

Structural Calculations for County of San Bernadino – 900 E Washington

900 E Washington
Colton, CA 92324

REVIEWED FOR CODE COMPLIANCE

Reviewed by: Maged Gannay
Date: 01/15/2025

Approval of these plans & specifications shall not be construed to be a permit for, or an approval of any violation of any Federal, State, County or City laws or ordinances. One set of approved plans must be kept on the job until completion.

WILLDAN ENGINEERING

Building Department Submittal
MI2404043.00
12/28/2024



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BY _____
CITY OF COLTON
BUILDING DEPARTMENT

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DIVISION 91 ANALYSIS AND DESIGN REQUIREMENTS

1. STRUCTURAL BASIS OF DESIGN

STRUCTURAL BASIS OF DESIGN

INTRODUCTION AND BUILDING DESCRIPTION

This project consists of diaphragm and wall anchorage structural work for the seismic retrofit of an existing 3-story office building. The existing building is type III construction and consists of a wood panelized roof system with wood 2x sub-purlins & 4x or 6x purlins with GLB girders supported by steel columns that stack to the foundation level. The 2nd and 3rd floor framing is a hybrid plywood floor system with TJI's spanning between steel wide flange girders. The columns supporting the 3rd floor, 2nd floor, and roof system are supported by concrete pad footings. The exterior bearing walls consist of four sides of intermediate concrete-tilt up shear walls. An interior steel moment frame aligns with the diaphragm re-entrant corners at the center of the building.

RETROFIT PROCEDURE AND STRUCTURAL SCOPE OF WORK:

The strengthening of the lateral-force resisting system has been designed to meet or exceed 75% of the seismic base shear specified in "The Equivalent Lateral Force Procedure" of the ASCE 7-16 building standard. The "Los Angeles County Division 91 Earthquake Hazard Reduction in Existing Tilt-Up Concrete Wall Buildings" provided guidance for the following scope of work:

- 1.) Analysis of existing roof and floor flexible diaphragm and sub-diaphragm demands and nailing patterns. It was determined that increased diaphragm capacity and strengthening is not required.
- 2.) Analysis and strengthening of roof purlins and girder for out of plate out-of-plane wall anchorage. Addition of continuity (cross) ties include an option for straps added on top of sheathing or bolted anchors added from below.
- 3.) Analysis of floor joists and girder anchorage and continuity (cross) ties determined increase capacity and strengthening is not required.

AS-BUILT DRAWINGS:

Original historical plans by SAA Consulting Engineers dated 9/28/1987 were available for review and were used to develop existing framing plans and details..

STRUCTURAL BASIS OF DESIGN

CODES AND STANDARDS

- 2022 California Building Code (Title 24, Part 2, California Code of Regulations), with inclusion of all amendments.
- Minimum Design Loads for Buildings and Other Structures (ASCE/SEI 7-16)
- Building Code Requirements for Structural Concrete (ACI 318-19)
- Specification for Structural Steel Buildings (ANSI/AISC 360-16)
- Seismic Provisions for Structural Steel Buildings (ANSI/AISC 341-16)
- Structural Welding Code – Steel (AWS D1.4/D1.4M – 2018)

DESIGN LOADS

- **Seismic Loads**
Importance Factor: $I = 1.0$
Occupancy Category: II
Site Class: D
Seismic Design Category: E
 $S_S = 2.146g$ $S_{DS} = 1.717g$
 $S_1 = 0.854g$ $S_{D1} = 0.968g$
Existing System: Intermediate Precast Shearwalls
 $R = 3$
 $\Omega_0 = 2\frac{1}{2}$ (Reduced to 2.0 for flexible diaphragm)
 $C_d = 3$
Maximum Story Drift = $0.025h_{sx}$

2. LOADS

2.1 GRAVITY LOADS

Load Takeoff

ROOF DEAD LOAD:

RAFTER

Roofing:	2.5 psf
1/2" Plywood:	1.6 psf
Insulation:	1.5 psf
Miscellaneous (for rafters only):	1.7 psf
Rafter Self-Weight	0.7 psf
Σ Rafter D.L.	8 psf

JOIST

Roofing:	2.5 psf
1/2" Plywood:	1.6 psf
Rafters:	0.7 psf
Sprinkers:	1.5 psf
Miscellaneous (for joists & Girders):	1.8 psf
Joist Self-Weight:	1.9 psf
Ceiling, Fixtures, A.C.	2 psf
Σ Joist D.L.	12 psf

GIRDER

MEP	2 psf
Girder Self-Weight:	2 psf
Σ Girder D.L.	16 psf

Lateral: 16 psf

Superimposed DL: 11 psf

ROOF LIVE LOAD: 20.0 psf (Reducible per ASCE 7-16 Sect. 4.8)

Load Takeoff

2nd FLOOR DEAD LOAD:

JOIST

Floor Finishes (carpet):	1.0 psf
3/4" Plywood:	2.4 psf
1-1/2" Gypcrete:	15.0 psf
Insulation:	1.5 psf
Sprinkers:	1.5 psf
Miscellaneous:	1.6 psf
Partitions (non-bearing, non-shear):	8.0 psf
TJI Joist Self-Weight:	8.0 psf
Ceiling, Fixtures, A.C.	2.0 psf
Σ Joist D.L.	41.0 psf

GIRDER

Girder Self-Weight:	2.1 psf
Σ Girder D.L.	43.1 psf

Lateral: 43.1 psf

LIVE LOAD:

Office	50.0 psf	(Reducible per ASCE 7-16 Sect. 4.7)
Corridors above 1st floor	80.0 psf	(Reducible per ASCE 7-16 Sect. 4.7)
Common Areas	100.0 psf	(Non-reducible)
Storage	125.0 psf	(Non-reducible)

SOIL PRESSURE:

ALLOWABLE SOIL BEARING CAPACITIES

2000 psf	CONTINUOUS FOOTING	(Up to 2000 psf, see Geo Report)
2000 psf	SPREAD FOOTING	(Up to 2000 psf, see Geo Report)

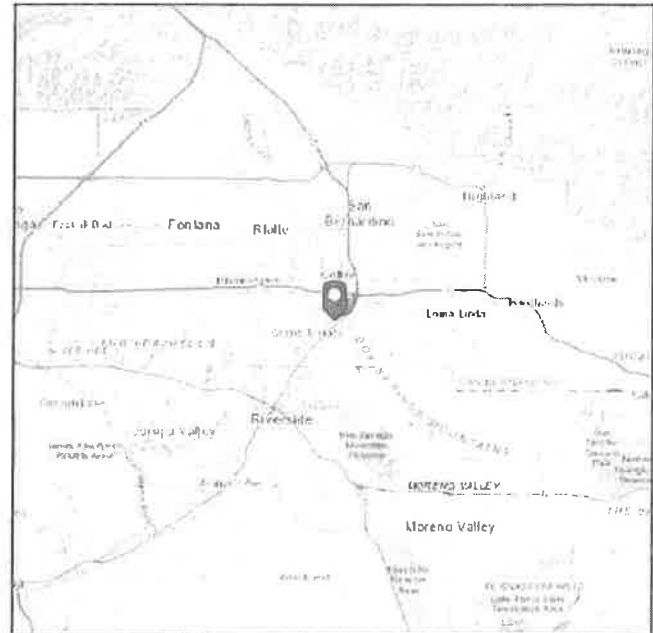
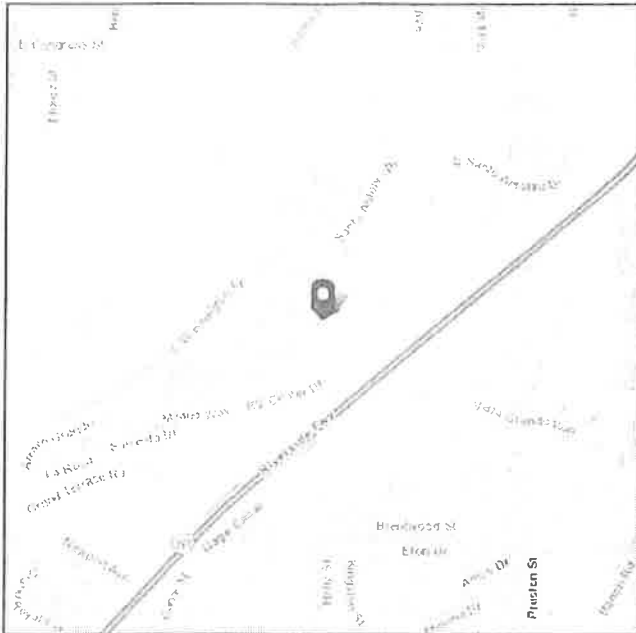
Allowable Passive Pressure = 360 pcf
Coefficient of Friction = 0.35

2.2 SEISMIC LOADS

ASCE Hazards Report

Standard: ASCE/SEI 7-16
Risk Category: II
Soil Class: D - Default (see Section 11.4.3)

Latitude: 34.047178
Longitude: -117.312787
Elevation: 929.8467057590307 ft
(NAVD 88)



ASCE
AMERICAN SOCIETY OF CIVIL ENGINEERS
Seismic

Site Soil Class: D - Default (see Section 11.4.3)

Results:

S_S :	2.145	S_{D1} :	N/A
S_1 :	0.854	T_L :	8
F_a :	1.2	PGA :	0.905
F_v :	N/A	PGA_M :	1.086
S_{MS} :	2.574	F_{PGA} :	1.2
S_{M1} :	N/A	I_e :	1
S_{DS} :	1.716	C_v :	1.5

Ground motion hazard analysis may be required. See ASCE/SEI 7-16 Section 11.4.8.

Data Accessed: Wed Jun 26 2024

Date Source: USGS Seismic Design Maps

Notes

- Mapped MCE Spectral Response values are obtained from the ATC Hazard Application
- References below are for ASCE 7-16 except as noted
- Values shown are at Strength Design or LRFD level and may be reduced for ASD

Structural & Site Specific Information

Project Address	=	900 Washington Colton CA		
Latitude	=	34.04718	Number of stories	= 3
Longitude	=	-117.313	Roof ht abv base	h_u = 41.2 ft
Mapped Resp Accel, short periods	S_s =	2.146 g	Regular structure?	= No
Mapped Resp Accel, 1s period	S_1 =	0.854 g	DSA/OSHDP project?	= No
Long Period Transition	T_L =	12 sec (Figs. 22-14 - 22-17)	$\rho = 1.0?$	= Yes
Seismic Force Resisting System	=	Intermediate precast		
Response Modification Coefficient	R =	4 (Table 12.2-1)	Site Class	= D (Table 20.3-1)
Building Period Coefficient	C_u =	0.020 (Table 12.8-2)	Site Class (for F_a)	= D (Sec 11.4.8) Exception 1
Period Parameter	α =	0.75 (Table 12.8-3)	Risk Category	= II (CBC TI 604.5)
Using Site Specific Ground Motion	=	No (Sec 21.1)	Importance Factor	I_e = 1.00 (Table 1.5-2)
Using Default Site Class?	=	No (Sec 11.4.3 & 11.4.4)		

Is ground motion hazard analysis required? <—Site Specific Ground Motion Required unless exempted by Sec 20.3.1 (Sec 11.4.6)

Design Spectral Acceleration Parameters

Short Period Site Coefficient	F_a =	1.000 (CBC TI 613.2.3(1))
Long Period Site Coefficient	F_v =	1.700 (Sec ASCE 7 Sec 11.4.8 for Add'l Reqs)
Design 5% Damped, Spectral Response Acceleration at short periods	S_{DS} =	1.431 g (CBC 2022 Eq. 16-20, 16-22)
Design 5% Damped, Spectral Response Acceleration at a period of 1 sec	S_{D1} =	0.968 g (CBC 2022 Eq. 16-21, 16-23)
Seismic Design Category Based on Short-Period Response Acceleration	=	D (CBC 2022 TI 613.2.5(1))
Seismic Design Category Based on a 1-Second Period Response Acceleration	=	D (CBC 2022 TI 613.2.5(2))
Seismic Design Category for large S_1 and Occupancy Category	=	E (ASCE 7-16 Sec 11.6)
Minimum Seismic Design Category if DSA/OSHDP	=	E (CBC 1613A.2.5)

Seismic Design Category: **E** (ASCE 7-16 Sec 11.6)

Design Period

Approx Fundamental Period	T_a =	0.325 sec (ASCE 7-16 Eq. 12.8-7)
Fundamental Period From Analysis	T_b =	1.500 sec enter 0 if not performed
T_0	$0.2(S_{DS}/S_{D1})$ =	0.135 sec (ASCE 7-16 Sec 11.4.6)
T_s	S_{DS}/S_{D1} =	0.677 sec (ASCE 7-16 Sec 11.4.6)
Coeff for Upper Limit on Period	C_u =	1.4 (ASCE 7-16 Table 12.8-1)
Maximum Fundamental Period	$C_u T_b$ =	0.455 sec (ASCE 7-16 Sec 12.8.2)

Design Period $T = 0.455$ sec (ASCE 7-16 Sec 12.8.2)

< Code Maximum Used

Seismic Response Coefficient

S_{DS} value for determination of C_s and E_s	=	1.431 (ASCE 7-16 Sec 12.8.1.3)
For $T \leq T_b$: C_s =	$S_{DS} / (R/I_e)$ =	0.358 < Short Period Design (ASCE 7-16 Eq. 12.8-2)
For $T_b < T \leq T_L$: $C_{s,max}$ =	$S_{D1} / (T(R/I_e))$ =	0.531 (ASCE 7-16 Eq. 12.8-3)
For $T > T_L$: $C_{s,max}$ =	$S_{D1} T_L / (T^2(R/I_e))$ =	14.005 (ASCE 7-16 Eq. 12.8-4)
Minimum: $C_{s,min}$ =	$\max(0.044 S_{DS} I_e, 0.01)$ =	0.063 (ASCE 7-16 Eq. 12.8-5)
Where $S_1 \geq 0.6g$: $C_{s,min}$ =	$0.5 S_1 / (R/I_e)$ =	0.107 (ASCE 7-16 Eq. 12.8-6)
Where $S_1 \geq 0.2g$ and site class D:	C_s =	0.358 (ASCE 7-16 Sec 11.4.8) Exception 2
Where $S_1 \geq 0.2g$ and site class E:		Use Calculated C_s Below (ASCE 7-16 Sec 11.4.8) Exception 3

Base Shear: $V_{base} = 0.358 W$ (ASCE 7-16 Eq. 12.8-1)

Design Response Spectrum: ASCE 7-16

Notes

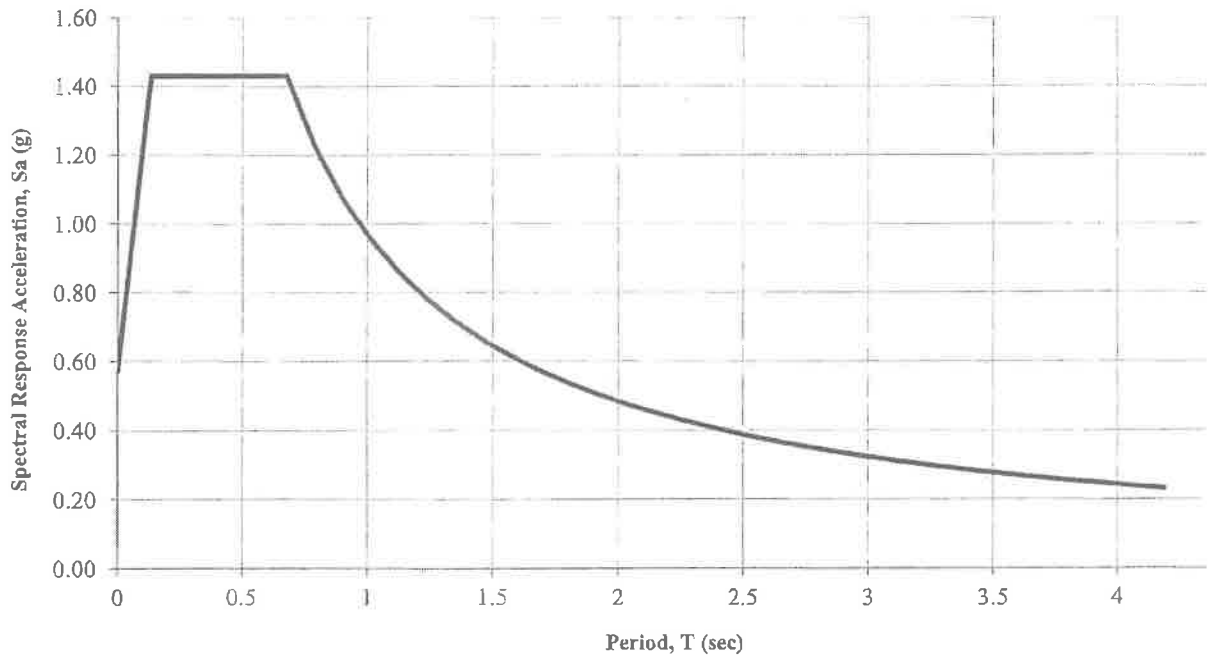
- References below are for ASCE 7-16 except as noted; see Figure 11.4-1
- Where an MCE response spectrum is required, multiply the design spectrum by 1.5 (Sec 11.4.7)
- Analysis must include a sufficient number of modes to obtain 100% mass participation in each direction (Sec 12.9.1.1)
- Force results shall be divided by (R / I) but may not be less than 100% of static results (Sec 12.9.1.4)
- Drift and displacement results shall be divided by (R/I) and multiplied by (Cd / I) for inelastic magnitude (Sec 12.9.1.2)
- Response parameters shall be combined using either the SRSS or the CQC methods (Sec 12.9.1.3)

Design Response Spectrum Parameters

	$T_0 =$	0.135 sec	(Sec 11.4.6)
	$T_s =$	0.677 sec	(Sec 11.4.6)
Long Period Transition	$T_L =$	12 sec	(Sec 11.4.6)
Design 5% Damped, Spectral Response Acceleration at short periods	$S_{DS} =$	1.431 g	(CBC 2022 E)
Design 5% Damped, Spectral Response Acceleration at a period of 1 sec	$S_{D1} =$	0.968 g	(CBC 2022 E)

Spectral Response Acceleration Curves

For $T < T_0$:	$S_a = S_{DS} (0.4 + 0.6 T/T_0)$	(Eq. 11.4-5)
For $T_0 \leq T \leq T_s$:	$S_a = S_{DS}$	(Sec 11.4.6 Itc)
For $T_s < T \leq T_L$:	$S_a = S_{D1} / T$	(Eq. 11.4-6)
For $T > T_L$:	$S_a = S_{D1} T_L / T^2$	(Eq. 11.4-7)



Design Response Spectrum

Vertical Distribution of Seismic Forces: ASCE 7-16

Notes

- References below are for ASCE 7-16 except as noted
- Values shown are at Strength Design or LRFD level and may be reduced for ASD

Seismic base shear coefficient	$C_s =$	0.358	(Sec 12.8.1)
Effective seismic weight	$W =$	5028 k	(Sec 12.7.2, sec
Seismic base shear	$V =$	1798 k	(Eq. 12.8-1)
Design period	$T =$	0.455 sec	(Sec 12.8.2)
Design 5% Damped, Spectral Response Acceleration at short periods	$S_{DS} =$	1.431 g	
Importance Factor	$I_e =$	1.00	(Table 1.5-2)
Exponent related to the structure period	$k =$	1.000	(Sec. 12.8.3)

Story Forces - Strength Level

Level	h (ft)	w floor (k)	w wall to N/S (k)	w wall to E/W (k)	w _x total (k)	w _x h _x ^k (kft)	Story Force F _x (k)	Story Shear ΣF _i (k)	Story Overturning F _x h _x (kft)
Roof	41.20	529	282	275	1086	44743	646	646	26613
Third	27.00	1032	476	463	1971	53217	768	1414	20743
Second	13.50	1032	476	463	1971	26609	384	1798	5186
		2593	1234	692	5028	124569	1798		

$$F_x = C_{vx} V \quad (\text{Eq. 12.8-11})$$

$$C_{vx} = \frac{w_x h_x}{\sum_{i=1}^n w_i h_i^k} \quad (\text{Eq. 12.8-12})$$

Diaphragm Forces - Strength Level

North-South Direction

Level	w _{px} (k) *	Σw _i (k)	F _{px} (k)	F _{px min} (k)	F _{px max} (k)	F _{px des} (k)
Roof	804	1086	478	230	460	460
Third	1495	3057	692	428	856	692
Second	1495	5028	535	428	856	535

maximum governs

East-West Direction

Level	w _{py} (k) *	Σw _i (k)	F _{py} (k)	F _{py min} (k)	F _{py max} (k)	F _{py des} (k)
Roof	811	1086	482	232	464	464
Third	1508	3057	698	431	863	698
Second	1508	5028	539	431	863	539

maximum governs

$$F_{px} = \frac{\sum_{i=1}^n F_i}{\sum_{i=1}^n w_i} w_{px} \quad (\text{Eq. 12.10-1})$$

$$F_{px min} = 0.2 S_{DS} I w_{px} \quad (\text{Eq. 12.10-2})$$

$$F_{px max} = 0.4 S_{DS} I w_{px} \quad (\text{Eq. 12.10-3})$$

* w_{px} = w_x - weight of parallel walls

Design Load Combinations:

Notes

- References below are for ASCE 7-16 except as noted
- Cantilevered components in Seismic Design Categories D, E, or F shall be designed for a min net upward force of 0.2 D
- Load combinations do not include load cases F (fluid), T (self-straining), or R (rain). See Sec 2.3 if these loads are appl
- Do not use loads from lateral soil pressure H to reduce structural actions imposed by W or E.
- Qe refers to horizontal seismic force per Sec 12.4, redundancy and overstrength factors are applied to load coefficient.

Load Combinations Input

Is the floor live load greater than 100 psf or used as garage or public assembly?

No

Is the structure subject to snow loads?

No

Is the structure subject to soil loads?

No

Redundancy Factor

$\rho =$

1.30 (Sec 12.3.4)

System Overstrength Factor

$\Omega_0 =$

3.0 (Table 12.2-1)

Seismic Design Category

E

Design 5% Damped, Spectral Response Acceleration at short periods

$S_{DS} =$

1.431 g

Vertical Seismic Force

$E_v = 0.2 S_{DS} D =$

0.286 D (Eq. 12.4-4a)

Load Combinations Using Strength Design (CBC 1605.2)

- 16-1: 1.4 D
- 16-2: 1.2 D + 1.6 L + 0.5 Lr
- 16-3a: 1.2 D + 0.5 L + 1.6 Lr
- 16-3b: 1.2 D ± 0.5 W + 1.6 Lr
- 16-4: 1.2 D ± W + 0.5 L + 0.5 Lr
- 16-5: 1.2 D + 0.5 L ± Qe
- 16-6: 0.9 D ± W
- 16-7: 0.9 D ± Qe

Load Combinations Using Special Seismic (CBC 1605.1 / ASCE 7-16 12.4.3)

- 16-5s: 1.49 D + 0.5 L ± 3 Qe
- 16-7s: 0.61 D ± 3 Qe

Design Load Combinations:

Notes

- References below are for ASCE 7-16 except as noted
- Cantilevered components in Seismic Design Categories D, E, or F shall be designed for a min net upward force of 0.2 D
- Load combinations do not include load cases F (fluid), T (self-straining), or R (rain). See Sec 2.3 if these loads are appl
- Do not use loads from lateral soil pressure H to reduce structural actions imposed by W or E.
- Qe refers to horizontal seismic force per Sec 12.4, redundancy and overstrength factors are applied to load coefficient.

Load Combinations Input

Is the floor live load greater than 100 psf or used as garage or public assembly?

No

Is the structure subject to snow loads?

No

Is the structure subject to soil loads?

No

Redundancy Factor

$\rho =$

1. (Sec 12.3.4)

System Overstrength Factor

$\Omega_0 =$

3.0 (Table 12.2-1)

Seismic Design Category

E

Design 5% Damped, Spectral Response Acceleration at short periods

$S_{DS} =$

1.431 g

Vertical Seismic Force

$E_v = 0.2 S_{DS} D =$

0.286 D (Eq. 12.4-4a)

Load Combinations Using Allowable Stress Design (CBC 1605.3.1)

- 16-8: D
- 16-9: D + L
- 16-10: D + Lr
- 16-11: D + 0.75 L + 0.75 Lr
- 16-12a: D ± 0.6 W
- 16-12b: 1.2 D ± 0.91 Qe
- 16-13: D ± 0.45 W + 0.75 L + 0.75 Lr
- 16-14: 1.15 D + 0.75 L ± 0.68 Qe + 0.75 Lr
- 16-15: 0.6 D ± 0.6 W
- 16-15: 0.4 D ± 0.91 Qe

Load Combinations Using Alternate Allowable Stress Design (CBC 1605.3.2)

- 16-17a: D
- 16-17b: D + L
- 16-17c: D + L + Lr
- 16-18a: 0.67 D ± 0.78 W
- 16-18b: D ± 0.78 W + L
- 16-19: D ± 0.78 W
- 16-20: D ± 0.39 W + L
- 16-21: 1.15 D + L ± 0.92 Qe
- 16-22: 0.7 D ± 0.92 Qe

3. LATERAL FORCE RESISTING SYSTEM (LFRS) DESIGN

3.1 DIAPHRAGM ANALYSIS

Roof Diaphragm Shear Force Calculation (ASD):

Roof Area: 25,177 ft²

N/S Force: 460K / 25,177ft² = 18.4 PSF

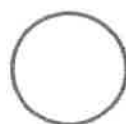
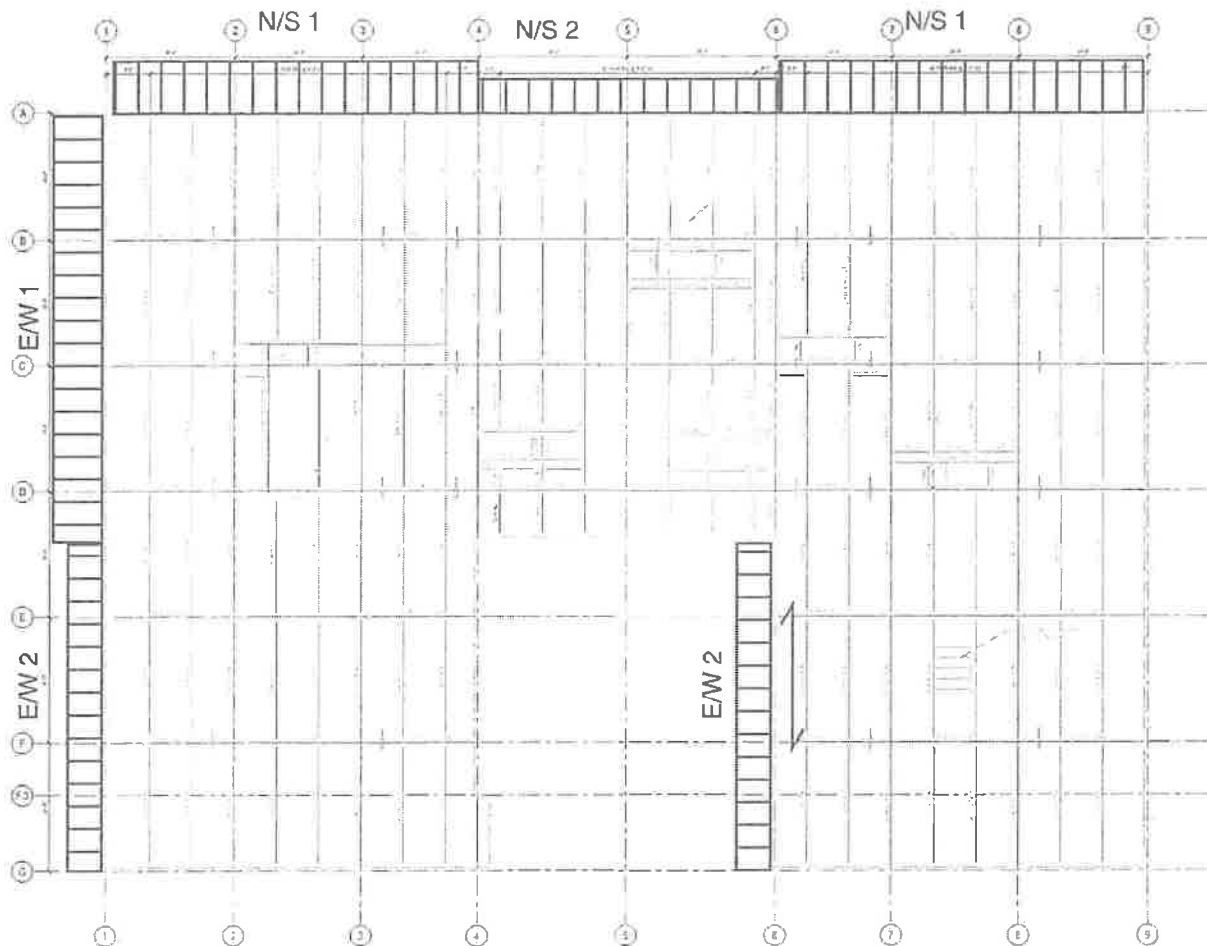
E/W Force: 464K / 25,177ft² = 18.4 PSF

N/S₁ = 0.7*18.4 PSF x 145' = 1,870 PLF

E/W₁ = 0.7*18.4 PSF x 197' = 2,537 PLF

N/S₂ = 0.7*18.4 PSF x 82.5' = 1,064 PLF

E/W₂ = 0.7* 18.4 PSF x 70.5' = 908 PLF



ROOF DIAPHRAGM SHEAR PLAN
(LRFD)


$$1'' = 50'-0''$$

North Grid E Diaphragm Type 4 $vu = 67 \text{ kips} / 70.5' = 950 \text{ plf} < 1030 \text{ OK @ 100\% DCE}$
 South Grid E Diaphragm Type 3 $vu = 43 \text{ kips} / 70.5' = 610 \text{ plf} < 910 \text{ plf OK @ 100\% DCE}$

Grid1 Diaphragm Type 2 $vu=56 \text{ kips}/145' = 386 \text{ plf} < 540 \text{ plf OK @ 100\% DCE}$
 East Grid 4 Diaphragm Type 2 $vu= 76.2 \text{ kips}/145' = 525 \text{ plf OK @ 100\% DCE}$
 West Grid 4 Diaphragm Type 1 $vu= 30.5 \text{ kips}/82.5' = 363 \text{ plf} < 405 \text{ plf 100\% DCE}$

EXISTING ROOF DIAPHRAGM NAILING SCHEDULE						
ZONE	PLYWOOD THICKNESS	TO LEDGERS, PURLINS, & BLOCKING (BN)	TO JOISTS & GLB's (EN)	FIELD (FN)	CAPACITY (ASD)	REMARKS
1	1/2"	10d @ 6"OC	10d @ 6"OC	10d @ 12"OC	405 PLF	
2	1/2"	10d @ 4"OC	10d @ 6"OC	10d @ 12"OC	540 PLF	
3	1/2"	10d @ 2 1/2"OC	10d @ 4"OC	10d @ 12"OC	910 PLF	W/ 3x SUB PURLIN @ ALL PANEL JOINTS
4	1/2"	10d @ 2"OC	10d @ 3"OC	10d @ 12"OC	1030 PLF	W/ 3x SUB PURLIN @ ALL PANEL JOINTS

NOTES:

- ALL NAILING @ BOUNDARY, PANEL EDGES & CONTINUOUS PANEL EDGES WITH SPACING LESS THAN 3" MUST BE STAGGERED
- 1/2" PLYWOOD SHEATHING STRUCT I INDEX 32/16

Table 4.2A Nominal Unit Shear Capacities for Sheathed Wood-Frame Diaphragms

Blocked Wood Structural Panel Diaphragms^{1,2,3,4,6}

Sheathing Grade	Common Nail Size ^a Length (in.) x Shank diameter (in.) x Head diameter (in.)	Minimum Nail Bearing Length In Framing Member or Blocking, ℓ_m (in.)	Minimum Nominal Panel Thickness (in.)	Minimum Nominal Width of Nailed Face at Adjoining Panel Edges and Boundaries (in.)	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)											
					6			4			2-1/2			2		
					Nail Spacing (in.) at other panel edges (Cases 1, 2, 3, & 4)											
					6			6			4			3		
					v_n (plf)	G_s (kips/in.)		v_n (plf)	G_s (kips/in.)		v_n (plf)	G_s (kips/in.)		v_n (plf)	G_s (kips/in.)	
	OSB	PLY	OSB	PLY	OSB	PLY	OSB	PLY	OSB	PLY	OSB	PLY				
Structural I	6d (2 x 0.113 x 0.266)	1-1/4	5/16	2	520	15	12	700	8.5	7.5	1050	12	10	1175	20	15
				3	590	12	9.5	785	7.0	6.0	1175	9.5	8.5	1330	17	13
	8d (2-1/2 x 0.131 x 0.281)	1-3/8	3/8	2	755	14	11	1010	9.0	7.5	1485	13	10	1680	21	15
				3	840	12	10	1120	7.5	6.5	1680	10	9.0	1890	18	13
	10d (3 x 0.148 x 0.312)	1-1/2	15/32	2	895	24	17	1190	15	12	1790	20	15	2045	31	21
				3	1010	20	15	1345	12	9.5	2015	16	13	2295	26	18
Sheathing and Single-Floor	6d (2 x 0.113 x 0.266)	1-1/4	5/16	2	475	15	10	630	9.0	7.0	940	13	9.5	1085	21	13
				3	530	12	9.0	700	7.0	6.0	1065	10	8.0	1205	17	12
			3/8	2	520	13	9.5	700	7.0	6.0	1050	10	8.0	1175	18	12
				3	590	10	8.0	785	5.5	5.0	1175	8.5	7.0	1330	14	10
	8d (2-1/2 x 0.131 x 0.281)	1-3/8	3/8	2	670	15	11	895	9.5	7.5	1345	13	9.5	1525	21	13
				3	755	12	9.5	1010	7.5	6.0	1510	11	8.5	1710	18	12
			7/16	2	715	14	10	950	8.5	7.0	1415	12	9.5	1610	20	13
				3	800	11	9.0	1065	7.0	6.0	1595	10	8.0	1805	17	12
			15/32	2	755	13	9.5	1010	7.5	6.5	1485	11	8.5	1680	19	13
				3	840	10	8.5	1120	6.0	5.5	1680	9.0	7.5	1890	15	11
	10d (3 x 0.148 x 0.312)	1-1/2	15/32	2	810	25	15	1080	15	11	1610	21	14	1835	33	18
				3	910	21	14	1205	12	9.5	1820	17	12	2050	28	16
			19/32	2	895	21	14	1190	13	9.5	1790	18	12	2045	28	17
				3	1010	17	12	1345	10	8.0	2015	14	11	2295	24	15

- Nominal unit shear capacities shall be adjusted in accordance with 4.1.4 to determine ASD allowable unit shear capacity and LRFD factored unit resistance. For general construction requirements see 4.2.7. For specific requirements, see 4.2.8.1 for wood structural panel diaphragms. See Appendix A for common nail dimensions.
- For species and grades of framing other than Douglas-Fir-Larch or Southern Pine, reduced nominal unit shear capacities shall be determined by multiplying the tabulated nominal unit shear capacity by the Specific Gravity Adjustment Factor = $[1 - (0.5 - G)]$, where G = Specific Gravity of the framing lumber from the NDS (Table 12.3.3A). The Specific Gravity Adjustment Factor shall not be greater than 1.
- Apparent shear stiffness values, G_s , are based on nail slip in framing with moisture content less than or equal to 19% at time of fabrication and panel stiffness values for diaphragms constructed with either OSB or 3-ply plywood panels. When 4-ply, or 5-ply plywood panels or composite panels are used, G_s values shall be permitted to be increased by 1.2
- Where moisture content of the framing is greater than 19% at time of fabrication, G_s values shall be multiplied by 0.8.

3rd Floor Diaphragm Shear Force Calculation (ASD):

Floor Area: 25,177 ft²

N/S Force: 692K / 25,177ft² = 27.4 PSF

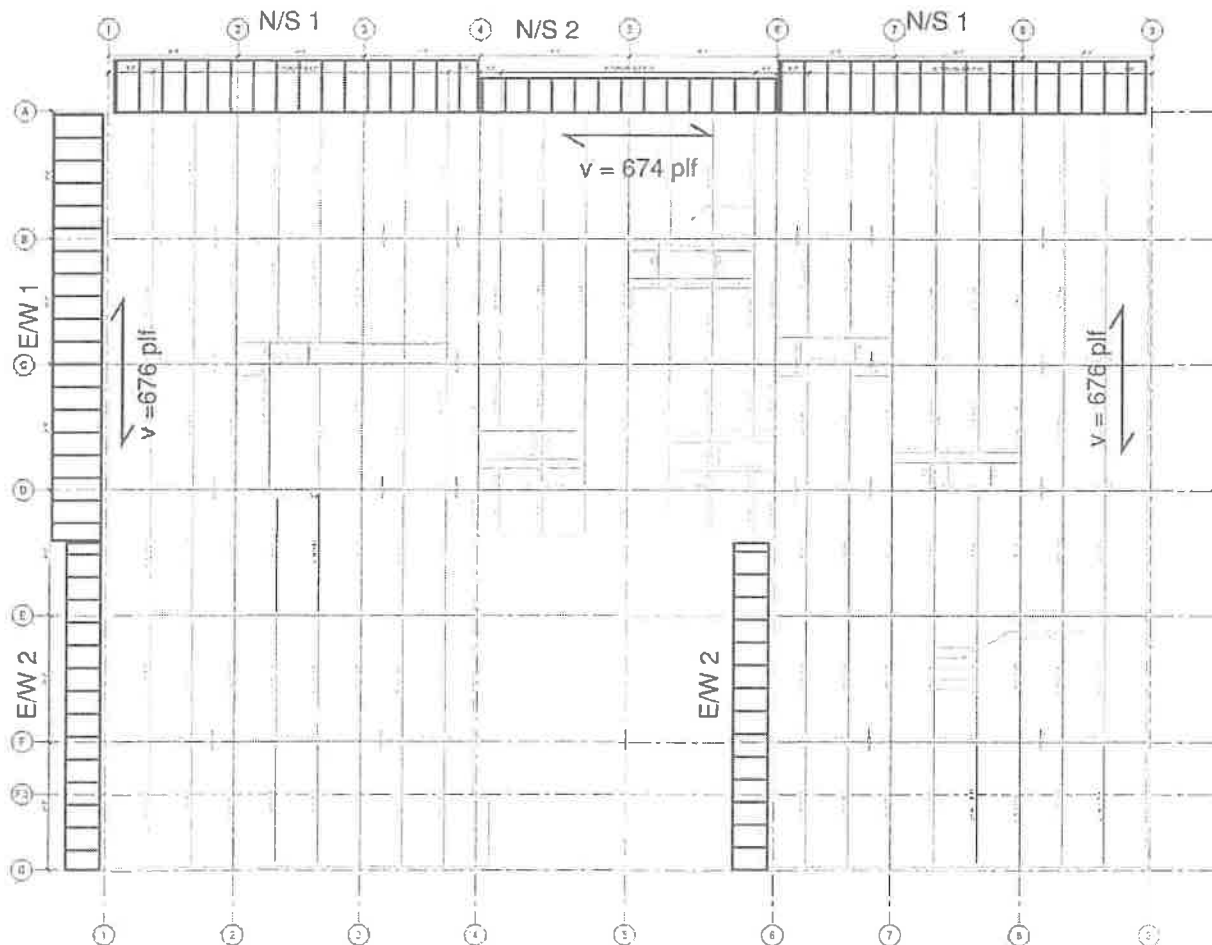
N/S₁ = 0.7* 27.4 PSF x 145' = 2781 PLF

N/S₂ = 0.7*27.4 PSF x 82.5' = 1,582 PSF

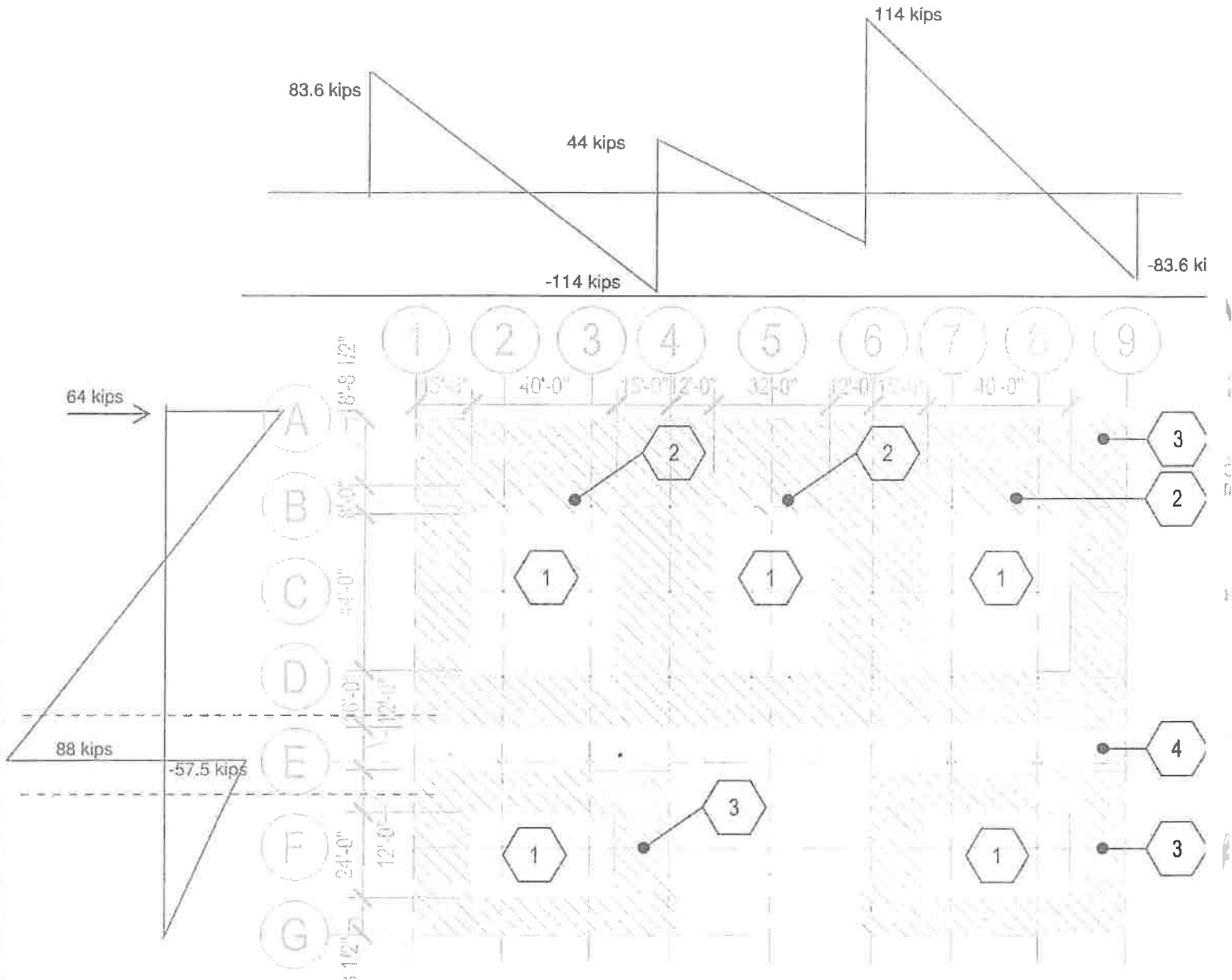
E/W Force: 698K / 25,177ft² = 25.7 PSF

E/W₁ = 0.7* 25.7 PSF x 197' = 3,220 PLF

E/W₂ = 0.7*25.7 PSF x 70.5' = 1,268 PLF



3rd FLOOR DIAPHRAGM SHEAR PLAN
 (LRFD)



EXISTING THIRD FLOOR DIAPHRAGM NAILING

1" = 50'-0"

East-West

Diaphragm Type 2 $vu = 64 \text{ kips} / (197' / 2) = 649 \text{ plf} < 672 \text{ plf OK @ 100\% DCE}$

North Grid E Diaphragm Type 4 $vu = 88 \text{ kips} / 70.5' = 1248 \text{ plf} < 1475 \text{ plf OK @ 100\% DCE}$

South Grid E Diaphragm Type 3 $vu = 57.5 \text{ kips} / 70.5' = 816 \text{ plf} < 1007 \text{ plf OK @ 100\% DCE}$

North South

Grid1 Diaphragm Type 2 $vu = 86.3 \text{ kips} / 145' = 595 \text{ plf} < 672 \text{ plf OK @ 100\% DCE}$

East Grid 4 Diaphragm Type 3 $vu = 114 \text{ kips} / 145' = 786 \text{ plf} < 1007 \text{ plf OK @ 100\% DCE}$

West Grid 4 Diaphragm Type 3 $vu = 44 \text{ kips} / 82.5' = 533 \text{ plf} < 672 \text{ plf 100\% DCE}$

2nd Floor Diaphragm Shear Force Calculation (ASD):

Floor Area: 25,177 ft²

N/S Force: 535K / 25,177ft² = 21.2 PSF

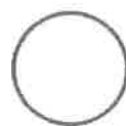
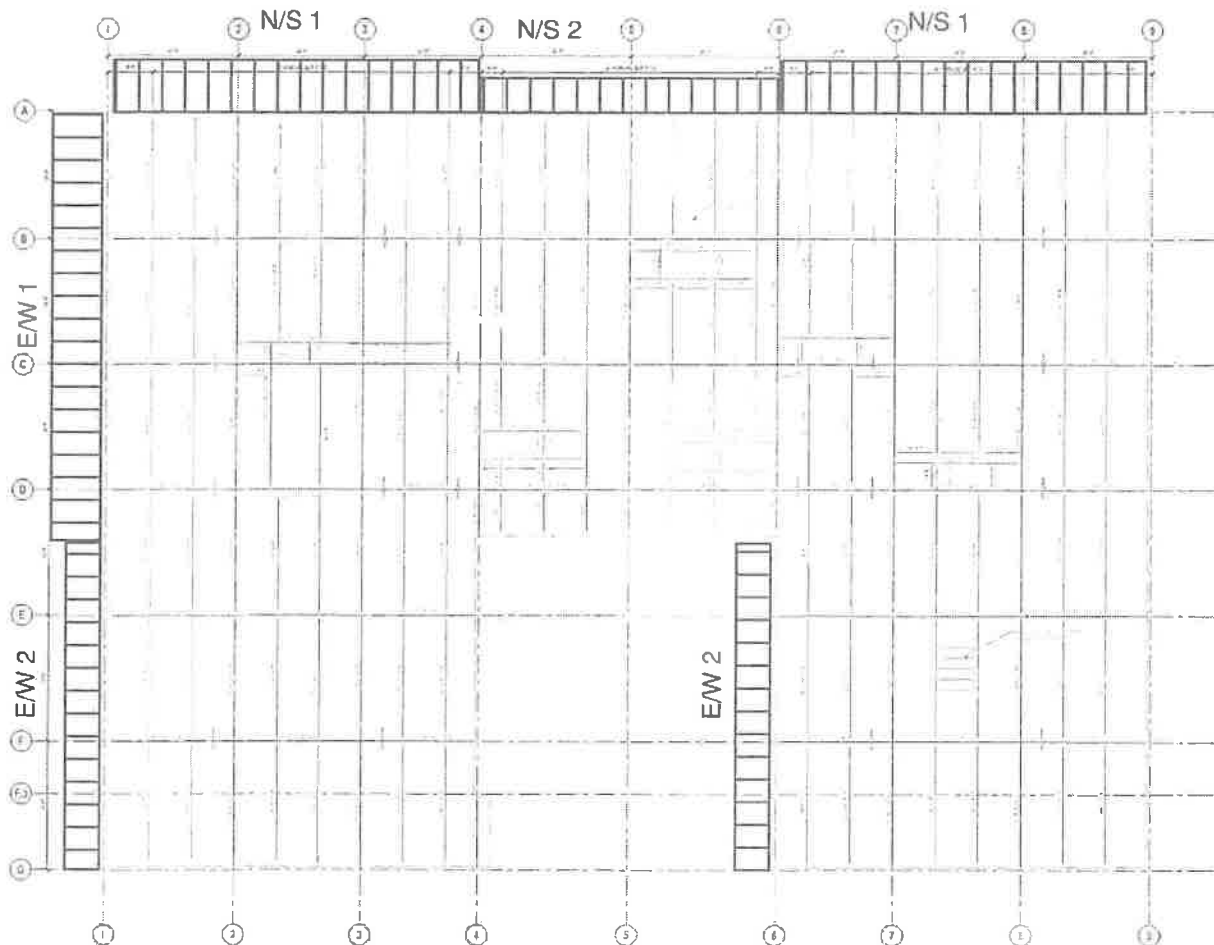
N/S₁ = 0.7* 21.2 PSF x 145' = 2152 PLF

N/S₂ = 0.7*21.2 PSF x 82.5' = 1,224 PSF

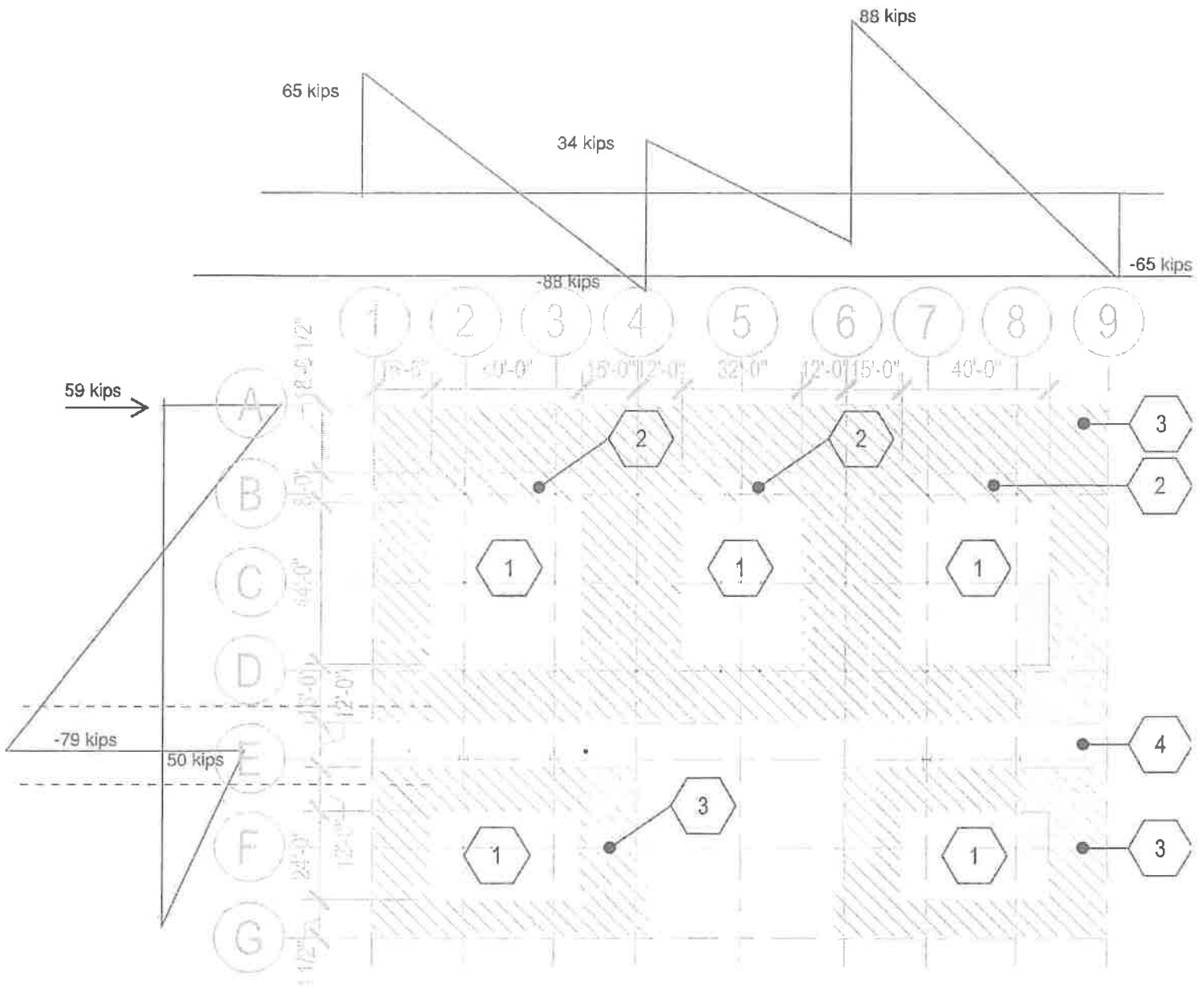
E/W Force: 539K / 25,177ft² = 21.4 PSF

E/W₁ = 0.7* 21.4 PSF x 197' = 2951 PLF

E/W₂ = 0.7*21.4 PSF x 70.5' = 1,056 PLF



3rd FLOOR DIAPHRAGM SHEAR PLAN
(LRFD)



EXISTING SECOND FLOOR DIAPHRAGM NAILING

1" = 50'-0"

Nailing pattern same as 3rd floor and is acceptable by inspection

EXISTING THIRD FLOOR DIAPHRAGM NAILING SCHEDULE

ZONE	PLYWOOD THICKNESS	TO LEDGERS, PURLINS, & BLOCKING (BN)	TO JOISTS & GLB's (EN)	FIELD (FN)	CAPACITY (l(ASD)	REMARKS
1	3/4"	10d @ 6"OC	10d @ 6"OC	10d @ 10"OC	505 'LF	
2	3/4"	10d @ 4"OC	10d @ 6"OC	10d @ 10"OC	672 'LF	
3	3/4"	10d @ 2 1/2"OC	10d @ 4"OC	10d @ 10"OC	1007 'LF	SEE NOTE
4	3/4"	10d @ 2"OC	10d @ 3"OC	10d @ 10"OC	1475 'LF	SEE NOTE

Table 4.2A Nominal Unit Shear Capacities for Sheathed Wood-Frame Diaphragms

Blocked Wood Structural Panel Diaphragms^{1,2,3,4,6}

Sheathing Grade	Common Nail Size ³ Length (in.) x Shank diameter (in.) x Head diameter (in.)	Minimum Nail Bearing Length In Framing Member or Blocking, ℓ_n (in.)	Minimum Nominal Panel Thickness (in.)	Minimum Nominal Width of Nailed Face at Adjoining Panel Edges and Boundaries (in.)	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)												
					6			4			2-1/2			2			
					Nail Spacing (in.) at other panel edges (Cases 1,2,3, & 4)												
					6			6			4			3			
					V_n (plf)	G_s (kips/in.)		V_n (plf)	G_s (kips/in.)		V_n (plf)	G_s (kips/in.)		V_n (plf)	G_s (kips/in.)		
OSB		PLY	OSB		PLY	OSB		PLY	OSB		PLY						
Structural I	6d (2 x 0.113 x 0.266)	1-1/4	5/16	2 3	520 590	15 12	700 9.5	8.5 7.0	7.5 6.0	1050 1175	12 9.5	10 8.5	1175 1330	20 17	15 13		
	8d (2-1/2 x 0.131 x 0.281)	1-3/8	3/8	2 3	755 840	14 12	11 10	1010 1120	9.0 7.5	7.5 6.5	1485 1680	13 10	10 9.0	1680 1890	21 18	15 13	
	10d (3 x 0.148 x 0.312)	1-1/2	15/32	2 3	895 1010	24 20	17 15	1190 1345	15 12	12 9.5	1790 2015	20 16	15 13	2045 2295	31 26	21 16	
	Sheathing and Single-Floor	6d (2 x 0.113 x 0.266)	1-1/4	5/16	2 3	475 530	15 12	10 9.0	630 700	9.0 7.0	7.0 6.0	940 1065	13 10	9.5 8.0	1085 1205	21 17	13 12
				3/8	2 3	520 590	13 10	9.5 8.0	700 785	7.0 5.5	6.0 5.0	1050 1175	10 8.5	8.0 7.0	1175 1330	18 14	12 10
			8d (2-1/2 x 0.131 x 0.281)	1-3/8	3/8	2 3	670 755	15 12	11 9.5	895 1010	9.5 7.5	7.5 6.0	1345 1510	13 11	9.5 8.5	1525 1710	21 18
7/16					2 3	715 800	14 11	10 9.0	950 1085	8.5 7.0	7.0 6.0	1415 1595	12 10	9.5 8.0	1610 1805	20 17	13 12
10d (3 x 0.148 x 0.312)		1-1/2		15/32	2 3	755 840	13 10	9.5 8.5	1010 1120	7.5 6.0	8.5 5.5	1485 1680	11 9.0	8.5 7.5	1680 1890	19 15	13 11
				15/32	2 3	810 910	25 21	15 14	1080 1205	15 12	11 9.5	1610 1820	21 17	14 12	1835 2060	33 28	16 16
				19/32	2 3	895 1010	21 17	14 12	1190 1345	13 10	9.5 8.0	1790 2015	18 14	12 11	2045 2295	28 24	17 15

- Nominal unit shear capacities shall be adjusted in accordance with 4.1.4 to determine ASD allowable unit shear capacity and LRFD factored unit resistance. For general construction requirements see 4.2.7. For specific requirements, see 4.2.8.1 for wood structural panel diaphragms. See Appendix A for common nail dimensions.
- For species and grades of framing other than Douglas-Fir-Larch or Southern Pine, reduced nominal unit shear capacities shall be determined by multiplying the tabulated nominal unit shear capacity by the Specific Gravity Adjustment Factor = $[1 - (0.5 - G)]$, where G = Specific Gravity of the framing lumber from the NDS (Table 12.3.3A). The Specific Gravity Adjustment Factor shall not be greater than 1.
- Apparent shear stiffness values, G_s , are based on nail slip in framing with moisture content less than or equal to 19% at time of fabrication and panel stiffness values for diaphragms constructed with either OSB or 3-ply plywood panels. When 4-ply, or 5-ply plywood panels or composite panels are used, G_s values shall be permitted to be increased by 1.2
- Where moisture content of the framing is greater than 19% at time of fabrication, G_s values shall be multiplied by 0.5.

3.2 WALL ANCHORAGE DESIGN

Joist and Girder Seismic Axial Loads (See 5/S-5.01 & 6/S-5.01)

$$S_{DS} = 1.717$$

$$I_E = 1.0$$

$F_p = 0.8 * S_{DS} * I * W_p$ (ASCE 7-16, 12.11-1)
 Use 75% F_p per 2022 LABC Ch 95, section 9506.1, 9506.4

Roof Framing	Rafter	Purlin	Girder						
Joist Trib. Width (ft)	4	8	24						
Wall Ht (ft)	14.5	14.5	14.5						
Roof Ht (ft)	13	13	13						
Wall Thickness (in)	8.5	8.5	8.5						
Seismic Axial Load (k) (LRFD)	3.5	7.1	21.2						
Seismic Axial Load (k) (ASD)	2.5	5.0	14.9						

(1) Seismic Axial load (ASD) = $0.75 * 0.7 * F_p \times 150 \text{ pcf Wall Thickness} / 12 \times \text{Wall Ht}^2 / (2 \times \text{Roof Ht}) \times \text{Joist Trib Width}$

Floor Joist	I-joist								
Girder Trib. Width (ft)	2.5								
Wall Ht (ft)	13.5								
Roof Ht (ft)	NA								
Wall Thickness (in)	10								
Seismic Axial Load (k) (ASD)	3.0								

(1) Seismic Axial load (ASD) = $0.75 * 0.7 * F_p \times 150 \text{ pcf Wall Thickness} / 12 \times \text{Wall Ht}^2 / (2 \times \text{Roof Ht}) \times \text{joist Trib Width}$

Roof Sub-Diaphragm Analysis (at Subpurlins) - Line "1" Grids "A" to "G"

Wall thickness = 8.50 in

TOW = 14.5 ft

Roof Ht. = 13.0 ft

Seismic Trib. Ht. = 8.00 ft

Wall weight = 106.25 psf

S_{DS} = 1.717

I_E = 1

Seismic, $F_p = 0.75 * 0.7 * 0.8 * S_{DS} * I_E * \text{Wall Weight} * \text{Seismic Trib. Ht. (ASD)}$

$F_{p_seismic} = 0.75 * 0.7 * 0.8 * 1.717 * 1 * 106.25 \text{ psf} * 8 \text{ ft} = 613.0 \text{ plf}$

(E) Subpurlin ties to wall 4.0 ft o.c.

Provide PAT18 each side of 2x4 with 2 - 1/2" dia. M.B. (See 5/S-5.01)

Capacity of (2) PAT18 straps

= 2 * 1610 lbs = 3220 lbs
= 3220 lbs > 2451.9 lbs O.K.

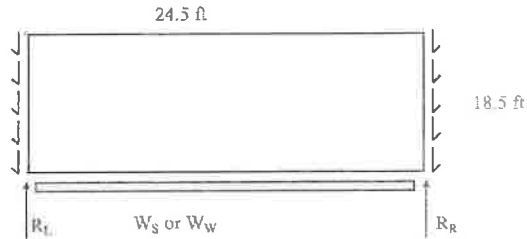
Capacity of 2x4 DF #2

= 1.5 * (3.5-0.5) * 575 psi * 1.6
= 4140 lbs > 2452.0 lbs O.K.

Capacity of 2 - 1/2" dia M.B. thru 2x

= 2 * 1640 * 1.6
= 5248 lbs > 2452.0 lbs O.K.

Roof Sub-Diaphragm Analysis (at Subpurlins) - Line "1" Grids "A" to "G"
Main Subdiaphragm Depth Check:



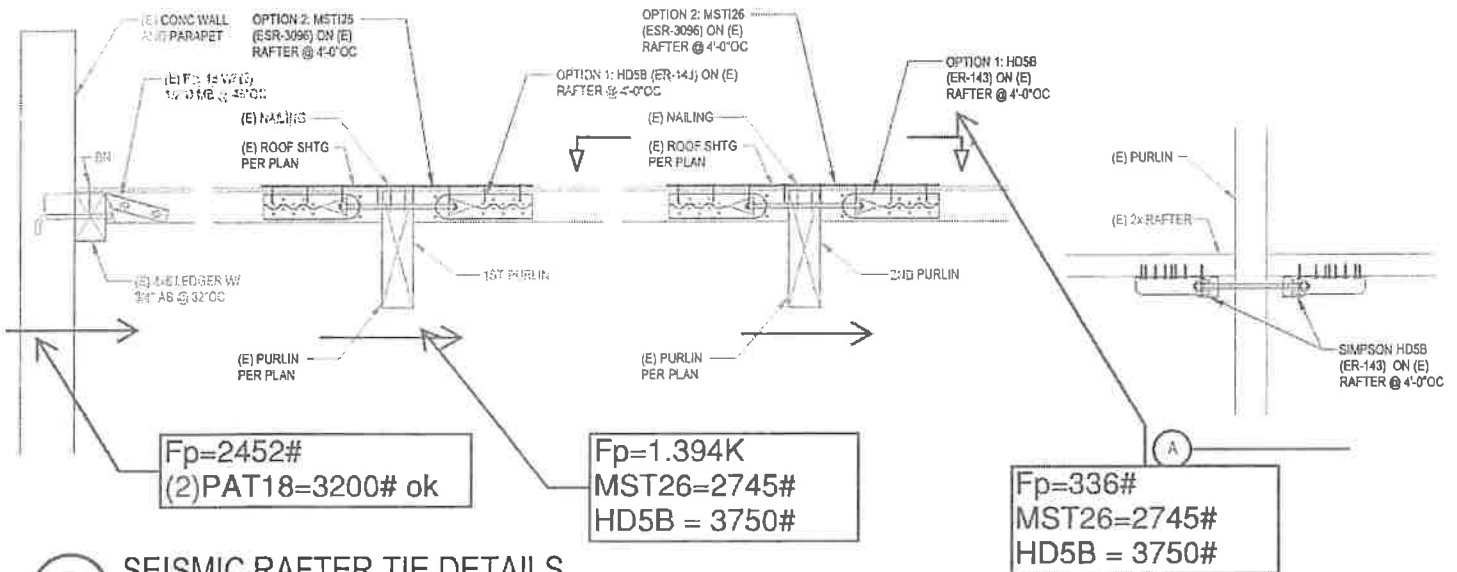
Subdiaphragm Width = 24.5 ft
Diaphragm Capacity = 540.0 plf (ASD)

Subdiaphragm Depth = (ws*Bay Width/2) / (0.75*Diaph. Capacity) = 18.5 ft

☒ LA County Amendments?

	Distance from Wall ft	Strap Spacing ft	Fp lbs	Strap Provided	Nails Provided (Total)	Capacity (adjusted) lbs	Overall Check
At Wall	0.0	4.0	2452				
1st Purlin	8.0	4.0	1394	Simpson MSTI26	26-10d	2745	OK
2nd Purlin	16.0	4.0	336	Simpson MSTI26	26-10d	2745	OK
3rd Purlin	24.0	4.0	0				
4th Purlin	32.0	4.0	0				
5th Purlin	40.0	4.0	0				
6th Purlin	48.0	4.0	0				
7th Purlin	56.0	4.0	0				
8th Purlin	64.0	4.0	0				

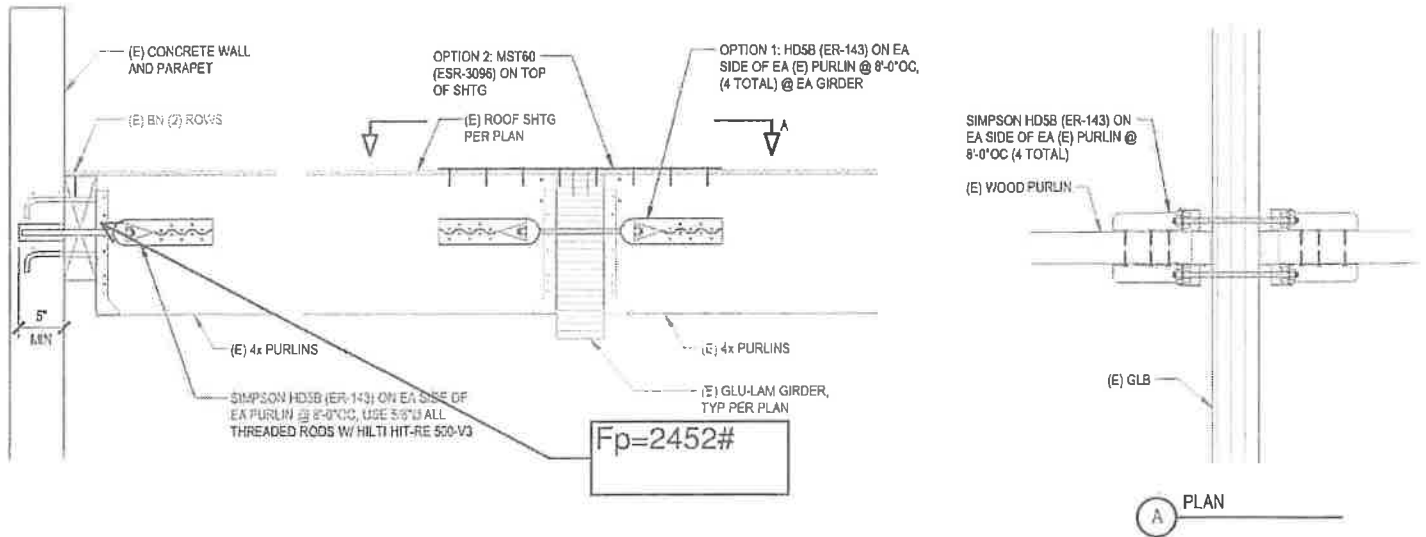
Max Moment = $F_p \times \text{Subdiaphragm Width}^2 / 8 = 45991.83 \text{ lb-ft}$
Chord Depth = Subdiaphragm Depth = 18.54 ft
Chord Force = Max Moment / Chord Depth = 2.48 k (ASD)



5

SEISMIC RAFTER TIE DETAILS

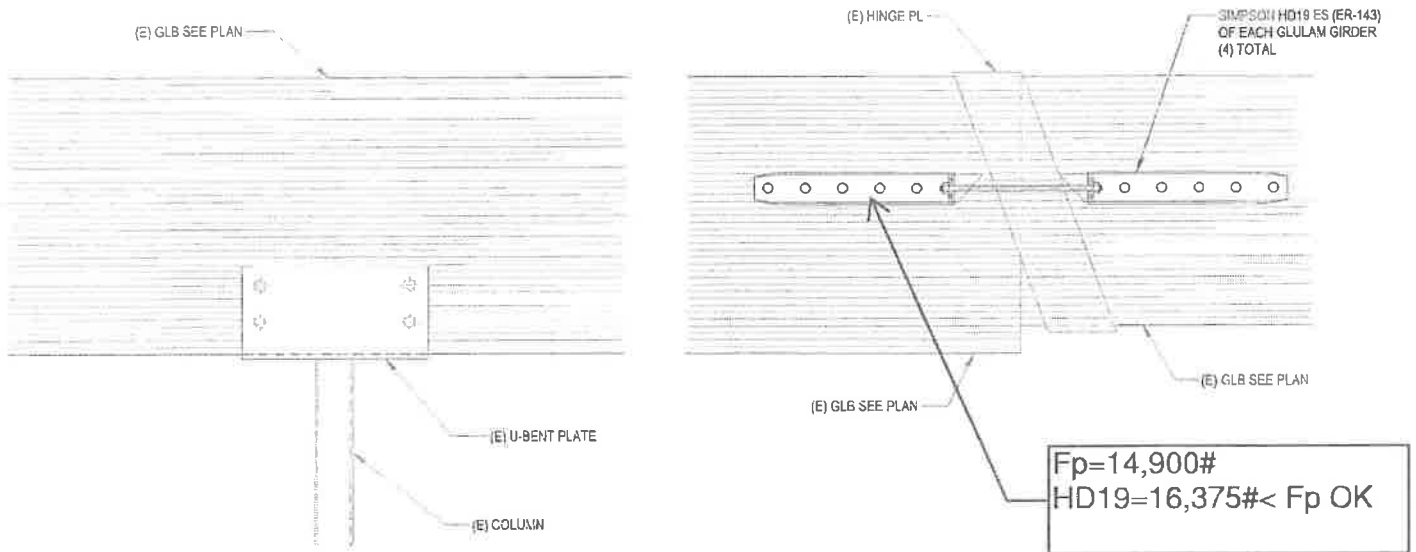
1" = 1'-0"



6

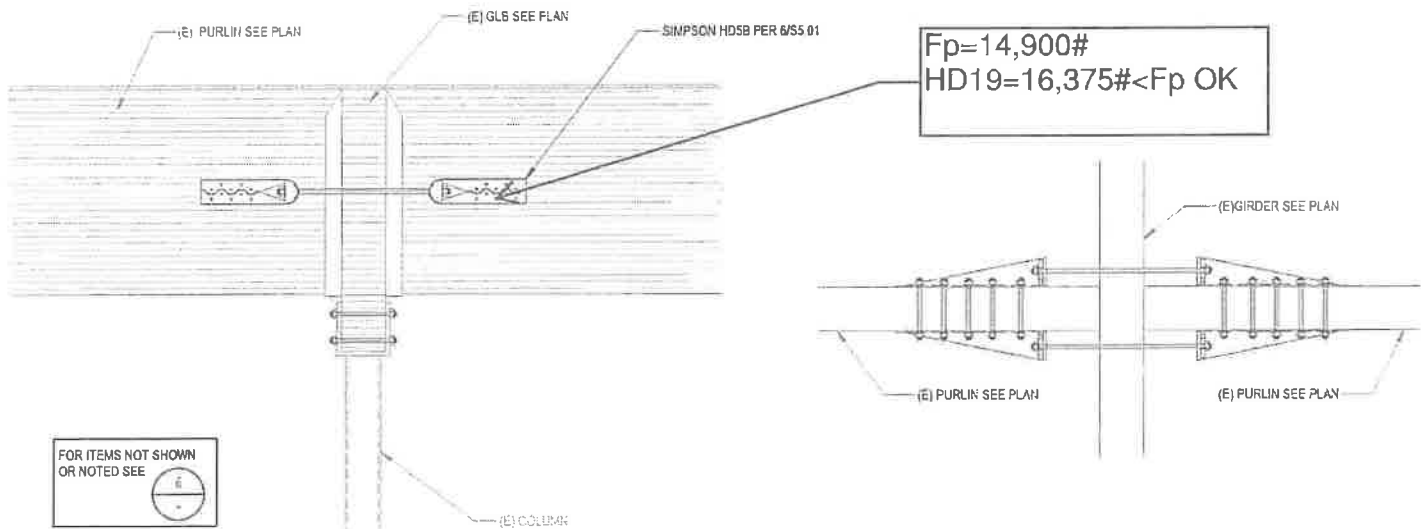
CONTINUOUS CROSS TIES FOR PURLINS

1" = 1'-0"



7 CROSS TIE AT HINGE CONNECTION

1" = 1'-0"



8 CONTINUOUS CROSS TIE AT COLUMN

1" = 1'-0"

Table 12G BOLTS: Reference Lateral Design Values, Z, for Double Shear (three member) Connections^{1,2}

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

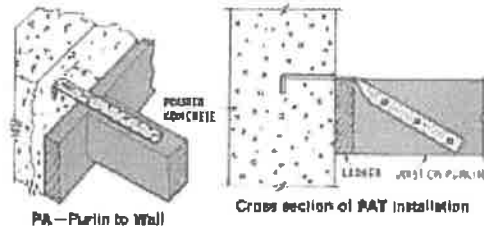
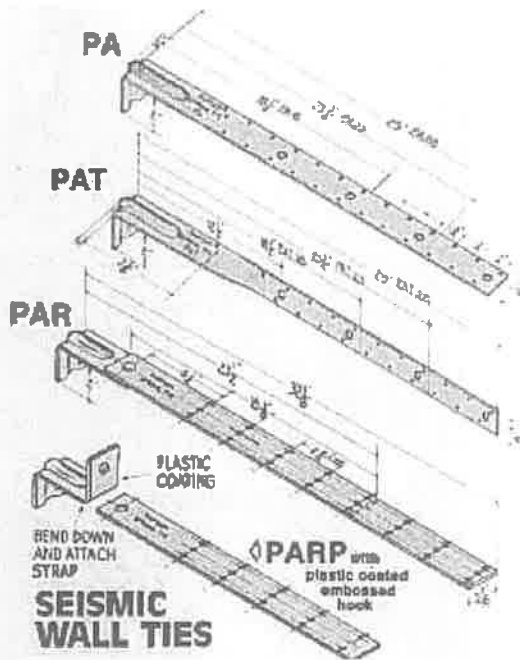


Thickness		Bolt Diameter	Main Member		Side Member		G=0.57 Red Oak		G=0.55 Mixed Maple Southern Pine		G=0.50 Douglas Fir-Larch		G=0.49 Douglas Fir-Larch (N)		G=0.46 Douglas Fir(S) Hem-Fir(N)		G=0.43 Hem-Fir		G=0.42 Spruce-Pine-Fir		G=0.37 Redwood		G=0.36 Eastern Softwoods Southern Pine-Fir(S) Western Cedars Western Woods		G=0.36 Northern Species	
t _a	t _s		D	Z _u	Z _s	Z _u	Z _s	Z _u	Z _s	Z _u	Z _s	Z _u	Z _s	Z _u	Z _s	Z _u	Z _s	Z _u	Z _s	Z _u	Z _s	Z _u	Z _s	Z _u	Z _s	
in.	in.		in.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.
1-1/2	1/4	1/2	1410	730	1150	550	1050	470	1030	460	970	420	900	380	880	370	780	310	760	290	730	290				
		5/8	1760	810	1440	610	1310	530	1280	520	1210	470	1130	420	1100	410	970	350	950	330	910	320				
		3/4	2110	890	1730	660	1580	590	1550	580	1450	520	1350	460	1320	450	1170	370	1140	360	1100	350				
		7/8	2460	960	2020	720	1840	630	1800	600	1690	550	1580	500	1540	490	1360	410	1330	390	1280	370				
		1	2810	1020	2310	770	2100	690	2060	650	1930	600	1800	540	1760	530	1560	440	1520	420	1460	410				
1-3/4	1/4	1/2	1640	850	1350	640	1230	550	1200	530	1130	490	1050	450	1030	430	910	360	890	340	850	330				
		5/8	2050	940	1660	710	1530	610	1500	600	1410	550	1310	490	1280	480	1130	400	1110	380	1070	370				
		3/4	2460	1040	2020	770	1840	660	1800	660	1690	600	1580	540	1540	530	1360	430	1330	420	1280	410				
		7/8	2870	1120	2350	840	2140	740	2110	700	1970	640	1840	580	1800	570	1590	470	1550	460	1490	430				
		1	3280	1190	2690	890	2450	790	2410	750	2250	700	2100	630	2060	610	1820	510	1770	490	1710	470				
2-1/2	1/4	1/2	1870	1210	1720	910	1650	790	1640	760	1590	700	1500	640	1470	610	1300	510	1270	490	1220	480				
		5/8	2740	1340	2400	1020	2190	880	2150	860	2010	790	1880	700	1840	690	1620	580	1580	550	1520	530				
		3/4	3520	1480	2860	1110	2830	980	2580	940	2410	860	2250	770	2200	750	1950	620	1900	600	1830	580				
		7/8	4100	1600	3360	1200	3060	1050	3010	1010	2820	920	2630	830	2570	810	2270	690	2210	680	2130	610				
		1	4690	1700	3840	1280	3500	1130	3440	1080	3220	1000	3000	900	2940	880	2590	730	2530	700	2440	680				
3-1/2	1/4	1/2	1870	1240	1720	1100	1650	1030	1640	1010	1590	970	1540	890	1530	860	1450	720	1430	680	1410	670				
		5/8	2740	1720	2510	1420	2410	1230	2390	1200	2330	1050	2260	980	2230	960	2110	810	2090	770	2060	740				
		3/4	3800	2070	3460	1550	3340	1370	3320	1310	3220	1210	3120	1080	3080	1050	2720	870	2660	840	2560	810				
		7/8	5060	2240	4630	1680	4290	1470	4210	1410	3940	1290	3680	1180	3600	1130	3180	950	3100	920	2990	860				
		1	5520	2380	5380	1790	4900	1580	4810	1510	4510	1400	4200	1280	4110	1230	3630	1020	3540	980	3410	950				
5-1/4	1/4	5/8	2740	1720	2510	1510	2410	1420	2390	1400	2330	1340	2260	1280	2230	1270	2110	1170	2090	1140	2060	1120				
		3/4	3800	2290	3460	2000	3340	1890	3320	1850	3220	1780	3120	1610	3090	1580	2920	1300	2890	1260	2840	1220				
		7/8	5060	2930	4630	2530	4440	2210	4410	2110	4280	1930	4150	1760	4110	1700	3880	1420	3840	1380	3770	1290				
		1	5520	3570	5960	2680	5720	2380	5670	2280	5510	2100	5330	1890	5280	1840	4990	1620	4930	1470	4850	1420				
5-1/2	1/4	5/8	2740	1720	2510	1510	2410	1420	2390	1400	2330	1340	2260	1280	2230	1270	2110	1170	2090	1140	2060	1120				
		3/4	3800	2290	3460	2000	3340	1890	3320	1850	3220	1780	3120	1690	3090	1650	2920	1360	2890	1320	2840	1280				
		7/8	5060	2930	4630	2570	4440	2310	4410	2210	4280	2020	4150	1830	4110	1780	3880	1490	3840	1440	3770	1350				
		1	5520	3640	5960	2810	5720	2480	5670	2370	5510	2200	5330	1980	5280	1930	4990	1600	4930	1540	4850	1490				
7-1/2	1/4	5/8	2740	1720	2510	1510	2410	1420	2390	1400	2330	1340	2260	1280	2230	1270	2110	1170	2090	1140	2060	1120				
		3/4	3800	2290	3460	2000	3340	1890	3320	1850	3220	1780	3120	1690	3090	1670	2920	1530	2890	1500	2840	1480				
		7/8	5060	2930	4630	2570	4440	2410	4410	2360	4280	2260	4150	2160	4110	2130	3880	1960	3840	1930	3770	1840				
		1	5520	3640	5960	3180	5720	3000	5670	2940	5510	2840	5330	2700	5280	2630	4990	2180	4930	2100	4850	2030				
9-1/2	1/4	3/4	3800	2290	3460	2000	3340	1890	3320	1850	3220	1780	3120	1690	3090	1670	2920	1530	2890	1500	2840	1480				
		7/8	5060	2930	4630	2570	4440	2410	4410	2360	4280	2260	4150	2160	4110	2130	3880	1960	3840	1930	3770	1870				
		1	5520	3640	5960	3180	5720	3000	5670	2940	5510	2840	5330	2700	5280	2660	4990	2440	4930	2400	4850	2350				
11-1/2	1/4	7/8	5060	2930	4630	2570	4440	2410	4410	2360	4280	2260	4150	2160	4110	2130	3880	1960	3840	1930	3770	1870				
		1	5520	3640	5960	3180	5720	3000	5670	2940	5510	2840	5330	2700	5280	2660	4990	2440	4930	2400	4850	2350				
13-1/2	1/4	1	5520	3640	5960	3180	5720	3000	5670	2940	5510	2840	5330	2700	5280	2660	4990	2440	4930	2400	4850	2350				

 1. Tabulated lateral design values, Z , for bolted connections shall be multiplied by all applicable adjustment factors (see Table 11.3.1).

 2. Tabulated lateral design values, Z , are for "full-body diameter" bolts (see Appendix Table E.1) with bolt bending yield strength, F_y , of 45,000 psi and dowel bearing strength, T_b , of 87,000 psi for ASTM A36 steel.

Existing Sub-purlin anchors



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 Simpson Strong-Tie Company, Inc.

PA/PAM/PAT PAR/PARP PURLIN ANCHORS

This Purlin Anchor line provides a tested 11,500 lbs. of pull value in 2000 psi concrete. The heavily embossed hook base is embedded 4" into concrete or masonry.

The Purlin Anchor Series offer a variety of wood-to-concrete or wood-to-concrete-block connections that satisfy engineering and code requirements.



PA Nomenclature

PA - Standard straight purlin anchor - attaches to top of purlin.
 PAT - Provides a 5d bar - attaches to the side of purlin.
 PAM - PATM - Provides for 4" of concrete embedment in a masonry block wall (PAM - Straight, PATM - 90° twist).
 PAR - 2 piece riveted anchor - adjusts for misaligned purlin (PAR - Standard embedment, PARM additional embedment see PATM, PATM).
 PARP - 2 piece bolted anchor.

PAR or PARP - Purlin to Wall with W Joist Hanger installed



MODEL NO.	LENGTH	FASTENERS		DESIGN LOADS			
		WALL	BOLE	Normal	Maximum	Normal	Maximum
PA18	18 1/2"	12-18d	2-1/2" MB	1800	2130	1610	2150
PA20	20 1/2"	18-18d	3-1/2" MB	2410	3200	2450	3200
PA28	28"	24-18d	4-1/2" MB	3140	3680	3140	3680
PAT18	18 1/2"	7-18d	2-1/2" MB	940	1175	1610	2150
PAT20	20 1/2"	13-18d	3-1/2" MB	1740	2180	2420	3200
PAT28	28"	18-18d	4-1/2" MB	2550	3190	3140	3680
PAM25	25 1/2"	18-18d	3-1/2" MB	2680	3200	2430	3200
PATM25	25"	13-18d	3-1/2" MB	1740	2180	2430	3200
**PAR	23 1/2"	3-H54A	—	780	1035	—	—
**PARP	23 1/2"	—	—	—	—	—	—
**PARM	26 1/2"	5-H54A	—	1300	1725	—	—
**PARB	30 1/2"	6-H54A	—	1560	2080	—	—
**PARR	30 1/2"	7-H54A	—	1830	2400	—	—
		8-H54A	—	2100	2450	—	—

**Uses 3/8" diameter bolt or rivet A54A fasteners included
 ACCEPTED - See Research Recommendation No. 1211 of the International Conference of Building Officials (Uniform Building Code) insufficient penetration (i.e. 1 1/2" into 2x4) will result in reduced values (1.5 x 75% BE of table ratings)

New girder cross-ties

Model No.	Material		Dimensions (in.)							Fasteners (in.)		Minimum Wood Member Size (in.)	Allowable Tension Loads (160)		Deflection at Highest Allowable Load
	Base (in.)	Body (ga.)	HB	SB	W	H	B	CL	SO	Anchor Dia. Bolt	Stud Bolts		DF/SP	SPF/HF	
HD12	¾	3	7	4	3½	20⅞	4¼	2⅞	3⅝	1	(4) 1	3½ x 3½	11,350	9,215	0.171
												3½ x 4½	12,665	10,765	0.171
												5½ x 5½	14,220	12,085	0.162
										1½	(4) 1	3½ x 3½	11,775	9,215	0.171
												3½ x 4½	13,335	11,055	0.177
												3½ x 7¼	15,435	13,120	0.194
												5½ x 5½	15,510	12,690	0.162
HD19	¾	3	7	4	3½	24½	4¼	2⅞	3⅝	1½	(5) 1	3½ x 7¼	16,735	14,225	0.191
												5½ x 5½	16,775	12,690	0.2
										1¼	(5) 1	3½ x 7¼	19,360	15,270	0.18
												5½ x 5½	19,070	16,210	0.137

1. To achieve published loads, machine bolts shall be installed with the nut on the opposite side of the holdown. If this orientation is reversed, the Designer shall reduce the allowable loads shown per NDS requirements when bolt threads are in the shear plane.
2. All references to bolts are for structural quality through bolts (not lag screw or carriage bolts) equal to or better than ASTM A307, Grade A.
3. HD19 with 1 1/4" anchor rod requires No.1 post (or better) to achieve published loads.

New sub-purlin and purlin cross-ties

These products are available with additional corrosion protection. For more information, see p. 12.

Model No.	Material		Dimensions (in.)							Fasteners (in.)		Minimum Wood Member Size (in.)	Allowable Tension Loads (160)		Deflection at Highest Allowable Load	Code Ref.
	Base (in.)	Body (ga.)	HB	SB	W	H	B	CL	SO	Anchor Bolt Dia.	Stud Bolts		DF/SP	SPF/HF		
HD3B	—	12	4%	2½	2½	8%	2¼	1½	¾	¾	(2) ¾	1½ x 3½	1,895	1,610	0.156	IBC*, FL, LA
												2½ x 3½	2,525	2,145	0.169	
												3 x 3½	3,130	3,050	0.12	
												3½ x 3½	3,130	3,050	0.12	
HD5B	¾	10	5½	3	2½	9%	2½	1¼	2	¾	(2) ¾	1½ x 3½	2,405	2,070	0.153	
												2½ x 3½	3,750	3,190	0.129	
												3 x 3½	4,505	3,785	0.156	
												3½ x 3½	4,935	4,195	0.15	
HD7B	¾	10	5½	3	2½	12%	2½	1¼	2	¾	(3) ¾	3 x 3½	6,645	5,650	0.142	
												3½ x 3½	7,310	6,215	0.154	
												3½ x 4½	7,345	6,245	0.155	
HD9B	¾	7	6½	3½	2½	14	2½	1¼	2½	¾	(3) ¾	3½ x 3½	7,740	6,580	0.159	
												3½ x 4½	9,920	8,430	0.178	
												3½ x 5½	9,920	8,430	0.178	
												3½ x 7½	10,035	8,530	0.179	
HD12	¾	3	7	4	3½	20%	4%	2½	3%	1	(4) 1	3½ x 3½	11,350	9,215	0.171	
										3½ x 4½	12,665	10,765	0.171			
										5½ x 5½	14,220	12,085	0.162			
										1½	(4) 1	3½ x 3½	11,775	9,215	0.171	
												3½ x 4½	13,335	11,055	0.177	
												3½ x 7½	15,435	13,120	0.194	
HD19	¾	3	7	4	3½	24½	4%	2½	3%	1½	(5) 1	3½ x 7½	16,735	14,225	0.191	
										5½ x 5½	16,775	12,690	0.2			
										1¼	(5) 1	3½ x 7½	19,360	15,270	0.18	
												5½ x 5½	19,070	16,210	0.137	

- To achieve published loads, machine bolts shall be installed with the nut on the opposite side of the holdown. If this orientation is reversed, the designer shall reduce the allowable loads shown per NDS requirements when bolt threads are in the shear plane.
- All references to bolts are for structural quality through bolts (not lag screw or carriage bolts) equal to or better than ASTM A307, Grade A.
- HD19 with 1¼" anchor rod requires No. 1 post (or better) to achieve published loads.

New purlin cross-ties Option #2

1985: ST/FHA/MST/HS1
 Strap Ties

Model No.	Clear Span (in.)	Fasteners (Total) (in.)	DF/SP Allowable Tension Loads	SPF/HF Allowable Tension Loads
			(160)	(160)
MST37	24	(14) 0.162 x 2 1/2	1,720	1,500
	18	(20) 0.162 x 2 1/2	2,460	2,140
	16	(22) 0.162 x 2 1/2	2,705	2,355
MST48	24	(26) 0.162 x 2 1/2	3,210	2,780
	18	(32) 0.162 x 2 1/2	3,950	3,425
	16	(34) 0.162 x 2 1/2	4,200	3,640
MST60	30	(34) 0.162 x 2 1/2	4,605	3,995
	24	(40) 0.162 x 2 1/2	5,240	4,700
	18	(46) 0.162 x 2 1/2	6,235	5,405
MST72	30	(48) 0.162 x 2 1/2	6,505	5,640
	24	(54) 0.162 x 2 1/2	6,730	6,345
	18	(62) 0.162 x 2 1/2	6,730	6,475

1. See **General Notes for Straps and Ties**.
2. Install bolts or nails as specified by designer. Bolt and nail values may not be combined.
3. Allowable bolt loads are based on parallel-to-grain loading and minimum member thickness: MST — 2 1/2".
4. Splitting may be a problem with installations on lumber smaller than 3 1/2"; either fill every nail hole with 0.148" x 1 1/2" nails or fill every other hole with 0.162" x 2 1/2" nails. Reduce the allowable load based on the size and quantity of fasteners used.
5. Fasteners: Nail dimensions in the table are listed diameter by length. For additional information, see **Fastener Types and Sizes Specified**

APPENDIX

Section 91.9108 Analysis and Design

For the purpose of this section, "**anchorage system(s)**" shall mean all structural elements, which supports the wall in the lateral direction, including wall anchorage and continuity tie (cross-tie) connectors in subdiaphragms and main diaphragms for retrofit and repairs.

91.9108.1 Wall Panel Anchorage

Concrete walls shall be anchored to all floors and roofs which provide lateral support for the wall. The anchorage shall provide a positive direct connection between the wall and floor or roof construction capable of resisting a horizontal force equal to 30 percent of the tributary wall weight for all buildings, and 45 percent of the tributary wall weight for essential buildings, or a minimum force of 250 pounds per linear foot of wall, whichever is greater. The required anchorage shall be based on the tributary wall panel assuming simple supports at floors and roof.

EXCEPTION: Alternate design may be approved by the Superintendent when justified by well established principles of mechanics.

91.9108.2 Special Requirements for Wall Anchors and Continuity Ties

The steel elements of the wall anchorage systems and continuity ties shall be designed by the allowable stress design method using a load factor of 1.7. The 1/3 stress increase permitted by CBC Section 1605.3.1.1 shall not be permitted for materials using allowable stress design methods.

The strength design specified in CBC Section 1912, using a load factor of 2.0 in lieu of 1.4 for earthquake loading, shall be used for design of embedment in concrete.

Wall anchors shall be provided to resist out-of-plane forces, independent of existing shear anchors.

EXCEPTION: Existing cast-in-place shear anchors may be used as wall anchors if the tie element can be readily attached to the anchors and if the engineer or architect can establish tension values for the existing anchors through the use of approved as-built plans or testing, and through analysis showing that the bolts are capable of resisting the total shear load while being acted upon by the maximum tension force due to earthquake. Criteria for analysis and testing shall be determined by the Superintendent.

Expansion anchors are not allowed without special approval of the Superintendent. Attaching the edge of plywood sheathing to steel ledgers is not considered as complying with the positive anchoring requirements of the Code; and attaching the edge of steel decks to steel ledgers is not considered as providing the positive anchorage of this Code unless testing and/or analysis are performed, which establish shear values for the attachment perpendicular to the edge of the deck.

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