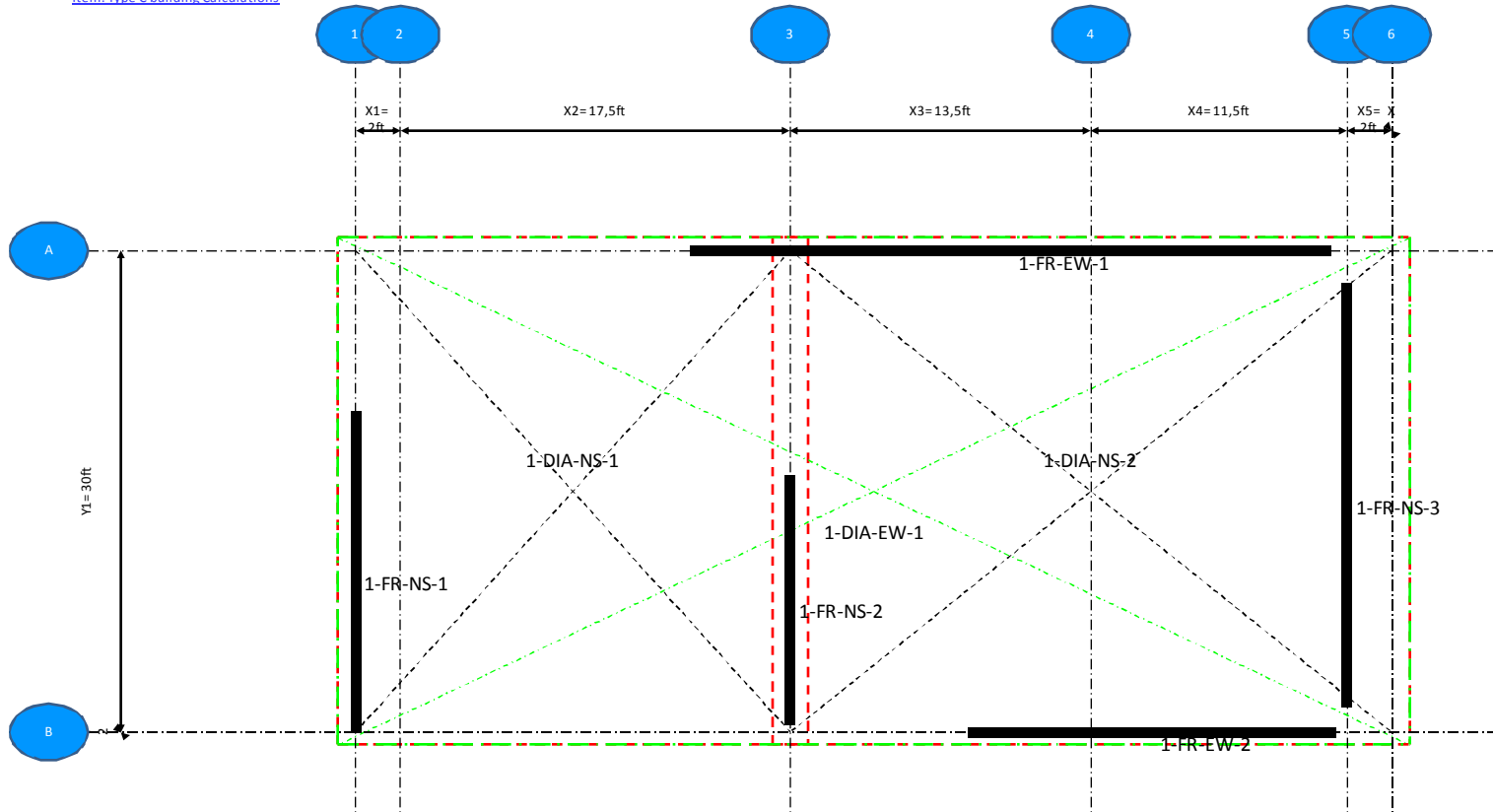


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XXXX W. IMPERIAL HWY
INGLEWOOD, CA 90303
(310)722-XXXX

Date: 9/3/2013
By: RS
Chk'd: F.K.L

Job Address: WEST 112TH STREET
LOS ANGELES COUNTY, CA 90304
Item: Type C building Calculations

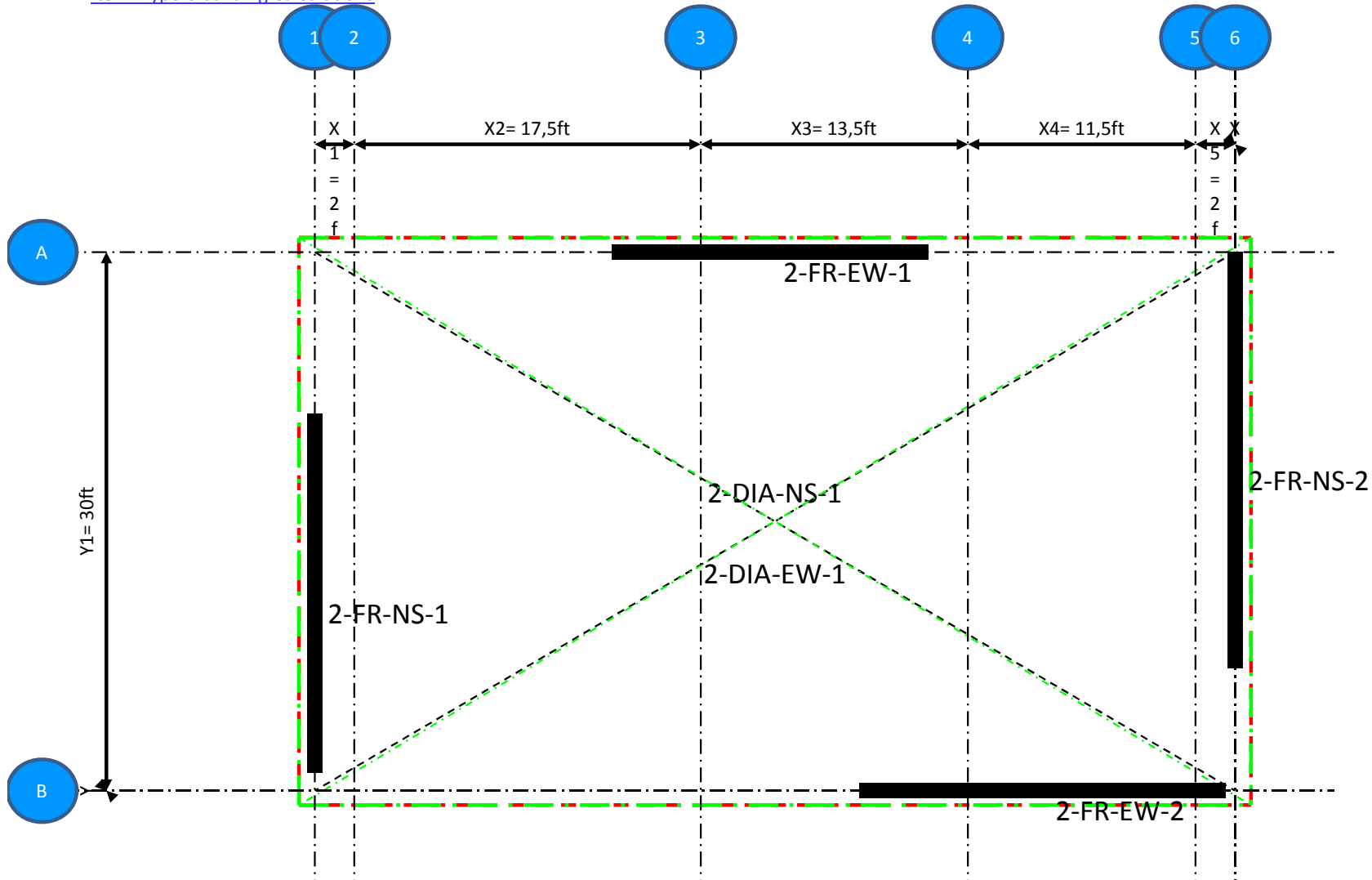


First Floor Plan

XXXX & ASSOCIATES
XXXX W. IMPERIAL HWY
INGLEWOOD, CA 90303
(310) 722-XXXX

Date: 9/3/2013
By: RS
Chk'd: F.K.L

Job Address: WEST 112TH STREET
LOS ANGELES COUNTY, CA 90304
[Item: Type C building Calculations](#)



2nd Floor Plan

ON

Gridlines

North-South Gridlines:

[illegible]

East-West Gridlines:

[illegible]

First Floor North-South Diaphragm boundaries:

DIAPH.	NORTH	SOUTH	EAST	WEST
1-DIA-NS-1	A	B	3	1
1-DIA-NS-2	A	B	6	3
1-DIA-NS-3				
1-DIA-NS-4				
1-DIA-NS-5				
1-DIA-NS-6				
1-DIA-NS-7				
1-DIA-NS-8				
1-DIA-NS-9				
1-DIA-NS-10				
1-DIA-NS-11				
1-DIA-NS-12				
1-DIA-NS-13				
1-DIA-NS-14				
1-DIA-NS-15				

First Floor East-West Diaphragm boundaries:

DIAPH.	NORTH	SOUTH	EAST	WEST
1-DIA-EW-1	A	B	6	
1-DIA-EW-2				
1-DIA-EW-3				
1-DIA-EW-4				
1-DIA-EW-5				
1-DIA-EW-6				
1-DIA-EW-7				
1-DIA-EW-8				
1-DIA-EW-9				
1-DIA-EW-10				
1-DIA-EW-11				
1-DIA-EW-12				
1-DIA-EW-13				
1-DIA-EW-14				
1-DIA-EW-15				

First Floor

North-South Shear Wall layout:

Shear wall	Grid	Length (ft)	distance From 0,0 (ft)
1-FR-NS-1	1.00	20.00	10.00
1-FR-NS-2	3.00	15.50	14.00
1-FR-NS-3	5.00	26.50	2.00
1-FR-NS-4			
1-FR-NS-5			
1-FR-NS-6			
1-FR-NS-7			
1-FR-NS-8			
1-FR-NS-9			
1-FR-NS-10			
1-FR-NS-11			
1-FR-NS-12			
1-FR-NS-13			
1-FR-NS-14			
1-FR-NS-15			

East-West Shear Wall layout:

Shear wall	Grid	Length (ft)	distance From 0,0 (ft)
1-FR-EW-1	A	28.75	15.00
1-FR-EW-2	B	16.50	27.50
1-FR-EW-3			
1-FR-EW-4			
1-FR-EW-5			
1-FR-EW-6			
1-FR-EW-7			
1-FR-EW-8			
1-FR-EW-9			
1-FR-EW-10			
1-FR-EW-11			
1-FR-EW-12			
1-FR-EW-13			
1-FR-EW-14			
1-FR-EW-15			

ON

Gridlines

North-South Gridlines:

[illegible]

East-West Gridlines:

[illegible]

2nd Floor

2nd Floor North-South Diaphragm boundaries:

DIAPH.	NORTH	SOUTH	EAST	WEST
2-DIA-NS-1	A	B	6.00	1.00
2-DIA-NS-2				
2-DIA-NS-3				
2-DIA-NS-4				
2-DIA-NS-5				
2-DIA-NS-6				
2-DIA-NS-7				
2-DIA-NS-8				
2-DIA-NS-9				
2-DIA-NS-10				
2-DIA-NS-11				
2-DIA-NS-12				
2-DIA-NS-13				
2-DIA-NS-14				
2-DIA-NS-15				

2nd Floor East-West Diaphragm boundaries:

DIAPH.	NORTH	SOUTH	EAST	WEST
2-DIA-EW-1	A	B	6.00	1.00
2-DIA-EW-2				
2-DIA-EW-3				
2-DIA-EW-4				
2-DIA-EW-5				
2-DIA-EW-6				
2-DIA-EW-7				
2-DIA-EW-8				
2-DIA-EW-9				
2-DIA-EW-10				
2-DIA-EW-11				
2-DIA-EW-12				
2-DIA-EW-13				
2-DIA-EW-14				
2-DIA-EW-15				

North-South Shear Wall layout:

	Grid	Length (ft)	distance From 0,0 (ft)
Shear wall			
2-FR-NS-1	1.00	20.00	9.00
2-FR-NS-2	6.00	23.20	0.00
2-FR-NS-3			
2-FR-NS-4			
2-FR-NS-5			
2-FR-NS-6			
2-FR-NS-7			
2-FR-NS-8			
2-FR-NS-9			
2-FR-NS-10			
2-FR-NS-11			
2-FR-NS-12			
2-FR-NS-13			
2-FR-NS-14			
2-FR-NS-15			

East-West Shear Wall layout:

			distance From
Shear wall	Grid	Length (ft)	0,0 (ft)
2-FR-EW-1	A	16.00	15.00
2-FR-EW-2	B	18.50	27.50
2-FR-EW-3			
2-FR-EW-4			
2-FR-EW-5			
2-FR-EW-6			
2-FR-EW-7			
2-FR-EW-8			
2-FR-EW-9			
2-FR-EW-10			
2-FR-EW-11			
2-FR-EW-12			
2-FR-EW-13			
2-FR-EW-14			
2-FR-EW-15			

XXXX & ASSOCIATES

XXXX W. IMPERIAL HWY
INGLEWOOD, CA 90303
(310) 722-XXXX

Client: Nguyen
Job Address: 22548 BYRON ST.
HAYWARD, CA 94541
Item: Type C building Calculations

Date: 7/3/2013
By: RS
Chk'd: HC

2) Building Gravity Loads:

Roof:		lbs/SF
Roofing:	Light Clay Tile	9.00
Sheathing:	1/2" DF Plywood	1.50
Rafters:	2x10 at 16" o.c.	2.61
Ceiling:	Ceiling: 5/8" Gypsum Board	2.80
	Joists 2x10 at 16" o.c.	2.61
Miscellaneous:		3.48
Roof Dead Load		16.84
Total Dead Load:		22.00
Min. Pitch	4	:12
Live Load:		20.00
Total Roof:		42.00 lbs/SF

Floors:		lbs/SF
Finish:	Carpet & Pad	1.00
Subfloor:	3/4" DF Plywood	2.30
Joists:	2x12 at 16" o.c.	3.17
Miscellaneous:		1.53
Dead Load:		8.00
Live Load:	Basic Residential	40.00
Total Floor:		48.00 lbs/SF

Walls:	Int. walls	Exterior walls
	lbs/SF	lbs/SF
Studs: 2x4 at 16" o.c.	0.98	0.98
Plates:	0.33	0.33
Sheathing: 1/2" DF Plywood		1.50
Wallboard: 1/2" Gypsum Board	4.40	2.20
Siding: 7/8" Stucco		10.00
Miscellaneous:	1.29	2.99
Total Wall self weight, Wsw:	7.00	18.00

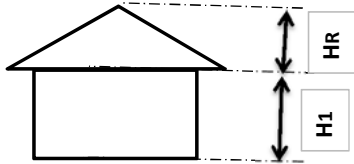
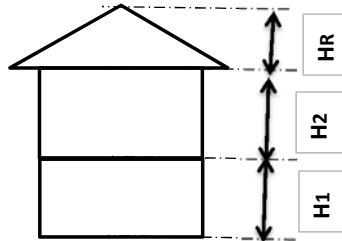
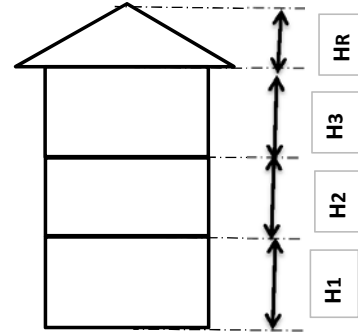
(of vertical surface area)

XXXX & ASSOCIATES

XXXX W. IMPERIAL HWY
 INGLEWOOD, CA 90303
 (310) 722-XXXX

Client: WEST 112TH STREET
 Job Address: WEST 112TH STREET
 LOS ANGELES COUNTY, CA 90304
 Item: Type C building Calculations

Date: 9/3/2013
 By: RS
 Chk'd: F.K.L

1) Input Geometry:**CASE : 1****CASE : 2****CASE : 2**

Building case: Case: 2

Roof	HR=	6.5	ft
2nd floor	H2=	10	ft
1st floor	H1=	10	ft

Footprint:

Roof	1,350	SF
2nd Floor	1,350	SF
1st Floor	1,350	SF
n/a	n/a	SF

Trib. Height (ft)	Floor Dead Load (lbs)	Ext Wall Length (ft)	Ext Wall Load (lbs)	Int Wall Length (ft)	Int Wall Load (lbs)	Wtotal (kips)
5	29,700	158	14,220	16	560	44.48
10	10,800	158	28,440	120	8,400	47.64

Material Weights

Roofing Material Weights	p.s.f.
(2) 15 & (1) 90 Comp	1.7
(3) 15 & (1) 90 Comp.	2.2
3 Ply & Gravel	5.6
4 Ply & Gravel	6
5 Ply & Gravel	6.5
Asphalt Shingles	5
Heavy Clay Tile	14
Light Clay Tile	9
Slate (3/8")	15
Steel 16 ga.	2.9
Steel 20 ga.	1.8
Wood Shakes	4

Spacing of members	ft
at 12" o.c.	1
at 16" o.c.	1.33
at 24" o.c.	2

Plywood Sheathing Weights	p.s.f.
1 1/8" DF Plywood	3.4
1 1/8" OSB	3.7
1/2" DF Plywood	1.5
1/2" OSB	1.7
3/4" DF Plywood	2.3
3/4" OSB	2.5
5/8" DF Plywood	1.8
5/8" OSB	2
7/8" OSB	2.9

Rafters/Joists	p.s.f.
2x10	3.47
2x12	4.22
2x14	4.97
2x4	1.31
2x6	2.06
2x8	2.72
TJI / 250 Pro	2.4
TJI / 350 Pro	3.3
TJI / 550 Pro	4.7
TJI / H60	3.1
TJI / H90	4.1

TJI / L60	2.8
TJI / L90	3.7
TJI / TJH	10
TJI / TJL (TJLX)	3.75
TJI / TJM	8
TJI / TJS	4.75
TJI / TJW	4.5
None	0

Insulation	p.s.f.
Fiberglass Wool	1
Rock Wool	2

Siding	p.s.f.
7/8" Stucco	10 psf
1" Plaster on Wood Lath	10 psf
Stone	4 psf
Vinyl Siding	3 psf

Ceilings	p.s.f.
1/2" Gypsum Board	2.2
5/8" Gypsum Board	2.8
Lath & Plaster (1")	8
Suspended Fiber Tile	1.8
None	0

Floors	p.s.f.
Carpet & Pad	1
Concrete (LtWt)	10
Concrete (Reg.)	12
Gypsum Concrete	6.5
Hardwood (1" Nom.)	4
Tile (3/4")	10
Vinyl	1

Wall type	p.s.f.
Interior	7.00
Exterior	18.00

Wall alignment	
Yes	
No	

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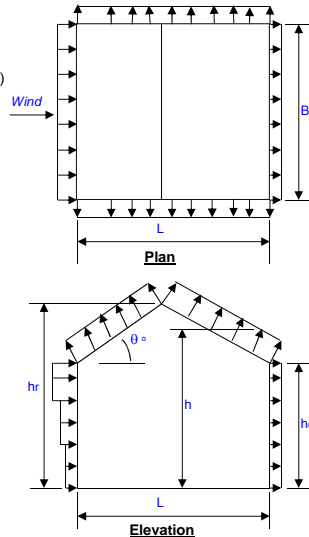
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INGLEWOOD, CA 90303
(310) 722-XXXX

Job Address: WEST 112TH STREET
LOS ANGELES COUNTY, CA 90304
Item: Type C building Calculations

Date: 9/3/2013
By: RS
Chk'd: F.K.L

3) Building Wind Loads:**ASCE 7-10 WIND Analysis**

Wind Direction =	Normal	(Normal or Parallel to building ridge)
Wind Speed, V =	110	mph (Wind Map, Figure 6-1)
Bldg. Classification =	II	(Table 1-1)
Exposure Category =	C	(Sect. 6.5.6)
Ridge Height, hr =	26.50	ft. (hr >= he)
Eave Height, he =	20.00	ft. (he <= hr)
Building Width =	30.00	ft. (Normal to Building Ridge)
Building Length =	45.00	ft. (Parallel to Building Ridge)
Roof Type =	Gable	(Gable or Monoslope)
Topo. Factor, Kzt =	1.00	(Sect. 6.5.7 & Figure 6-4)
Direct. Factor, Kd =	0.85	(Table 6-4)
Enclosed? (Y/N)	Y	(Sect. 6.2 & Figure 6-5)
Damping Ratio, β =	0.030	(Suggested Range = 0.010-0.070)
Period Coef., Ct =	0.0200	(Suggested Range = 0.020-0.035)
(Assume: $T = Ct \cdot h^{3/4}$, and $f = 1/T$)		

**Resulting Parameters and Coefficients:**

Roof Angle, θ =	23.43	deg.	
Mean Roof Ht., h =	23.25	ft. ($h = (hr + he)/2$, for roof angle > 10 deg.)	L = 30 ft.
Windward Wall Cp =	0.80	(Fig. 6-6)	B = 45 ft.
Leeward Wall Cp =	-0.50	(Fig. 6-6)	
Side Walls Cp =	-0.70	(Fig. 6-6)	
Windward Roof Cp =	-0.46	(Fig. 6-6) (Condition #1)	
Windward Roof Cp =	0.03	(Fig. 6-6) (Condition #2)	
Leeward Roof Cp =	-0.60	(Fig. 6-6)	
+GCpi Coef. =	0.18	(Figure 6-5) (positive internal pressure)	
-GCpi Coef. =	-0.18	(Figure 6-5) (negative internal pressure)	

If $z \leq 15$ then: $K_z = 2.01 \cdot (15/zg)^{(2/\alpha)}$, If $z > 15$ then: $K_z = 2.01 \cdot (z/zg)^{(2/\alpha)}$ (Table 6-3, Case 2a)

$\alpha = 9.50$ $zg = 900$ (Table 6-2)

$K_h = 0.93$ ($K_h = K_z$ evaluated at $z = h$)

$I = 1.00$ (Table 6-1) (Importance factor)

Velocity Pressure: $q_z = 0.00256 \cdot K_z \cdot K_{zt} \cdot K_d \cdot V^2 \cdot I$ (Sect. 6.5.10, Eq. 6-15)

$q_h = 24.51$ psf $q_h = 0.00256 \cdot K_h \cdot K_{zt} \cdot K_d \cdot V^2 \cdot I$ (q_z evaluated at $z = h$)

Ratio $h/L = 0.775$ freq., $f = 4.722$ hz. ($f \geq 1$, Rigid structure)

Gust Factor, $G = 0.850$ (Sect. 6.5.8)

Design Net External Wind Pressures (Sect. 6.5.12.2):

$p = q_z \cdot G \cdot C_p - q_i \cdot (+/-GCpi)$ for windward wall (psf), where: $q_i = q_h$ (Eq. 6-17, Sect. 6.5.12.2.1)

$p = q_h \cdot G \cdot C_p - q_i \cdot (+/-GCpi)$ for leeward wall, sidewalls, and roof (psf), where: $q_i = q_h$ (Sect. 6.5.12.2.1)

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Job Address: WEST 112TH STREET
LOS ANGELES COUNTY, CA 90304
Item: Type C building Calculations

Date: 9/3/2013
By: RS
Chk'd: F.K.L

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	10.79	19.61
	15.00	0.85	22.35	0.80	10.79	19.61
	20.00	0.90	23.75	0.80	11.74	20.56
	25.00	0.95	24.89	0.80	12.51	21.34
	26.50	0.96	25.20	0.80	12.72	21.55
For z = hr:						
For z = he:	20.00	0.90	23.75	0.80	11.74	20.56
For z = h:	23.25	0.93	24.51	0.80	12.26	21.08
Leeward Wall	All	-	-	-0.50	-14.83	-6.01
Side Walls	All	-	-	-0.70	-19.00	-10.17
Roof (windward) conc	-	-	-	-0.46	-13.97	-5.14
Roof (windward) conc	-	-	-	0.03	-3.77	5.05
Roof (leeward)	-	-	-	-0.60	-16.91	-8.09
	-	-	-			

- Notes: 1. (+) and (-) signs signify wind pressures acting toward & away from respective surfaces.
2. Per Code Section 6.1.4.1, the minimum wind load for MWFRS shall not be less than 10 psf.
3. Refer: a. ASCE 7-02, "Minimum Design Loads for Buildings and Other Structures".
b. "Guide to the Use of the Wind Load Provisions of ASCE 7-05"
by: Kishor C. Mehta and James M. Delahay (2004).

Determination of Gust Effect Factor, G:

Is Building Flexible? No $f \geq 1$ Hz.

1: Simplified Method for Rigid Building

G = 0.850

Parameters Used in Both Item #2 and Item #3 Calculations (from Table 6-2):

α^A = 0.105
 b^A = 1.00
 $\alpha(\text{bar})$ = 0.154
 $b(\text{bar})$ = 0.65
c = 0.20

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LOS ANGELES COUNTY, CA 90304
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Date: 9/3/2013
By: RS
Chk'd: F.K.L

$l = 500$ ft.
 $s(\text{bar}) = 0.200$
 $z(\text{min}) = 15$ ft.

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

$z(\text{bar}) = 15.00 = 0.6 \cdot h$, but not $< z(\text{min})$, ft.
 $l_z(\text{bar}) = 0.228 = c \cdot (33/z(\text{bar}))^{1/6}$, Eq. 6-5
 $L_z(\text{bar}) = 427.06 = l \cdot (z(\text{bar})/33)^{1/3} \cdot s(\text{bar})$, Eq. 6-7
 $gq = 3.4$ (3.4, per Sect. 6.5.8.1)
 $gv = 3.4$ (3.4, per Sect. 6.5.8.1)
 $gr = 4.545 = (2 \cdot (\ln(3600 \cdot f)))^{1/2} + 0.577 / (2 \cdot \ln(3600 \cdot f))^{1/2}$, Eq. 6-9
 $Q = 0.913 = (1 / (1 + 0.63 \cdot ((B+h)/L_z(\text{bar}))^{0.63}))^{1/2}$, Eq. 6-6

2: Calculation of G for Rigid Building

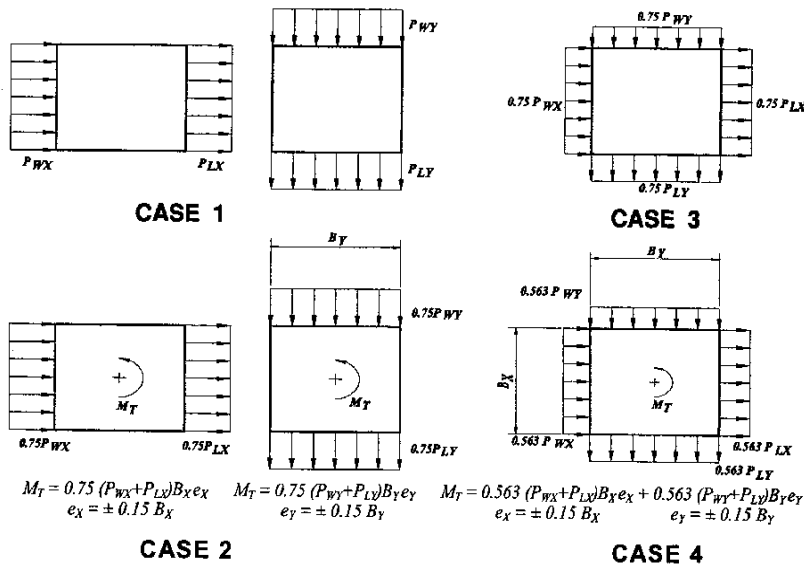
$G = 0.879 = 0.925 \cdot ((1 + 1.7 \cdot gq \cdot l_z(\text{bar}) \cdot Q) / (1 + 1.7 \cdot gv \cdot l_z(\text{bar})))$, Eq. 6-4

3: Calculation of Gf for Flexible Building

$\beta = 0.030$ Damping Ratio
 $C_t = 0.020$ Period Coefficient
 $T = 0.212 = C_t \cdot h^{3/4}$, sec. (Period)
 $f = 4.722 = 1/T$, Hz. (Natural Frequency)
 $V(\text{fps}) = \text{N.A.} = V(\text{mph}) \cdot (88/60)$, ft./sec.
 $V(\text{bar}, z\text{bar}) = \text{N.A.} = b(\text{bar}) \cdot (z(\text{bar})/33)^{\alpha(\text{bar})} \cdot V \cdot (88/60)$, ft./sec., Eq. 6-14
 $N_1 = \text{N.A.} = f \cdot L_z(\text{bar}) / (V(\text{bar}, z\text{bar}))$, Eq. 6-12
 $R_n = \text{N.A.} = 7.47 \cdot N_1 / (1 + 10.3 \cdot N_1^{5/3})$, Eq. 6-11
 $\eta h = \text{N.A.} = 4.6 \cdot f \cdot h / (V(\text{bar}, z\text{bar}))$
 $R_h = \text{N.A.} = (1/\eta h) - 1 / (2 \cdot \eta h^2) \cdot (1 - e^{-(2 \cdot \eta h)})$ for $\eta h > 0$, or $= 1$ for $\eta h = 0$, Eq. 6-13
 $\eta B = \text{N.A.} = 4.6 \cdot f \cdot B / (V(\text{bar}, z\text{bar}))$
 $R_B = \text{N.A.} = (1/\eta B) - 1 / (2 \cdot \eta B^2) \cdot (1 - e^{-(2 \cdot \eta B)})$ for $\eta B > 0$, or $= 1$ for $\eta B = 0$, Eq. 6-13
 $\eta L = \text{N.A.} = 15.4 \cdot f \cdot L / (V(\text{bar}, z\text{bar}))$
 $R_L = \text{N.A.} = (1/\eta L) - 1 / (2 \cdot \eta L^2) \cdot (1 - e^{-(2 \cdot \eta L)})$ for $\eta L > 0$, or $= 1$ for $\eta L = 0$, Eq. 6-13
 $R = \text{N.A.} = ((1/\beta) \cdot R_n \cdot R_h \cdot R_B \cdot (0.53 + 0.47 \cdot R_L))^{1/2}$, Eq. 6-10
 $G_f = \text{N.A.} = 0.925 \cdot (1 + 1.7 \cdot l_z(\text{bar}) \cdot (gq^2 \cdot Q^2 + gr^2 \cdot R^2)^{1/2}) / (1 + 1.7 \cdot gv \cdot l_z(\text{bar}))$,
Use: $G = 0.850$

Eq. 6-8

Figure 6-9 - 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

XXXX & ASSOCIATES

XXXX W. IMPERIAL HWY
INGLEWOOD, CA 90303
(310) 722-XXXX

Job Address: WEST 112TH STREET
LOS ANGELES COUNTY, CA 90304
Item: Type C building Calculations

Date: 9/3/2013
By: RS
Chk'd: F.K.L

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 6.5.12.2.1 and 6.5.12.2.3 as applicable for buildings of all heights.

2. Above diagrams show plan views of building.

3. Notation:

Pwx, Pwy = Windward face pressure acting in the X, Y principal axis, respectively.

PLx, PLy = Leeward face pressure acting in the X, Y principal axis, respectively.

e (ex, ey) = Eccentricity for the X, Y principal axis of the structure, respectively.

MT = Torsional moment per unit height acting about a vertical axis of the building.

p_s, Use a conservative approach and design with this 21.55 psf
pressure for entire structure

Horizontal Pressure: 21.55 lbs/SF
Uplift Pressure: 15 lbs/SF (Conservative)

N-S Projection:

	Exterior wall Projection Area		N-S Projection Area at level:		Wind Force	
Roof:	265 SF		533 SF		11,473 lbs	
2nd Floor:	535 SF		535 SF		11,527 lbs	
1st Floor:	535 SF		0 SF		0 lbs	
N/A	0 SF					
Base Shear					22,999 lbs	

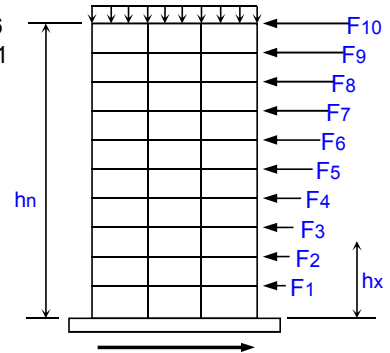
E-W Projection:

	Exterior wall Projection Area		E-W Projection Area at level:			
Roof:	92 SF		232 SF		4,998 lbs	
2nd Floor:	280 SF		280 SF		6,033 lbs	
1st Floor:	280 SF		0 SF		0 lbs	
N/A	0 SF					
Base Shear					11,031 lbs	

4) Building Seismic Loads:

Input Data:

Occupancy Category = **I** IBC 2009, Table 1604.5, page 307
 Importance Factor, I = **1.00** ASCE 7-05 Table 11.5-1, page 116
 Soil Site Class = **D** IBC 2009 Table 1613.5.2, page 341
 Location Zip Code = **90304**
 Spectral Accel., Ss = **1.750** ASCE 7-05 Figures 22-1 to 22-14
 Spectral Accel., S1 = **0.750** ASCE 7-05 Figures 22-2 to 22-14
 Long. Trans. Period, TL = **8.000** sec. ASCE 7 Fig's. 22-15 to 22-20
 Structure Height, hn = **20.000** ft.
 Actual Calc. Period, Tc = **0.000** sec. from independent analysis
 Seismic Resist. System = **A13** Light-framed walls sheathed with wood structural panels rated for shear or steel sheets (ASCE 7-05 Table 12.2-1)



$$V = C_s * W = \sum(F_i) = 16.55 \text{ kips}$$

Seismic Base Shear

Structure Weight Distribution:

No. of Seismic Levels = **2**

Seismic Level x	Height, hx (ft.)	Weight, Wx (kips)
2	20.000	44.50
1	10.000	47.70
	0.000	0.00

Total Weight, W = $\sum W_x$ = **92.20** kips (ASCE 7-05 Section 12.7.2)

Results:

Site Coefficients:

Fa = **1.000** IBC 2009 Table 1613.5.3(1), page 341
 Fv = **1.500** IBC 2009 Table 1613.5.3(2), page 341

Maximum Spectral Response Accelerations for Short and 1-Second Periods:

SMS = **1.750** SMS = Fa*Ss, IBC 2009 Eqn. 16-36, page 340
 SM1 = **1.125** SM1 = Fv*S1, IBC 2009 Eqn. 16-37, page 340

Design Spectral Response Accelerations for Short and 1-Second Periods :

SDS = **1.167** SDS = 2*SMS/3, IBC 2009 Eqn. 16-38, page 342
 SD1 = **0.750** SD1 = 2*SM1/3, IBC 2009 Eqn. 16-39, page 342

(continued:)

Seismic Design Category:

Category(for S_{DS}) = **E** IBC 2009 Table 1613.5.6(1), page 343
 Category(for S_{D1}) = **E** IBC 2009 Table 1613.5.6(2), page 343
 Use Category = **E** Most critical of either category case above controls

Fundamental Period:

Period Coefficient, C_T = **0.020** ASCE 7-05 Table 12.8-2, page 129
 Period Exponent, x = **0.75** ASCE 7-05 Table 12.8-2, page 129
 Approx. Period, T_a = **0.189** sec., T_a = C_T*h_n^x, ASCE 7-05 Section 12.8.2.1, Eqn. 12.8-7
 Upper Limit Coef., C_u = **1.400** ASCE 7-05 Table 12.8-1, page 129
 Period max., T_(max) = **0.265** sec., T_(max) = C_u*T_a, ASCE 7-05 Section 12.8-2, page 129
 Fundamental Period, T = **0.189** sec., T = T_a <= C_u*T_a, ASCE 7-05 Section 12.8.2, page 129

Seismic Design Coefficients and Factors:

Response Mod. Coef., R = **6.5** ASCE 7-05 Table 12.2-1, pages 120-122
 Overstrength Factor, Ω_o = **3** ASCE 7-05 Table 12.2-1, pages 120-122
 Defl. Amplif. Factor, C_d = **4** ASCE 7-05 Table 12.2-1, pages 120-122
 C_s = **0.179** C_s = S_{DS}/(R/I), ASCE 7-05 Section 12.8.1.1, Eqn. 12.8-2
 C_{S(max)} = **0.610** For T <= T_L, C_{S(max)} = S_{D1}/(T*(R/I)), ASCE 7-05 Eqn. 12.8-3
 C_{S(min)} = **0.058** C_{S(min)} = 0.5*S₁/(R/I), ASCE 7-05 Eqn. 12.8-6
 Use: C_s = **0.179** C_{S(min)} <= C_s <= C_{S(max)}

Seismic Base Shear:

V = **16.55** kips, V = C_s*W, ASCE 7-05 Section 12.8.1, Eqn. 12.8-1

Seismic Shear Vertical Distribution:

Distribution Exponent, k = **1.00** k = 1 for T <= 0.5 sec., k = 2 for T >= 2.5 sec.
 k = (2-1)*(T-0.5)/(2.5-0.5)+1, for 0.5 sec. < T < 2.5 sec.

Lateral Force at Any Level: F_x = C_{vx}*V, ASCE 7-05 Section 12.8.3, Eqn. 12.8-11, page 130

Vertical Distribution Factor: C_{vx} = W_x*h_x^k/(ΣW_i*h_i^k), ASCE 7-05 Eqn. 12.8-12, page 130

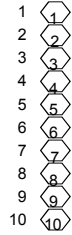
Seismic Level x	Weight, W _x (kips)	h _x ^k (ft.)	W _x *h _x ^k (ft-kips)	C _{vx} (%)	Shear, F _x (kips)	Σ Story Shears
2	44.50	20.000	890.0	0.651	10.77	10.77
1	47.70	10.000	477.0	0.349	5.77	16.55
Σ =	92.20		1367.0	1.000	16.55	

Comments:

5) SCHEDULES:

PLYWOOD SCHEDULE

	Plywood
0	3
260	7
310	4
380	8
460	5
490	9
600	6
640	10
770	NEW WALL



Plywood Thickness	Plywood Grade	Common Nail Size	Edge Nailing (in o.c.)	Field Nailing	Unsupported Panel Edges	Panel Edge Studs Required	Allowable Shear lbs/ft	Nailing 16d Face	Sole Plate Nailing	Simpson A35 Roof Shear Conn. Spacing
1/2"	APA 24/0	8d	6	12" o.c.	Unblocked	2x	180	15" o.c.	2'-6" o.c.	
3/4"	APA 24/0	10d	6	10" o.c.	Unblocked	2x	255	10" o.c.	1'-9" o.c.	
1/2"	APA 24/0	8d	6	12" o.c.	Blocked	2x	260	10" o.c.	1'-8" o.c.	
1/2"	APA 24/0	8d	4	12" o.c.	Blocked	2x	380	7" o.c.	1'-2" o.c.	
1/2"	APA 24/0	8d	3	12" o.c.	Blocked	2x	490	5 1/2" o.c.	10" o.c.	
1/2"	APA 24/0	8d	2	12" o.c.	Blocked	3x	640	4" o.c.	8" o.c.	
1/2"	APA 24/0	10d	6	12" o.c.	Blocked	2x	310	8 1/2" o.c.	1'-4" o.c.	
1/2"	APA 24/0	10d	4	12" o.c.	Blocked	2x	460	5 1/2" o.c.	10" o.c.	
1/2"	APA 24/0	10d	3	12" o.c.	Blocked	3x	600	4 1/2" o.c.	9" o.c.	
1/2"	APA 24/0	10d	2	12" o.c.	Blocked	3x	770	3 1/2" o.c.	6" o.c.	

NOTE: Where 3x framing is required at adjoining panel edges, nails shall be staggered.

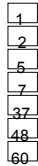
NOTE: If box or sinker nails are used in lieu of common nails either 1) increase nails by one size i.e. (from 8d to 10d or 10d to 16d) or 2) decrease edge nail spacing by 25% i.e. (from 6" to 4.5") (decreased nail spacing not allowed for shearwall type 6 or 10).

NOTE: If horizontal edges of structural panels are edge nailed to common horizontal floor framing members or wall plates, sole plate nail spacing can be increased to 16d @ 16" o.c.

Floor To Floor Strap schedule

Floor To Floor Strap

-5000	LFTA-1.5	1
1205	FTA2-1.5	2
1575	FTA5-1.5	5
1865	MST37	37
1905	FTA2-3.5	2
2820	FTA7-1.5	7
3095	MST48	48
3765	FTA5-3.5	5
4050	MST60	60
4785	FTA7-3.5	7
7395	New Strap	New Strap



Simpson Designation	1 1/2" Stud All Ld lbs	3 1/2" Stud All Ld lbs
LFTA	1205	
FTA2	1575	2820
FTA5	1865	4050
FTA7	3095	7395
MST37		2465
MST48		3695
MST60		4785

Floor To Foundation schedule

Load (lbs) strap

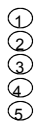
anchor

type	Simpson Designation	Min. Member Thickness (in)	Allowable Load (lbs)	Stud Required	Anchor Bolt
					Not Req'd
2	HDU2-SDS2.5		3075	2-2x or 4x	SSTB 20
4	HDU4-SDS2.5		4565	2-2x or 4x	SSTB 24
5	HDU5-SDS2.5		5645	2-2x or 4x	SSTB 24
8	HDU8-SDS2.5		5980	2-2x or 4x	SSTB 34
11	HDU11-SDS2.5		9535	3-2x or 6x	1" x 36"
14	HDU14-SDS2.5		10770	4x6	1" x 36"
18	(2)HDU11-SDS2.5		19070	3-2x or 6x	(2) 1" x 36"

ANCHOR BOLT SCHEDULE

Anchor Bolt

0	1
293	2
390	3
585	4
780	5
1170	New



Bolt Diameter	Bolt Spacing	Sill Plate Thickness	All. Shear lbs/ft
5/8"	4'-0"	2x	293
5/8"	3'-0"	3x	390
5/8"	2'-0"	3x	585
5/8"	1'-6"	3x	780
5/8"	1'-0"	3x	1170

All anchor bolts shall be ASTM A-307 material with a minimum embedment of 7" and include a 3" x 3" x 3/16" pl. washer.

Holdowns & Tension Ties

HDU/DTT2Z Holdowns



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.

HDU holdowns are pre-deflected during the manufacturing process, virtually eliminating deflection under load due to material stretch. They use Simpson Strong-Tie® Strong-Drive® SDS screws which install easily, reduce fastener slip and provide a greater net section when compared to bolts.

The HDU series of holdowns are designed to replace previous versions of the product such as PHD's as well as bolted holdowns. The HDU2, 4 and 5 are direct replacements for the PHD2, 5 and 6, respectively.

The DTT2Z tension tie is suitable for lighter-duty holddown applications on single or double 2x posts, and installs easily with Strong-Drive SDS screws (included). The DTT2Z has been tested in accordance with the ICC-ES acceptance criteria for Holdowns Attached to Wood Members (AC155) and meets the minimum requirements for many alternate braced wall panels per section R602.10.3.2 of the 2009 IRC (see table R602.10.6, item 1).

For more information on holddown options, contact Simpson Strong-Tie.

HDU SPECIAL FEATURES:

- Pre-deflected body virtually eliminates deflection due to material stretch.
- Uses SDS screws which install easily, reduce fastener slip, and provide a greater net section area of the post compared to bolts.
- SDS screws are supplied with the holdowns to ensure proper fasteners are used.
- No stud bolts to countersink at openings.

MATERIAL: See table

FINISH: HDU – Galvanized; DTT2Z – ZMAX® coating;

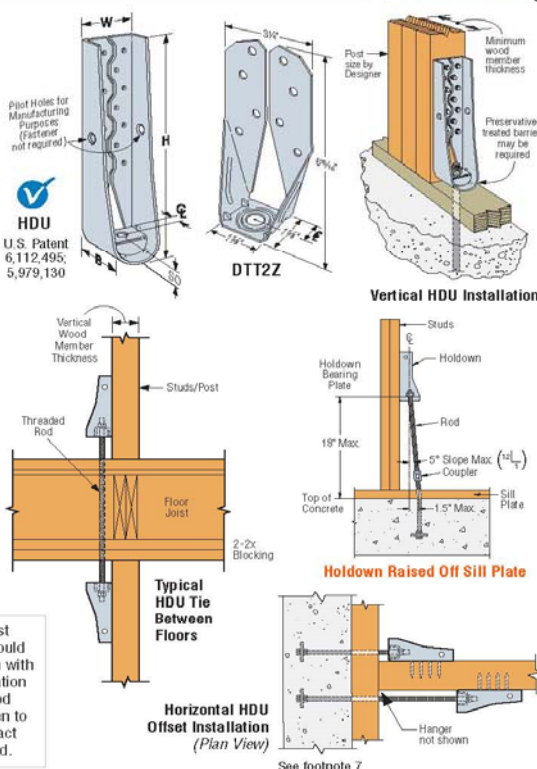
DTT2SS – stainless steel

INSTALLATION: • Use all specified fasteners. See General Notes.

- For use in vertical and horizontal applications.
- The HDU requires no additional washer, the DTT requires a standard cut washer (included) be installed between the nut and the seat.
- To tie multiple 2x members together, the Designer must determine the fasteners required to join the members without splitting the wood. See page 26 for SDS values.
- See SB and SSTB Anchor Bolts on pages 33-37 for anchorage options.
- SDS screws install best with a low speed high torque drill with a 3/8" hex head driver.

CODES: See page 13 for Code Reference Key Chart.

For holdowns, per ASTM test standards, anchor bolt nut should be finger-tight plus 1/4 to 1/2 turn with a hand wrench, with consideration given to possible future wood shrinkage. Care should be taken to not over-torque the nut. Impact wrenches should not be used.



These products are available with additional corrosion protection. Additional products on this page may also be available with this option, check with Simpson Strong-Tie for details.

Model No.	Ga	Dimensions (in.)					Fasteners		Minimum Wood Member Thickness ⁴ (in.)	Allowable Tension Loads (lbs.) (160) ¹			Code Ref.
		W	H	B	Q	SO	Anchor Bolt Dia. (in.)	SDS Screws		DF/SP	SPF/HF	Deflection at Allowable Load (in.)	
DTT2Z	14	3 3/4	6 1/8	1 1/2	1 1/8	3/8	1/2	8-SDS 1/4"x1 1/2"	1 1/2	1825	1800	0.105	I6, L8, F5
DTT2Z-SDS2.5								8-SDS 1/4"x1 1/2"	3	2145	1835	0.128	170
HDU2-SDS2.5	14	3	8 1/8	3/4	1 1/8	1 1/8	3/8	8-SDS 1/4"x2 1/2"	3	2145	2105	0.128	
HDU4-SDS2.5	14	3	10 1/8	3/4	1 1/8	1 1/8	3/8	6-SDS 1/4"x2 1/2"	3	3075	2215	0.088	
HDU5-SDS2.5	14	3	13 1/8	3/4	1 1/8	1 1/8	3/8	10-SDS 1/4"x2 1/2"	3	4565	3285	0.114	
								14-SDS 1/4"x2 1/2"	3	5645	4065	0.115	
HDU8-SDS2.5	10	3	16 1/8	3/4	1 1/8	1 1/8	3/8	20-SDS 1/4"x2 1/2"	3	5980	4305	0.084	I6, L8, F5
									3 1/2	6970	5020	0.116	
									4 1/2	7870	5665	0.113	
HDU11-SDS2.5	10	3	22 1/4	3/4	1 1/8	1 1/8	1	30-SDS 1/4"x2 1/2"	5 1/2	9535	6865	0.137	
									7 1/4	11175	8045	0.137	
HDU14-SDS2.5	7	3	25 1/8	3/4	1 1/8	1 1/8	1	36-SDS 1/4"x2 1/2"	4x6 ¹⁰	10770	7755	0.122	170
									7 1/4	14375 ⁸	10435 ⁹	0.177	I6, L8, F5
									5 1/2 ⁸	14445 ^{8,9}	10350 ⁹	0.177	

1. Allowable loads have been increased for earthquake or wind load durations with no further increase allowed; reduce where other load durations govern.
2. The Designer must specify anchor bolt type, length and embedment. See SB and SSTB Anchor Bolts (pages 33-37).
3. Structural composite lumber columns have sides that show either the wide face or the edges of the lumber strands/veneers. Values in the tables reflect installation into the wide face. See technical bulletin T-SCLCOLUMN for values on the narrow face (edge) (see page 232 for details).
4. Post design by Specifier. Tabulated loads are based on a minimum 3 1/2" wide post (in a 3 1/2" wall). Post may consist of multiple members provided they are connected independently of the holdown fasteners. See pages 226-227 for common post allowable loads.

5. Tension values are valid for holdowns flush or raised off of sill plate.
6. Deflection at Allowable Tension Load includes fastener slip, holdown deformation and anchor rod elongation for holdowns installed up to 6" above top of concrete. Holdowns may be installed raised up to 18" above top of concrete with no load reduction provided that additional elongation of the anchor rod is accounted for.
7. Tabulated loads may be doubled when the holdowns are installed on opposite sides of the wood member provided either the post is large enough to prevent opposing holdown screw interference or the holdowns are offset to eliminate screw interference.
8. Note HDU14 allowable loads are based on a 5 1/2" wide post (6x6 min.).
9. Requires heavy hex anchor nut to achieve tabulated loads (supplied with holdown).
10. Loads are applicable to installation on either narrow or wide face of post.

FTA/LFTA Floor Tie Anchors

Designed for use as a floor-to-floor tension tie, one FTA replaces two comparably sized holdowns and the threaded rod.

The LFTA Light Floor Tie Anchor is for nailed installations.

MATERIAL: See table

FINISH: LFTA—galvanized;

FTA—Simpson Strong-Tie® gray paint

INSTALLATION: • Use all specified fasteners.

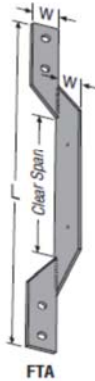
See General Notes.

- Washers required on side opposite FTA for full loads.
- Nail holes between floors allow preattachment to the joist during installation; these nails are not required.

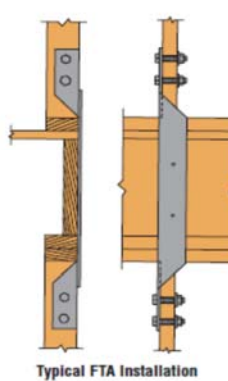
OPTIONS:

- The standard model's clear span of 17" will accommodate up to a 12" joist. The clear span of the FTA may be increased with a corresponding increase in overall length.

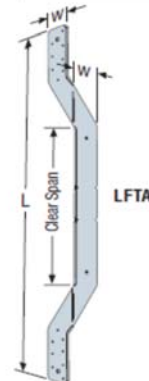
CODES: See page 13 for Code Reference Key Chart.



FTA



Typical FTA Installation



LFTA

SIMPSON
Strong-Tie

Model No.	Ga	Dimensions			Fasteners (Total)		Allowable Uplift Loads ¹ (160)						Code Ref.
		W	Clear Span	L	Qty	Dia	Vertical Member Thickness					LFTA ²	
							1½	2	2½	3	3½		
LFTA	16	2¼	17	38%	16-10d	—	—	—	—	—	—	1205	I17, L6, F16
FTA2	10	3	17	37%	4	½	1890	2515	3120	3385	3385	—	
FTA5	10	3½	17	45%	4	¾	2240	3000	3750	4400	4400	—	
FTA7	3	3½	17	56	6	¾	3715	5020	6210	7600	7600	—	

1. Allowable loads have been increased for wind or earthquake loading with no further increase allowed. Reduce where other loads govern.

2. Reduce the allowable load for the LFTA according to the code when nails penetrate wood less than 1 1/4".

3. **NAILS:** 10d = 0.148" dia. x 3" long. See page 22-23 for other nail sizes and information.

Concrete Connectors & Anchors**PAB Pre-Assembled Anchor Bolt**

The PAB anchor bolt is a versatile cast-in-place anchor bolt ideal for high-tension-load applications. It features a plate washer at the embedded end sandwiched between two fixed hex nuts and a head stamp for easy identification after the pour.

- Available in diameters from 1/2" to 1 1/2" in lengths from 12" to 36" (in 6" increments)
- Available in standard and high-strength steel
- Head stamp contains the No Equal sign, diameter designation and an "HS" on high-strength rods.

MATERIAL: Standard Steel — ASTM F1554 Grade 36, A36 or A307 — $F_u = 58$ ksi
High-Strength Steel (up to 1" dia.) — ASTM A449 — $F_u = 120$ ksi
High-Strength Steel (1 1/2" and 1 3/4" dia.) — ASTM A578 or F1554 Grade 105 — $F_u = 125$ ksi

FINISH: None

The Simpson Strong-Tie® Anchor Designer Software™ analyzes and suggests anchor solutions using the ACI 318 Appendix B strength design methodology (or ACI 318 Appendix B strength design methodology for 2008 ACI 318 Appendix B). It provides cracked and uncracked concrete anchorage solutions for numerous Simpson Strong-Tie mechanical and adhesive anchors as well as the PAB anchor bolt. With its easy-to-use graphical user interface, the software makes it easy for the Designer to identify anchorage solutions without having to perform time-consuming calculations by hand.

**PAB Anchor Bolt — Standard Steel**

Diameter (in.)	Plate Washer Size (in.)	L ₁ (in.)	Root Model No.	Lengths (in.)
1/2"	1/2" x 1 1/2" x 1 1/2"	1	PAB1-XX	12" to 36"
5/8"	5/8" x 1 1/2" x 1 1/2"	1 1/2"	PAB1 1/2-XX	(in 6" increments)
3/4"	3/4" x 2" x 2"	1 1/2"	PAB1 1/2-XX	
1"	1" x 2 1/2" x 2 1/2"	1 1/2"	PAB1 1/2-XX	
1 1/4"	1 1/4" x 2 1/2" x 2 1/2"	1 1/2"	PAB1 1/2-XX	
1 1/2"	1 1/2" x 3" x 3"	2"	PAB1 1/2-XX	

PAB Anchor Bolt — High-Strength Steel

Diameter (in.)	Plate Washer Size (in.)	L ₁ (in.)	Root Model No.	Lengths (in.)
1/2"	1/2" x 1 1/2" x 1 1/2"	1	PAB1H-XX	12" to 36"
5/8"	5/8" x 1 1/2" x 1 1/2"	1 1/2"	PAB1 1/2H-XX	(in 6" increments)
3/4"	3/4" x 2" x 2"	1 1/2"	PAB1 1/2H-XX	
1"	1" x 2 1/2" x 2 1/2"	1 1/2"	PAB1 1/2H-XX	
1 1/4"	1 1/4" x 2 1/2" x 2 1/2"	1 1/2"	PAB1 1/2H-XX	
1 1/2"	1 1/2" x 3" x 3"	2"	PAB1 1/2H-XX	

PAB Anchor Bolt — Anchorage Solutions

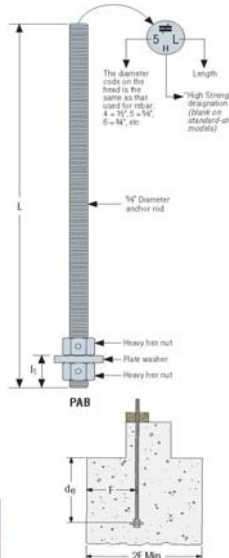
Design Criteria	Diameter (in.)	Anchor Bolt Model No.	7500 psi Concrete				3000 psi Concrete			
			Dimensions (in.)	Tension Load (lbs.)	ASD	LRF D	Dimensions (in.)	Tension Load (lbs.)	ASD	LRF D
Wind	1/2"	PAB1	4	6	3705	6175	4	6	3705	6175
	5/8"	PAB1 1/2	4	6	4030	6720	4	6	4415	7360
	3/4"	PAB1 1/2	5	8	5800	9830	5	7 1/2	5900	9830
	1"	PAB1 1/2	7	10 1/2	8720	14350	6 1/2	10	8720	14350
	1 1/4"	PAB1 1/2	9	13 1/2	12345	20085	8 1/2	13 1/2	11340	18000
	1 1/2"	PAB1 1/2	10	15	15830	26350	9 1/2	14 1/2	15810	26350
	1"	PAB1H	10	15	15840	26365	9 1/2	14 1/2	15815	26365
	1 1/4"	PAB1H	16	24	32710	54515	15	22 1/2	32710	54515
	1 1/2"	PAB1H	9	13 1/2	12610	22680	8	12	12495	20820
	1 3/4"	PAB1H	12	18	19920	33200	11	16 1/2	19920	33200
Seismic	1/2"	PAB1	5	7 1/2	4320	6175	4 1/2	7	4320	6175
	5/8"	PAB1 1/2	6	10	6880	9830	6	9	6880	9830
	3/4"	PAB1 1/2	7 1/2	11 1/2	9060	12940	7	10 1/2	8945	12780
	1"	PAB1 1/2	9	13 1/2	11905	17010	8 1/2	13	11970	17100
	1 1/4"	PAB1 1/2	10 1/2	16	14060	20985	9 1/2	14 1/2	14060	20985
	1 1/2"	PAB1 1/2	14 1/2	22	25310	36715	13 1/2	20 1/2	24650	35715
	1"	PAB1H	16	24	29090	41555	15	22 1/2	29090	41555
	1 1/4"	PAB1H	11	16 1/2	15995	22850	10 1/2	16	16435	23480
	1 1/2"	PAB1H	12	18	18445	26350	11 1/2	17 1/2	18445	26350
	1 3/4"	PAB1H	17	25 1/2	32045	47205	16	24	32720	46180

How to specify and order:

When calling out PAB anchor bolts, substitute the desired length for the "XX" in the Root Model Number.

For a 1/2"x16" anchor bolt, the model number would be PAB1-16 (or PAB1H-16 for high strength).

1. Plate washers are designed to develop the capacity of the bolt.



Design loads are calculated using a full shear cone. Coverage on each side of the bolt shall be a minimum of 2" or reductions must be taken.

Naming Scheme:

PAB1H-12
PAB Anchor Bolt
Diameter and Grade
Length (12", 16", 24", 30" or 36")
* Units in 1/4" increments (Ex: 9 = 9/16" or 1 1/8")

1. Anchorage designs conform to ACI 318-11 Appendix D and assume cracked concrete with no supplementary reinforcement.
2. Seismic indicates Seismic Design Category C through F. Detached one- and two-family dwellings in SDC C may use wind anchorage solutions. Seismic anchorage designs conform to ACI 318-11 Section 13.3.4.
3. Wind includes Seismic Design Category A and B.
4. Foundation dimensions are for anchorage only. Foundation design (size and reinforcement) by Designer. The registered design professional may specify alternate embedment, footing size, and anchor bolt.
5. Allowable Stress Design (ASD) values are obtained by multiplying Load Factor Resistance Design (LRF D) capacities by 0.7 for Seismic and 0.6 for Wind.

MSTC48B3/MSTC66B3 Pre-Bent Straps

The MSTC48B3 and MSTC66B3 are pre-bent straps designed to transfer tension load from an upper story shearwall to a beam on the story below.

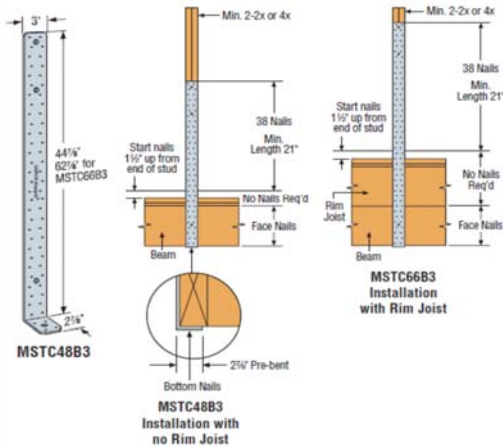
MATERIAL: 14 gauge

FINISH: Galvanized

CODES: See page 13 for Code Reference Key Chart.

Model No.	Dimensions		Fasteners			Allowable Tension Loads		Code Ref.
	Width (min)	Depth (min)	Face	Bottom	Studs/Post	DF/SP (160)	SPF/HF (160)	
MSTC48B3	3	9 1/4	12-10d		4-10d	3930	3380	F26
MSTC66B3	3 1/2	11 1/4	14-10d		38-10d	4440	3820	

- Using fewer than 38 nails in the studs/post will reduce the capacity of the connection. To calculate a reduced capacity use 129 lbs. per nail for DFL/SYP or 112 lbs. per nail for HF/SPF.
- Nails in studs/post shall be installed symmetrically. Nails may be installed over the entire length of the strap over the studs/post.
- The 3" wide beam may be double 2x members.
- MSTC48B3 and MSTC66B3 installed over wood structural panel sheathing up to 1/2" thick achieve 0.85 of table loads.
- Loads governed by the lower of .125" deflection from static tests on wood members, steel ultimate divided by 2, or the calculated nail values.
- NAILS:** 16d = 0.162" dia. x 3 1/2" long, 10d = 0.148" dia. x 3" long. See page 22-23 for other nail sizes and information.



6) LOAD COMBINATIONS:

The gravity loads prevent the shear wall from overturning or toppling over by creating a resisting moment that opposes the overturning moment. The magnitude of this resisting moment is the summation of moments about the same point that the overturning occurs. The applied loads in the overturning analysis are to be in accordance with the code-required load combinations, which are shown below for reference:

- | | |
|---|----------------------|
| 1. $D + F$ | (IBC Equation 16-8) |
| 2. $D + H + F + L + T$ | (IBC Equation 16-9) |
| 3. $D + H + F + (L_r \text{ or } S \text{ or } R)$ | (IBC Equation 16-10) |
| 4. $D + H + F + 0.75(L + T) + 0.75 (L_r \text{ or } S \text{ or } R)$ | (IBC Equation 16-11) |
| 5. $D + H + F + (W \text{ or } 0.7E)$ | (IBC Equation 16-12) |
| 6. $D + H + F + 0.75(W \text{ or } 0.7E) + 0.75L + 0.75(L_r \text{ or } S \text{ or } R)$ | (IBC Equation 16-13) |
| 7. $0.6D + W + H$ | (IBC Equation 16-14) |
| 8. $0.6D + 0.7E + H$ | (IBC Equation 16-15) |

It can be seen by inspection that the first four combinations are eliminated for shear wall analysis and design since they do not include any lateral load terms. It can also be seen by inspection that the combination $D + H + F + 0.75(W \text{ or } 0.7E) + 0.75L + 0.75(L_r \text{ or } S \text{ or } R)$ will produce the worst-case compression chord force, and the combinations $0.6D + W + H$ or $0.6D + 0.7E + H$ will produce the worst-case tension chord forces. Since these combinations all include either wind or seismic loads, the load duration factor for the member designs will be $C_D = 1.6$. For simplicity, the following terms will not be considered since they do not typically apply to shear walls: H (earth pressure), F (fluid pressure), and T (temperature and shrinkage change).

NDS-Shear Wall chord capacity calculations

F _C =	1350	psi	DF Larch #2
E =	1600000	psi	DF Larch #2
E _{min} =	580000	psi	DF Larch #2
C _F =	1.00		
C _M =	1.00		
C _t =	1.00		
C _i =	1.00		
C _T =	1.00		
C _D =	1.60		
K _e =	1.00		

$$C_P := \left(\frac{1 + \alpha_c}{2 \cdot c} \right) - \sqrt{\left(\frac{1 + \alpha_c}{2 \cdot c} \right)^2 - \frac{\alpha_c}{c}}$$

$c =$ 0.8 For sawn lumber

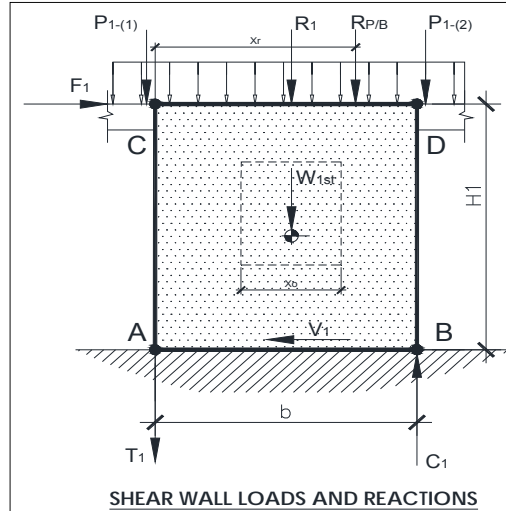
Floor:	Chords	P _{load} Max (compression)	Width (in)	Depth (in)	Area (in ²)	Length (ft)	I _e = K _e *Length(in)	E' _{min} = E _{min} *CM* ³ Ct*Ci* CD, (psi)	F _{cd} = 0.822* E'/min/ (le/depth) ² , (psi)	F* _c =F _c *CM* Ct*Ci*CF*CD	$\alpha_c = \frac{F_c E}{F_{c_star}}$	C _p	F' _c =F _c *CM*Ct *Ci*CF*CD*C _p (psi)	P _{load} Max = F' _c *Area (lbs)
3rd floor	2x4	#N/A	1.5	3.5	5.25	#N/A	#N/A	928000	#N/A	2160.00	#N/A	#N/A	#N/A	#N/A
3rd floor	2x6	#N/A	1.5	5.5	8.25	#N/A	#N/A	928000	#N/A	2160.00	#N/A	#N/A	#N/A	#N/A
3rd floor	2x8	#N/A	1.5	7.5	11.25	#N/A	#N/A	928000	#N/A	2160.00	#N/A	#N/A	#N/A	#N/A
3rd floor	(2) 2x4	#N/A	3	3.5	10.5	#N/A	#N/A	928000	#N/A	2160.00	#N/A	#N/A	#N/A	#N/A
3rd floor	(2) 2x6	#N/A	3	5.5	16.5	#N/A	#N/A	928000	#N/A	2160.00	#N/A	#N/A	#N/A	#N/A
3rd floor	(2) 2x8	#N/A	3	7.5	22.5	#N/A	#N/A	928000	#N/A	2160.00	#N/A	#N/A	#N/A	#N/A
3rd floor	(3) 2x4	#N/A	4.5	3.5	15.75	#N/A	#N/A	928000	#N/A	2160.00	#N/A	#N/A	#N/A	#N/A
3rd floor	(3) 2x6	#N/A	4.5	5.5	24.75	#N/A	#N/A	928000	#N/A	2160.00	#N/A	#N/A	#N/A	#N/A
3rd floor	(3) 2x8	#N/A	4.5	7.5	33.75	#N/A	#N/A	928000	#N/A	2160.00	#N/A	#N/A	#N/A	#N/A
3rd floor	(4) 2x4	#N/A	6	3.5	21	#N/A	#N/A	928000	#N/A	2160.00	#N/A	#N/A	#N/A	#N/A
3rd floor	(5) 2x4	#N/A	7.5	3.5	26.25	#N/A	#N/A	928000	#N/A	2160.00	#N/A	#N/A	#N/A	#N/A
3rd floor	(4) 2x6	#N/A	6	5.5	33	#N/A	#N/A	928000	#N/A	2160.00	#N/A	#N/A	#N/A	#N/A
3rd floor	(4) 2x8	#N/A	6	7.5	45	#N/A	#N/A	928000	#N/A	2160.00	#N/A	#N/A	#N/A	#N/A
3rd floor	N/A													
2nd floor	2x4	3,162	1.5	3.5	5.25	10	120	928000	648.92	2160.00	0.30	0.28	602.34	3,162
2nd floor	2x6	10,351	1.5	5.5	8.25	10	120	928000	1602.44	2160.00	0.74	0.58	1254.68	10,351
2nd floor	2x8	19,175	1.5	7.5	11.25	10	120	928000	2979.75	2160.00	1.38	0.79	1704.42	19,175
2nd floor	(2) 2x4	6,325	3	3.5	10.5	10	120	928000	648.92	2160.00	0.30	0.28	602.34	6,325
2nd floor	(2) 2x6	20,702	3	5.5	16.5	10	120	928000	1602.44	2160.00	0.74	0.58	1254.68	20,702
2nd floor	(2) 2x8	38,349	3	7.5	22.5	10	120	928000	2979.75	2160.00	1.38	0.79	1704.42	38,349
2nd floor	(3) 2x4	9,487	4.5	3.5	15.75	10	120	928000	648.92	2160.00	0.30	0.28	602.34	9,487
2nd floor	(3) 2x6	31,053	4.5	5.5	24.75	10	120	928000	1602.44	2160.00	0.74	0.58	1254.68	31,053
2nd floor	(3) 2x8	57,524	4.5	7.5	33.75	10	120	928000	2979.75	2160.00	1.38	0.79	1704.42	57,524
2nd floor	(4) 2x4	12,649	6	3.5	21	10	120	928000	648.92	2160.00	0.30	0.28	602.34	12,649
2nd floor	(5) 2x4	15,811	7.5	3.5	26.25	10	120	928000	648.92	2160.00	0.30	0.28	602.34	15,811
2nd floor	(4) 2x6	41,404	6	5.5	33	10	120	928000	1602.44	2160.00	0.74	0.58	1254.68	41,404
2nd floor	(4) 2x8	76,699	6	7.5	45	10	120	928000	2979.75	2160.00	1.38	0.79	1704.42	76,699
2nd floor	N/A													
1st floor	2x4	3,162	1.5	3.5	5.25	10	120	928000	648.92	2160.00	0.30	0.28	602.34	3,162
1st floor	2x6	10,351	1.5	5.5	8.25	10	120	928000	1602.44	2160.00	0.74	0.58	1254.68	10,351
1st floor	2x8	19,175	1.5	7.5	11.25	10	120	928000	2979.75	2160.00	1.38	0.79	1704.42	19,175
1st floor	(2) 2x4	6,325	3	3.5	10.5	10	120	928000	648.92	2160.00	0.30	0.28	602.34	6,325
1st floor	(2) 2x6	20,702	3	5.5	16.5	10	120	928000	1602.44	2160.00	0.74	0.58	1254.68	20,702
1st floor	(2) 2x8	38,349	3	7.5	22.5	10	120	928000	2979.75	2160.00	1.38	0.79	1704.42	38,349
1st floor	(3) 2x4	9,487	4.5	3.5	15.75	10	120	928000	648.92	2160.00	0.30	0.28	602.34	9,487
1st floor	(3) 2x6	31,053	4.5	5.5	24.75	10	120	928000	1602.44	2160.00	0.74	0.58	1254.68	31,053
1st floor	(3) 2x8	57,524	4.5	7.5	33.75	10	120	928000	2979.75	2160.00	1.38	0.79	1704.42	57,524
1st floor	(4) 2x4	12,649	6	3.5	21	10	120	928000	648.92	2160.00	0.30	0.28	602.34	12,649
1st floor	(5) 2x4	15,811	7.5	3.5	26.25	10	120	928000	648.92	2160.00	0.30	0.28	602.34	15,811
1st floor	(4) 2x6	41,404	6	5.5	33	10	120	928000	1602.44	2160.00	0.74	0.58	1254.68	41,404
1st floor	(4) 2x8	76,699	6	7.5	45	10	120	928000	2979.75	2160.00	1.38	0.79	1704.42	76,699
1st floor	N/A													

XXXX & ASSOCIATES

XXXX W. IMPERIAL HWY
INGLEWOOD, CA 90303
(310) 722-XXXX

Client: WEST 112TH STREET
Job Address: WEST 112TH STREET
LOS ANGELES COUNTY, CA 90304
Item: Type C building Calculations

Date: 9/3/2013
By: RS
Chk'd: F.K.L

Shearwalls Calculations:

Shear Wall ID/number:	2-FR-NS-2	2-FR-EW-2	1-FR-NS-2	1-FR-NS-3
Bottom of Shear Wall level:	2nd floor	2nd floor	1st floor	1st floor
Top of Shear Wall level:	Roof	Roof	2nd floor	2nd floor
As, Trib. floor area seismic (ft2)	710.0	710.0	710.0	300.0
Seismic Load from floor area (not adjusted for H/b), (lbs)	5,385.0	5,385.0	3,000.0	1,300.0
Seismic Load (Shear) from walls on floor above, (lbs)	0.0	0.0	0.0	1,450.0
Wind Load (Shear) from walls on floor above, (lbs)	0.0	0.0		5,385.0
Total Seismic Load (not adjusted for H/b), (lbs)	5,385.0	5,385.0	3,000.0	2,750.0
Aw, E-W wind projected Area (ft2)	230.0	170.0	300.0	90.0
F1, Wind Load (lbs)	4,955.4	3,662.7	6,463.5	7,324.1
Wall location: interior/exterior?	Exterior	Exterior	Interior	Exterior
Wall length, b (ft)	23.0	18.5	15.3	26.4
WD 1, Floor Uniform Dead Load (lbs/ft)	120.0	120.0	30.0	30.0
WL1, Floor Uniform Live Load (lbs/ft)	160.0	160.0	30.0	30.0
P 1-1-Dead (header) (lbs)	0.0	0.0	0.0	0.0
P 1-1-Live (header) (lbs)	0.0	0.0	0.0	0.0
P 1-2-Dead (header) (lbs)	0.0	0.0	0.0	0.0
P 1-2- Live (header) (lbs)	0.0	0.0	0.0	0.0
P p/b, Dead Load from post/beam (lbs)	0.0	0.0	0.0	0.0
P p/b, Live Load from post/beam (lbs)	0.0	0.0	0.0	0.0
Xr, post/beam location from A (ft)	10.0	10.0	10.0	10.0
Wall height, H1 (ft)	10.0	10.0	10.0	10.0
H/b < 3.5 ?	Yes, OK	Yes, OK	Yes, OK	Yes, OK
H/b > 2 for seismic, multiply V by 2b/H?	No	No	No	No
F1 seismic Seismic Load, adjusted for H/b (lbs)	5,385	5,385	3,000	2,750
Wsw, Wall self weight= (psf)	18.0	18.0	7.0	18.0
Wall opening width (if any), xo (ft)	8.0	4.6	0.0	9.0
Wall opening height (if any), yo (ft)	4.0	4.0	0.0	4.0

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W _{1st} , Wall self weight = $b \cdot H_1 \cdot W_{sw}$, (lb)	3,564.0	2,998.8	1,071.0	4,104.0
F ₁ , floor shear (V ₁) = max. (seismic/wind)	5,385.0	5,385.0	6,463.5	7,324.1
b _e , effective shear resist. Width (ft)	15.0	13.9	15.3	17.4
Unit shear, $v_1 = V_1 / b$ effective (lbs/ft)	359.0	387.4	422.5	420.9
Shear Wall Type needed, (from plywood schedule, using blocked walls only)	4.0	8.0	8.0	8.0
Override Shear Wall Type? If yes, write wall type below:				
R _{1 Dead} = W _{D1} * b; (lbs)	2,760.0	2,220.0	459.0	792.0
R _{1 Live} = W _{L1} * b; (lbs)	3,680.0	2,960.0	459.0	792.0
OTM _{1, Wind} = F _{1 Wind} * H ₁ (lb*ft)	49,553.6	36,626.5	64,635.1	73,240.5
OTM _{1, Seismic} = F _{1 Seismic} * H ₁ (lb*ft)	53,850.0	53,850.0	30,000.0	27,500.0
RM _{R1} = R _{1 Dead} * b/2, DL resisting moment (lb*ft)	31,740.0	20,535.0	3,511.4	10,454.4
RM _{P1} = Min (P ₁) * b, DL minimum resisting moment (lb*ft)	0.0	0.0	0.0	0.0
RM _{W1} = W _{1st} * b/2, Wall self weight resisting moment (lb*ft)	40,986.0	27,738.9	8,193.2	54,172.8
RM _{P/B1} = W _{P/B Dead} * min(Xr, b-Xr), post/beam DL resisting moment (lb*ft)	0.0	0.0	0.0	0.0
RM _{DL} , sum of DL resisting moments (lb*ft)	72,726.0	48,273.9	11,704.5	64,627.2
RM _{R1 Live} = R _{1 Live} * b/2, LL resisting moment (lb*ft)	42,320.0	27,380.0	3,511.4	10,454.4
RM _{P1} = Min (P ₁) * b, LL minimum resisting moment (lb*ft)	0.0	0.0	0.0	0.0
RM _{P/B1} = W _{P/B Live} * min(Xr, b-Xr), post/beam LL resisting moment (lb*ft)	0.0	0.0	0.0	0.0
RM _{LL} , sum of LL resisting moments (lb*ft)	42,320.0	27,380.0	3,511.4	10,454.4
Chord Forces: Tension Case, Wind load; governing combo: 0.6D + W: T ₁ = (OTM _{1, Wind} - 0.6 * RM _{DL}) / b; (lbs)	257.3	414.2	3,765.5	1,305.5
Chord Forces: Tension Case, Seismic load; governing combo: 0.6D + 0.7E: T ₁ = (0.7 * OTM _{1, Seismic} - 0.6 * RM _{DL}) / b; (lbs)	-258.3	471.9	913.5	-739.6
Chord Forces: Compression Case, Wind load; governing combo: D + 0.75W + 0.75L: C ₁ = (0.75 * OTM _{1, Wind} + RM _{DL} + 0.75 * RM _{LL}) / b; (lbs)	6,157.9	5,204.3	4,105.5	4,825.7
Chord Forces: Compression Case, Seismic load; governing combo: D + 0.75(0.7E + L): C ₁ = (0.525 * OTM _{1, Seismic} + RM _{DL} + 0.75 * RM _{LL}) / b; (lbs)	5,771.2	5,247.6	1,966.5	3,291.9
Floor to floor uplift; (lbs), NEGATIVE NUMBER MEANS NO UPLIFT.	257.3	471.9	3,765.5	1,305.5
Floor to Floor Strap Symbol, from strap schedule (if applicable)	1	1		

XXXX & ASSOCIATES

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Item: Type C building Calculations

Date: 9/3/2013
By: RS
Chk'd: F.K.L

Floor to Floor Strap Call Out, from strap schedule	LFTA-1.5	LFTA-1.5		
Override Floor to Floor Strap Call Out?				
Select Chord Size	(2) 2x4	(2) 2x4	(2) 2x4	(2) 2x4
Maximum Chord Compression Capacity (lbs)	6,325	6,325	6,325	6,325
Maximum Chord Tension Capacity (lbs)	4,830	4,830	4,830	4,830
Chord Size Compression code check?	OK	OK	OK	OK
Chord Size Tension code check?	OK	OK	OK	OK
Floor To Foundation hold down designation, from hold down schedule			HDU4-SDS2.5	HDU2-SDS2.5
Floor To Foundation hold down type, from hold down schedule			4	2
Floor To Foundation anchor bolt, from anchor bolt schedule			SSTB 24	SSTB 20
Override Floor To Foundation anchor bolt?			PBA5-18	PBA5-18
Sill anchor bolt type, from anchor bolt schedule			3	3
Override Sill anchor bolt type?				
A_{chord} , Chord Area (in ²)	10.50	10.50	10.50	10.50
$\Delta b = 8v_1H_1^3 / EAb$, (in)	0.01	0.01	0.01	0.01
Gt, panel rigidity through the thickness in lb/in	25,000.0	25,001.0	25,000.0	25,000.0
$\Delta v = v_1H_1 / Gt$, (in)	0.14	0.15	0.17	0.17
Nail spacing, Sn	6	7	4	6
Nail size	10d	10d	10d	10d
Load per nail (lbs) = $v_1 / (12/Sn)$	180	226	141	210
e_n , nail deformation TABLE 2305.2.2(1), (in)	0.01	0.01	0.01	0.01
$\Delta n = 0.75 * h * e_n$, (in)	0.075	0.075	0.075	0.075
Δa , (in)	0.125	0.125	0.125	0.125
Δs = story drift, (in)	0.35	0.36	0.38	0.38
Is Δs smaller than $0.02 * H$?	OK	OK	OK	OK

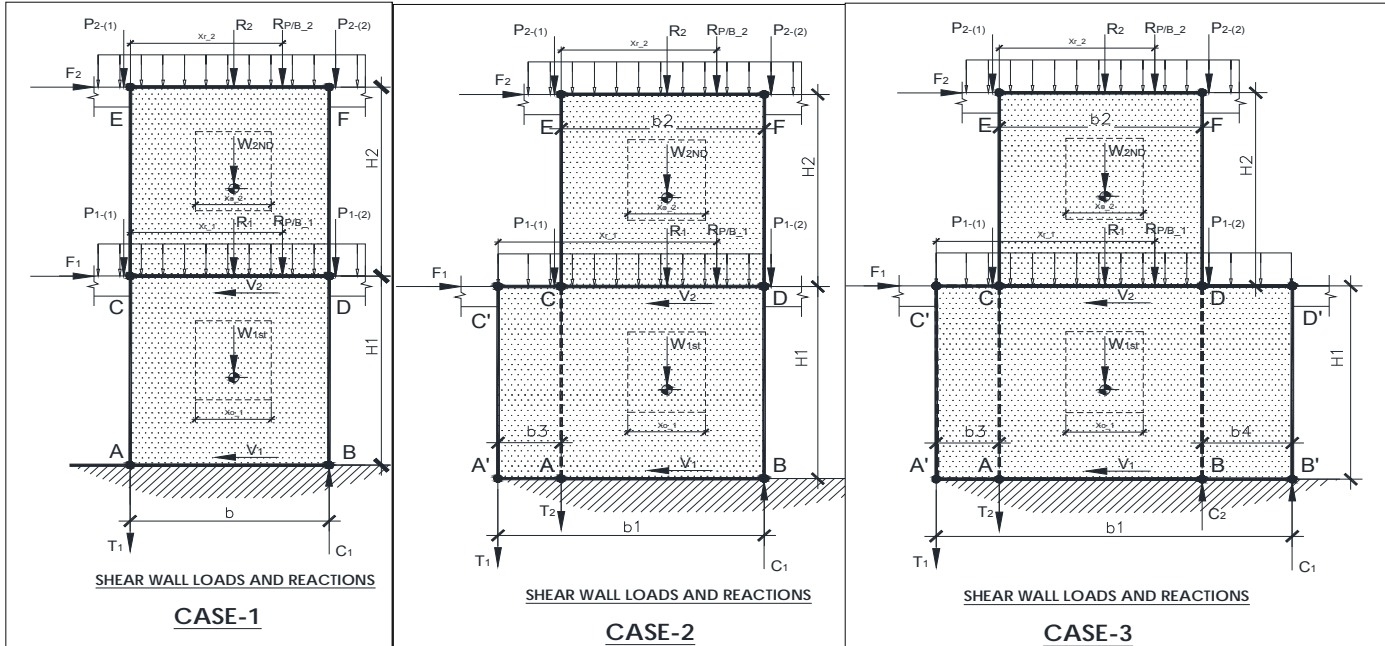
XXXX & ASSOCIATES

XXXX W. IMPERIAL HWY
INGLEWOOD, CA 90303
(310) 722-XXXX

Date: 9/3/2013
By: RS
Chk'd: F.K.L

Client: WEST 112TH STREET
Job Address: WEST 112TH STREET
LOS ANGELES COUNTY, CA 90304
Item: Type C building Calculations

Shearwalls Calculations:



Shear Wall ID/number:	1-FR-NS-1/2-FR-NS-1	1-FR-EW-1/2-FR-EW-1	
Level 2, Bottom of Shear Wall level:	2nd floor	2nd floor	
Level 2, Top of Shear Wall level:	Roof	Roof	
Left shear wall chord aligned?	Yes	Yes	
Right shear wall chord aligned?	Yes	Yes	
Distance A-A'(if chords are not aligned)			
Distance B-B'(if chords are not aligned)			
As2,Trib. floor area seismic (ft2)	710.0	710.0	
F2 seismic,Seismic Load from floor area (not adjusted for H/b), (lbs)	5,385.0	5,385.0	
SL2,Seismic Load (Shear) from walls on floor above, (lbs)	0.0	0.0	
Level 2,Total Seismic Load(not adjusted for H/b), (lbs)	5,385.0	5,385.0	
Level 2, Aw, E-W wind projected Area (ft2)	230.0	165.0	
F2,wind Load (lbs)	4,955.4	3,554.9	
Level 2, Wall location: interior/exterior?	Exterior	Exterior	
Level 2, Wall length, b2 (ft)	20.0	16.0	
WD 2, Floor Uniform Dead Load (lbs/ft)	0.0	0.0	
WL2, Floor Uniform Live Load (lbs/ft)	0.0	0.0	
P 2-1-Dead (header) (lbs)	0.0	50.0	
P 2-1-Live (header) (lbs)	0.0	50.0	
P 2-2-Dead (header) (lbs)	0.0	50.0	
P 2-2- Live (header) (lbs)	0.0	50.0	
P p/b 2, Dead Load from post/beam (lbs)	0.0	150.0	
P p/b 2, Live Load from post/beam (lbs)	0.0	300.0	
Xr 2, post/beam location from A (ft)	10.2	10.2	
Wall height, H2 (ft)	10.0	10.0	
H2/b2 <3.5 ?	Yes, OK	Yes, OK	
H2/b2 >2 for seismic, multiply V by 2b/H?	No	No	
F2 seismic, Seismic Load, adjusted for H/b (lbs)	5,385	5,385	
W2sw, Wall self weight= (psf)	18.0	18.0	
Wall opening width (if any), xo 2 (ft)	5.0	3.8	
Wall opening height (if any), yo 2 (ft)	4.0	4.0	

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Date:
By:
Chk'd:

9/3/2013
RS
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W2nd, Wall self weight= $b2 \cdot H2 \cdot W2sw$, (lb)	3,240.0	2,606.4	
F2, floor shear (V2)=max. (seismic/wind)	5,385.0	5,385.0	
be2, effective shear resist. Width (ft)	15.0	12.2	
Unit shear, $v2=V2/be2$ (lbs/ft)	359.0	441.4	
Level 2, Shear Wall Type needed, (from plywood schedule)	4.0	8.0	
Override Shear Wall Type? If yes, write wall type below:			
$R2_{Dead} = W_{D2} \cdot b2$; (lbs)	0.0	0.0	
$R2_{Live} = W_{L2} \cdot b2$; (lbs)	0.0	0.0	
Level 2, $OTM2_{Wind} = F2_{Wind} \cdot H2$ (lb*ft)	49,553.6	35,549.3	
Level 2, $OTM2_{Seismic} = F2_{Seismic} \cdot H2$ (lb*ft)	53,850.0	53,850.0	
$RM_{R2} = R2_{Dead} \cdot b2/2$, DL resisting moment (lb*ft)	0.0	0.0	
$RM_{P2} = \text{Min}(P2) \cdot b2$, DL minimum resisting moment (lb*ft)	0.0	800.0	
$RM_{W2} = W2nd \cdot b2/2$, Wall self weight resisting moment (lb*ft)	32,400.0	20,851.2	
$RM_{P/B2} = W_{P/B_Dead} \cdot \text{min}(Xr, b2-Xr)$, post/beam DL resisting moment (lb*ft)	0.0	870.0	
RM_{D2} , sum of DL resisting moments (lb*ft)	32,400.0	22,521.2	
$RM_{R2_Live} = R2_{Live} \cdot b2/2$, LL resisting moment (lb*ft)	0.0	0.0	
$RM_{P2} = \text{Min}(P2) \cdot b2$, LL minimum resisting moment (lb*ft)	0.0	800.0	
$RM_{P/B2} = W_{P/B_Live} \cdot \text{min}(Xr, b2-Xr)$, post/beam LL resisting moment (lb*ft)	0.0	1,740.0	
RM_{L2} , sum of LL resisting moments (lb*ft)	0.0	2,540.0	
Level 2, Chord Forces: Tension Case, Wind load; governing combo: 0.6D + W: $T2=(OTM2_{Wind}-0.6 \cdot RM_{D1})/b2$; (lbs)	1,505.7	1,377.3	
Level 2, Chord Forces: Tension Case, Seismic load; governing combo: 0.6D + 0.7E: $T2=(0.7 \cdot OTM2_{Seismic}-0.6 \cdot RM_{D2})/b2$; (lbs)	912.8	1,511.4	
Level 2, Chord Forces: Compression Case, Wind load; governing combo: D + 0.75W+0.75L: $C2=(0.75 \cdot OTM2_{Wind}+RM_{D2}+0.75 \cdot RM_{L2})/b2$; (lbs)	3,478.3	3,193.0	
Level 2, Chord Forces: Compression Case, Seismic load; governing combo: D + 0.75(0.7E+L): $C2=(0.525 \cdot OTM2_{Seismic}+RM_{D2}+0.75 \cdot RM_{L2})/b2$; (lbs)	3,033.6	3,293.6	
Level 2, Floor to floor uplift; (lbs), NEGATIVE NUMBER MEANS NO UPLIFT.	1,505.7	1,511.4	
Level 2, Floor to Floor Strap Symbol, from strap schedule (if applicable)	2	2	
Level 2, Floor to Floor Strap Call Out, from strap schedule	FTA2-1.5	FTA2-1.5	
Override Floor to Floor Strap Call Out?			
Level 2, Select Chord Size	(3) 2x4	(2) 2x4	
Level 2, Maximum Chord Capacity	9,487	6,325	
Level 2, Chord Size code check?	OK	OK	
Level 2, Floor To Foundation hold down designation, from hold down schedule	HDU2-SDS2.5	HDU2-SDS2.5	
Level 2, Floor To Foundation hold down type, from hold down schedule	2	2	
Level 2, Floor To Foundation anchor bolt, from anchor bolt schedule	SSTB 20	SSTB 20	

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Level 2, Override Floor To Foundation anchor bolt?	Not Applicable at this level	Not Applicable at this level	
Level 2, Sill anchor bolt type, from anchor bolt schedule	2	3	
Level 2, Override Sill anchor bolt type?	Not Applicable at this level	Not Applicable at this level	
Level 2, A_{chord} , Chord Area (in ²)	15.75	10.50	
Level 2, $\Delta b = 8 \cdot v_2 \cdot H^2 / E \cdot a \cdot b^2$, (in)	0.01	0.01	
Level 2, Gt, panel rigidity through the thickness in lb/in	25,000.0	25,000.0	
Level 2, $\Delta v = v_2 H^2 / Gt$, (in)	0.14	0.18	
Level 2, Nail spacing, Sn	4	4	
Level 2, Nail size	8d	10d	
Level 2, Load per nail (lbs) = $v_2 / (12 / Sn)$	120	147	
Level 2, en, nail deformation TABLE 2305.2.2(1), (in)	0.01	0.01	
Level 2, $\Delta n = 0.75 \cdot h \cdot e_n$, (in)	0.075	0.075	
Level 2, Δa , (in)	0.125	0.125	
Level 2, Δs = story drift, (in)	0.35	0.39	
Level 2, Is Δs smaller than $0.02 \cdot H$?	OK	OK	

Level 1, Bottom of Shear Wall level:	1st floor	1st floor	
Level 1, Top of Shear Wall level:	2nd floor	2nd floor	
As1, Trib. floor area seismic (ft ²)	265.0	710.0	
Level 1, F1 seismic, Seismic Load from floor area (not adjusted for H/b), (lbs)	1,080.0	2,900.0	
Level 1, SL1, Seismic Load (Shear) from other walls on floor above, (lbs)	0.0	0.0	
Level 1, Total Seismic Load (not adjusted for H/b), (lbs)	1,080.0	2,900.0	
Level 1, Aw, E-W wind projected Area (ft ²)	130.0	140.0	
Level 1, F1, Wind Load (lbs)	2,800.9	3,016.3	
Level 1, Wall location: interior/exterior?	Exterior	Exterior	
Level 1, Wall length, b1 (ft)	20.0	20.4	
Level 1, WD 1, Floor Uniform Dead Load (lbs/ft)	0.0	0.0	
Level 1, WL1, Floor Uniform Live Load (lbs/ft)	0.0	0.0	
Level 1, P 1-1-Dead (header) (lbs)	50.0	50.0	
Level 1, P 1-1-Live (header) (lbs)	50.0	50.0	
Level 1, P 1-2-Dead (header) (lbs)	50.0	50.0	
Level 1, P 1-2- Live (header) (lbs)	50.0	50.0	
Level 1, P p/b 1, Dead Load from post/beam (lbs)	0.0	1,000.0	
Level 1, P p/b 1, Live Load from post/beam (lbs)	0.0	2,500.0	
Level 1, Xr_1, post/beam location from A (ft)	0.0	10.2	
Level 1, Wall height, H1 (ft)	10.0	10.0	
Level 1, $(H1+H2)/b1 < 3.5$?	Yes, OK	Yes, OK	
Level 1, $(H1+H2)/b1 > 2$ for seismic, multiply V by $2b/H$?	No	No	
Level 1, F1 seismic, Seismic Load, adjusted for H/b (lbs)	1,080	2,900	
Level 1, W1sw, Wall self weight = (psf)	18.0	18.0	
Level 1, Wall opening width (if any), xo_1 (ft)	7.3	0.0	
Level 1, Wall opening height (if any), yo_1 (ft)	4.0	0.0	
Level 1, W1st, Wall self weight = $b1 \cdot H1 \cdot W1sw$, (lb)	3,074.4	3,672.0	
Level 1, F1, floor shear (V1) = max. (seismic/wind)	2,800.9	3,016.3	
Level 1, be1, effective shear resist. Width (ft)	12.7	20.4	

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Level 1, Unit shear, $v1=(V1+V2)/be1$ (lbs/ft)	644.6	411.8	
Level 1, Shear Wall Type needed, (from plywood schedule)	10.0	8.0	
Level 1, Override Shear Wall Type? If yes, write wall type below:			
Level 1, $R1_{Dead} = W_{D1} * b1$; (lbs)	0.0	0.0	
Level 1, $R1_{Live} = W_{L1} * b1$; (lbs)	0.0	0.0	
Level 1, $OTM2_{Wind} = F2_{wind} * (H1+H2)$ (lb*ft)	99,107.1	71,098.6	
Level 1, $OTM2_{Seismic} = F2_{seismic} * (H1+H2)$, (lb*ft)	107,700.0	107,700.0	
Level 1, $OTM1_{Wind} = (F1_{wind} * H1) + F2_{wind} * (H1+H2)$, (lb*ft)	127,115.6	101,261.6	
Level 1, $OTM1_{Seismic} = (F1_{seismic} * H1) + F2_{seismic} * (H1+H2)$ (lb*ft)	118,500.0	136,700.0	
Level 1, $RM_{R1} = R1_{Dead} * b1/2$, DL resisting moment (lb*ft)	0.0	0.0	
Level 1, $RM_{P1} = \text{Min}(P1) * b1$, DL minimum resisting moment (lb*ft)	1,000.0	1,020.0	
Level 1, $RM_{W1} = W1 * b1/2$, Wall self weight resisting moment (lb*ft)	30,744.0	37,454.4	
Level 1, $RM_{P/B1} = W_{P/B_{Dead}} * \text{min}(Xr, b1 - Xr)$, post/beam DL resisting moment (lb*ft)	0.0	10,200.0	
Level 1, RM_{D1} , sum of DL resisting moments (lb*ft)	64,144.0	71,195.6	
Level 1, $RM_{R1_{Live}} = R1_{Live} * b1/2$, LL resisting moment (lb*ft)	0.0	0.0	
Level 1, $RM_{P1} = \text{Min}(P1) * b1$, LL minimum resisting moment (lb*ft)	1,000.0	1,020.0	
Level 1, $RM_{P/B1} = W_{P/B_{Live}} * \text{min}(Xr, b1 - Xr)$, post/beam LL resisting moment (lb*ft)	0.0	25,500.0	
Level 1, RM_{L1} , sum of LL resisting moments (lb*ft)	1,000.0	29,060.0	
Level 1, Chord Forces: Tension Case, Wind load; governing combo: 0.6D + W: $T1 = (OTM1_{Wind} - 0.6 * RM_{D1}) / b1$; (lbs)	4,431.5	2,869.8	
Level 1, Chord Forces: Tension Case, Seismic load; governing combo: 0.6D + 0.7E: $T1 = (0.7 * OTM1_{Seismic} - 0.6 * RM_{D1}) / b1$; (lbs)	2,223.2	2,596.7	
Level 1, Chord Forces: Compression Case, Wind load; governing combo: D + 0.75W + 0.75L: $C1 = (0.75 * OTM1_{Wind} + RM_{D1} + 0.75 * RM_{L1}) / b1$; (lbs)	8,011.5	8,281.2	
Level 1, Chord Forces: Compression Case, Seismic load; governing combo: D + 0.75(0.7E + L): $C1 = (0.525 * OTM1_{Seismic} + RM_{D1} + 0.75 * RM_{L1}) / b1$; (lbs)	6,355.3	8,076.4	
Level 1, Floor to floor uplift; (lbs), NEGATIVE NUMBER MEANS NO UPLIFT.	4,431.5	2,869.8	
Level 1, Floor to Floor Strap Symbol, from strap schedule (if applicable)	N/A	N/A	
Level 1, Floor to Floor Strap Call Out, from strap schedule	N/A	N/A	
Level 1, Override Floor to Floor Strap Call Out?			
Level 1, Select Chord Size	(3) 2x4	(3) 2x4	
Level 1, Maximum Chord Tension Capacity	9,563	9,563	

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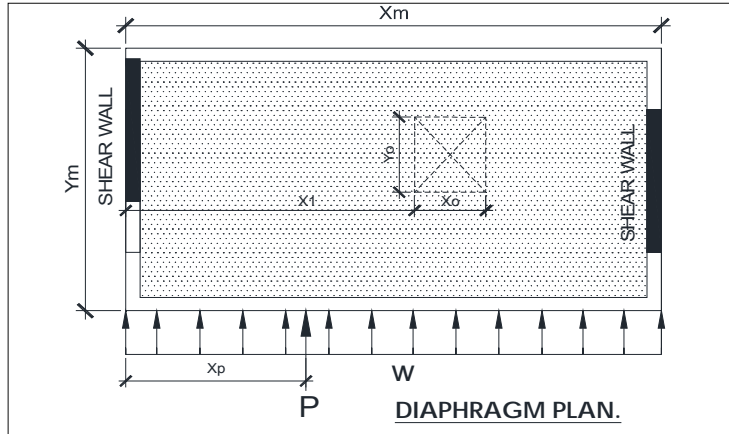
Level 1, Maximum Chord Compression Capacity	9,487	9,487	
Level 1, Chord Size Compression code check?	OK	OK	
Level 1, Chord Size Tension code check?	OK	OK	
Level 1, Floor To Foundation hold down designation, from hold down schedule	HDU4-SDS2.5	HDU2-SDS2.5	
Level 1, Floor To Foundation hold down type, from hold down schedule	4	2	
Level 1, Floor To Foundation anchor bolt, from anchor bolt schedule	SSTB 24	SSTB 20	
Level 1, Override Floor To Foundation anchor bolt?	Use PBA5-18	Use PBA5-18	
Level 1, Sill anchor bolt type, from anchor bolt schedule	4	3	
Level 1, Override Sill anchor bolt type?			
Level 1, A_{chord} , Chord Area (in ²)	15.75	15.75	
Level 1, $\Delta b = 8 * v1 * H1^3 / E * a * b1$, (in)	0.01	0.01	
Level 1, Gt, panel rigidity through the thickness in lb/in	25,000.0	25,000.0	
Level 1, $\Delta v = v1 H1 / Gt$, (in)	0.26	0.16	
Level 1, Nail spacing, Sn	4	4	
Level 1, Nail size	10d	10d	
Level 1, Load per nail (lbs) = $v1 / (12 / Sn)$	215	137	
Level 1, en, nail deformation TABLE 2305.2.2(1), (in)	0.01	0.01	
Level 1, $\Delta n = 0.75 * h * en$, (in)	0.075	0.075	
Level 1, Δa , (in)	0.125	0.125	
Level 1, Δs = story drift, (in)	0.47	0.37	
Level 1, Is Δs smaller than $0.02 * H$?	OK	OK	

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Diaphragm Design:

Diaphragm ID/number:	2-DIA-NS-1		2-DIA-EW-1	1-DIA-NS-1	1-DIA-NS-2	1-DIA-EW-1
Diaphragm level:	Roof		Roof	2nd floor	2nd floor	2nd floor
Xm, (ft)	44.0		30.0	20.0	25.0	30.0
Ym, (ft)	30.0		44.0	30.0	30.0	44.0
Opening width (if any), xo (ft)	0.0		4.0	0.0	4.0	4.0
Opening height (if any), yo (ft)	0.0		8.0	0.0	8.0	8.0
Opening location (if any), x1 (ft)	0.0		1.0	0.0	1.0	1.0
Trib. Floor area seismic, ft2	1,320.0		1,320.0	600.0	750.0	1,320.0
Total Floor seismic load (lbs)	10,770.0		10,770.0	5,770.0	5,770.0	5,770.0
Total Floor Area (ft2)	1,350.0		1,350.0	1,350.0	1,350.0	1,350.0
W _{N-S} SEISMIC, LB/FT	239.3		351.0	128.2	128.2	188.1
P _{seismic} , point load from shear walls from floor above (lbs)	0.0		0.0	0.0	0.0	0.0
P _{wind} , point load from shear walls from floor above (lbs)	0.0		0.0	0.0	0.0	0.0
Point load location Xp (ft)	0.0		0.0	0.0	13.5	0.0
N-S wind projected height (ft)	10.5		10.5	10.0	10.0	10.0
W _{WIND} , lb/ft	226.2		226.2	215.5	215.5	215.5
V = W*1/2 + (P*a/Xm or P*b/Xm), (lbs)	5,265.3		5,265.3	2,154.5	2,693.1	3,231.8
M = W*x ² /8 + P*xp*(Xm-xp)/Xm, (lb-ft)	57,918.7		39,490.0	10,772.5	16,832.1	24,238.2
Ye, effective shear resisting width=Ym-Yo (ft),	30.0		36.0	30.0	22.0	36.0
Max. unit shear=V/Ye, (lbs/ft)	175.5		146.3	71.8	122.4	89.8
Top plate force=M/Ym, (lbs)	1,930.6		897.5	359.1	561.1	550.9
Select Top plate size	(2) 2x4		(2) 2x4	(2) 2x4	(2) 2x4	(2) 2x4
Maximum Chord Capacity	4,830		4,830	4,830	4,830	4,830
Top plate Size code check?	OK		OK	OK	OK	OK
PANEL GRADE	APA Rated		APA Rated	APA Rated	APA Rated	APA Rated
COMMON NAIL SIZE OR STAPLE LENGTH AND GAGE	10d		10d	10d	10d	10d
MINIMUM FASTENER PENETRATION IN FRAMING (inches)	1		1	1	1	1
MINIMUM NOMINAL PANEL THICKNESS (inch)	19/32		19/32	19/32	19/32	19/32
MINIMUM NOMINAL WIDTH OF FRAMING MEMBERS AT ADJOINING PANEL EDGES AND BOUNDARIES (inches)	2		2	2	2	2
DIAPHRAGM BLOCKED/UNBLOCKED	unblocked		unblocked	unblocked	unblocked	unblocked
BLOCKED DIAPHRAGM Fastener spacing (inches)						
CASE	1		3	1	1	1
ALLOWABLE SHEAR (POUNDS PER FOOT)	285		215	285	285	285
Diaphragm shear capacity code check?	OK		OK	OK	OK	OK

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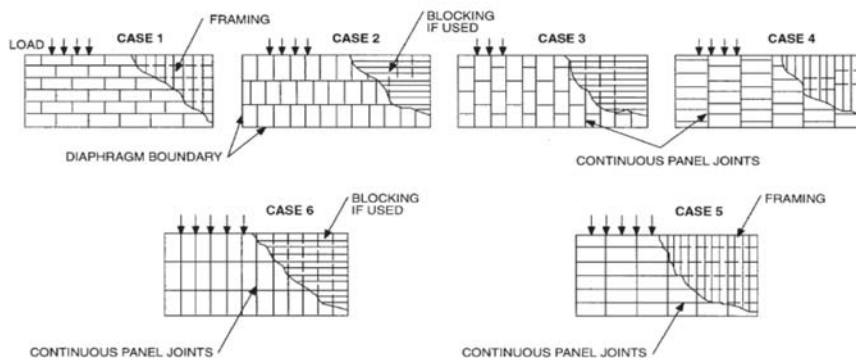
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A_{chord} , Chord Area (in ²)	10.50		10.50	10.50	10.50	10.50
$\Delta b = 5v^3 L^3 / 8EAb$, (in)	0.02		0.00	0.00	0.00	0.00
Gt, panel rigidity through the thickness in lb/in	25,000.0		25,000.0	25,000.0	25,000.0	25,000.0
$\Delta v = vL / 4Gt$, (in)	0.08		0.04	0.01	0.03	0.03
Nail spacing, S_n	6		6	6	6	6
Nail size	10d		10d	10d	10d	10d
Load per nail (lbs) = $vL / (12/S_n)$	88		73	36	61	45
e_n , nail deformation TABLE 2305.2.2(1), (in)	0.01		0.01	0.01	0.01	0.01
$\Delta n = 0.188^* L^* e_n$, (in)	0.08272		0.0564	0.0376	0.047	0.0564
$\Delta c = \Sigma (\Delta cX) / 2b$, (in)	0.100		0.100	0.100	0.100	0.100
Δs = story drift, (in)	0.28		0.20	0.15	0.18	0.19
Is Δs smaller than $0.02^* H$?	OK		OK	OK	OK	OK

ALLOWABLE SHEAR (POUNDS PER FOOT) FOR HORIZONTAL APA PANEL DIAPHRAGMS WITH FRAMING OF DOUGLAS-FIR, LARCH OR SOUTHERN PINE^(a) FOR WIND OR SEISMIC LOADING^(b) (See also IBC Table 2306.3.1)

Panel Grade	Common Nail Size ^(b)	Minimum Nail Penetration in Framing (in.)	Minimum Nominal Panel Thickness (in.)	Adjoining Panel Edges and Boundaries (in.)	Blocked Diaphragms				Unblocked Diaphragms	
					Nail Spacing (in.) at diaphragm boundaries (all cases), at continuous panel edges parallel to load (Cases 3 & 4), and at all panel edges (Cases 5 & 6) ^(b)				Nails Spaced 6" max. at Supported Edges ^(b)	
					Minimum Nominal Width of Framing Member at				Case 1	
					6 4 2-1/2 ^(c) 2 ^(c)				(No unblocked edges or continuous joints parallel to load)	All other configurations (Cases 2, 3, 4, 5 & 6)
APA STRUCTURAL I grades	6d ^(d) (0.113" dia.)	1-1/4	5/16	2 3	185 210	250 280	375 420	420 475	165 185	125 140
	8d (0.131" dia.)	1-3/8	3/8	2 3	270 300	360 400	530 600	600 675	240 265	180 200
	10d ^(d) (0.148" dia.)	1-1/2	15/32	2 3	320 360	425 480	640 720	730 820	285 320	215 240
	6d ^(d) (0.113" dia.)	1-1/4	5/16	2 3	170 190	225 250	335 380	380 430	150 170	110 125
			3/8	2 3	185 210	250 280	375 420	420 475	165 185	125 140
			3/8	2 3	240 270	320 360	480 540	545 610	215 240	160 180
APA RATED SHEATHING; APA RATED STURD-I-FLOOR and other APA grades except Species Group 5	8d (0.131" dia.)	1-3/8	7/16	2 3	255 285	340 380	505 570	575 645	230 255	170 190
			15/32	2 3	270 300	360 400	530 600	600 675	240 265	180 200
			15/32	2 3	290 325	385 430	575 650	655 735	255 290	190 215
	10d ^(d) (0.148" dia.)	1-1/2	19/32	2 3	320 360	425 480	640 720	730 820	285 320	215 240

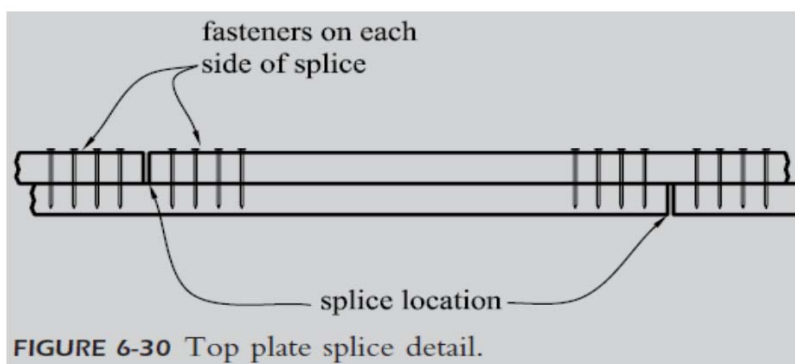


Top Plate Calculations

F_c =	1350 psi	DF Larch #2
F_t =	575 psi	DF Larch #2
C_F =	1.00	
C_M =	1.00	
C_t =	1.00	
C_i =	1.00	
C_T =	1.00	
C_D =	1.60	

As a result of the splicing of the double plates, only one plate is effective in resisting the tension loads, whereas both members are effective in resisting the axial compression force in the top plates (Figure 6.30).

Chords	$P_{load\ Max}$	Width (in)	Depth (in)	Area (in ²) for compression	Area (in ²) for tension	$F'_c = F_c * C_M * C_t * C_i * C_F * C_D * C_p$ (psi)	$P_{load\ Max_comp.} = F'_c * Area$ (lbs)	$F'_t = F_t * C_M * C_t * C_i * C_F * C_D * C_p$ (psi)	$T_{load\ Max_tension} = F'_t * Area$ (lbs)
2x4	2,415	1.5	3.5	5.25	2.625	2160.00	11,340	920.00	2,415
2x6	3,795	1.5	5.5	8.25	4.125	2160.00	17,820	920.00	3,795
2x8	5,175	1.5	7.5	11.25	5.625	2160.00	24,300	920.00	5,175
(2) 2x4	4,830	3	3.5	10.5	5.25	2160.00	22,680	920.00	4,830
(2) 2x6	7,590	3	5.5	16.5	8.25	2160.00	35,640	920.00	7,590
(2) 2x8	10,350	3	7.5	22.5	11.25	2160.00	48,600	920.00	10,350
(3) 2x4	9,563	4.5	3.5	15.75	10.395	2160.00	34,020	920.00	9,563
(3) 2x6	15,028	4.5	5.5	24.75	16.335	2160.00	53,460	920.00	15,028
(3) 2x8	20,493	4.5	7.5	33.75	22.275	2160.00	72,900	920.00	20,493
(4) 2x4	14,490	6	3.5	21	15.75	2160.00	45,360	920.00	14,490
(5) 2x4	18,113	7.5	3.5	26.25	19.6875	2160.00	56,700	920.00	18,113
(4) 2x6	22,770	6	5.5	33	24.75	2160.00	71,280	920.00	22,770
(4) 2x8	31,050	6	7.5	45	33.75	2160.00	97,200	920.00	31,050
N/A	N/A								

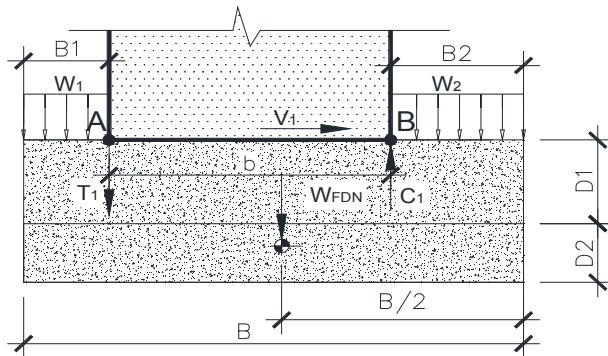
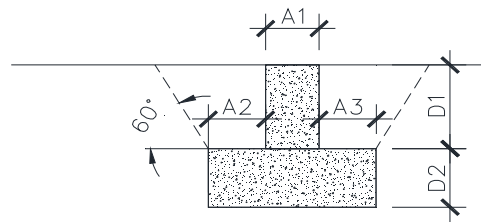


XXXX & ASSOCIATES

XXXX W. IMPERIAL HWY
INGLEWOOD, CA 90303
(310) 722-XXXX

Client: WEST 112TH STREET
Job Address: WEST 112TH STREET
LOS ANGELES COUNTY, CA 90304
Item: Type C building Calculations

Date: 9/3/2013
By: RS
Chk'd: F.K.L

Shear Walls: Footing overturning check:**FOUNDATION ELEVATION****FOUNDATION SECTION**

Shear Wall ID/number:	1-FR-NS-1	1-FR-NS-2	1-FR-NS-3		1-FR-EW-1	1-FR-EW-2
A1, (ft)	1.33	1.33	1.33		1.33	1.33
A2, (ft)	0	0	0		0	0
A3, (ft)	0	0	0		0	0
b, (ft)	20	15.3	26.3		28.7	16.5
B1, (ft)	1	1	1		1	1
B2, (ft)	1	1	1		1	1
D1, (ft)	1	1.67	1		1	1
D2, (ft)	0	0	0		0	0
B, (ft)	22	15.3	28.3		30.7	18.5
Shear Wall uplift, Tu (lbs)	4,500	3,265	785		703	2,175
Overturning Moment from uplift, Omu=Tu*b (lb-ft)	90,000	49,955	20,646		20,176	35,888
W _{ID} , (lbs/ft)	4,200	4,200	1,200		2,200	2,200
W _{2D} , (lbs/ft)	4,200	3,800	1,000		2,000	2,000
W _{concrete wall} (lbs)	200	260	200		200	200
W _{concrete footing} (lbs)	0	0	0		0	0
W _{soil A2 side} (lbs)	0	0	0		0	0
W _{soil A3 side} (lbs)	0	0	0		0	0
W _{foundation total} (lbs)	200	260	200		200	200
Resiting Moment from foundation, RM _{found} =W _{found} *B/2, (lb-ft)	2,200	1,989	2,830		3,070	1,850
Resiting Moment from wall load RM _{w1} , =W _{ID} *(B-B1/2), (lb-ft)	90,300	62,160	33,360		66,440	39,600
Resiting Moment from wall load RM _{w2} , =W _{2D} *(B-B2/2), (lb-ft)	90,300	56,240	27,800		60,400	36,000
Total Resiting Moment RM, =Min(RM _{w1} , RM _{w2})+RM _{found}	92,500	58,229	30,630		63,470	37,850

XXXX & ASSOCIATES

XXXX W. IMPERIAL HWY
 INGLEWOOD, CA 90303
 (310) 722-XXXX

Client: WEST 112TH STREET
Job Address: WEST 112TH STREET
 LOS ANGELES COUNTY, CA 90304
Item: Type C building Calculations

Date: 9/3/2013
By: RS
Chk'd: F.K.L

Can foundation resist uplift?	Yes, OK	Yes, OK	Yes, OK		Yes, OK	Yes, OK
Allowable soil bearing pressure (lbs/ft ²)	1,500	1,500	1,500		1,500	1,500
DL+LL at footing (lb/ft)	190	150	150		151	152
W _{foundation total} (lb/ft)	390	410	350		351	352
Footing width (ft)	1.25	1.25	1.25		2.25	3.25
Check foundation for soil bearing pressure.	OK	OK	OK		OK	OK

Shear Wall Deflection Literature:

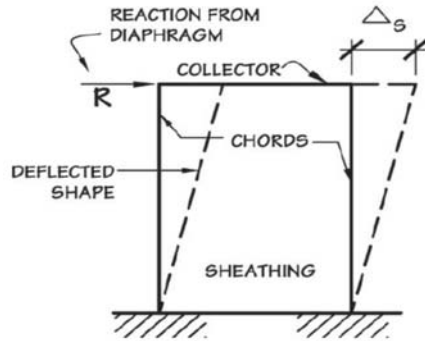


Figure 10.14 Deflection of a shearwall is termed *story drift* Δ_s .

$$\Delta_s = \Delta_b + \Delta_v + \Delta_n + \Delta_a$$

where Δ_b = bending deflection of the shearwall

Δ_v = shear deflection of the shearwall

Δ_n = deflection of the shearwall due to nail slip (deformation)

Δ_a = deflection of the shearwall due to anchorage slip and rotation

$$\Delta_b = \frac{8vh^3}{EAb}$$

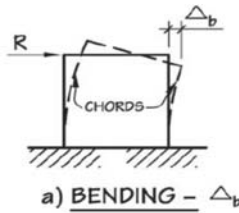
where v = shear force at the top of the wall (lb/ft)

h = height of the wall (ft)

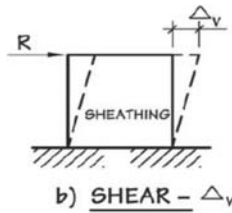
b = width of the wall (ft)

E = modulus of elasticity of the chord (psi)

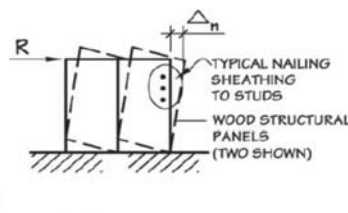
A = cross-sectional area of chord (in.²)



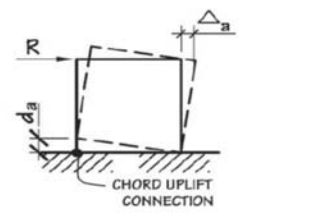
a) BENDING - Δ_b



b) SHEAR - Δ_v



c) NAIL SLIP - Δ_n



d) ANCHORAGE SLIP - Δ_a

$$\Delta_v = \frac{vh}{Gt}$$

where Gt = panel rigidity through the thickness in lb/in [IBC Table 2305.2.2(2)]

$$\Delta_n = 0.75he_n$$

where e_n = nail deformation (in.).

$$\Delta_a = \frac{h}{b} d_a$$

where d_a = anchorage slip (in.).

TABLE 2305.2.2(2)
VALUES OF G_f FOR USE IN CALCULATING DEFLECTION OF WOOD STRUCTURAL PANEL SHEAR WALLS AND DIAPHRAGMS

PANEL TYPE	SPAN RATING	VALUES OF G_f (lb/in. panel depth or width)							
		OTHER				STRUCTURAL I			
		3-ply Plywood	4-ply Plywood	5-ply Plywood ^a	OSB	3-ply Plywood	4-ply Plywood	5-ply Plywood ^a	OSB
Sheathing	24/0	25,000	32,500	37,500	77,500	32,500	42,500	41,500	77,500
	24/16	27,000	35,000	40,500	83,500	35,000	45,500	44,500	83,500
	32/16	27,000	35,000	40,500	83,500	35,000	45,500	44,500	83,500
	40/20	28,500	37,000	43,000	88,500	37,000	48,000	47,500	88,500
	48/24	31,000	40,500	46,500	96,000	40,500	52,500	51,000	96,000
Single Floor	16 o.c.	27,000	35,000	40,500	83,500	35,000	45,500	44,500	83,500
	20 o.c.	28,000	36,500	42,000	87,000	36,500	47,500	46,000	87,000
	24 o.c.	30,000	39,000	45,000	93,000	39,000	50,500	49,500	93,000
	32 o.c.	36,000	47,000	54,000	110,000	47,000	61,000	59,500	110,000
	48 o.c.	50,500	65,500	76,000	155,000	65,500	85,000	83,500	155,000

TABLE 2304.7(3)
ALLOWABLE SPANS AND LOADS FOR WOOD STRUCTURAL PANEL SHEATHING AND SINGLE-FLOOR GRADES CONTINUOUS OVER TWO OR MORE SPANS WITH STRENGTH AXIS PERPENDICULAR TO SUPPORTS^{a,b}

SHEATHING GRADES		ROOF ^c				FLOOR ^d
Panel span rating roof/floor span	Panel thickness (inches)	Maximum span (inches)		Load ^e (psf)		Maximum span (inches)
		With edge support ^f	Without edge support	Total load	Live load	
12/0	$\frac{5}{16}$	12	12	40	30	0
16/0	$\frac{5}{16}, \frac{3}{8}$	16	16	40	30	0
20/0	$\frac{5}{16}, \frac{3}{8}$	20	20	40	30	0
24/0	$\frac{3}{8}, \frac{7}{16}, \frac{1}{2}$	24	20 ^g	40	30	0
24/16	$\frac{7}{16}, \frac{1}{2}$	24	24	50	40	16
32/16	$\frac{15}{32}, \frac{1}{2}, \frac{5}{8}$	32	28	40	30	16 ^h
40/20	$\frac{19}{32}, \frac{5}{8}, \frac{3}{4}, \frac{7}{8}$	40	32	40	30	20 ^{h,i}
48/24	$\frac{23}{32}, \frac{3}{4}, \frac{7}{8}$	48	36	45	35	24
54/32	$\frac{7}{8}, 1$	54	40	45	35	32
60/32	$\frac{7}{8}, 1, \frac{1}{8}$	60	48	45	35	32
SINGLE FLOOR GRADES		ROOF ^c				FLOOR ^d
Panel span rating	Panel thickness (inches)	Maximum span (inches)		Load ^e (psf)		Maximum span (inches)
		With edge support ^f	Without edge support	Total load	Live load	
16 o.c.	$\frac{1}{2}, \frac{19}{32}, \frac{5}{8}$	24	24	50	40	16 ^h
20 o.c.	$\frac{19}{32}, \frac{5}{8}, \frac{3}{4}$	32	32	40	30	20 ^{h,i}
24 o.c.	$\frac{23}{32}, \frac{3}{4}$	48	36	35	25	24
32 o.c.	$\frac{7}{8}, 1$	48	40	50	40	32
48 o.c.	$1\frac{1}{32}, 1\frac{1}{8}$	60	48	50	40	48

TABLE 2305.2.2(1)
 e_n VALUES (Inches) FOR USE IN CALCULATING DIAPHRAGM DEFLECTION DUE TO FASTENER SLIP (Structural I)^{a,d}

LOAD PER FASTENER ^a (pounds)	FASTENER DESIGNATIONS ^b			
	6d	8d	10d	14-Ga staple x 2 inches long
60	0.01	0.00	0.00	0.011
80	0.02	0.01	0.01	0.018
100	0.03	0.01	0.01	0.028
120	0.04	0.02	0.01	0.04
140	0.06	0.03	0.02	0.053
160	0.10	0.04	0.02	0.068
180	—	0.05	0.03	—
200	—	0.07	0.47	—
220	—	0.09	0.06	—
240	—	—	0.07	—