		
C_O=	0.9144		
	61 plf		Blocking Unit Shear
	152.49		Force Calculated

T = **Not Req'd** lbs

OSB Wall Sheathing attachment

Min Shear Wall Segment: *2.57 ft*

Provide: 7/16" OSB W/ 8d Nails @ 6" O.C. Va= 336

OR: 7/16" OSB W/ 1½ 16 Gage Staples @ 3" O.C. Va=

W1

Blocking / Nailing Framing Attachment

"No Blocking Required"

Description: *1-1 Shear Wall*

Perforated Shear Wall Calculation Sheet: This spreadsheet is made in conformance to the IBC Chapters 2305-2308 and AFPA's "SDPWS: Lateral Force Resisting Systems".

Shear Wall Forces

	28.50	ft	Total length of wall
L =	28.50	ft	Total length of shear wall
L_w =	18.50	ft	Total length of full height segments
H =	10.00	ft	height of shear wall
H' =	8.00	ft	Maximum opening height
V_1 =	3633	lbs	Total Wind force at top of wall (Windward or Leeward only)
$w_{DL\ self}$ =	120	plf	Self weight
$w_{DL\ above}$ =	168.98	plf	Applied dead load
	7/16	in	Prefered OSB thickness
	1/2	in	Prefered Gyp thickness
	Y	y/n	Wall Connected to Concrete
	N	y/n	Wall Connected to Truss or Joist
	Y	y/n	Wall Connected to Gable / Drag Truss or Rim

Shearwall segments
7
5.25
6.25

Unit Base Shear

$\%_{fh}$ = L_w/L =	0.649		Percent of full height segments
$\%_{oh}$ = H'/H =	0.800		Percent of maximum opening height
SCAF =	0.67		Shear capacity adjustment factors (NDS SDPWS Table)
v_{base} = V_1/L_w =	196	plf	Unit base shear
v_{req} = v_{base}/SCAF	**293**	plf	Effective unit base shear
OTM =	**54,178**	lb ft	Overturning moment of total length of wall

Shear wall adjustment factor

RM =	**117,362**	lb ft	Resisting moment of total length of wall
r=	0.6981		
C_O=	0.6706		
	127 plf		Blocking Unit Shear
	292.86		Force Calculated

Shear Transfer to Concrete:

1/2 Anchor Bolts @ 72 " O.C. (4) Minimum

T = **Not Req'd** lbs

OSB Wall Sheathing attachment

Min Shear Wall Segment: *2.86 ft*

Provide: 7/16" OSB W/ 8d Nails @ 6" O.C. Va= 336

OR: 7/16" OSB W/ 1½ 16 Gage Staples @ 3" O.C. Va=

W1

Blocking / Nailing Framing Attachment

"Typ. Gable / Drag Truss or Rim Nailing"

Description: 2-1 Shear Wall (2 PANELS)

Perforated Shear Wall Calculation Sheet: This spreadsheet is made in conformance to the IBC Chapters 2305-2308 and AFPA's "SDPWS: Lateral Force Resisting Systems".

Shear Wall Forces

				Shearwall segments
	19.00	ft	Total length of wall	1.5
L =	1.50	ft	Total length of shear wall	
L_w =	1.50	ft	Total length of full height segments	
H =	9.00	ft	height of shear wall	
H' =	0.00	ft	Maximum opening height	
V_1 =	774	lbs	Total Wind force at top of wall	
$w_{DL\ self}$ =	108	plf	Self weight	
$w_{DL\ above}$ =	68.00	plf	Applied dead load	
	7/16	in	Prefered OSB thickness	
	1/2	in	Prefered Gyp thickness	
	Y	y/n	Wall Connected to Concrete	
	N	y/n	Wall Connected to Truss or Joist	
	Y	y/n	Wall Connected to Gable / Drag Truss or Rim	

Unit Base Shear

$\%_{fh}$ = L_w/L =	1.000		Percent of full height segments
$\%_{oh}$ = H'/H =	0.000		Percent of maximum opening height
SCAF =	1.00		Shear capacity adjustment factors (NDS SDPWS Table)
v_{base} = V_1/L_w =	516	plf	Unit base shear
v_{req} = v_{base}/SCAF	516	plf	Effective unit base shear
OTM =	6,967	lb ft	Overturning moment of shortest panel

Shear wall adjustment factor

RM =	198	lb ft	Resisting moment of shortest panel
r=	1.0000		
C_O=	1.0000		
	81 plf		Blocking Unit Shear
	516.07		Force Calculated

			Holdown	Ta	Type
T =	3500	lbs	Simpson STHD14	5645	Strap
		OR:	Simpson HDU5	5645	Holdown

Portal Frame

Provide: (2) Eng. Portal Frame Va = 1454# EA. 2908# Total

Description: *2-1 (2) Shear Wall* *(2 PANELS)*

Perforated Shear Wall Calculation Sheet: This spreadsheet is made in conformance to the IBC Chapters 2305-2308 and AFPA's "SDPWS: Lateral Force Resisting Systems".

Shear Wall Forces

				Shearwall segments
	12.50	ft	Total length of wall	4
L =	4.00	ft	Total length of shear wall	
L_w =	4.00	ft	Total length of full height segments	
H =	9.00	ft	height of shear wall	
H' =	0.00	ft	Maximum opening height	
V_1 =	2064	lbs	Total Wind force at top of wall	
$w_{DL\ self}$ =	108	plf	Self weight	
$w_{DL\ above}$ =	68.00	plf	Applied dead load	
	7/16	in	Prefered OSB thickness	
	1/2	in	Prefered Gyp thickness	
	Y	y/n	Wall Connected to Concrete	
	N	y/n	Wall Connected to Truss or Joist	
	Y	y/n	Wall Connected to Gable / Drag Truss or Rim	

Unit Base Shear

$\%_{fh}$ = L_w/L =	1.000		Percent of full height segments
$\%_{oh}$ = H'/H =	0.000		Percent of maximum opening height
SCAF =	1.00		Shear capacity adjustment factors (NDS SDPWS Table)
v_{base} = V_1/L_w =	516	plf	Unit base shear
v_{req} = v_{base}/SCAF	**516**	plf	Effective unit base shear
OTM =	**18,579**	lb ft	Overturning moment of shortest panel

Shear wall adjustment factor

RM =	**1,408**	lb ft	Resisting moment of shortest panel
r=	1.0000		
C_O=	1.0000		
	330 plf		Blocking Unit Shear
	516.07		Force Calculated

			Holdown	Ta	Type
T =	**3500**	lbs	**Simpson STHD14**	**5645**	**Strap**
		OR:	**Simpson HDU5**	**5645**	**Holdown**

OSB Wall Sheathing attachment

Min Shear Wall Segment: *1.33 ft*

Provide: 7/16" OSB W/ 8d Nails @ 3" O.C.

Va= 630

W3

Blocking / Nailing Framing Attachment

Nail Gable / Drag Truss or Rim to Top Plate W/ 10d's @ 4" O.C. to Top Plate

Description: *A-1 Shear Wall*

Perforated Shear Wall Calculation Sheet: This spreadsheet is made in conformance to the IBC Chapters 2305-2308 and AFPA's "SDPWS: Lateral Force Resisting Systems".

Shear Wall Forces

				Shearwall segments
	16.00	ft	Total length of wall	4
L =	16.00	ft	Total length of shear wall	3.7
L_w =	7.70	ft	Total length of full height segments	
H =	10.00	ft	height of shear wall	
H' =	6.00	ft	Maximum opening height	
V_1 =	2926	lbs	Total Wind force at top of wall (Windward or Leeward only)	
$w_{DL\ self}$ =	120	plf	Self weight	
$w_{DL\ above}$ =	68.00	plf	Applied dead load	
	7/16	in	Prefered OSB thickness	
	1/2	in	Prefered Gyp thickness	
	Y	y/n	Wall Connected to Concrete	
	N	y/n	Wall Connected to Truss or Joist	
	Y	y/n	Wall Connected to Gable / Drag Truss or Rim	

Unit Base Shear

$\%_{fh}$ = L_w/L =	0.481		Percent of full height segments
$\%_{oh}$ = H'/H =	0.600		Percent of maximum opening height
SCAF =	0.71		Shear capacity adjustment factors (NDS SDPWS Table)
v_{base} = V_1/L_w =	380	plf	Unit base shear
v_{req} = v_{base}/SCAF	**538**	plf	Effective unit base shear
OTM =	**41,406**	lb ft	Overturning moment of shortest panel

Shear wall adjustment factor

RM =	**24,064**	lb ft	Resisting moment of shortest panel
r=	0.6073		
C_O=	0.7067		
	183 plf		Blocking Unit Shear
	537.74		Force Calculated

Shear Transfer to Concrete:

1/2 Anchor Bolts @ 72 " O.C.

(3) Minimum

			Holdown	Ta	Type
T =	**3502**	lbs	**Simpson STHD14**	**5645**	**Strap**
		OR:	**Simpson HDU5**	**5645**	**Holdown**

OSB Wall Sheathing attachment

Min Shear Wall Segment: *2.86 ft*

Provide: 7/16" OSB W/ 8d Nails @ 3" O.C.

Va= 630

W3

Blocking / Nailing Framing Attachment

Nail Gable / Drag Truss or Rim to Top Plate W/ 10d's @ 6" O.C. to Top Plate

Description: *A-1 (2) Shear Wall*

Perforated Shear Wall Calculation Sheet: This spreadsheet is made in conformance to the IBC Chapters 2305-2308 and AFPA's "SDPWS: Lateral Force Resisting Systems".

Shear Wall Forces

				Shearwall segments
	7.67	ft	Total length of wall	7.67
L =	7.67	ft	Total length of shear wall	
L_w =	7.67	ft	Total length of full height segments	
H =	10.00	ft	height of shear wall	
H' =	0.00	ft	Maximum opening height	
V_1 =	2926	lbs	Total Wind force at top of wall (Windward or Leeward only)	
$w_{DL\ self}$ =	120	plf	Self weight	
$w_{DL\ above}$ =	68.00	plf	Applied dead load	
	7/16	in	Prefered OSB thickness	
	1/2	in	Prefered Gyp thickness	
	Y	y/n	Wall Connected to Concrete	
	N	y/n	Wall Connected to Truss or Joist	
	Y	y/n	Wall Connected to Gable / Drag Truss or Rim	

Unit Base Shear

$\%_{fh}$ = L_w/L =	1.000		Percent of full height segments
$\%_{oh}$ = H'/H =	0.000		Percent of maximum opening height
SCAF =	1.00		Shear capacity adjustment factors (NDS SDPWS Table)
v_{base} = V_1/L_w =	382	plf	Unit base shear
v_{req} = v_{base}/SCAF	382	plf	Effective unit base shear
OTM =	29,262	lb ft	Overturning moment of total length of wall

Shear wall adjustment factor

RM =	5,530	lb ft	Resisting moment of total length of wall
r=	1.0000		
C_O=	1.0000		
	382 plf		Blocking Unit Shear
	381.52		Force Calculated

Shear Transfer to Concrete:

1/2 Anchor Bolts @ 36 " O.C. (3) Minimum

			Holdown	Ta	Type
T =	3383	lbs	Simpson STHD14	5645	Strap
		OR:	Simpson HDU5	5645	Holdown

OSB Wall Sheathing attachment

Min Shear Wall Segment: *2.86 ft*

Provide: 7/16" OSB W/ 8d Nails @ 4" O.C. Va= 490

OR: 7/16" OSB W/ 1½ 16 Gage Staples @ 2" O.C. Va=

W2

Blocking / Nailing Framing Attachment

Nail Gable / Drag Truss or Rim to Top Plate W/ 10d's @ 4" O.C. to Top Plate

Description: *B-1 Shear Wall* *(2 PANELS)*

Perforated Shear Wall Calculation Sheet: This spreadsheet is made in conformance to the IBC Chapters 2305-2308 and AFPA's "SDPWS: Lateral Force Resisting Systems".

Shear Wall Forces

				Shearwall segments
	22.00	ft	Total length of wall	16
L =	16.00	ft	Total length of shear wall	
L_w =	16.00	ft	Total length of full height segments	
H =	10.00	ft	height of shear wall	
H' =	0.00	ft	Maximum opening height	
V_1 =	4372	lbs	Total Wind force at top of wall	
$w_{DL\ self}$ =	120	plf	Self weight	
$w_{DL\ above}$ =	80.00	plf	Applied dead load	
	7/16	in	Prefered OSB thickness	
	1/2	in	Prefered Gyp thickness	
	Y	y/n	Wall Connected to Concrete	
	N	y/n	Wall Connected to Truss or Joist	
	Y	y/n	Wall Connected to Gable / Drag Truss or Rim	

Unit Base Shear

$\%_{fh}$ = L_w/L =	1.000		Percent of full height segments
$\%_{oh}$ = H'/H =	0.000		Percent of maximum opening height
SCAF =	1.00		Shear capacity adjustment factors (NDS SDPWS Table)
v_{base} = V_1/L_w =	273	plf	Unit base shear
v_{req} = v_{base}/SCAF	**273**	plf	Effective unit base shear
OTM =	**43,725**	lb ft	Overturning moment of total length of wall

Shear wall adjustment factor

RM =	**25,600**	lb ft	Resisting moment of total length of wall
r=	1.0000		
C_O=	1.0000		
	397 plf		Blocking Unit Shear
	273.28		Force Calculated

Shear Transfer to Concrete:

1/2 Anchor Bolts @ 60 " O.C.

(5) Minimum

			Holdown	Ta	Type
T =	**1773**	lbs	**Simpson LSTHD8**	**2145**	**Strap**
		OR:	**Simpson DTT2Z**	**2145**	**Holdown**

OSB Wall Sheathing attachment

Min Shear Wall Segment: *2.86 ft*

Provide: 7/16" OSB W/ 8d Nails @ 6" O.C. Va= 336

OR: 7/16" OSB W/ 1½ 16 Gage Staples @ 3" O.C. Va=

W1

Blocking / Nailing Framing Attachment

Nail Gable / Drag Truss or Rim to Top Plate W/ 10d's @ 4" O.C. to Top Plate

Description: *C-1 Shear Wall*

Perforated Shear Wall Calculation Sheet: This spreadsheet is made in conformance to the IBC Chapters 2305-2308 and AFPA's "SDPWS: Lateral Force Resisting Systems".

Shear Wall Forces

				Shearwall segments
	40.00	ft	Total length of wall	40
L =	*40.00*	ft	Total length of shear wall	
L_w =	*40.00*	ft	Total length of full height segments	
H =	*10.00*	ft	height of shear wall	
H' =	*0.00*	ft	Maximum opening height	
V_1 =	*8622*	lbs	Total Wind force at top of wall	
$w_{DL\ self}$ =	*120*	plf	Self weight	
$w_{DL\ above}$ =	*68.00*	plf	Applied dead load	
	7/16	in	Prefered OSB thickness	
	1/2	in	Prefered Gyp thickness	
	Y	y/n	Wall Connected to Concrete	
	N	y/n	Wall Connected to Truss or Joist	
	Y	y/n	Wall Connected to Gable / Drag Truss or Rim	

Unit Base Shear

$\%_{fh}$ = L_w/L =	1.000		Percent of full height segments
$\%_{oh}$ = H'/H =	0.000		Percent of maximum opening height
SCAF =	*1.00*		Shear capacity adjustment factors (NDS SDPWS Table)
v_{base} = V_1/L_w =	216	plf	Unit base shear
v_{req} = v_{base}/SCAF	**216**	plf	Effective unit base shear
OTM =	**86,219**	lb ft	Overturning moment of total length of wall

Shear wall adjustment factor

RM =	**150,400**	lb ft	Resisting moment of total length of wall
r=	1.0000		
C_O=	1.0000		
	216 plf		Blocking Unit Shear
	215.55		Force Calculated

Shear Transfer to Concrete:

1/2 Anchor Bolts @ 60 " O.C. (9) Minimum

T = **Not Req'd** lbs

Gyp Board Wall Sheathing attachment Min Shear Wall Segment: *5.00 ft*

Provide: 1/2" GYP Board W/ 5d Cooler Nails or Screws, Blocked @ 6' O.C. (BOTH SIDES) Va= 250 **WB**

OR: 1/2" GYP Board W/ 5d Cooler Nails or Screws, Unblocked @ 4" O.C. (BOTH SIDES)

Blocking / Nailing Framing Attachment

Nail Gable / Drag Truss or Rim to Top Plate W/ 10d's @ 6" O.C. to Top Plate

Description: *D-1 Shear Wall*

Perforated Shear Wall Calculation Sheet: This spreadsheet is made in conformance to the IBC Chapters 2305-2308 and AFPA's "SDPWS: Lateral Force Resisting Systems".

Shear Wall Forces

				Shearwall segments
	30.79	ft	Total length of wall	3.7
L =	30.79	ft	Total length of shear wall	7
L_w =	19.37	ft	Total length of full height segments	8.67
H =	10.00	ft	height of shear wall	
H' =	6.00	ft	Maximum opening height	
V_1 =	4449	lbs	Total Wind force at top of wall	
$w_{DL\ self}$ =	120	plf	Self weight	
$w_{DL\ above}$ =	68.00	plf	Applied dead load	
	7/16	in	Prefered OSB thickness	
	1/2	in	Prefered Gyp thickness	
	Y	y/n	Wall Connected to Concrete	
	N	y/n	Wall Connected to Truss or Joist	
	Y	y/n	Wall Connected to Gable / Drag Truss or Rim	

Unit Base Shear

$\%_{fh}$ = L_w/L =	0.629		Percent of full height segments
$\%_{oh}$ = H'/H =	0.600		Percent of maximum opening height
SCAF =	0.77		Shear capacity adjustment factors (NDS SDPWS Table)
v_{base} = V_1/L_w =	230	plf	Unit base shear
v_{req} = v_{base}/SCAF	**298**	plf	Effective unit base shear
OTM =	**57,693**	lb ft	Overturning moment of total length of wall

Shear wall adjustment factor

RM =	**89,114**	lb ft	Resisting moment of total length of wall
r=	0.7387		
C_O=	0.7712		
	144 plf		Blocking Unit Shear
	297.85		Force Calculated

Shear Transfer to Concrete:

1/2 Anchor Bolts @ 72 " O.C.

(5) Minimum

			Holdown	Ta	Type
T =	**218**	lbs	**Intersecting wall**	**500**	**Misc**

OSB Wall Sheathing attachment

Min Shear Wall Segment: *2.86 ft*

Provide: 7/16" OSB W/ 8d Nails @ 6" O.C. Va= 336

OR: 7/16" OSB W/ 1½ 16 Gage Staples @ 3" O.C. Va=

W1

Blocking / Nailing Framing Attachment

"Typ. Gable / Drag Truss or Rim Nailing"

Project Name: 19/051 Woodlawn Townhomes
Project #: 2010-09655
Location: Boise, Idaho

PERFORMANCE ENGINEERS

Engineering: Spencer Bradshaw
Checker: Shawn Reeder
Date: 05/06/2020

Steel Beam

File: 05 Beams.ec6

Lic. # : KW-06007473
SHAWN REEDER

DESCRIPTION: Deck Beam

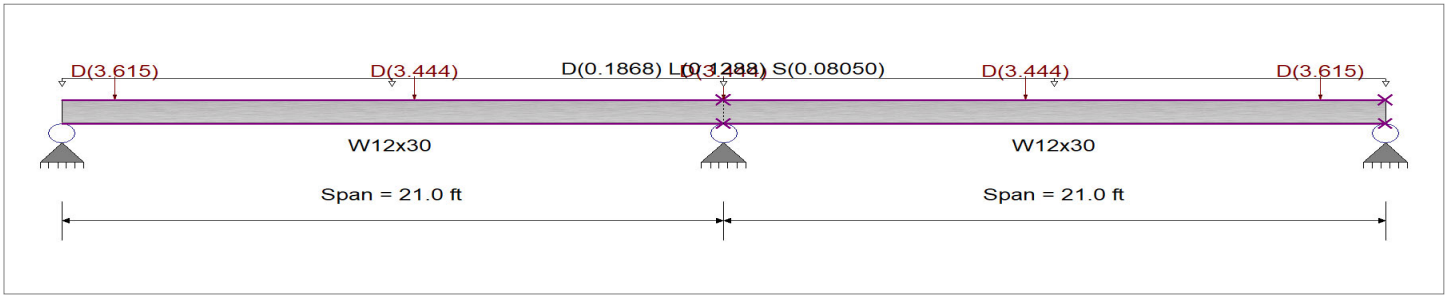
CODE REFERENCES

Calculations per AISC 360-10, IBC 2018, CBC 2016, ASCE 7-16
Load Combination Set : IBC 2018

Material Properties

Analysis Method : Allowable Strength Design
Beam Bracing : Beam is Fully Braced against lateral-torsional buckling
Bending Axis : Major Axis Bending

Fy : Steel Yield : 50.0 ksi
E: Modulus : 29,000.0 ksi



Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Beam self weight NOT internally calculated and added
Loads on all spans...
Uniform Load on ALL spans : D = 0.0580, L = 0.040, S = 0.0250 ksf, Tributary Width = 3.220 ft

Load(s) for Span Number 1
Point Load : D = 3.615 k @ 1.70 ft, (Beam PL)

Point Load : D = 3.444 k @ 11.20 ft, (Beam PL)

Point Load : D = 3.444 k @ 21.0 ft, (Beam PL)

Load(s) for Span Number 2
Point Load : D = 3.444 k @ 9.580 ft, (Beam PL)

Point Load : D = 3.615 k @ 18.958 ft, (Beam PL)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	0.336 : 1	Maximum Shear Stress Ratio =	0.170 : 1
Section used for this span	W12x30	Section used for this span	W12x30
Ma : Applied	36.128 k-ft	Va : Applied	10.903 k
Mn / Omega : Allowable	107.535 k-ft	Vn/Omega : Allowable	63.960 k
Load Combination	+D+0.750L+0.750S+H	Load Combination	+D+0.750L+0.750S+H
Location of maximum on span	21.000 ft	Location of maximum on span	21.000 ft
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.034 in	Ratio =	7,362 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.190 in	Ratio =	1327 >=180
Max Upward Total Deflection	0.000 in	Ratio =	0 <180

Maximum Forces & Stresses for Load Combinations

Load Combination / Segment Length	Span #	Max Stress Ratios M	Max Stress Ratios V	Summary of Moment Values Mmax +	Mmax -	Ma Max	Mnx	Mnx/Omega	Cb	Rm	Summary of Shear Values Va Max	Vnx	Vnx/Omega
+D+H													
Dsgn. L = 21.00 ft	1	0.255	0.138	16.51	-27.47	27.47	179.58	107.53	1.00	1.00	8.84	95.94	63.96
Dsgn. L = 21.00 ft	2	0.255	0.086	16.77	-27.47	27.47	179.58	107.53	1.00	1.00	5.49	95.94	63.96
+D+0.750L+0.750S+H													
Dsgn. L = 21.00 ft	1	0.322	0.165	20.23	-34.57	34.57	179.58	107.53	1.00	1.00	10.53	95.94	63.96
Dsgn. L = 21.00 ft	2	0.322	0.112	20.58	-34.57	34.57	179.58	107.53	1.00	1.00	7.18	95.94	63.96
+D+Lr+H													

Steel Beam

DESCRIPTION: Deck Beam

Load Combination / Segment Length	Span #	Max Stress Ratios M	Max Stress Ratios V	Summary of Moment Values Mmax +	Mmax -	Ma Max	Mnx	Mnx/Omega	Cb	Rm	Summary of Shear Values Va Max	Vnx	Vnx/Omega
Dsgn. L = 21.00 ft	1	0.255	0.138	16.51	-27.47	27.47	179.58	107.53	1.00	1.00	8.84	95.94	63.96
Dsgn. L = 21.00 ft	2	0.255	0.086	16.77	-27.47	27.47	179.58	107.53	1.00	1.00	5.49	95.94	63.96
+D+S+H													
Dsgn. L = 21.00 ft	1	0.297	0.155	18.80	-31.91	31.91	179.58	107.53	1.00	1.00	9.90	95.94	63.96
Dsgn. L = 21.00 ft	2	0.297	0.102	19.13	-31.91	31.91	179.58	107.53	1.00	1.00	6.55	95.94	63.96
+D+0.750Lr+0.750L+H													
Dsgn. L = 21.00 ft	1	0.305	0.158	19.28	-32.80	32.80	179.58	107.53	1.00	1.00	10.11	95.94	63.96
Dsgn. L = 21.00 ft	2	0.305	0.106	19.61	-32.80	32.80	179.58	107.53	1.00	1.00	6.76	95.94	63.96
+D+0.750L+0.750S+H													
Dsgn. L = 21.00 ft	1	0.336	0.170	21.07	-36.13	36.13	179.58	107.53	1.00	1.00	10.90	95.94	63.96
Dsgn. L = 21.00 ft	2	0.336	0.118	21.44	-36.13	36.13	179.58	107.53	1.00	1.00	7.55	95.94	63.96
+D+0.60W+H													
Dsgn. L = 21.00 ft	1	0.255	0.138	16.51	-27.47	27.47	179.58	107.53	1.00	1.00	8.84	95.94	63.96
Dsgn. L = 21.00 ft	2	0.255	0.086	16.77	-27.47	27.47	179.58	107.53	1.00	1.00	5.49	95.94	63.96
+D+0.70E+H													
Dsgn. L = 21.00 ft	1	0.255	0.138	16.51	-27.47	27.47	179.58	107.53	1.00	1.00	8.84	95.94	63.96
Dsgn. L = 21.00 ft	2	0.255	0.086	16.77	-27.47	27.47	179.58	107.53	1.00	1.00	5.49	95.94	63.96
+D+0.750Lr+0.750L+0.450W+H													
Dsgn. L = 21.00 ft	1	0.305	0.158	19.28	-32.80	32.80	179.58	107.53	1.00	1.00	10.11	95.94	63.96
Dsgn. L = 21.00 ft	2	0.305	0.106	19.61	-32.80	32.80	179.58	107.53	1.00	1.00	6.76	95.94	63.96
+D+0.750L+0.750S+0.450W+H													
Dsgn. L = 21.00 ft	1	0.336	0.170	21.07	-36.13	36.13	179.58	107.53	1.00	1.00	10.90	95.94	63.96
Dsgn. L = 21.00 ft	2	0.336	0.118	21.44	-36.13	36.13	179.58	107.53	1.00	1.00	7.55	95.94	63.96
+D+0.750L+0.750S+0.5250E+H													
Dsgn. L = 21.00 ft	1	0.336	0.170	21.07	-36.13	36.13	179.58	107.53	1.00	1.00	10.90	95.94	63.96
Dsgn. L = 21.00 ft	2	0.336	0.118	21.44	-36.13	36.13	179.58	107.53	1.00	1.00	7.55	95.94	63.96
+0.60D+0.60W+0.60H													
Dsgn. L = 21.00 ft	1	0.153	0.083	9.90	-16.48	16.48	179.58	107.53	1.00	1.00	5.31	95.94	63.96
Dsgn. L = 21.00 ft	2	0.153	0.052	10.06	-16.48	16.48	179.58	107.53	1.00	1.00	3.30	95.94	63.96
+0.60D+0.70E+0.60H													
Dsgn. L = 21.00 ft	1	0.153	0.083	9.90	-16.48	16.48	179.58	107.53	1.00	1.00	5.31	95.94	63.96
Dsgn. L = 21.00 ft	2	0.153	0.052	10.06	-16.48	16.48	179.58	107.53	1.00	1.00	3.30	95.94	63.96

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+0.750L+0.750S+0.5250E+H	1	0.1839	8.988		0.0000	0.000
+D+0.750L+0.750S+0.5250E+H	2	0.1899	12.096		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2	Support 3
Overall MAXimum	6.818	18.457	6.723
Overall MINimum	0.634	2.113	0.634
+D+H	5.582	14.336	5.487
+D+L+H	6.597	17.717	6.502
+D+Lr+H	5.582	14.336	5.487
+D+S+H	6.216	16.450	6.121
+D+0.750Lr+0.750L+H	6.343	16.872	6.248
+D+0.750L+0.750S+H	6.818	18.457	6.723
+D+0.60W+H	5.582	14.336	5.487
+D+0.70E+H	5.582	14.336	5.487
+D+0.750Lr+0.750L+0.450W+H	6.343	16.872	6.248
+D+0.750L+0.750S+0.450W+H	6.818	18.457	6.723
+D+0.750L+0.750S+0.5250E+H	6.818	18.457	6.723
+0.60D+0.60W+0.60H	3.349	8.602	3.292
+0.60D+0.70E+0.60H	3.349	8.602	3.292
D Only	5.582	14.336	5.487
Lr Only			
L Only	1.014	3.381	1.014
S Only	0.634	2.113	0.634
W Only			
E Only			
H Only			

Steel Column

Lic. # : KW-06007473 SHAWN REEDER

DESCRIPTION: --None--

Code References

Calculations per AISC 360-10, IBC 2018, CBC 2016, ASCE 7-16
Load Combinations Used : IBC 2018

USE (2) HSS6x1-1/2x3/16

General Information

Steel Section Name :	**HSS6x2x3/16**	Overall Column Height	8.50 ft
Analysis Method :	Allowable Strength	Top & Bottom Fixity	Top & Bottom Pinned
Steel Stress Grade		Brace condition for deflection (buckling) along columns :	
Fy : Steel Yield	36.0 ksi	X-X (width) axis :	
E : Elastic Bending Modulus	29,000.0 ksi	Fully braced against buckling ABOUT Y-Y Axis	
		Y-Y (depth) axis :	
		Fully braced against buckling ABOUT X-X Axis	

Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Column self weight included : 80.117 lbs * Dead Load Factor
AXIAL LOADS . . .
Axial Load at 8.50 ft, D = 19.0 k

DESIGN SUMMARY

Bending & Shear Check Results

PASS Max. Axial+Bending Stress Ratio = **0.3431** : 1

Load Combination	+D+H
Location of max.above base	0.0 ft
At maximum location values are . . .	
Pa : Axial	19.080 k
Pn / Omega : Allowable	55.617 k
Ma-x : Applied	0.0 k-ft
Mn-x / Omega : Allowable	8.228 k-ft
Ma-y : Applied	0.0 k-ft
Mn-y / Omega : Allowable	3.719 k-ft

PASS Maximum Shear Stress Ratio = **0.0** : 1

Load Combination	0.0
Location of max.above base	0.0 ft
At maximum location values are . . .	
Va : Applied	0.0 k
Vn / Omega : Allowable	0.0 k

Maximum Load Reactions . .

Top along X-X	0.0 k
Bottom along X-X	0.0 k
Top along Y-Y	0.0 k
Bottom along Y-Y	0.0 k

Maximum Load Deflections . . .

Along Y-Y 0.0 in at 0.0 ft above base
for load combination :

Along X-X 0.0 in at 0.0 ft above base
for load combination :

Load Combination Results

Load Combination	Maximum Axial + Bending Stress Ratios: Stress Ratio	Status	Location	Cbx	Cby	KxLx/Rx	KyLy/Ry	Maximum Shear Ratios: Stress Ratio	Status	Location
+D+H	0.343	PASS	0.00 ft	1.00	1.00	0.00	0.00	0.000	PASS	0.00 ft
+D+L+H	0.343	PASS	0.00 ft	1.00	1.00	0.00	0.00	0.000	PASS	0.00 ft
+D+Lr+H	0.343	PASS	0.00 ft	1.00	1.00	0.00	0.00	0.000	PASS	0.00 ft
+D+S+H	0.343	PASS	0.00 ft	1.00	1.00	0.00	0.00	0.000	PASS	0.00 ft
+D+0.750Lr+0.750L+H	0.343	PASS	0.00 ft	1.00	1.00	0.00	0.00	0.000	PASS	0.00 ft
+D+0.750L+0.750S+H	0.343	PASS	0.00 ft	1.00	1.00	0.00	0.00	0.000	PASS	0.00 ft
+D+0.60W+H	0.343	PASS	0.00 ft	1.00	1.00	0.00	0.00	0.000	PASS	0.00 ft
+D+0.70E+H	0.343	PASS	0.00 ft	1.00	1.00	0.00	0.00	0.000	PASS	0.00 ft
+D+0.750Lr+0.750L+0.450W+H	0.343	PASS	0.00 ft	1.00	1.00	0.00	0.00	0.000	PASS	0.00 ft
+D+0.750L+0.750S+0.450W+H	0.343	PASS	0.00 ft	1.00	1.00	0.00	0.00	0.000	PASS	0.00 ft
+D+0.750L+0.750S+0.5250E+H	0.343	PASS	0.00 ft	1.00	1.00	0.00	0.00	0.000	PASS	0.00 ft
+0.60D+0.60W+0.60H	0.206	PASS	0.00 ft	1.00	1.00	0.00	0.00	0.000	PASS	0.00 ft
+0.60D+0.70E+0.60H	0.206	PASS	0.00 ft	1.00	1.00	0.00	0.00	0.000	PASS	0.00 ft

Maximum Reactions

Note: Only non-zero reactions are listed.

Load Combination	Axial Reaction @ Base (k)	X-X Axis Reaction @ Base (k)	X-X Axis Reaction @ Top (k)	Y-Y Axis Reaction @ Base (k)	Y-Y Axis Reaction @ Top (k)	Mx - End Moments @ Base (k-ft)	Mx - End Moments @ Top (k-ft)	My - End Moments @ Base (k-ft)	My - End Moments @ Top (k-ft)
+D+H	19.080								
+D+L+H	19.080								

Steel Column

Lic. # : KW-06007473 SHAWN REEDER

DESCRIPTION: --None--

Maximum Reactions

Note: Only non-zero reactions are listed.

Load Combination	Axial Reaction @ Base	X-X Axis Reaction @ Base	@ Top	k	Y-Y Axis Reaction @ Base	@ Top	Mx - End Moments @ Base	@ Top	k-ft	My - End Moments @ Base	@ Top
+D+Lr+H	19.080										
+D+S+H	19.080										
+D+0.750Lr+0.750L+H	19.080										
+D+0.750L+0.750S+H	19.080										
+D+0.60W+H	19.080										
+D+0.70E+H	19.080										
+D+0.750Lr+0.750L+0.450W+H	19.080										
+D+0.750L+0.750S+0.450W+H	19.080										
+D+0.750L+0.750S+0.5250E+H	19.080										
+0.60D+0.60W+0.60H	11.448										
+0.60D+0.70E+0.60H	11.448										
D Only	19.080										
Lr Only											
L Only											
S Only											
W Only											
E Only											
H Only											

Extreme Reactions

Item	Extreme Value	Axial Reaction @ Base	X-X Axis Reaction @ Base	@ Top	k	Y-Y Axis Reaction @ Base	@ Top	Mx - End Moments @ Base	@ Top	k-ft	My - End Moments @ Base	@ Top
Axial @ Base	Maximum	19.080										
"	Minimum											
Reaction, X-X Axis Base	Maximum	19.080										
"	Minimum	19.080										
Reaction, Y-Y Axis Base	Maximum	19.080										
"	Minimum	19.080										
Reaction, X-X Axis Top	Maximum	19.080										
"	Minimum	19.080										
Reaction, Y-Y Axis Top	Maximum	19.080										
"	Minimum	19.080										
Moment, X-X Axis Base	Maximum	19.080										
"	Minimum	19.080										
Moment, Y-Y Axis Base	Maximum	19.080										
"	Minimum	19.080										
Moment, X-X Axis Top	Maximum	19.080										
"	Minimum	19.080										
Moment, Y-Y Axis Top	Maximum	19.080										
"	Minimum	19.080										

Maximum Deflections for Load Combinations

Load Combination	Max. X-X Deflection		Distance		Max. Y-Y Deflection		Distance	
+D+H	0.0000	in	0.000	ft	0.000	in	0.000	ft
+D+L+H	0.0000	in	0.000	ft	0.000	in	0.000	ft
+D+Lr+H	0.0000	in	0.000	ft	0.000	in	0.000	ft
+D+S+H	0.0000	in	0.000	ft	0.000	in	0.000	ft
+D+0.750Lr+0.750L+H	0.0000	in	0.000	ft	0.000	in	0.000	ft
+D+0.750L+0.750S+H	0.0000	in	0.000	ft	0.000	in	0.000	ft
+D+0.60W+H	0.0000	in	0.000	ft	0.000	in	0.000	ft
+D+0.70E+H	0.0000	in	0.000	ft	0.000	in	0.000	ft
+D+0.750Lr+0.750L+0.450W+H	0.0000	in	0.000	ft	0.000	in	0.000	ft
+D+0.750L+0.750S+0.450W+H	0.0000	in	0.000	ft	0.000	in	0.000	ft
+D+0.750L+0.750S+0.5250E+H	0.0000	in	0.000	ft	0.000	in	0.000	ft
+0.60D+0.60W+0.60H	0.0000	in	0.000	ft	0.000	in	0.000	ft
+0.60D+0.70E+0.60H	0.0000	in	0.000	ft	0.000	in	0.000	ft
D Only	0.0000	in	0.000	ft	0.000	in	0.000	ft
Lr Only	0.0000	in	0.000	ft	0.000	in	0.000	ft
L Only	0.0000	in	0.000	ft	0.000	in	0.000	ft

Steel Column

DESCRIPTION: --None--

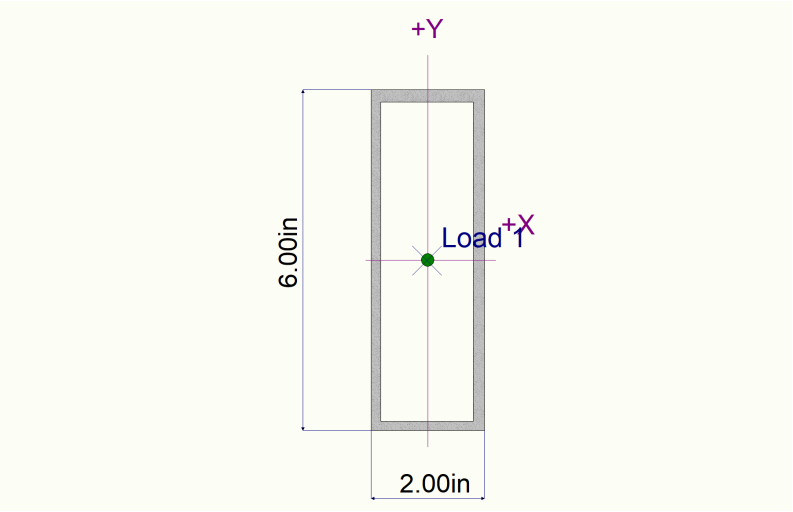
Maximum Deflections for Load Combinations

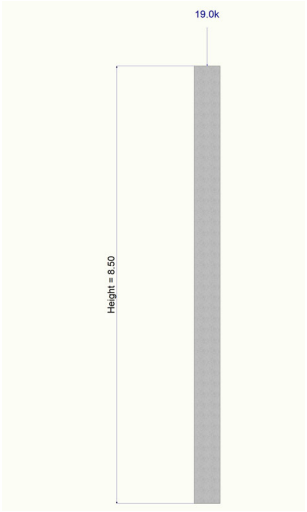
Load Combination	Max. X-X Deflection		Distance		Max. Y-Y Deflection		Distance	
S Only	0.0000	in	0.000	ft	0.000	in	0.000	ft
W Only	0.0000	in	0.000	ft	0.000	in	0.000	ft
E Only	0.0000	in	0.000	ft	0.000	in	0.000	ft
H Only	0.0000	in	0.000	ft	0.000	in	0.000	ft

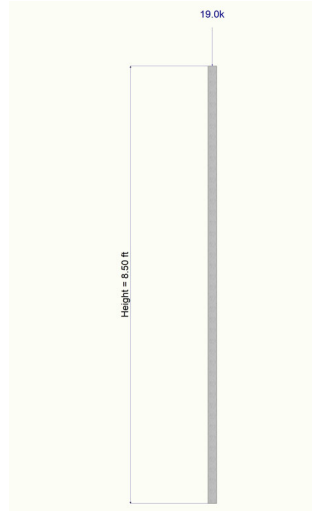
Steel Section Properties : HSS6x2x3/16

Depth	=	6.000	in	I xx	=	10.50	in^4	J	=	5.240	in^4
Design Thick	=	0.174	in	S xx	=	3.49	in^3	Cw	=	3.68	in^6
Width	=	2.000	in	R xx	=	2.010	in				
Wall Thick	=	0.187	in	Zx	=	4.580	in^3				
Area	=	2.580	in^2	I yy	=	1.800	in^4	C	=	3.680	in^3
Weight	=	9.426	plf	S yy	=	1.800	in^3				
				R yy	=	0.836	in				
				Zy	=	2.070	in^3				
Ycg	=	0.000	in								

Sketches







Wood Column

Lic. # : KW-06007473 SHAWN REEDER

DESCRIPTION: EndColumn

Code References

Calculations per NDS 2018, IBC 2018, CBC 2016, ASCE 7-16
Load Combinations Used : IBC 2018

General Information

Analysis Method :	Allowable Stress Design		
End Fixities	Top & Bottom Pinned		
Overall Column Height	10.1 ft		
(Used for non-slender calculations)			
Wood Species	DouglasFir-Larch		
Wood Grade	No.2		
Fb +	750.0 psi	Fv	170.0 psi
Fb -	750.0 psi	Ft	475.0 psi
Fc - Prll	700.0 psi	Density	31.20 pcf
Fc - Perp	625.0 psi		

E : Modulus of Elasticity . . .	x-x Bending	y-y Bending	Axial
Basic	1,300.0	1,300.0	1,300.0 ksi
Minimum	470.0	470.0	

Wood Section Name	**8x10**		
Wood Grading/Manuf.	Graded Lumber		
Wood Member Type	Sawn		
Exact Width	**7.50** in	Allow Stress Modification Factors	
Exact Depth	**9.50** in	Cf or Cv for Bending	1.0
Area	71.250 in^2	Cf or Cv for Compression	1.0
Ix	535.86 in^4	Cf or Cv for Tension	1.0
Iy	333.984 in^4	Cm : Wet Use Factor	1.0
		Ct : Temperature Factor	1.0
		Cfu : Flat Use Factor	1.0
		Kf : Built-up columns	1.0 *NDS 15.3.2*
		Use Cr : Repetitive ?	No

Brace condition for deflection (buckling) along columns :
X-X (width) axis : Fully braced against buckling ABOUT Y-Y Axis
Y-Y (depth) axis : Fully braced against buckling ABOUT X-X Axis

Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Column self weight included : 155.919 lbs * Dead Load Factor
AXIAL LOADS . . .
Axial Load at 10.10 ft, D = 18.450 k
BENDING LOADS . . .
Lat. Point Load at 9.0 ft creating My-y, D = 4.50 k
Lat. Point Load at 0.660 ft creating My-y, D = -4.50 k

DESIGN SUMMARY

Bending & Shear Check Results

PASS Max. Axial+Bending Stress Ratio = **0.5757 : 1**

Load Combination	+D+H
Governing NDS Forumla	Comp + Myy, NDS Eq. 3.9-3
Location of max.above base	8.948 ft
At maximum location values are . . .	
Applied Axial	18.606 k
Applied Mx	0.0 k-ft
Applied My	4.046 k-ft
Fc : Allowable	630.0 psi

PASS Maximum Shear Stress Ratio = **0.5113 : 1**

Load Combination	+D+H
Location of max.above base	10.10 ft
Applied Design Shear	78.228 psi
Allowable Shear	153.0 psi

Maximum SERVICE Lateral Load Reactions . .

Top along Y-Y	0.0 k	Bottom along Y-Y	0.0 k
Top along X-X	3.716 k	Bottom along X-X	3.716 k

Maximum SERVICE Load Lateral Deflections . . .

Along Y-Y 0.0 in at 0.0 ft above base
for load combination : n/a

Along X-X 0.06214 in at 6.982 ft above base
for load combination : +D+H

Other Factors used to calculate allowable stresses . . .

	Bending	Compression	Tension

Load Combination Results

Load Combination	C_D	C_P	Maximum Axial + Bending Stress Ratios: Stress Ratio	Status	Location	Maximum Shear Ratios: Stress Ratio	Status	Location
+D+H	0.900	1.000	0.5757	PASS	8.948 ft	0.5113	PASS	10.10 ft
+D+L+H	1.000	1.000	0.5026	PASS	8.948 ft	0.4602	PASS	10.10 ft
+D+Lr+H	1.250	1.000	0.3798	PASS	8.948 ft	0.3681	PASS	10.10 ft
+D+S+H	1.150	1.000	0.4213	PASS	8.948 ft	0.4001	PASS	10.10 ft
+D+0.750Lr+0.750L+H	1.250	1.000	0.3798	PASS	8.948 ft	0.3681	PASS	10.10 ft
+D+0.750L+0.750S+H	1.150	1.000	0.4213	PASS	8.948 ft	0.4001	PASS	10.10 ft
+D+0.60W+H	1.600	1.000	0.2815	PASS	8.948 ft	0.2876	PASS	10.10 ft
+D+0.70E+H	1.600	1.000	0.2815	PASS	8.948 ft	0.2876	PASS	10.10 ft

Wood Column

DESCRIPTION: EndColumn

Load Combination Results

Load Combination	C_D	C_P	Maximum Axial + Bending Stress Ratios			Maximum Shear Ratios		
			Stress Ratio	Status	Location	Stress Ratio	Status	Location
+D+0.750Lr+0.750L+0.450W+H	1.600	1.000	0.2815	PASS	8.948 ft	0.2876	PASS	10.10 ft
+D+0.750L+0.750S+0.450W+H	1.600	1.000	0.2815	PASS	8.948 ft	0.2876	PASS	10.10 ft
+D+0.750L+0.750S+0.5250E+H	1.600	1.000	0.2815	PASS	8.948 ft	0.2876	PASS	10.10 ft
+0.60D+0.60W+0.60H	1.600	1.000	0.1559	PASS	8.948 ft	0.1726	PASS	0.6101 ft
+0.60D+0.70E+0.60H	1.600	1.000	0.1559	PASS	8.948 ft	0.1726	PASS	0.6101 ft

Maximum Reactions

Note: Only non-zero reactions are listed.

Load Combination	X-X Axis Reaction k		Y-Y Axis Reaction k		Axial Reaction	My - End Moments k-ft		Mx - End Moments	
	@ Base	@ Top	@ Base	@ Top	@ Base	@ Base	@ Top	@ Base	@ Top
+D+H	-3.716	3.716			18.606				
+D+L+H	-3.716	3.716			18.606				
+D+Lr+H	-3.716	3.716			18.606				
+D+S+H	-3.716	3.716			18.606				
+D+0.750Lr+0.750L+H	-3.716	3.716			18.606				
+D+0.750L+0.750S+H	-3.716	3.716			18.606				
+D+0.60W+H	-3.716	3.716			18.606				
+D+0.70E+H	-3.716	3.716			18.606				
+D+0.750Lr+0.750L+0.450W+H	-3.716	3.716			18.606				
+D+0.750L+0.750S+0.450W+H	-3.716	3.716			18.606				
+D+0.750L+0.750S+0.5250E+H	-3.716	3.716			18.606				
+0.60D+0.60W+0.60H	-2.230	2.230			11.164				
+0.60D+0.70E+0.60H	-2.230	2.230			11.164				
D Only	-3.716	3.716			18.606				
Lr Only									
L Only									
S Only									
W Only									
E Only									
H Only									

Maximum Deflections for Load Combinations

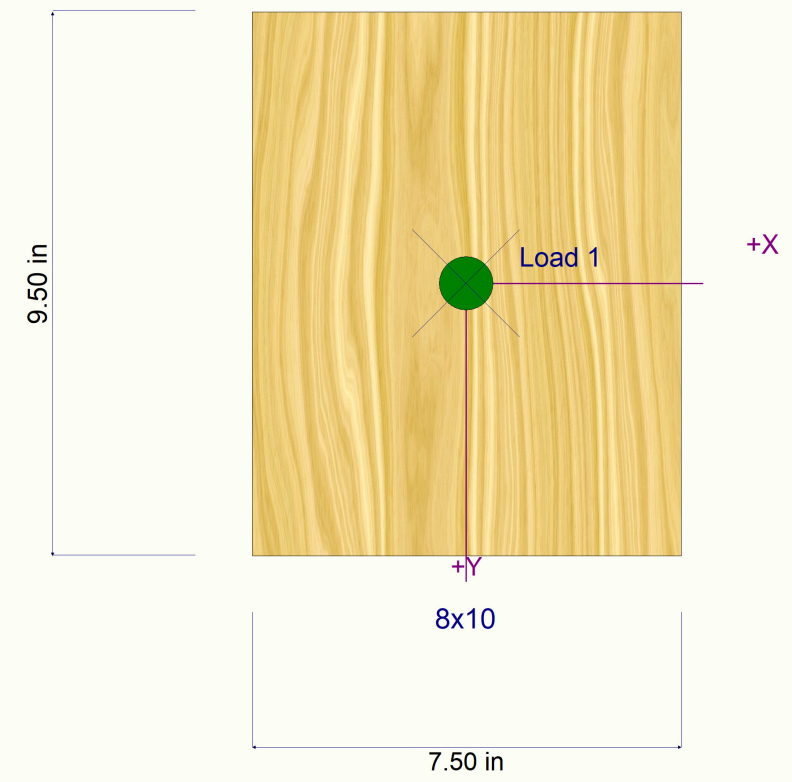
Load Combination	Max. X-X Deflection	Distance	Max. Y-Y Deflection	Distance
+D+H	0.0621 in	6.982 ft	0.000 in	0.000 ft
+D+L+H	0.0621 in	6.982 ft	0.000 in	0.000 ft
+D+Lr+H	0.0621 in	6.982 ft	0.000 in	0.000 ft
+D+S+H	0.0621 in	6.982 ft	0.000 in	0.000 ft
+D+0.750Lr+0.750L+H	0.0621 in	6.982 ft	0.000 in	0.000 ft
+D+0.750L+0.750S+H	0.0621 in	6.982 ft	0.000 in	0.000 ft
+D+0.60W+H	0.0621 in	6.982 ft	0.000 in	0.000 ft
+D+0.70E+H	0.0621 in	6.982 ft	0.000 in	0.000 ft
+D+0.750Lr+0.750L+0.450W+H	0.0621 in	6.982 ft	0.000 in	0.000 ft
+D+0.750L+0.750S+0.450W+H	0.0621 in	6.982 ft	0.000 in	0.000 ft
+D+0.750L+0.750S+0.5250E+H	0.0621 in	6.982 ft	0.000 in	0.000 ft
+0.60D+0.60W+0.60H	0.0373 in	6.982 ft	0.000 in	0.000 ft
+0.60D+0.70E+0.60H	0.0373 in	6.982 ft	0.000 in	0.000 ft
D Only	0.0621 in	6.982 ft	0.000 in	0.000 ft
Lr Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
L Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
S Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
W Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
E Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
H Only	0.0000 in	0.000 ft	0.000 in	0.000 ft

Wood Column

Lic. # : KW-06007473 SHAWN REEDER

DESCRIPTION: EndColumn

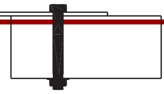
Sketches



BOLTS

Table 12B BOLTS: Reference Lateral Design Values, Z, for Single Shear (two member) Connections[1,2]

for sawn lumber or SCL main member with 1/4" ASTM A 36 steel side plate



Thickness: Main Member t_m in.	Thickness: Side Member t_s in.	Bolt Diameter D in.	G=0.67 Red Oak $Z_{\parallel}$ lbs.	G=0.67 Red Oak $Z_{\perp}$ lbs.	G=0.55 Mixed Maple Southern Pine $Z_{\parallel}$ lbs.	G=0.55 Mixed Maple Southern Pine $Z_{\perp}$ lbs.	G=0.50 Douglas Fir-Larch $Z_{\parallel}$ lbs.	G=0.50 Douglas Fir-Larch $Z_{\perp}$ lbs.	G=0.49 Douglas Fir-Larch(N) $Z_{\parallel}$ lbs.	G=0.49 Douglas Fir-Larch(N) $Z_{\perp}$ lbs.	G=0.46 Douglas Fir(S) Hem-Fir(N) $Z_{\parallel}$ lbs.	G=0.46 Douglas Fir(S) Hem-Fir(N) $Z_{\perp}$ lbs.	G=0.43 Hem-Fir $Z_{\parallel}$ lbs.	G=0.43 Hem-Fir $Z_{\perp}$ lbs.	G=0.42 Spruce-Pine-Fir $Z_{\parallel}$ lbs.	G=0.42 Spruce-Pine-Fir $Z_{\perp}$ lbs.	G=0.37 Redwood (open grain) $Z_{\parallel}$ lbs.	G=0.37 Redwood (open grain) $Z_{\perp}$ lbs.	G=0.36 Eastern Softwoods Spruce-Pine-Fir(S) Western Cedars Western Woods $Z_{\parallel}$ lbs.	G=0.36 Eastern Softwoods Spruce-Pine-Fir(S) Western Cedars Western Woods $Z_{\perp}$ lbs.	G=0.35 Northern Species $Z_{\parallel}$ lbs.	G=0.35 Northern Species $Z_{\perp}$ lbs.
1-1/2	1/4	1/2	730	420	620	350	580	310	580	310	550	290	520	280	510	270	470	240	460	240	450	230
		5/8	910	480	780	400	730	360	720	360	690	340	650	320	640	320	590	290	580	280	560	270
		3/4	1090	550	940	450	870	420	860	410	820	390	780	360	770	360	710	320	690	320	680	310
		7/8	1270	600	1090	510	1020	470	1010	450	960	430	910	410	900	400	820	370	810	360	790	350
		1	1460	660	1250	550	1170	510	1150	500	1100	480	1040	450	1030	450	940	400	930	400	900	390
1-3/4	1/4	1/2	810	460	690	370	640	340	630	330	600	310	570	290	560	280	510	250	500	250	490	240
		5/8	1020	520	870	430	800	390	790	380	750	360	710	340	700	330	640	300	630	290	610	280
		3/4	1220	590	1040	480	960	440	950	430	900	410	860	380	840	370	770	330	750	330	730	320
		7/8	1420	650	1210	540	1130	490	1110	480	1050	450	1000	420	980	420	890	380	880	370	850	360
		1	1630	710	1380	580	1290	540	1270	520	1200	500	1140	470	1120	460	1020	410	1000	410	980	400
2-1/2	1/4	1/2	930	600	860	470	830	410	820	400	780	380	740	350	720	340	650	300	640	290	620	280
		5/8	1370	670	1150	530	1050	470	1040	470	980	430	920	400	910	390	810	340	800	330	770	320
		3/4	1640	750	1370	590	1270	530	1250	520	1180	490	1110	450	1090	440	980	380	960	370	930	360
		7/8	1910	820	1600	650	1480	590	1450	570	1370	530	1290	490	1270	480	1140	420	1120	410	1080	400
		1	2190	880	1830	700	1690	640	1660	620	1570	580	1480	540	1450	530	1300	460	1280	450	1240	440
3-1/2	1/4	1/2	930	620	860	550	830	510	820	510	800	480	770	450	770	430	720	370	720	360	710	350
		5/8	1370	860	1260	690	1210	610	1200	600	1160	550	1130	500	1120	490	1060	420	1050	410	1020	400
		3/4	1900	990	1740	760	1670	680	1660	660	1580	610	1480	560	1450	540	1290	460	1260	450	1220	440
		7/8	2530	1070	2170	840	1990	740	1950	710	1840	660	1720	610	1690	590	1510	510	1480	500	1430	470
		1	2980	1150	2480	890	2270	800	2230	770	2100	730	1970	660	1930	650	1720	560	1690	540	1630	530
5-1/4	1/4	5/8	1370	860	1260	760	1210	710	1200	700	1160	670	1130	640	1120	630	1060	580	1050	560	1030	540
		3/4	1900	1140	1740	1000	1670	940	1660	930	1610	860	1560	770	1550	760	1460	640	1450	620	1420	600
		7/8	2530	1460	2320	1190	2220	1050	2200	1010	2140	920	2070	840	2050	820	1940	700	1920	680	1890	640
		1	3260	1660	2980	1270	2860	1130	2840	1080	2750	1010	2670	920	2640	890	2490	750	2450	730	2360	710
5-1/2	1/4	5/8	1370	860	1260	760	1210	710	1200	700	1160	670	1130	640	1120	630	1060	580	1050	570	1030	560
		3/4	1900	1140	1740	1000	1670	940	1660	930	1610	890	1560	810	1550	790	1460	660	1450	640	1420	620
		7/8	2530	1460	2320	1240	2220	1090	2200	1050	2140	960	2070	880	2050	860	1940	730	1920	710	1890	660
		1	3260	1730	2980	1320	2860	1170	2840	1130	2750	1050	2670	950	2640	930	2490	780	2470	760	2420	740
7-1/2	1/4	5/8	1370	860	1260	760	1210	710	1200	700	1160	670	1130	640	1120	630	1060	580	1050	570	1030	560
		3/4	1900	1140	1740	1000	1670	940	1660	930	1610	890	1560	850	1550	840	1460	760	1450	750	1420	740
		7/8	2530	1460	2320	1280	2220	1210	2200	1180	2140	1130	2070	1080	2050	1070	1940	960	1920	930	1890	870
		1	3260	1820	2980	1590	2860	1500	2840	1470	2750	1400	2670	1270	2640	1230	2490	1030	2470	1000	2420	960
9-1/2	1/4	3/4	1900	1140	1740	1000	1670	940	1660	930	1610	890	1560	850	1550	840	1460	760	1450	750	1420	740
		7/8	2530	1460	2320	1280	2220	1210	2200	1180	2140	1130	2070	1080	2050	1070	1940	980	1920	970	1890	930
		1	3260	1820	2980	1590	2860	1500	2840	1470	2750	1420	2670	1350	2640	1330	2490	1220	2470	1200	2420	1180
11-1/2	1/4	7/8	2530	1460	2320	1280	2220	1210	2200	1180	2140	1130	2070	1080	2050	1070	1940	980	1920	970	1890	930
		1	3260	1820	2980	1590	2860	1500	2840	1470	2750	1420	2670	1350	2640	1330	2490	1220	2470	1200	2420	1180
13-1/2	1/4	1	3260	1820	2980	1590	2860	1500	2840	1470	2750	1420	2670	1350	2640	1330	2490	1220	2470	1200	2420	1180

1. Tabulated lateral design values, Z, for bolted connections shall be multiplied by all applicable adjustment factors (see Table 11.3.1).
2. Tabulated lateral design values, Z, are for "full-body diameter" bolts (see Appendix Table L1) with bolt bending yield strength, F_{yb}, of 45,000 psi and dowel bearing strength, F_e, of 87,000 psi for ASTM A36 steel.

1670*12=20,040

Location:

Weld Calculation

Capacity of Weld Parallel to Beam

Fexx	70	ksi	
Angle of Weld to Load	0	degrees	
Sin of Angle	0		
Fw	42	ksi	
Safety Factor	2		
Length of Weld	4	in	
Size of Weld	0.19	in	
Weld Capacity	15750	#	
Number Welds	1		
Moment Arm	8	inches	(Width of Beam)
Weld Moment Capacity	10500	ft-#	

Capacity of Weld Perpendicular to Beam

Fexx	70	ksi	
Angle of Weld to Load	0	degrees	
Sin of Angle	0		
Fw	42	ksi	
Safety Factor	2		
Length of Weld	8	in	
Size of Weld	0.19	in	
Weld Capacity	31500	#	
Number Welds	2		(One on Each Side of Column)
Moment Arm	4	inches	(Half Width of Column)
Weld Moment Capacity	21000	ft-#	
Total Moment Capacity	31500	ft-#	
Actual Moment	62500	ft-#	

Therefore, with a 3/16" fillet Weld on one side of each member is sufficient.