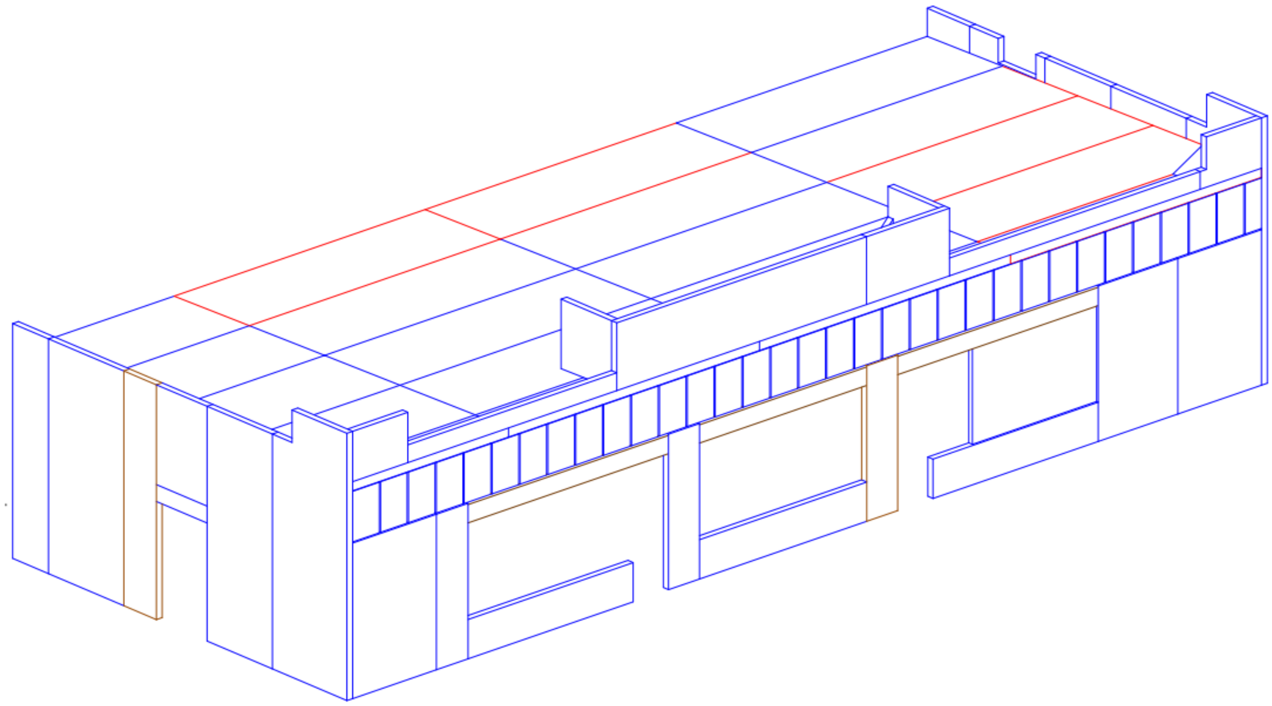
Pinnacle Engineering, Inc.

Structural Calculations for:

Circle K Main Building – Angier, NC



Angier, NC

January 31, 2023

8180 Corporate Park Drive ▲ Suite 235 ▲ Cincinnati, Ohio 45242 ▲ (513) 984-1663 ▲ Fax (513) 984-1688

Pinnacle Engineering, Inc.

Index of Calculations

Note - Engineer’s seal on this calculation booklet only applies to calculations and information provided by Pinnacle Engineering, Inc. It does not apply to manufacturer information and documentation (pages 46 through 59). The engineering data provided only applies to PorterSIPs panel systems.

NORTH CAROLINA PROFESSIONAL SEAL 028089 ENGINEER JAMES KIPTON PING

This item was digitally signed and sealed by James Kipton Ping on 2023.02.16 14:39:00 -05'00'

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GENERAL DESIGN INFORMATION

PROJECT DESCRIPTION

A one story SIP structure with pre-fabricated wood trusses and concrete footings

STORE BUILDING PARAMETERS

Building Width	Width = 35.5 ft
Building Length	Length = 88 ft
Roof Slope (per foot)	$Slope_{12}$ = 0.25:12
Roof slope (degrees)	$Slope_{Deg}$ = **1.19°**
Eave height	h_{eave} = 16.75 ft
Parapet height (from eave height)	h_p = 3.917 ft
Height for wind calcs (mean or t/parapet)	h_w = 20.667

TYPICAL LOAD PARAMETERS

Occupancy Category	II
Ground Snow	P_g = 15 psf
Wind Speed	116 mph
Terrain Category/Exposure	C
Mapped Spectral Response (0.2 second)	S_S = 0.172
Mapped Spectral Response (1 second)	S_1 = 0.083
Seismic Site Class	D

TYPICAL MATERIALS

Concrete (strengths at 28 days):

Footings:	3000 psi
Walls:	4000 psi
Slabs, interior:	4000 psi
Slabs, exterior:	4500 psi
Rebar:	ASTM A615 or A996, Grade 60, deformed

Rough Carpentry

2x4 and 2x6:	Spruce Pine Fir, Stud Grade
2x8, 2x10, 2x12:	Southern Pine, No. 2 Grade
LVL:	F_b = 2600 psi, E = 1800 ksi
LSL:	F_b = 1700 psi, E = 1300 ksi
Bolts:	ASTM A307

Structurally Insulated Panels (SIPs)

6.5" Panels:	5.625 in Core Thickness
8.25" Panels:	7.375 in Core Thickness

REFERENCES

- NCBC-2018, *North Carolina State Building Code*
- ASCE 7-10, *Minimum Design Loads for Buildings and Other Structures*
- ACI 318-14, *Building Code Requirements for Structural Concrete*
- ANSI/AF&PA NDS-2015, *National Design Specification (NDS) for Wood Construction*
- PorterCorp ESR-4692 issued April 2022

SERVICEABILITY LIMITS

DEFLECTION LIMITS:

Roof members:	
Live, Snow, or Wind Load	L /360
Total Load	L /240
Members supporting masonry:	
Live, Snow, or Wind Load	L / 600
Total Load	L / 400

LATERAL DRIFT LIMITS:

Wind Load Drift Limit (as calculated under ultimate load per ASCE7)

Overal building drift limit: H / 168 (equivalent to H / 400 at 0.7 of 0.6 of ultimate load load)

Seismic Load Drift Limit (as calcluated under ultimate load per ASCE7)

Interstory drift limit: 0.025 x H (Per ASCE 7, Table 12.12-1)

DEAD LOADING

DEAD LOAD CONSTRUCTION

Roof

Material	Thickness (in)	γ (lb/ft^3)	Weight (lb/ft^2)
40mil Dura-Last	0.040		0.3
8.25" SIP	8.250		3.6
FRP Ceiling	0.100		0.8
Sprinklers	0.000		3.0
HVAC	0.000		3.0
Misc.	0.000		2.0
Wood Truss	40.000		3.0
Plasterboard	0.250	60	1.2
Totals	**48.640**		**17.0**

Exterior Walls

Material	Thickness (in)	γ (lb/ft^3)	Weight (lb/ft^2)
EnduraWall	0.500		3.0
6.5" SIP	6.500		3.6
FRP	0.100		0.8
Nichiha Panels	0.625		1.0
Totals	**7.725**		**8.4**

Interior Walls

Material	Thickness (in)	γ (lb/ft^3)	Weight (lb/ft^2)
Framing	5.500		2.0
Plasterboard	1.250	60	6.3
Totals	**6.750**		**8.3**

LIVE LOADS

FLOOR LIVE LOAD (BUILDING CODE SECTION 1603.1.1):

Restaraunts	100 psf

ROOF LIVE LOAD (BUILDING CODE SECTION 1603.1.2):

Non-Occupied Areas	20 psf (reduced where applicable per building code section 1607.11.2)

SNOW LOADING (LENGTH)

SNOW LOADING

In accordance with ASCE7-10

Tedds calculation version 1.0.10

Building details

Roof type	Flat
Width of roof	$b = \mathbf{88.00}$ ft

Ground snow load

Ground snow load (Figure 7-1)	$p_g = \mathbf{15.00}$ lb/ft^2
Density of snow	$\gamma = \min(0.13 \times p_g / 1\text{ft} + 14\text{lb/ft}^3, 30\text{lb/ft}^3) = \mathbf{15.95}$ lb/ft^3
Terrain typeSect. 26.7	C
Exposure condition (Table 7-2)	Partially exposed
Exposure factor (Table 7-2)	$C_e = \mathbf{1.00}$
Thermal condition (Table 7-3)	Others with cold roofs
Thermal factor (Table 7-3)	$C_t = \mathbf{1.10}$
Importance category (Table 1.5-1)	II
Importance factor (Table 1.5-2)	$I_s = \mathbf{1.00}$
Min snow load for low slope roofs (Sect 7.3.4)	$p_{f\_min} = I_s \times p_g = \mathbf{15.00}$ lb/ft^2
Flat roof snow load (Sect 7.3)	$p_f = 0.7 \times C_e \times C_t \times I_s \times p_g = \mathbf{11.55}$ lb/ft^2

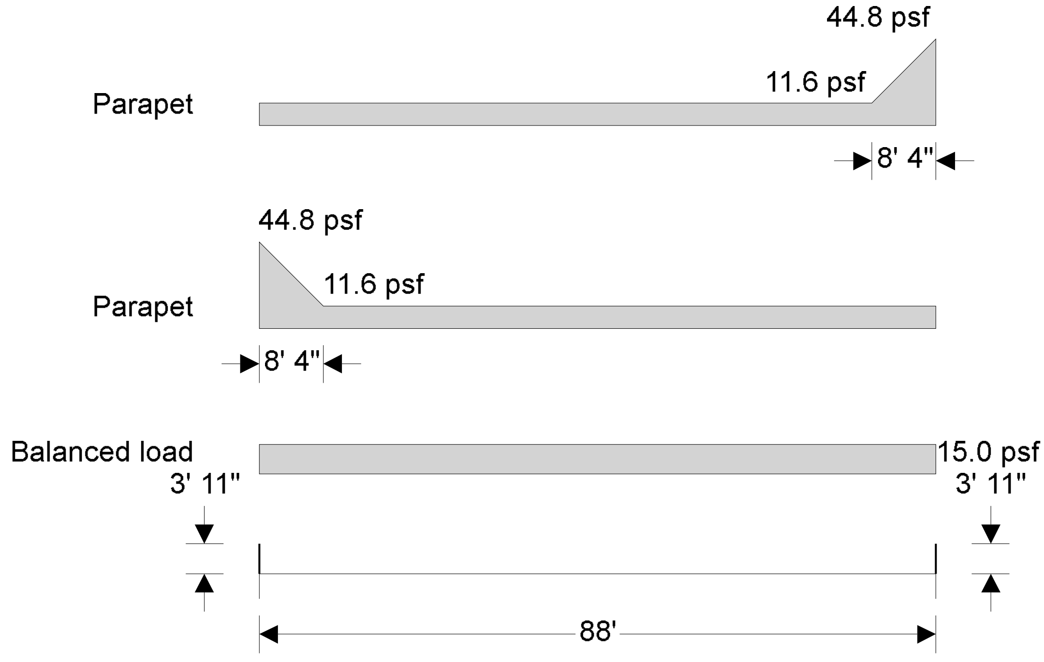
Left parapet

Balanced snow load height	$h_b = p_f / \gamma = \mathbf{0.72}$ ft
Height of left parapet	$h_{pptL} = \mathbf{3.92}$ ft
Height from balance load to top of left parapet	$h_{c\_pptL} = h_{pptL} - h_b = \mathbf{3.19}$ ft
Length of roof - left parapet	$l_{u\_pptL} = b = \mathbf{88.00}$ ft
Drift height windward drift - left parpet	$h_{d\_l\_pptL} = 0.75 \times (0.43 \times (\max(20\text{ ft}, l_{u\_pptL}) \times 1\text{ft}^2)^{1/3} \times (p_g / 1\text{lb/ft}^2 + 10)^{1/4} - 1.5\text{ft}) = \mathbf{2.08}$ ft
Drift height - left parapet	$h_{d\_pptL} = \min(h_{d\_l\_pptL}, h_{pptL} - h_b) = \mathbf{2.08}$ ft
Drift width	$W_{d\_pptL} = \min(4 \times h_{d\_l\_pptL}, 8 \times (h_{pptL} - h_b), b) = \mathbf{8.33}$ ft
Drift surcharge load - left parapet	$p_{d\_pptL} = h_{d\_pptL} \times \gamma = \mathbf{33.22}$ lb/ft^2

Right parapet

Height of right parapet	$h_{pptR} = \mathbf{3.92}$ ft
Height from balance load to top of right parapet	$h_{c\_pptR} = h_{pptR} - h_b = \mathbf{3.19}$ ft
Length of roof - right parapet	$l_{u\_pptR} = b = \mathbf{88.00}$ ft
Drift height windward drift - right parpet	$h_{d\_l\_pptR} = 0.75 \times (0.43 \times (\max(20\text{ ft}, l_{u\_pptR}) \times 1\text{ft}^2)^{1/3} \times (p_g / 1\text{lb/ft}^2 + 10)^{1/4} - 1.5\text{ft}) = \mathbf{2.08}$ ft
Drift height - right parapet	$h_{d\_pptR} = \min(h_{d\_l\_pptR}, h_{pptR} - h_b) = \mathbf{2.08}$ ft

Drift width	$W_{d\_pptR} = \min(4 \times h_{d\_l\_pptR}, 8 \times (h_{pptR} - h_b), b) =$ **8.33** ft
Drift surcharge load - right parapet	$p_{d\_pptR} = h_{d\_pptR} \times \gamma =$ **33.22** lb/ft^2



Roof elevation

SNOW LOADING (WIDTH)

SNOW LOADING

In accordance with ASCE7-10

Tedds calculation version 1.0.10

Building details

Roof type	Flat
Width of roof	b = **35.50** ft

Ground snow load

Ground snow load (Figure 7-1)	$p_g =$ **15.00** lb/ft^2
Density of snow	$\gamma = \min(0.13 \times p_g / 1\text{ft} + 14\text{lb/ft}^3, 30\text{lb/ft}^3) =$ **15.95** lb/ft^3
Terrain typeSect. 26.7	C
Exposure condition (Table 7-2)	Partially exposed
Exposure factor (Table 7-2)	$C_e =$ **1.00**
Thermal condition (Table 7-3)	Others with cold roofs
Thermal factor (Table 7-3)	$C_t =$ **1.10**
Importance category (Table 1.5-1)	II
Importance factor (Table 1.5-2)	$I_s =$ **1.00**
Min snow load for low slope roofs (Sect 7.3.4)	$p_{f\_min} = I_s \times p_g =$ **15.00** lb/ft^2
Flat roof snow load (Sect 7.3)	$p_f = 0.7 \times C_e \times C_t \times I_s \times p_g =$ **11.55** lb/ft^2

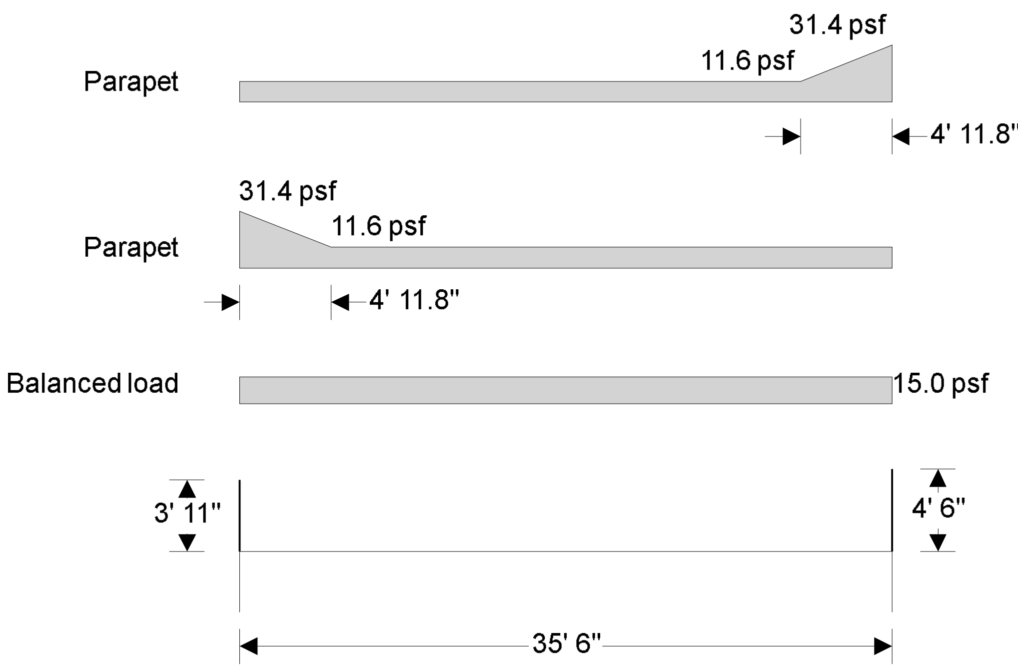
Left parapet

Balanced snow load height	$h_b = p_f / \gamma =$ **0.72** ft

Height of left parapet	h_{pptL} = **3.92** ft
Height from balance load to top of left parapet	$h_{c\_pptL} = h_{pptL} - h_b$ = **3.19** ft
Length of roof - left parapet	l_{u_pptL} = b = **35.50** ft
Drift height windward drift - left parpet	$h_{d\_l\_pptL} = 0.75 \times (0.43 \times (\max(20\text{ ft}, l_{u\_pptL}) \times 1\text{ft}^2)^{1/3} \times (p_g / 1\text{lb/ft}^2 + 10)^{1/4} - 1.5\text{ft})$ = **1.25** ft
Drift height - left parapet	$h_{d\_pptL} = \min(h_{d\_l\_pptL}, h_{pptL} - h_b)$ = **1.25** ft
Drift width	$W_{d\_pptL} = \min(4 \times h_{d\_l\_pptL}, 8 \times (h_{pptL} - h_b), b)$ = **4.98** ft
Drift surcharge load - left parapet	$p_{d\_pptL} = h_{d\_pptL} \times \gamma$ = **19.86** lb/ft^2

Right parapet

Height of right parapet	h_{pptR} = **4.50** ft
Height from balance load to top of right parapet	$h_{c\_pptR} = h_{pptR} - h_b$ = **3.78** ft
Length of roof - right parapet	l_{u_pptR} = b = **35.50** ft
Drift height windward drift - right parpet	$h_{d\_l\_pptR} = 0.75 \times (0.43 \times (\max(20\text{ ft}, l_{u\_pptR}) \times 1\text{ft}^2)^{1/3} \times (p_g / 1\text{lb/ft}^2 + 10)^{1/4} - 1.5\text{ft})$ = **1.25** ft
Drift height - right parapet	$h_{d\_pptR} = \min(h_{d\_l\_pptR}, h_{pptR} - h_b)$ = **1.25** ft
Drift width	$W_{d\_pptR} = \min(4 \times h_{d\_l\_pptR}, 8 \times (h_{pptR} - h_b), b)$ = **4.98** ft
Drift surcharge load - right parapet	$p_{d\_pptR} = h_{d\_pptR} \times \gamma$ = **19.86** lb/ft^2



Roof elevation

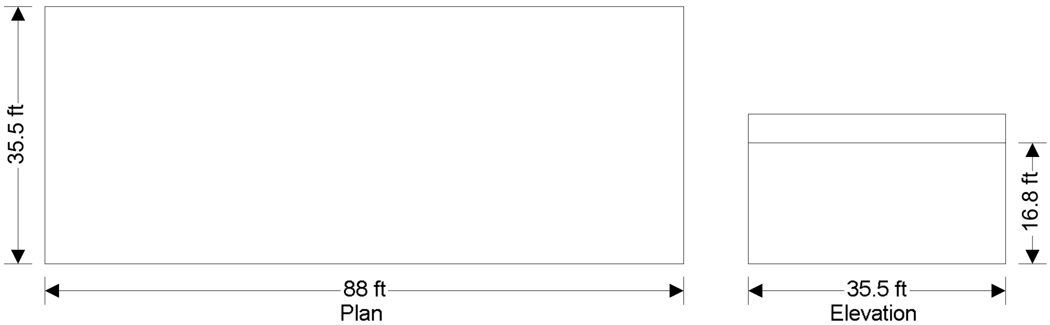
WIND LOADING (MWFRS)

WIND LOADING

In accordance with ASCE7-10

Using the directional design method

Tedds calculation version 2.1.12



Building data

Type of roof	Flat
Length of building	b = **88.00** ft
Width of building	d = **35.50** ft
Height to eaves	H = **16.75** ft
Height of parapet	h_p = **3.92** ft
Mean height	h = **16.75** ft

General wind load requirements

Basic wind speed	V = **116.0** mph
Risk category	II
Velocity pressure exponent coef (Table 26.6-1)	K_d = **0.85**
Exposure category (cl 26.7.3)	C
Enclosure classification (cl.26.10)	Enclosed buildings
Internal pressure coef +ve (Table 26.11-1)	GC_{pi_p} = **0.18**
Internal pressure coef –ve (Table 26.11-1)	GC_{pi_n} = **-0.18**
Gust effect factor	G_f = **0.85**
Minimum design wind loading (cl.27.1.5)	p_{min_r} = **8** lb/ft^2

Topography

Topography factor not significant	K_{zt} = 1.0
Velocity pressure equation	$q = 0.00256 \times K_z \times K_{zt} \times K_d \times V^2 \times 1psf/mph^2$

Velocity pressures table

z (ft)	K_z (Table 27.3-1)	q_z (psf)
15.00	0.85	24.89
16.75	0.87	25.40
20.67	0.91	26.51

Peak velocity pressure for internal pressure

Peak velocity pressure – internal (as roof press.)	q_i = **25.40** psf

Parapet pressures and forces

Velocity pressure at top of parapet	q_p = **26.51** psf
Combined net pressure coefficient, leeward	GC_{pnl} = **-1.0**
Combined net parapet pressure, leeward	$p_{pl} = q_p \times GC_{pnl}$ = **-26.51** psf
Combined net pressure coefficient, windward	GC_{pnw} = **1.5**
Combined net parapet pressure, windward	$p_{pw} = q_p \times GC_{pnw}$ = **39.76** psf
Wind direction 0 deg:	
Leeward parapet force	$F_{w,wpl\_0} = p_{pl} \times h_p \times b$ = **-9.1** kips
Windward parapet force	$F_{w,wpw\_0} = p_{pw} \times h_p \times b$ = **13.7** kips
Wind direction 90 deg:	
Leeward parapet force	$F_{w,wpl\_90} = p_{pl} \times h_p \times d$ = **-3.7** kips

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Windward parapet force $F_{w,wpw\_90} = p_{pw} \times h_p \times d =$ **5.5** kips

Pressures and forces

Net pressure $p = q \times G_f \times C_{pe} - q_i \times GC_{pi}$

Net force $F_w = p \times A_{ref}$

Roof load case 1 - Wind 0, GC_{pi} 0.18, $-c_{pe}$

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
A (-ve)	16.75	-0.90	25.40	-24.00	737.00	-17.69
B (-ve)	16.75	-0.90	25.40	-24.00	737.00	-17.69
C (-ve)	16.75	-0.50	25.40	-15.37	1474.00	-22.65
D (-ve)	16.75	-0.30	25.40	-11.05	176.00	-1.94

Total vertical net force $F_{w,v} =$ **-59.98** kips

Total horizontal net force $F_{w,h} =$ **0.00** kips

Walls load case 1 - Wind 0, GC_{pi} 0.18, $-c_{pe}$

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
A_1	15.00	0.80	24.89	12.35	1320.00	16.30
A_2	16.75	0.80	25.40	12.70	154.00	1.96
B	16.75	-0.50	25.40	-15.37	1474.00	-22.65
C	16.75	-0.70	25.40	-19.69	594.63	-11.71
D	16.75	-0.70	25.40	-19.69	594.63	-11.71

Overall loading

Projected vertical plan area of wall $A_{vert\_w\_0} = b \times (H + h_p) =$ **1818.70** ft^2

Projected vertical area of roof $A_{vert\_r\_0} =$ **0.00** ft^2

Minimum overall horizontal loading $F_{w,total\_min} = p_{min\_w} \times A_{vert\_w\_0} + p_{min\_r} \times A_{vert\_r\_0} =$ **29.10** kips

Leeward net force $F_l = F_{w,wB} + F_{w,wpl\_0} =$ **-31.8** kips

Windward net force $F_w = F_{w,wA\_1} + F_{w,wA\_2} + F_{w,wpw\_0} =$ **32.0** kips

Overall horizontal loading $F_{w,total} = \max(F_w - F_l + F_{w,h}, F_{w,total\_min}) =$ **63.8** kips

Roof load case 2 - Wind 0, GC_{pi} -0.18, $-1c_{pe}$

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
A (+ve)	16.75	-0.18	25.40	0.69	737.00	0.51
B (+ve)	16.75	-0.18	25.40	0.69	737.00	0.51
C (+ve)	16.75	-0.18	25.40	0.69	1474.00	1.01
D (+ve)	16.75	-0.18	25.40	0.69	176.00	0.12

Total vertical net force $F_{w,v} =$ **2.14** kips

Total horizontal net force $F_{w,h} =$ **0.00** kips

Walls load case 2 - Wind 0, GC_{pi} -0.18, $-1c_{pe}$

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
A_1	15.00	0.80	24.89	21.50	1320.00	28.37

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft²)	Net force F_w (kips)
A_2	16.75	0.80	25.40	21.84	154.00	3.36
B	16.75	-0.50	25.40	-6.22	1474.00	-9.17
C	16.75	-0.70	25.40	-10.54	594.63	-6.27
D	16.75	-0.70	25.40	-10.54	594.63	-6.27

Overall loading

Projected vertical plan area of wall — $A_{vert\_w\_0} = b \times (H + h_p) = \mathbf{1818.70}$ ft²

Projected vertical area of roof — $A_{vert\_r\_0} = \mathbf{0.00}$ ft²

Minimum overall horizontal loading — $F_{w,total\_min} = p_{min\_w} \times A_{vert\_w\_0} + p_{min\_r} \times A_{vert\_r\_0} = \mathbf{29.10}$ kips

Leeward net force — $F_l = F_{w,wB} + F_{w,wpl\_0} = \mathbf{-18.3}$ kips

Windward net force — $F_w = F_{w,wA\_1} + F_{w,wA\_2} + F_{w,wpw\_0} = \mathbf{45.4}$ kips

Overall horizontal loading — $F_{w,total} = \max(F_w - F_l + F_{w,h}, F_{w,total\_min}) = \mathbf{63.8}$ kips

Roof load case 3 - Wind 90, GC_{pi} 0.18, $-c_{pe}$

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft²)	Net force F_w (kips)
A (-ve)	16.75	-0.90	25.40	-24.00	297.31	-7.14
B (-ve)	16.75	-0.90	25.40	-24.00	297.31	-7.14
C (-ve)	16.75	-0.50	25.40	-15.37	594.63	-9.14
D (-ve)	16.75	-0.30	25.40	-11.05	1934.75	-21.38

Total vertical net force — $F_{w,v} = \mathbf{-44.79}$ kips

Total horizontal net force — $F_{w,h} = \mathbf{0.00}$ kips

Walls load case 3 - Wind 90, GC_{pi} 0.18, $-c_{pe}$

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft²)	Net force F_w (kips)
A_1	15.00	0.80	24.89	12.35	532.50	6.58
A_2	16.75	0.80	25.40	12.70	62.13	0.79
B	16.75	-0.28	25.40	-10.53	594.63	-6.26
C	16.75	-0.70	25.40	-19.69	1474.00	-29.02
D	16.75	-0.70	25.40	-19.69	1474.00	-29.02

Overall loading

Projected vertical plan area of wall — $A_{vert\_w\_90} = d \times (H + h_p) = \mathbf{733.68}$ ft²

Projected vertical area of roof — $A_{vert\_r\_90} = \mathbf{0.00}$ ft²

Minimum overall horizontal loading — $F_{w,total\_min} = p_{min\_w} \times A_{vert\_w\_90} + p_{min\_r} \times A_{vert\_r\_90} = \mathbf{11.74}$ kips

Leeward net force — $F_l = F_{w,wB} + F_{w,wpl\_90} = \mathbf{-9.9}$ kips

Windward net force — $F_w = F_{w,wA\_1} + F_{w,wA\_2} + F_{w,wpw\_90} = \mathbf{12.9}$ kips

Overall horizontal loading — $F_{w,total} = \max(F_w - F_l + F_{w,h}, F_{w,total\_min}) = \mathbf{22.8}$ kips

Roof load case 4 - Wind 90, GC_{pi} -0.18, $+c_{pe}$

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft²)	Net force F_w (kips)
A (+ve)	16.75	-0.18	25.40	0.69	297.31	0.20

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
B (+ve)	16.75	-0.18	25.40	0.69	297.31	0.20
C (+ve)	16.75	-0.18	25.40	0.69	594.63	0.41
D (+ve)	16.75	-0.18	25.40	0.69	1934.75	1.33

Total vertical net force $F_{w,v}$ = **2.14** kips

Total horizontal net force $F_{w,h}$ = **0.00** kips

Walls load case 4 - Wind 90, GC_{pi} -0.18, +c_{pe}

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
A_1	15.00	0.80	24.89	21.50	532.50	11.45
A_2	16.75	0.80	25.40	21.84	62.13	1.36
B	16.75	-0.28	25.40	-1.39	594.63	-0.83
C	16.75	-0.70	25.40	-10.54	1474.00	-15.54
D	16.75	-0.70	25.40	-10.54	1474.00	-15.54

Overall loading

Projected vertical plan area of wall $A_{vert\_w\_90} = d \times (H + h_p) = \mathbf{733.68}\ ft^2$

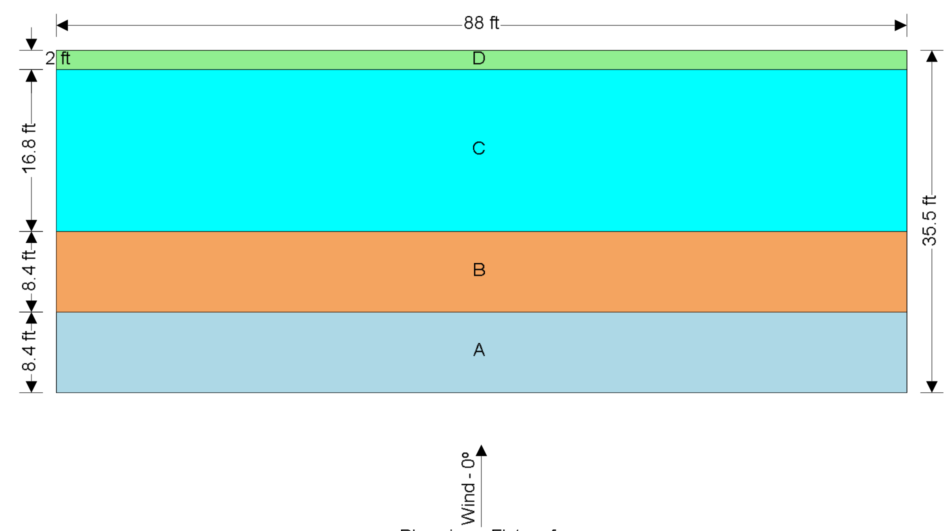
Projected vertical area of roof $A_{vert\_r\_90} = \mathbf{0.00}\ ft^2$

Minimum overall horizontal loading $F_{w,total\_min} = p_{min\_w} \times A_{vert\_w\_90} + p_{min\_r} \times A_{vert\_r\_90} = \mathbf{11.74}$ kips

Leeward net force $F_l = F_{w,wB} + F_{w,wpl\_90} = \mathbf{-4.5}$ kips

Windward net force $F_w = F_{w,wA\_1} + F_{w,wA\_2} + F_{w,wpw\_90} = \mathbf{18.3}$ kips

Overall horizontal loading $F_{w,total} = \max(F_w - F_l + F_{w,h}, F_{w,total\_min}) = \mathbf{22.8}$ kips



Plan view - Flat roof

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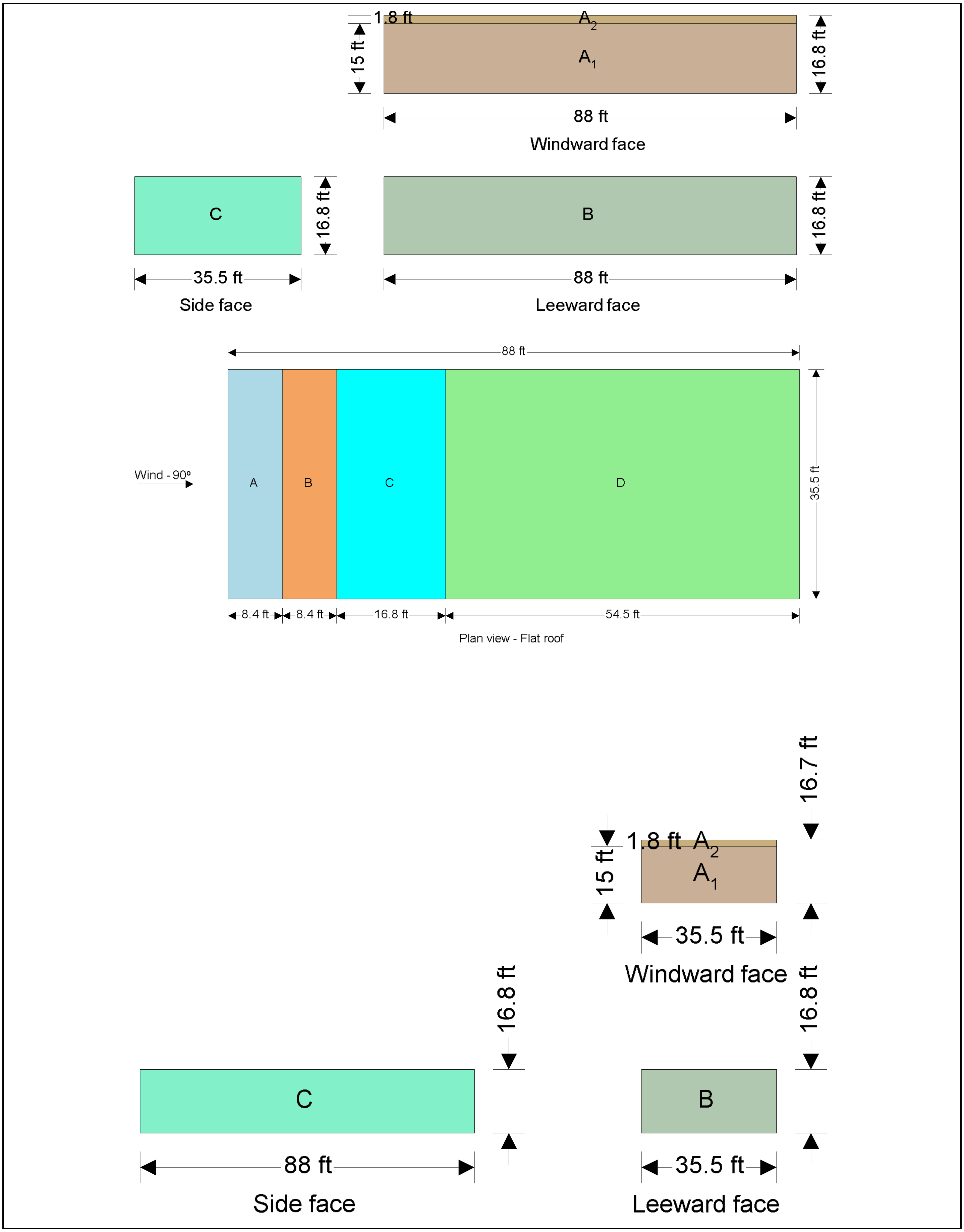

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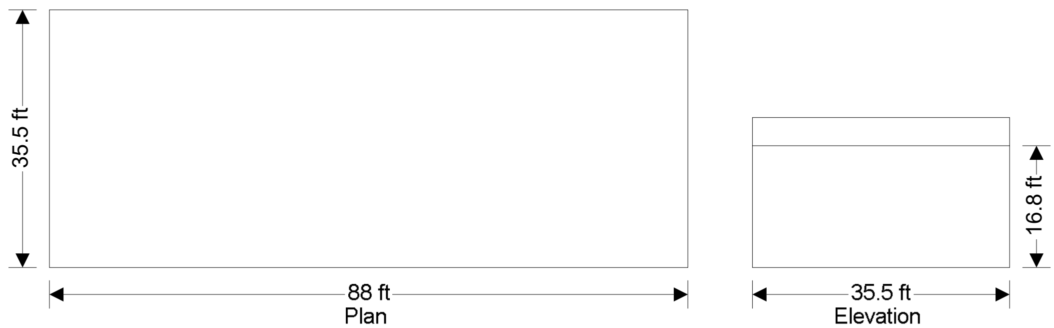
WIND LOAD (C&C - STORE)

WIND LOADING

In accordance with ASCE7-10

Using the components and cladding design method

Tedds calculation version 2.1.12



Building data

Type of roof	Flat
Length of building	b = **88.00** ft
Width of building	d = **35.50** ft
Height to eaves	H = **16.75** ft
Height of parapet	h_p = **3.92** ft
Mean height	h = **16.75** ft

General wind load requirements

Basic wind speed	V = **116.0** mph
Risk category	II
Velocity pressure exponent coef (Table 26.6-1)	K_d = **0.85**
Exposure category (cl 26.7.3)	C
Enclosure classification (cl.26.10)	Enclosed buildings
Internal pressure coef +ve (Table 26.11-1)	GC_{pi_p} = **0.18**
Internal pressure coef –ve (Table 26.11-1)	GC_{pi_n} = **-0.18**
Parapet internal pressure coef +ve (Table 26.11-1)	GC_{pi_pp} = **0.18**
Parapet internal pressure coef –ve (Table 26.11-1)	GC_{pi_np} = **-0.18**
Gust effect factor	G_f = **0.85**

Topography

Topography factor not significant	K_{zt} = 1.0

Velocity pressure

Velocity pressure coefficient (T.30.3-1)	K_z = **0.87**
Velocity pressure	$q_h = 0.00256 \times K_z \times K_{zt} \times K_d \times V^2 \times 1\text{psf/mph}^2$ = **25.4** psf

Velocity pressure at parapet

Velocity pressure coefficient (T.30.3-1)	K_z = **0.91**
Velocity pressure	$q_p = 0.00256 \times K_z \times K_{zt} \times K_d \times V^2 \times 1\text{psf/mph}^2$ = **26.5** psf

Peak velocity pressure for internal pressure

Peak velocity pressure – internal (as roof press.)	q_i = **25.40** psf

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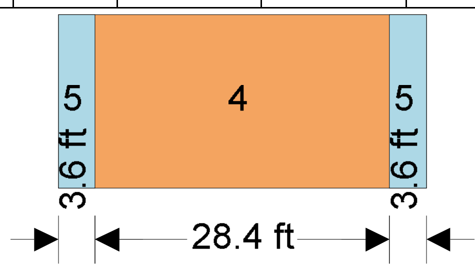
Equations used in tables

Net pressure $p = q_h \times [GC_p - GC_{pi}]$

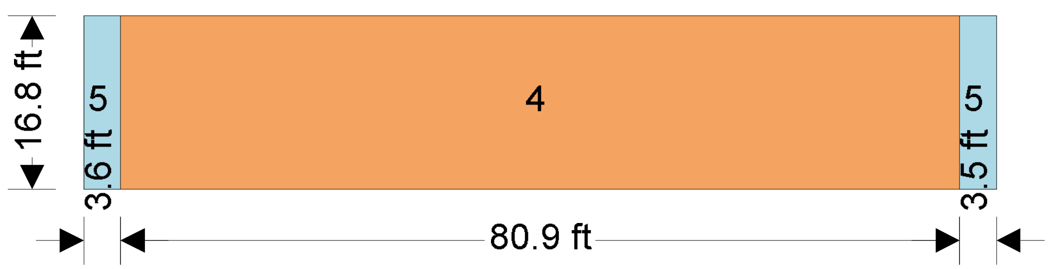
Parapet net pressure $p = q_p \times [GC_p - GC_{pi\_p}]$

Components and cladding pressures - Wall (Table 30.4-1 and Figure 30.4-2A)

Component	Zone	Length (ft)	Width (ft)	Eff. area (ft^2)	$+GC_p$	$-GC_p$	Pres (+ve) (psf)	Pres (-ve) (psf)
<=10 sf	4	-	-	10.0	0.90	-0.99	27.4	-29.7
50 sf	4	-	-	50.0	0.79	-0.88	24.6	-26.9
200 sf	4	-	-	200.0	0.69	-0.78	22.2	-24.5
>500 sf	4	-	-	500.1	0.63	-0.72	20.6	-22.9
<=10 sf	5	-	-	10.0	0.90	-1.26	27.4	-36.6
50 sf	5	-	-	50.0	0.79	-1.04	24.6	-30.9
200 sf	5	-	-	200.0	0.69	-0.85	22.2	-26.1
>500 sf	5	-	-	500.1	0.63	-0.72	20.6	-22.9



Elevation of gable wall



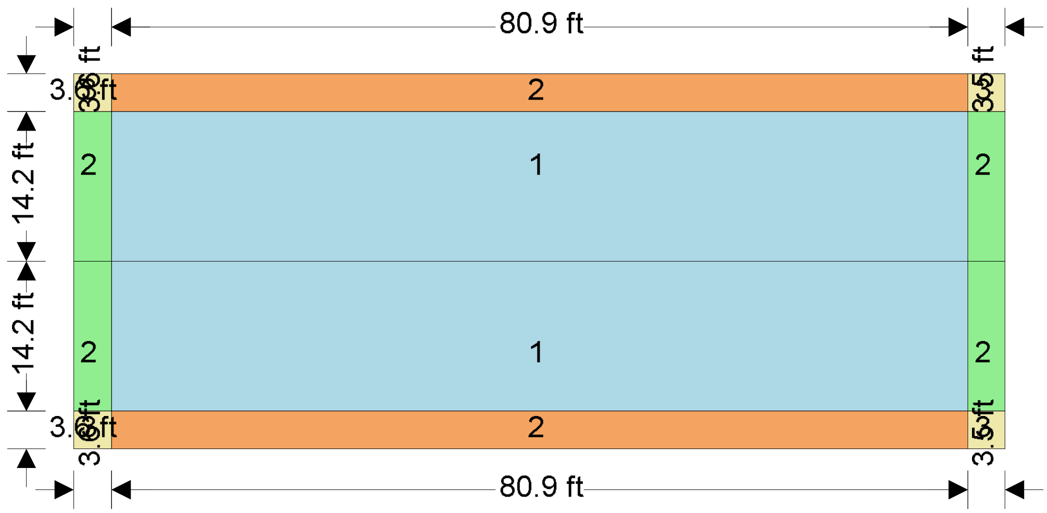
Elevation of side wall

Components and cladding pressures - Roof (Figure 30.4-2A)

Component	Zone	Length (ft)	Width (ft)	Eff. area (ft^2)	$+GC_p$	$-GC_p$	Pres (+ve) (psf)	Pres (-ve) (psf)
<=10 sf	1	-	-	10.0	0.30	-1.00	12.2 #	-30.0
25 sf	1	-	-	25.0	0.26	-0.96	11.2 #	-29.0
50 sf	1	-	-	50.0	0.23	-0.93	10.4 #	-28.2
>100 sf	1	-	-	100.1	0.20	-0.90	9.7 #	-27.4
<=10 sf	2	-	-	10.0	0.90	-1.80	27.4	-50.3
25 sf	2	-	-	25.0	0.84	-1.52	25.8	-43.2
50 sf	2	-	-	50.0	0.79	-1.31	24.6	-37.9
>100 sf	2	-	-	100.1	0.74	-1.10	23.4	-32.5
<=10 sf	3	-	-	10.0	0.90	-1.80	27.4	-50.3

Component	Zone	Length (ft)	Width (ft)	Eff. area (ft^2)	$+GC_p$	$-GC_p$	Pres (+ve) (psf)	Pres (-ve) (psf)
25 sf	3	-	-	25.0	0.84	-1.52	25.8	-43.2
50 sf	3	-	-	50.0	0.79	-1.31	24.6	-37.9
>100 sf	3	-	-	100.1	0.74	-1.10	23.4	-32.5

The final net design wind pressure, including all permitted reductions, used in the design shall not be less than 16psf acting in either direction



Plan on roof

SEISMIC FORCES

SEISMIC FORCES

In accordance with ASCE 7-10

Tedds calculation version 3.1.03

Site parameters

Site class D

Mapped acceleration parameters (Section 11.4.1)

at short period S_S = **0.172**

at 1 sec period S_1 = **0.083**

Site coefficientat short period (Table 11.4-1) F_a = **1.600**

at 1 sec period (Table 11.4-2) F_v = **2.400**

Spectral response acceleration parameters

at short period (Eq. 11.4-1) $S_{MS} = F_a \times S_S$ = **0.275**

at 1 sec period (Eq. 11.4-2) $S_{M1} = F_v \times S_1$ = **0.199**

Design spectral acceleration parameters (Sect 11.4.4)

at short period (Eq. 11.4-3) $S_{DS} = 2 / 3 \times S_{MS}$ = **0.183**

at 1 sec period (Eq. 11.4-4) $S_{D1} = 2 / 3 \times S_{M1}$ = **0.133**

Seismic design category

Risk category (Table 1.5-1) II

Seismic design category based on short period response acceleration (Table 11.6-1)

B

Seismic design category based on 1 sec period response acceleration (Table 11.6-2)	
	B
Seismic design category	B
Approximate fundamental period	
Height above base to highest level of building	h_n = **22** ft
From Table 12.8-2:	
Structure type	All other systems
Building period parameter C_t	C_t = **0.02**
Building period parameter x	x = **0.75**
Approximate fundamental period (Eq 12.8-7)	$T_a = C_t \times (h_n)^x \times$ 1sec / (1ft)^x= **0.203** sec
Building fundamental period (Sect 12.8.2)	$T = T_a$ = **0.203** sec
Long-period transition period	T_L = **8** sec
Seismic response coefficient	
Seismic force-resisting system (Table 12.2-1)	A. Bearing_Wall_Systems
	15. Light-frame (wood) walls sheathed with wood structural panels
Response modification factor (Table 12.2-1)	R = **6.5**
Seismic importance factor (Table 1.5-2)	I_e = **1.000**
Seismic response coefficient (Sect 12.8.1.1)	
Calculated (Eq 12.8-2)	$C_{s\_calc} = S_{DS} / (R / I_e)$ = **0.0282**
Maximum (Eq 12.8-3)	$C_{s\_max} = S_{D1} / ((T / 1\text{ sec}) \times (R / I_e))$ = **0.1006**
Minimum (Eq 12.8-5)	$C_{s\_min} = \max(0.044 \times S_{DS} \times I_e, 0.01)$ = **0.0100**
Seismic response coefficient	C_s = **0.0282**
Seismic base shear (Sect 12.8.1)	
Effective seismic weight of the structure	W = **125.0** kips
Seismic response coefficient	C_s = **0.0282**
Seismic base shear (Eq 12.8-1)	$V = C_s \times W$ = **3.5** kips

SIPS

Wall Panels:

Width $w_{wall} = 8$ ft
Span $L_{wall} = 16$ ft
Thickness $t_{wall} = 6.5$ in
Allowable Transverse Load (Porter Code Report, adjusted for capacity of SIP skin nailing)
@ L/180 23.8 psf
@ L/240 22.0 psf
@ L/360 14.0 psf

Demand $D_{wall} = 0.6 * 0.7 * 37.9$ psf = **15.918** psf
Capacity $C_{wall} = 22.0$ psf
Check $check_{wall}$ = if($C_{wall} > D_{wall}$, “OK”, “NO GOOD”) = **"OK"**

“Note- The SIPA Master Report wall panel values (table 7) are based on zero bearing and provide a Cp = 0.4. This does not account for the capacity of the nailing through the SIP skin. See full table of adjusted values on page 35 for accurate wall panel capacities that factor in the additional capacity gained through nailing through the SIP skins.”

Roof Panels:

Width $w_{roof} = 8$ ft
Span $L_{roof} = 32$ in
Thickness $t_{roof} = 8.25$ in
Allowable Transverse Load (Porter, Table 5)
@ L/180 90 psf
@ L/240 90 psf
@ L/360 90 psf

Demand D_{roof} = 20 psf + 20 psf = **40.000** psf
Capacity $C_{roof} = 90$ psf
Check $check_{roof}$ = if($C_{roof} > D_{roof}$, “OK”, “NO GOOD”) = **"OK"**

Roof Fasteners:

Tributary Width TW = L_{roof} / 2 = **1.333** ft
Fastener Spacing (c/c) s = 9 in

Performance Data (per Trufast SIP Fasteners):
Safety Factor SF = 3
Withdrawal W = (917 lb/in / SF) * t_{roof} = **2521.750** lb
Head Pull-Thru P = 630 lb / SF = **210.000** lb

Design Loads (ASCE 7-16 & Plans):
Velocity Winds Pressure (S 30.3.2) $q_z = 25.4$ psf
Internal Pressure Coefficient (S 26.11) GC_{pi} = max(0.18 , -0.18)
Roof Uplift Pressure (S 30.4.2) p = -50.3 psf

Net Roof Pressure P_{net} = 0.6*(p + 20 psf) = **-18.180** psf
Uplift Per Fastener $P_{fastener} = P_{net}$ * TW * s = **-18.180** lb

Check $check_{fastener}$ = if(P > abs($P_{fastener}$) , “OK” , “NO GOOD”) = **"OK"**

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ASD SEGMENTED SHEAR WALL (WALL 1)

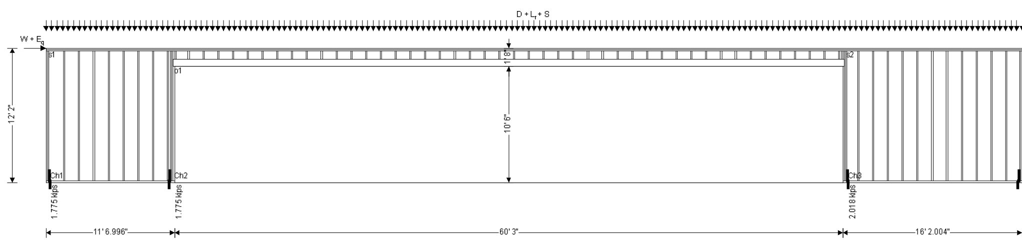
WOOD SHEAR WALL DESIGN (NDS)

In accordance with NDS2015 allowable stress design and the segmented shear wall method

Tedds calculation version 1.2.09

Panel details

Structural wood panel sheathing on one side	
Panel height	h = **12.167** ft
Panel length	b = **88** ft



Panel opening details

Width of opening	w_{o1} = **60.25** ft
Height of opening	h_{o1} = **10.5** ft
Height to underside of lintel over opening	l_{o1} = **10.5** ft
Position of opening	P_{o1} = **11.583** ft
Total area of wall	$A = h \times b - w_{o1} \times h_{o1}$ = **438.045** ft^2

Panel construction

Nominal stud size	2" x 6"
Dressed stud size	1.5" x 5.5"
Cross-sectional area of studs	A_s = **8.25** in^2
Stud spacing	s = **16** in
Nominal end post size	2 x 2" x 6"
Dressed end post size	2 x 1.5" x 5.5"
Cross-sectional area of end posts	A_e = **16.5** in^2
Hole diameter	Dia = **1** in
Net cross-sectional area of end posts	A_{en} = **13.5** in^2
Nominal collector size	2 x 2" x 6"
Dressed collector size	2 x 1.5" x 5.5"
Service condition	Dry
Temperature	100 degF or less
Vertical anchor stiffness	k_a = **50000** lb/in

From NDS Supplement Table 4A - Reference design values for visually graded dimension lumber (2" - 4" thick)

Species, grade and size classification	Spruce-Pine-Fir, no.2 grade, 2" & wider
Specific gravity	G = **0.42**
Tension parallel to grain	F_t = **450** lb/in^2
Compression parallel to grain	F_c = **1150** lb/in^2
Modulus of elasticity	E = **1400000** lb/in^2
Minimum modulus of elasticity	E_{min} = **510000** lb/in^2

Sheathing details

Sheathing material	15/32" wood panel oriented strandboard sheathing

Fastener type	8d common nails at 3"centers

From SDPWS Table 4.3A Nominal Unit Shear Capacities for Wood-Frame Shear Walls - Wood-based Panels

Nominal unit shear capacity for seismic design	v_s = 980 plf × min[1 - (0.5 - G), 1] = **901.6** lb/ft
Nominal unit shear capacity for wind design	v_w = 1370 plf × min[1 - (0.5 - G), 1] = **1260.4** lb/ft
Apparent shear wall shear stiffness	G_a = **25** kips/in

Loading details

Dead load acting on top of panel	D = **355** lb/ft
Roof live load acting on top of panel	L_r = **355** lb/ft
Snow load acting on top of panel	S = **266** lb/ft
Self weight of panel	S_{wt} = **10** lb/ft^2
In plane wind load acting at head of panel	W = **8010** lbs
Wind load serviceability factor	f_{Wserv} = **0.60**
In plane seismic load acting at head of panel	E_q = **1750** lbs
Design spectral response accel. par., short periods	S_{DS} = **0.184**

From IBC 2015 cl.1605.3.1 Basic load combinations

Load combination no.1	D + 0.6W
Load combination no.2	D + 0.7E
Load combination no.3	D + 0.45W + 0.75L_f + 0.75(L_r or S or R)
Load combination no.4	D + 0.525E + 0.75L_f + 0.75S
Load combination no.5	0.6D + 0.6W
Load combination no.6	0.6D + 0.7E

Adjustment factors

Load duration factor – Table 2.3.2	C_D = **1.60**
Size factor for tension – Table 4A	C_{Ft} = **1.30**
Size factor for compression – Table 4A	C_{Fc} = **1.10**
Wet service factor for tension – Table 4A	C_{Mt} = **1.00**
Wet service factor for compression – Table 4A	C_{Mc} = **1.00**
Wet service factor for modulus of elasticity – Table 4A	C_{ME} = **1.00**
Temperature factor for tension – Table 2.3.3	C_{tt} = **1.00**
Temperature factor for compression – Table 2.3.3	C_{tc} = **1.00**
Temperature factor for modulus of elasticity – Table 2.3.3	C_{tE} = **1.00**
Incising factor – cl.4.3.8	C_i = **1.00**
Buckling stiffness factor – cl.4.4.2	C_T = **1.00**
Adjusted modulus of elasticity	$E_{min}' = E_{min} \times C_{ME} \times C_{tE} \times C_i \times C_T$ = **510000** psi
Critical buckling design value	$F_{cE} = 0.822 \times E_{min}' / (h / d)^2$ = **595** psi
Reference compression design value	$F_c^* = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i$ = **2024** psi
For sawn lumber	c = **0.8**
Column stability factor – eqn.3.7-1	$C_P = (1 + (F_{cE} / F_c^*)) / (2 \times c) - \sqrt{([(1 + (F_{cE} / F_c^*)) / (2 \times c)]^2 - (F_{cE} / F_c^*) / c)}$ = **0.27**

From SDPWS Table 4.3.4 Maximum Shear Wall Aspect Ratios

Maximum shear wall aspect ratio	3.5
Segment 1 wall length	b_1 = **11.583** ft
Shear wall aspect ratio	h / b_1 = **1.05**
Segment 2 wall length	b_2 = **16.167** ft

Shear wall aspect ratio — h / b_2 = **0.753**

Segmented shear wall capacity - Equal deflection method

Wind loading:

Segment 2 stiffness — $k_2 = 1 / (2 \times h^3 / (3 \times E \times A_e \times b_2^2) + h / (G_a \times b_2) + h^2 / (k_a \times b_2^2))$ = **22.823** kips/in

Unit shear capacity, widest segment — $v_{sww2} = v_w / 2$ = **630.2** plf

Deflection under capacity load — $\delta_{Cap} = 2 \times v_{sww2} \times h^3 / (3 \times E \times A_e \times b_2) + v_{sww2} \times h / (G_a) + h^2 \times v_{sww2} / (k_a \times b_2)$ = **0.446** in

Segment 1 stiffness — $k_1 = 1 / (2 \times h^3 / (3 \times E \times A_e \times b_1^2) + h / (G_a \times b_1) + h^2 / (k_a \times b_1^2))$ = **14.549** kips/in

Segment 1 unit shear at δ_{Cap} — $v_{dsww1} = \delta_{Cap} \times k_1 / b_1$ = **560.75** plf

Segment 1 shear capacity — $v_{sww1} = v_w / 2$ = **630.2** plf

v_{dsww1} / v_{sww1} = **0.890**

PASS - Segment shear capacity exceeds segment unit shear at δ_{Cap}

Maximum shear force under wind loading — $V_{w\_max} = 0.6 \times W$ = **4.806** kips

Shear capacity for wind loading — $V_w = \min(v_{sww1}, v_{dsww1}) \times b_1 + v_{sww2} \times b_2$ = **16.684** kips

V_{w_max} / V_w = **0.288**

PASS - Shear capacity for wind load exceeds maximum shear force

Seismic loading:

Segment 2 stiffness — $k_2 = 1 / (2 \times h^3 / (3 \times E \times A_e \times b_2^2) + h / (G_a \times b_2) + h^2 / (k_a \times b_2^2))$ = **22.823** kips/in

Unit shear capacity, widest segment — $v_{sws2} = v_s / 2$ = **450.8** plf

Deflection under capacity load — $\delta_{Cap} = 2 \times v_{sws2} \times h^3 / (3 \times E \times A_e \times b_2) + v_{sws2} \times h / (G_a) + h^2 \times v_{sws2} / (k_a \times b_2)$ = **0.319** in

Segment 1 stiffness — $k_1 = 1 / (2 \times h^3 / (3 \times E \times A_e \times b_1^2) + h / (G_a \times b_1) + h^2 / (k_a \times b_1^2))$ = **14.549** kips/in

Segment 1 unit shear at δ_{Cap} — $v_{dsws1} = \delta_{Cap} \times k_1 / b_1$ = **401.12** plf

Segment 1 shear capacity — $v_{sws1} = v_s / 2$ = **450.8** plf

v_{dsws1} / v_{sws1} = **0.890**

PASS - Segment shear capacity exceeds segment unit shear at δ_{Cap}

Maximum shear force under seismic loading — $V_{s\_max} = 0.7 \times E_q$ = **1.225** kips

Shear capacity for seismic loading — $V_s = \min(v_{sws1}, v_{dsws1}) \times b_1 + v_{sws2} \times b_2$ = **11.934** kips

V_{s_max} / V_s = **0.103**

PASS - Shear capacity for seismic load exceeds maximum shear force

Chord capacity for chords 1 and 2

Shear wall aspect ratio — h / b_1 = **1.05**

Load combination 5

Shear force for maximum tension — $V = 0.6 \times W$ = **4.806** kips

Axial force for maximum tension — $P = (0.6 \times (D + S_{wt} \times h)) \times s / 2$ = **0.191** kips

Maximum tensile force in chord — $T = V \times (k_1 / \mathrm{sum}(k_1,k_2)) \times h / b_1 - P$ = **1.775** kips

Maximum applied tensile stress — $f_t = T / A_{en}$ = **131** lb/in²

Design tensile stress — $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i$ = **936** lb/in²

f_t / F_t' = **0.140**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 1

Shear force for maximum compression — $V = 0.6 \times W$ = **4.806** kips

Axial force for maximum compression — $P = ((D + S_{wt} \times h)) \times s / 2$ = **0.318** kips

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Maximum compressive force in chord	$C = V \times (k_1 / \mathrm{sum}(k_1,k_2)) \times h / b_1 + P =$ **2.283** kips
Maximum applied compressive stress	$f_c = C / A_e =$ **138** lb/in^2
Design compressive stress	$F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P =$ **553** lb/in^2
	$f_c / F_c' =$ **0.250**

PASS - Design compressive stress exceeds maximum applied compressive stress

Chord capacity for chords 3 and 4

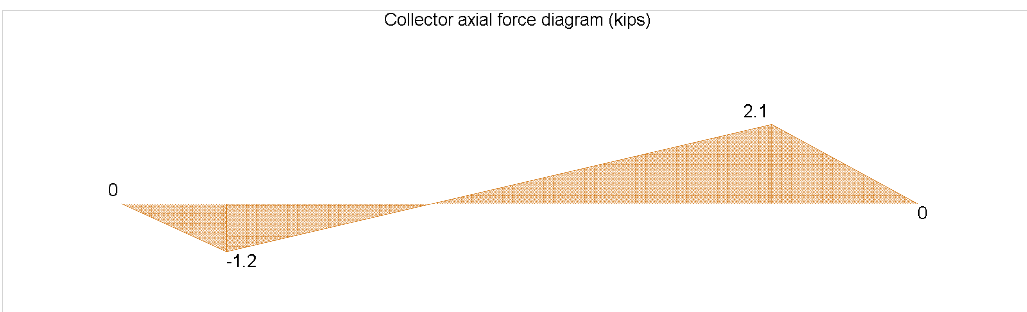
Shear wall aspect ratio	$h / b_2 =$ **0.753**
Load combination 5	
Shear force for maximum tension	$V = 0.6 \times W =$ **4.806** kips
Axial force for maximum tension	$P = (0.6 \times (D + S_{wt} \times h)) \times s / 2 =$ **0.191** kips
Maximum tensile force in chord	$T = V \times (k_2 / \mathrm{sum}(k_1,k_2)) \times h / b_2 - P =$ **2.018** kips
Maximum applied tensile stress	$f_t = T / A_{en} =$ **149** lb/in^2
Design tensile stress	$F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i =$ **936** lb/in^2
	$f_t / F_t' =$ **0.160**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 1	
Shear force for maximum compression	$V = 0.6 \times W =$ **4.806** kips
Axial force for maximum compression	$P = ((D + S_{wt} \times h)) \times s / 2 =$ **0.318** kips
Maximum compressive force in chord	$C = V \times (k_2 / \mathrm{sum}(k_1,k_2)) \times h / b_2 + P =$ **2.527** kips
Maximum applied compressive stress	$f_c = C / A_e =$ **153** lb/in^2
Design compressive stress	$F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P =$ **553** lb/in^2
	$f_c / F_c' =$ **0.277**

PASS - Design compressive stress exceeds maximum applied compressive stress

Collector capacity



Collector seismic design force factor	$F_{Coll} =$ **1**
Maximum shear force on wall	$V_{max} = \max(F_{Coll} \times V_{s\_max}, V_{w\_max}) =$ **4.806** kips
Maximum force in collector	$P_{coll} =$ **2.052** kips
Maximum applied tensile stress	$f_t = P_{coll} / (2 \times A_s) =$ **124** lb/in^2
Design tensile stress	$F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i =$ **936** lb/in^2
	$f_t / F_t' =$ **0.133**

PASS - Design tensile stress exceeds maximum applied tensile stress

Maximum applied compressive stress	$f_c = P_{coll} / (2 \times A_s) =$ **124** lb/in^2
Column stability factor	$C_P =$ **1.00**
Design compressive stress	$F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P =$ **2024** lb/in^2
	$f_c / F_c' =$ **0.061**

PASS - Design compressive stress exceeds maximum applied compressive stress

Hold down force

Chord 1 T_1 = **1.775** kips
Chord 2 T_2 = **1.775** kips
Chord 3 T_3 = **2.018** kips
Chord 4 T_4 = **2.018** kips

Wind load deflection

Design shear force $V_{\delta w} = f_{Wserv} \times W$ = **4.806** kips
Deflection limit $\Delta_{w\_allow} = h / 400$ = **0.365** in
Segment 1
Induced unit shear $v_{\delta w} = V_{\delta w} \times (k_1 / sum(k_1,k_2)) / b_1$ = **161.53** lb/ft
Anchor tension force $T_\delta = max(0\ kips, v_{\delta w} \times h - 0.6 \times (D + S_{wt} \times h) \times s / 2)$ = **1.775** kips
Segment 1 deflection – Eqn. 4.3-1 $\delta_{sww1} = 2 \times v_{\delta w} \times h^3 / (3 \times E \times A_e \times b_1) + v_{\delta w} \times h / (G_a) + h \times T_\delta / (k_a \times b_1)$ = **0.125** in
$\delta_{sww1} / \Delta_{w\_allow}$ = **0.341**
PASS - Shear wall deflection is less than deflection limit
Segment 2
Induced unit shear $v_{\delta w} = V_{\delta w} \times (k_2 / sum(k_1,k_2)) / b_2$ = **181.54** lb/ft
Anchor tension force $T_\delta = max(0\ kips, v_{\delta w} \times h - 0.6 \times (D + S_{wt} \times h) \times s / 2)$ = **2.018** kips
Segment 2 deflection – Eqn. 4.3-1 $\delta_{sww2} = 2 \times v_{\delta w} \times h^3 / (3 \times E \times A_e \times b_2) + v_{\delta w} \times h / (G_a) + h \times T_\delta / (k_a \times b_2)$ = **0.126** in
$\delta_{sww2} / \Delta_{w\_allow}$ = **0.344**
PASS - Shear wall deflection is less than deflection limit

Seismic deflection

Design shear force $V_{\delta s} = E_q$ = **1.75** kips
Deflection limit $\Delta_{s\_allow} = 0.020 \times h$ = **2.92** in
Deflection ampification factor $C_{d\delta}$ = **4**
Seismic importance factor I_e = **1.25**
Segment 1
Induced unit shear $v_{\delta s} = V_{\delta s} \times (k_1 / sum(k_1,k_2)) / b_1$ = **58.82** lb/ft
Anchor tension force $T_\delta = max(0\ kips, v_{\delta s} \times h - (0.6 - 0.2 \times S_{DS}) \times (D + S_{wt} \times h) \times s / 2)$ = **0.537** kips
Segment 1 deflection – Eqn. 4.3-1 $\delta_{swse1} = 2 \times v_{\delta s} \times h^3 / (3 \times E \times A_e \times b_1) + v_{\delta s} \times h / (G_a) + h \times T_\delta / (k_a \times b_1)$ = **0.043** in
Amp. seis. deflection – ASCE7 Eqn. 12.8-15 $\delta_{sws1} = C_{d\delta} \times \delta_{swse1} / I_e$ = **0.138** in
$\delta_{sws1} / \Delta_{s\_allow}$ = **0.047**
PASS - Shear wall deflection is less than deflection limit
Segment 2
Induced unit shear $v_{\delta s} = V_{\delta s} \times (k_2 / sum(k_1,k_2)) / b_2$ = **66.1** lb/ft
Anchor tension force $T_\delta = max(0\ kips, v_{\delta s} \times h - (0.6 - 0.2 \times S_{DS}) \times (D + S_{wt} \times h) \times s / 2)$ = **0.625** kips
Segment 2 deflection – Eqn. 4.3-1 $\delta_{swse2} = 2 \times v_{\delta s} \times h^3 / (3 \times E \times A_e \times b_2) + v_{\delta s} \times h / (G_a) + h \times T_\delta / (k_a \times b_2)$ = **0.044** in
Amp. seis. deflection – ASCE7 Eqn. 12.8-15 $\delta_{sws2} = C_{d\delta} \times \delta_{swse2} / I_e$ = **0.141** in
$\delta_{sws2} / \Delta_{s\_allow}$ = **0.048**
PASS - Shear wall deflection is less than deflection limit

ASD PERFORATED SHEAR WALL (WALL 2)

WOOD SHEAR WALL DESIGN (NDS)

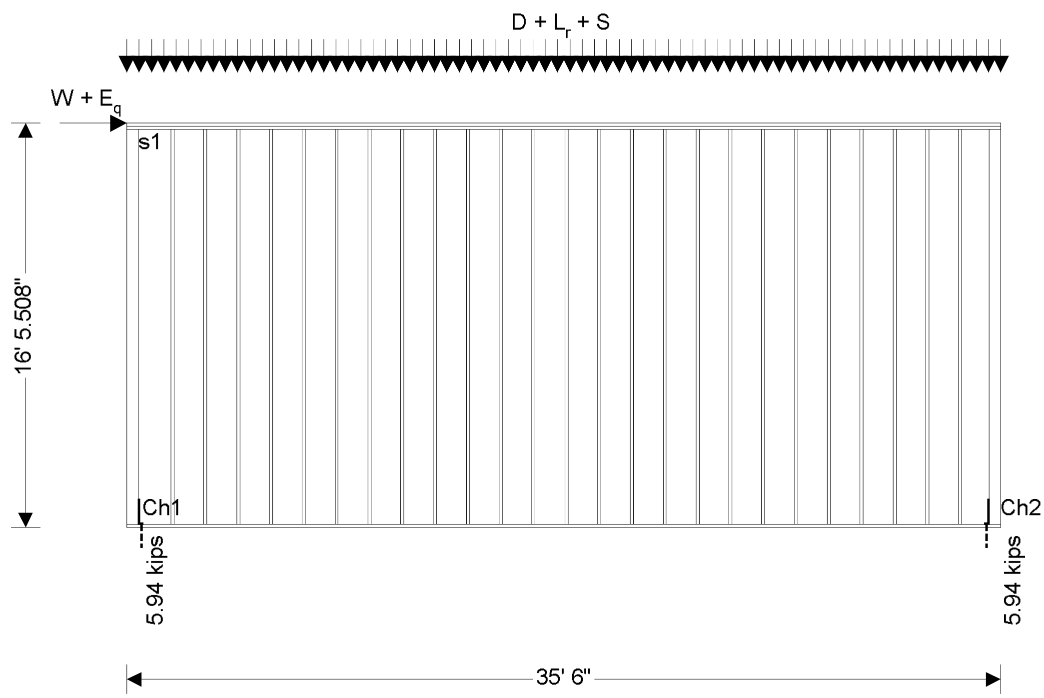
In accordance with NDS2015 allowable stress design and the segmented shear wall method

Tedds calculation version 1.2.09

Panel details

Structural wood panel sheathing on one side

Panel height	h = **16.459** ft
Panel length	b = **35.5** ft
Total area of wall	A = h × b = **584.295** ft^2



Panel construction

Nominal stud size	2" x 6"
Dressed stud size	1.5" x 5.5"
Cross-sectional area of studs	A_s = **8.25** in^2
Stud spacing	s = **16** in
Nominal end post size	6" x 6"
Dressed end post size	5.5" x 5.5"
Cross-sectional area of end posts	A_e = **30.25** in^2
Hole diameter	Dia = **1** in
Net cross-sectional area of end posts	A_{en} = **24.75** in^2
Nominal collector size	2 x 2" x 6"
Dressed collector size	2 x 1.5" x 5.5"
Service condition	Dry
Temperature	100 degF or less
Vertical anchor stiffness	k_a = **50000** lb/in

From NDS Supplement Table 4A - Reference design values for visually graded dimension lumber (2" - 4" thick)

Species, grade and size classification	Spruce-Pine-Fir, no.2 grade, 2" & wider

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Specific gravity	G = **0.42**
Tension parallel to grain	F_t = **450** lb/in^2
Compression parallel to grain	F_c = **1150** lb/in^2
Modulus of elasticity	E = **1400000** lb/in^2
Minimum modulus of elasticity	E_{min} = **510000** lb/in^2

Sheathing details

Sheathing material	15/32" wood panel oriented strandboard sheathing
Fastener type	8d common nails at 3"centers

From SDPWS Table 4.3A Nominal Unit Shear Capacities for Wood-Frame Shear Walls - Wood-based Panels

Nominal unit shear capacity for seismic design	v_s = 980 plf × min[1 - (0.5 - G), 1] = **901.6** lb/ft
Nominal unit shear capacity for wind design	v_w = 1370 plf × min[1 - (0.5 - G), 1] = **1260.4** lb/ft
Apparent shear wall shear stiffness	G_a = **25** kips/in

Loading details

Dead load acting on top of panel	D = **27** lb/ft
Roof live load acting on top of panel	L_r = **27** lb/ft
Snow load acting on top of panel	S = **60** lb/ft
Self weight of panel	S_{wt} = **10** lb/ft^2
In plane wind load acting at head of panel	W = **21630** lbs
Wind load serviceability factor	f_{Wserv} = **0.60**
In plane seismic load acting at head of panel	E_q = **1750** lbs
Design spectral response accel. par., short periods	S_{DS} = **0.184**

From IBC 2015 cl.1605.3.1 Basic load combinations

Load combination no.1	D + 0.6W
Load combination no.2	D + 0.7E
Load combination no.3	D + 0.45W + 0.75L_f + 0.75(L_r or S or R)
Load combination no.4	D + 0.525E + 0.75L_f + 0.75S
Load combination no.5	0.6D + 0.6W
Load combination no.6	0.6D + 0.7E

Adjustment factors

Load duration factor – Table 2.3.2	C_D = **1.60**
Size factor for tension – Table 4A	C_{Ft} = **1.30**
Size factor for compression – Table 4A	C_{Fc} = **1.10**
Wet service factor for tension – Table 4A	C_{Mt} = **1.00**
Wet service factor for compression – Table 4A	C_{Mc} = **1.00**
Wet service factor for modulus of elasticity – Table 4A	C_{ME} = **1.00**
Temperature factor for tension – Table 2.3.3	C_{tt} = **1.00**
Temperature factor for compression – Table 2.3.3	C_{tc} = **1.00**
Temperature factor for modulus of elasticity – Table 2.3.3	C_{tE} = **1.00**
Incising factor – cl.4.3.8	C_i = **1.00**
Buckling stiffness factor – cl.4.4.2	C_T = **1.00**
Adjusted modulus of elasticity	$E_{min}' = E_{min} \times C_{ME} \times C_{tE} \times C_i \times C_T$ = **510000** psi
Critical buckling design value	$F_{cE} = 0.822 \times E_{min}' / (h / d)^2$ = **325** psi
Reference compression design value	$F_c^* = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i$ = **2024** psi
For sawn lumber	c = **0.8**

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Column stability factor – eqn.3.7-1	$C_P = (1 + (F_{cE} / F_c^*)) / (2 \times c) - \sqrt{([(1 + (F_{cE} / F_c^*)) / (2 \times c)]^2 - (F_{cE} / F_c^*) / c)} =$ **0.15**

From SDPWS Table 4.3.4 Maximum Shear Wall Aspect Ratios

Maximum shear wall aspect ratio	3.5
Shear wall length	$b =$ **35.5** ft
Shear wall aspect ratio	$h / b =$ **0.464**

Segmented shear wall capacity

Maximum shear force under wind loading	$V_{w\_max} = 0.6 \times W =$ **12.978** kips
Shear capacity for wind loading	$V_w = v_w \times b / 2 =$ **22.372** kips
	$V_{w\_max} / V_w =$ **0.58**
	PASS - Shear capacity for wind load exceeds maximum shear force
Maximum shear force under seismic loading	$V_{s\_max} = 0.7 \times E_q =$ **1.225** kips
Shear capacity for seismic loading	$V_s = v_s \times b / 2 =$ **16.003** kips
	$V_{s\_max} / V_s =$ **0.077**
	PASS - Shear capacity for seismic load exceeds maximum shear force

Chord capacity for chords 1 and 2

Shear wall aspect ratio	$h / b =$ **0.464**
Load combination 5	
Shear force for maximum tension	$V = 0.6 \times W =$ **12.978** kips
Axial force for maximum tension	$P = (0.6 \times (D + S_{wt} \times h)) \times s / 2 =$ **0.077** kips
Maximum tensile force in chord	$T = V \times h / (b) - P =$ **5.940** kips
Maximum applied tensile stress	$f_t = T / A_{en} =$ **240** lb/in^2
Design tensile stress	$F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i =$ **936** lb/in^2
	$f_t / F_t' =$ **0.256**
	PASS - Design tensile stress exceeds maximum applied tensile stress
Load combination 1	
Shear force for maximum compression	$V = 0.6 \times W =$ **12.978** kips
Axial force for maximum compression	$P = ((D + S_{wt} \times h)) \times s / 2 =$ **0.128** kips
Maximum compressive force in chord	$C = V \times h / (b) + P =$ **6.145** kips
Maximum applied compressive stress	$f_c = C / A_e =$ **203** lb/in^2
Design compressive stress	$F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P =$ **314** lb/in^2
	$f_c / F_c' =$ **0.648**

PASS - Design compressive stress exceeds maximum applied compressive stress

Hold down force

Chord 1	$T_1 =$ **5.94** kips
Chord 2	$T_2 =$ **5.94** kips

Wind load deflection

Design shear force	$V_{\delta w} = f_{Wserv} \times W =$ **12.978** kips
Deflection limit	$\Delta_{w\_allow} = h / 400 =$ **0.494** in
Induced unit shear	$v_{\delta w} = V_{\delta w} / b =$ **365.58** lb/ft
Anchor tension force	$T_\delta = \max(0 \text{ kips}, v_{\delta w} \times h - 0.6 \times (D + S_{wt} \times h) \times s / 2) =$ **5.940** kips
Shear wall deflection – Eqn. 4.3-1	$\delta_{sww} = 2 \times v_{\delta w} \times h^3 / (3 \times E \times A_e \times b) + v_{\delta w} \times h / (G_a) + h \times T_\delta / (k_a \times b) =$ **0.304** in
	$\delta_{sww} / \Delta_{w\_allow} =$ **0.617**
	PASS - Shear wall deflection is less than deflection limit

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Seismic deflection

Design shear force	$V_{\delta s} = E_q =$ **1.75** kips
Deflection limit	$\Delta_{s\_allow} = 0.020 \times h =$ **3.95** in
Induced unit shear	$v_{\delta s} = V_{\delta s} / b =$ **49.3** lb/ft
Anchor tension force	$T_\delta = \max(0 \text{ kips}, v_{\delta s} \times h - (0.6 - 0.2 \times S_{DS}) \times (D + S_{wt} \times h) \times s / 2) =$ **0.739** kips
Shear wall elastic deflection – Eqn. 4.3-1	$\delta_{swse} = 2 \times v_{\delta s} \times h^3 / (3 \times E \times A_e \times b) + v_{\delta s} \times h / (G_a) + h \times T_\delta / (k_a \times b) =$ **0.04** in
Deflection ampification factor	$C_{d\delta} =$ **4**
Seismic importance factor	$I_e =$ **1.25**
Amp. seis. deflection – ASCE7 Eqn. 12.8-15	$\delta_{sws} = C_{d\delta} \times \delta_{swse} / I_e =$ **0.13** in
	$\delta_{sws} / \Delta_{s\_allow} =$ **0.033**

PASS - Shear wall deflection is less than deflection limit

ASD SEGMENTED SHEAR WALL (WALL 3)

WOOD SHEAR WALL DESIGN (NDS)

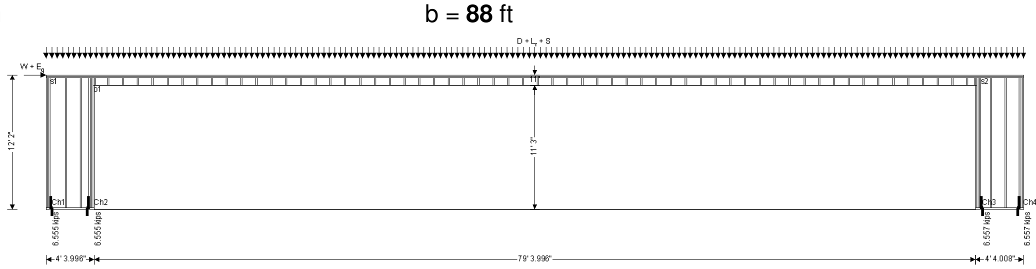
In accordance with NDS2015 allowable stress design and the segmented shear wall method

Tedds calculation version 1.2.09

Panel details

Structural wood panel sheathing on one side

Panel height	$h =$ **12.167** ft
Panel length	$b =$ **88** ft



Panel opening details

Width of opening	$w_{o1} =$ **79.333** ft
Height of opening	$h_{o1} =$ **11.25** ft
Height to underside of lintel over opening	$l_{o1} =$ **11.25** ft
Position of opening	$P_{o1} =$ **4.333** ft
Total area of wall	$A = h \times b - w_{o1} \times h_{o1} =$ **178.173** ft^2

Panel construction

Nominal stud size	2" x 6"
Dressed stud size	1.5" x 5.5"
Cross-sectional area of studs	$A_s =$ **8.25** in^2
Stud spacing	$s =$ **16** in
Nominal end post size	3 x 2" x 6"

Dressed end post size	3 x 1.5" x 5.5"
Cross-sectional area of end posts	A_e = **24.75** in^2
Hole diameter	Dia = **1** in
Net cross-sectional area of end posts	A_{en} = **20.25** in^2
Nominal collector size	2 x 2" x 6"
Dressed collector size	2 x 1.5" x 5.5"
Service condition	Dry
Temperature	100 degF or less
Vertical anchor stiffness	k_a = **50000** lb/in

From NDS Supplement Table 4A - Reference design values for visually graded dimension lumber (2" - 4" thick)

Species, grade and size classification	Spruce-Pine-Fir, stud grade, 2" & wider
Specific gravity	G = **0.42**
Tension parallel to grain	F_t = **350** lb/in^2
Compression parallel to grain	F_c = **725** lb/in^2
Modulus of elasticity	E = **1200000** lb/in^2
Minimum modulus of elasticity	E_{min} = **440000** lb/in^2

Sheathing details

Sheathing material	15/32" wood panel oriented strandboard sheathing
Fastener type	8d common nails at 2"centers

From SDPWS Table 4.3A Nominal Unit Shear Capacities for Wood-Frame Shear Walls - Wood-based Panels

Nominal unit shear capacity for seismic design	v_s = 1280 plf $\times$ min[1 - (0.5 - G), 1] = **1177.6** lb/ft
Nominal unit shear capacity for wind design	v_w = 1790 plf $\times$ min[1 - (0.5 - G), 1] = **1646.8** lb/ft
Apparent shear wall shear stiffness	G_a = **39** kips/in

Loading details

Dead load acting on top of panel	D = **355** lb/ft
Roof live load acting on top of panel	L_r = **355** lb/ft
Snow load acting on top of panel	S = **266** lb/ft
Self weight of panel	S_{wt} = **10** lb/ft^2
In plane wind load acting at head of panel	W = **8010** lbs
Wind load serviceability factor	f_{Wserv} = **0.60**
In plane seismic load acting at head of panel	E_q = **1750** lbs
Design spectral response accel. par., short periods	S_{DS} = **0.184**

From IBC 2015 cl.1605.3.1 Basic load combinations

Load combination no.1	D + 0.6W
Load combination no.2	D + 0.7E
Load combination no.3	D + 0.45W + 0.75L_f + 0.75(L_r or S or R)
Load combination no.4	D + 0.525E + 0.75L_f + 0.75S
Load combination no.5	0.6D + 0.6W
Load combination no.6	0.6D + 0.7E

Adjustment factors

Load duration factor – Table 2.3.2	C_D = **1.60**
Size factor for tension – Table 4A	C_{Ft} = **1.00**
Size factor for compression – Table 4A	C_{Fc} = **1.00**
Wet service factor for tension – Table 4A	C_{Mt} = **1.00**
Wet service factor for compression – Table 4A	C_{Mc} = **1.00**
Wet service factor for modulus of elasticity – Table 4A	C_{ME} = **1.00**

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Temperature factor for tension – Table 2.3.3 C_{tt} = **1.00**

Temperature factor for compression – Table 2.3.3 C_{tc} = **1.00**

Temperature factor for modulus of elasticity – Table 2.3.3 C_{tE} = **1.00**

Incising factor – cl.4.3.8 C_i = **1.00**

Buckling stiffness factor – cl.4.4.2 C_T = **1.00**

Adjusted modulus of elasticity $E_{min}' = E_{min} \times C_{ME} \times C_{tE} \times C_i \times C_T$ = **440000** psi

Critical buckling design value $F_{cE} = 0.822 \times E_{min}' / (h / d)^2$ = **513** psi

Reference compression design value $F_c^* = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i$ = **1160** psi

For sawn lumber c = **0.8**

Column stability factor – eqn.3.7-1 $C_P = (1 + (F_{cE} / F_c^*)) / (2 \times c) - \sqrt{([(1 + (F_{cE} / F_c^*)) / (2 \times c)]^2 - (F_{cE} / F_c^*) / c)}$ = **0.39**

From SDPWS Table 4.3.4 Maximum Shear Wall Aspect Ratios

Maximum shear wall aspect ratio 3.5

Segment 1 wall length b_1 = **4.333** ft

Shear wall aspect ratio h / b_1 = **2.808**

Segment 2 wall length b_2 = **4.334** ft

Shear wall aspect ratio h / b_2 = **2.807**

Segmented shear wall capacity - Strength distribution method

Maximum shear force under wind loading $V_{w\_max} = 0.6 \times W$ = **4.806** kips

Shear capacity for wind loading $V_w = v_w \times (2 \times b_1^2 / h + 2 \times b_2^2 / h) / 2$ = **5.084** kips

V_{w_max} / V_w = **0.945**

PASS - Shear capacity for wind load exceeds maximum shear force

Maximum shear force under seismic loading $V_{s\_max} = 0.7 \times E_q$ = **1.225** kips

Shear capacity for seismic loading $V_s = v_s \times (2 \times b_1^2 / h + 2 \times b_2^2 / h) / 2$ = **3.635** kips

V_{s_max} / V_s = **0.337**

PASS - Shear capacity for seismic load exceeds maximum shear force

Chord capacity for chords 1 and 2

Shear wall aspect ratio h / b_1 = **2.808**

Load combination 5

Shear force for maximum tension $V = 0.6 \times W$ = **4.806** kips

Axial force for maximum tension $P = (0.6 \times (D + S_{wt} \times h)) \times s / 2$ = **0.191** kips

Maximum tensile force in chord $T = V \times (2 \times b_1^2 / h / (2 \times b_1^2 / h + 2 \times b_2^2 / h)) \times (h / b_1) - P$ = **6.555** kips

Maximum applied tensile stress $f_t = T / A_{en}$ = **324** lb/in²

Design tensile stress $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i$ = **560** lb/in²

f_t / F_t' = **0.578**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 1

Shear force for maximum compression $V = 0.6 \times W$ = **4.806** kips

Axial force for maximum compression $P = ((D + S_{wt} \times h)) \times s / 2$ = **0.318** kips

Maximum compressive force in chord $C = V \times (2 \times b_1^2 / h / (2 \times b_1^2 / h + 2 \times b_2^2 / h)) \times (h / b_1) + P$ = **7.064** kips

Maximum applied compressive stress $f_c = C / A_e$ = **285** lb/in²

Design compressive stress $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P$ = **455** lb/in²

f_c / F_c' = **0.628**

PASS - Design compressive stress exceeds maximum applied compressive stress

Chord capacity for chords 3 and 4

Shear wall aspect ratio — $h / b_2 =$ **2.807**

Load combination 5

Shear force for maximum tension — $V = 0.6 \times W =$ **4.806** kips

Axial force for maximum tension — $P = (0.6 \times (D + S_{wt} \times h)) \times s / 2 =$ **0.191** kips

Maximum tensile force in chord — $T = V \times (2 \times b_2^2 / h / (2 \times b_1^2 / h + 2 \times b_2^2 / h)) \times (h / b_2) - P =$ **6.557** kips

Maximum applied tensile stress — $f_t = T / A_{en} =$ **324** lb/in^2

Design tensile stress — $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i =$ **560** lb/in^2

$f_t / F_t' =$ **0.578**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 1

Shear force for maximum compression — $V = 0.6 \times W =$ **4.806** kips

Axial force for maximum compression — $P = ((D + S_{wt} \times h)) \times s / 2 =$ **0.318** kips

Maximum compressive force in chord — $C = V \times (2 \times b_2^2 / h / (2 \times b_1^2 / h + 2 \times b_2^2 / h)) \times (h / b_2) + P =$ **7.065** kips

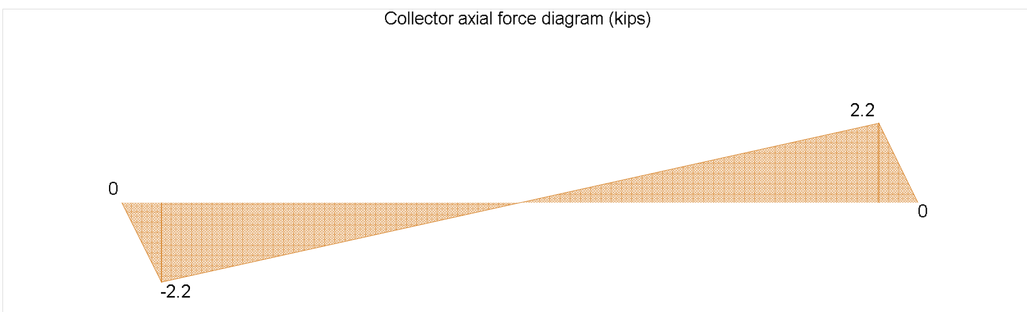
Maximum applied compressive stress — $f_c = C / A_e =$ **285** lb/in^2

Design compressive stress — $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P =$ **455** lb/in^2

$f_c / F_c' =$ **0.628**

PASS - Design compressive stress exceeds maximum applied compressive stress

Collector capacity



Collector seismic design force factor — $F_{Coll} =$ **1**

Maximum shear force on wall — $V_{max} = \max(F_{Coll} \times V_{s\_max}, V_{w\_max}) =$ **4.806** kips

Maximum force in collector — $P_{coll} =$ **2.167** kips

Maximum applied tensile stress — $f_t = P_{coll} / (2 \times A_s) =$ **131** lb/in^2

Design tensile stress — $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i =$ **560** lb/in^2

$f_t / F_t' =$ **0.235**

PASS - Design tensile stress exceeds maximum applied tensile stress

Maximum applied compressive stress — $f_c = P_{coll} / (2 \times A_s) =$ **131** lb/in^2

Column stability factor — $C_P =$ **1.00**

Design compressive stress — $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P =$ **1160** lb/in^2

$f_c / F_c' =$ **0.113**

PASS - Design compressive stress exceeds maximum applied compressive stress

Hold down force

Chord 1 — $T_1 =$ **6.555** kips

Chord 2 — $T_2 =$ **6.555** kips

Chord 3 — $T_3 =$ **6.557** kips

Chord 4 — $T_4 =$ **6.557** kips

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DESIGN WARNING - Design using the strength distribution method to distribute shear loads to individual shear wall segments does not include a deflection check as results to that method are not reliable. Under seismic loads, a drift check is required as part of the design. Suggest using the equal deflections load distribution method to include the seismic drift check in this design.

ASD PERFORATED SHEAR WALL (WALL 4)

WOOD SHEAR WALL DESIGN (NDS)

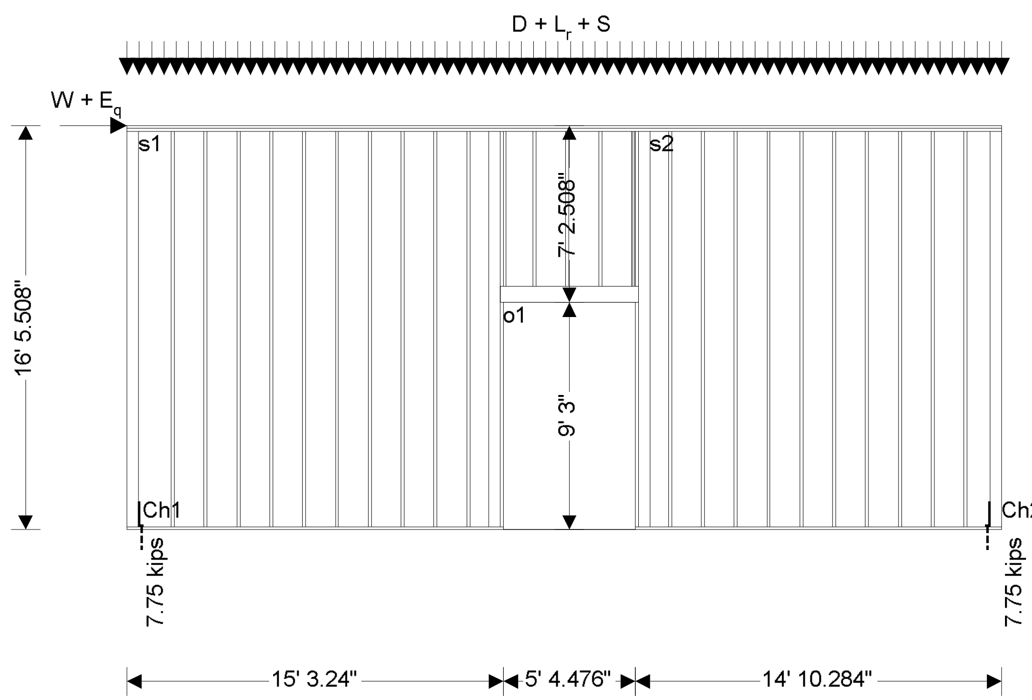
In accordance with NDS2015 allowable stress design and the perforated shear wall method

Tedds calculation version 1.2.09

Panel details

Structural wood panel sheathing on one side

Panel height	h = **16.459** ft
Panel length	b = **35.5** ft



Panel opening details

Width of opening	w_{o1} = **5.373** ft
Height of opening	h_{o1} = **9.25** ft
Height to underside of lintel over opening	l_{o1} = **9.25** ft
Position of opening	P_{o1} = **15.27** ft
Total area of wall	$A = h \times b - w_{o1} \times h_{o1}$ = **534.594** ft^2

Panel construction

Nominal stud size	2" x 6"
Dressed stud size	1.5" x 5.5"
Cross-sectional area of studs	A_s = **8.25** in^2
Stud spacing	s = **16** in
Nominal end post size	6" x 6"
Dressed end post size	5.5" x 5.5"
Cross-sectional area of end posts	A_e = **30.25** in^2

Hole diameter	Dia = **1** in
Net cross-sectional area of end posts	A_{en} = **24.75** in^2
Nominal collector size	2 x 2" x 6"
Dressed collector size	2 x 1.5" x 5.5"
Service condition	Dry
Temperature	100 degF or less
Vertical anchor stiffness	k_a = **50000** lb/in

From NDS Supplement Table 4A - Reference design values for visually graded dimension lumber (2" - 4" thick)

Species, grade and size classification	Spruce-Pine-Fir, no.2 grade, 2" & wider
Specific gravity	G = **0.42**
Tension parallel to grain	F_t = **450** lb/in^2
Compression parallel to grain	F_c = **1150** lb/in^2
Modulus of elasticity	E = **1400000** lb/in^2
Minimum modulus of elasticity	E_{min} = **510000** lb/in^2

Sheathing details

Sheathing material	15/32" wood panel oriented strandboard sheathing
Fastener type	8d common nails at 3"centers

From SDPWS Table 4.3A Nominal Unit Shear Capacities for Wood-Frame Shear Walls - Wood-based Panels

Nominal unit shear capacity for seismic design	v_s = min(980 plf × min[1 - (0.5 - G), 1], 1740 plf) = **901.6** lb/ft
Nominal unit shear capacity for wind design	v_w = min(1370 plf × min[1 - (0.5 - G), 1], 2435 plf) = **1260.4** lb/ft
Apparent shear wall shear stiffness	G_a = **25** kips/in

Loading details

Dead load acting on top of panel	D = **27** lb/ft
Roof live load acting on top of panel	L_r = **27** lb/ft
Snow load acting on top of panel	S = **60** lb/ft
Self weight of panel	S_{wt} = **10** lb/ft^2
In plane wind load acting at head of panel	W = **21630** lbs
Wind load serviceability factor	f_{Wserv} = **0.60**
In plane seismic load acting at head of panel	E_q = **1750** lbs
Design spectral response accel. par., short periods	S_{DS} = **0.184**

From IBC 2015 cl.1605.3.1 Basic load combinations

Load combination no.1	D + 0.6W
Load combination no.2	D + 0.7E
Load combination no.3	D + 0.45W + 0.75L_f + 0.75(L_r or S or R)
Load combination no.4	D + 0.525E + 0.75L_f + 0.75S
Load combination no.5	0.6D + 0.6W
Load combination no.6	0.6D + 0.7E

Adjustment factors

Load duration factor – Table 2.3.2	C_D = **1.60**
Size factor for tension – Table 4A	C_{Ft} = **1.30**
Size factor for compression – Table 4A	C_{Fc} = **1.10**
Wet service factor for tension – Table 4A	C_{Mt} = **1.00**
Wet service factor for compression – Table 4A	C_{Mc} = **1.00**
Wet service factor for modulus of elasticity – Table 4A	C_{ME} = **1.00**
Temperature factor for tension – Table 2.3.3	C_{tt} = **1.00**
Temperature factor for compression – Table 2.3.3	

	C_{tc} = **1.00**
Temperature factor for modulus of elasticity – Table 2.3.3	
	C_{tE} = **1.00**
Incising factor – cl.4.3.8	C_i = **1.00**
Buckling stiffness factor – cl.4.4.2	C_T = **1.00**
Adjusted modulus of elasticity	$E_{min}' = E_{min} \times C_{ME} \times C_{tE} \times C_i \times C_T$ = **510000** psi
Critical buckling design value	$F_{cE} = 0.822 \times E_{min}' / (h / d)^2$ = **325** psi
Reference compression design value	$F_c^* = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i$ = **2024** psi
For sawn lumber	c = **0.8**
Column stability factor – eqn.3.7-1	$C_P = (1 + (F_{cE} / F_c^*)) / (2 \times c) - \sqrt{[(1 + (F_{cE} / F_c^*)) / (2 \times c)]^2 - (F_{cE} / F_c^*) / c}$ = **0.15**

From SDPWS Table 4.3.4 Maximum Shear Wall Aspect Ratios

Maximum shear wall aspect ratio	3.5
Perforated wall length	b_1 = **15.27** ft
Shear wall aspect ratio	h / b_1 = **1.078**
Perforated wall length	b_2 = **14.857** ft
Shear wall aspect ratio	h / b_2 = **1.108**

Shear capacity adjustment factor – cl.4.3.3.5

Sum of perforated shear wall lengths	$\Sigma L_i = b_1 + b_2$ = **30.127** ft
Total length of perforated shear wall	$L_{tot} = b_1 + w_{o1} + b_2$ = **35.5** ft
Total area of openings	$A_o = w_{o1} \times h_{o1}$ = **49.7** ft^2
Sheathing area ratio (eqn. 4.3-6)	$r = 1 / (1 + A_o /(h \times \Sigma L_i))$ = **0.909**
Shear capacity adjustment factor (eqn. 4.3-5)	C_o = **0.906**

Perforated shear wall capacity

Maximum shear force under wind loading	$V_{w\_max} = 0.6 \times W$ = **12.978** kips
Shear capacity for wind loading	$V_w = v_w \times C_o \times \Sigma L_i / 2$ = **17.2** kips
	V_{w_max} / V_w = **0.755**
	PASS - Shear capacity for wind load exceeds maximum shear force
Maximum shear force under seismic loading	$V_{s\_max} = 0.7 \times E_q$ = **1.225** kips
Shear capacity for seismic loading	$V_s = v_s \times C_o \times \Sigma L_i / 2$ = **12.304** kips
	V_{s_max} / V_s = **0.1**
	PASS - Shear capacity for seismic load exceeds maximum shear force

Chord capacity for chords 1 and 2

Load combination 5	
Shear force for maximum tension	$V = 0.6 \times W$ = **12.978** kips
Axial force for maximum tension	$P = (0.6 \times (D + S_{wt} \times h)) \times s / 2$ = **0.077** kips
Maximum tensile force in chord	$T = V \times h / ((C_o \times \Sigma L_i)) - P$ = **7.750** kips
Maximum applied tensile stress	$f_t = T / A_{en}$ = **313** lb/in^2
Design tensile stress	$F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i$ = **936** lb/in^2
	f_t / F_t' = **0.335**
	PASS - Design tensile stress exceeds maximum applied tensile stress
Load combination 1	
Shear force for maximum compression	$V = 0.6 \times W$ = **12.978** kips
Axial force for maximum compression	$P = ((D + S_{wt} \times h)) \times s / 2$ = **0.128** kips
Maximum compressive force in chord	$C = V \times h / ((C_o \times \Sigma L_i)) + P$ = **7.954** kips
Maximum applied compressive stress	$f_c = C / A_e$ = **263** lb/in^2
Design compressive stress	$F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P$ = **314** lb/in^2

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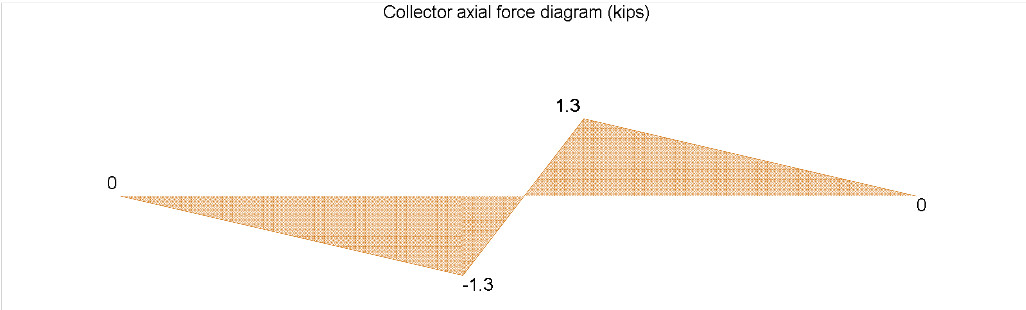
f_c / F_c' = **0.839**

PASS - Design compressive stress exceeds maximum applied compressive stress

Collector capacity



Collector seismic design force factor	F_{Coll} = **1**
Maximum shear force on wall	$V_{max} = \max(F_{Coll} \times V_{s\_max}, V_{w\_max})$ = **12.978** kips
Uniform shear applied to wall	$v_a = V_{max} / ((C_o \times \Sigma L_i))$ = **475.5** plf
Shear resisted by wall segments	$v_b = v_a \times b / (b_1 + b_2)$ = **560.3** plf
Maximum force in collector	P_{coll} = **1.295** kips
Maximum applied tensile stress	$f_t = P_{coll} / (2 \times A_s)$ = **78** lb/in^2
Design tensile stress	$F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i$ = **936** lb/in^2
	f_t / F_t' = **0.084**

PASS - Design tensile stress exceeds maximum applied tensile stress

Maximum applied compressive stress	$f_c = P_{coll} / (2 \times A_s)$ = **78** lb/in^2
Column stability factor	C_P = **1.00**
Design compressive stress	$F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P$ = **2024** lb/in^2
	f_c / F_c' = **0.039**

PASS - Design compressive stress exceeds maximum applied compressive stress

Hold down force

Chord 1	T_1 = **7.75** kips
Chord 2	T_2 = **7.75** kips

Wind load deflection

Design shear force	$V_{\delta w} = f_{Wserv} \times W$ = **12.978** kips
Deflection limit	Δ_{w_allow}= h / 400 = **0.494** in
Induced unit shear	$v_{\delta w\_max} = V_{\delta w} / (C_o \times \Sigma L_i)$ = **475.5** lb/ft
Anchor tension force	$T_\delta = \max(0 \text{ kips}, v_{\delta w\_max} \times h - 0.6 \times (D + S_{wt} \times h) \times s / 2)$ = **7.750** kips
Shear wall deflection – Eqn. 4.3-1	$\delta_{sww} = 2 \times v_{\delta w\_max} \times h^3 / (3 \times E \times A_e \times \Sigma L_i) + v_{\delta w\_max} \times h / (G_a) + h \times T_\delta / (k_a \times \Sigma L_i)$ = **0.411** in
	$\delta_{sww} / \Delta_{w\_allow}$ = **0.832**

PASS - Shear wall deflection is less than deflection limit

Seismic deflection

Design shear force	$V_{\delta s} = E_q$ = **1.75** kips
Deflection limit	Δ_{s_allow}= 0.020 × h = **3.95** in
Induced unit shear	$v_{\delta s\_max} = V_{\delta s} / (C_o \times \Sigma L_i)$ = **64.12** lb/ft
Anchor tension force	$T_\delta = \max(0 \text{ kips}, v_{\delta s\_max} \times h - (0.6 - 0.2 \times S_{DS}) \times (D + S_{wt} \times h) \times s / 2)$ = **0.983** kips

Shear wall elastic deflection – Eqn. 4.3-1	$\delta_{swse} = 2 \times v_{\delta s\_max} \times h^3 / (3 \times E \times A_e \times \Sigma L_i) + v_{\delta s\_max} \times h / (G_a) + h \times T_\delta / (k_a \times \Sigma L_i)$ = **0.055** in
Deflection ampification factor	$C_{d\delta}$ = **4**
Seismic importance factor	I_e = **1.25**
Amp. seis. deflection – ASCE7 Eqn. 12.8-15	$\delta_{sws} = C_{d\delta} \times \delta_{swse} / I_e$ = **0.175** in
	$\delta_{sws} / \Delta_{s\_allow}$ = **0.044**

PASS - Shear wall deflection is less than deflection limit

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STORE FOUNDATION LINE LOADS

Foundation Line 1

DL_{1_roof} = (20 psf * 35.5 ft * 0.5) = **355.000** plf
DL_{1_wall} = 10 psf * 22 ft = **220.000** plf
LL_{1_roof} = (20 psf * 35.5 ft * 0.5) = **355.000** plf
SL_{1_roof} = (15 psf * 35.5 ft *0.5) = **266.250** plf

Foundation Line 2

DL_{2_roof} = DL_{1_roof} = **355.000** plf
DL_{2_wall} = 10 psf * 20.667 ft = **206.670** plf
LL_{2_roof} = LL_{1_roof} = **355.000** plf
SL_{2_roof} =SL_{1_roof} = **266.250** plf

Foundation Line A

DL_{A_roof} = (20 psf * 32 in * 0.5) = **26.667** plf
DL_{A_wall} = 10 psf * 20.667 ft = **206.670** plf
LL_{A_roof} = (20 psf * 32 in * 0.5) = **26.667** plf
SL_{A_roof} = (44.8 psf * 32 in * 0.5) = **59.733** plf

Foundation Line F

DL_{F_roof} = DL_{A_roof} = **26.667** plf
DL_{F_wall} = DL_{A_wall} = **206.670** plf
LL_{F_roof} = LL_{A_roof} = **26.667** plf
SL_{F_roof} = SL_{A_roof} = **59.733** plf

Loads do not include self weight of concrete footings

WALL 3 HSS COLUMN UPLIFT (C&C)

Roof Dead Load	DL = 20 psf
Tributary Area 1	A_1 = (17.75 ft * 18.0 ft) = **319.500** ft^2
Area 1 Uplift Pressure	p_1 = -32.5 psf
Uplift Force 1	W_1 = 0.6 * (A_1 * DL) + 0.6 * (A_1 * p_1) = **-2396.250** lbf
Tributary Area 2	A_2 = (17.75 ft * 15.7 ft) = **278.675** ft^2
Area 2 Uplift Pressure	p_2 = -32.5 psf
Uplift Force 2	W_2 = 0.6 * (A_2 * DL) + 0.6 * (A_2 * p_2) = **-2090.062** lbf
Tributary Area 3	A_3 = (17.75 ft * 13.7 ft) = **243.175** ft^2
Area 3 Uplift Pressure	p_3 = -32.5 psf
Uplift Force 3	W_3 = 0.6 * (A_3 * DL) + 0.6 * (A_3 * p_3) = **-1823.813** lbf
Tributary Area 4	A_4 = (17.75 ft * 15.0 ft) = **266.250** ft^2
Area 4 Uplift Pressure	p_4 = -32.5 psf
Uplift Force 4	W_4 = 0.6 * (A_4 * DL) + 0.6 * (A_4 * p_4) = **-1996.875** lbf
Tributary Area Left End	A_L = (17.75 ft * 8.977 ft) = **159.342** ft^2
Area Left End Uplift Pressure	p_L = -32.5 psf
Additional Uplift Force Left End	W_L = 0.6 * (A_L * DL) + 0.6 * (A_L * p_L) = **-1195.063** lbf
Tributary Area Right End	A_R = (17.75 ft * 7.937 ft) = **140.882** ft^2
Area Right End Uplift Pressure	p_R = -32.5 psf
Additional Uplift Force Right End	W_R = 0.6 * (A_R * DL) + 0.6 * (A_R * p_R) = **-1056.613** lbf

Adjusted Wall Capacity Table - SIPA Code Report

*Highlighted cells are controlled by zero bearing end reaction connection

Load Duration Factor (C_D) = 1.6

8d **Common nail @** 6 **"oc**

	Panel Thickness								
	4.625			6.5			8.25		
Panel Length	L/180	L/240	L/360	L/180	L/240	L/360	L/180	L/240	L/360
8	46.7	46.7	34.0	48.7	48.7	48.7	49.7	49.7	49.7
10	37.3	33.0	22.0	38.3	38.3	38.0	40.3	40.3	40.3
12	30.0	23.0	15.0	32.4	32.4	27.0	33.4	33.4	33.4
14	21.0	16.0		27.4	27.4	19.0	28.4	28.4	28.4
16				23.8	22.0	14.0	24.8	24.8	22.0
18				21.6	16.0		21.6	21.6	17.0
20							19.7	19.7	13.0

8d **Common nail @** 4 **"oc**

	Panel Thickness								
	4.625			6.5			8.25		
Panel Length	L/180	L/240	L/360	L/180	L/240	L/360	L/180	L/240	L/360
8	57.5	51.0	34.0	59.5	59.5	56.0	60.5	60.5	60.5
10	45.0	33.0	22.0	47.0	47.0	38.0	49.0	49.0	49.0
12	30.0	23.0	15.0	39.7	39.7	27.0	40.7	40.7	39.0
14	21.0	16.0		33.6	29.0	19.0	34.6	34.6	29.0
16				29.0	22.0	14.0	30.2	30.2	22.0
18				22.0	16.0		26.4	25.0	17.0
20							24.0	20.0	13.0

8d **Common nail @** 3 **"oc**

	Panel Thickness								
	4.625			6.5			8.25		
Panel Length	L/180	L/240	L/360	L/180	L/240	L/360	L/180	L/240	L/360
8	68.0	51.0	34.0	70.3	70.3	56.0	71.3	71.3	71.3
10	45.0	33.0	22.0	55.7	55.7	38.0	57.7	57.7	54.0
12	30.0	23.0	15.0	46.9	40.0	27.0	47.9	47.9	39.0
14	21.0	16.0		39.0	29.0	19.0	40.8	40.8	29.0
16				29.0	22.0	14.0	35.7	33.0	22.0
18				22.0	16.0		31.3	25.0	17.0
20							26.0	20.0	13.0

#8 **Screw (1" min. pen.) @** 6 **"oc**

	Panel Thickness								
	4.625			6.5			8.25		
Panel Length	L/180	L/240	L/360	L/180	L/240	L/360	L/180	L/240	L/360
8	51.0	51.0	34.0	53.0	53.0	53.0	54.0	54.0	54.0
10	40.8	33.0	22.0	41.8	41.8	38.0	43.8	43.8	43.8
12	30.0	23.0	15.0	35.3	35.3	27.0	36.3	36.3	36.3
14	21.0	16.0		29.9	29.0	19.0	30.9	30.9	29.0
16				26.0	22.0	14.0	27.0	27.0	22.0
18				22.0	16.0		23.6	23.6	17.0
20							21.4	20.0	13.0

#8 **Screw (1" min. pen.) @** 4 **"oc**

	Panel Thickness								
	4.625			6.5			8.25		
Panel Length	L/180	L/240	L/360	L/180	L/240	L/360	L/180	L/240	L/360
8	64.0	51.0	34.0	66.0	66.0	56.0	67.0	67.0	67.0
10	45.0	33.0	22.0	52.2	52.2	38.0	54.2	54.2	54.0
12	30.0	23.0	15.0	44.0	40.0	27.0	45.0	45.0	39.0
14	21.0	16.0		37.3	29.0	19.0	38.3	38.3	29.0
16				29.0	22.0	14.0	33.5	33.0	22.0
18				22.0	16.0		29.3	25.0	17.0
20							26.0	20.0	13.0

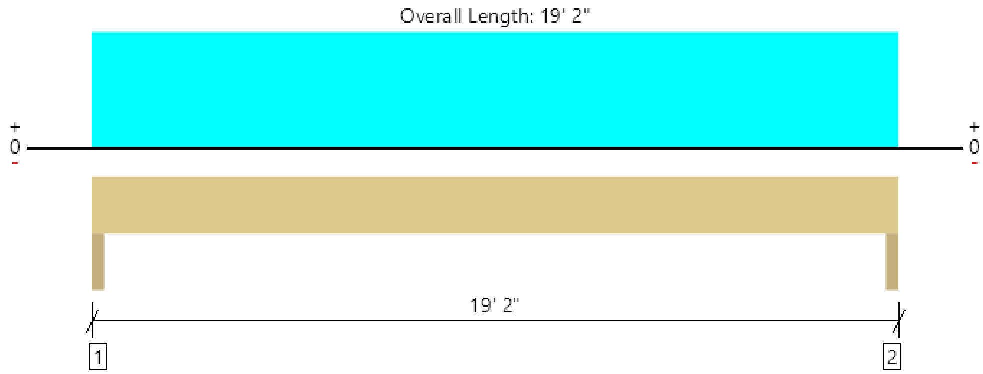
#8 **Screw (1" min. pen.) @** 3 **"oc**

	Panel Thickness								
	4.625			6.5			8.25		
Panel Length	L/180	L/240	L/360	L/180	L/240	L/360	L/180	L/240	L/360
8	68.0	51.0	34.0	79.0	79.0	56.0	80.0	80.0	78.0
10	45.0	33.0	22.0	62.6	57.0	38.0	64.6	64.6	54.0
12	30.0	23.0	15.0	51.0	40.0	27.0	53.7	53.7	39.0
14	21.0	16.0		39.0	29.0	19.0	45.7	43.0	29.0
16				29.0	22.0	14.0	40.0	33.0	22.0
18				22.0	16.0		34.0	25.0	17.0
20							26.0	20.0	13.0

Level			
Member Name	Results	Current Solution	Comments
Header: Wall 1 Max	Passed	3 piece(s) 1 3/4" x 16" 2.0E Microllam® LVL	
Header: Wall 3 Max	Passed	3 piece(s) 1 3/4" x 16" 2.0E Microllam® LVL	
Header: Wall 3 Critical	Passed	3 piece(s) 1 3/4" x 16" 2.0E Microllam® LVL	
Header: Wall 4	Passed	3 piece(s) 1 3/4" x 14" 2.0E Microllam® LVL	

Level, Header: Wall 1 Max

3 piece(s) 1 3/4" x 16" 2.0E Microllam® LVL



All locations are measured from the outside face of left support (or left cantilever end). All dimensions are horizontal.

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	7414 @ 1 1/2"	11419 (3.00")	Passed (65%)	--	1.0 D + 1.0 Lr (All Spans)
Shear (lbs)	6189 @ 1' 7"	19950	Passed (31%)	1.25	1.0 D + 1.0 Lr (All Spans)
Moment (Ft-lbs)	34606 @ 9' 7"	58339	Passed (59%)	1.25	1.0 D + 1.0 Lr (All Spans)
Live Load Defl. (in)	0.307 @ 9' 7"	0.631	Passed (L/739)	--	1.0 D + 1.0 Lr (All Spans)
Total Load Defl. (in)	0.669 @ 9' 7"	0.946	Passed (L/339)	--	1.0 D + 1.0 Lr (All Spans)

System : Wall
Member Type : Header
Building Use : Residential
Building Code : IBC 2018
Design Methodology : ASD

- Deflection criteria: LL (L/360) and TL (L/240).
- Allowed moment does not reflect the adjustment for the beam stability factor.

Supports	Bearing Length			Loads to Supports (lbs)				Accessories
	Total	Available	Required	Dead	Roof Live	Snow	Factored	
1 - Trimmer - SPF	3.00"	3.00"	1.95"	4012	3402	2549	7414	None
2 - Trimmer - SPF	3.00"	3.00"	1.95"	4012	3402	2549	7414	None

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	9' 7" o/c	
Bottom Edge (Lu)	19' 2" o/c	

•Maximum allowable bracing intervals based on applied load.

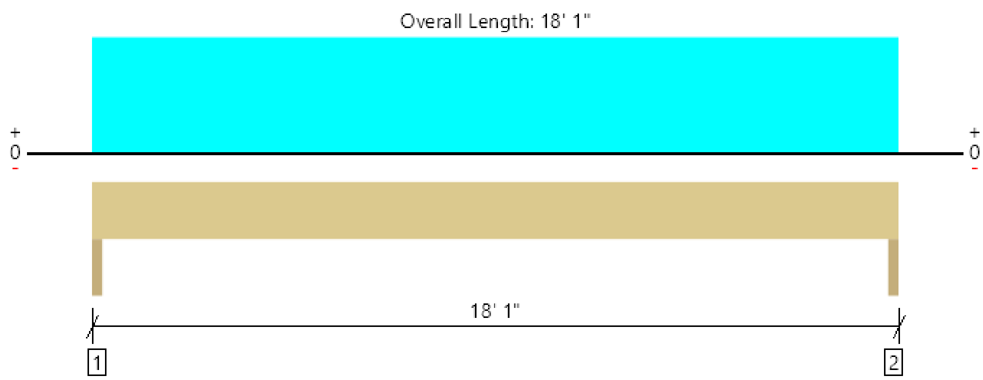
Vertical Loads	Location	Tributary Width	Dead (0.90)	Roof Live (non-snow: 1.25)	Snow (1.15)	Comments
0 - Self Weight (PLF)	0 to 19' 2"	N/A	24.5	--	--	
1 - Uniform (PLF)	0 to 19' 2"	N/A	355.0	355.0	266.0	roof load
2 - Uniform (PSF)	0 to 19' 2"	3' 11"	10.0	-	-	wall load

Weyerhaeuser Notes

ForteWEB Software Operator	Job Notes
Mike Streicher Pinnacle Engineering (513) 247-1090 mstreicher@pinneng.com	

Level, Header: Wall 3 Max

3 piece(s) 1 3/4" x 16" 2.0E Microllam® LVL



All locations are measured from the outside face of left support (or left cantilever end). All dimensions are horizontal.

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	6995 @ 1"	9516 (2.50")	Passed (74%)	--	1.0 D + 1.0 Lr (All Spans)
Shear (lbs)	5803 @ 1' 6 1/2"	19950	Passed (29%)	1.25	1.0 D + 1.0 Lr (All Spans)
Moment (Ft-lbs)	31044 @ 9' 1/2"	58339	Passed (53%)	1.25	1.0 D + 1.0 Lr (All Spans)
Live Load Defl. (in)	0.249 @ 9' 1/2"	0.597	Passed (L/863)	--	1.0 D + 1.0 Lr (All Spans)
Total Load Defl. (in)	0.543 @ 9' 1/2"	0.896	Passed (L/396)	--	1.0 D + 1.0 Lr (All Spans)

- Deflection criteria: LL (L/360) and TL (L/240).
- Allowed moment does not reflect the adjustment for the beam stability factor.

System : Wall
Member Type : Header
Building Use : Residential
Building Code : IBC 2018
Design Methodology : ASD

Supports	Bearing Length			Loads to Supports (lbs)				Accessories
	Total	Available	Required	Dead	Roof Live	Snow	Factored	
1 - Trimmer - SPF	2.50"	2.50"	1.84"	3785	3210	2405	6995	None
2 - Trimmer - SPF	2.50"	2.50"	1.84"	3785	3210	2405	6995	None

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	11' 2" o/c	
Bottom Edge (Lu)	18' 1" o/c	

•Maximum allowable bracing intervals based on applied load.

Vertical Loads	Location	Tributary Width	Dead (0.90)	Roof Live (non-snow: 1.25)	Snow (1.15)	Comments
0 - Self Weight (PLF)	0 to 18' 1"	N/A	24.5	--	--	
1 - Uniform (PLF)	0 to 18' 1"	N/A	355.0	355.0	266.0	roof load
2 - Uniform (PSF)	0 to 18' 1"	3' 11"	10.0	-	-	wall load

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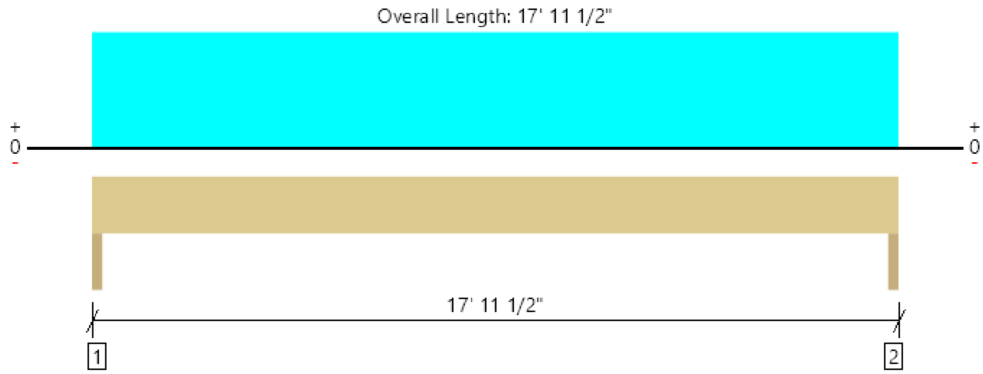
The product application, input design loads, dimensions and support information have been provided by ForteWEB Software Operator

ForteWEB Software Operator	Job Notes
Mike Streicher Pinnacle Engineering (513) 247-1090 mstreicher@pinneng.com	

Level, Header: Wall 3 Critical

3 piece(s) 1 3/4" x 16" 2.0E Microllam® LVL



All locations are measured from the outside face of left support (or left cantilever end). All dimensions are horizontal.

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	6947 @ 1"	9516 (2.50")	Passed (73%)	--	1.0 D + 1.0 Lr (All Spans)
Shear (lbs)	5754 @ 1' 6 1/2"	19950	Passed (29%)	1.25	1.0 D + 1.0 Lr (All Spans)
Moment (Ft-lbs)	30612 @ 8' 11 3/4"	58339	Passed (52%)	1.25	1.0 D + 1.0 Lr (All Spans)
Live Load Defl. (in)	0.243 @ 8' 11 3/4"	0.593	Passed (L/880)	--	1.0 D + 1.0 Lr (All Spans)
Total Load Defl. (in)	0.529 @ 8' 11 3/4"	0.890	Passed (L/404)	--	1.0 D + 1.0 Lr (All Spans)

- Deflection criteria: LL (L/360) and TL (L/240).
- Allowed moment does not reflect the adjustment for the beam stability factor.

System : Wall
Member Type : Header
Building Use : Residential
Building Code : IBC 2018
Design Methodology : ASD

Supports	Bearing Length			Loads to Supports (lbs)				Accessories
	Total	Available	Required	Dead	Roof Live	Snow	Factored	
1 - Trimmer - SPF	2.50"	2.50"	1.83"	3759	3188	2388	6947	None
2 - Trimmer - SPF	2.50"	2.50"	1.83"	3759	3188	2388	6947	None

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	11' 4" o/c	
Bottom Edge (Lu)	18' o/c	

•Maximum allowable bracing intervals based on applied load.

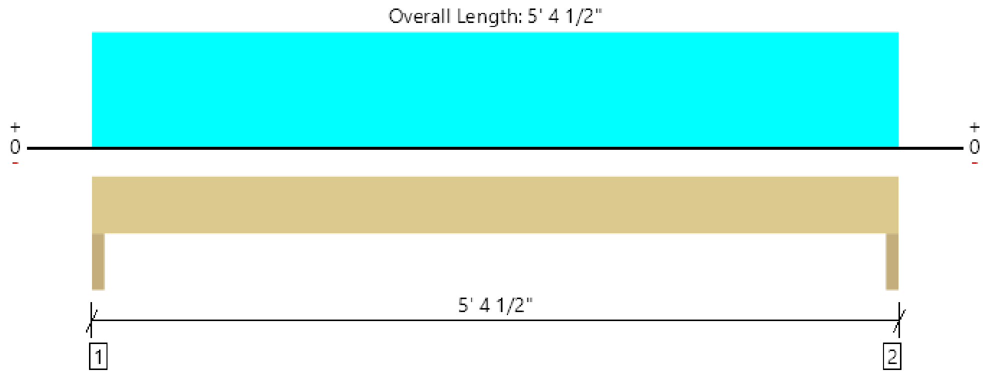
Vertical Loads	Location	Tributary Width	Dead (0.90)	Roof Live (non-snow: 1.25)	Snow (1.15)	Comments
0 - Self Weight (PLF)	0 to 17' 11 1/2"	N/A	24.5	--	--	
1 - Uniform (PLF)	0 to 17' 11 1/2"	N/A	355.0	355.0	266.0	roof load
2 - Uniform (PSF)	0 to 17' 11 1/2"	3' 11"	10.0	-	-	wall load

Weyerhaeuser Notes

ForteWEB Software Operator	Job Notes
Mike Streicher Pinnacle Engineering (513) 247-1090 mstreicher@pinneng.com	

Level, Header: Wall 4

3 piece(s) 1 3/4" x 14" 2.0E Microllam® LVL



All locations are measured from the outside face of left support (or left cantilever end). All dimensions are horizontal.

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	501 @ 1 1/2"	11419 (3.00")	Passed (4%)	--	1.0 D + 1.0 S (All Spans)
Shear (lbs)	237 @ 1' 5"	16060	Passed (1%)	1.15	1.0 D + 1.0 S (All Spans)
Moment (Ft-lbs)	612 @ 2' 8 1/4"	41846	Passed (1%)	1.15	1.0 D + 1.0 S (All Spans)
Live Load Defl. (in)	0.001 @ 2' 8 1/4"	0.171	Passed (L/999+)	--	1.0 D + 1.0 S (All Spans)
Total Load Defl. (in)	0.002 @ 2' 8 1/4"	0.256	Passed (L/999+)	--	1.0 D + 1.0 S (All Spans)

- Deflection criteria: LL (L/360) and TL (L/240).
- Allowed moment does not reflect the adjustment for the beam stability factor.

System : Wall
Member Type : Header
Building Use : Residential
Building Code : IBC 2018
Design Methodology : ASD

Supports	Bearing Length			Loads to Supports (lbs)				Accessories
	Total	Available	Required	Dead	Roof Live	Snow	Factored	
1 - Trimmer - SPF	3.00"	3.00"	1.50"	340	73	161	501	None
2 - Trimmer - SPF	3.00"	3.00"	1.50"	340	73	161	501	None

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	5' 5" o/c	
Bottom Edge (Lu)	5' 5" o/c	

•Maximum allowable bracing intervals based on applied load.

Vertical Loads	Location	Tributary Width	Dead (0.90)	Roof Live (non-snow: 1.25)	Snow (1.15)	Comments
0 - Self Weight (PLF)	0 to 5' 4 1/2"	N/A	21.5	--	--	
1 - Uniform (PLF)	0 to 5' 4 1/2"	N/A	27.0	27.0	60.0	roof load
2 - Uniform (PSF)	0 to 5' 4 1/2"	7' 9 5/8"	10.0	-	-	wall load

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The product application, input design loads, dimensions and support information have been provided by ForteWEB Software Operator

RAM® Elements CONNECT Edition

Current Date: 1/31/2023 13:47
Units system: English
File name: P:\23\11-20\23018 - Circle K - Angier, NC\Engineering\HSS Col Design.retx

Steel Code Check

Report: Comprehensive

Members: Hot-rolled
Design code: AISC 360-2016 LRFD

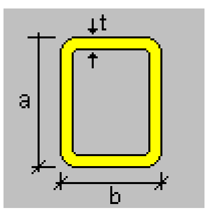
Member	:	**1 (HSS Col)**
Design status	:	**OK**

DESIGN WARNINGS

Section information

Section name: HSS_SQR 4X4X1_4 **(US)**

Dimensions



a	=	4.000	[in]	Height
b	=	4.000	[in]	Width
T	=	0.233	[in]	Thickness

Properties

Section properties	Unit	Major axis	Minor axis
Gross area of the section. (Ag)	[in2]	3.370	
Moment of Inertia (local axes) (I)	[in4]	7.800	7.800
Moment of Inertia (principal axes) (I')	[in4]	7.800	7.800
Bending constant for moments (principal axis) (J')	[in]	0.000	0.000
Radius of gyration (local axes) (r)	[in]	1.521	1.521
Radius of gyration (principal axes) (r')	[in]	1.521	1.521
Saint-Venant torsion constant. (J)	[in4]	12.800	
Section warping constant. (Cw)	[in6]	0.000	
Distance from centroid to shear center (principal axis) (xo,yo)	[in]	0.000	0.000
Top elastic section modulus of the section (local axis) (Ssup)	[in3]	3.900	3.900
Bottom elastic section modulus of the section (local axis) (Sinf)	[in3]	3.900	3.900
Top elastic section modulus of the section (principal axis) (S'sup)	[in3]	3.900	3.900
Bottom elastic section modulus of the section (principal axis) (S'inf)	[in3]	3.900	3.900
Plastic section modulus (local axis) (Z)	[in3]	4.700	4.700
Plastic section modulus (principal axis) (Z')	[in3]	4.700	4.700
Polar radius of gyration. (ro)	[in]	2.150	
Area for shear (Aw)	[in2]	1.538	1.538
Torsional constant. (C)	[in3]	6.563	

Material : A500 GrB rectangular

Properties	Unit	Value
Yield stress (Fy):	[Kip/in2]	46.00
Tensile strength (Fu):	[Kip/in2]	58.00
Elasticity Modulus (E):	[Kip/in2]	29000.00
Shear modulus for steel (G):	[Kip/in2]	11153.85

DESIGN CRITERIA

Description	Unit	Value
Length for tension slenderness ratio (L)	[ft]	10.00

Distance between member lateral bracing points

Length (Lb) [ft]	
Top	Bottom
10.00	10.00

Laterally unbraced length

Length [ft]			Effective length factor		
Major axis(L33)	Minor axis(L22)	Torsional axis(Lt)	Major axis(K33)	Minor axis(K22)	Torsional axis(Kt)
10.00	10.00	10.00	1.0	1.0	1.0

Additional assumptions

Continuous lateral torsional restraint	No
Tension field action	No
Continuous flexural torsional restraint	No
Effective length factor value type	None
Major axis frame type	Sway
Minor axis frame type	Sway

DESIGN CHECKS

AXIAL TENSION DESIGN ✔

<u>Axial tension</u>

Ratio	:	0.01			
Capacity	:	139.52 [Kip]	Reference	:	Cl.D2
Demand	:	1.70 [Kip]	Ctrl Eq.	:	D10 at 0.00%

Intermediate results	Unit	Value	Reference
Factored axial tension capacity(ϕP_n):	[Kip]	139.52	Cl.D2
Nominal axial tension capacity (P_n)	[Kip]	155.02	Eq.D2-1

AXIAL COMPRESSION DESIGN ✔

<u>Compression in the major axis 33</u>

Ratio	:	0.15			
Capacity	:	91.81 [Kip]	Reference	:	Cl.E3
Demand	:	13.94 [Kip]	Ctrl Eq.	:	D2 at 0.00%

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	--	Non slender	
Unstiffened element slenderness (λ)	--	14.17	
Unstiffened element limiting slenderness (λ_r)	--	35.15	Table.4.1a.Case6
Stiffened element classification	--	Non slender	
Stiffened element slenderness (λ)	--	14.17	
Stiffened element limiting slenderness (λ_r)	--	35.15	Table.4.1a.Case6
Factored flexural buckling strength(ϕP_{n33}):	[Kip]	91.81	Cl.E3
Unbraced length (L_{33})	[ft]	10.00	Cl.E2
Effective slenderness ($(KL/r)_{33}$)	--	78.88	Cl.E2
Elastic critical buckling stress (F_{e33})	[Kip/in2]	46.00	Eq.E3-4
Effective area of the cross section based on the effective width (A...	[in2]	3.37	
Critical stress for flexural buckling (F_{cr33})	[Kip/in2]	30.27	Eq.E3-2
Nominal flexural buckling strength (P_{n33})	[Kip]	102.01	Eq.E3-1

Compression in the minor axis 22

Ratio	:	0.15			
Capacity	:	91.81 [Kip]	Reference	:	Cl.E3
Demand	:	13.94 [Kip]	Ctrl Eq.	:	D2 at 0.00%

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	--	Non slender	
Unstiffened element slenderness (λ)	--	14.17	
Unstiffened element limiting slenderness (λ_r)	--	35.15	Table.4.1a.Case6
Stiffened element classification	--	Non slender	
Stiffened element slenderness (λ)	--	14.17	
Stiffened element limiting slenderness (λ_r)	--	35.15	Table.4.1a.Case6
Factored flexural buckling strength(ϕP_{n22}):	[Kip]	91.81	Cl.E3
Unbraced length (L_{22})	[ft]	10.00	Cl.E2
Effective slenderness ($(KL/r)_{22}$)	--	78.88	Cl.E2
Elastic critical buckling stress (F_{e22})	[Kip/in2]	46.00	Eq.E3-4
Effective area of the cross section based on the effective width (A...	[in2]	3.37	
Critical stress for flexural buckling (F_{cr22})	[Kip/in2]	30.27	Eq.E3-2
Nominal flexural buckling strength (P_{n22})	[Kip]	102.01	Eq.E3-1

FLEXURAL DESIGN ✔

Bending about major axis, M33

Ratio	:	0.00			
Capacity	:	16.22 [Kip*ft]	Reference	:	Cl.F7.1
Demand	:	0.00 [Kip*ft]	Ctrl Eq.	:	D1 at 0.00%

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	--	Compact	
Unstiffened element slenderness (λ)	--	14.17	
Limiting slenderness for noncompact unstiffened element (λ_r)	--	35.15	
Limiting slenderness for compact unstiffened element (λ_p)	--	28.12	
Stiffened element classification	--	Compact	
Stiffened element slenderness (λ)	--	14.17	
Limiting slenderness for noncompact stiffened element (λ_r)	--	143.12	
Limiting slenderness for compact stiffened element (λ_p)	--	60.76	
Factored yielding strength(ϕM_n):	[Kip*ft]	16.22	Cl.F7.1
Yielding (M_n)	[Kip*ft]	18.02	Eq.F7-1

Bending about minor axis, M22

Ratio	: 0.00		
Capacity	: 16.22 [Kip*ft]	Reference	: Cl.F7.1
Demand	: 0.00 [Kip*ft]	Ctrl Eq.	: D1 at 0.00%

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	--	Compact	
Unstiffened element slenderness (λ)	--	14.17	
Limiting slenderness for noncompact unstiffened element (λ_r)	--	35.15	
Limiting slenderness for compact unstiffened element (λ_p)	--	28.12	
Stiffened element classification	--	Compact	
Stiffened element slenderness (λ)	--	14.17	
Limiting slenderness for noncompact stiffened element (λ_r)	--	143.12	
Limiting slenderness for compact stiffened element (λ_p)	--	60.76	
Factored yielding strength about a geometric axis(ϕM_n):	[Kip*ft]	16.22	Cl.F7.1
Yielding (M_n)	[Kip*ft]	18.02	Eq.F7-1

DESIGN FOR SHEAR ✔

Shear in major axis 33

Ratio	: 0.00		
Capacity	: 38.21 [Kip]	Reference	: Cl.G1
Demand	: 0.00 [Kip]	Ctrl Eq.	: D1 at 0.00%

Intermediate results	Unit	Value	Reference
Factored shear capacity(ϕV_n):	[Kip]	38.21	Cl.G1
Web buckling coefficient (k_v)	--	5.00	Cl.G4
Web buckling coefficient (C_v)	--	1.00	Eq.G2-9
Nominal shear strength (V_n)	[Kip]	42.46	Eq.G4-1

Shear in minor axis 22

Ratio	: 0.00		
Capacity	: 38.21 [Kip]	Reference	: Cl.G1
Demand	: 0.00 [Kip]	Ctrl Eq.	: D1 at 0.00%

Intermediate results	Unit	Value	Reference
Factored shear capacity(ϕV_n):	[Kip]	38.21	Cl.G1
Web buckling coefficient (k_v)	--	5.00	Cl.G4
Web buckling coefficient (C_v)	--	1.00	Eq.G2-9
Nominal shear strength (V_n)	[Kip]	42.46	Eq.G4-1

TORSION DESIGN ✔

Torsion

Ratio	: 0.00		
Capacity	: 13.58 [Kip*ft]	Reference	: Cl.H3.1
Demand	: 0.00 [Kip*ft]	Ctrl Eq.	: D1 at 0.00%

Intermediate results	Unit	Value	Reference
Factored torsion capacity(ϕT_n):	[Kip*ft]	13.58	Cl.H3.1
Critical torsional buckling stress (F_{cr})	[Kip/in2]	27.60	Eq.H3-3
Nominal torsion capacity (T_n)	[Kip*ft]	15.09	Eq.H3-1

COMBINED ACTIONS DESIGN ✔

Combined flexure and axial

Ratio	: 0.08		
Ctrl Eq.	: D2 at 0.00%	Reference	: Eq.H1-1b

Intermediate results	Unit	Value	Reference
Interaction of flexure and axial force:	--	0.08	Eq.H1-1b
Available flexural strength about strong axis (M_{c33})	[Kip*ft]	16.22	Cl.H1.1
Available flexural strength about weak axis (M_{c22})	[Kip*ft]	16.22	Cl.H1.1
Available axial strength (P_c)	[Kip]	91.81	Cl.H1.1

ICC EVALUATION SERVICE®

ICC-ES Evaluation Report

ESR-4692

Reissued April 2022

This report is subject to renewal April 2023.

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DIVISION: 06 00 00—WOOD, PLASTICS, AND COMPOSITES
Section: 06 12 00—Structural Panels

REPORT HOLDER:

PORTERCORP

EVALUATION SUBJECT:

STRUCTURAL INSULATED PANELS

1.0 EVALUATION SCOPE

Compliance with the following codes:

- 2018 and 2015 *International Building Code®* (IBC)
- 2018 and 2015 *International Residential Code®* (IRC)

Property evaluated:

- Structural

2.0 USES

2.1 General:

Structural Insulated Panels are used as structural insulated roof and wall panels capable of resisting transverse, axial and in-plane shear loads.

2.2 Construction Types:

Structural Insulated Panels shall be considered combustible building elements when determining the construction type in accordance with IBC Chapter 6.

2.3 Fire Resistive Assemblies:

Structural Insulated Panels shall not be used as part of a fire-rated assembly unless suitable evidence and details are submitted and approved by the authority having jurisdiction.

3.0 DESCRIPTION

3.1 General:

Structural Insulated Panels are factory-assembled, engineered-wood-faced, structural insulated panels (SIPs) with an expanded polystyrene (EPS) foam core. The product is intended for use as load-bearing or non-load-bearing wall and roof panels. Structural Insulated Panels are available in $4^5/_8$-inch (117.5 mm) through 15-inch (381 mm) overall thicknesses and are custom-made to the specifications for each use. The maximum product size is 8 feet (2438 mm) wide and up to 24 feet (7315 mm) in length.

3.2 Materials:

3.2.1 Facing: The facing consists of two single-ply oriented strand board (OSB) facings a minimum of $^7/_{16}$-inch-thick (11.1 mm) conforming to the properties shown in Table 3. Additionally, facing materials shall conform to DOC PS 2, Exposure 1, Rated Sheathing with a span index of 24/16. Panels may be manufactured with the facing strength axis oriented in either direction with respect to the direction of product bending provided the appropriate design values are used.

3.2.2 Core: The core material is EPS foam plastic insulation conforming to ASTM C578, Type I. The foam core, up to 4-inch (101.6 mm) thickness, has a flame spread rating not exceeding 75 and a smoke-developed rating not exceeding 450 when tested in accordance with ASTM E84. Cores used in structural insulated panels up to 15 inches (381 mm) thick, comply with IBC Section 2603.3 Exception 4.

3.2.3 Adhesive: Facing materials are adhered to the core material using a thin-film adhesive. The adhesive is applied during the lamination process in accordance with the in-plant quality system documentation.

3.2.4 Material Sources: The facing, core and adhesive used in the construction of Structural Insulated Panels must be materials from approved sources as identified in the in-plant quality system documentation. A list of material suppliers is provided in Table 16.

3.2.5 Splines: Structural Insulated Panels are interconnected with surface splines, block splines, or I-joists (Figure 1). Connections using dimensional lumber splines or engineered structural splines not specifically addressed in this report must be designed in accordance with accepted engineering practice to meet applicable code requirements.

3.2.5.1 Surface Splines: Surface splines (Figure 1) consist of 3-inch-wide (76.2 mm) by $^7/_{16}$-inch-thick (11.1 mm) or thicker OSB. At each panel joint, one surface spline is inserted into each of two tight-fitting slots in the core. The slots in the core are located just inside the facing.

3.2.5.2 Block Splines: Block splines (Figure 1) are manufactured in the same manner as the SIP except with an overall thickness that is 1 inch (25.4 mm) less than the overall thickness of the panels to be joined.

3.2.5.3 I-Joist Splines: Structural capacities for prefabricated wood I-joists splines (Figure 1) shall be established and monitored in accordance with ASTM D5055 with properties equal to or greater than those shown in Table 4. The overall depth of the joist is 1 inch (25.4 mm) less than the overall thickness of the panels to be joined.

4.0 DESIGN AND INSTALLATION

4.1 Design:

4.1.1 Overall Structural System: The scope of this report is limited to the evaluation of the SIP component. Panel connections and other details related to incorporation of the product into the overall structural system of a building are beyond the scope of this report.

4.1.2 Design Approval: Where required by the authority having jurisdiction, structures using Structural Insulated Panels shall be designed by a registered design professional. Construction documents, including engineering calculations and drawings providing floor plans, window details, door details and connector details, shall be submitted to the code official when application is made for a permit. The individual preparing such documents shall possess the necessary qualifications as required by the applicable code and the professional registration laws of the state where the construction is undertaken. Approved construction documents shall be available at all times on the jobsite during installation.

4.1.3 Design Loads: Design loads to be resisted by the product shall be as required under the applicable code. Loads on the panels shall not exceed the loads noted in this report. Where loading conditions result in superimposed stresses, the sum of the ratio of actual loads over allowable loads shall not exceed one. Calculations demonstrating that the loads applied are less than the allowable loads described in this report shall be submitted to the code official for approval.

4.1.4 Allowable Loads: Allowable axial, transverse and in-plane shear loads may be calculated using the panel properties provided in Tables 1, 2 and 4 or selected from Tables 5 through 15. For loading conditions not specifically addressed herein, structural members designed in accordance with accepted engineering practice shall be provided to meet applicable code requirements.

4.1.5 Concentrated Loads: Axial loads shall be applied to the product through continuous members such as structural insulated roof or floor panels or repetitive members such as joists, trusses or rafters spaced at regular intervals of 24 inches (610 mm) on center or less. Such members shall be fastened to a rim board or similar member to distribute the load to the product. For other loading conditions, reinforcement shall be provided. This reinforcement shall be designed in accordance with accepted engineering practice.

4.1.6 Eccentric and Side Loads: Axial loads shall be applied concentrically to the top of the product. Loads shall not be applied eccentrically or through framing attached to one side of the panel (such as balloon framing) except where additional engineering documentation is provided.

4.1.7 Openings: Openings in panels are permitted when the header depth is at least 12 inches (305 mm), and the interior of the opening is reinforced with minimum 0.42 SG lumber graded #2 around the perimeter, secured in place with not less than 0.131-inch x $2^1/_2$-inch (2.9 mm x 63.5 mm) nails, spaced 6 inches (152 mm) on center. The panels are not used to resist in-plane shear loads. SIP splines are not permitted within 6 inches of the end of the header and are not permitted within the header. Allowable loads for maximum header spans of 36 inches may be selected from Tables 10 and 12. Allowable loads for maximum header spans of 72 inches (1829 mm) may be selected from Tables 11 and 13. Openings in panels beyond the scope of this report shall be reinforced with wood or steel designed in accordance with accepted engineering practice to resist all loads applied to the opening as required by the adopted code. Details for door and window openings shall be provided to clarify the manner of supporting axial, transverse and/or in-plane shear loads at openings. Such details shall be subject to approval by the local authority having jurisdiction.

4.1.8 In-Plane Shear Design: Shear walls utilizing block or surface splines shall be sized to resist all code required wind and seismic loads without exceeding the allowable loads provided herein. Shear wall chords, hold-downs and connections to transfer shear forces between the wall and surrounding structure shall be designed in accordance with accepted engineering practice. Allowable strengths for SIP shear walls with structural splines along each panel edge shall be designed in accordance with accepted engineering practice and are subject to the limitations for wood sheathed shear walls.

4.1.9 Seismic Design Categories A, B, and C: Use of the shear wall configurations in Table 14 is limited to structures in Seismic Design Categories A, B and C. Where SIPs are used to resist seismic forces the following factors shall be used for design: Response Modification Coefficient, R = 2.0; System Overstrength Factor, Ω_0 = 2.5; Deflection Amplification Factor, C_d = 2.0. The maximum panel height-to-width ratio shall be 2:1.

4.1.10 Horizontal Diaphragms: Horizontal diaphragms shall be sized to resist all code required wind and seismic loads without exceeding the allowable loads provided herein. Diaphragm chords and connections to transfer shear forces between the diaphragm and surrounding structure shall be designed in accordance with accepted engineering practice. The maximum diaphragm length-to-width ratio shall not exceed 3:1.

4.1.11 Combined Loads: Panels subjected to any combination of transverse, axial or in-plane shear loads shall be analyzed utilizing a straight-line interaction.

4.1.12 Panel Reinforcements: Allowable transverse loads for panels reinforced with I-joists meeting the minimum properties shown in Table 4 are presented in Table 8. Panels reinforced with I-joists have not been evaluated for use in wall applications. Panels reinforced with I-joist splines may be designed in accordance with accepted engineering practice.

4.2 Installation:

4.2.1 General: Structural Insulated Panels shall be fabricated, identified and erected in accordance with this report, the approved construction documents and the applicable codes. In the event of a conflict between the manufacturer's published installation instructions and this report, this report shall govern. Approved construction documents shall be available at all times on the jobsite during installation.

4.2.2 Splines: Structural Insulated Panels are interconnected at the panel edges through the use of a spline. The spline type may be of any configuration listed in Section 3.2.5 as required by the specific design. The spline shall be secured in place with not less than 0.131-inch x $2^1/_2$-inch (2.9 mm x 63.5 mm) nails, spaced 6 inches on center on both sides of the panel, or an approved equivalent fastener. All joints shall be sealed in accordance with the SIP manufacturer's installation instructions. Alternate spline connections may be required for panels subjected to in-plane shear forces. Such panels shall be interconnected exactly as required in Tables 14 and 15 or as directed by the designer.

4.2.3 Plates: The top and bottom plates of the panels shall be dimensional or engineered lumber sized to match the core thickness of the panel. The plates shall be secured using not less than 0.131-inch x 2 1/2-inch (2.9 mm x 63.5 mm) nails, spaced 6 inches on center on both sides of the panel, or an approved equivalent fastener. A second top plate of 1 1/8-inch (29 mm) minimum thickness dimensional or engineered lumber with a specific gravity of 0.42 that is cut to the full thickness of the panel shall be secured to the first top plate using 0.131-inch x 3-inch (2.9 mm x 76 mm) nails or an approved equivalent fastener.

4.2.4 Cutting and Notching: No field cutting or routing of the panels shall be permitted except as shown on approved construction documents.

4.2.5 Protection from Decay: SIPs that rest on exterior foundation walls shall not be located within 8 inches of exposed earth. SIPs supported by concrete or masonry that is in direct contact with earth shall be protected from the concrete or masonry by a moisture barrier.

4.2.6 Protection from Termites: In areas subject to damage from termites, SIPs shall be protected from termites using an approved method. Panels shall not be installed below grade or in contact with earth.

4.2.7 Heat-Producing Fixtures: Heat-producing fixtures shall not be installed in the panels unless protected by a method approved by the code official or documented in test reports. This limitation shall not be interpreted to prohibit heat-producing elements with suitable protection.

4.2.8 Plumbing Installation Restrictions: Plumbing and waste lines may extend at right angles through the wall panels but are not permitted vertically within the core. Lines shall not interrupt splines or panel plates unless approved by a registered design professional.

4.2.9 Voids and Holes:

4.2.9.1 Voids in Core: In lieu of openings designed in accordance with Section 4.1.7, the following voids are permitted. Voids may be provided in the panel core during fabrication at predetermined locations only. Voids parallel to the panel span shall be limited to a single 1-inch-maximum-diameter (25.4 mm) hole. Such voids shall be spaced a minimum of 4 feet (1219 mm) on center measured perpendicular to the panel span. Two 1/2-inch-diameter (12.7 mm) holes may be substituted for the single 1-inch hole provided they are maintained parallel and within 2 inches of each other. Voids perpendicular to the panel span shall be limited to a single 1-inch-maximum-diameter (25.4 mm) hole placed not closer than 16 inches (406 mm) from the support. Additional voids in the same direction shall be spaced not less than 28 inches (711 mm) on center.

4.2.9.2 Holes in Panels: Holes may be placed in panels during fabrication at predetermined locations only. Holes shall be limited to 4 inches by 4 inches (102 mm by 102 mm) square. The minimum distance between holes shall not be less than 4 feet (1219 mm) on center measured perpendicular to the panel span and 24 inches (610 mm) on center measured parallel to the panel span. Not more than three holes shall be permitted in a single line parallel to the panel span. The holes may intersect voids permitted elsewhere in this report.

4.2.10 Panel Cladding:

4.2.10.1 Roof Covering: The roof covering, underlayment and flashing shall comply with the applicable codes. All roofing materials must be installed in accordance with the manufacturer's installation instructions. The use of roof coverings requiring the application of heat during installation shall be reviewed and approved by a registered design professional.

4.2.10.2 Exterior Wall Covering: Panels shall be covered on the exterior by a water-resistive barrier as required by the applicable code. The water-resistive barrier shall be attached with flashing in such a manner as to provide a continuous water-resistive barrier behind the exterior wall veneer. The exterior facing of the SIP wall shall be covered with weather protection as required by the adopted building code or other approved materials.

4.2.11 Interior Finish: The SIP foam plastic core shall be separated from the interior of the building by an approved thermal barrier of ½-inch (12.7 mm) gypsum wallboard or equivalent thermal barrier where required by IBC Section 2603.4.

5.0 CONDITIONS OF USE

The Structural Insulated Panels described in this report comply with, or are a suitable alternative to what is specified in, those codes listed in Section 1.0 of this report, subject to the following conditions:

5.1 Installation complies with this report and the approved construction documents.

5.2 This report applies only to the panel thicknesses specifically listed herein.

5.3 In-use panel heights/spans shall not exceed the values listed herein. Extrapolation beyond the values listed herein is not permitted.

5.4 The panels are manufactured at the production facility listed in Section 7.2 of this evaluation report.

6.0 EVIDENCE SUBMITTED

6.1 Reports of axial load, transverse load, and in-plane racking shear tests of panels, conducted in accordance with the general guidelines of ASTM E72.

6.2 Reports of diaphragm tests of panels, conducted in accordance with ASTM E455.

7.0 IDENTIFICATION

7.1 Structural Insulation Panels are identified with the following information:

7.1.1 The ICC-ES Evaluation Report number (ESR-4692).

7.1.2 Project or batch number

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

PORTERCORP
4240 NORTH 136th AVENUE
HOLLAND, MI 49424

TABLE 1—BASIC PROPERTIES[1]

Property	Weak-Axis Bending	Strong-Axis Bending
Allowable Tensile Stress, F_t (psi)	245	495
Allowable Compressive Stress, F_c (psi)	340	580
Elastic Modulus (Bending), E_b (psi)	738900	658800
Shear Modulus, G (psi)	270	405
Allowable Core Shear Stress, F_v (psi)	4.5	5.0
Core Compressive Modulus, E_c (psi)	360	360
Reference Depth, h_o (in.)	4.625	4.625
Shear Depth Factor Exponent, m	0.84	0.86
Face Peeling Factor, C_p	0.4	0.4

For **SI:** 1 inch = 25.4 mm; 1 psi = 6894.8 Pa.
[1] All properties are based on a minimum panel width of 24-in.

TABLE 2—SECTION PROPERTIES

Panel Thickness, *h* (in.)	Core Thickness, *c* (in.)	Dead Weight, w_d (psf)	Facing Area, A_f (in.^{2}/ft)	Shear Area, A_v (in.^{2}/ft)	Moment of Inertia, *I* (in.^{4}/ft)	Section Modulus, *S* (in.^{3}/ft)	Radius of Gyration, r (in.)	Centroid-to-Facing Dist., y_c (in.)
4.625	3.75	3.2	10.5	50.3	46.0	19.9	2.09	2.31
6.50	5.625	3.3	10.5	72.8	96.5	29.7	3.03	3.25
8.25	7.375	3.5	10.5	93.8	160.2	38.8	3.91	4.13
10.25	9.375	3.6	10.5	117.8	252.7	49.3	--	--
12.25	11.375	3.8	10.5	141.8	366.3	59.8	--	--
15	14.125	4.0	10.5	174.8	556.7	74.2	--	--

For **SI:** 1 inch = 25.4 mm; 1 foot = 304.8 mm; 1 psf = 47.88 Pa.; 1 in.^{2}/ft = 2116.66mm^2/m 1 in.^{3} = 16387.064 mm^3; 1 in.^{4}/ft = 1365588.67mm^4/m

TABLE 3—OSB FACING MINIMUM PROPERTIES

Thickness (in.)	Flatwise Stiffness (lb_f-in.^{2}/ft)		Flatwise Strength (lb_f-in./ft)		Tension (lb_f/ft)		Density (pcf)
	Along	Across	Along	Across	Along	Across	
7/16	54,700	27,100	950	870	6,800	6,500	35

For **SI:** 1 inch = 25.4 mm; 1 foot = 304.8 mm; 1 lbf = 4.448 N; 1 pcf = 0.006366 N/m^3; 1 lbf-in/ft = 370.833 N-mm/m; 1 lbf/ft = 14.59 N/m; 1 lb_f-in.^{2}/ft = 9419.167 N-mm/m

TABLE 4—MINIMUM I-JOIST PROPERTIES FOR USE AS REINFORCEMENTS[1]

Depth	Bending Stiffness EI	Moment Capacity M	Shear Capacity V	Coefficient of Shear Deflection K
(in.)	(lb_f-in.^{2}) x 10^6	(lb_f-ft)	(lb_f)	(lb_f) x 10^6
9.25	185	2715	1155	4.81
11.25	296	3410	1405	5.85
14	482	4270	1710	7.28

For **SI:** 1 inch = 25.4 mm; 1 foot = 304.8 mm; 1 lbf = 4.448 N; 1lbf-in.^{2} = 2870.962 N-mm
[1] Properties are based on certification in accordance with ASTM D5055 or equivalent.

TABLE 5—ALLOWABLE ROOF UNIFORM TRANSVERSE LOADS, BLOCKED BEARING, SHORT DURATION (PSF) [1, 4]

Panel Length (ft)	PANEL THICKNESS (inch)								
	$4^5/_8$			$6^1/_2$			$8^1/_4$		
	Deflection Limit[2]			Deflection Limit[2]			Deflection Limit[2]		
	L/180	L/240	L/360	L/180	L/240	L/360	L/180	L/240	L/360
8 WAB[3]	50	40	27	73	64	43	80	80	58
8	68	51	34	82	82	56	90	90	78
10	45	33	22	63	57	38	68	68	54
12	30	23	15	51	40	27	55	55	39
14	21	16	--	39	29	19	46	43	29
16	--	--	--	29	22	14	40	33	22
18	--	--	--	22	16	--	34	25	17
20	--	--	--	--	--	--	26	20	13
22	--	--	--	--	--	--	21	15	--
24	--	--	--	--	--	--	17	12	--

For **SI:** 1 inch = 25.4 mm; 1 foot = 304.8 mm; 1 psf = 47.88 Pa.

See Table 6 for notes.

TABLE 6—ALLOWABLE ROOF UNIFORM TRANSVERSE LOADS, BLOCKED BEARING, SHORT DURATION (PSF) [1, 4]

Panel Length (ft)	PANEL THICKNESS (inch)								
	$10^1/_4$			$12^1/_4$			15		
	Deflection Limit[2]			Deflection Limit[2]			Deflection Limit[2]		
	L/180	L/240	L/360	L/180	L/240	L/360	L/180	L/240	L/360
8 WAB[3]	88	88	75	93	96	96	108	108	108
8	98	98	98	107	107	107	121	121	121
10	73	73	73	79	79	79	87	87	87
12	59	59	54	63	63	63	68	68	68
14	49	49	41	52	52	52	56	56	56
16	42	42	31	44	44	41	47	47	47
18	37	36	24	39	39	32	41	41	41
20	32	29	19	34	34	26	36	36	36
22	29	23	15	31	31	21	33	33	29
24	25	19	12	28	26	17	29	29	24

For **SI:** 1 inch = 25.4 mm; 1 foot = 304.8 mm; 1 psf = 47.88 Pa.

[1] Table values assume a simply supported panel with $1^1/_2$ in. of continuous bearing on facing at supports (C_p = 1.0) with solid wood plates at bearing locations. Values do not include the dead weight of the panel.
[2] Deflection limit shall be selected by building designer based on the serviceability requirements of the structure and the requirements of adopted building code. Values are based on loads of short duration only and do not consider the effects of creep.
[3] Tabulated values are based on the strong-axis of the facing material oriented parallel to the direction of panel bending. WAB indicates weak-axis bending of the facing material; the strong-axis of the facing material is oriented perpendicular to the direction of panel bending.
[4] Permanent loads, such as dead load, shall not exceed 0.50 times the tabulated load.

TABLE 7—ALLOWABLE WALL UNIFORM TRANSVERSE LOADS (PSF) [1, 4]

Panel Length (ft)	PANEL THICKNESS (inch)								
	$4^5/_8$			$6^1/_2$			$8^1/_4$		
	Deflection Limit[2]			Deflection Limit[2]			Deflection Limit[2]		
	L/180	L/240	L/360	L/180	L/240	L/360	L/180	L/240	L/360
8 WAB[3]	22	22	22	24	24	24	25	25	25
8	25	25	25	27	27	27	28	28	28
10	20	20	20	21	21	21	23	23	23
12	16	16	15	18	18	18	19	19	19
14	14	14	--	15	15	15	16	16	16
16	--	--	--	13	13	13	14	14	14
18	--	--	--	12	12	11	12	12	12
20	--	--	--	--	--	--	11	11	11

For **SI:** 1 inch = 25.4 mm; 1 foot = 304.8 mm; 1 psf = 47.88 Pa.

[1] Table values represent wall panel capacities ($4^5/_8$-in., $6^1/_2$-in. and $8^1/_4$-in. thickness panels only) utilizing a zero bearing configuration (Figure 2). Allowable loads are determined based on C_p reported in Table 1.
[2] Deflection limit shall be selected by building designer based on the serviceability requirements of the structure and the requirements of adopted building code. Values are based on loads of short duration only and do not consider the effects of creep.
[3] Tabulated values are based on the strong-axis of the facing material oriented parallel to the direction of panel bending. WAB indicates weak-axis bending of the facing material; the strong-axis of the facing material is oriented perpendicular to the direction of panel bending.
[4] Permanent loads, such as dead load, shall not exceed 0.50 times the tabulated load.

TABLE 8—ALLOWABLE UNIFORM TRANSVERSE LOADS WITH I-JOIST REINFORCEMENTS (PSF) [1, 3, 4]

Panel Length (ft)	PANEL THICKNESS (inch)								
	$10^1/_4$-in. SIP thickness			$12^1/_4$-in. SIP thickness			15-in. SIP thickness		
	Deflection Limit[2]			Deflection Limit[2]			Deflection Limit[2]		
	L/180	L/240	L/360	L/180	L/240	L/360	L/180	L/240	L/360
8	115	115	115	124	124	124	123	123	123
10	92	92	92	99	99	99	98	98	98
12	76	76	76	82	82	82	82	82	82
14	65	65	65	71	71	71	70	70	70
16	57	57	57	62	62	62	61	61	61
18	51	51	44	55	55	55	54	54	54
20	46	46	33	49	49	48	48	48	48
22	41	38	25	45	45	37	44	44	44
24	36	30	20	41	41	29	41	41	41

For **SI:** 1 inch = 25.4 mm; 1 foot = 304.8 mm; 1 psf = 47.88 Pa.

[1] Values assume a simply supported panel with $1^1/_2$ in. of continuous bearing on facing at supports. Values do not include the dead weight of the panel.
[2] Deflection limit shall be selected by building designer based on the serviceability requirements of the structure and the requirements of adopted building code. Values are based on loads of short duration only and do not consider the effects of creep.
[3] Tabulated values are based on the strong-axis of the facing material oriented parallel to the direction of panel bending.
[4] Permanent loads, such as dead load, shall not exceed 0.50 times the tabulated load.

TABLE 9—ALLOWABLE AXIAL LOADS (PLF) [1,2,3,4]

Lateral Brace Spacing (ft)	PANEL THICKNESS (inch)		
	$4^5/_8$	$6^1/_2$	$8^1/_4$
8 WAB[5]	2320	2470	2530
8	3630	4070	4240
10	3260	3890	4130
12	2810	3660	4000
14	--	3390	3830
16	--	3090	3640
18	--	2790	3430
20	--	--	3190

For **SI:** 1 inch = 25.4 mm; 1 foot = 304.8 mm; 1 PLF = 14.59 N/m.

[1] Permanent loads, such as dead load, shall not exceed 0.50 times the tabulated load.
[2] All values are for normal duration and may not be increased for other durations.
[3] Axial loads shall be applied concentrically to the top of the panel through repetitive members spaced not more than 24-in. on center. Such members shall be fastened to a rim board or similar member to distribute along the top of the SIP.
[4] The ends of both facings must bear on the supporting foundation or structure to achieve the tabulated axial loads.
[5] Tabulated values are based on the strong-axis of the facing material oriented parallel to the direction of panel bending. WAB indicates weak-axis bending of the facing material; the strong-axis of the facing material is oriented perpendicular to the direction of panel bending.

TABLE 10—ALLOWABLE UNIFORM TRANSVERSE LOADS FOR SIPS WITH OPENINGS, 36-IN. MAXIMUM SPAN (PSF) [1,4,5,6]

Panel Length (ft)	PANEL THICKNESS (inch)								
	$4^5/_8$			$6^1/_2$			$8^1/_4$		
	Deflection Limit[2]			Deflection Limit[2]			Deflection Limit[2]		
	L/180	L/240	L/360	L/180	L/240	L/360	L/180	L/240	L/360
8 WAB[3]	23	17	11	42	31	21	62	47	31
8	31	23	15	57	43	28	75	65	43
10	17	13	8	33	25	16	48	39	26
12	10	8	5	21	16	10	33	25	16
14	7	5	--	14	10	7	22	16	11
16	--	--	--	9	7	--	15	11	7
18	--	--	--	7	5	--	11	8	5
20	--	--	--	--	--	--	8	6	--

For **SI:** 1 inch = 25.4 mm; 1 foot = 304.8 mm; 1 psf = 47.88 Pa.

See Table 11 for notes.

TABLE 11—ALLOWABLE UNIFORM TRANSVERSE LOADS FOR SIPS WITH OPENINGS, 72-INCH MAXIMUM SPAN (PSF) [1,4,5,6]

Panel Length (ft)	PANEL THICKNESS (inch)								
	4 5/8			6 1/2			8 1/4		
	Deflection Limit[2]			Deflection Limit[2]			Deflection Limit[2]		
	L/180	L/240	L/360	L/180	L/240	L/360	L/180	L/240	L/360
8 WAB[3]	16	12	8	29	23	15	39	36	24
8	23	17	11	37	33	22	49	49	34
10	12	9	6	24	19	12	31	29	19
12	7	5	--	15	11	7	21	18	12
14	5	--	--	10	7	5	16	12	8
16	--	--	--	7	5	--	11	8	5
18	--	--	--	5	--	--	8	6	--
20	--	--	--	--	--	--	6	--	--

For **SI:** 1 inch = 25.4 mm; 1 foot = 304.8 mm; 1 psf = 47.88 Pa.

[1] Table values represent wall panel capacities utilizing a zero bearing configuration (Figure 2). Construction shall be as described in Section 4.1.7 of this report.
[2] Deflection limit shall be selected by building designer based on the serviceability requirements of the structure and the requirements of adopted building code. Values are based on loads of short duration only and do not consider the effects of creep.
[3] Tabulated values are based on the strong-axis of the facing material oriented parallel to the direction of panel bending. WAB indicates weak-axis bending of the facing material; the strong-axis of the facing material is oriented perpendicular to the direction of panel bending.
[4] Permanent loads, such as dead load, shall not exceed 0.50 times the tabulated load.
[5] Tabulated values assume header depths ranging from 12-in. to 36-in.
[6] SIP splines are not permitted within 6-in. of the end of the header and are not permitted within the header.

TABLE 12—ALLOWABLE AXIAL LOADS FOR SIPS WITH OPENINGS, 36-IN. MAXIMUM SPAN (PLF) [1,2,3,4,6,7]

Lateral Brace Spacing (ft)	Panel Thickness (inch)		
	4 5/8	6 1/2	8 1/4
8 WAB[5]	770	820	840
8	1210	1355	1410
10	1085	1295	1375
12	935	1220	1330
14	--	1130	1275
16	--	1030	1210
18	--	930	1140
20	--	--	1060

For **SI:** 1 inch = 25.4 mm; 1 foot = 304.8 mm; ; 1 plf = 14.59 N/m.

See Table 13 for notes.

TABLE 13—ALLOWABLE AXIAL LOADS FOR SIPS WITH OPENINGS, 72-IN. MAXIMUM SPAN (PLF) [1,2,3,4,6,7]

Lateral Brace Spacing (ft)	Panel Thickness (inch)		
	$4^{5}/_{8}$	$6^{1}/_{2}$	$8^{1}/_{4}$
8 WAB[5]	460	490	505
8	725	810	845
10	650	775	825
12	560	730	800
14	--	675	765
16	--	615	725
18	--	555	685
20	--	--	635

For **SI:** 1 inch = 25.4 mm; 1 foot = 304.8 mm; 1 plf = 14.59 N/m.

[1] Permanent loads, such as dead load, shall not exceed 0.50 times the tabulated load.
[2] All values are for normal duration and may not be increased for other durations.
[3] Axial loads shall be applied concentrically to the top of the panel through repetitive members spaced not more than 24-in. on center. Such members shall be fastened to a rim board or similar member to distribute along the top of the SIP.
[4] The ends of both facings must bear on the supporting foundation or structure to achieve the tabulated axial loads.
[5] Tabulated values are based on the strong-axis of the facing material oriented parallel to the direction of panel bending. WAB indicates weak-axis bending of the facing material; the strong-axis of the facing material is oriented perpendicular to the direction of panel bending.
[6] Tabulated values assume header depths ranging from 12-in. to 36-in.
[7] SIP splines are not permitted within 6-in. of the end of the header and are not permitted within the header.

TABLE 14—ALLOWABLE IN-PLANE SHEAR STRENGTH (POUNDS PER FOOT) FOR SIP SHEAR WALLS (WIND AND SEISMIC LOADS IN SEISMIC DESIGN CATEGORIES A, B AND C) [1, 2]

Spline Type[3]	Minimum Nominal SIP Thickness (in.)	Minimum Facing Connections[2,4]			Shear Strength(plf)
		Chord[2]	Plate[2]	Spline[3]	
Block or Surface Spline	$4^{5}/_{8}$	0.131-in. x $2^{1}/_{2}$-in. nails, 6-in. on center	0.131-in. x $2^{1}/_{2}$-in. nails, 6-in. on center	0.131-in. x $2^{1}/_{2}$-in. nails, 6-in. on center	380
	$8^{1}/_{4}$	0.131-in. x $2^{1}/_{2}$-in. nails, 6-in. on center	0.131-in. x $2^{1}/_{2}$-in. nails, 6-in. on center	0.131-in. x $2^{1}/_{2}$-in. nails, 6-in. on center	400

For **SI:** 1 inch = 25.4 mm; 1 foot = 304.8 mm; 1 psf = 47.88 Pa.; 1 plf = 14.59 N/m.

[1] Maximum shear wall dimensions ratio shall not exceed 2:1 (height: width) for resisting wind or seismic loads.
[2] Chords, hold downs and connections to other structural elements must be designed by a registered design professional in accordance with accepted engineering practice.
[3] Spline type at interior panel-to-panel joints only. Solid chord members are required at each end of each shear wall segment.
[4] Required connections must be made on each side of the panel. Dimensional or engineered lumber shall have an equivalent specific gravity of 0.42 or greater.

TABLE 15—ALLOWABLE IN-PLANE SHEAR STRENGTH FOR HORIZONTAL DIAPHRAGMS SUBJECTED TO WIND OR SEISMIC LOADING

Minimum Nominal SIP Thickness (in.)	Minimum Connections			Shear Strength (plf)	Max. Aspect Ratio
	Surface Spline[1] (Figure 3b)	Boundary Support Element (Figure 3c)	Interior Support Spline[2,3] (Figure 3a)		
8-1/4	0.131-in. x 2$^1/_2$-in. nails, 6-in. on center $^7/_{16}$-in. x 3-in. OSB Surface Spline	10-in. length, 0.190-in. shank diameter, 0.255-in. thread o.d., 2.750-in. thread length, 0.625-in. head diameter SIP screw, 6-in. on center	0.131-in. x 2$^1/_2$-in. nails, 6-in. on center	265	3:1
	0.131-in. x 2$^1/_2$-in. nails, 4-in. on center $^7/_{16}$-in. x 3-in. OSB Surface Spline	10-in. length, 0.190-in. shank diameter, 0.255-in. thread o.d., 2.750-in. thread length, 0.625-in. head diameter SIP screw, 4-in. on center	0.131-in. x 2$^1/_2$-in. nails, 4-in. on center	330	3:1
	0.131-in. x 2$^1/_2$-in. nails, 2-in. on center, two rows staggered $^3/_8$-in. $^7/_{16}$-in. x 3-in. OSB Surface Spline	10-in. length, 0.190-in. shank diameter, 0.255-in. thread o.d., 2.750-in. thread length, 0.625-in. head diameter SIP screw, 3-in. on center	0.131-in. x 2$^1/_2$-in. nails, 2-in. on center, two rows staggered $^3/_8$-in.	575	3:1

For **SI:** 1 inch = 25.4 mm, 1 PLF = 14.59 N/m

[1]Surface or block spline only at interior panel-to-panel joints. Specified fasteners are required on both sides of panel joint through the top surface only, as shown in Figure 3b.
[2]Interior support spline shall be solid lumber 1$^1/_2$-inch-wide minimum and have a specific gravity of 0.42 or greater. Specified fasteners are required through both facings as shown in Figure 3c.
[3]Attachment of panels to interior supports is the responsibility of the designer and are not included with the shear strength capacities in this table.

TABLE 16—COMPONENT MATERIAL SOURCES

Facing	Core	Adhesive
Louisiana-Pacific Corporation Sagola, MI Distributed by: Viking Forest Products, LLC 7615 Smetana Lane Eden Prairie, MN 55344	Atlas Molded Products, A Division of Atlas Roofing Corporation 8240 Byron Center Road SW Byron Center, MI 49315	Ashland, LLC 5475 Rings Road Dublin, OH 43017
Norbord, Inc. 1 Toronto Street, Suite 600 Toronto ON, Canada M5C 2W4	Benchmark Foam, Inc. 401 Pheasant Ridge Drive Watertown, SD 57201	DuPont Specialty Products 200 Larkin Center 1501 Larkin Center Drive Midland, MI 48674
Tolko Industries, Ltd. 3203 30th Avenue Vernon BC, Canada V1T 6M1	Carpenter Foam 1021 E Springfield Road High Point, NC 27263	
	Creative Packaging Company 6301 Midland Industrial Drive Shelbyville, KY 40065	
	Insulfoam, a Carlisle Company 1507 Sunburst Lane Mead, NE 68041 (I-41)	
	Iowa EPS Products, Inc. 5554 N.E. 16th Street Des Moines, IA 50313	
	OPCO, Inc. P.O. Box 101 Latrobe, PA 15650	
	Plymouth Foam 1 Southern Gateway Drive Gnadenhutten, OH 44629	
	Polar Industries, Inc. 32 Gramar Avenue Prospect, CT 06712	
	Powerfoam Insulation Division of Metl-Span LTD. 550 Murray Street, Highway 287 Midlothian, TX 76065	
	Thermal Foams, Inc. 2101 Kenmore Avenue Buffalo, NY 14207	

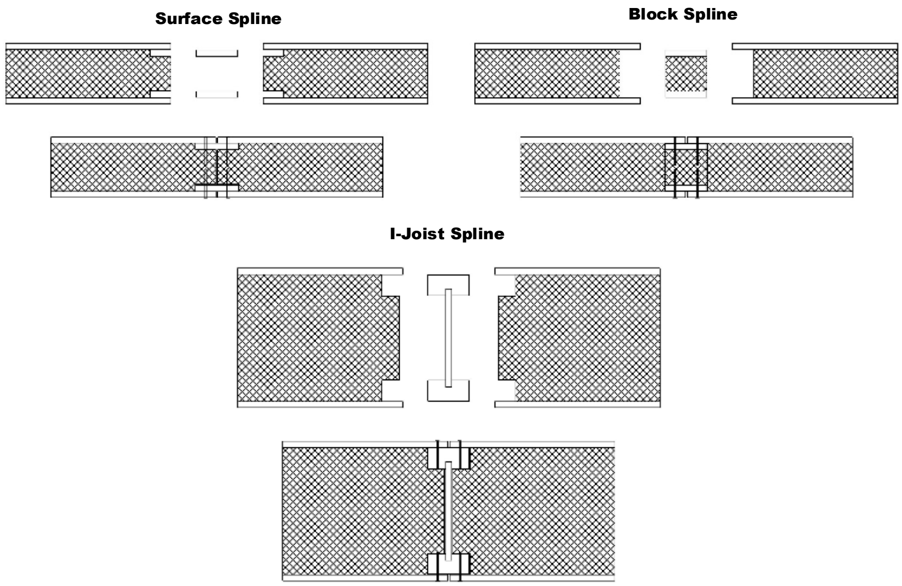


FIGURE 1—SIP SPLINE TYPES

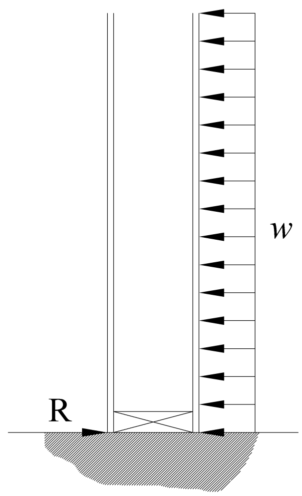


FIGURE 2—ZERO BEARING SUPPORT

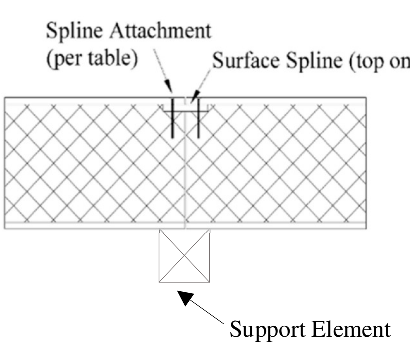


FIGURE 3A—INTERIOR SUPPORT SPLINE

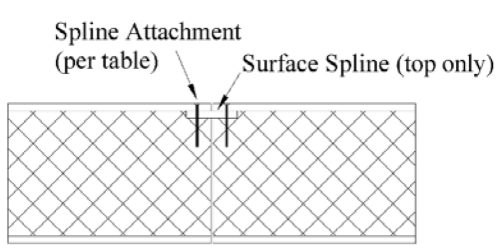


FIGURE 3B—SURFACE SPLINE

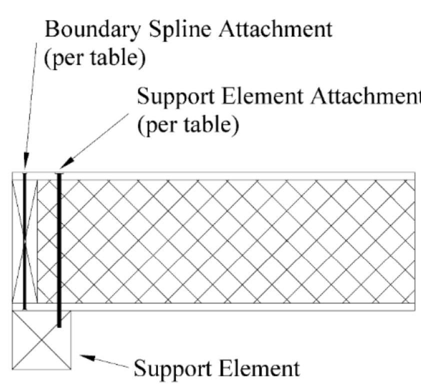


FIGURE 3C—BOUNDARY SUPPORT ELEMENT

SIP Fasteners

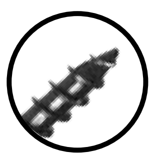
For Structural Insulated Panel and Nail Base Construction

APPLICATION

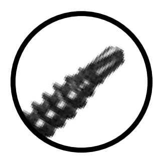
TRUFAST SIP Fasteners are specifically engineered for attaching structural insulated panels (sips) and nail base panels to wood and metal framing. Featuring a large, pancake head style with a 6-lobe drive, TRUFAST SIP Fasteners drive quickly and smoothly, and draw panels securely without the need of a washer. And only TRUFAST offers three fastener styles for use in wood, corrugated steel, and steel members without pre-drilling! Contact your panel manufacturer or distributor and ask to test drive a TRUFAST SIP Fastener, and see why they're the #1 fastener in the SIP industry.

PRODUCT FEATURES

- Case hardened and tempered for easy installation and long term durability.
- Large diameter, low profile pancake head provides excellent pull-through resistance without the need for a washer while eliminating "telegraphing" on shingles, metal panels and other roof surface materials.
- 6-Lobe internal drive offers excellent bit engagement during installation, especially in high torque applications.
- Widest selection of fastener lengths in the industry for proper sizing to panel thickness.
- Choice of 3 thread/point styles for job-matched performance in either wood or steel substrates.



SIPTP
Thread Point for Wood & Timber Applications



SIPLD
Light Duty - Drill Point for Corrugated Steel Deck & Wood Applications

SIPHD
Heavy Duty - Drill Point for Thick Steel Member Applications

PRODUCT SPECIFICATIONS

Material:	Case hardened and tempered carbon steel
Head Style/Drive:	Pancake Head with T-30 Internal Drive
Head Diameter:	0.625"
Nominal Shank Diameter:	SIPTP and SIPLD: 0.190" SIPHD: 0.212"
Thread Length:	SIPTP* and SIPLD: 2.750" SIPHD: 3.875" ** 3" and longer fasteners; 2" and 2-1/2" fasteners are full thread*
Overall Lengths:	SIPTP: 2" thru 18" SIPLD: 3" thru 18" SIPHD: 6" thru 13-3/4"
Point:	SIPTP: Gimlet Thread SIPLD: #2 (0.135" dia.) Drill Point SIPHD: #4 (0.225" dia.) Drill Point
Coating:	Epoxy e-coat (black) *Passes more than 15 cycles (Kesternich) in accordance with DIN 50012*

SIP Fasteners

For Structural Insulated Panel and Nail Base Construction

PRODUCT SELECTION

Length in.	(mm)	SIPTP Part #	SIPLD Part #	Pkg. Qty.
2	(51)	**SIPTP-2000**	NA	500/Pail
2-1/2	(64)	**SIPTP-2500**	NA	500/Pail
3	(76)	**SIPTP-3000**	**SIPLD-3000**	500/Pail
3-1/2	(89)	**SIPTP-3500**	**SIPLD-3500**	500/Pail
4	(102)	**SIPTP-4000**	**SIPLD-4000**	500/Pail
4-1/2	(114)	**SIPTP-4500**	**SIPLD-4500**	500/Pail
5	(127)	**SIPTP-5000**	**SIPLD-5000**	500/Pail
5-1/2	(140)	**SIPTP-5500**	**SIPLD-5500**	500/Pail
6	(152)	**SIPTP-6000**	**SIPLD-6000**	500/Pail
6-1/2	(165)	**SIPTP-6500**	**SIPLD-6500**	500/Pail
7	(178)	**SIPTP-7000**	**SIPLD-7000**	500/Pail
7-1/2	(191)	**SIPTP-7500**	**SIPLD-7500**	500/Pail
8	(203)	**SIPTP-8000**	**SIPLD-8000**	500/Pail
8-1/2	(216)	NA	**SIPLD-8500**	250/Pail
9	(229)	**SIPTP-9000**	**SIPLD-9000**	250/Pail
10	(254)	**SIPTP-10000**	**SIPLD-10000**	250/Pail
11	(279)	**SIPTP-11000**	**SIPLD-11000**	250/Pail
12	(305)	**SIPTP-12000**	**SIPLD-12000**	250/Pail
13	(330)	**SIPTP-13000**	**SIPLD-13000**	250/Box
14	(356)	**SIPTP-14000**	**SIPLD-14000**	250/Box
15	(381)	**SIPTP-15000**	**SIPLD-15000**	250/Box
16	(406)	**SIPTP-16000**	**SIPLD-16000**	250/Box
18	(483)	**SIPTP-18000**	**SIPLD-18000**	250/Box

NOTE: Two T-30 Driver Bits included in each package

Length in.	(mm)	SIPHD Part #	Pkg. Qty.
6	(152)	**SIPHD-6000**	500/Pail
8	(203)	**SIPHD-8000**	250/Pail
9-3/4	(248)	**SIPHD-9750**	250/Pail
11-3/4	(298)	**SIPHD-11750**	250/Pail
13-3/4	(349)	**SIPHD-13750**	250/Box

NOTE: Two T-30 Driver Bits included in each package

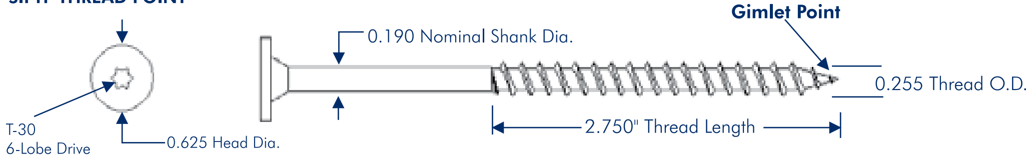
T-30 Driver Bit

FM APPROVED

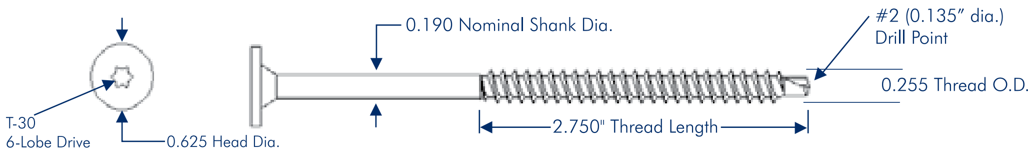
NOTE: All tests were conducted by an independent testing laboratory. Test results are offered only as a guide and are not guaranteed in any way by TRUFAST, LLC.
"Head Pull-Thru", "Withdrawal", and "Lateral Load" data reflect average ultimate values.

FASTENER DIMENSIONS

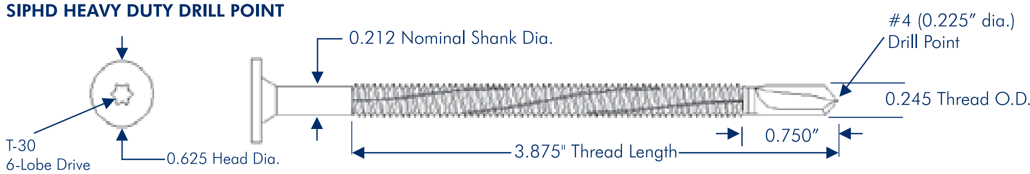
SIPTP THREAD POINT



SIPLD LIGHT DUTY DRILL POINT



SIPHD HEAVY DUTY DRILL POINT



PERFORMANCE DATA

Fastener	Tensile Strength	Shear Strength	Head Pull-Thru Values 7/16" OSB	SIP Panel
SIPTP	3380 lbf.	2900 lbf.	545 lbf.	630 lbf.
SIPLD	3380 lbf.	2900 lbf.	545 lbf.	630 lbf.
SIPHD	6000 lbf.	3400 lbf.	545 lbf.	630 lbf.

Withdrawal Values in Wood*

Specific Gravity	0.67	0.55	0.50	0.46	0.43	0.36	0.31
SIPTP & SIPLD:	1429	1173	1067	981	917	768	661

*Values are in lb/in. of thread penetration

Withdrawal Values in Steel

Type B Corrugated	22 ga	20 ga	18 ga		
SIPLD:	510 lbf	645 lbf	920 lbf		
Structural Steel	16 ga	13 ga	12 ga	3/16"	1/4"
SIPHD:	770 lbf	1130 lbf	1690 lbf	3100 lbf	4500 lbf

Lateral Load Resistance

Fastener	Main Member	Side Member	Load (lbf.)
SIPTP	SPF 2x4	SIP Panel	943
SIPLD	22 ga. Corrugated Steel	Nail Base	411
SIPLD	7/16" OSB	Nail Base	112
SIPHD	1/8" Structural Steel	SIP Panel	929

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