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CLIENT	B+A Architecture	Page	TOC
		Date	12/19
JOB	Woodside Ridge Clubhouse NW Pryor Road & NW O'Brien Road Lee's Summit, MO	Job No.	BtA19005
		Made By	DJP

Calculations Table of Contents (Structural)

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

Dl J Packard
12-27-19

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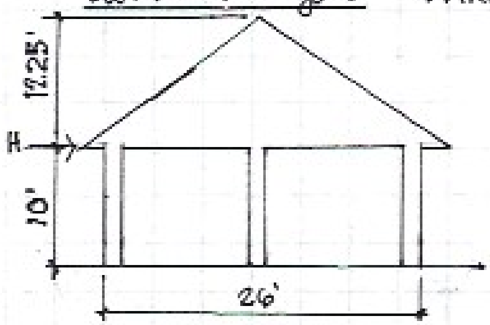
Lateral Analysis



Wind is Critical (vs. Seismic)

At Patio Roof - Transverse (Open)

Try Flagpole columns w/ $H_{t_{max}} = 10'$

$H = 30\,psf(12.25')8.5' = 3124^{\#}$

$H_{per\ column} = \frac{3124^{\#}}{3} = 1041^{\#}$

$M_{column} = 1041^{\#}(10') = 10413^{\#\cdot'}$

$S_r = \frac{10413^{\#\cdot'}(12''/')}{0.5(46000\,psi)} = 5.43\,in^3$

$\Delta_a = \frac{120''}{120} = 1''$

$I_r = \frac{1041^{\#}(120'')^3}{3(29\times10^6\,psi)1''} = 20.68\,in^4$ HSS6x6x1/4

Rafter Uplift at Bearing $= (-51\,psf + 10\,psf)1.33'(\frac{13'}{2} + 2') = -4625^{\#}$

Simpson H25A

Page L1.A

WIND LOADING ANALYSIS - Main Wind-Force Resisting System

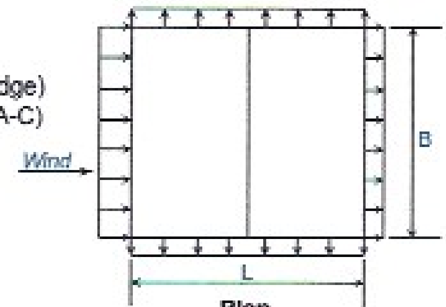
Per ASCE 7-10 Code for Enclosed or Partially Enclosed Buildings

Using Method 2: Analytical Procedure (Section 27) for Buildings of Any Height

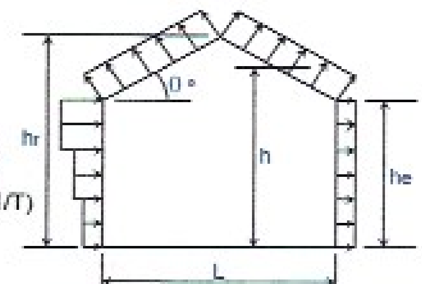
Job Name:	Woodside Ridge Clubhouse	Subject:	Patio Roof		
Job Number:		Originator:		Checker:	

Input Data:

Wind Direction =	Normal	(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 =	22.34	ft. (hr >= he)
Eave Height, he =	10.00	ft. (he <= hr)
Building Width =	26.00	ft. (Normal to Building Ridge)
Building Length =	15.00	ft. (Parallel to Building Ridge)
Roof Type =	Gable	(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 = 26 ft.
B = 15 ft.

Resulting Parameters and Coefficients:

Roof Angle, θ =	43.51	deg.	
Mean Roof Ht., h =	16.17	ft. (h = (hr+he)/2, for roof angle >10 deg.)	
Windward Wall Cp =	0.80	(Fig. 27.4-1)	
Leeward Wall Cp =	-0.35	(Fig. 27.4-1)	
Side Walls Cp =	-0.70	(Fig. 27-4.1)	
Windward Roof Cp =	-0.03	(Fig. 27-4.1)	(Condition #1)
Windward Roof Cp =	0.36	(Fig. 27-4.1)	(Condition #2)
Leeward Roof Cp =	-0.60	(Fig. 27-4.1)	
		(Fig. 27-4.1)	
+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 =	0.86	(Kh = Kz evaluated at z = h)		

Velocity Pressure: qz = 0.00256*Kz*Kzt*Kd*V^2*I (Eq. 27.3-1)

qh =	22.71	psf	qh = 0.00256*Kh*Kzt*Kd*V^2 (qz evaluated at z = h)		
Ratio h/B =	0.622	freq., f =	3.543	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)

Normal 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	2.71	27.69
	15.00	0.85	22.35	0.80	2.71	27.69
	20.00	0.90	23.75	0.80	3.66	28.64
For z = hr:	22.34	0.92	24.31	0.80	4.04	29.02
For z = he:	10.00	0.85	22.35	0.80	2.71	27.69
For z = h:	16.17	0.86	22.71	0.80	2.95	27.93
Leeward Wall	All	-	-	-0.35	-19.31	5.67
Side Walls	All	-	-	-0.70	-26.00	-1.02
Roof (windward) cond. 1	-	-	-	-0.03	-13.06	11.91
Roof (windward) cond. 2	-	-	-	0.36	-5.53	19.45
	-	-	-			
Roof (leeward)	-	-	-	-0.60	-24.07	0.91
	-	-	-			

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.

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 =	4.481	= (2*(LN(3600*f)))^(1/2)+0.577/(2*LN(3600*f))^(1/2) , Eq. 26.9-11
Q =	0.944	= (1/(1+0.63*((B+h)/Lz(bar))^0.63))^(1/2) , Eq. 26.9-8

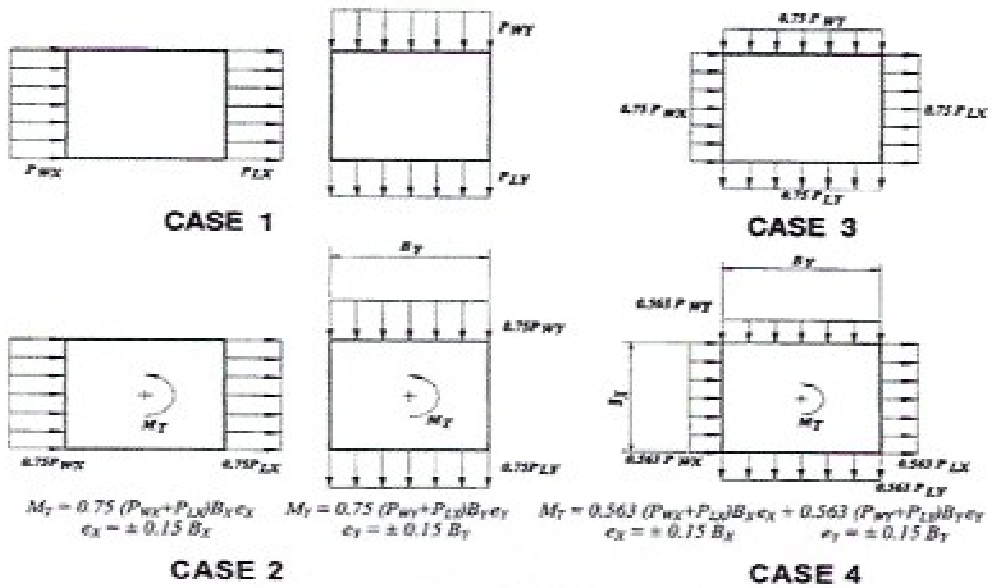
2: Calculation of G for Rigid Building

G = 0.896 = 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 =	0.282	= Ct*h^(3/4) , sec. (Approximate fundamental period)
f =	3.543	= 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.

H/TSP

SIMPSON Strong-Tie

Page LI.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.

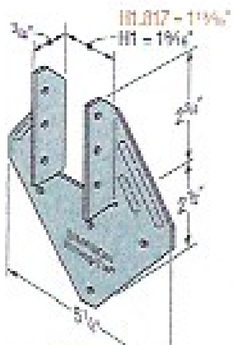
Material: See table

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.**

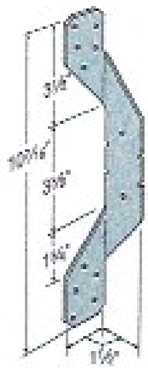
Installation:

- 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.

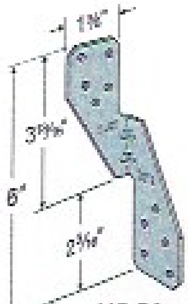
Codes: See p. 12 for Code Reference Key Chart



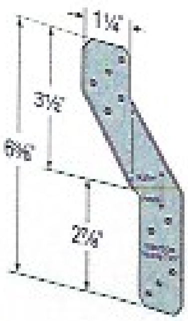
H1
(H1.81Z similar)



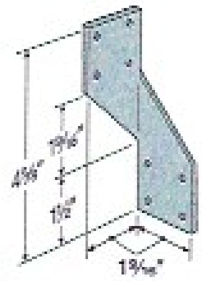
H2A
(H2ASS similar)



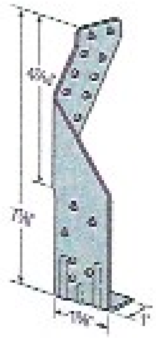
H2.5A
(H2.5ASS similar)



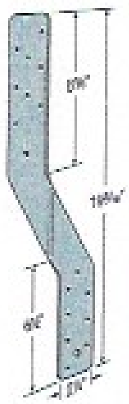
H2.5T



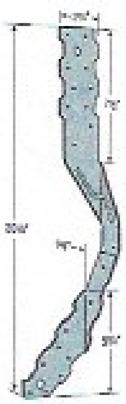
H3



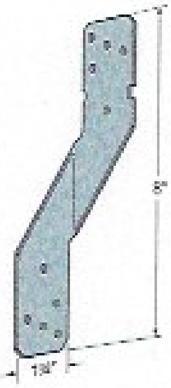
TSP



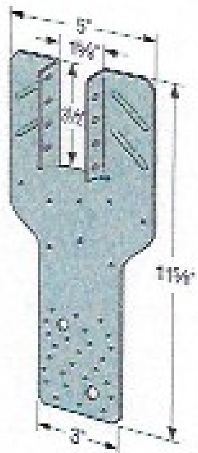
H6



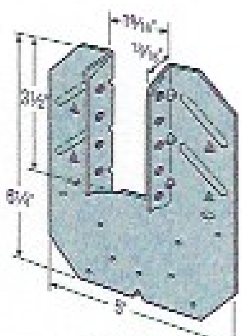
H7Z



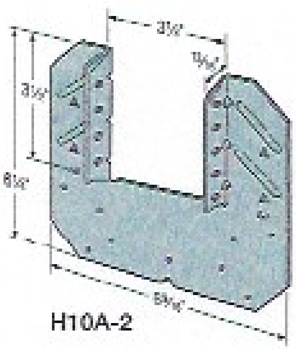
H8



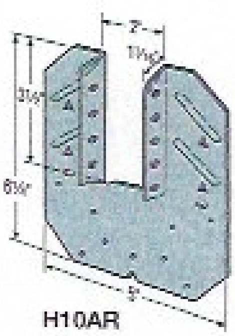
H10S



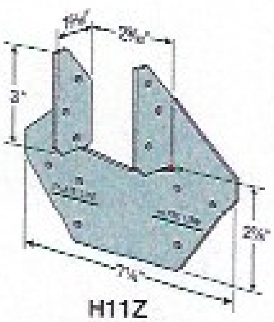
H10A
(H10ASS similar)



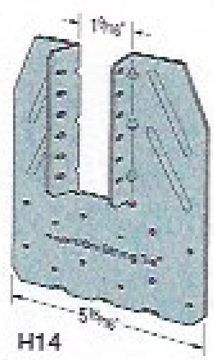
H10A-2



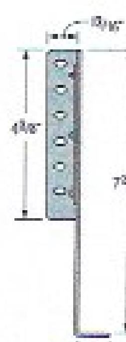
H10AR



H11Z



H14



H14 Profile

Straps and Ties

H/TSP

SIMPSON Strong-Tie®

Page L.I.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	425	440	165	370	IBC, FL, LA
H1.81Z	18	(6) 0.131 x 1½	(4) 0.131 x 2½		350	305	195	330	300	290	150	260	—
H2A	18	(5) 0.131 x 1½	(2) 0.131 x 1½	(5) 0.131 x 1½	575	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	385	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	205	—	
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	285	—	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

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CLIENT BtA

JOB Woodside Ridge Clubhouse

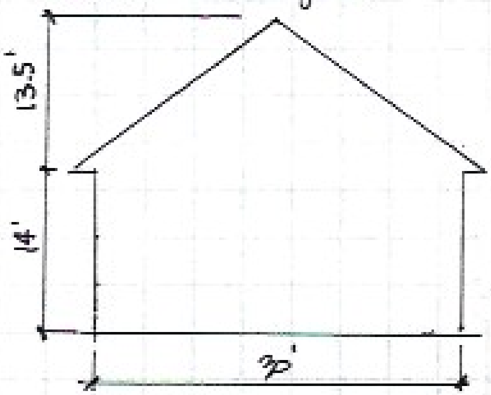
Page L2

Date 10/19

Job No. BtA19005

Made By

Lateral Analysis



Critical at Back/Gable endwall w/ Sliding Doors

$H = 30\text{psf}\,(17'(13.5')0.5 + 15'(7')) = 6593^{\#}$

Use LVL shear frame @ Sliding Doors & Drag shear to North Restroom Wall

$v = \dfrac{6593^{\#}}{2(2.25') + 16'} = 322\,\text{plf} \rightarrow$ 19/32" APA (40/20) Sheathing nailed w/ 10d @ 6" o.c.

WIND LOADING ANALYSIS - Main Wind-Force Resisting System

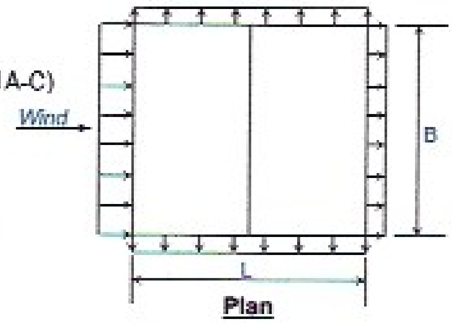
Per ASCE 7-10 Code for Enclosed or Partially Enclosed Buildings

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

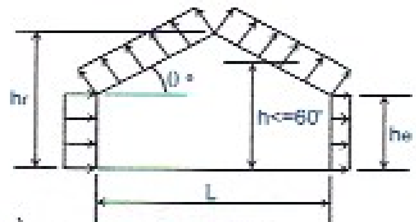
Job Name:	Woodside Ridge Clubhouse	Subject:	Main Roof		
Job Number:		Originator:		Checker:	

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 =	27.50	ft. (hr >= he)
Eave Height, he =	14.00	ft. (he <= hr)
Building Width =	30.00	ft. (Normal to Building Ridge)
Building Length =	29.00	ft. (Parallel to Building Ridge)
Roof Type =	Gable	(Gable or Monoslope)
Topo. Factor, Kzt =	1.00	(Sect. 26.8 & Figure 26.8-1)
Direct. Factor, Kd =	0.85	(Table 26.6)
Enclosed? (Y/N)	Y	(Sect. 26.2 & Table 26.11-1)
Hurricane Region?	N	



Plan



Elevation

Resulting Parameters and Coefficients:

Roof Angle, θ =	41.99	deg.
Mean Roof Ht., h =	20.75	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 =	0.91	(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 = 23.93 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.

Page L 2.B

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	0.56	9.09	17.71	Zone 1	-0.45	-15.08	-6.46
Zone 2	0.21	0.72	9.33	Zone 2	-0.69	-20.82	-12.20
Zone 3	-0.43	-14.60	-5.98	Zone 3	-0.37	-13.16	-4.55
Zone 4	-0.37	-13.16	-4.55	Zone 4	-0.45	-15.08	-6.46
Zone 5	---	---	---	Zone 5	0.40	5.26	13.88
Zone 6	---	---	---	Zone 6	-0.29	-11.25	-2.63
Zone 1E	0.69	12.20	20.82	Zone 1E	-0.48	-15.79	-7.18
Zone 2E	0.27	2.15	10.77	Zone 2E	-1.07	-29.91	-21.30
Zone 3E	-0.53	-16.99	-8.38	Zone 3E	-0.53	-16.99	-8.38
Zone 4E	-0.48	-15.79	-7.18	Zone 4E	-0.48	-15.79	-7.18
Zone 5E	---	---	---	Zone 5E	0.61	10.29	18.91
Zone 6E	---	---	---	Zone 6E	-0.43	-14.60	-5.98

*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. = N.A. ft.

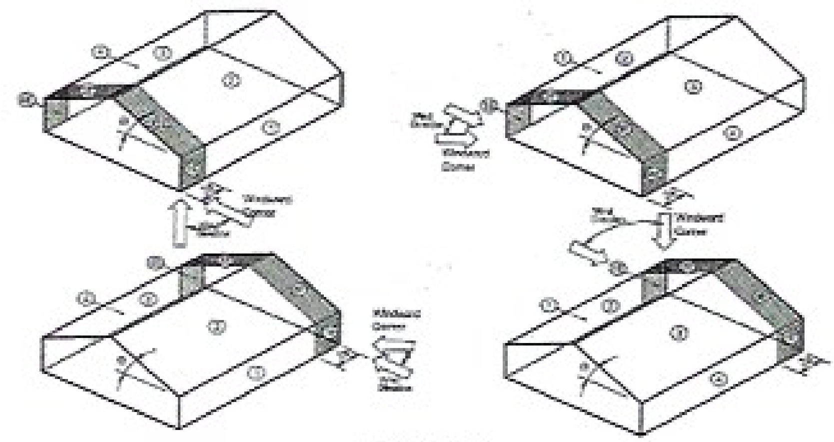
For Case B when GCpf is neg. in Zones 2/2E: Zones 2/2E dist. = 14.50 ft.

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

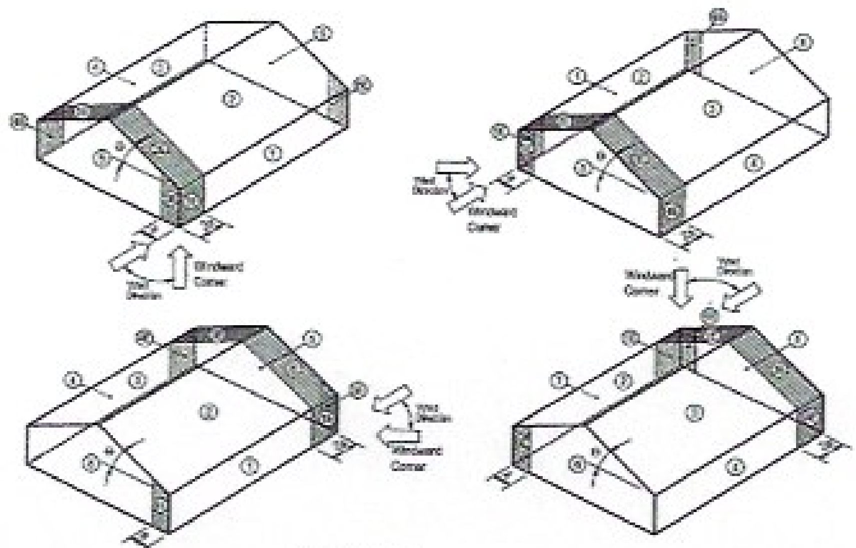
MWFRS Wind Load for Load Case A, Torsional Case				MWFRS Wind Load for Case B, Torsional Case			
Surface	GCpf	p = Net Pressure (psf)		Surface	GCpf	p = Net Pressure (psf)	
		(w/ +GCpi)	(w/ -GCpi)			(w/ +GCpi)	(w/ -GCpi)
Zone 1T	---	2.27	4.43	Zone 1T	---	-3.77	-1.62
Zone 2T	---	0.18	2.33	Zone 2T	---	-5.21	-3.05
Zone 3T	---	-3.65	-1.50	Zone 3T	---	-3.29	-1.14
Zone 4T	---	-3.29	-1.14	Zone 4T	---	-3.77	-1.62
Zone 5T	---	---	---	Zone 5T	---	1.32	3.47
Zone 6T	---	---	---	Zone 6T	---	-2.81	-0.66

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'

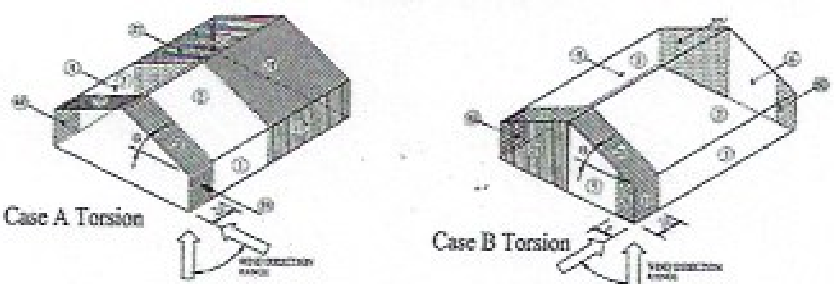


Load Case A



Load Case B

Basic Load Cases



Case A Torsion

Case B Torsion

Transverse Direction

Longitudinal Direction

Torsional Load Cases

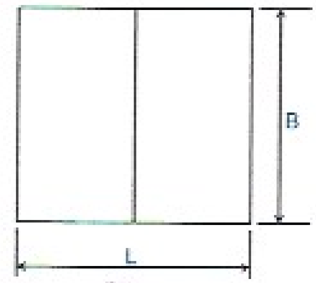
WIND LOADING ANALYSIS - Wall Components and Cladding

Per ASCE 7-10 Code for Buildings of Any Height
Using Part 1 & 3: Analytical Procedure (Section 30.4 & 30.6)

Job Name:	Woodside Ridge 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 =	27.50	ft. (hr >= he)
Eave Height, he =	14.00	ft. (he <= hr)
Building Width =	30.00	ft. (Normal to Building Ridge)
Building Length =	29.00	ft. (Parallel to Building Ridge)
Roof Type =	Gable	(Gable or Monoslope)
Topo. Factor, Kzt =	1.00	(Sect. 26.8 & Figure 26.8-1)
Direct. Factor, Kd =	0.85	(Table 26.6)
Enclosed? (Y/N)	Y	(Sect. 26.6-1 & Figure 26.11-1)
Hurricane Region?	N	
Component Name =	Wall	(Girt, Siding, Wall, or Fastener)
Effective Area, Ae =	19	ft.^2 (Area Tributary to C&C)



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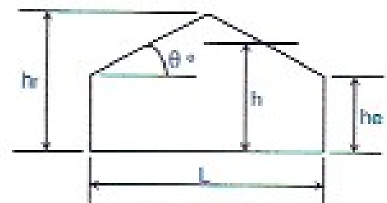


Elevation

Resulting Parameters and Coefficients:

Roof Angle, θ =	41.99	deg.
Mean Roof Ht., h =	20.75	ft. (h = (hr+he)/2, for roof angle >10 deg.)

Wall External Pressure Coefficients, GCp:

GCp Zone 4 Pos. =	0.95	(Fig. 30.4-1)
GCp Zone 5 Pos. =	0.95	(Fig. 30.4-1)
GCp Zone 4 Neg. =	-1.05	(Fig. 30.4-1)
GCp Zone 5 Neg. =	-1.30	(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 =	0.91	(Kh = Kz evaluated at z = h)

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

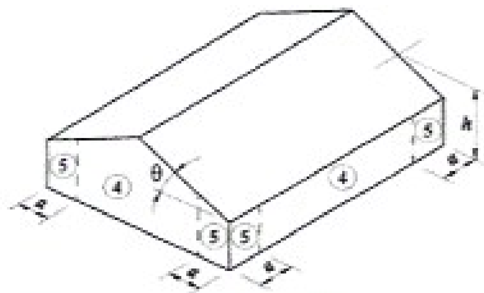
qh = 23.93 psf   qh = 0.00256*Kh*Kzt*Kd*V^2 (qz evaluated at z = h)

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)

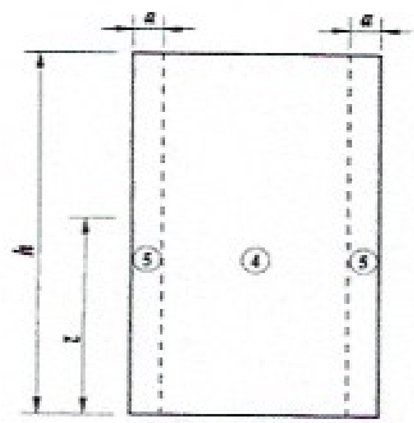
Wind Load Tabulation for Wall Components & Cladding							
Component	z (ft.)	Kh	qh (psf)	p = Net Design Pressures (psf)			
				Zone 4 (+)	Zone 4 (-)	Zone 5 (+)	Zone 5 (-)
Wall	0	0.91	23.93	27.06	-29.45	27.06	-35.45
	15.00	0.91	23.93	27.06	-29.45	27.06	-35.45
	20.00	0.91	23.93	27.06	-29.45	27.06	-35.45
	25.00	0.91	23.93	27.06	-29.45	27.06	-35.45
For z = hr:	27.50	0.91	23.93	27.06	-29.45	27.06	-35.45
For z = he:	14.00	0.91	23.93	27.06	-29.45	27.06	-35.45
For z = h:	20.75	0.91	23.93	27.06	-29.45	27.06	-35.45

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 ELEVATION

Wall Zones for Buildings with h > 60 ft.

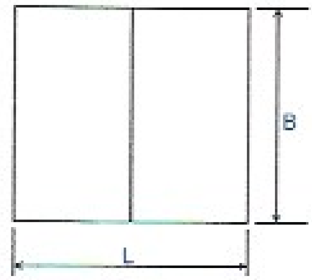
WIND LOADING ANALYSIS - Roof Components and Cladding

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:	Woodside Ridge 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 =	27.50	ft. (hr >= he)
Eave Height, he =	14.00	ft. (he <= hr)
Building Width =	30.00	ft. (Normal to Building Ridge)
Building Length =	29.00	ft. (Parallel to Building Ridge)
Roof Type =	Gable	(Gable or Monoslope)
Topo. Factor, Kzt =	1.00	(Sect. 26.8 & Figure 26.8-1)
Direct. Factor, Kd =	0.85	(Table 26.6)
Enclosed? (Y/N)	Y	(Sect. 26.6-1 & Figure 26.11-1)
Hurricane Region?	N	
Component Name =	Joist	(Purlin, Joist, Decking, or Fastener)
Effective Area, Ae =	20	ft.^2 (Area Tributary to C&C)
Overhangs? (Y/N)	Y	(if used, overhangs on all sides)



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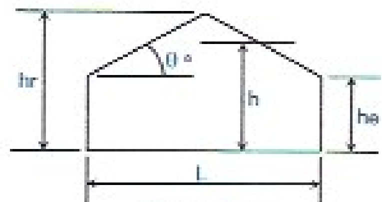


Elevation

Resulting Parameters and Coefficients:

Roof Angle, θ =	41.99	deg.
Mean Roof Ht., h =	20.75	ft. (h = (hr+he)/2, for roof angle >10 deg.)

Roof External Pressure Coefficients, GCp:

GCp Zone 1-3 Pos. =	0.87	(Fig. 30.4-2A, 30.4-2B, and 30.4-2C)
GCp Zone 1 Neg. =	-0.94	(Fig. 30.4-2A, 30.4-2B, and 30.4-2C)
GCp Zone 2 Neg. =	-1.94	(Fig. 30.4-2A, 30.4-2B, and 30.4-2C)
GCp Zone 3 Neg. =	-1.94	(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 =	0.91	(Kh = Kz evaluated at z = h)

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

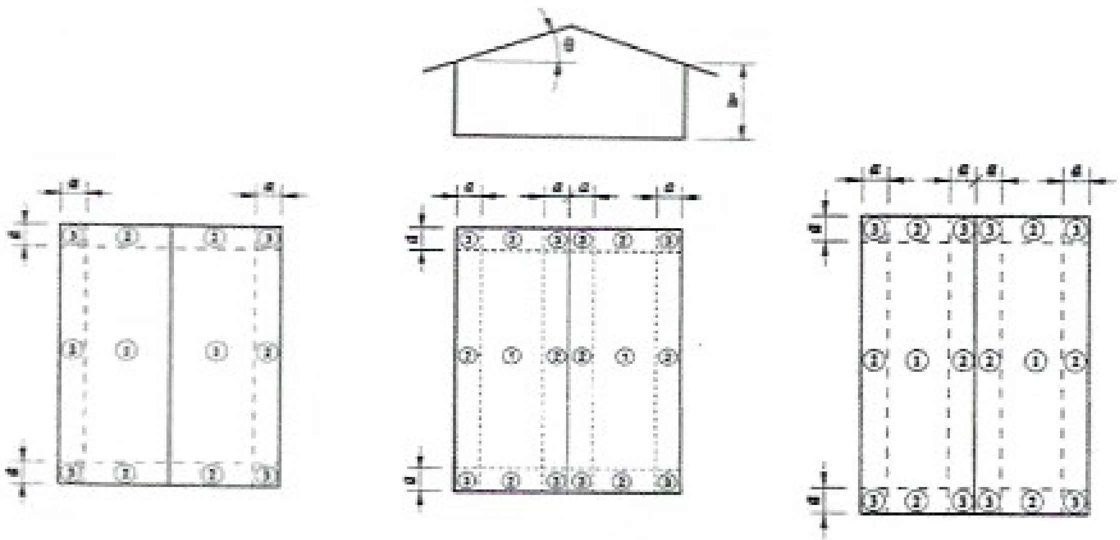
qh = 23.93 psf   qh = 0.00256*Kh*Kzt*Kd*V^2 (qz evaluated at z = h)

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	0.91	23.93	25.13	-26.80	-50.73	-50.73
	15.00	0.91	23.93	25.13	-26.80	-50.73	-50.73
	20.00	0.91	23.93	25.13	-26.80	-50.73	-50.73
	25.00	0.91	23.93	25.13	-26.80	-50.73	-50.73
For z = hr:	27.50	0.91	23.93	25.13	-26.80	-50.73	-50.73
For z = he:	14.00	0.91	23.93	25.13	-26.80	-50.73	-50.73
For z = h:	20.75	0.91	23.93	25.13	-26.80	-50.73	-50.73

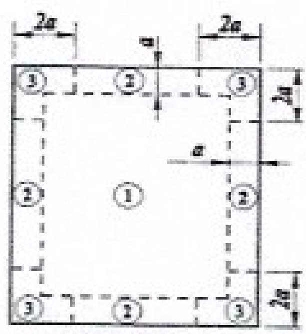
Notes: 1. (+) and (-) signs signify wind pressures acting toward & away from respective surfaces.
2. Width of Zone 2 (edge), 'a' = 3.00 ft.
3. Width of Zone 3 (corner), 'a' = 3.00 ft.
4. For monoslope roofs with θ <= 3 degrees, use Fig. 30.4-2A for 'GCp' values with 'qh'.
5. For buildings with h > 60' 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°)



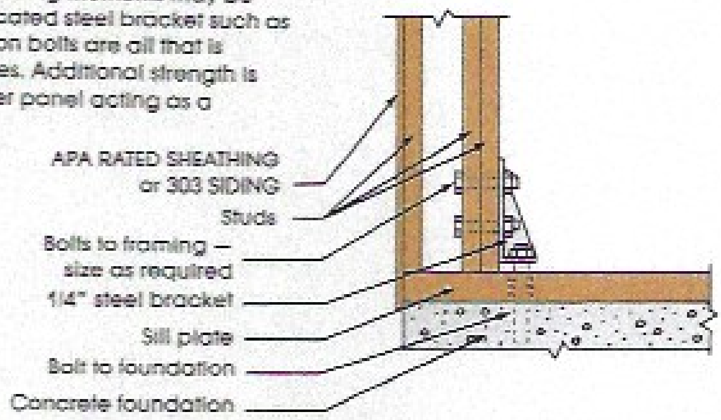
ROOF PLAN

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

Page L2.3

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

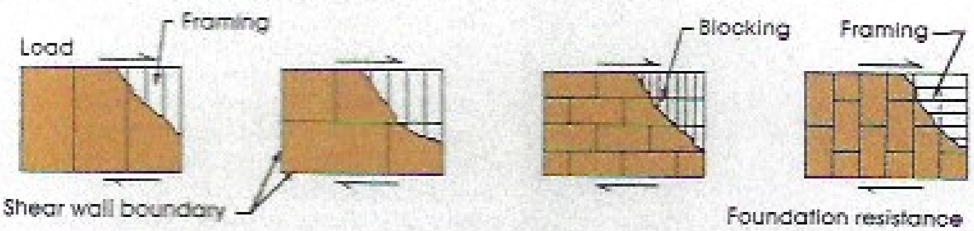


Table 23 Recommended Shear (pounds per foot) for APA Panel Shear Walls with Framing of Douglas Fir, Larch, or Southern Pine[a] for Wind or Seismic Loading[b]

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	230[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[e]	530[d]	10d	260	380	490[e]	640
	7/16			240[d]	350[d]	450[e]	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.

W335

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CLIENT B&A
Page R1
Date 10/19
JOB Woodside Ridge Clubhouse
Job No. B&A19005
Made By DJP

Roof Framing

Say LL = 20 psf
DL = 15 psf

Rafters – $l_{max} = 16'$ 2x10 @ 16" o.c.
$l = 15'$ " " " "
$l = 13'$ 2x10 @ 24" o.c.
$l = 12'$ 2x10 @ 24" o.c. or 2x8 @ 16"

Ridge Beams – $l_{main\ room} = 28'$, w = 15'(35 psf) = 525 plf, R = 7350#, M = 51450#-' **3-1 3/4" x 18" LVL (1.9E)**

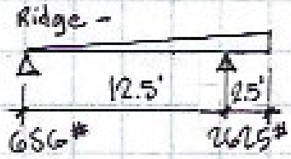
$l_{patio} = 15'$, w = 13'(35 psf) = 455 plf, R = 3413#, M = 12797#-' **2-1 3/4" x 11 1/4" LVL (1.9E)**

$l_{Restroom} = 15'$, w = 16'(35 psf) = 560 plf, R = 4200#, M = 15750#-' **2-1 3/4" x 11 1/4" LVL (1.9E)**

$l_{Equipment} = 8'$, w = 12'(35 psf) = 420 plf, R = 1680#, M = 3360#-' **2-2x12**

$l_{mech.} = 8'$, w = 16'(35 psf) = 560 plf, R = 2240#, M = 4480#-' **2-2x12**

Dormer Beams –
Ridge –



w_{max} = 12.5'(35 psf) = 438 plf → W = $\frac{438\text{ plf}(15')}{2}$ = 3281#

−M = 438 plf (2.5') 1.25' = −1369#-' **1-2x12**

$l = 12'$, w = 4'(35 psf) = 140 plf, R = 840#, M = 2520#-' **1-2x12** ok

Page R1.A

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

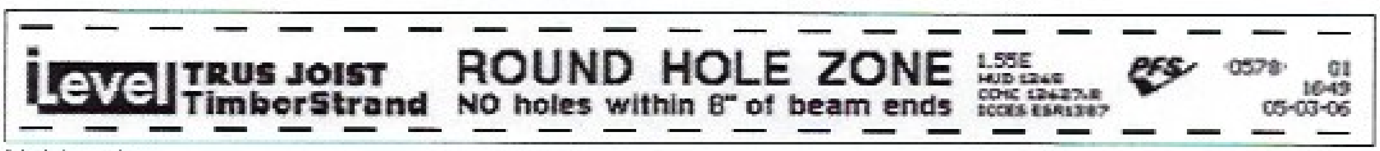
Grade	Width	Design Property	Depth 4⅜"	5½"	5½" Plank Orientation	7¼"	8⅝"	9¼"	9½"	11¼"	11⅞"	14"	16"	18"	20"
TimberStrand® LSL															
1.3E	3½"	Moment (ft-lbs)	1,735	2,685	1,780	4,550	6,335	7,240		10,520					
		Shear (lbs)	4,085	5,135	1,925	6,765	8,050	8,635		10,500					
		Moment of Inertia (in.⁴)	24	49	20	111	187	231		415					
		Weight (plf)	4.5	5.5	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	10,470	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	2[11]/[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	13,065	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.

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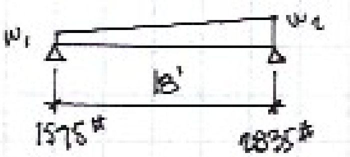
Roof Framing (cont'd)

Dormer Beams (cont'd)

Valley -

12 / 9 / 15

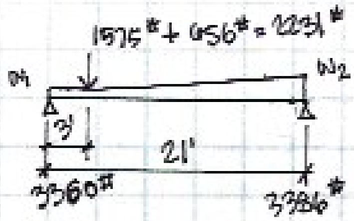


$w_1 = 1'(35psf) = 35plf$, $w_2 = 13'(35psf) = 455plf$

$M = 0.1283(3780^{\#})18' + 35plf(18')^2/8 = 10147^{\#\cdot\prime}$

$w_e = 250.5plf$, $\Delta_a = \frac{216''}{240} = 0.9$, $I_r = \frac{5(250.5)(216)^4}{12(384)1.9\times10^6(0.9)} = 346in^4$

1-1¾" x 14" LVL (1.9E)



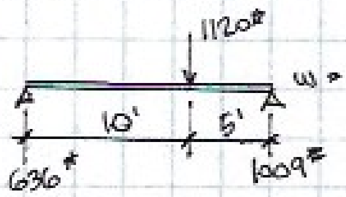
$w_e = \frac{3386^{\#}}{10.5'} = 322plf$, $M_e = 17750^{\#\cdot\prime}$,

$\Delta_a = \frac{252''}{240} = 1.05''$, $I_r = \frac{5(322)(252)^4}{12(384)1.9\times10^6(1.05)} = 706in^4$

2-1¾" x 14" LVL (1.9E)

Header $l = 8'$, $w = 8'(35psf) = 280plf$, $R = 1120^{\#}$, $M = 2240^{\#\cdot\prime}$, 2-2x10

Trimmer



$w = 35plf$ $M = \frac{636^{\#} + 286^{\#}}{2}(10') = 4610^{\#\cdot\prime}$, 3-2x10

'Truss' Beams (Top Chord) $l = 9'$, $w = 3'(35psf) = 105plf$, $R = 473^{\#} + 2'(105plf) = 683^{\#}$, $M = 1063^{\#\cdot\prime}$ 2-2x8 (P.T.)

Patio Roof Ends $l = 14'$, $w = (6.5' + 2')(35psf) = 298plf$, $R = 2086^{\#}$, $M = 7301^{\#\cdot\prime}$ 3-2x12 P.T.

Drag Beam $l = 8'$, $w = 8'(35psf) = 280plf$, $R = 1120^{\#}$, $M = 2240^{\#\cdot\prime}$ 3-2x10 P.T.

$l = 7'$, $w = 4'(35psf) = 140plf$, $R = 490^{\#}$, $M = 858^{\#\cdot\prime}$ 3-2x8 P.T.

LSSR/LSU

SIMPSON Strong-Tie

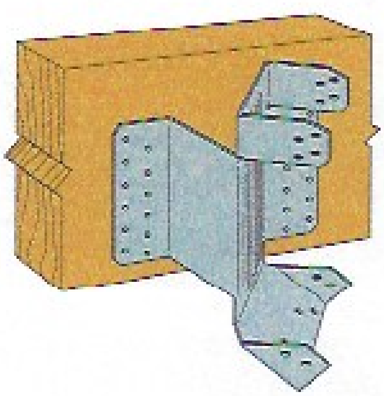
Page R2.A

Slopeable/Skewable Rafter Hanger (cont.)

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

Actual Joist Width (in.)	Model No.	Ga.	Dimensions (in.) W	Dimensions (in.) H	Dimensions (in.) A	Fasteners (in.) Face	Fasteners (in.) Joist	DF/SP Species Header Uplift (160)	DF/SP Floor (100)	DF/SP Roof Snow (115)	DF/SP Roof Const. (125)	SPF/HF Species Header Uplift (160)	SPF/HF Floor (100)	SPF/HF Roof Snow (115)	SPF/HF Roof Const. (125)	Code Ref.
Sloped Only Hangers																
1¾	LSSR1.81Z	18	1¹³⁄₁₆	8⁹⁄₁₆	4⅛	(14) 0.148 x 2½	(12) 0.148 x 1½	510	1,415	1,565	1,565	440	1,215	1,345	1,345	IBC
2 to 2⁵⁄₁₆	LSSR2.1Z	18	2⅛	8⁹⁄₁₆	4⅛	(14) 0.148 x 2½	(12) 0.148 x 1½	510	1,415	1,565	1,565	440	1,215	1,345	1,345	IBC
2⁵⁄₁₆	LSSR2.37Z	18	2⅜	8⁹⁄₁₆	4⅛	(14) 0.148 x 2½	(12) 0.148 x 1½	510	1,415	1,565	1,565	440	1,215	1,345	1,345	IBC
2½	LSSR2.56Z	18	2⁹⁄₁₆	8⁹⁄₁₆	4⅛	(14) 0.148 x 2½	(12) 0.148 x 1½	510	1,415	1,565	1,565	440	1,215	1,345	1,345	IBC
3	LSSR210-2Z	16	3⅛	8¹³⁄₁₆	5⅛	(22) 0.162 x 2½	(18) 0.162 x 2½	695	2,365	2,365	2,365	600	2,035	2,035	2,035	IBC
3½	LSSR410Z	16	3⁹⁄₁₆	8¹³⁄₁₆	5⅛	(22) 0.162 x 2½	(18) 0.162 x 2½	695	2,365	2,365	2,365	600	2,035	2,035	2,035	IBC
Dbl 2	LSU4.12	14	4⅛	9	2¼	(24) 0.162 x 3½	(16) 0.148 x 1½	1,150	3,215	3,700	4,020	990	2,785	3,200	3,480	—
Dbl 2⅛	LSU4.28	14	4¼	9	2⅜	(24) 0.162 x 3½	(16) 0.148 x 1½	1,150	3,215	3,700	4,020	990	2,785	3,200	3,480	—
Dbl 2⁵⁄₁₆	LSU3510-2	14	4⅝	8⅞	3⅜	(24) 0.162 x 3½	(16) 0.148 x 1½	1,150	3,215	3,700	4,020	990	2,785	3,200	3,480	—
Dbl 2½	LSU5.12	14	5⅛	9	2¼	(24) 0.162 x 3½	(16) 0.148 x 1½	885	3,215	3,700	3,785	760	2,785	3,200	3,280	—
Skewed Hangers or Sloped and Skewed Hangers																
1¾	LSSR1.81Z	18	1¹³⁄₁₆	8⁹⁄₁₆	4⅛	(13) 0.148 x 2½	(9) 0.148 x 1½	510	1,060	1,205	1,205	440	910	1,035	1,035	IBC
2 to 2⁵⁄₁₆	LSSR2.1Z	18	2⅛	8⁹⁄₁₆	4⅛	(13) 0.148 x 2½	(9) 0.148 x 1½	510	1,060	1,205	1,205	440	910	1,035	1,035	IBC
2⁵⁄₁₆	LSSR2.37Z	18	2⅜	8⁹⁄₁₆	4⅛	(13) 0.148 x 2½	(9) 0.148 x 1½	510	1,060	1,205	1,205	440	910	1,035	1,035	IBC
2½	LSSR2.56Z	18	2⁹⁄₁₆	8⁹⁄₁₆	4⅛	(13) 0.148 x 2½	(9) 0.148 x 1½	510	1,060	1,205	1,205	440	910	1,035	1,035	IBC
3	LSSR210-2Z	16	3⅛	8¹³⁄₁₆	5⅛	(20) 0.162 x 2½	(13) 0.162 x 2½	695	1,810	1,810	1,810	600	1,555	1,555	1,555	IBC
3½	LSSR410Z	16	3⁹⁄₁₆	8¹³⁄₁₆	5⅛	(20) 0.162 x 2½	(13) 0.162 x 2½	695	1,810	1,810	1,810	600	1,555	1,555	1,555	IBC
Dbl 2	LSU4.12[4]	14	4⅛	9	2¼	(24) 0.162 x 3½	(16) 0.148 x 1½	1,150	2,300	2,300	2,300	990	1,990	1,990	1,990	—
Dbl 2⅛	LSU4.28[4]	14	4¼	9	2⅜	(24) 0.162 x 3½	(16) 0.148 x 1½	1,150	2,300	2,300	2,300	990	1,990	1,990	1,990	—
Dbl 2⁵⁄₁₆	LSU3510-2[4]	14	4⅝	8⅞	3⅜	(24) 0.162 x 3½	(16) 0.148 x 1½	1,150	2,300	2,300	2,300	990	1,990	1,990	1,990	—
Dbl 2½	LSU5.12[4]	14	5⅛	9	2¼	(24) 0.162 x 3½	(16) 0.148 x 1½	885	1,790	1,790	1,790	760	1,550	1,550	1,550	—

1. Uplift loads have been increased for earthquake or wind loading with no further increase allowed. Reduce where other loads govern.
2. For slope-only installations, the four triangle holes may be filled for an allowable roof download of 3,015 lb. for LSSR 160A.
3. Roof loads are 125% of floor loads unless limited by other criteria.
4. LSU3510-2, LSU4.12, LSU4.28, and LSU5.12 skew options must be factory-ordered.
5. Minimum 11" joist height for LSU3510-2, LSU4.12, LSU5.12; 9¼" for all others.
6. On the acute side of the skewed LSSR hangers, fill obround holes only.
7. **Fasteners:** Nail dimensions in the table are listed diameter by length. See pp. 21–22 for fastener information.



The LSU5.12 must be factory-skewed 0° to 45°. It may be field-sloped to 45°.
(LSU4.12, LSU4.28 and LSU3510-2 similar)

Hanger shown skewed right.

I-Joist, Glulam and Structural Composite Lumber Connectors

Face-Mount Hangers — I-Joists, Glulam and SCL

Page R2.B

Codes: See p. 12 for Code Reference Key Chart.

Actual Joist Size (in.)	Model No.	Carried Member: Glulam	SCL	I-Joist	Web Stiff Reqd.	Dimensions (in.): W	H	B	Min./Max.	Fasteners (in.): Face	Joist	Allowable Loads DF/SP Species Header: Uplift (160)	Floor (100)	Snow (115)	Roof (125)	SPF/HF Species Header: Floor (100)	Snow (115)	Roof (125)	Code Ref.
1½ x 9½	U210			•	✓	1 9/16	7 13/16	2	—	(6) 0.148 x 3	(6) 0.148 x 1½	990	1,220	1,380	1,480	1,050	1,185	1,275	
	MIU1.56/9			•	—	1 9/16	8 13/16	2½	—	(16) 0.162 x 3½	(2) 0.148 x 1½	230	2,305	2,615	2,820	1,980	2,245	2,425	
1½ x 11⅞	U210			•	✓	1 9/16	7 13/16	2	—	(6) 0.148 x 3	(6) 0.148 x 1½	990	1,220	1,380	1,480	1,050	1,185	1,275	
	MIU1.56/11			•	—	1 9/16	11 1/16	2½	—	(20) 0.162 x 3½	(2) 0.148 x 1½	230	2,880	3,135	3,135	2,475	2,695	2,695	
1¾ x 5½	HU1.81/5		•		—	1 13/16	5⅜	2½	Min.	(12) 0.162 x 3½	(4) 0.148 x 1½	610	1,785	2,015	2,165	1,540	1,735	1,865	
									Max.	(16) 0.162 x 3½	(6) 0.148 x 1½	915	2,380	2,685	2,890	2,050	2,315	2,490	
1¾ x 7¼	HU7		•		—	1 13/16	6 15/16	2½	Min.	(12) 0.162 x 3½	(4) 0.148 x 1½	610	1,785	2,015	2,165	1,540	1,735	1,865	
									Max.	(16) 0.162 x 3½	(6) 0.148 x 1½	1515	2,380	2,685	2,890	2,050	2,315	2,490	
1¾ x 9½	IUS1.81/9.5			•	—	1¾	9½	2	—	(8) 0.148 x 3	—	70	950	1,080	1,165	815	925	1,000	
	HU9		•	•	✓	1 13/16	9 1/16	2½	Min.	(18) 0.162 x 3½	(6) 0.148 x 1½	915	2,680	3,020	3,250	2,305	2,605	2,800	
									Max.	(24) 0.162 x 3½	(10) 0.148 x 1½	1,795	3,570	4,030	4,335	3,075	3,470	3,735	
	HUS1.81/10		•		—	1 13/16	8¾	3	—	(30) 0.162 x 3½	(10) 0.162 x 3½	2,675	5,135	5,295	5,400	4,415	5,105	5,195	
	HUCQ1.81/9-SDS		•		—	1 13/16	9	3	—	(8) ¼" x 1¾" SDS	(4) ¼" x 1¾" SDS	1,310	2,000	2,300	2,500	1,440	1,655	1,800	
	MIU1.81/9		•	•	—	1 13/16	8 13/16	2½	—	(16) 0.162 x 3½	(2) 0.148 x 1½	230	2,305	2,615	2,820	1,980	2,245	2,425	
1¾ x 11⅞	IUS1.81/11.88			•	—	1¾	11⅞	2	—	(10) 0.148 x 3	—	70	1,185	1,345	1,455	1,020	1,160	1,250	
	MIU1.81/11		•	•	—	1 13/16	11 1/16	2½	—	(20) 0.162 x 3½	(2) 0.148 x 1½	230	2,880	3,135	3,135	2,475	2,695	2,695	
	HUS1.81/10		•		—	1 13/16	8¾	3	—	(30) 0.162 x 3½	(10) 0.162 x 3½	2,675	5,135	5,295	5,400	4,705	5,105	5,195	
	HU11		•	•	✓	1 13/16	11 1/16	2½	Min.	(22) 0.162 x 3½	(6) 0.148 x 1½	915	3,275	3,695	3,970	2,820	3,180	3,425	
									Max.	(30) 0.162 x 3½	(10) 0.148 x 1½	1,795	4,465	4,705	4,810	3,845	4,340	4,600	
	HUCQ1.81/11-SDS		•		—	1 13/16	11	3	—	(10) ¼" x 1¾" SDS	(4) ¼" x 1¾" SDS	1,310	2,500	2,875	3,125	1,800	2,070	2,250	
1¾ x 14	IUS1.81/14		•	•	—	1¾	14	2	Min.	(12) 0.148 x 3	—	70	1,420	1,615	1,745	1,220	1,390	1,500	
									Max.	(14) 0.148 x 3	—	70	1,660	1,805	1,805	1,425	1,550	1,550	
	MIU1.81/14		•	•	—	1 13/16	13 13/16	2½	—	(22) 0.162 x 3½	(2) 0.148 x 1½	230	3,170	3,595	3,875	2,725	3,090	3,335	
	HUS1.81/10		•		—	1 13/16	8¾	3	—	(30) 0.162 x 3½	(10) 0.162 x 3½	2,675	5,135	5,295	5,400	4,705	5,105	5,195	IBC, FL, LA
	U14		•	•	✓	1 9/16	10⅛	2	—	(14) 0.162 x 3½	(6) 0.148 x 1½	970	2,015	2,285	2,465	1,735	1,965	2,120	
	HU14		•	•	✓	1 13/16	13 11/16	2½	Min.	(28) 0.162 x 3½	(8) 0.148 x 1½	1,515	4,165	4,420	4,505	3,580	4,050	4,335	
									Max.	(36) 0.162 x 3½	(14) 0.148 x 1½	1,795	5,055	5,275	5,420	4,615	5,000	5,130	
	HUCQ1.81/11-SDS		•		—	1 13/16	11	3	—	(10) ¼" x 1¾" SDS	(4) ¼" x 1¾" SDS	1,310	2,500	2,875	3,125	1,800	2,070	2,250	
1¾ x 16	IUS1.81/16			•	—	1¾	16	2	Min.	(14) 0.148 x 3	—	70	1,660	1,805	1,805	1,425	1,555	1,555	
									Max.	(16) 0.148 x 3	—	70	1,805	1,805	1,805	1,555	1,555	1,555	
	MIU1.81/16			•	—	1 13/16	15 13/16	2½	—	(24) 0.162 x 3½	(2) 0.148 x 1½	230	3,455	3,920	4,045	2,970	3,370	3,480	
1¾ x 18	MIU1.81/18			•	—	1 13/16	17 13/16	2½	—	(26) 0.162 x 3½	(2) 0.148 x 1½	230	3,745	4,070	4,045	3,220	3,460	3,480	
2 x 9½	IUS2.06/9.5			•	—	2 1/16	9½	2	—	(8) 0.148 x 3	—	70	950	1,080	1,165	815	925	1,000	
	HU2.1/9			•	✓	2⅛	9 1/16	2½	—	(14) 0.162 x 3½	(6) 0.148 x 1½	915	2,085	2,350	2,530	1,795	2,025	2,180	
2 x 11⅞	IUS2.06/11.88			•	—	2 1/16	11⅞	2	—	(10) 0.148 x 3	—	70	1,185	1,345	1,455	1,020	1,160	1,250	
	MIU2.1/11			•	✓	2⅛	11 1/16	2½	—	(20) 0.162 x 3½	(2) 0.148 x 1½	230	2,880	3,135	3,135	2,475	2,695	2,695	
	HU2.1/11			•	✓	2⅛	11	2½	—	(16) 0.162 x 3½	(6) 0.148 x 1½	915	2,380	2,685	2,890	2,050	2,315	2,490	
2 x 14	IUS2.06/14			•	—	2 1/16	14	2	Min.	(12) 0.148 x 3	—	70	1,420	1,615	1,745	1,220	1,390	1,500	
									Max.	(14) 0.148 x 3	—	70	1,660	1,805	1,805	1,425	1,555	1,555	
	MIU2.1/11			•	✓	2⅛	11 1/16	2½	—	(20) 0.162 x 3½	(2) 0.148 x 1½	230	2,880	3,135	3,135	2,475	2,695	2,695	
	HU2.1/11			•	✓	2⅛	11	2½	—	(16) 0.162 x 3½	(6) 0.148 x 1½	915	2,380	2,685	2,890	2,050	2,315	2,490	
2 x 16	IUS2.06/16			•	—	2 1/16	16	2	Min.	(14) 0.148 x 3	—	70	1,660	1,805	1,805	1,425	1,555	1,555	
									Max.	(16) 0.148 x 3	—	70	1,805	1,805	1,805	1,555	1,555	1,555	
	HU2.1/11			•	✓	2⅛	11	2½	—	(16) 0.162 x 3½	(6) 0.148 x 1½	915	2,380	2,685	2,890	2,050	2,315	2,490	
2 5/16 x 9½	IUS2.06/9.5			•	—	2 1/16	9½	2	—	(8) 0.148 x 3	—	70	950	1,080	1,165	815	925	1,000	
	HU2.1/9			•	✓	2⅛	9 1/16	2½	—	(14) 0.162 x 3½	(6) 0.148 x 1½	915	2,085	2,350	2,530	1,795	2,025	2,180	
2 5/16 x 11⅞	IUS2.06/11.88			•	—	2 1/16	11⅞	2	—	(10) 0.148 x 3	—	70	1,185	1,345	1,455	1,020	1,160	1,250	
	MIU2.1/11			•	✓	2⅛	11 1/16	2½	—	(20) 0.162 x 3½	(2) 0.148 x 1½	230	2,880	3,135	3,135	2,475	2,695	2,695	
	HU2.1/11			•	✓	2⅛	11	2½	—	(16) 0.162 x 3½	(6) 0.148 x 1½	915	2,380	2,685	2,890	2,050	2,315	2,490	

See footnotes on p. 150.

SIMPSON Strong-Tie

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CCQ/ECCQ

Column Caps

This product is preferable to similar connectors because of (a) easier installation, (b) higher loads, (c) lower installed cost, or a combination of these features.

Column caps provide a strong connection for column-beam combinations. This design uses Strong-Drive® SDS Heavy-Duty Connector screws to provide faster installation and provides a greater net section area of the column compared to bolts. The SDS screws provide for a lower profile compared to standard through bolts.

Material: CCQ3, ECCQ3, CCQ4, CCQ4.62, ECCQ4, ECCQ4.62, CCQ6, ECCQ6 — 7 gauge; all others — 3 gauge

Finish: Simpson Strong-Tie gray paint; available in HDG and stainless steel; CCOQ and ECCOQ — no coating

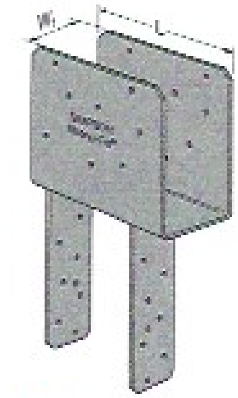
Installation:

- Install ¼" x 2½" Strong-Drive SDS Heavy-Duty Connector screws, which are provided with the column cap. (Lag screws will not achieve the same load.) Install stainless-steel Strong-Drive screws with stainless-steel connectors.
- CCOQ and ECCOQ column caps only (no straps) may be ordered for field-welding to pipe or other columns. Dimensions are same as CCQ and ECCQ. Weld by Designer.
- For rough-cut lumber sizes, provide dimensions. An optional W_2 dimension may be specified with any column size given. (Note that the W_2 dimension on straps rotated 90° is limited by the W_1 dimension.)

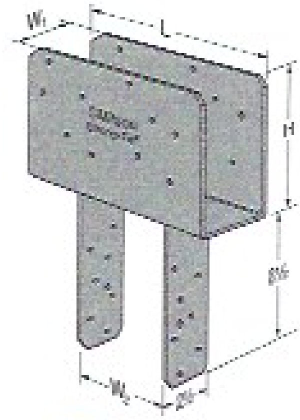
Options:

- For end conditions, specify ECCQ.
- Straps may be rotated 90° where $W_1 > W_2$ and for CCQ5-6.
- Other custom column caps are available. Contact Simpson Strong-Tie.

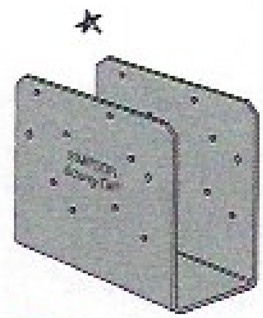
Codes: See p. 12 for Code Reference Key Chart



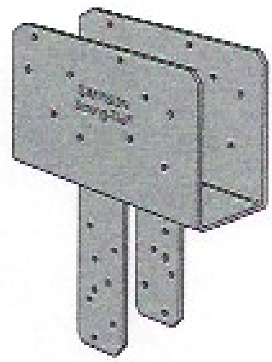
ECCQ46SDS2.5



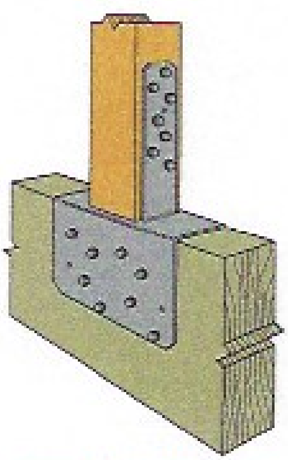
CCQ46SDS2.5



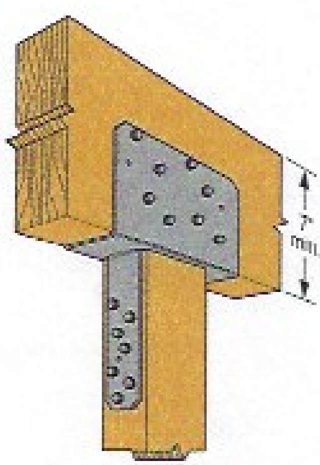
CCOQ4-SDS2.5
(no coating)



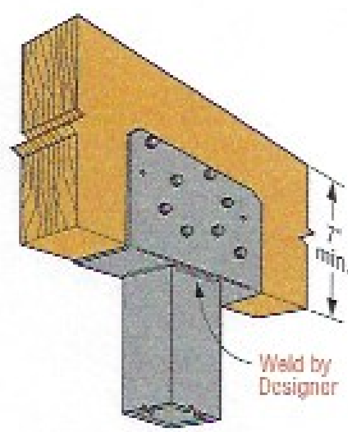
Optional CCQ with Straps Rotated 90°



Inverted CCQ44SDS2.5 Post-to-Beam Installation



Typical CCQ46SDS2.5 Installation



CCOQ Installation on Steel Column

CCQ/ECCQ

SIMPSON Strong-Tie

Page R2.0

Column Caps (cont.)

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

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

	Model No.	Beam Width (in.)	Dimensions (in.) W1	W2	L CCQ	L ECCQ	H	No. of ¼" x 2½" SDS Screws Beam CCQ	Beam ECCQ	Post	Allowable Loads (DF/SP) CCQ Uplift (160)	CCQ Down (100)	ECCQ Uplift (160)	ECCQ Down (100)	Code Ref.
SS	CCQ3-4SDS2.5	3⅛	3¼	3⅝	11	8½	7	16	14	14	5,370	16,980	3,465	6,125	IBC, FL, LA
SS	CCQ3-6SDS2.5	3⅛	3¼	5½	11	8½	7	16	14	14	5,370	21,485	3,465	10,740	
SS	CCQ44SDS2.5	3½	3⅝	3⅝	11	8½	7	16	14	14	5,370	19,020	3,785	7,655	
SS	CCQ46SDS2.5	3½	3⅝	5½	11	8½	7	16	14	14	6,785	24,065	3,785	12,030	
SS	CCQ48SDS2.5	3½	3⅝	7½	11	8½	7	16	14	14	6,785	24,065	3,785	16,405	
	CCQ4.62-3.62SDS	4⅝	4¾	3⅝	11	8½	7	16	14	14	5,370	23,390	3,785	9,845	
	CCQ4.62-4.62SDS	4⅝	4¾	4⅝	11	8½	7	16	14	14	5,370	30,070	3,785	12,655	
	CCQ4.62-5.50SDS	4⅝	4¾	5½	11	8½	7	16	14	14	6,785	30,940	3,785	15,470	
SS	CCQ5-4SDS2.5	5⅛	5¼	3⅝	11	8½	7	16	14	14	5,370	26,635	4,040	11,210	
SS	CCQ5-6SDS2.5	5⅛	5¼	5½	11	8½	7	16	14	14	6,785	28,190	5,355	17,615	
SS	CCQ5-8SDS2.5	5⅛	5¼	7½	11	8½	7	16	14	14	6,785	35,235	5,355	24,025	
SS	CCQ64SDS2.5	5¼, 5½	5½	3⅝	11	8½	7	16	14	14	5,370	28,585	3,785	12,030	
SS	CCQ66SDS2.5	5¼, 5½	5½	5½	11	8½	7	16	14	14	6,785	33,275	3,785	18,905	
SS	CCQ68SDS2.5	5¼, 5½	5½	7½	11	8½	7	16	14	14	6,785	37,815	3,785	25,780	
SS	CCQ6-7.13SDS2.5	5¼, 5½	5½	7⅛	11	8½	7	16	14	14	6,785	37,815	3,785	24,490	
SS	CCQ74SDS2.5	6¾	6⅞	3⅝	11	8½	7	16	14	14	5,370	33,490	4,040	15,355	
SS	CCQ76SDS2.5	6¾	6⅞	5½	11	8½	7	16	14	14	6,785	37,125	5,355	24,130	
	CCQ77SDS2.5	6¾	6⅞	6⅞	11	8½	7	16	14	14	6,785	48,265	5,355	29,615	
	CCQ78SDS2.5	6¾	6⅞	7½	11	8½	7	16	14	14	6,785	48,265	5,355	32,905	
SS	CCQ7.1-4SDS2.5	7	7⅛	3⅝	11	8½	7	16	14	14	5,370	34,730	4,040	18,375	
SS	CCQ7.1-6SDS2.5	7	7⅛	5½	11	8½	7	16	14	14	6,785	38,500	5,355	28,875	
	CCQ7.1-7.1SDS2.5	7	7⅛	7⅛	11	8½	7	16	14	14	6,785	57,750	5,355	36,750	
	CCQ7.1-8SDS2.5	7	7⅛	7½	11	8½	7	16	14	14	6,785	52,500	5,355	39,375	
	CCQ84SDS2.5	7½	7⅝	3⅝	11	8½	7	16	14	14	6,785	37,210	5,355	16,405	
	CCQ86SDS2.5	7½	7⅝	5½	11	8½	7	16	14	14	6,785	41,250	5,355	25,780	
	CCQ88SDS2.5	7½	7⅝	7½	11	8½	7	16	14	14	6,785	51,565	5,355	35,155	
	CCQ94SDS2.5	8¾	8⅞	3⅝	11	8½	7	16	14	14	6,785	47,545	5,355	19,905	
	CCQ96SDS2.5	8¾	8⅞	5½	11	8½	7	16	14	14	6,785	48,125	5,355	31,280	
	CCQ98SDS2.5	8¾	8⅞	7½	11	8½	7	16	14	14	6,785	62,565	5,355	42,655	
	CCQ106SDS2.5	9¼	9⅜	5½	11	8½	7	16	14	14	6,785	52,250	5,355	32,655	

CCOQ/ECCOQ Model No. (No Legs)
CCOQ3-SDS2.5 ECCOQ3-SDS2.5
CCOQ4-SDS2.5 ECCOQ4-SDS2.5
CCOQ4.62-SDS2.5 ECCOQ4.62-SDS2.5 ★
CCOQ5-SDS2.5 ECCOQ5-SDS2.5
CCOQ6-SDS2.5 ECCOQ6-SDS2.5
CCOQ7-SDS2.5 ECCOQ7-SDS2.5
CCOQ7.12-SDS2.5 ECCOQ7.12-SDS2.5
CCOQ8-SDS2.5 ECCOQ8-SDS2.5
CCOQ9-SDS2.5 ECCOQ9-SDS2.5
CCOQ10-SDS2.5 ECCOQ10-SDS2.5

1. Uplift loads have been increased for earthquake or wind loading with no further increase allowed. Reduce where other loads govern.
2. Downloads shall be reduced where limited by capacity of the post.
3. 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.
4. Spliced conditions must be detailed by the Designer to transfer tension loads between spliced members by means other than the column cap.
5. Column sides are assumed to be aligned in the same vertical plane as the beam sides. CCQ4.62 models assume a minimum 3½"-wide post.
6. 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.
7. Beam depth must be a minimum of 7".
8. For 5¼" engineered lumber, use 5½" models.
9. CCOQ and ECCOQ welded to a steel column will achieve maximum load listed as CCQ and ECCQ. The steel column width shall match the beam width. Weld by Designer.

LTP4/LTP5/A34/A35

SIMPSON Strong-Tie®

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Framing Angles and Plates

The larger LTP5 spans subfloor at the top of the blocking or rim board. The embossments enhance performance.

The LTP4 lateral tie plate transfers shear forces for top plate-to-rim board or blocking connections. Nail holes are spaced to prevent wood splitting for single and double top-plate applications. May be installed over plywood sheathing.

The A35 angle's exclusive bending slot allows instant, accurate field bends for all two- and three-way ties. Balanced, completely reversible design permits the A35 to secure a great variety of connections.

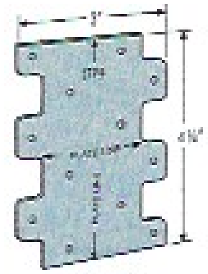
Material: LTP4/LTP5 — 20 gauge; all others — 18 gauge

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

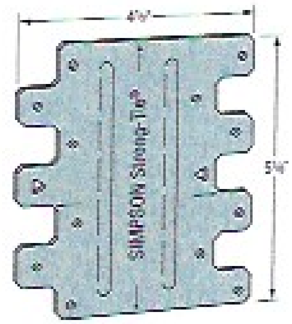
Installation:

- Use all specified fasteners; see General Notes
- A35 — Bend one time only

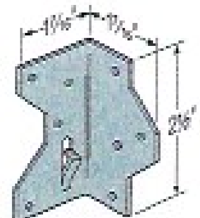
Codes: See p. 12 for Code Reference Key Chart



LTP4



LTP5



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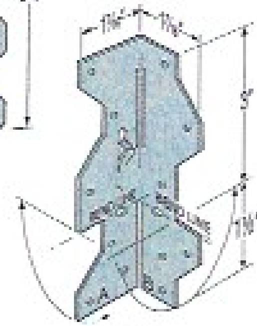

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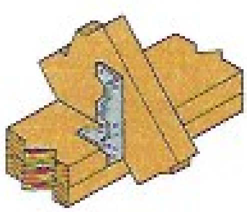
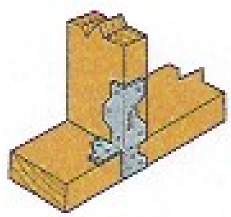
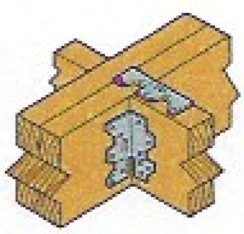


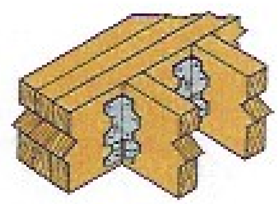
Joists to Plate with A Leg Inside



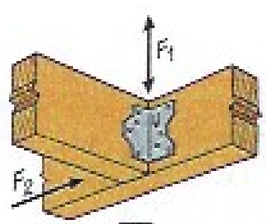
Studs to Plate with B Leg Outside



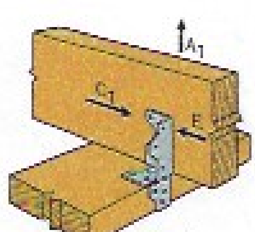
Joists to Beams



Ceiling Joists to Beam



1 A34



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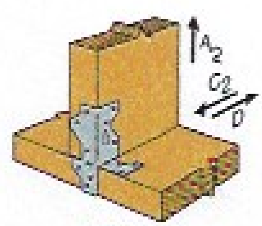

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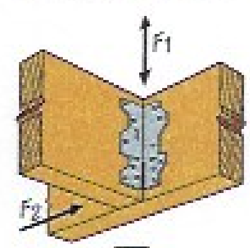

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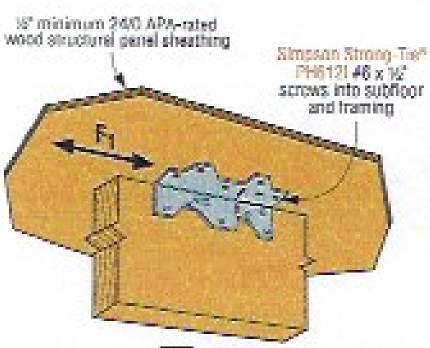

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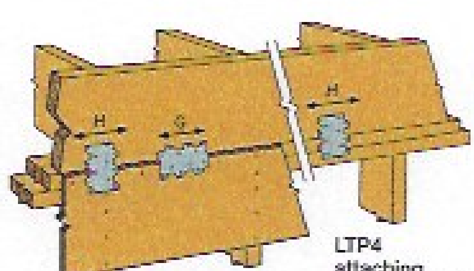
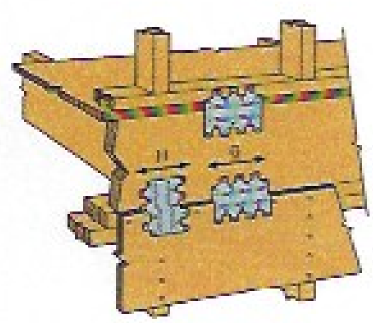


6 LTP4 Installed over Wood Structural Panel Sheathing

LTP4 attaching Top Plates to Rim Board



7 LTP5 Installed over Wood Structural Panel Sheathing

Straps and Ties

LTP4/LTP5/A34/A35

SIMPSON Strong-Tie

Page K2.F

Framing Angles and Plates (cont.)

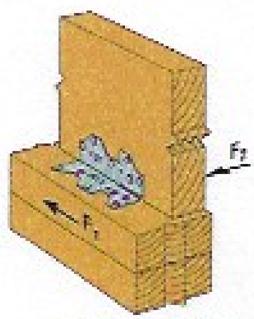
These products are available with additional corrosion protection. For more information, see p. 15.

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

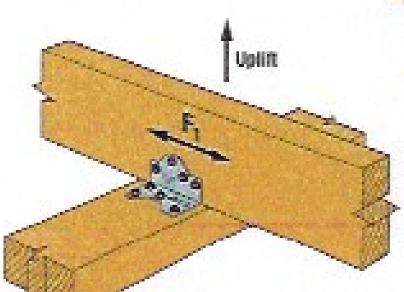
SD Many of these products are approved for installation with Strong-Drive® SD Connector screws. See pp. 335–337 for more information.

Model No.	Type of Connection	Fasteners (in.)	Direction of Load	DF/SP Allowable Loads			SPF/HF Allowable Loads			Code Ref.
				Floor (100)	Roof (125)	(160)	Floor (100)	Roof (125)	(160)	
A34	1	(8) 0.131 x 1½	F_1	395	465	465	340	400	400	IBC, FL, LA
			F_2[6]	395	430	430	340	370	370	
	1	(8) #9 x 1½ SD	F_1	640	640	640	550	550	550	
			F_2	495	495	495	425	425	425	
			Uplift	240	240	240	170	170	170	—
A35	2	(9) 0.131 x 1½	A_1	295	350	350	255	300	300	IBC, FL, LA
			E	295	360	385	255	310	330	
			C_1	185	185	185	160	160	160	
	3	(12) 0.131 x 1½	A_2	295	325	325	255	280	280	
			C_2	295	330	330	255	285	285	
			D	225	225	225	195	195	195	
	4	(12) 0.131 x 1½	F_1	590	650	650	510	560	560	
			F_2[6]	590	670	670	510	575	575	
	5	(12) PH612I	F_1	420	420	420	360	360	360	—
LTP4	6	(12) 0.131 x 1½	G	580	625	625	500	540	540	IBC, FL, LA
			H	580	525	525	500	450	450	
LTP5	7	(12) 0.131 x 1½	G	580	565	565	500	485	485	
			H	545	490	480	470	420	420	

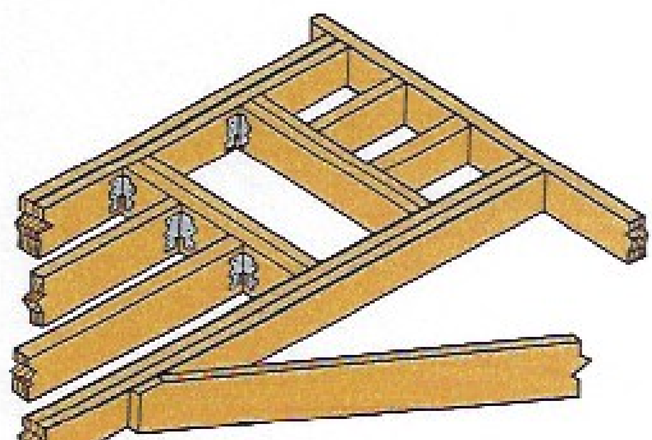
1. Allowable loads are for one angle. When angles are installed on each side of the joist, the minimum joist thickness is 3".
2. Some illustrations show connections that could cause cross-grain tension or bending of the wood during loading if not reinforced sufficiently. In this case, mechanical reinforcement should be considered.
3. LTP4 can be installed over ⅜" wood structural panel sheathing with 0.131" x 1½" nails and achieve 0.72 of the listed load, or over ½" sheathing and achieve 0.64 of the listed load. 0.131" x 2½" nails will achieve 100% load.
4. LTP4 satisfies the IRC continuously sheathed portal frame (CS-PF) framing anchor requirements when installed over raised wood floor framing per Figure R602.10.6.4.
5. The LTP5 may be installed over wood structural panel sheathing up to ½" thick using 0.131" x 1½" nails with no reduction in load.
6. Connectors are required on both sides to achieve F_2 loads in both directions.
7. **Fasteners:** Nail dimensions in the table are diameter by length. SD screws are Simpson Strong-Tie® Strong-Drive® screws. PH612I is a pan-head #6 x ½" screw available from Simpson Strong-Tie. See pp. 21–22 for other nail sizes and information.



4 Typical A35 Installation



1 A34 Typical Installation



Chimney Framing

Straps and Ties

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Roof Framing (cont'd)

Wall Headers $l = 24'$, $w = 9.5'(35\text{psf}) = 333\text{plf}$, $R = 3990^{\#}$, $+M = 23976^{\#\cdot'}$

$\Delta_a = \frac{288''}{360} = 0.80''$ $I_r = \frac{5(333\text{plf})(288'')^4}{12^4 (384)\, 1.9\times10^6\text{psi}\,(0.8'')} = 1635\ \text{in}^4 \rightarrow \underline{3\text{-}1^3\!/\!_4'' \times 16''\ \text{LVL}}$ (1.9E)

$l = 12'$, $w = 4'(35\text{psf}) = 140\text{plf}$, $R = 840^{\#}$, $+M = 2520^{\#\cdot'} \rightarrow \underline{3\text{-}2\times10}$

$l = 5'$, $w = 10'(35\text{psf}) = 350\text{plf}$, $R = 875^{\#}$, $+M = 1094^{\#\cdot'} \rightarrow \underline{3\text{-}2\times6}$

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Foundations

Interior Footings (thickened slab) say $q_a = 1500$ psf

$P_{\text{main room}} = 7350\#$ $A_r = \frac{7350\#}{1500\text{ psf}} = 4.9\text{ sf} = \underline{2'\text{-}3''\text{ sq} \times 16''}$

$P_{\text{restrooms}} = 4200\# + 1680\# = 5880\#$ $A_r = 3.92\text{ sf} = 2'\text{-}0''\text{ sq}$ $\underline{2'\text{ sq} \times 16''}$ (min.)

Exterior Footings

$w = 14(10\text{ psf}) + 5.5'(20\text{ psf}) + 9'(35\text{ psf}) + 2.5'(200\text{ psf}) + 4'(100\text{ psf}) = 1405\text{ plf}$

$b_r < 1.33'$ (min.), ok