PACKARD ENGINEERING
21021 Oak Drive
Belton, MO 64012
Phone: (816) 767-7222
Fax: (816) 767-8105
E-mail: dan@packardstructuralengineering.com

CLIENT	BHA Architecture	Page	T.O.C.
		Date	7/20
JOB	Osage Clubhouse 2025 SW M150 Hwy Lee's Summit, MO	Job No.	BHA20001
		Made By	DJP

Calculations Table of Contents (Structural)

STATE OF MISSOURI
DANIEL J. PACKARD
NUMBER
E-24486
PROFESSIONAL ENGINEER

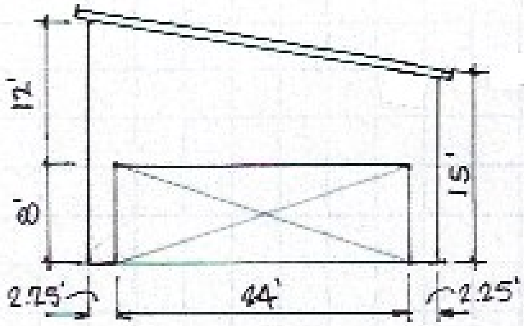
D J Packard
07-31-20

PACKARD ENGINEERING
21021 Oak Drive
Belton, MO 64012
Phone: (816) 767-7222
Fax: (816) 767-8105
E-mail: dan@packardstructural engineering.com

CLIENT B+A Architecture — Page L1
Date 03/20
JOB Osage Clubhouse — Job No. BtA20001
150 Highway & Pryor Rd.
Lee's Summit Mo — Made By DJP

Lateral

- See following pages for Lateral (Wind Controls) loads @ MWFRS & Components/Cladding
- Adequate Shearwalls exist - but check at N&W walls of Great Room (& Patio Roof - Open End)
- North Wall of Great Room: Drag shear to North Wall of Pool Equip't room w/ Roof Framing
  ← 11' long →



$$F_W = \underbrace{\left(24\,psf\,(7.5'+5')17'\right)}_{5100\#} \frac{4.5'}{(4.5'+14')} = 1120\#$$

$5100\# - 1120\# = 3980\#$ Drag Shear $v = \frac{3980\#}{10.67'} = 373$ plf ← 8d @ 4" o.c.

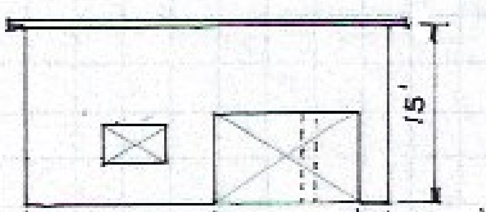
$$M_o = 1120\#\left(\frac{12.5'}{2} + 7.5'\right) = 15396\#'$$

$$T = C = \frac{15396\#'}{26.25'} = 587\#, \quad V = \frac{1120\#}{2} = 560\#$$

$$v = \frac{1120\#}{2(2.25')} = 249\text{ plf} \leftarrow 8d @ 6" o.c., typ, UNO$$

$$M_{pillar} = 560\#(8') = 4480\#'$$

- East Wall of Great Room:



$$F_W = 24\,psf\left(3.5'(6.25') + (1.67'+7.5')14.33'\right) = 3677\#$$

$$M_o = 3677\#(12') = 44124\#'$$

$$T = C = \frac{44124\#'}{15'} = 2942\#$$

$$v = \frac{3677\#}{15.33'} = 240\text{ plf}$$

WIND LOADING ANALYSIS - Main Wind-Force Resisting System

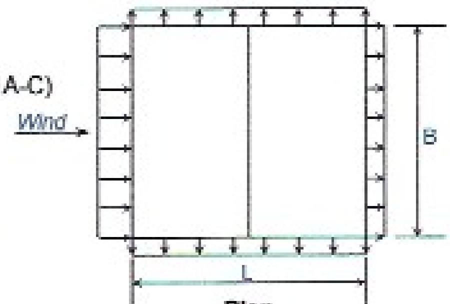
Per ASCE 7-10 Code for Enclosed or Partially Enclosed Buildings

Using Method 2: Analytical Procedure (Section 27 & 28) for Low-Rise Buildings

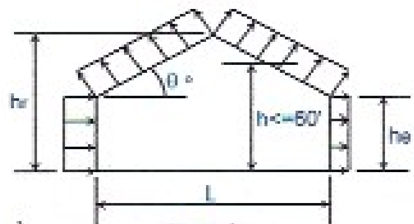
Job Name:	Clubhouse	Subject:	Main Roof - Great Room
Job Number:		Originator:	Checker:

Input Data:

Wind Speed, V =	110	mph (Wind Map, Figure 26.5-1A-C)
Bldg. Classification =	II	(Table 1.5-1 Risk Category)
Exposure Category =	C	(Sect. 26.7)
Ridge Height, hr =	21.00	ft. (hr >= he)
Eave Height, he =	15.00	ft. (he <= hr)
Building Width =	28.67	ft. (Normal to Building Ridge)
Building Length =	30.00	ft. (Parallel to Building Ridge)
Roof Type =	Monoslope	(Gable or Monoslope)
Topo. Factor, Kzt =	1.00	(Sect. 26.8 & Figure 26.8-1)
Direct. Factor, Kd =	0.85	(Table 26.6)
Enclosed? (Y/N)	Y	(Sect. 26.2 & Table 26.11-1)
Hurricane Region?	N	



Plan



Elevation

Resulting Parameters and Coefficients:

Roof Angle, θ =	11.82	deg.
Mean Roof Ht., h =	18.00	ft. (h = (hr+he)/2, for angle >10 deg.)

Check Criteria for a Low-Rise Building:

1. Is h <= 60' ? Yes, O.K.   2. Is h <= Lesser of L or B? Yes, O.K.

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

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

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

α =	9.50	(Table 26.9-1)
zg =	900	(Table 26.9-1)
Kh =	0.88	(Kh = Kz evaluated at z = h)

Velocity Pressure: qz = 0.00256*Kz*Kzt*Kd*V^2 (Sect. 28.3.2, Eq. 28.3-1)

qh = 23.23 psf   qh = 0.00256*Kh*Kzt*Kd*V^2 (qz evaluated at z = h)

Design Net External Wind Pressures (Sect. 28.4.1):
p = qh*[(GCpf) - (+/-GCpi)] (psf, Eq. 28.4-1)

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

a =	3.00	ft.
2*a =	6.00	ft.

MWFRS Wind Load for Load Case A				MWFRS Wind Load for Load Case B			
Surface	GCpf	p = Net Pressures (psf)		Surface	*GCpf	p = Net Pressures (psf)	
		(w/ +GCpi)	(w/ -GCpi)			(w/ +GCpi)	(w/ -GCpi)
Zone 1	0.46	6.48	14.84	Zone 1	-0.45	-14.63	-6.27
Zone 2	-0.69	-20.21	-11.84	Zone 2	-0.69	-20.21	-11.84
Zone 3	-0.42	-13.94	-5.57	Zone 3	-0.37	-12.77	-4.41
Zone 4	-0.35	-12.39	-4.03	Zone 4	-0.45	-14.63	-6.27
Zone 5	---	---	---	Zone 5	0.40	5.11	13.47
Zone 6	---	---	---	Zone 6	-0.29	-10.92	-2.55
Zone 1E	0.70	11.99	20.35	Zone 1E	-0.48	-15.33	-6.97
Zone 2E	-1.07	-29.03	-20.67	Zone 2E	-1.07	-29.03	-20.67
Zone 3E	-0.60	-18.18	-9.82	Zone 3E	-0.53	-16.49	-8.13
Zone 4E	-0.53	-16.39	-8.02	Zone 4E	-0.48	-15.33	-6.97
Zone 5E	---	---	---	Zone 5E	0.61	9.99	18.35
Zone 6E	---	---	---	Zone 6E	-0.43	-14.17	-5.81

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

For Case A when GCpf is neg. in Zones 2/2E: Zones 2/2E dist. = 14.34 ft.

For Case B when GCpf is neg. in Zones 2/2E: Zones 2/2E dist. = 15.00 ft.

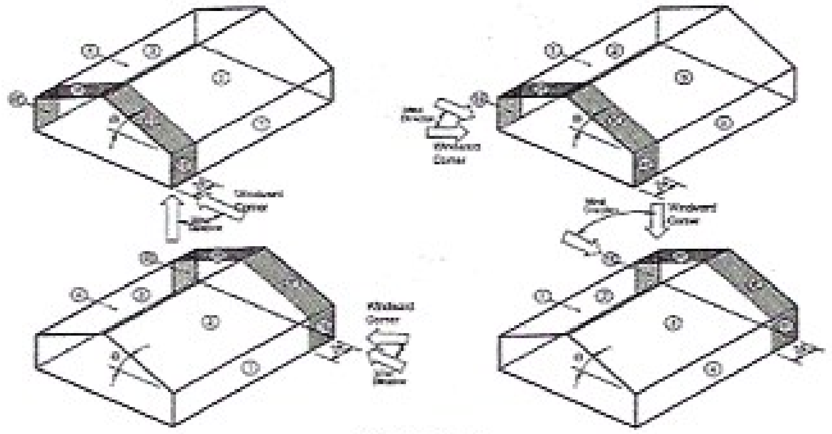
Remainder of roof Zones 2/2E extending to ridge line shall use roof Zones 3/3E pressure coefficients.

MWFRS Wind Load for Load Case A, Torsional Case				MWFRS Wind Load for Case B, Torsional Case			
Surface	GCpf	p = Net Pressure (psf)		Surface	GCpf	p = Net Pressure (psf)	
		(w/ +GCpi)	(w/ -GCpi)			(w/ +GCpi)	(w/ -GCpi)
Zone 1T	---	1.62	3.71	Zone 1T	---	-3.66	-1.57
Zone 2T	---	-5.05	-2.96	Zone 2T	---	-5.05	-2.96
Zone 3T	---	-3.48	-1.39	Zone 3T	---	-3.19	-1.10
Zone 4T	---	-3.10	-1.01	Zone 4T	---	-3.66	-1.57
Zone 5T	---	---	---	Zone 5T	---	1.28	3.37
Zone 6T	---	---	---	Zone 6T	---	-2.73	-0.64

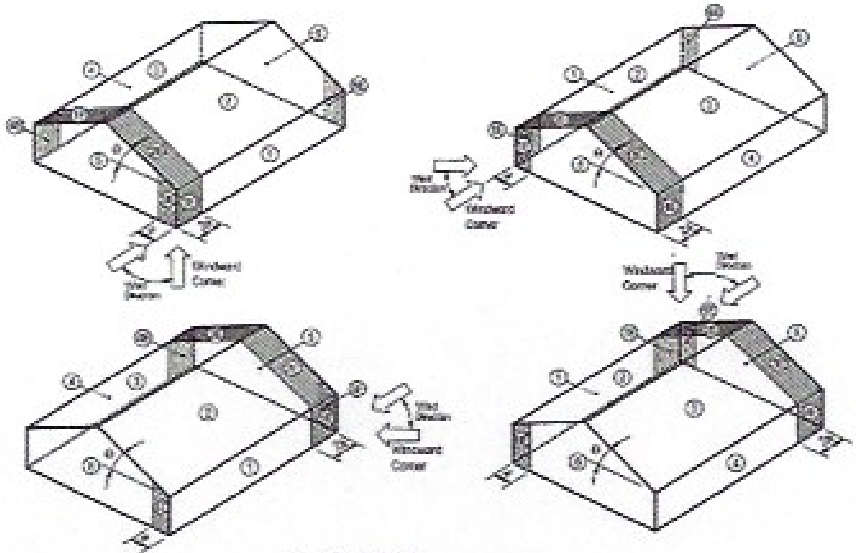
Notes: 1. For Load Case A (Transverse), Load Case B (Longitudinal), and Torsional Cases:
- Zone 1 is windward wall for interior zone.
- Zone 2 is windward roof for interior zone.
- Zone 3 is leeward roof for interior zone.
- Zone 4 is leeward wall for interior zone.
- Zones 5 and 6 are sidewalls.
- Zone 1T is windward wall for torsional case
- Zone 3T is leeward roof for torsional case
- Zones 5T and 6T are sidewalls for torsional case.
- Zone 1E is windward wall for end zone.
- Zone 2E is windward roof for end zone.
- Zone 3E is leeward roof for end zone.
- Zone 4E is leeward wall for end zone.
- Zone 5E & 6E is sidewalls for end zone.
- Zone 2T is windward roof for torsional case.
- Zone 4T is leeward wall for torsional case.

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

Low-Rise Buildings h <= 60'

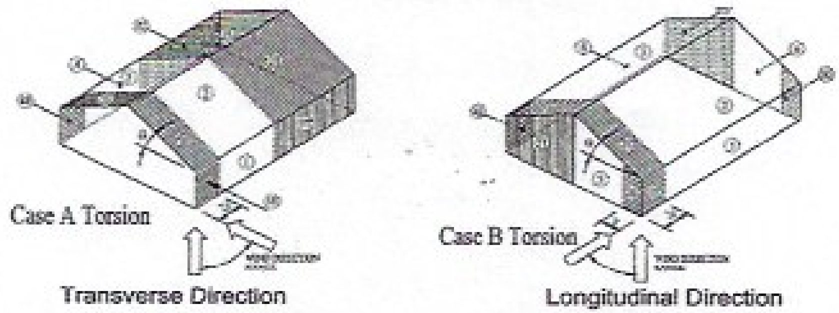


Load Case A



Load Case B

Basic Load Cases



Torsional Load Cases

Page Li.b

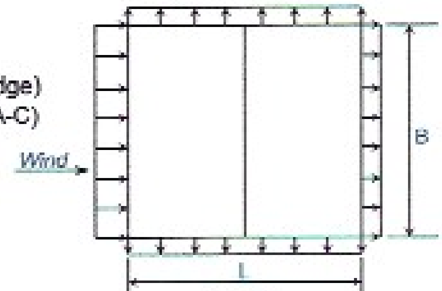
WIND LOADING ANALYSIS - Main Wind-Force Resisting System

Per ASCE 7-10 Code for Enclosed or Partially Enclosed Buildings
Using Method 2: Analytical Procedure (Section 27) for Buildings of Any Height

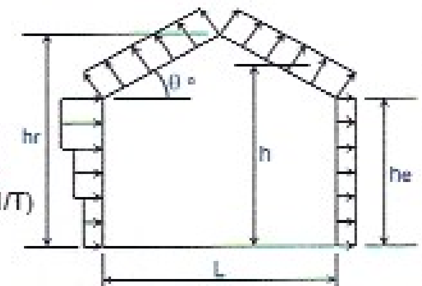
Job Name:	Clubhouse	Subject:	Patio Roof		
Job Number:		Originator:		Checker:	

Input Data:

Wind Direction = Parallel (Normal or Parallel to building ridge)
Wind Speed, V = 110 mph (Wind Map, Figure 26.5-1A-C)
Bldg. Classification = II (Table 1.4-1 Risk Cat.)
Exposure Category = C (Sect. 26.7)
Ridge Height, hr = 13.00 ft. (hr >= he)
Eave Height, he = 9.50 ft. (he <= hr)
Building Width = 16.00 ft. (Normal to Building Ridge)
Building Length = 30.00 ft. (Parallel to Building Ridge)
Roof Type = Monoslope (Gable or Monoslope)
Topo. Factor, Kzt = 1.00 (Sect. 26.8 & Table 26.8-1)
Direct. Factor, Kd = 0.85 (Table 26.6-1)
Enclosed? (Y/N) N (Sect. 6.2 & Figure 6-5)
Hurricane Region? N
Damping Ratio, β = 0.050 (Suggested Range = 0.010-0.070)
Period Coef., Ct = 0.0350 (Suggested Range = 0.020-0.035)
(Assume: T = Ct*h^(3/4) , and f = 1/T)



Plan



Elevation

L = 30 ft.
B = 16 ft.

Resulting Parameters and Coefficients:

Roof Angle, θ = 12.34 deg.
Mean Roof Ht., h = 11.25 ft. (h = (hr+he)/2, for roof angle >10 deg.)
Windward Wall Cp = 0.80 (Fig. 27.4-1)
Leeward Wall Cp = -0.33 (Fig. 27.4-1)
Side Walls Cp = -0.70 (Fig. 27-4.1)

Roof Cp (zone #1) =	-0.90	-0.18	(Fig. 27-4.1)	(zone #1 for 0 to h/2)
Roof Cp (zone #2) =	-0.90	-0.18	(Fig. 27-4.1)	(zone #2 for h/2 to h)
Roof Cp (zone #3) =	-0.50	-0.18	(Fig. 27-4.1)	(zone #3 for h to 2*h)
Roof Cp (zone #4) =	-0.30	-0.18	(Fig. 27-4.1)	(zone #4 for > 2*h)

+GCpi Coef. = 0.55 (Table 26.11- (positive internal pressure)
-GCpi Coef. = -0.55 (Table 26.11- (negative internal pressure)

If z <= 15 then: Kz = 2.01*(15/zg)^(2/α) , If z > 15 then: Kz = 2.01*(z/zg)^(2/α) (Table 27.3-1)
α = 9.50 zg = 900 (Table 26.9-1)
Kh = 0.85 (Kh = Kz evaluated at z = h)

Velocity Pressure: qz = 0.00256*Kz*Kzt*Kd*V^2*I (Eq. 27.3-1)
qh = 22.35 psf qh = 0.00256*Kh*Kzt*Kd*V^2 (qz evaluated at z = h)
Ratio h/L = 0.375 freq., f = 4.651 hz. (f >= 1, Rigid structure)
Gust Factor, G = 0.850 (Sect. 26.9)
Design Net External Wind Pressures (Sect. 27.4):
p = qz*G*Cp - qi*(+/-GCpi) for windward wall (psf), where: qi =qh (Eq. 27.4-1)
p = qh*G*Cp - qi*(+/-GCpi) for leeward wall, sidewalls, and roof (psf), where: qi = qh (Eq. 27.4-1)

Parallel to Ridge Wind Load Tabulation for MWFRS - Buildings of Any Height						
Surface	z (ft.)	Kz	qz (psf)	Cp	p = Net Design Press. (psf)	
					(w/ +GCpi)	(w/ -GCpi)
Windward Wall	0	0.85	22.35	0.80	2.91	27.49
For z = hr:	13.00	0.85	22.35	0.80	2.91	27.49
For z = he:	9.50	0.85	22.35	0.80	2.91	27.49
For z = h:	11.25	0.85	22.35	0.80	2.91	27.49
Leeward Wall	All	-	-	-0.33	-18.47	6.12
Side Walls	All	-	-	-0.70	-25.59	-1.01
Roof (zone #1) cond. 1	-	-	-	-0.90	-29.39	-4.81
Roof (zone #1) cond. 2	-	-	-	-0.18	-15.71	8.87
Roof (zone #2) cond. 1	-	-	-	-0.90	-29.39	-4.81
Roof (zone #2) cond. 2	-	-	-	-0.18	-15.71	8.87
Roof (zone #3) cond. 1	-	-	-	-0.50	-21.79	2.79
Roof (zone #3) cond. 2	-	-	-	-0.18	-15.71	8.87
Roof (zone #4) cond. 1	-	-	-	-0.30	-17.99	6.59
Roof (zone #4) cond. 2	-	-	-	-0.18	-15.71	8.87

Notes: 1. (+) and (-) signs signify wind pressures acting toward & away from respective surfaces.
2. Per Code Section 27.4.7, the minimum wind load for MWFRS shall not be less than 16 psf.

4. Roof zone #1 is applied for horizontal distance of 0 to h/2 from windward edge.
5. Roof zone #2 is applied for horizontal distance of h/2 to h from windward edge.
6. Roof zone #3 is applied for horizontal distance of h to 2*h from windward edge.
7. Roof zone #4 is applied for horizontal distance of > 2*h from windward edge.

Determination of Gust Effect Factor, G:

Is Building Flexible? No — f >=1 Hz.

1: Simplified Method for Rigid Building

G = 0.850

Parameters Used in Both Item #2 and Item #3 Calculations (from Table 26.9-1):

α^ =	0.105	
b^ =	1.00	
α(bar) =	0.154	
b(bar) =	0.65	
c =	0.20	
ℓ =	500	ft.
ε(bar) =	0.200	
z(min) =	15	ft.

Calculated Parameters Used in Both Rigid and/or Flexible Building Calculations:

z(bar) =	15.00	= 0.6*h , but not < z(min) , ft. Table 26.9-1
Iz(bar) =	0.228	= c*(33/z(bar))^(1/6) , Eq. 26.9-7
Lz(bar) =	427.06	= ℓ*(z(bar)/33)^(ε(bar)) , Eq. 26.9-9
gq =	3.4	(3.4, per Sect. 26.9.4)
gv =	3.4	(3.4, per Sect. 26.9.4)
gr =	4.541	= (2*(LN(3600*f)))^(1/2)+0.577/(2*LN(3600*f))^(1/2) , Eq. 26.9-11
Q =	0.949	= (1/(1+0.63*((B+h)/Lz(bar))^0.63))^(1/2) , Eq. 26.9-8

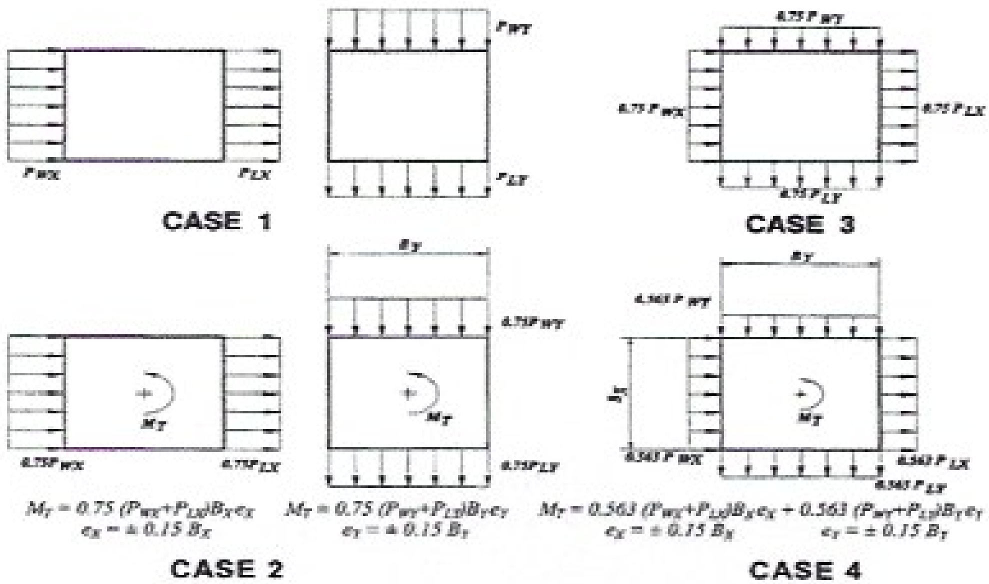
2: Calculation of G for Rigid Building

G = 0.898 = 0.925*((1+1.7*gq*Iz(bar)*Q)/(1+1.7*gv*Iz(bar))) , Eq. 26.9-6

3: Calculation of Gf for Flexible Building

β =	0.050	Damping Ratio
Ct =	0.035	Period Coefficient
T =	0.215	= Ct*h^(3/4) , sec. (Approximate fundamental period)
f =	4.651	= 1/T , Hz. (Natural Frequency)
V(fps) =	N.A.	= V(mph)*(88/60) , ft./sec.
V(bar,zbar) =	N.A.	= b(bar)*(z(bar)/33)^(α(bar))*V*(88/60) , ft./sec. , Eq. 26.9-16
N1 =	N.A.	= f*Lz(bar)/(V(bar,zbar)) , Eq. 26.9-14
Rn =	N.A.	= 7.47*N1/(1+10.3*N1)^(5/3) , Eq. 26.9-13
ηh =	N.A.	= 4.6*f*h/(V(bar,zbar))
Rh =	N.A.	= (1/ηh)-1/(2*ηh^2)*(1-e^(-2*ηh)) for ηh>0, or = 1 for ηh=0 ,Eq. 26.9-15a, b
ηb =	N.A.	= 4.6*f*B/(V(bar,zbar))
RB =	N.A.	= (1/ηb)-1/(2*ηb^2)*(1-e^(-2*ηb)) for ηb>0, or = 1 for ηb=0,Eq. 26.9-15a, b
ηd =	N.A.	= 15.4*f*L/(V(bar,zbar))
RL =	N.A.	= (1/ηd)-1/(2*ηd^2)*(1-e^(-2*ηd)) for ηd>0, or = 1 for ηd=0 ,Eq. 26.9-15a, b
R =	N.A.	= ((1/β)*Rn*Rh*RB*(0.53+0.47*RL))^(1/2) , Eq. 26.9-12
Gf =	N.A.	= 0.925*(1+1.7*Iz(bar)*(gq^2*Q^2+gr^2*R^2)^(1/2))/(1+1.7*gv*Iz(bar)) , Eq. 26.9-10
Use: G =	0.850	

Figure 27.4-1 - Design Wind Load Cases of MWFRS for Buildings of All Heights



Case 1: Full design wind pressure acting on the projected area perpendicular to each principal axis of the structure, considered separately along each principal axis.

Case 2: Three quarters of the design wind pressure acting on the projected area perpendicular to each principal axis of the structure in conjunction with a torsional moment as shown, considered separately for each principal axis.

Case 3: Wind pressure as defined in Case 1, but considered to act simultaneously at 75% of the specified value.

Case 4: Wind pressure as defined in Case 2, but considered to act simultaneously at 75% of the specified value.

Notes:
1. Design wind pressures for windward (Pw) and leeward (PL) faces shall be determined in accordance with the provisions of Section 27.4.1 and 27.4.2 as applicable for buildings of all heights.
2. Above diagrams show plan views of building.
3. Notation:
   - P_{wx}, P_{wy} = Windward face pressure acting in the X, Y principal axis, respectively.
   - P_{Lx}, P_{Ly} = Leeward face pressure acting in the X, Y principal axis, respectively.
   - e (e_x, e_y) = Eccentricity for the X, Y principal axis of the structure, respectively.
   - M_T = Torsional moment per unit height acting about a vertical axis of the building.

WIND LOADING ANALYSIS - Wall Components and Cladding

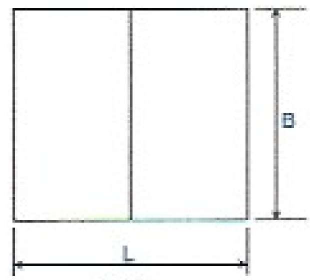
Per ASCE 7-10 Code for Buildings of Any Height

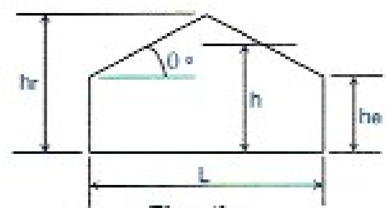
Using Part 1 & 3: Analytical Procedure (Section 30.4 & 30.6)

Job Name:	Clubhouse	Subject:	Main Roof Wall		
Job Number:		Originator:		Checker:	

Input Data:

Wind Speed, V =	110	mph (Wind Map, Figure 26.5-1A-C)
Bldg. Classification =	II	(Table 1.5-1 Risk Category)
Exposure Category =	C	(Sect. 26.7)
Ridge Height, hr =	21.00	ft. (hr >= he)
Eave Height, he =	15.00	ft. (he <= hr)
Building Width =	28.67	ft. (Normal to Building Ridge)
Building Length =	30.00	ft. (Parallel to Building Ridge)
Roof Type =	Monoslope	(Gable or Monoslope)
Topo. Factor, Kzt =	1.00	(Sect. 26.8 & Figure 26.8-1)
Direct. Factor, Kd =	0.85	(Table 26.6)
Enclosed? (Y/N)	Y	(Sect. 26.6-1 & Figure 26.11-1)
Hurricane Region?	N	
Component Name =	Wall	(Girt, Siding, Wall, or Fastener)
Effective Area, Ae =	23	ft.^2 (Area Tributary to C&C)





Resulting Parameters and Coefficients:

Roof Angle, θ =	11.82	deg.
Mean Roof Ht., h =	18.00	ft. (h = (hr+he)/2, for roof angle >10 deg.)

Wall External Pressure Coefficients, GCp:

GCp Zone 4 Pos. =	0.94	(Fig. 30.4-1)
GCp Zone 5 Pos. =	0.94	(Fig. 30.4-1)
GCp Zone 4 Neg. =	-1.04	(Fig. 30.4-1)
GCp Zone 5 Neg. =	-1.27	(Fig. 30.4-1)

Positive & Negative Internal Pressure Coefficients, GCpi (Figure 26.11-1):

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

If z <= 15 then: Kz = 2.01*(15/zg)^(2/α) , If z > 15 then: Kz = 2.01*(z/zg)^(2/α) (Table 30.3-1)

α =	9.50	(Table 26.9-1)
zg =	900	(Table 26.9-1)
Kh =	0.88	(Kh = Kz evaluated at z = h)

Velocity Pressure: qz = 0.00256*Kz*Kzt*Kd*V^2 (Sect. 30.3.2, Eq. 30.3-1)

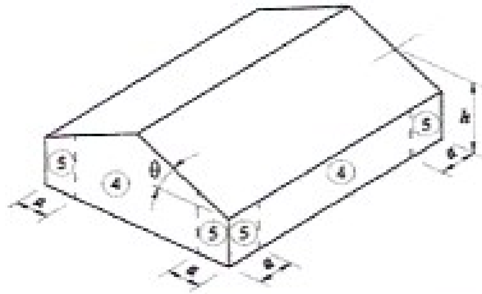
qh = 23.23 psf   qh = 0.00256*Kh*Kzt*Kd*V^2 (qz evaluated at z = h)

Design Net External Wind Pressures (Sect. 30.4 & 30.6):
For h <= 60 ft.: p = qh*((GCp) - (+/-GCpi)) (psf)
For h > 60 ft.: p = q*(GCp) - qi*(+/-GCpi) (psf)
where: q = qz for windward walls, q = qh for leeward walls and side walls
qi = qh for all walls (conservatively assumed per Sect. 30.6)

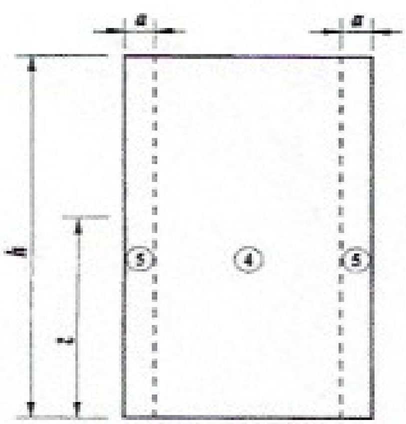
Wind Load Tabulation for Wall Components & Cladding							
Component	z (ft.)	Kh	qh (psf)	p = Net Design Pressures (psf)			
				Zone 4 (+)	Zone 4 (-)	Zone 5 (+)	Zone 5 (-)
Wall	0	0.88	23.23	25.92	-28.24	25.92	-33.73
	15.00	0.88	23.23	25.92	-28.24	25.92	-33.73
	20.00	0.88	23.23	25.92	-28.24	25.92	-33.73
For z = hr:	21.00	0.88	23.23	25.92	-28.24	25.92	-33.73
For z = he:	15.00	0.88	23.23	25.92	-28.24	25.92	-33.73
For z = h:	18.00	0.88	23.23	25.92	-28.24	25.92	-33.73

Notes: 1. (+) and (-) signs signify wind pressures acting toward & away from respective surfaces.
2. Width of Zone 5 (end zones), 'a' = 3.00 ft.
3. Per Code Section 30.2.2, the minimum wind load for C&C shall not be less than 16 psf.
4. References : a. ASCE 7-10, "Minimum Design Loads for Buildings and Other Structures".
b. "Guide to the Use of the Wind Load Provisions of ASCE 7-02"
by: Kishor C. Mehta and James M. Delahay (2004).

Wall Components and Cladding:



Wall Zones for Buildings with h <= 60 ft.



Wall Zones for Buildings with h > 60 ft.

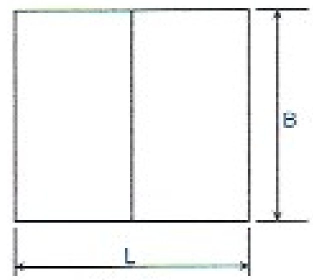
WIND LOADING ANALYSIS - Roof Components and Cladding

Per ASCE 7-10 Code for Bldgs. of Any Height with Gable Roof θ <= 45° or Monoslope Roof θ <= 3°
Using Part 1 & 3: Analytical Procedure (Section 30.4 & 30.6)

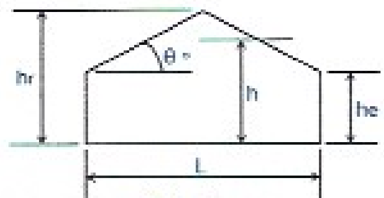
Job Name:	Clubhouse	Subject:	Main Roof		
Job Number:		Originator:		Checker:	

Input Data:

Wind Speed, V =	110	mph (Wind Map, Figure 26.5-1A-C)
Bldg. Classification =	II	(Table 1.5-1 Risk Category)
Exposure Category =	C	(Sect. 26.7)
Ridge Height, hr =	21.00	ft. (hr >= he)
Eave Height, he =	15.00	ft. (he <= hr)
Building Width =	28.67	ft. (Normal to Building Ridge)
Building Length =	30.00	ft. (Parallel to Building Ridge)
Roof Type =	Monoslope	(Gable or Monoslope)
Topo. Factor, Kzt =	1.00	(Sect. 26.8 & Figure 26.8-1)
Direct. Factor, Kd =	0.85	(Table 26.6)
Enclosed? (Y/N)	Y	(Sect. 26.6-1 & Figure 26.11-1)
Hurricane Region?	N	
Component Name =	Joist	(Purlin, Joist, Decking, or Fastener)
Effective Area, Ae =	38	ft.^2 (Area Tributary to C&C)
Overhangs? (Y/N)	Y	(if used, overhangs on all sides)



Plan



Elevation

Resulting Parameters and Coefficients:

Roof Angle, θ =	11.82	deg.
Mean Roof Ht., h =	18.00	ft. (h = (hr+he)/2, for roof angle >10 deg.)

Monoslope Roof > 3 degrees!

Roof External Pressure Coefficients, GCp:

GCp Zone 1-3 Pos. =	0.38	(Fig. 30.4-2A, 30.4-2B, and 30.4-2C)
GCp Zone 1 Neg. =	-0.84	(Fig. 30.4-2A, 30.4-2B, and 30.4-2C)
GCp Zone 2 Neg. =	-2.20	(Fig. 30.4-2A, 30.4-2B, and 30.4-2C)
GCp Zone 3 Neg. =	-3.00	(Fig. 30.4-2A, 30.4-2B, and 30.4-2C)

Positive & Negative Internal Pressure Coefficients, GCpi (Figure 26.11-1):

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

If z <= 15 then: Kz = 2.01*(15/zg)^(2/α) , If z > 15 then: Kz = 2.01*(z/zg)^(2/α) (Table 30.3-1)

α =	9.50	(Table 26.9-1)
zg =	900	(Table 26.9-1)
Kh =	0.88	(Kh = Kz evaluated at z = h)

Velocity Pressure: qz = 0.00256*Kz*Kzt*Kd*V^2 (Sect. 30.3.2, Eq. 30.3-1)

qh = 23.23 psf   qh = 0.00256*Kh*Kzt*Kd*V^2 (qz evaluated at z = h)

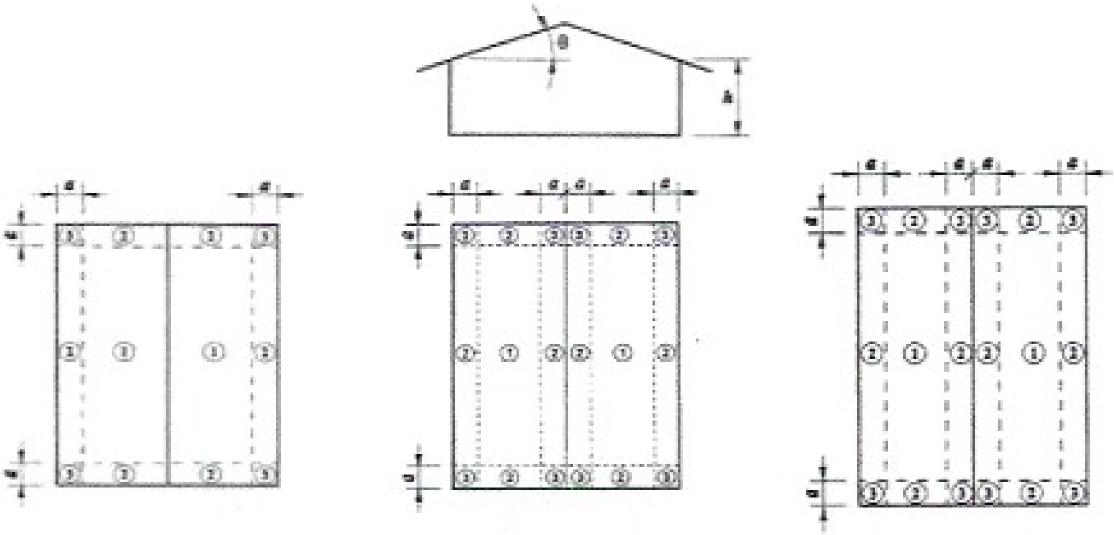
Design Net External Wind Pressures (Sect. 30.4 & 30.6):
For h <= 60 ft.: p = qh*((GCp) - (+/-GCpi)) (psf)
For h > 60 ft.: p = q*(GCp) - qi*(+/-GCpi) (psf)
where: q = qh for roof
qi = qh for roof (conservatively assumed per Sect. 30.6)

Wind Load Tabulation for Roof Components & Cladding

Component	z (ft.)	Kh	qh (psf)	p = Net Design Pressures (psf)			
				Zone 1,2,3 (+)	Zone 1 (-)	Zone 2 (-)	Zone 3 (-)
Joist	0	0.88	23.23	13.10	-23.74	-55.28	-73.96
	15.00	0.88	23.23	13.10	-23.74	-55.28	-73.96
	20.00	0.88	23.23	13.10	-23.74	-55.28	-73.96
For z = hr:	21.00	0.88	23.23	13.10	-23.74	-55.28	-73.96
For z = he:	15.00	0.88	23.23	13.10	-23.74	-55.28	-73.96
For z = h:	18.00	0.88	23.23	13.10	-23.74	-55.28	-73.96

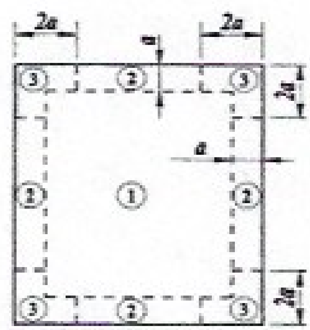
Notes: 1. (+) and (-) signs signify wind pressures acting toward & away from respective surfaces.
2. Width of Zone 2 (edge), 'a' = 3.00 ft.
3. Width of Zone 3 (corner), 'a' = 3.00 ft.
4. For monoslope roofs with θ <= 3 degrees, use Fig. 30.4-2A for 'GCp' values with 'qh'.
5. For buildings with h > 60' and θ > 10 degrees, use Fig. 30.6-1 for 'GCpi' values with 'qh'.
6. For all buildings with overhangs, use Fig. 30.4-2B for 'GCp' values per Sect. 30.10.
7. If a parapet >= 3' in height is provided around perimeter of roof with θ <= 10 degrees, Zone 3 shall be treated as Zone 2.
8. Per Code Section 30.2.2, the minimum wind load for C&C shall not be less than 16 psf.
9. References : a. ASCE 7-02, "Minimum Design Loads for Buildings and Other Structures".
b. "Guide to the Use of the Wind Load Provisions of ASCE 7-02"
by: Kishor C. Mehta and James M. Delahay (2004).

Roof Components and Cladding:



θ <= 7 deg. | 7 deg. < θ <= 27 deg. | 27 deg. < θ <= 45 deg.

Roof Zones for Buildings with h <= 60 ft.
(for Gable Roofs <= 45° and Monoslope Roofs <= 3°)



ROOF PLAN

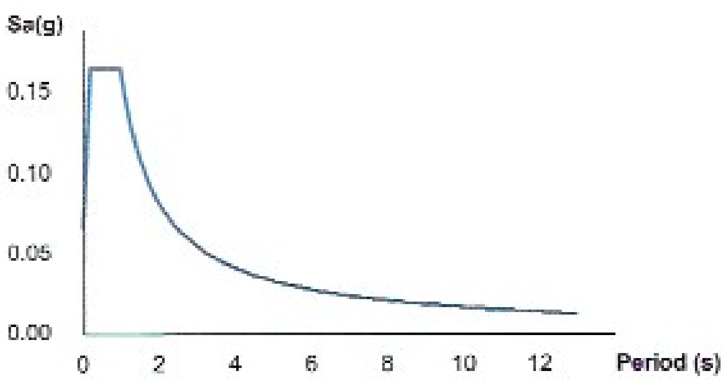
Roof Zones for Buildings with h > 60 ft.
(for Gable Roofs <= 10° and Monoslope Roofs <= 3°)

ATC Hazards by Location

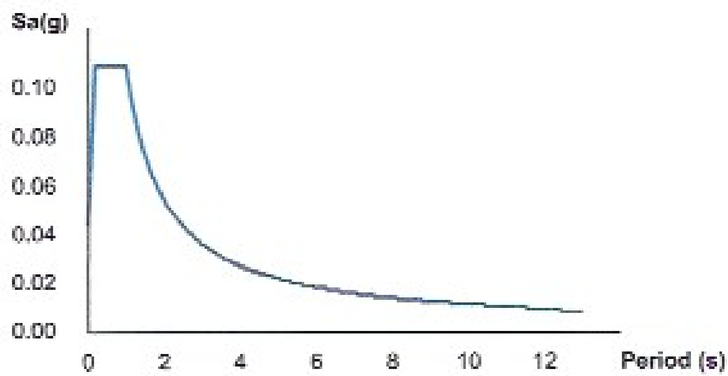
Search Information

Address:	150 Highway and Pryor Road, Lee's Summit, Missouri
Coordinates:	38.8536739, -94.4172055
Elevation:	1037 ft
Timestamp:	2020-03-20T19:30:00.180Z
Hazard Type:	Seismic
Reference Document:	ASCE7-16
Risk Category:	II
Site Class:	D-default

MCER Horizontal Response Spectrum



Design Horizontal Response Spectrum



Basic Parameters

Name	Value	Description
S_S	0.1	MCE_R ground motion (period=0.2s)
S_1	0.068	MCE_R ground motion (period=1.0s)
S_{MS}	0.16	Site-modified spectral acceleration value
S_{M1}	0.164	Site-modified spectral acceleration value
S_{DS}	0.107	Numeric seismic design value at 0.2s SA
S_{D1}	0.109	Numeric seismic design value at 1.0s SA

Additional Information

Name	Value	Description
SDC	B	Seismic design category
F_a	1.6	Site amplification factor at 0.2s
F_v	2.4	Site amplification factor at 1.0s

CR_S	0.926	Coefficient of risk (0.2s)
CR_1	0.876	Coefficient of risk (1.0s)
PGA	0.047	MCE_G peak ground acceleration
F_{PGA}	1.6	Site amplification factor at PGA
PGA_M	0.076	Site modified peak ground acceleration
T_L	12	Long-period transition period (s)
SsRT	0.1	Probabilistic risk-targeted ground motion (0.2s)
SsUH	0.108	Factored uniform-hazard spectral acceleration (2% probability of exceedance in 50 years)
SsD	1.5	Factored deterministic acceleration value (0.2s)
S1RT	0.068	Probabilistic risk-targeted ground motion (1.0s)
S1UH	0.078	Factored uniform-hazard spectral acceleration (2% probability of exceedance in 50 years)
S1D	0.6	Factored deterministic acceleration value (1.0s)
PGAd	0.5	Factored deterministic acceleration value (PGA)

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

Disclaimer

Hazard loads are provided by the U.S. Geological Survey Seismic Design Web Services.

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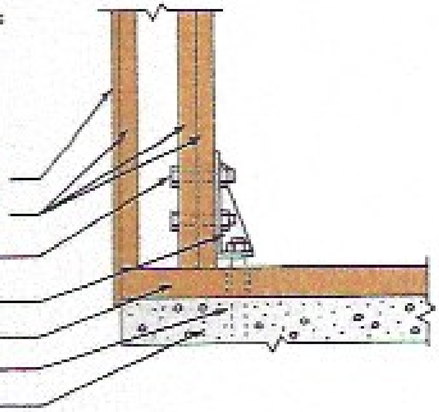
Page L1.P

Figure 19 Shear Wall Foundation Anchor

High shear wall overturning moments may be transferred by a fabricated steel bracket such as this. Regular foundation bolts are all that is required in many cases. Additional strength is furnished by the corner panel acting as a nailed gusset plate.



Typical Layout for Shear Walls

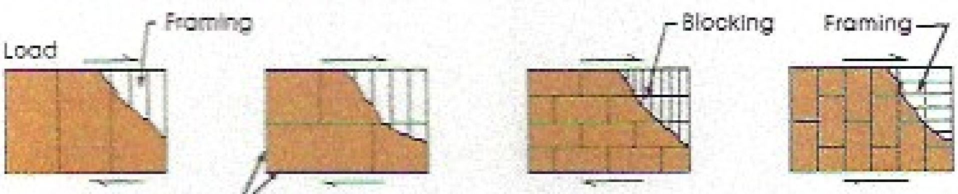


Table 23 Recommended Shear (pounds per foot) for APA Panel Shear Walls with Framing of Douglas Fir, Larch, or Southern Pine(a) for Wind or Seismic Loading(b)

Panel Grade	Minimum Nominal Panel Thickness (in.)	Minimum Nail Penetration in Framing (in.)	Panels Applied Direct to Framing: Nail Size (common or galvanized box)	Nail Spacing at Panel Edges (in.): 6	4	3	2(e)	Panels Applied Over 1/2" Gypsum Sheathing: Nail Size (common or galvanized box)	Nail Spacing at Panel Edges (in.): 6	4	3	2(e)
APA STRUCTURAL I RATED SHEATHING EXP 1 or EXT	5/16	1-1/4	6d	200	300	390	510	8d	200	300	390	510
	3/8	1-1/2	8d	220(d)	360(d)	460(d)	610(d)	10d	280	430	550(e)	730
	7/16			255(d)	395(d)	505(d)	670(d)					
	15/32			280	430	550	730					
	15/32	1-5/8	10d	340	510	665(e)	870	—	—	—	—	—
APA RATED SHEATHING EXP 1, EXP 2 or EXT; APA STRUCTURAL II RATED SHEATHING EXP 1 or EXT; APA panel siding(f) and other APA grades except species Group 5	5/16 or 1/4(f)	1-1/4	6d	180	270	350	450	8d	180	270	350	450
	3/8			200	300	390	510		200	300	390	510
	3/8	1-1/2	8d	220(d)	320(d)	410(d)	530(d)	10d	260	380	490(e)	640
	7/16			240(d)	350(d)	450(d)	585(d)					
	15/32			260	380	490	640					
	15/32	1-5/8	10d	310	460	600(e)	770	—	—			
	19/32			340	510	665(e)	870	—	—			
APA panel siding(f) and other APA grades except species Group 5	5/16(f)	1-1/4	Nail Size (galvanized casing) 6d	140	210	275	360	Nail Size (galvanized casing) 8d	140	210	275	360
	3/8	1-1/2	8d	160	240	310	410	10d	160	240	310(e)	410

(a) For framing of other species: (1) Find species group of lumber in the NFPA National Design Spec. (2)(a) For common or galvanized box nails, find shear value from table for nail size, and for STRUCTURAL I panels (regardless of actual grade). (b) For galvanized casing nails, take shear value directly from table. (3) Multiply this value by 0.82 for Lumber Group III or 0.65 for Lumber Group IV.

(b) All panel edges backed with 2-inch nominal or wider framing. Install panels either horizontally or vertically. Space nails 6 inches oc along intermediate framing members for 3/8-inch and 7/16-inch panels installed on studs spaced 24 inches oc. For other conditions and panel thicknesses, space nails 12 inches oc on intermediate supports.

(c) 3/8-inch or 303-16 oc is minimum recommended when applied direct to framing as exterior siding.

(d) Shears may be increased to values shown for 15/32 inch sheathing with same nailing provided (1) studs are spaced a maximum of 16 inches oc, or (2) if panels are applied with long dimension across studs.

(e) Framing shall be 3 inch nominal or wider, and nails shall be staggered where nails are spaced 2 inches oc, and where 10d nails having penetration into framing of more than 1-5/8 inches are spaced 3 inches oc. Exception: Unless otherwise required, 2-inch nominal framing may be used where full nailing surface width is available and nails are staggered.

(f) 303-16 oc plywood may be 11/32-inch, 3/8-inch or thicker. Thickness at point of nailing on panel edges governs shear values.

W895

PACKARD ENGINEERING

Phone: (816) 767-7222
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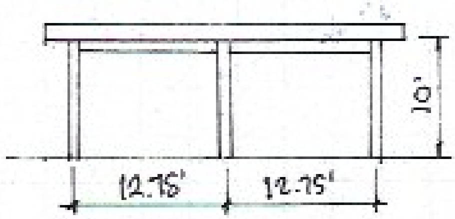
CLIENT	B+A Architecture	Page	L2
		Date	03/20
JOB	Osage Clubhouse	Job No.	B+A20001
		Made By	DJP

- Open End of Patio:



$$F_w = 24\,psf\left(\frac{3'+1'}{2}\right)11' = 528^{\#}$$

$$M_o = 528^{\#}(10') = 5280^{\#\text{-}\prime}$$

$$T = C = \frac{5280^{\#\text{-}\prime}}{23.5'} = 207^{\#}$$

$$V = \frac{528^{\#}}{3} = 176^{\#}$$

$$M_{posts} = 176^{\#}(10') = 1760^{\#\text{-}\prime}$$

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Fax: (816) 767-8105

Roof Framing (Low Slope) → $LL_{snow} = 25$ psf   DL = 15 psf

NA → (Flat) ⟶ $LL_{snow} = 30$ psf, 50 psf drifted, DL = 20 psf ← N/A

Decking = 15/32" APA Rated (32/16) Sheathing

Joists – High roof – $l = 28'$, 16" o.c., GP Wood I Beam, WI60 → 14"; Uplift = 1.33'[(16')24 psf + 3'(31 plf)] = 634#

Patio roof – $l = 15'$, 2x10 @ 16" o.c. (SP#2) Uplift = 1.33'[(9.5')24 psf + 3'(31 plf)] = 427#

Low roof – $l = 10'$, 2x10 @ 16" o.c.

Header – $l = 24'$, $w = 10'(10\text{ psf}) + 4'(40\text{ psf}) = 260$ plf, R = 3120#, $+M = \frac{260\text{ plf}(24')^2}{8} = 18720$#·'

$-M_{ends} = -4480$#·' $- \frac{260\text{ plf}(24')^2}{12} = -16960$#·'   + 587# = 3707# (w/ wind)

3-1¾" × 11¼" LVL (1.9E)?

$\Delta_{a,TL} = \frac{288''}{360} = 0.8''$, $I_r = \frac{5(260\text{ plf})(288'')^4}{12\frac{''}{'}(384)1.9\times10^6(0.80'')} = 1277\text{ in}^4 (\div 3 = 426\text{ in}^4) \to 14''$

use 3-1¾" × 14" LVL (1.9E)

$l = 8'$, $w = 22'(40\text{ psf}) + 7'(10\text{ psf}) = 950$ plf, R = 3800#, +M = 7600#·'

3-2x12

$l = 5'$, $w = 950$ plf, R = 2375#, +M = 2969#·'   3-2x10

Beams – $l_{patio} = 2 \times 12.5' = 25'$, $w = 7.5' + 2'(40\text{ psf}) = 380$ plf, $R_{min} = 1781$#, $R_{max} = 5938$# (middle)

$+M = \frac{380\text{ plf}(12.5')^2}{10} = 5938$#·', $-M = -7422$#·'   + 207# = 1988#   6x12 P.T. #1 SP

Post – P = 5938#, −M = 1760#·' → 6x6 post w/ Simpson MPB66Z base

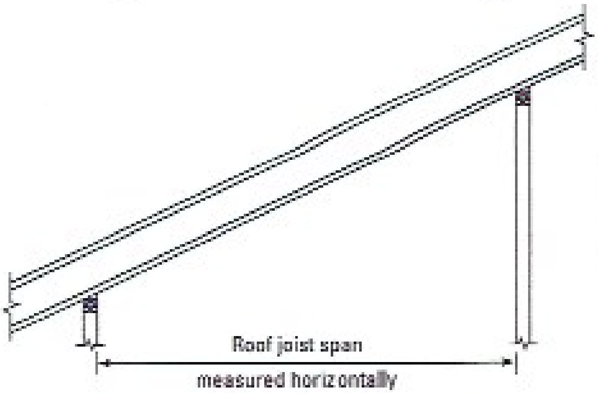
$l = 16.5'$, $w = 40\text{ psf}(4') = 160$ plf, R = 1320#, +M = 5445#·', T = 3980#

drag-strut beam $S_r = \frac{5445\text{\#·'}(12''/')}{750\text{ psi}(1.15)} = 75.76\text{ in}^3$, $A_r = \frac{3980\#}{450\text{ psi}} = 8.84\text{ in}^2$, use 6x12 or built-up 3-2x12

$\frac{8.84}{61.88} + \frac{75.76}{116.0} = 0.80 < 1.0$, ok

Page R1.A

Roof Joist Maximum Spans



Notes:

1. **Roof joists to be sloped min. ¼" in 12".** No camber provided.
2. Maximum deflection is limited to L/180 at total load, L/240 at live load.
3. **Maximum slope is limited to 12" in 12"** for use of these tables.
4. Tables are based on the more restrictive of simple or multiple spans.
5. End spans of multiple-span joists must be at least 40% of the adjacent span.
6. For other loading conditions or on-center spacings, refer to Uniform Load Tables or use FASTBeam® selection software.
7. Minimum end bearing length is 1¾". Minimum intermediate bearing length is 3½".
8. Spans shown below cover a broad range of applications. It may be possible to exceed these spans by analyzing a specific application using FASTBeam software.
9. Tables apply to gravity loads only.
10. Dead load is calculated along the joist length.
11. 20 psf non-snow live loads have been reduced per code for slopes of over 8/12 through 12/12.

Roof Joist Maximum Spans – 115% (Snow)

Refer to Notes above.

Load (PSF)	Joist Series	Joist Depth	Slope of 4/12 or less			Slope of over 4/12 through 8/12			Slope of over 8/12 through 12/12		
			16" o.c.	19.2" o.c.	24" o.c.	16" o.c.	19.2" o.c.	24" o.c.	16" o.c.	19.2" o.c.	24" o.c.
Snow 115% Live 25 Dead 15	GPI 20	9½"	19'-09"	18'-07"	17'-02"	18'-07"	17'-06"	16'-02"	17'-03"	16'-02"	15'-00"
		11⅞"	23'-09"	22'-04"	20'-08"	22'-04"	21'-00"	19'-05"	20'-09"	19'-05"	18'-00"
		14"	27'-02"	25'-04"	22'-08"	25'-07"	24'-00"	22'-01"	23'-08"	22'-03"	20'-07"
	GPI 40	9½"	21'-01"	19'-10"	18'-04"	19'-10"	18'-08"	17'-03"	18'-05"	17'-03"	16'-00"
		11⅞"	25'-03"	23'-06"	21'-00"	23'-09"	22'-04"	20'-05"	22'-00"	20'-08"	19'-02"
		14"	28'-03"	25'-09"	23'-00"	27'-00"	25'-01"	22'-05"	25'-00"	23'-06"	21'-07"
	GPI 65	11⅞"	27'-08"	26'-00"	24'-00"	26'-01"	24'-06"	22'-08"	24'-02"	22'-08"	21'-00"
		14"	31'-06"	29'-07"	27'-05"	29'-08"	27'-11"	25'-10"	27'-06"	25'-10"	23'-11"
		16"	35'-00"	32'-11"	29'-10"	33'-00"	31'-00"	28'-08"	30'-07"	28'-09"	26'-07"
	GPI 90	11⅞"	31'-09"	29'-10"	27'-07"	29'-11"	28'-01"	26'-00"	27'-09"	26'-01"	24'-01"
		14"	36'-01"	33'-10"	31'-04"	34'-00"	31'-11"	29'-07"	31'-06"	29'-07"	27'-05"
		16"	39'-11"	37'-06"	34'-09"	37'-08"	35'-04"	32'-09"	34'-10"	32'-09"	30'-04"
	WI 40	9½"	21'-01"	19'-05"	17'-04"	19'-10"	18'-08"	16'-11"	18'-05"	17'-03"	16'-00"
		11⅞"	24'-03"	22'-02"	19'-09"	23'-07"	21'-07"	19'-03"	22'-00"	20'-08"	18'-07"
		14"	26'-08"	24'-04"	21'-09"	25'-11"	23'-08"	21'-02"	25'-00"	22'-10"	20'-05"
	WI 60	11⅞"	26'-10"	25'-02"	23'-03"	25'-03"	23'-09"	22'-00"	23'-05"	22'-00"	20'-04"
		14"	30'-07"	28'-07"	25'-07"	28'-10"	27'-01"	24'-11"	26'-08"	25'-01"	23'-03"
		16"	33'-09"	30'-10"	27'-06"	32'-00"	30'-00"	26'-10"	29'-08"	27'-10"	25'-09"
	WI 80	11⅞"	29'-10"	28'-00"	25'-11"	28'-01"	26'-05"	24'-05"	26'-01"	24'-06"	22'-08"
		14"	33'-11"	31'-10"	29'-06"	32'-00"	30'-00"	27'-10"	29'-08"	27'-10"	25'-09"
		16"	37'-08"	35'-04"	32'-09"	35'-06"	33'-04"	30'-10"	32'-10"	30'-11"	28'-07"
Snow 115% Live 30 Dead 15	GPI 20	9½"	19'-00"	17'-10"	16'-06"	17'-11"	16'-10"	15'-07"	16'-08"	15'-08"	14'-06"
		11⅞"	22'-10"	21'-05"	19'-06"	21'-06"	20'-03"	18'-09"	20'-00"	18'-09"	17'-05"
		14"	26'-01"	23'-11"	21'-04"	24'-08"	23'-02"	20'-10"	22'-11"	21'-06"	19'-11"
	GPI 40	9½"	20'-03"	19'-00"	17'-05"	19'-01"	17'-11"	16'-07"	17'-09"	16'-08"	15'-05"
		11⅞"	24'-03"	22'-02"	19'-10"	22'-11"	21'-06"	19'-04"	21'-03"	20'-00"	18'-06"
		14"	26'-08"	24'-04"	21'-09"	26'-00"	23'-09"	21'-02"	24'-02"	22'-08"	20'-06"
	GPI 65	11⅞"	26'-07"	24'-11"	23'-01"	25'-01"	23'-07"	21'-10"	23'-04"	21'-11"	20'-03"
		14"	30'-09"	28'-05"	26'-04"	28'-07"	26'-10"	24'-10"	26'-07"	24'-11"	23'-01"
		16"	33'-08"	31'-07"	28'-06"	31'-09"	29'-10"	27'-05"	29'-06"	27'-09"	25'-08"
	GPI 90	11⅞"	30'-06"	28'-08"	26'-05"	28'-10"	27'-01"	25'-00"	26'-09"	25'-02"	23'-03"
		14"	34'-08"	32'-07"	30'-01"	32'-09"	30'-09"	28'-05"	30'-05"	28'-07"	26'-05"
		16"	38'-05"	36'-00"	33'-04"	36'-03"	34'-00"	31'-06"	33'-08"	31'-07"	29'-03"
	WI 40	9½"	20'-01"	18'-04"	16'-04"	19'-01"	17'-11"	16'-00"	17'-09"	16'-08"	15'-05"
		11⅞"	22'-11"	20'-11"	18'-08"	22'-04"	20'-05"	18'-02"	21'-03"	19'-09"	17'-08"
		14"	25'-02"	22'-11"	20'-06"	24'-07"	22'-05"	20'-00"	23'-09"	21'-08"	19'-04"
	WI 60	11⅞"	25'-08"	24'-02"	22'-00"	24'-04"	22'-10"	21'-02"	22'-07"	21'-03"	19'-08"
		14"	29'-05"	27'-00"	24'-01"	27'-09"	26'-01"	23'-07"	25'-09"	24'-02"	22'-05"
		16"	31'-10"	[illegible]	26'-04"	30'-10"	28'-05"	[illegible]	28'-07"	26'-11"	24'-07"
	WI 80	11⅞"	28'-08"	26'-11"	24'-11"	27'-01"	25'-05"	23'-06"	25'-02"	23'-07"	21'-10"
		14"	32'-07"	30'-07"	28'-04"	30'-10"	28'-11"	26'-08"	28'-07"	26'-10"	24'-11"
		16"	36'-02"	34'-00"	30'-08"	34'-02"	32'-01"	29'-08"	31'-09"	29'-10"	27'-07"

Table continues on next page.

Page E1.B

Southern Pine Reference Design Values

Based on SPIB Grading Rules and AWC National Design Specification®
Values in pounds per square inch (psi)

Table 1 Dimension Lumber – 2″ to 4″ thick, 2″ and wider

Effective June 1, 2013

Based on Normal Load Duration and Dry Service (MC ≤ 19%) — See Tables A-1 thru A-4 for Adjustment Factors

NEW DESIGN VALUES

Effective June 1, 2013

Size	Grade	Bending F_b	Tension Parallel to Grain F_t	Shear Parallel to Grain F_v	Compression Perpendicular to Grain $F_{c\perp}$	Compression Parallel to Grain F_c	Modulus of Elasticity E	Modulus of Elasticity E_{min}
2″ to 4″ thick, 2″ to 4″ wide. Includes: 2x2, 2x3, 2x4, 3x3, 3x4, 4x4	Dense Select Structural	2700	1900	175	660	2050	1,900,000	690,000
	Select Structural	2350	1650	175	565	1900	1,800,000	660,000
	Non-Dense Select Structural	2050	1450	175	480	1800	1,600,000	580,000
	No.1 Dense	1650	1100	175	660	1750	1,800,000	660,000
	No.1	1500	1000	175	565	1650	1,600,000	580,000
	No.1 Non-Dense	1300	875	175	480	1550	1,400,000	510,000
	No.2 Dense	1200	750	175	660	1500	1,600,000	580,000
	No.2	1100	675	175	565	1450	1,400,000	510,000
	No.2 Non-Dense	1050	600	175	480	1450	1,300,000	470,000
	No.3 and Stud	650	400	175	565	850	1,300,000	470,000
	Construction	875	500	175	565	1600	1,400,000	510,000
	Standard	475	275	175	565	1300	1,200,000	440,000
	Utility[1]	225	125	175	565	850	1,200,000	440,000
2″ to 4″ thick, 5″ to 6″ wide. Includes: 2x5, 2x6, 3x5, 3x6, 4x5, 4x6	Dense Select Structural	2400	1650	175	660	1900	1,900,000	690,000
	Select Structural	2100	1450	175	565	1800	1,800,000	660,000
	Non-Dense Select Structural	1850	1300	175	480	1700	1,600,000	580,000
	No.1 Dense	1500	1000	175	660	1650	1,800,000	660,000
	No.1	1350	875	175	565	1550	1,600,000	580,000
	No.1 Non-Dense	1200	775	175	480	1450	1,400,000	510,000
	No.2 Dense	1050	650	175	660	1450	1,600,000	580,000
	No.2	1000	600	175	565	1400	1,400,000	510,000
	No.2 Non-Dense	950	525	175	480	1350	1,300,000	470,000
	No.3 and Stud	575	350	175	565	800	1,300,000	470,000
2″ to 4″ thick, 8″ wide. Includes: 2x8, 3x8, 4x8[2]	Dense Select Structural	2200	1550	175	660	1850	1,900,000	690,000
	Select Structural	1950	1350	175	565	1700	1,800,000	660,000
	Non-Dense Select Structural	1700	1200	175	480	1650	1,600,000	580,000
	No.1 Dense	1350	900	175	660	1600	1,800,000	660,000
	No.1	1250	800	175	565	1500	1,600,000	580,000
	No.1 Non-Dense	1100	700	175	480	1400	1,400,000	510,000
	No.2 Dense	975	600	175	660	1400	1,600,000	580,000
	No.2	925	550	175	565	1350	1,400,000	510,000
	No.2 Non-Dense	875	500	175	480	1300	1,300,000	470,000
	No.3 and Stud	525	325	175	565	775	1,300,000	470,000
2″ to 4″ thick, 10″ wide. Includes: 2x10, 3x10, 4x10[2]	Dense Select Structural	1950	1300	175	660	1800	1,900,000	690,000
	Select Structural	1700	1150	175	565	1650	1,800,000	660,000
	Non-Dense Select Structural	1500	1050	175	480	1600	1,600,000	580,000
	No.1 Dense	1200	800	175	660	1550	1,800,000	660,000
	No.1	1050	700	175	565	1450	1,600,000	580,000
	No.1 Non-Dense	950	625	175	480	1400	1,400,000	510,000
	No.2 Dense	850	525	175	660	1350	1,600,000	580,000
	No.2	800	475	175	565	1300	1,400,000	510,000
	No.2 Non-Dense	750	425	175	480	1250	1,300,000	470,000
	No.3 and Stud	475	275	175	565	750	1,300,000	470,000
2″ to 4″ thick, 12″ wide[3]. Includes: 2x12, 3x12, 4x12[2]	Dense Select Structural	1800	1250	175	660	1750	1,900,000	690,000
	Select Structural	1600	1100	175	565	1650	1,800,000	660,000
	Non-Dense Select Structural	1400	975	175	480	1550	1,600,000	580,000
	No.1 Dense	1100	750	175	660	1500	1,800,000	660,000
	No.1	1000	650	175	565	1400	1,600,000	580,000
	No.1 Non-Dense	900	575	175	480	1350	1,400,000	510,000
	No.2 Dense	800	500	175	660	1300	1,600,000	580,000
	No.2	750	450	175	565	1250	1,400,000	510,000
	No.2 Non-Dense	700	400	175	480	1250	1,300,000	470,000
	No.3 and Stud	450	250	175	565	725	1,300,000	470,000

(1) For Utility, design values apply to 4″-wide lumber only. (2) For lumber 4″ thick and 8″ or wider, multiply the F_b value by $C_F = 1.1$. (3) For lumber wider than 12″, multiply these 12″-width design values for F_b, F_t and F_c by $C_F = .90$, and use these 12″-width design values for the other properties.

The Southern Forest Products Association (SFPA) does not test lumber or establish design values. Neither SFPA, nor its members, warrant that the design values are correct, and disclaim responsibility for injury or damage resulting from the use of such design values.

Page R1.C

Allowable Design Properties(1) (100% Load Duration)

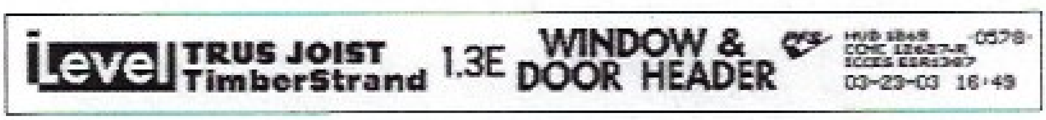
Grade	Width	Design Property	Depth 4⅜"	5½"	5½" Plank Orientation	7¼"	8⅝"	9¼"	9½"	11¼"	11⅞"	14"	16"	18"	20"
TimberStrand® LSL															
1.3E	3½"	Moment (ft-lbs)	1,735	2,685	1,780	4,550	6,335	7,240		10,520					
		Shear (lbs)	4,085	5,135	1,925	6,765	8,050	8,635		10,500					
		Moment of Inertia (in.⁴)	24	49	20	111	187	231		415					
		Weight (plf)	4.5	5.6	5.6	7.4	8.8	9.4		11.5					
1.55E	1¾"	Moment (ft-lbs)						4,950	5,210	7,195	7,975	10,920	14,090		
		Shear (lbs)						3,345	3,435	4,070	4,295	5,065	5,785		
		Moment of Inertia (in.⁴)						115	125	208	244	400	597		
		Weight (plf)						5.1	5.2	6.2	6.5	7.7	8.8		
	3½"	Moment (ft-lbs)						9,905	10,420	14,390	15,955	21,840	28,180		
		Shear (lbs)						6,690	6,870	8,140	8,590	10,125	11,575		
		Moment of Inertia (in.⁴)						231	250	415	488	800	1,195		
		Weight (plf)						10.1	10.4	12.3	13	15.3	17.5		
Microllam® LVL															
1.9E	1¾"	Moment (ft-lbs)		2,125		3,555		5,600	5,885	8,070	8,925	12,130	15,555	19,375	23,580
		Shear (lbs)		1,830		2,410		3,075	3,160	3,740	3,950	4,655	5,320	5,985	6,650
		Moment of Inertia (in.⁴)		24		56		115	125	208	244	400	597	851	1,167
		Weight (plf)		2.8		3.7		4.7	4.8	5.7	6.1	7.1	8.2	9.2	10.2
Parallam® PSL															
2.0E	2¹¹⁄₁₆"	Moment (ft-lbs)						9,535	10,025	13,800	15,280	20,855	26,840	33,530	
		Shear (lbs)						4,805	4,935	5,845	6,170	7,275	8,315	9,350	
		Moment of Inertia (in.⁴)						175	192	319	375	615	917	1,305	
		Weight (plf)						7.8	8.0	9.5	10.0	11.8	13.4	15.1	
	3½"	Moment (ft-lbs)						12,415	13,055	17,970	19,900	27,160	34,955	43,665	
		Shear (lbs)						6,260	6,430	7,615	8,035	9,475	10,825	12,180	
		Moment of Inertia (in.⁴)						231	250	415	488	800	1,195	1,701	
		Weight (plf)						10.1	10.4	12.3	13.0	15.3	17.5	19.7	
	5¼"	Moment (ft-lbs)						18,625	19,585	26,955	29,855	40,740	52,430	65,495	
		Shear (lbs)						9,390	9,645	11,420	12,055	14,210	16,240	18,270	
		Moment of Inertia (in.⁴)						346	375	623	733	1,201	1,792	2,552	
		Weight (plf)						15.2	15.6	18.5	19.5	23.0	26.3	29.5	
	7"	Moment (ft-lbs)						24,830	26,115	35,940	39,805	54,325	69,905	87,325	
		Shear (lbs)						12,520	12,855	15,225	16,070	18,945	21,655	24,360	
		Moment of Inertia (in.⁴)						462	500	831	977	1,601	2,389	3,402	
		Weight (plf)						20.2	20.8	24.6	26.0	30.6	35.0	39.4	

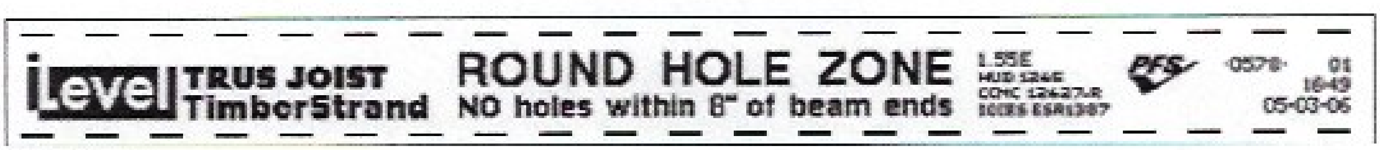
(1) For product in beam orientation, unless otherwise noted.

Some sizes may not be available in your region.

TimberStrand® LSL Grade Verification

TimberStrand® LSL is available in more than one grade. The product will be stamped with its grade information, as shown in the examples below. With 1.55E TimberStrand® LSL, larger holes can be drilled through the beam. See **Allowable Holes** on page 36.





Actual stamps shown.

Page K1.D

Design Stresses

Grade	Orientation	G Shear Modulus of Elasticity (psi)	E Modulus of Elasticity (psi)	E_{min} Adjusted Modulus of Elasticity[1] (psi)	F_b Flexural Stress[2] (psi)	F_t Tension Stress[3] (psi)	$F_{c\perp}$ Compression Perpendicular to Grain[4] (psi)	$F_{c\parallel}$ Compression Parallel to Grain (psi)	F_v Horizontal Shear Parallel to Grain (psi)	SG Equivalent Specific Gravity[5]
TimberStrand® LSL										
1.3E	Beam/Column	81,250	1.3×10^6	660,750	1,700	1,075	680	1,400	400	0.50[6]
1.3E	Plank	81,250	1.3×10^6	660,750	1,900[7]	1,075	435	1,400	150	0.50[6]
1.55E	Beam	96,875	1.55×10^6	787,815	2,325	1,070[8]	800	2,050	310[8]	0.50[6]
Microllam® LVL										
1.9E	Beam	118,750	1.9×10^6	965,710	2,600	1,555	750	2,510	285	0.50
Parallam® PSL										
1.8E and 2.0E	Column	112,500	1.8×10^6	914,880	2,400[9]	1,755	425[9]	2,500	190[9]	0.50
2.0E[10]	Beam	125,000	2.0×10^6	1,016,535	2,900	2,025	750	2,900	290	0.50

(1) Reference modulus of elasticity for beam stability and column stability calculations, per NDS® 2005.

(2) For 12" depth. For other depths, multiply F_b by the appropriate factor as follows:

– For TimberStrand® LSL, multiply by $\left[\frac{12}{d}\right]^{0.092}$

– For Microllam® LVL, multiply by $\left[\frac{12}{d}\right]^{0.136}$

– For Parallam® PSL, multiply by $\left[\frac{12}{d}\right]^{0.111}$

(3) F_t has been adjusted to reflect the volume effects for most standard applications.

(4) $F_{c\perp}$ may not be increased for duration of load.

(5) For lateral connection design only.

(6) Specific gravity of 0.58 may be used for bolts installed perpendicular to face and loaded perpendicular to grain.

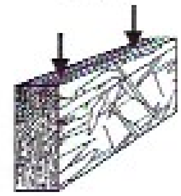
(7) Values are for thickness up to 3½".

(8) Values account for large hole capabilities. See **Allowable Holes** on page 36.

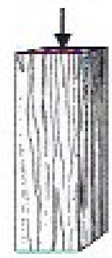
(9) Values are for plank orientation.

(10) Use the **Parallam® PSL 1.8E and 2.0E Column** orientation design stresses when designing columns.

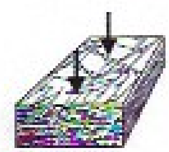
Beam



Column



Plank



General Assumptions for iLevel® Trus Joist® Residential Beams

- Lateral support is required at bearing and along the span at 24" on-center, maximum.
- Bearing lengths are based on each product's bearing stress for applicable grade and orientation.
- All members 7¼" and less in depth are restricted to a maximum deflection of 5⁄16".
- Beams that are 1¾" x 16" and deeper require multiple plies.
- No camber.
- Beams and columns must remain straight to within (L)/(4608) (in.) of true alignment. L is the unrestrained length of the member in feet.
- Tables on pages 8–15 include load reductions applied in accordance with code.

For applications not covered in this brochure, contact your iLevel representative.

See pages 38 and 39 for multiple-member beam connections.

TimberStrand® LSL, Microllam® LVL, and untreated Parallam® PSL are intended for dry-use applications

Product Storage

H/TSP

SIMPSON Strong-Tie

Page R1.E

Seismic and Hurricane Ties

Simpson Strong-Tie hurricane ties provide a positive connection between truss/rafter and the wall of the structure to resist wind and seismic forces.

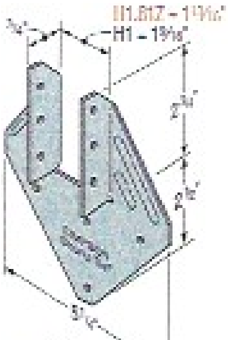
Material: See table

Finish: Galvanized. H1.81Z, H7Z and H11Z — ZMAX® coating. Some models available in stainless steel or ZMAX; see Corrosion Information, pp. 13–15 or visit **strongtie.com**.

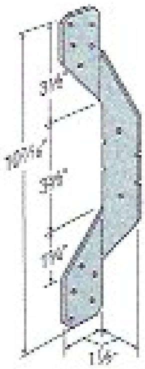
Installation:

- Use all specified fasteners; see General Notes.
- Hurricane ties can be installed with flanges facing inward or outward.
- H2.5T, H3 and H6 ties are shipped in equal quantities of right and left versions (right versions shown).
- Hurricane ties do not replace solid blocking.
- When installing ties on plated trusses (on the side opposite the truss plate) do not fasten through the truss plate from behind. This can force the truss plate off of the truss and compromise truss performance.
- H10A optional nailing to connect shear blocking, use 8d nails. Slots allow maximum field bending up to a pitch of 6:12, use H10A sloped loads for field-bent installation.

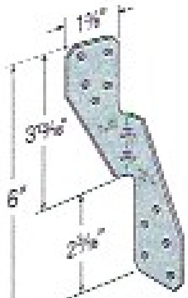
Codes: See p. 12 for Code Reference Key Chart



H1
(H1.81Z similar)



H2A
(H2ASS similar)



H2.5A
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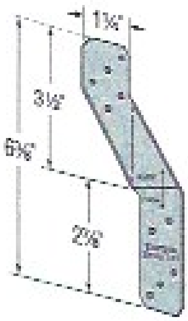
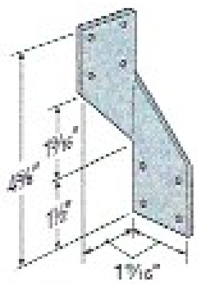
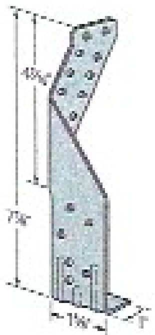


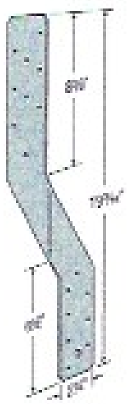
H2.5T



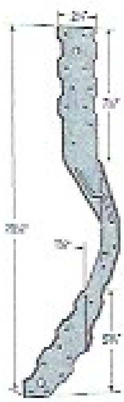
H3



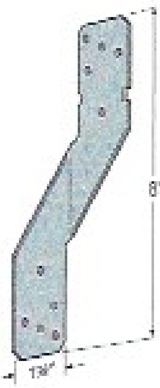
TSP



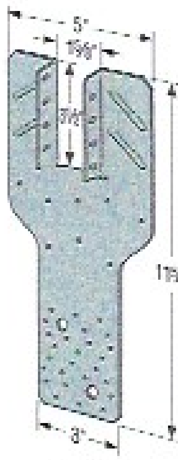
H6



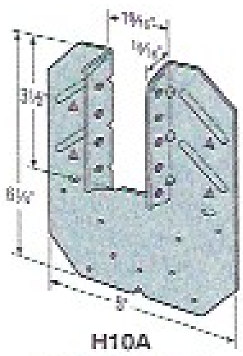
H7Z



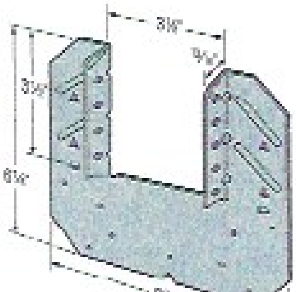
H8



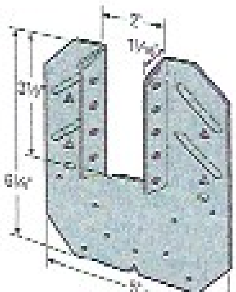
H10S



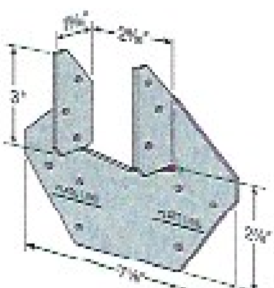
H10A
(H10ASS similar)



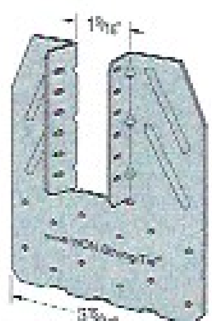
H10A-2



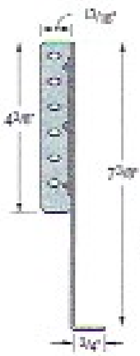
H10AR



H11Z



H14



H14 Profile

Straps and Ties

SIMPSON Strong-Tie

Page R3.F

Seismic and Hurricane Ties (cont.)

These products are available with additional corrosion protection. For more information, see p. 15.

SS For stainless steel fasteners, see p. 21.

SD Many of these products are approved for installation with Strong-Drive® SD Connector screws. See pp. 335–337 for more information.

Model No.	Ga.	Fasteners (in.) To Rafters/ Truss	Fasteners (in.) To Plates	Fasteners (in.) To Studs	DF/SP Allowable Loads Uplift (160)	DF/SP Lateral (160) F_1	DF/SP Lateral (160) F_2	Uplift with 0.131" x 1½" Nails (160)	SPF/HF Allowable Loads Uplift (160)	SPF/HF Lateral (160) F_1	SPF/HF Lateral (160) F_2	Uplift with 0.131" x 1½" Nails (160)	Code Ref.
H1	18	(6) 0.131 x 1½	(4) 0.131 x 2½	—	480	510	190	455	475	440	165	370	IBC, FL, LA
H1.81Z	18	(6) 0.131 x 1½	(4) 0.131 x 2½	—	350	335	195	330	300	290	150	260	—
H2A	18	(5) 0.131 x 1½	(2) 0.131 x 1½	(5) 0.131 x 1½	525	130	55	—	495	130	55	—	IBC, FL, LA
H2ASS	18	(5) 0.131 x 1½	(2) 0.131 x 1½	(5) 0.131 x 1½	400	130	55	400	345	130	55	345	—
H2.5A	18	(5) 0.131 x 2½	(5) 0.131 x 2½	—	565	110	110	575	535	110	110	495	IBC, FL, LA
H2.5ASS	18	(5) 0.131 x 2½	(5) 0.131 x 2½	—	440	75	70	365	380	75	70	310	—
H2.5T	18	(5) 0.131 x 2½	(5) 0.131 x 2½	—	495	135	145	420	495	135	145	420	IBC, FL, LA
H3	18	(4) 0.131 x 2½	(4) 0.131 x 2½	—	400	210	170	415	365	180	145	290	
H6	16	—	(8) 0.131 x 2½	(8) 0.131 x 2½	1,230	—	—	—	1,055	—	—	—	IBC, FL
H7Z	16	(4) 0.131 x 2½	(2) 0.131 x 1½	(8) 0.131 x 2½	930	410	—	—	715	355	—	—	
H8	18	(5) 0.148 x 1½	(5) 0.148 x 1½	—	780	95	90	630	710	95	90	510	IBC, FL, LA
H10A Field Bent	18	(9) 0.148 x 1½	(9) 0.148 x 1½	—	855	590	285	—	760	505	285	—	
H10A	18	(9) 0.148 x 1½	(9) 0.148 x 1½	—	1,040	565	285	—	1,015	485	285	—	
H10ASS	18	(9) 0.148 x 1½	(9) 0.148 x 1½	—	970	565	170	—	835	485	170	—	—
H10AR	18	(9) 0.148 x 1½	(9) 0.148 x 1½	—	1,050	490	285	—	905	420	285	—	
H10S	18	(8) 0.131 x 1½	(8) 0.131 x 1½"	(8) 0.131 x 2½	910	660	215	550	785	570	185	475	IBC, FL, LA
H10A-2	18	(9) 0.148 x 1½	(9) 0.148 x 1½	—	1,080	680	260	—	930	585	225	—	
H11Z	18	(6) 0.162 x 2½	(6) 0.162 x 2½	—	830	525	760	—	715	450	655	—	—
H14	18	(12) 0.131 x 1½	(13) 0.131 x 2½	—	1,275	725	205	—	1,050	480	245	—	IBC, FL, LA
		(12) 0.131 x 1½	(15) 0.131 x 2½	—	1,340	670	230	—	1,050	480	245	—	
TSP	16	(9) 0.148 x 1½	(6) 0.148 x 1½	—	755	310	190	—	650	265	160	—	FL
		(9) 0.148 x 1½	(6) 0.148 x 3	—	1,015	310	190	—	875	265	160	—	

1. See pp. 260–261 for Straps and Ties General Notes.
2. Allowable loads are for one anchor. A minimum rafter thickness of 2½" must be used when framing anchors are used on each side of the joist and on the same side of the plate (exception: connectors installed such that nails on opposite side don't interfere).
3. Allowable DF/SP uplift load for stud-to-bottom plate installation (see detail 15) is 390 lb. (H2.5A); 265 lb. (H2.5ASS); and 310 lb. (H8). For SPF/HF values, multiply these values by 0.86.
4. Allowable loads in the F_1 direction are not intended to replace diaphragm boundary members or cross-grain bending of the truss or rafter members.
5. When cross-grain bending or cross-grain tension cannot be avoided in the members, mechanical reinforcement to resist such forces shall be considered by the Designer.
6. Hurricane ties are shown on the outside of the wall for clarity and assume a minimum overhang of 3½". Installation on the inside of the wall is acceptable (see General Instructions for the Installer, note "p" on p. 18). For uplift continuous load path, install connectors in the same area (e.g., truss-to-plate connector and plate-to-stud connector) on the same side of the wall, unless detailed by designer. See technical bulletin T-C-HTIECON at **strongtie.com** for more information.
7. Southern pine allowable uplift loads for H10A = 1,340 lb. and for the H14 = 1,465 lb.
8. Refer to Simpson Strong-Tie® technical bulletin T-C-HTIEBEAR at **strongtie.com** for allowable bearing enhancement loads.
9. H10S can have the stud offset a maximum of 1" from the rafter (center to center) for a reduced uplift of 890 lb. (DF/SP) and 765 lb. (SPF).
10. H10S nails to plates are optional for uplift but required for lateral loads.
11. Some load values for the stainless-steel connectors shown here are lower than those for the carbon-steel versions. Ongoing test programs have shown this also to be the case with other stainless-steel connectors in the product line that are installed with nails. Visit **strongtie.com/corrosion** for updated information.
12. The allowable loads of stainless-steel connectors match carbon-steel connectors when installed with stainless-steel Strong-Drive® SCNR Ring-Shank Connector nails. For more information, refer to engineering letter L-F-SSNAILS at **strongtie.com**.
13. Allowable DF/SP/SPF uplift load for the H2.5A fastened to a 2x4 truss bottom chord and double top plates using (5) 0.131" x 1½" nails in the top plates and (3) 0.131" x 1½" nails in the lowest three flange holes into the truss bottom chord is 260 lb. (160).
14. For TSP installed stud to single plate see p. 276.
15. **Fasteners:** Nail dimensions in the table are listed diameter by length. See pp. 21–22 for fastener information.

Straps and Ties

MPBZ

Page R1.G

Moment Post Base

The patent-pending MPBZ is specifically designed to provide moment resistance for columns or posts. An innovative overlapping sleeve design encapsulates the post, helping to resist rotation around its base. It is available for 4x4, 6x6 and 8x8 posts. The MPBZ is ideal for outdoor structures, such as carports, fences and decks. Built-in stand-off tabs provide the required 1" stand-off to resist decay of the post while eliminating multiple parts and assembly. Additionally, the MPBZ is available in ZMAX® as the standard finish to meet exposure conditions in many environments.

Features:

- Internal top-of-concrete tabs
- 1" standoff tabs
- Additional holes provided to attach trim material
- Weep hole provided for water drainage

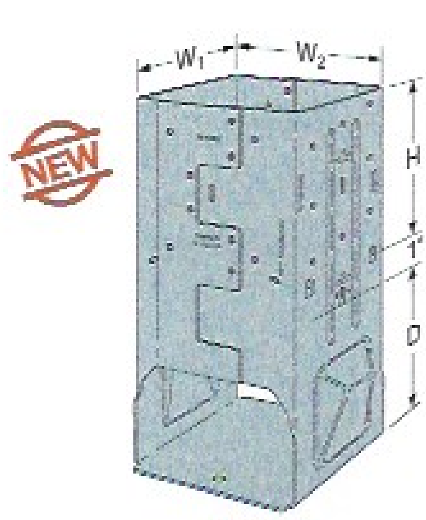
Material: 12 gauge

Finish: ZMAX coating

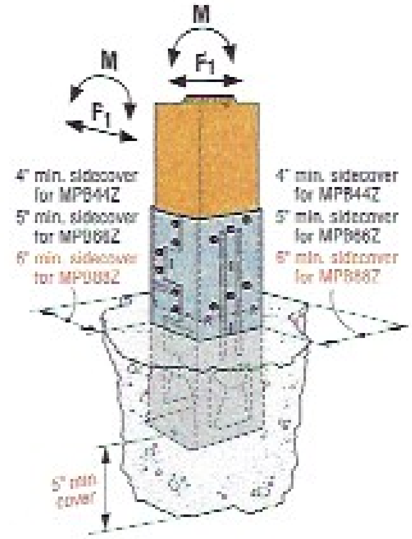
Installation:

- Use all specified fasteners; see General Notes.
- Install MPBZ before concrete is placed using embedment level indicators and form board attachment holes.
- Place post on tabs 1" above top of concrete.
- Install Strong-Drive SDS Heavy-Duty Connector screws, which are supplied with the MPBZ. (Lag screws will not achieve the same load.)
- Concrete level inside the part must not exceed ¼" above embedment line to allow for water drainage.
- Annual inspection of connectors used in outdoor application is advised. If significant corrosion is apparent or suspected, then the wood, fasteners and connectors should be evaluated by a qualified engineer or inspector.

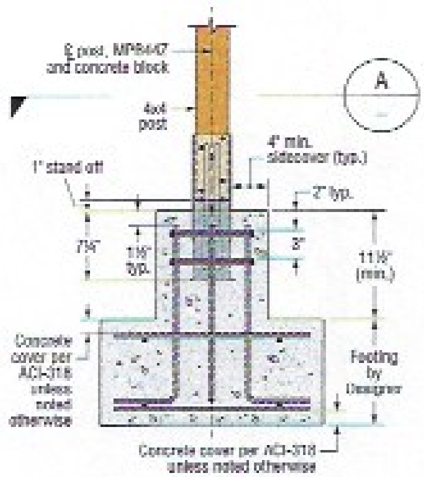
Codes: See p. 12 for Code Reference Key Chart



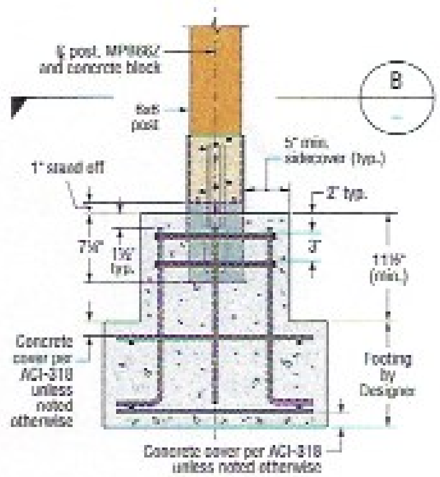
MPB88Z
(MPB44Z, MPB66Z similar)
U.S. Patent Pending



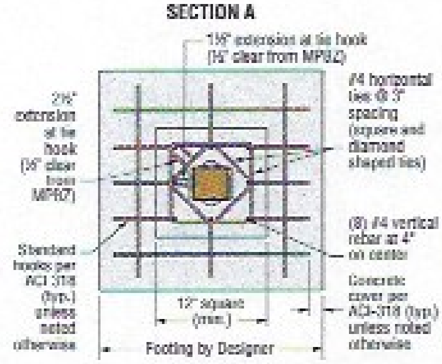
Typical MPB66Z
Non-Reinforced Installation
(others similar)



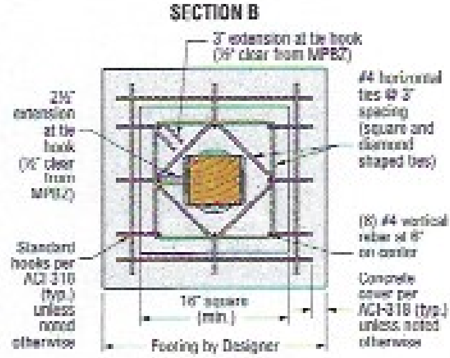
SECTION A



SECTION B



MPB44Z
Reinforced Concrete Footing
Footing (size and reinforcement) by Designer. Standard hook geometry in accordance with ACI 318 unless noted otherwise.



MPB66Z
Reinforced Concrete Footing
Footing (size and reinforcement) by Designer. Standard hook geometry in accordance with ACI 318 unless noted otherwise.

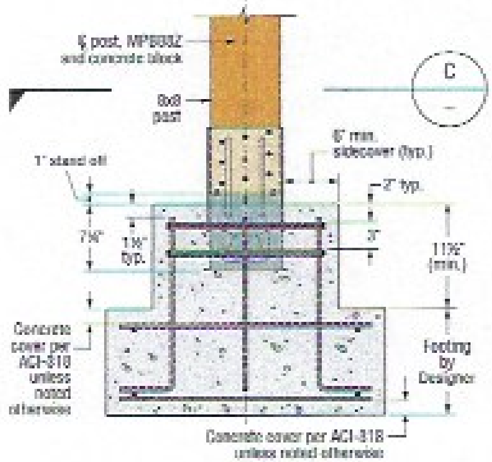
These reinforced MPBZ details are available on strongtie.com/mpbz.

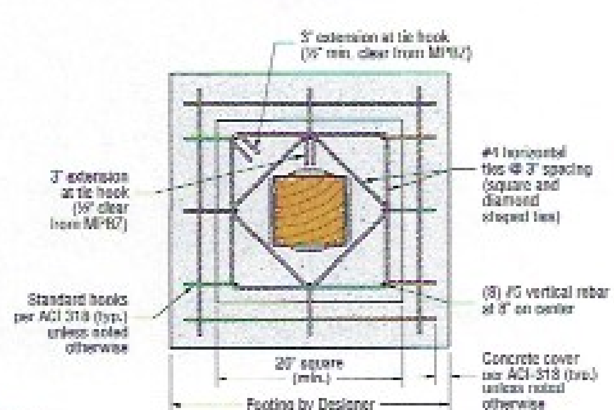
Moment Post Base (cont.)

These products are available with additional corrosion protection. For more information, see p. 15.

Model No.	Nominal Column Size	Dimensions (in.) W1 / W2	D	H	Strong-Drive® SDS Screws	Concrete Allowable Loads: Uplift Uncracked	Uplift Cracked	Lateral F_1 Uncracked	Lateral F_1 Cracked	Moment M (ft.-lb.) Uncracked	Moment M (ft.-lb.) Cracked	Wood Assembly Allowable Loads (DF/SP): Download (100)	Download (160)	Moment M (ft.-lb.) (160)	Rotational Stiffness (in.-lb./rad.)	Code Ref.
Non-Reinforced Concrete																
Wind and Seismic Design Category A&B																
MPB44Z	4x4	3 9/16	7 1/4	7 1/4	(16) 1/4" x 2 1/2"	4,900	3,820	1,750	1,225	1,350	985	6,240	6,410	1,540	1,245,000	IBC, FL, LA
MPB66Z	6x6	5 9/16	7 1/4	7 1/4	(24) 1/4" x 2 1/2"	5,815	5,815	3,545	2,405	2,680	1,875	9,360	10,855	3,730	2,405,000	
MPB88Z	8x8	7 9/16	7 1/4	7 1/4	(36) 1/4" x 3"	9,945	6,960	7,200	5,560	4,160	2910	15,120	17,585	4,525	5,500,000	—
Seismic Design Category C–F																
MPB44Z	4x4	3 9/16	7 1/4	7 1/4	(16) 1/4" x 2 1/2"	4,785	3,350	1,535	1,075	1,180	830	6,240	6,410	1,540	1,245,000	IBC, FL, LA
MPB66Z	6x6	5 9/16	7 1/4	7 1/4	(24) 1/4" x 2 1/2"	5,815	5,815	3,015	2,055	2,055	1,645	9,360	10,855	3,730	2,405,000	
MPB88Z	8x8	7 9/16	7 1/4	7 1/4	(36) 1/4" x 3"	7,420	6,100	6,965	4,875	3,470	2550	15,120	17,585	4,525	5,500,000	—
Reinforced Concrete																
Wind and Seismic Design Category A&B																
MPB44Z	4x4	3 9/16	7 1/4	7 1/4	(16) 1/4" x 2 1/2"	4,900	3,820	1,750	1,225	1,540	1,540	6,240	6,410	1,540	1,245,000	—
MPB66Z	6x6	5 9/16	7 1/4	7 1/4	(24) 1/4" x 2 1/2"	5,815	5,815	3,545	2,405	3,730	3,190	9,360	10,855	3,730	2,405,000	
MPB88Z	8x8	7 9/16	7 1/4	7 1/4	(36) 1/4" x 3"	9,945	6,960	7,200	5,560	4,525	4,525	15,120	17,585	4,525	5,500,000	
Seismic Design Category C–F																
MPB44Z	4x4	3 9/16	7 1/4	7 1/4	(16) 1/4" x 2 1/2"	4,785	3,350	1,535	1,075	1,540	1,540	6,240	6,410	1,540	1,245,000	—
MPB66Z	6x6	5 9/16	7 1/4	7 1/4	(24) 1/4" x 2 1/2"	5,815	5,815	3,015	2,110	3,350	2,795	9,360	10,855	3,730	2,405,000	
MPB88Z	8x8	7 9/16	7 1/4	7 1/4	(36) 1/4" x 3"	7,420	6,100	6,965	4,875	4,525	4,525	15,120	17,585	4,525	5,500,000	

1. Loads may not be increased for duration of load.
2. Higher download can be achieved by solidly packing grout in the 1" standoff area before installation of the post. Allowable download shall be based on either the wood post design or the concrete design calculated per code.
3. Concrete shall have a minimum compressive strength of f'_c = 2,500 psi.
4. Tabulated rotational stiffness accounts for the rotation of the base assembly attributable to deflection of the connector, fastener slip, and post deformation. Designer must account for additional deflection attributable to bending of the post.
5. To obtain LRFD values, multiply ASD seismic load values by 1.4 and wind load values by 1.67 (1.6 for 2012 IBC).
6. In accordance with IBC, Section 1613.1, detached one- and two-family dwellings in Seismic Design Category (SDC) C may use "Wind and SDC A&B" allowable loads.
7. Foundation dimensions are for anchorage only. Foundation design (size and reinforcement) by Designer.
8. Allowable load shall be the lesser of the wood assembly or concrete allowable load. To achieve full wood assembly allowable moment loads, additional concrete design and reinforcement by Designer is required.
9. For loading simultaneously in more than one direction, the allowable load must be evaluated using the following equation: (Design Uplift / Allowable Uplift, or Design Download / Allowable Download) + (Design Moment / Allowable Moment) + (Design Lateral / Allowable Lateral) ≤ 1.0.
10. To account for shrinkage up to 3%, multiply rotational stiffness by 0.75. Reduction may be linearly interpolated for shrinkage less than 3%.





MPB88Z
Reinforced Concrete Footing

Footing (size and reinforcement) by Designer. Standard hook geometry in accordance with ACI 318 unless noted otherwise.

Bases and Caps

AC/LPCZ/LCE/RTC

SIMPSON Strong-Tie

Page R1.1

Post Caps

The universal design of the LCE4 post cap provides high capacity while eliminating the need for rights and lefts. For use with 4x or 6x lumber. LPCZ — Adjustable design allows greater connection versatility.

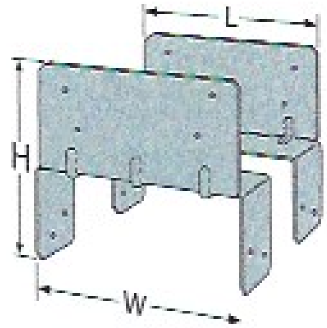
Material: LCE4 — 20 gauge;
AC, LPC4Z — 18 gauge;
LPC6Z — 16 gauge;
RTC — 14 gauge

Finish: Galvanized.
Some products available in ZMAX® coating and stainless steel.
See Corrosion Information, pp. 13–15.

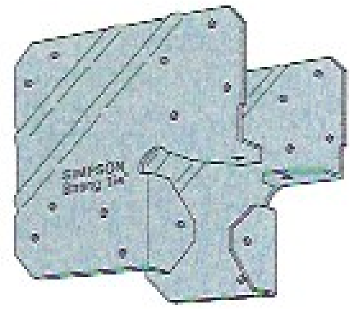
Installation:

- Use all specified fasteners, see General Notes
- Install all models in pairs. LPCZ — 2½" beams may be used if 0.148" x 1½" nails are substituted for 0.148" x 3" nails

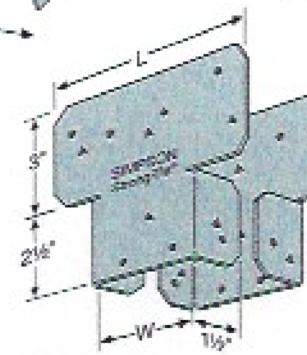
Codes: See p. 12 for Code Reference Key Chart



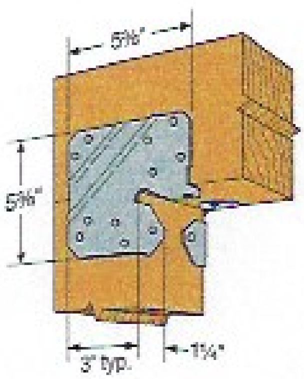
LPCZ



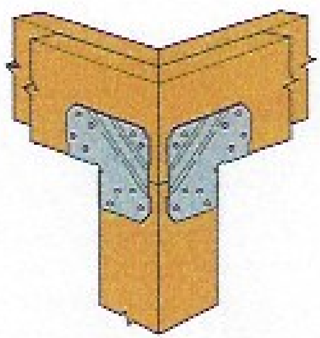
LCE4



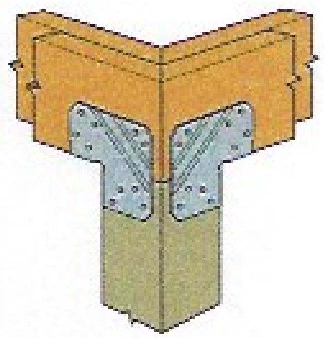
AC



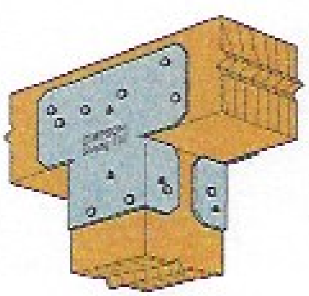
Typical LCE4 Installation
(for 4x or 6x lumber)



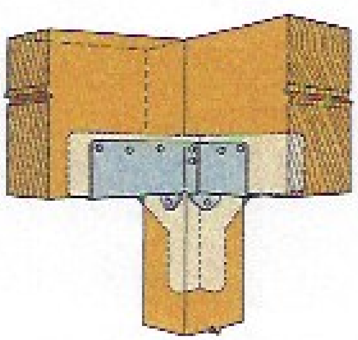
Typical LCE4 Corner Installation
(see note 7)



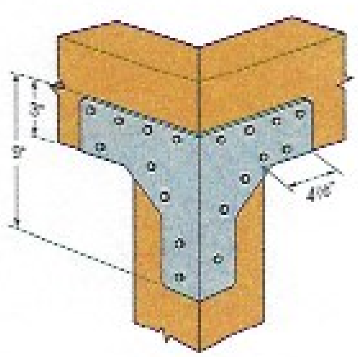
Typical LCE4Z Installation
(mitered corner)



Typical AC4 Installation



RTC44 Installation
(square cut)



RTC44 Installation
(mitered corner)

AC/LPCZ/LCE/RTC

SIMPSON Strong-Tie

Page R1.3

Post Caps (cont.)

These products are available with additional corrosion protection. For more information, see p. 15.

SS For stainless-steel fasteners, see p. 21.

SD Many of these products are approved for installation with Strong-Drive® SD Connector screws. See pp. 335–337 for more information.

Model No.	Dimensions (in.)		Min. / Max.	Total No. Fasteners (in.)		Allowable Loads (DF/SP) (160)		Code Ref.
	W	L		Beam	Post	Uplift	Lateral	
AC4	3⁹⁄₁₆	6½	Min.	(8) 0.162 x 3½	(8) 0.162 x 3½	1,745	1,610	IBC, FL, LA
	3⁹⁄₁₆	6½	Max.	(14) 0.162 x 3½	(14) 0.162 x 3½	2,490	1,475	
AC4RZ	4	7	Min.	(8) 0.162 x 3½	(8) 0.162 x 3½	1,745	1,610	
	4	7	Max.	(14) 0.162 x 3½	(14) 0.162 x 3½	2,490	2,075	
LCE4	—	5½	—	(14) 0.162 x 3½	(10) 0.162 x 3½	1,955	1,350	
AC6	5½	8½	Min.	(8) 0.162 x 3½	(8) 0.162 x 3½	1,665	1,565	
	5½	8½	Max.	(14) 0.162 x 3½	(14) 0.162 x 3½	2,815	2,075	
AC6RZ	6	9	Min.	(8) 0.162 x 3½	(8) 0.162 x 3½	1,665	1,565	
	6	9	Max.	(14) 0.162 x 3½	(14) 0.162 x 3½	3,055	2,450	
LPC4Z	3⁹⁄₁₆	3½	—	(8) 0.148 x 3	(8) 0.148 x 3	755	760	
LPC6Z	5⁹⁄₁₆	5½	—	(8) 0.148 x 3	(8) 0.148 x 3	920	885	

1. Allowable loads have been increased for wind or earthquake loading with no further increase allowed. Reduce where other loads govern.
2. Loads apply only when used in pairs.
3. LPCZ lateral load is in the direction parallel to the beam.
4. For minimum nailing quantity and load values, fill all round holes; for maximum nailing quantity and load values, fill all round and triangular holes.
5. Uplift loads do not apply to spliced conditions. Spliced conditions must be detailed by the Designer to transfer tension loads between spliced members by means other than the post cap.
6. LCE4 uplift load for mitered-corner conditions is 985 lb. (DF/SP) or 845 lb. (SPF). Lateral loads do not apply.
7. Structural composite lumber columns have sides that show either the wide face or the edges of the lumber strands/veneers known as the narrow face. Values in the tables reflect installation into the wide face. See technical bulletin T-C-SCLCLM at **strongtie.com** for load reductions resulting from narrow-face installations.
8. **Fasteners:** Nail dimensions in the table are listed diameter by length. See pp. 21–22 for fastener information.

Model No.	Dimensions (in.)		Total No. of Fasteners (in.)		DF/SP Uplift Loads			SPF Uplift Loads		
	W	L	Beam	Post	Side Beam	Main Beam	Post	Side Beam	Main Beam	Post
RTC44[1] (Mitered corner)	3⁹⁄₁₆	4⅜	(16) 0.162 x 3½	(10) 0.162 x 3½	900	900	1,800	775	775	1,550
RTC44[2] (Square cut)	3⁹⁄₁₆	4⅜	(16) 0.162 x 3½	(10) 0.162 x 3½	925	1,230	1,760	795	1,060	1,515
LCE4Z[1] (Mitered corner)	5⅝	5⅝	(14) 0.162 x 3½	(10) 0.162 x 3½	—	—	985	—	—	845

1. The allowable download for the mitered RTC44 and LCE4Z connection is limited to the bearing of the mitered beams on the post and shall be determined by the Designer.
2. The allowable download for the main beam in the square-cut RTC44 connection is limited to the bearing of the beam on the post and shall be determined by the Designer. The side beam allowable download is 1,170 lb.
3. The combined uplift loads applied to all the beams must not exceed the post allowable uplift load listed in the table.
4. Connectors must be installed in pairs to achieve listed loads.

Bases and Caps

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Phone: (816) 767-7222
Fax: (816) 767-8105

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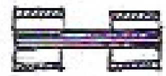
		Made By	DJP

Pillar @ 24' Header - P=3707# & M=16960#·' LVL 1¾"x18" (1.9E) w/ (4) 2x6 & 3/8" PLYWD



Patio End Header - l=13' w=4'(40psf)+2'(20psf)=200plf, R=1300#

$M = \frac{200plf(13')^2}{8} = 4225$#·' 3-2x10

Entry 'Eyebrow' Beam -

l=8.0' w=50psf(4')=200plf, R=800#, M=1600#·', 3-2x6

PACKARD ENGINEERING
21021 Oak Drive
Belton, MO 64012
Phone: (816) 767-7222
Fax: (816) 767-8105

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Foundation

trenched footings - assume $q_a = 1500psf$ (net)

$w_{max} = 16'(40psf) + 4'(70psf) + 20'(10psf) + 4'(150psf)2 = 2320 plf$
@Great Room East

$b_r = \frac{2320}{1500} = 1.55'$

R_{max} addl $= 260plf(12') = 3120^{\#}$ $b_r = \frac{3120^{\#}}{1500psf(6')} = 0.35$

$w = 4'(40psf) + 16'(10psf) + 4'(150psf) = 920plf$

$b_r = \frac{920plf}{1500psf} = 0.61' \rightarrow b_{r\,total} = 0.35' + 0.61' = 0.96'$

use 1.33' (minimum)

Patio footings @ Posts

$R_{max} = 5938^{\#}$ & $M = 1760^{\#'}$ $A_r = \frac{5938^{\#}}{1500psf} = 4sf < 1.50' \times 3' = 4.5sf$ OK.