Structural Calculations

For

C & S Family Restaurant - Commercial Building

Owner:
Agustin Marroquin

Project Address / Location:
55795 Twenty Nine 29 Palms Hwy
Yucca Valley, CA 92284
APN# 0586-351-16

Architect / Designer
Monica Hoffmann
57556 Twenty Nine 29 Palms Hwy
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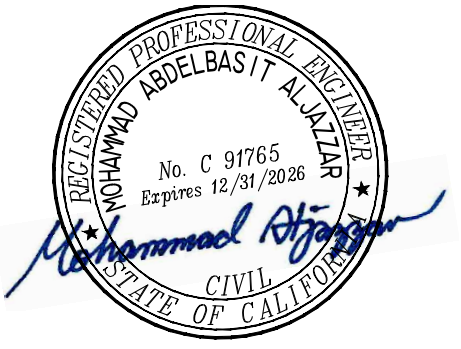

Signed and Sealed Dated:
Friday, June 20, 2025

Revision: **Revision 0** /

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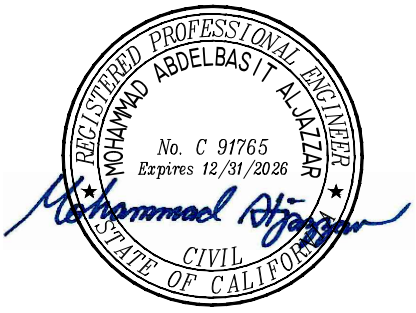


<< END OF CALCULATIONS >>

A. Gravity Design Load Breakdown

A.1 Gravity Loads
Dead Loads &
Roof Live Loads

Structure Loadings

1. Gravity Loading

1.1 Roof Areas [Self-weight and Superimposed Dead]:

a) Roof Above Conditioned Space θ = 1.2 | 0.25 | : 12 Slope

Material	psf.	psf.	psf.
Solar Panels:			
Photo Voltaic Panels allowance ***	4 **	4 **	4 **
Roof System:			
Roof Covering TPO Membrane (or PVC).[1]	0.5	0.5	0.5
Cover Sheild / Fire-rated Board (5/8" Gypsum or Cement Board)	2.5	2.5	2.5
Thermal Insulation[3]	1.5	1.5	1.5
MVP Vapor Retarder.	0.5	0.5	0.5
Douglas Fir Sheathing * : 3/4" Plywood / *OSB*	*2.5*	*2.5*	*2.5*
Insulation Below Roof Deck (Option C: None) (Option B: R13 - 3.5") 0.1pcf/1'	0.5	0.5	0.5
Fire Sprinklers, HVAC, Ducts (MEP)	4.0	4.0	4.0
Miscellaneous	2.0		
A. (Roof Finishes) Sheathing Rating & Framing/Roof Truss (Top Chord)	**14**		
Ceiling (Soffit) Framing:			
Ceiling Not Finished. Exposed (Spray Painted)			
Roof Joists 230 TJI 14"@ 24"o.c.			2.0
Miscellaneous			2.0
B. Total Load Applied as Roof Dead including Roof Truss			**16**

b) Roof Above Un-Conditioned Space θ = 4.8 | 1 | : 12 Slope

Material	psf.
Roof System:	
Standing Seam Roof Panel [4]	2.0
Douglas Fir Sheathing * : 1/2" Plywood / *OSB*	*2.0*
Fire Sprinklers	2.0
Miscellaneous	2.0
A. (Roof Finishes) Sheathing Rating (Framing weight Not Included)	**8**
Ceiling (Soffit) Framing:	
Ceiling Not Finished. Exposed (Spray Painted)	
Roof Joists TJI @ 24"o.c.	2.0
B. Total Load Applied as Roof Dead including Roof Framing	**10**

Roofing Notes:

* *For Dead Load Only Load Combination with E (Seismic) and For calculating the Seismic Weight W_s USE [Add + 4 psf for PV] where applies*

** PhotoVoltaic Panels use 4 psf min. per CBC Code ,
However the weight is displaced by the Roof Live Load, (D = 4 psf / [0.9 Duration Factor] = 4.44 psf << Lr = 16 psf/ [1.25 Duration Factor] = 12.8 psf)

[1] Single Ply Roofing i.e. TPO: 45 mil is 0.23 psf, 60 mil is 0.31 psf. PVC: 45 mil is 0.30 psf, 60 mil is 0.40 psf.

[3] Thermal Insulation, w/ overlapping/staggered joint
Extruded Polystyrene (XPS) Rigid Board (R5/in) 8"x 1.5 pcf.
Polyisocyanurate (Polyiso) Rigid Board (R6.5) 6" x 3 pcf.

[4] Weight per square foot varies per gauge, profile and material;
Cold Formed Coated Steel Panels i.e. 20 ga. Max. 2.68 psf > 22 ga. (1/32") > 24 ga. Min. 1.40 psf, Aluminum panels 0.040in at 0.75 psf min.
Corrugated Sheet 20 gauge 2 psf max (Non Loaded and supported by the roof deck plywood, and rafters)

1.2 Wall Framing and Finishes: [Dead]

a) Exterior Walls:

Typicla Exterior Wall: Material (From outside to Inside)	psf.
Exterior Finishes: 7/8" Stucco 10.00 psf ,	10.00
Douglas Fir Sheathing 1/2" Plywood / *OSB*	*1.70*
x6 Framing;	**1.70**
Insulation (Cavity) 0.1pcf/1"	0.30
Interior: Gypsum Board [1-Layer, 1/2" Min. or 5/8" Max.]	2.80
Miscellaneous	1.50
A. Exterior Walls (x6 Framing)	**18.00**

At Front Wall Facing 29-Palms Hwy: Material (From outside to Inside)	psf.	
Exterior Finish: Board and Batten Siding	3.00	
Douglas Fir Sheathing 1/2" Plywood / *OSB*	*1.70*	
x6 Framing;	**1.70**	
Insulation (Cavity) 0.1pcf/1"	0.30	
Interior: Gypsum Board [1-Layer, 1/2" Min. or 5/8" Max.]	2.80	
Miscellaneous	1.50	
A. Exterior Walls (x6 Framing)	**11.00**	***Use 12 psf***

b) Interior Walls

***[for x6 Wall Configurations (ADD + 1.00 psf)]**

Material (From outside to Inside)	psf.	psf.
Gypsum Board [1-Layer, 1/2" Min. or 5/8" Max.]	2.80	2.80
Douglas Fir Sheathing 1/2" Plywood / *OSB*		*1.70*
2x Framing	**1.10**	**1.10**
Insulation (Cavity) 0.1pcf/1"	0.40	0.40
Gypsum Board [1-Layer, 1/2" Min. or 5/8" Max.]	2.80	2.80
Miscellaneous	1.90	2.20
A. Interior Wall	**9.00***	
B. Interior Wall [at Kitchens, ADA Bathroom and Shear walls]		**11.00***

2.0 Roof Areas [Live]

Commercial Live Loads:	**Area Load [psf]**	**Concentrated Load [lbs.] [1]**
L_r : Roof - Flat	20	300

Live Load Notes:

* No Live Load Reduction; $L_o \geq 100$ psf or $A_T < 200$ ft²

[1] Mid-span for flexure / at "d" from supports for shear, for beams apply full concentrated load, for joists or rafter use (Spacing in. / 30 in.) x Conc. Load. Concentrated Load allowed to be applied at an area 2.5 ft x 2.5 ft

A.2 Snow Load & Design Parameters

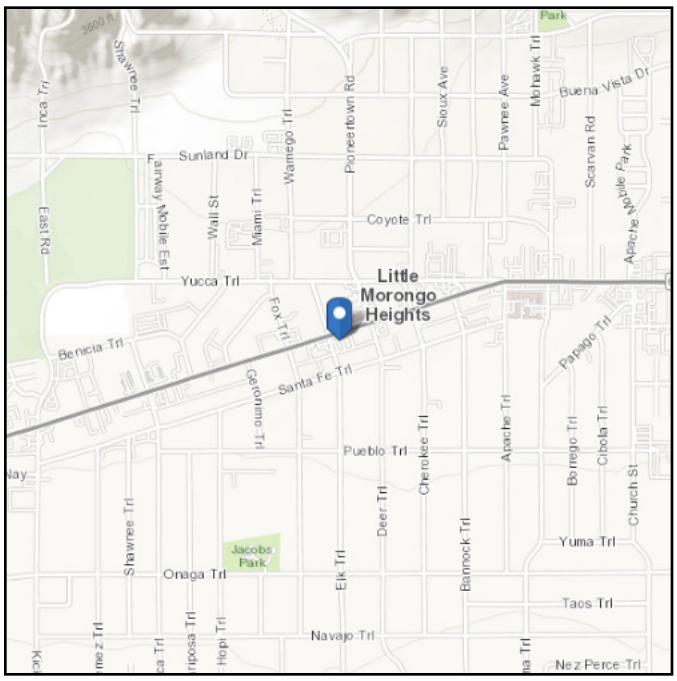
* ASCE 7-16 HAZARDS REPORT
* Riverside County Handout

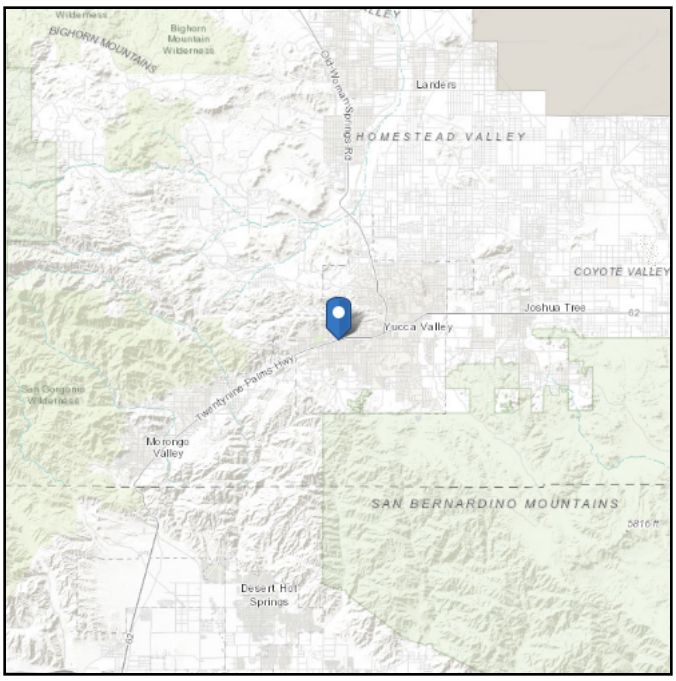
ASCE Hazards Report

SNOW

Address:
55795 29 Palms Hwy
Yucca Valley, California
92284

Standard:	ASCE/SEI 7-16	**Latitude:**	34.118916
Risk Category:	II	**Longitude:**	-116.447035
Soil Class:	D - Default (see Section 11.4.3)	**Elevation:**	3326.2407060656838 ft (NAVD 88)





Snow

Results:

Ground Snow Load, p_g:	5 lb/ft^2
Mapped Elevation:	3326.2 ft
Data Source:	ASCE/SEI 7-16, Table 7.2-8
Date Accessed:	Thu May 15 2025

SEE ASCE7-22 Snow Load Requirements

Values provided are ground snow loads. In areas designated "case study required," extreme local variations in ground snow loads preclude mapping at this scale. Site-specific case studies are required to establish ground snow loads at elevations not covered.

Snow load values are mapped to a 0.5 mile resolution. This resolution can create a mismatch between the mapped elevation and the site-specific elevation in topographically complex areas. Engineers should consult the local authority having jurisdiction in locations where the reported 'elevation' and 'mapped elevation' differ significantly from each other.

Threshold Height for Snow Load
Up to 2,000 ft , pg = 0 psf
2,000 ft -- 3,600 ft , pg = 5 psf

site at h = 3,326 < 3,600 ft.

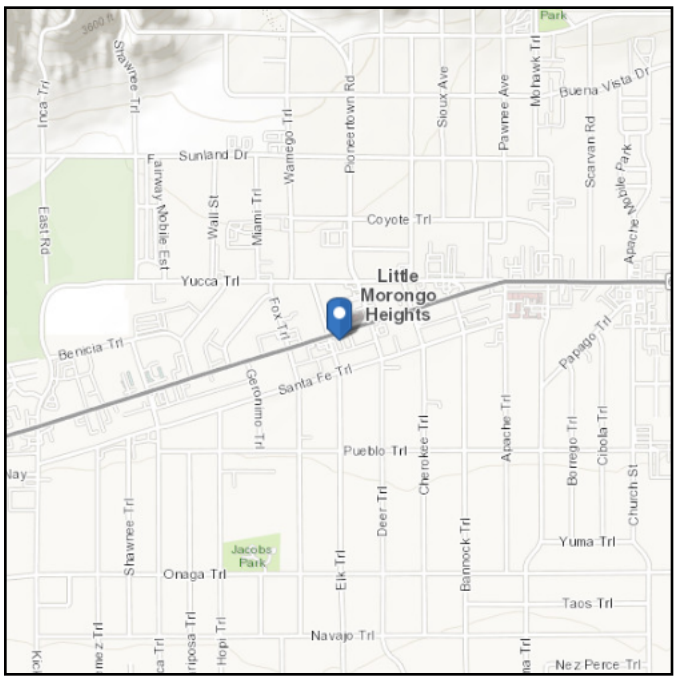
ASCE Hazards Report

SNOW

Address:
55795 29 Palms Hwy
Yucca Valley, California
92284

Standard: ASCE/SEI 7-22
Risk Category: II
Soil Class: Default

Latitude: 34.118916
Longitude: -116.447035
Elevation: 3326.2407060656838 ft (NAVD 88)



Snow

Results:

Ground Snow Load, p_g:	24 lb/ft^2
20-year MRI Value:	6.3 lb/ft^2
Winter Wind Parameter:	0.25
Mapped Elevation:	3510.2 ft
Data Source:	ASCE/SEI 7-22, Figures 7.6-1 and 7.6-2 A-D
Date Accessed:	Thu May 15 2025

Snow Design Loads will be in accordance to ASCE7-22 Snow Load, and ASCE7-22 Load Combinations

Values provided are ground snow loads. In areas designated "case study required," extreme local variations in ground snow loads preclude mapping at this scale. Site-specific case studies are required to establish ground snow loads at elevations not covered.

Snow load values are mapped to a 0.5 mile resolution. This resolution can create a mismatch between the mapped elevation and the site-specific elevation in topographically complex areas. Engineers should consult the local authority having jurisdiction in locations where the reported 'elevation' and 'mapped elevation' differ significantly from each other.

SAN BERNARDINO COUNTY	Land Use Services Building and Safety Division **Information Bulletin**	**Number:** IB-0003
		Code References: 2019 CBC ASCE 7-16
Building Official Signature: Jack Leonard, PE, CBO		**Original Effective Date:** November 28, 1972 **Updated:** July 2, 2020
Subject: Rainfall Water Flow Rate, Wind Speed Requirements, and Snow Load Design Values		

1.0 PURPOSE

The purpose of this Information Bulletin is to provide the rainfall water flow rate, wind speed requirements, and snow load design values for the communities listed below and all other areas where the elevation exceeds 3,600 feet above sea level. American Society of Civil Engineers (ASCE) 7-16 requires snow loads above 3,600 feet elevation to be determined by the authority having jurisdiction.

2.0 HISTORY

Original Effective Date: November 28, 1972; Updated: July 2, 2020

3.0 PROCEDURE

Rainfall Water Flow Rate

A. Rainfall Water Flow Rate: Throughout the County of San Bernardino, the recognized rainfall water flow rate is 2.5 inches per hour for a 100-year return.

B. Alternatively, the rainfall water flow rate may be determined based on the site location using the program available at:

https://hdsc.nws.noaa.gov/hdsc/pfds/pfds_map_cont.html?bkmrk=ca

Use the design value of an hour for a 100-year return in inch.

Wind Speed Requirements

Wind loads shall be based on the California Building Code (CBC) Section 1609 with the following design criteria:

A. Minimum Exposure C category shall be used throughout the County of San Bernardino. The ultimate design wind speed, V_{ult} (mph), shall be determined by Figures 1609.3(1), 1609.3(2) and 1609.3(3) of CBC. Design values of 3-second gust wind speeds per Risk Category are as follows:

	V_{ult}	Special Wind Region
Risk Category I:	95 mph	110 mph
Risk Category II:	95 mph	120 mph
Risk Category III & IV:	107 mph	125 mph

B. Alternatively, the ultimate design wind speed may be determined based on the site location using the program available at: http://windspeed.atcouncil.org/

Snow Load Design Values

A. Snow loads shall be considered for the communities listed below in addition to the dead loads of the structure. Unbalanced loads shall be considered.

B. Potential accumulation of snow at valleys, parapets, roof structures and offsets in roofs of uneven configuration shall be considered.

C. All building exits under down-slope eaves having a slope greater than 4:12 shall be protected from sliding snow and ice.

D. Roof members supporting plaster shall be designed for deflection.

E. Snow loads shall not be reduced due to the tributary area.

F. The roof snow loads given in the table below may be reduced due to the slope of the roof when the pitch meets or exceeds 7:12. The roof snow loads may be further modified in accordance with the procedures found in ASCE 7-16. Ground snow loads shall not be a factor where roof snow loads have already been determined in accordance with this procedure. Minor adjustments to reduce the design snow load may be made by the Building Official when he or she has knowledge of less snowfall in a particular location.

G. 20% of the uniform design snow load, regardless of actual roof slope, shall be included in effective seismic weight of the structure where the flat roof snow load exceeds 30 psf.

AREA	ROOF SNOW LOAD (psf)	AREA	ROOF SNOW LOAD (psf)
Angelus Oaks	75	Lake Gregory	75
Arrowbear Lake	75	Minnelusa	75
Arrowhead Highlands	75	Moonridge	75
Baldwin Lake	75	Mt. Baldy	75
Barton Flats	100	Oak Glen	50
Big Bear City	75	Oak Hills (> 3,600')	30
Big Bear Lake	100	Onyx Summit	100
Blue Jay	100	Phelan (> 3,600')	30
Cajon Canyon	50	Pinion Hills (> 3,600')	30
Camp Angeles	75	Rim Forest	50
Cedarpines Park	50	Running Springs	75
Crestline	75	Seven Oaks	75
Erwin Lake	75	Skyforest	75
Fallsvale (Forest Falls)	75	Smiley Park	50
Fawnskin	75	Snow Valley	75
Forest Home	75	Sugarloaf	75
Green Valley	100	Twin Peaks	75
Lake Arrowhead	75	Valley of Enchantment	75
		Wrightwood	50

Note:

1. For communities not listed above use the snow load design values from adjacent communities. Elevations above 3,600 feet use a minimum of 30 psf.
2. Frost Line Depth: 18 inches of frost line depth shall be used for the foundation design in Mountain Region.

Snow Load **Typical All Roofs**

Roof Pitch θ = 1.8 0.375 :12 Roof Slope

min. 3/8": 12" to Avoid Ponding

Ground Snow Load:

p_g = 24.00 psf Ground Snow Load Risk Cat. II

Site Height = 3,326 ft

W_2 = 0.25 *Winter Wind Parameter*

γ = 17.12 psf, $\gamma = 0.13\, p_g + 14$ psf $\leq$ 30 psf Snow Density ASCE 7-22 (Eq. 7.7-1)

Flat Roof Snow Load:

$p_f = 0.7\, C_e\, C_t\, p_g$ ASCE 7-22 (Eq. 7.3-1)

C_e = 0.9 Exposure Factor ASCE 7-22 Table 7.3-1 [Terrain C, Fully Exposed]

C_t = 1.18 Thermal Factor ASCE 7-22 Table 7.3-2 / 7.3-3

p_f = 17.8 psf

Minimum Snow Load: ***for Roof Slopes < 15 °*** **Required** ASCE7-22 (7.3.3)

for $p_g \leq p_{m,max}$ = 30 psf per ASCE7-22 (Table 7.3-4)

$p_m = p_g$

for $p_g > p_{m,max}$

$p_m = p_{m,max}$ p_m = 24 psf Uniform - full span as a separate Load Case

Sloped Roof Snow Load:

$p_s = C_s\, p_f$ ASCE 7-22 (Eq. 7.4-1)

C_s = 1.0 Slope Factor ASCE 7-22 Fig. 7.4-1

p_s = 17.8 psf

For Parapets:

$h_d = 1.5 [(P_g^{0.74}) (l_u^{0.70}) (W_2^{1.7}) / \gamma]$ Design Drift Ht. = $3/4\ h_d \leq h_c$ Clear Ht. Abv. Balanced Snow

Design Drift Length = $w = 8\ (0.75\ h_d)$

		Parapet Ht. (ft)	Upwind Length $l_{u,upwind}$ (ft)	Balanced Snow Ht. h_b (ft)	Clear Ht. Abv. Balanced Snow h_c (ft)	Design Drift Ht. h_b (ft)	Design Drift Width. w (ft)	Drift Surcharge p_d (psf)
Line (B)	*Wind Blowing From North*	3.167	28.000	1.042	2.125	0.871	6.965	14.9
Line (B) High	*Wind Blowing From South*	1.500	70.000	1.042	0.458	0.458	3.663	7.8
Line (B) Low	*Wind Blowing From South*	3.000	70.000	1.042	1.958	1.200	9.598	20.5
Line (F) High	*Wind Blowing From North*	1.500	70.000	1.042	0.458	0.458	3.663	7.8
Line (F) Low	*Wind Blowing From North*	3.000	70.000	1.042	1.958	1.200	9.598	20.5
Line (1)	*Wind Blowing from East*	1.500	45.000	1.042	0.458	0.458	3.663	7.8
Line (3)	*Wind Blowing from West*	3.000	45.000	1.042	1.958	1.028	8.223	17.6

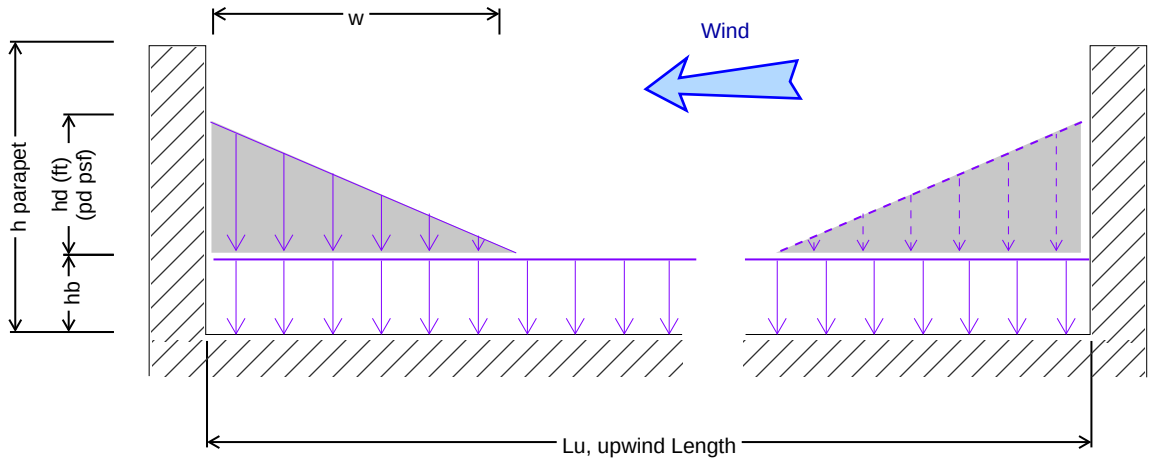


Table 7.3-1. Exposure Factor, C_e.

Surface Roughness Category	Exposure of Roof[a]		
	Fully Exposed[b]	Partially Exposed	Sheltered
B (see Section 26.7)	0.9	1.0	1.2
C (see Section 26.7)	0.9	1.0	1.1
D (see Section 26.7)	0.8	0.9	1.0
Above the tree line in windswept mountainous areas	0.7	0.8	N/A
In Alaska, in areas where trees do not exist within a 2 mi (3 km) radius of the site	0.7	0.8	N/A

Table 7.3-4. Minimum Snow Loads for Low-Slope Roofs.

Risk Category	$p_{m,max}$
I	25 lb/ft^2 (1.20 kN/m^2)
II	30 lb/ft^2 (1.44 kN/m^2)
III	35 lb/ft^2 (1.68 kN/m^2)
IV	40 lb/ft^2 (1.92 kN/m^2)

Table 7.3-2. Thermal Factor, C_t.

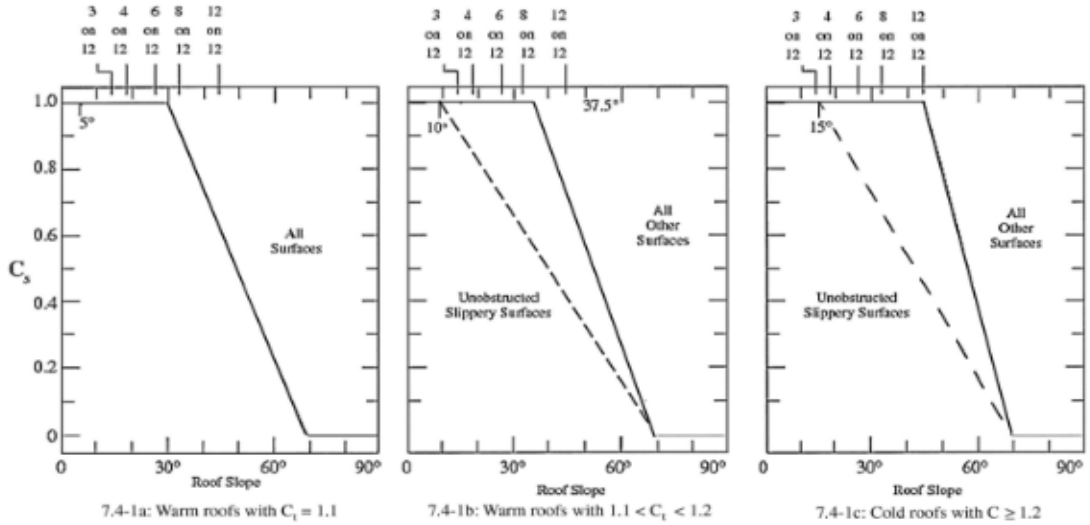
Thermal condition[a]	C_t
All structures except as indicated as follows	See Table 7.3-3
Unheated structures, open-air structures, structures kept just above freezing [40 to 50 °F (4 to 10 °C)], and other structures with cold, ventilated roofs meeting the minimum requirements of the applicable energy code	1.2
Freezer building	1.3
Continuously heated greenhouses[b] with a roof having a thermal resistance (R-value) less than 2.0 h·ft^2·°F/Btu (0.4 m^2·K/W) or a thermal transmittance (U-factor) greater than 0.5 Btu/h·ft^2·°F (2.5 W/m^2·K)	0.85

[a] These conditions shall be representative of the anticipated conditions during winters for the life of the structure.
[b] Greenhouses with a constantly maintained interior temperature of 50°F (10°C) or more, at any point 3 ft (0.9 m) above the floor level during winters and having either a maintenance attendant on duty at all times or a temperature alarm system to provide warning in the event of a heating failure.

Table 7.3-3. Thermal Factor, C_t, for Heated Structures with Unventilated Roofs.[a]

R_{roof} (h·ft^2·°F/Btu [m^2·K/W])	U_{roof} (Btu/h·ft^2·°F [W/m^2·K])	P_g (psf [kPa])						
		≤10 [0.48]	20 [0.96]	30 [1.44]	40 [1.92]	50 [2.40]	60 [2.88]	≥70 [3.36]
≤20 [3.52]	≥0.050 [0.284]	1.20	1.11	1.05	1.01	1.00	1.00	1.00
30 [5.28]	0.033 [0.189]	1.20	1.17	1.14	1.13	1.12	1.11	1.10
40 [7.04]	0.025 [0.142]	1.20	1.19	1.17	1.16	1.16	1.15	1.15
50 [8.80][b]	0.020 [0.114][b]	1.20	1.20	1.19	1.19	1.19	1.18	1.18

[a] For values of P_g and R_{roof} that fall between those shown in the table, linear interpolation may be used to determine the value of C_t.
[b] For values of R_{roof} > 50 h·ft^2·°F/Btu (8.80 m^2·K/W) or U_{roof} < 0.020 Btu/h·ft^2·°F (0.114 W/m^2·K), C_t should be taken as equal to 1.2.



7.4-1a: Warm roofs with $C_t = 1.1$ 7.4-1b: Warm roofs with $1.1 < C_t < 1.2$ 7.4-1c: Cold roofs with $C \geq 1.2$

Figure 7.4-1. Graphs for determining slope factor, C_s.
Note: See Tables 7.3-2 and 7.3-3 for C_t definitions.

B. Lateral Design Load Calculation

B.1 Lateral Forces Design

Seismic Load Calculation

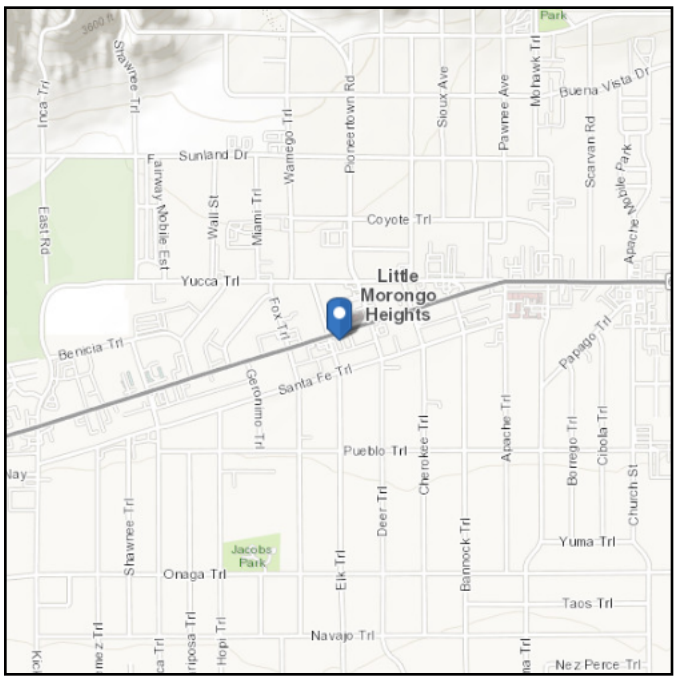
SEISMIC

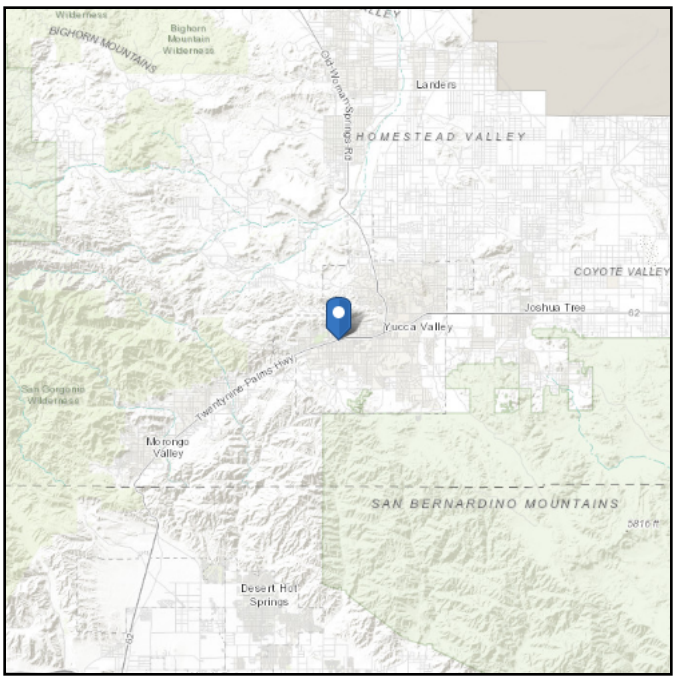
ASCE Hazards Report

Address:
55795 29 Palms Hwy
Yucca Valley, California
92284

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

Latitude: 34.118916
Longitude: -116.447035
Elevation: 3326.2407060656838 ft (NAVD 88)





Seismic

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

Results:

S1 > 0.75 , SDC: E

S_S :	2.221	S_{D1} :	N/A
S_1 :	0.794	T_L :	8
F_a :	1.2	PGA :	0.948
F_v :	N/A	PGA_M :	1.137
S_{MS} :	2.666	F_{PGA} :	1.2
S_{M1} :	N/A	I_e :	1
S_{DS} :	1.777	C_v :	1.5

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

Data Accessed: Thu May 15 2025

Date Source: USGS Seismic Design Maps

Seismic Design Parameters per Geotech Report

Based on the exception, the spectral response accelerations presented below were calculated using the site coefficients (Fa and Fv) from tables 1613.2.3(1) and 1613.2.3(2) presented in Section 16.4.4 of the 2022 CBC.

Parameter		Value
Mapped MCE_R Acceleration at 0.2 sec Period	S_s	2,22
Mapped MCE_R Acceleration at 1.0 sec Period	S_1	0,79
Site Class	---	D
Site Modified Spectral Acceleration at 0.2 sec Period	S_{MS}	2,66
Site Modified Spectral Acceleration at 1.0 sec Period	S_{M1}	1,34
Design Spectral Acceleration at 0.2 sec Period	S_{DS}	1,77
Design Spectral Acceleration at 1.0 sec Period	S_{D1}	0,89

S1 > 0.75 , SDC: E

Seismic Calculation — 1-Story Building

w/ Sheated Wall Panels

Site Parameters :

Site Class : Site Class D — *See Geotech Report*

S_S =	2.221			S_{DS} =	1.777	Fa =	1.200
S_1 =	0.794	> 0.75	SDC (E)	S_{D1} =	0.900	Fv =	1.700

Seismic Design Parameters:

* Risk Category = II
* Importance Factor I_e = 1.00
* Seismic Design Category : **E**

Per 11.4.8 Site specific ground motion Procedures, required for Site Class D also see footnote a Table 11.4-2
Site response analysis shall be performed per Ch.20.3.1 , However not required per exception (2) where:
Structures on site Class D
S1 ≥ 0.2 provided that seismic load is calculated for building with;
T ≤ 1.5 Ts , Cs is per Eq. 12.8-2 i.e. plateu extended to 1.5 Ts
T > 1.5 Ts , Cs' = 1.5 Cs is per Eq. 12.8-3 or 12.8-4 i.e. Cs is multiplied by 1.5

Fv =	1.70	for calculation of Ts
S_{D1} =	0.900	for calculation of Ts
Ts =	0.506	sec.
1.5 Ts =	0.760	sec.
T max =	0.213	

T,max < 1.5 Ts Cs calculated per Equation 12.8-2

Lateral Force Resisting System:

* Seismic Force-Resisting System, Bearing Wall systems,
  Light-frame (wood) walls sheathed with wood structural panels rated for shear resistance.
  Simpson Strong Wall ICC-ER-ESR-2652
* Response Modification Coefficient, R = 6 1/2
* Deflection Amplification Factor, C_d = 4
* Overstrength Factor, Ω_o = 2 1/2 Per Footnote b, ASCE 7-16 Table 12.2-1 for Flexible Diaphragm 1/2 less the tabulated Ωo = 3
* Redundancy Factor, ρ = 1.3 **SDC E**

Period Determination : *From Table 12.8-2*

Building Height (ft) = 15.00 *Average Roof Ht.* C_t = 0.02

Approximate period per Eq. 12.8-7 ; ($T_a = C_t H_n^x$) T_a = 0.152 s x = 0.75

C_u = 1.40 T max = T_a x C_u = 0.213 s

Base Shear Coefficient :

$C_s = S_{DS} / (R/I_e)$	0.273	*Eq. 12.8-2*
$C_{s,max} = S_{D1} / T(R/I_e)$	0.908	*Eq. 12.8-3*
$C_{s,min} = 0.044 S_{DS} I_e \geq 0.01$	0.078	*Eq. 12.8-5*
$C_{s,min} = 0.5 S_1 / (R/I_e)$ for $S_1 \geq 0.6$	0.061	*Eq. 12.8-6*

Cs = 0.273

Cs with ρ , ρ Cs = 0.355

Cs with Ω , Ω Cs = 0.683 Ωo = 2 1/2

Cs with Ω , Ω Cs = 0.819 Ωo = 3

Roof Diaphragm

East-West Direction

Seismic Mass and Seismic Load Per Line

[ASD] **0.7**
ρ = **1.3**
Cs = 0.273

	L (ft)	Ht. (ft)	psf	Wt. (lbs)		0.7 ρ E (lbs) 34,304		Line A [Wall] 0.7 ρ E (lbs) 1,274		Line A [Side Facing B] 0.7 ρ E (lbs) 2,340		Line B [Side Facing A] 0.7 ρ E (lbs) 1,964		Line B [Wall] 0.7 ρ E (lbs) 4,192		Line B [Side Facing D] 0.7 ρ E (lbs) 5,843		Line D [Side Facing B] 0.7 ρ E (lbs) 5,843		Line D [Wall] 0.7 ρ E (lbs) 702		Line D [Side Facing F] 0.7 ρ E (lbs) 4,808		Line F [Side Facing D] 0.7 ρ E (lbs) 4,621		Line F [Wall] 0.7 ρ E (lbs) 2,717
						Ws (lbs) 138,084		Ws (lbs) 5,130		Ws (lbs) 9,419		Ws (lbs) 7,907		Ws (lbs) 16,875		Ws (lbs) 23,519		Ws (lbs) 23,519		Ws (lbs) 2,828		Ws (lbs) 19,354		Ws (lbs) 18,600		Ws (lbs) 10,935
Interior Walls																										
Interior Partitions Between B & D:																										
-Future Use-	60.00	9.75	9	5,265	50%	2,633		0		0		0		0	50%	1,316	50%	1,316		0		0		0		0
Interior Partitions Between D & F:																										
Restrooms	18.58	9.75	11	1,993	50%	997		0		0		0		0		0		0		0	34%	335	66%	661		0
	12.92	9.75	11	1,385	50%	693		0		0		0		0		0		0		0	68%	470	32%	223		0
	7.50	9.75	11	804	50%	402		0		0		0		0		0		0		0	49%	195	51%	207		0
	5.42	9.75	11	581	50%	290		0		0		0		0		0		0		0	58%	168	42%	123		0
	3.67	9.75	11	393	50%	197		0		0		0		0		0		0		0	42%	82	58%	115		0
	12.92	9.75	11	1,385	50%	693		0		0		0		0		0		0		0	35%	239	65%	454		0
	7.50	9.75	11	804	50%	402		0		0		0		0		0		0		0	21%	83	79%	320		0
	5.42	9.75	11	581	50%	290		0		0		0		0		0		0		0	12%	34	88%	257		0
	3.67	9.75	11	393	50%	197		0		0		0		0		0		0		0	28%	54	72%	142		0
Kitchen	21.75	9.75	9	1,909	50%	954		0		0		0		0		0		0		0	100%	954	0%	0		0
	28.00	9.75	9	2,457	50%	1,229		0		0		0		0		0		0		0	52%	636	48%	592		0
	21.75	9.75	9	1,909	50%	954		0		0		0		0		0		0		0	40%	381	60%	574		0
	16.00	9.75	9	1,404	50%	702		0		0		0		0		0		0		0	69%	483	31%	219		0
-Future Use-	20.00	9.75	9	1,755	50%	878		0		0		0		0		0		0		0	80%	702	20%	176		0
Exterior Walls																										
Line A	45.00	9.50	12	5,130	100%	5,130	100%	5,130		0		0		0		0		0		0		0		0		0
Line B Dbl. Wall	45.00	15.00	25	16,875	100%	16,875		0		0		0	100%	16,875		0		0		0		0		0		0
Line D - Interior Shear Wall	21.75	10.00	13	2,828	100%	2,828		0		0		0		0		0		0	100%	2,828		0		0		0
Line F	45.00	13.50	18	10,935	100%	10,935		0		0		0		0		0		0		0		0		0	100%	10,935
Exterior Walls																										
Line 1 A-B	25.00	10.50	18	4,725	50%	2,363		0	50%	1,181	50%	1,181		0		0		0		0		0		0		0
Line 1 B-D	42.00	13.50	18	10,206	50%	5,103		0		0		0		0	50%	2,552	50%	2,552		0		0		0		0
Line 1 D-F	28.00	13.50	18	6,804	50%	3,402		0		0		0		0		0		0		0	50%	1,701	50%	1,701		0
Line 3 A-B	25.00	10.50	18	4,725	50%	2,363		0	50%	1,181	50%	1,181		0		0		0		0		0		0		0
Line 3 B-D	42.00	13.50	18	10,206	50%	5,103		0		0		0		0	50%	2,552	50%	2,552		0		0		0		0
Line 3 D-F	28.00	13.50	18	6,804	50%	3,402		0		0		0		0		0		0		0	50%	1,701	50%	1,701		0
Roof Diaphragm	Area (ft²)																									
Roof																										
1--3 A--B	1260.00	1.00	10	12,600	100%	12,600		0	56%	7,056	44%	5,544		0		0		0		0		0		0		0
1--3 B--D	1890.00	1.00	16	30,240	100%	30,240		0		0		0		0	50%	15,120	50%	15,120		0		0		0		0
1--3 D--F	1260.00	1.00	16	20,160	100%	20,160		0		0		0		0		0		0		0	50%	10,080	50%	10,080		0
PV Photo-Voltaic Solar																										
1--3 B--D	990.00	1.00	4	3,960	100%	3,960		0		0		0		0	50%	1,980	50%	1,980		0		0		0		0
1--3 D--F	528.00	1.00	4	2,112	100%	2,112		0		0		0		0		0		0		0	50%	1,056	50%	1,056		0

Snow added to Roof Weight 20% were p,flat > 30 psf

pf Calculated = 17.8 psf

Not required to be included

Roof Diaphragm

Seismic Mass and Seismic Load Per Line

North-South Direction

							Line 1 [Wall]		**Line 1 [Side Facing 3]**		**Line 3 [Side Facing 1]**		**Line 3 [Wall]**	
		[ASD]	**0.7**		**Total**									
		ρ =	**1.3**		**0.7 ρ E (lbs)**		**0.7 ρ E (lbs)**		**0.7 ρ E (lbs)**		**0.7 ρ E (lbs)**		**0.7 ρ E (lbs)**	
		Cs =	**0.273**		**35,261**		**5,400**		**12,106**		**12,355**		**5,400**	
	L (ft)	**Ht. (ft)**	**psf**	**Wt. (lbs)**		**Ws (lbs)**		**Ws (lbs)**		**Ws (lbs)**		**Ws (lbs)**		**Ws (lbs)**
						141,935		**21,735**		**48,731**		**49,734**		**21,735**
Interior Walls														
Interior Partitions Between B & D:														
-Future Use-	60.00	9.75	9	5,265	50%	2,633		0	50%	1,316	50%	1,316		0
Interior Partitions Between D & F:														
Restrooms	18.58	9.75	11	1,993	50%	997		0	29%	293	71%	703		0
	12.92	9.75	11	1,385	50%	693		0	15%	106	85%	586		0
	7.50	9.75	11	804	50%	402		0	9%	37	91%	365		0
	5.42	9.75	11	581	50%	290		0	17%	50	83%	240		0
	3.67	9.75	11	393	50%	197		0	15%	29	85%	167		0
	12.92	9.75	11	1,385	50%	693		0	15%	106	85%	586		0
	7.50	9.75	11	804	50%	402		0	9%	37	91%	365		0
	5.42	9.75	11	581	50%	290		0	17%	50	83%	240		0
	3.67	9.75	11	393	50%	197		0	15%	29	85%	167		0
Kitchen	21.75	9.75	9	1,909	50%	954		0	75%	714	25%	240		0
	28.00	9.75	9	2,457	50%	1,229		0	51%	628	49%	601		0
	21.75	9.75	9	1,909	50%	954		0	75%	714	25%	240		0
	16.00	9.75	9	1,404	50%	702		0	88%	615	12%	87		0
-Future Use-	20.00	9.75	9	1,755	50%	878		0	20%	176	80%	702		0
Exterior Walls														
Line A	45.00	9.50	12	5,130	50%	2,565		0	50%	1,283	50%	1,283		0
Line B Dbl. Wall	45.00	15.00	25	16,875	50%	8,438		0	50%	4,219	50%	4,219		0
Line D - Interior Shear Wall	21.75	10.00	13	2,828	50%	1,414		0	75%	1,058	25%	356		0
Line F	45.00	13.50	18	10,935	50%	5,468		0	50%	2,734	50%	2,734		0
Exterior Walls														
Line 1 A-B	25.00	10.50	18	4,725	100%	4,725	100%	4,725		0		0		0
Line 1 B-D	42.00	13.50	18	10,206	100%	10,206	100%	10,206		0		0		0
Line 1 D-F	28.00	13.50	18	6,804	100%	6,804	100%	6,804		0		0		0
Line 3 A-B	25.00	10.50	18	4,725	100%	4,725		0		0		0	100%	4,725
Line 3 B-D	42.00	13.50	18	10,206	100%	10,206		0		0		0	100%	10,206
Line 3 D-F	28.00	13.50	18	6,804	100%	6,804		0		0		0	100%	6,804
Roof Diaphragm	Area (ft^2)													
Roof														
1--3 A--B	1260.00	1.00	10	12,600	100%	12,600		0	50%	6,300	50%	6,300		0
1--3 B--D	1890.00	1.00	16	30,240	100%	30,240		0	50%	15,120	50%	15,120		0
1--3 D--F	1260.00	1.00	16	20,160	100%	20,160		0	50%	10,080	50%	10,080		0
PV Photo-Voltaic Solar														
1--3 B--D	990.00	1.00	4	3,960	100%	3,960		0	50%	1,980	50%	1,980		0
1--3 D--F	528.00	1.00	4	2,112	100%	2,112		0	50%	1,056	50%	1,056		0

Snow added to Roof Weight 20% were p,flat > 30 psf

pf Calculated = 17.8 psf

Not required to be included

Seismic Forces for Shear Wall per Lines: **0.7 ρ E (lbs)** ρ = **1.30**

Roof Diaphragm

East-West Direction

									Diaph. Shear (lbs)		Diaph. Depth (ft)	Diaph. Shear (plf)
Line A [Wall]	**3,614**	lbs	Line A [Wall]	1,274	lbs							
			Line A[Side Facing B]	2,340	lbs	x	100%	2,340	lbs	44.000	53
			Line B[Side Facing A]	1,964	lbs	x	100%	1,964	lbs	44.000	45
Line B [Wall]	**11,999**	lbs	Line B [Wall]	4,192	lbs							
			Line B[Side Facing D]	5,843	lbs	x	100%	5,843	lbs	44.000	133
			Line D[Side Facing B]	5,843	lbs	x	100%	5,843	lbs	44.000	133
Line D [Wall]	**11,353**	lbs	Line D [Wall]	702	lbs							
			Line D[Side Facing F]	4,808	lbs	x	100%	4,808	lbs	44.000	109
			Line F[Side Facing D]	4,621	lbs	x	100%	4,621	lbs	44.000	105
Line F [Wall]	**7,337**	lbs	Line F [Wall]	2,717	lbs							
Total	**34,304**	**lbs**										

North-South Direction

									Diaph. Shear (lbs)		Diaph. Depth (ft)	Diaph. Shear (plf)
Line 1 [Wall]	**17,506**	lbs	Line 1 [Wall]	5,400	lbs							
			Line 1[Side Facing 3]	12,106	lbs	x	100%	12,106	lbs	95	127
			Line 3[Side Facing 1]	12,355	lbs	x	100%	12,355	lbs	95	130
Line 3 [Wall]	**17,755**	lbs	Line 3 [Wall]	5,400	lbs							
Total	**35,261**	**lbs**										

B.2 Lateral Forces Design

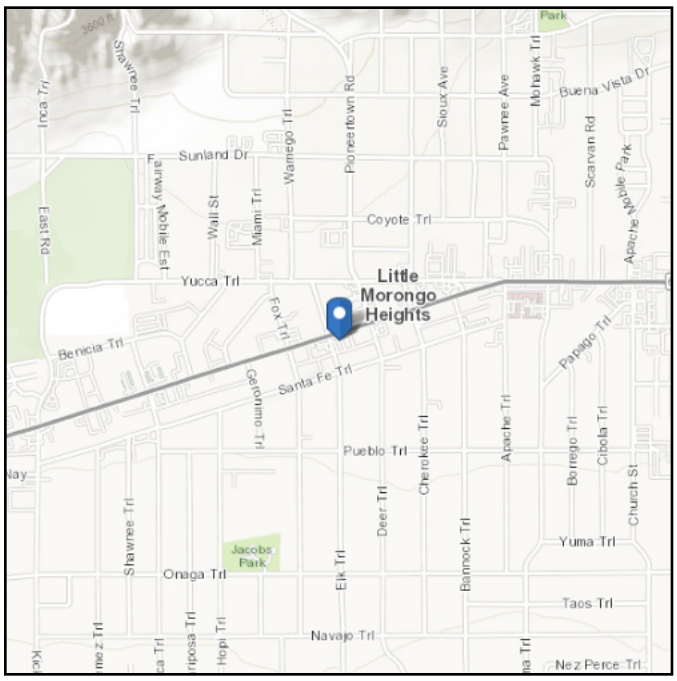
Wind Load Calculation (MWFRS)

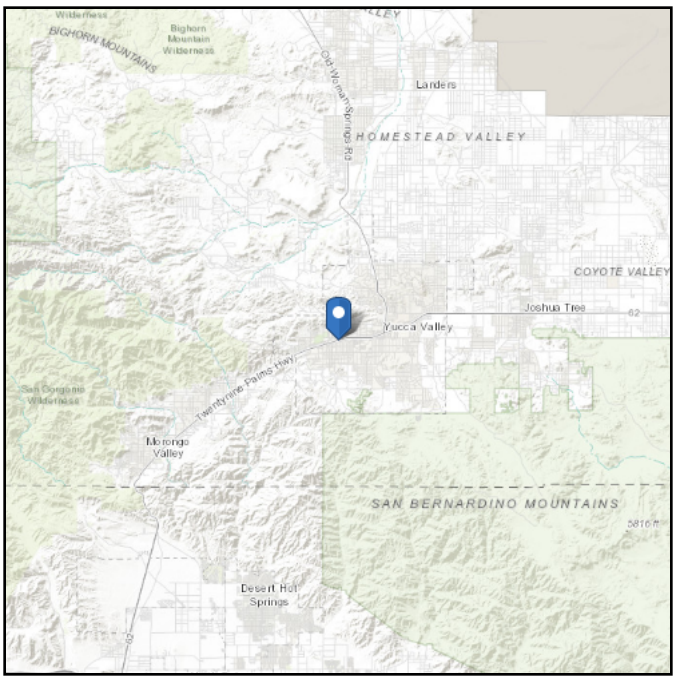
ASCE Hazards Report

WIND

Address:
55795 29 Palms Hwy
Yucca Valley, California
92284

Standard:	ASCE/SEI 7-16	**Latitude:**	34.118916
Risk Category:	II	**Longitude:**	-116.447035
Soil Class:	D - Default (see Section 11.4.3)	**Elevation:**	3326.2407060656838 ft (NAVD 88)





Wind

Results:

Wind Speed	97 Vmph
10-year MRI	67 Vmph
25-year MRI	73 Vmph
50-year MRI	78 Vmph
100-year MRI	83 Vmph
Special	Special Wind Region -- Mountainous terrain, gorges, and special wind regions shown in Fig. 26.5-1 shall be examined for unusual wind conditions. The Authority Having Jurisdiction shall, if necessary, adjust the values given in Fig. 26.5-1 to account for higher local wind speeds. Such adjustment shall be based on meteorological information and an estimate of the basic wind speed obtained in accordance with the provisions in Section 26.5.3.

120 MPH PER San Bernardino County IB-0003 for wind. For Special Wind Zone

Data Source: ASCE/SEI 7-16, Fig. 26.5-1B and Figs. CC.2-1–CC.2-4, and Section 26.5.2

Date Accessed: Thu May 15 2025

SAN BERNARDINO COUNTY	Land Use Services Building and Safety Division **Information Bulletin**	**Number:** IB-0003
		Code References: 2019 CBC ASCE 7-16
Building Official Signature: Jack Leonard, PE, CBO		**Original Effective Date:** November 28, 1972 **Updated:** July 2, 2020
Subject: Rainfall Water Flow Rate, Wind Speed Requirements, and Snow Load Design Values		

1.0 PURPOSE

The purpose of this Information Bulletin is to provide the rainfall water flow rate, wind speed requirements, and snow load design values for the communities listed below and all other areas where the elevation exceeds 3,600 feet above sea level. American Society of Civil Engineers (ASCE) 7-16 requires snow loads above 3,600 feet elevation to be determined by the authority having jurisdiction.

2.0 HISTORY

Original Effective Date: November 28, 1972; Updated: July 2, 2020

3.0 PROCEDURE

Rainfall Water Flow Rate

A. Rainfall Water Flow Rate: Throughout the County of San Bernardino, the recognized rainfall water flow rate is 2.5 inches per hour for a 100-year return.

B. Alternatively, the rainfall water flow rate may be determined based on the site location using the program available at:

https://hdsc.nws.noaa.gov/hdsc/pfds/pfds_map_cont.html?bkmrk=ca

Use the design value of an hour for a 100-year return in inch.

Wind Speed Requirements

Wind loads shall be based on the California Building Code (CBC) Section 1609 with the following design criteria:

A. Minimum Exposure C category shall be used throughout the County of San Bernardino. The ultimate design wind speed, V_{ult} (mph), shall be determined by Figures 1609.3(1), 1609.3(2) and 1609.3(3) of CBC. Design values of 3-second gust wind speeds per Risk Category are as follows:

	V_{ult}	Special Wind Region
Risk Category I:	95 mph	110 mph
Risk Category II:	95 mph	120 mph
Risk Category III & IV:	107 mph	125 mph

B. Alternatively, the ultimate design wind speed may be determined based on the site location using the program available at: http://windspeed.atcouncil.org/

Snow Load Design Values

A. Snow loads shall be considered for the communities listed below in addition to the dead loads of the structure. Unbalanced loads shall be considered.

Wind Load Calculation Wind Pressure Determination

Building is Low-rise	**1-Story Building**	
* Ht. ≤ 60 ft	15'-0" < 60'	
* Ht ≤ Min. L or B	15'-0" < 45'-0"	
Risk Category:	II	
Exposure:	C	
Elevation at site:	3,326	ft.
Basic Wind Speed V:	120	mph — *120 mph Per San Bernardino County for Special Wind Region*

Building Dimensions:

Main Residence

L =	98.00	ft.	approx.
B =	45.00	ft.	approx.
h =	15.00	ft.	mean roof ht

	MWFRS	**C & C**	ASCE 7-16	
G	0.85	0.85	*Gust Factor*	
K_d	0.85	0.85	*Directional Factor, Table 26.6-1 , p266*	
K_{zt}	1.00	1.00	*Topography Factor Ch. 26.8.2 , p268*	Not Applicable
K_e	0.89	0.89	*Elevation Factor (above sea level) Table 26.9-1 , p268*	

z ↓	K_z			*Velocity pressure Coefficients , p268*
		Wind pressure		***@ Heights***
12.000	0.85	0.85	Low Roof (Front)	
15.000	0.85	0.85	Roof Mean Height	

q_z @	psf	psf	$q_z = 0.00256 K_z K_{zt} K_d K_e V^2$	**Use qz**	
12.000	23.6	23.6	Low Roof (Front)	**24**	**psf**
15.000	23.6	23.6	Roof Mean Height	**24**	**psf**

Use 24 psf
For All Heights

Wind Blowing [West --> East] ***Roof Diaphragm***

B = 95.00 ft h = 12.00 ft qz = 24 psf 0.25 :12 Roof Slope θ = 1.2 deg.
L = 45.00 ft qh = 24 psf

Wall Pressure Coeffecient Cp: **[Leeward]** **[Windward]**
L/B = 0.474 *0 < L/B < 1* **Cp = -0.50** **Cp = 0.80**

			factor*	b (ft)	h (ft) d Rise	A (ft²)	q (psf)	G	Cp	p (psf)	P (lbs)	P,Horiz. (lbs)	A,horiz. (ft²)	p,min (psf)	P,min (lbs)
Walls															
Windward Wall															
Line	Between						qz								
1	*A*	*B*	1.00	25.00	6.25	156	24	0.85	0.80	16.32	2,550	2,550	156	16	2,500
1	*B*	*D*	1.00	42.00	6.25	263	24	0.85	0.80	16.32	4,284	4,284	263	16	4,200
1	*D*	*F*	1.00	28.00	6.25	175	24	0.85	0.80	16.32	2,856	2,856	175	16	2,800
Parapets															
Line	Between						qp		GCpn						
1	*A*	*B*				0	24	1	1.50	36.00	0	0	0	16	0
1	*B*	*D*	1.00	42.00	2.00	84	24	1	1.50	36.00	3,024	3,024	84	16	1,344
1	*D*	*F*	1.00	28.00	2.00	56	24	1	1.50	36.00	2,016	2,016	56	16	896
Leeward Wall															
Line	Between						qh								
3	*A*	*B*	1.00	25.00	6.25	156	24	0.85	-0.50	-10.20	-1,594	1,594	156	0	0
3	*B*	*D*	1.00	42.00	5.79	243	24	0.85	-0.50	-10.20	-2,481	2,481	243	0	0
3	*D*	*F*	1.00	28.00	5.79	162	24	0.85	-0.50	-10.20	-1,654	1,654	162	0	0
Parapets															
Line	Between						qp		GCpn						
3	*A*	*B*				0	24	1	-1.00	-24.00	0	0	0	16	0
3	*B*	*D*	1.00	42.00	2.92	123	24	1	-1.00	-24.00	-2,940	2,940	123	16	1,960
3	*D*	*F*	1.00	28.00	2.92	82	24	1	-1.00	-24.00	-1,960	1,960	82	16	1,307

Ult. [1.0 W] **Total** **P = 25,360 lbs** **P min. = 15,008 lbs**
ASD [0.6 W] **P = 15,216 lbs** **P min. = 9,005 lbs**

Wind Blowing [East --> West] ***Roof Diaphragm***

			factor*	b (ft)	h (ft) d Rise	A (ft²)	q (psf)	G	Cp	p (psf)	P (lbs)	P,Horiz. (lbs)	A,horiz. (ft²)	p,min (psf)	P,min (lbs)
Walls															
Windward Wall															
Line	Between						qz								
3	*A*	*B*	1.00	25.00	6.25	156	24	0.85	0.80	16.32	2,550	2,550	156	16	2,500
3	*B*	*D*	1.00	42.00	5.79	243	24	0.85	0.80	16.32	3,970	3,970	243	16	3,892
3	*D*	*F*	1.00	28.00	5.79	162	24	0.85	0.80	16.32	2,647	2,647	162	16	2,595
Parapets															
Line	Between						qp		GCpn						
3	*A*	*B*				0	24	1	1.50	36.00	0	0	0	16	0
3	*B*	*D*	1.00	42.00	2.92	123	24	1	1.50	36.00	4,415	4,415	123	16	1,962
3	*D*	*F*	1.00	28.00	2.92	82	24	1	1.50	36.00	2,943	2,943	82	16	1,308
Leeward Wall															
Line	Between						qh								
1	*A*	*B*	1.00	25.00	6.25	156	24	0.85	-0.50	-10.20	-1,594	1,594	156	0	0
1	*B*	*D*	1.00	42.00	6.25	263	24	0.85	-0.50	-10.20	-2,678	2,678	263	0	0
1	*D*	*F*	1.00	28.00	6.25	175	24	0.85	-0.50	-10.20	-1,785	1,785	175	0	0
Parapets															
Line	Between						qp		GCpn						
1	*A*	*B*				0	24	1	-1.00	-24.00	0	0	0	16	0
1	*B*	*D*	1.00	42.00	2.00	84	24	1	-1.00	-24.00	-2,016	2,016	84	16	1,344
1	*D*	*F*	1.00	28.00	2.00	56	24	1	-1.00	-24.00	-1,344	1,344	56	16	896

Ult. [1.0 W] **Total** **P = 25,942 lbs** **P min. = 14,497 lbs**
ASD [0.6 W] **P = 15,565 lbs** **P min. = 8,698 lbs**

ASD [0.6 W] **P = 15,565 lbs** **P min. = 9,005 lbs**

Wind Blowing [West --> East] ***Roof Diaphragm***

			Line A			Line B			Line D			Line F		
				P (lbs)	$P_{,min}$ (lbs)		P (lbs)	$P_{,min}$ (lbs)		P (lbs)	$P_{,min}$ (lbs)		P (lbs)	$P_{,min}$ (lbs)
Walls														
Windward Wall														
Line	Between													
1	*A*	*B*	50%	1,275	1,250	50%	1,275	1,250		0	0		0	0
1	*B*	*D*		0	0	50%	2,142	2,100	50%	2,142	2,100		0	0
1	*D*	*F*		0	0		0	0	50%	1,428	1,400	50%	1,428	1,400
Line	Between													
1	*A*	*B*	50%	0	0	50%	0	0		0	0		0	0
1	*B*	*D*		0	0	50%	1,512	672	50%	1,512	672		0	0
1	*D*	*F*		0	0		0	0	50%	1,008	448	50%	1,008	448
Leeward Wall														
Line	Between													
3	*A*	*B*	50%	797	0	50%	797	0		0	0		0	0
3	*B*	*D*		0	0	50%	1,241	0	50%	1,241	0		0	0
3	*D*	*F*		0	0		0	0	50%	827	0	50%	827	0
Line	Between													
3	*A*	*B*	50%	0	0	50%	0	0		0	0		0	0
3	*B*	*D*		0	0	50%	1,470	980	50%	1,470	980		0	0
3	*D*	*F*		0	0		0	0	50%	980	653	50%	980	653
Ult. [1.0 W]				**2,072**	**1,250**		**8,437**	**5,002**		**10,608**	**6,254**		**4,243**	**2,502**
ASD [0.6 W]				**1,243**	**750**		**5,062**	**3,001**		**6,365**	**3,752**		**2,546**	**1,501**

Wind Blowing [East --> West] ***Roof Diaphragm***

			Line A			Line B			Line D			Line F		
				P (lbs)	$P_{,min}$ (lbs)		P (lbs)	$P_{,min}$ (lbs)		P (lbs)	$P_{,min}$ (lbs)		P (lbs)	$P_{,min}$ (lbs)
Walls														
Windward Wall														
Line	Between													
3	*A*	*B*	50%	1,275	1,250	50%	1,275	1,250		0	0		0	0
3	*B*	*D*		0	0	50%	1,985	1,946	50%	1,985	1,946		0	0
3	*D*	*F*		0	0		0	0	50%	1,323	1,297	50%	1,323	1,297
Line	Between													
3	*A*	*B*	50%	0	0	50%	0	0		0	0		0	0
3	*B*	*D*		0	0	50%	2,208	981	50%	2,208	981		0	0
3	*D*	*F*		0	0		0	0	50%	1,472	654	50%	1,472	654
Leeward Wall														
Line	Between													
1	*A*	*B*	50%	797	0	50%	797	0		0	0		0	0
1	*B*	*D*		0	0	50%	1,339	0	50%	1,339	0		0	0
1	*D*	*F*		0	0		0	0	50%	893	0	50%	893	0
Line	Between													
1	*A*	*B*	50%	0	0	50%	0	0		0	0		0	0
1	*B*	*D*		0	0	50%	1,008	672	50%	1,008	672		0	0
1	*D*	*F*		0	0		0	0	50%	672	448	50%	672	448
Ult. [1.0 W]				**2,072**	**1,250**		**8,611**	**4,849**		**10,899**	**5,999**		**4,360**	**2,399**
ASD [0.6 W]				**1,243**	**750**		**5,167**	**2,910**		**6,539**	**3,599**		**2,616**	**1,440**

	Line A		Line B		Line D		Line F	
ASD [0.6 W]	**1,243**	**lbs**	**5,167**	**lbs**	**6,539**	**lbs**	**2,616**	**lbs**

Wind Blowing [North --> South] ***Roof Diaphragm***

B = 45.00 ft h = 12.00 ft qz = 24 psf 1 :12 Roof Slope θ = 4.8 deg.
L = 95.00 ft qh = 24 psf

Wall Pressure Coeffecient Cp: **[Leeward]** **[Windward]**
L/B = 2.111 *2 < L/B < 4* **Cp = -0.29** **Cp = 0.80**

			factor*	b (ft)	h (ft) d Rise	A (ft²)	q (psf)	G	Cp	p (psf)	P (lbs)	P,Horiz. (lbs)	A,horiz. (ft²)	p,min (psf)	P,min (lbs)
Walls															
Windward Wall															
Line	Between						qz								
A	*1*	*1/2 Span*	1.00	22.50	6.25	141	24	0.85	0.80	16.32	2,295	2,295	141	16	2,250
A	*1/2 Span*	*3*	1.00	22.50	6.25	141	24	0.85	0.80	16.32	2,295	2,295	141	16	2,250
Parapets															
Line	Between						qp		GCpn						
B	*1*	*1/2 Span*	1.00	22.50	3.20	72	24	1	1.50	36.00	2,592	2,592	72	16	1,152
B	*1/2 Span*	*3*	1.00	22.50	3.20	72	24	1	1.50	36.00	2,592	2,592	72	16	1,152
Leeward Wall															
Line	Between						qh								
F	*1*	*1/2 Span*	1.00	22.50	6.13	138	24	0.85	-0.29	-6.01	-828	828	138	0	0
F	*1/2 Span*	*3*	1.00	22.50	5.75	129	24	0.85	-0.29	-6.01	-777	777	129	0	0
Parapets															
Line	Between						qp		GCpn						
F	*1*	*1/2 Span*	1.00	22.50	3.42	77	24	1	-1.00	-24.00	-1,845	1,845	77	16	1,230
F	*1/2 Span*	*3*	1.00	22.50	3.83	86	24	1	-1.00	-24.00	-2,070	2,070	86	16	1,380

Ult. [1.0 W] **Total** **P = 15,294 lbs** **P min. = 9,414 lbs**
ASD [0.6 W] **P = 9,176 lbs** **P min. = 5,648 lbs**

Wind Blowing [South --> North] ***Roof Diaphragm***

			factor*	b (ft)	h (ft) d Rise	A (ft²)	q (psf)	G	Cp	p (psf)	P (lbs)	P,Horiz. (lbs)	A,horiz. (ft²)	p,min (psf)	P,min (lbs)
Walls															
Windward Wall															
Line	Between						qz								
F	*1*	*1/2 Span*	1.00	22.50	6.13	138	24	0.85	0.80	16.32	2,249	2,249	138	16	2,205
F	*1/2 Span*	*3*	1.00	22.50	5.75	129	24	0.85	0.80	16.32	2,111	2,111	129	16	2,070
Parapets															
Line	Between						qp		GCpn						
F	*1*	*1/2 Span*	1.00	22.50	3.42	77	24	1	1.50	36.00	2,768	2,768	77	16	1,230
F	*1/2 Span*	*3*	1.00	22.50	3.83	86	24	1	1.50	36.00	3,105	3,105	86	16	1,380
Leeward Wall															
Line	Between						qh								
A	*1*	*1/2 Span*	1.00	22.50	6.25	141	24	0.85	-0.29	-6.01	-845	845	141	0	0
A	*1/2 Span*	*3*	1.00	22.50	6.25	141	24	0.85	-0.29	-6.01	-845	845	141	0	0
Parapets															
Line	Between						qp		GCpn						
B	*1*	*1/2 Span*	1.00	22.50	3.20	72	24	1	-1.00	-24.00	-1,728	1,728	72	16	1,152
B	*1/2 Span*	*3*	1.00	22.50	3.20	72	24	1	-1.00	-24.00	-1,728	1,728	72	16	1,152

Ult. [1.0 W] **Total** **P = 15,379 lbs** **P min. = 9,189 lbs**
ASD [0.6 W] **P = 9,227 lbs** **P min. = 5,514 lbs**

ASD [0.6 W] **P = 9,227 lbs** **P min. = 5,648 lbs**

Wind Blowing [North --> South] ***Roof Diaphragm***

				Line 1			Line 3	
				P (lbs)	P,min (lbs)		P (lbs)	P,min (lbs)
Walls								
Windward Wall								
Line	Between							
A	*1*	*1/2 Span*	75%	1,721	1,688	25%	574	563
A	*1/2 Span*	*3*	25%	574	563	75%	1,721	1,688
Line	Between							
B	*1*	*1/2 Span*	75%	1,944	864	25%	648	288
B	*1/2 Span*	*3*	25%	648	288	75%	1,944	864
Leeward Wall								
Line	Between							
F	*1*	*1/2 Span*	75%	621	0	25%	207	0
F	*1/2 Span*	*3*	25%	194	0	75%	583	0
Line	Between							
F	*1*	*1/2 Span*	75%	1,384	923	25%	461	308
F	*1/2 Span*	*3*	25%	517	345	75%	1,552	1,035
Ult. [1.0 W]				**7,604**	**4,670**		**7,691**	**4,745**
ASD [0.6 W]				**4,562**	**2,802**		**4,614**	**2,847**

Wind Blowing [South --> North] ***Roof Diaphragm***

				Line 1			Line 3	
				P (lbs)	P,min (lbs)		P (lbs)	P,min (lbs)
Walls								
Windward Wall								
Line	Between							
F	*1*	*1/2 Span*	75%	1,687	1,654	25%	562	551
F	*1/2 Span*	*3*	25%	528	518	75%	1,584	1,553
Line	Between							
F	*1*	*1/2 Span*	75%	2,076	923	25%	692	308
F	*1/2 Span*	*3*	25%	776	345	75%	2,329	1,035
Leeward Wall								
Line	Between							
A	*1*	*1/2 Span*	75%	634	0	25%	211	0
A	*1/2 Span*	*3*	25%	211	0	75%	634	0
Line	Between							
B	*1*	*1/2 Span*	75%	1,296	864	25%	432	288
B	*1/2 Span*	*3*	25%	432	288	75%	1,296	864
Ult. [1.0 W]				**7,640**	**4,591**		**7,739**	**4,598**
ASD [0.6 W]				**4,584**	**2,755**		**4,644**	**2,759**

	Line 1		Line 3	
ASD [0.6 W]	**4,584**	**lbs**	**4,644**	**lbs**

Roof Diaphragm

Wind [MWFRS] Forces for Shear Wall per Lines:	ASD 0.6 W (lbs)	
West-East Direction		
Line A	1,243	lbs
Line B	5,167	lbs
Line D	6,539	lbs
Line F	2,616	lbs
Total	15,565	lbs
North - South Direction		
Line 1	4,584	lbs
Line 3	4,644	lbs
Total	9,227	lbs

B.3 Lateral Forces Design

Wind Load Calculation (C&C) Components and Cladding

Wind : Components & Cladding **Walls**

Walls

Wind Velocity Pressure
Based on: 120 mph
q_h = 24 psf

Roof Pitch
1 :12 Roof Slope
θ = 4.8
Reduced by 10% for θ ≤ 10

Building Dimensions:

Length 95.00 ft — *Larger dim. of the L-shaped Hipped Roof*
Width 45.00 ft — *Smaller dim. of the L-shaped Hipped Roof*
Height 15.00 ft

Zone Parameter

a = max { min { 10 % Least Horizontal Dim. , 0.4 h } , 4% Least Horizontal Dim. , 3 ft }

min { 4.50 ft , 6.00 ft } , 1.8 ft

a = 4.50 ft

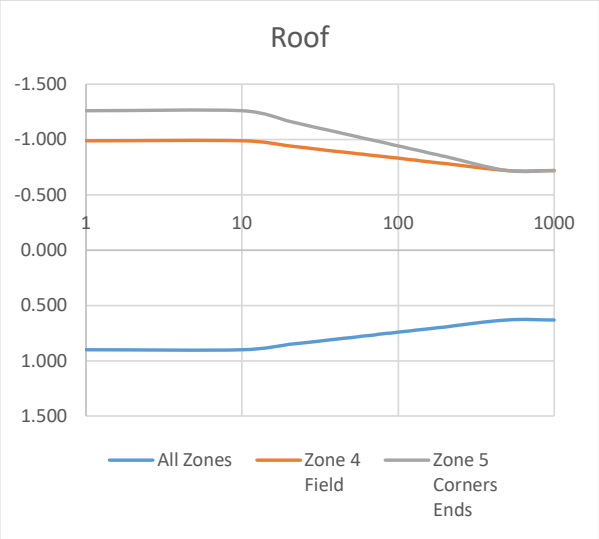
a = 5.00 ft

Walls [Fig. 30.3-1] **ASCE 7-16 p. 335**

External Pressure Coefficient (GC_p)

Positive with and without Overhang
↓ Negative without Overhang

EWA	All Zones	Zone 4 Field	Zone 5 Corners Ends
1	0.900	-0.990	-1.260
10	0.900	-0.990	-1.260
20	0.852	-0.942	-1.164
30	0.824	-0.914	-1.108
40	0.804	-0.894	-1.069
50	0.789	-0.879	-1.038
60	0.776	-0.866	-1.013
70	0.766	-0.856	-0.991
80	0.756	-0.846	-0.973
90	0.748	-0.838	-0.957
100	0.741	-0.831	-0.942
120	0.728	-0.818	-0.917
140	0.718	-0.808	-0.896
160	0.709	-0.799	-0.877
180	0.700	-0.790	-0.861
200	0.693	-0.783	-0.846
500	0.630	-0.720	-0.720
1000	0.630	-0.720	-0.720



External Pressures psf

$p = q_h (GC_p)$

Positive with and without Overhang
↓ Negative without Overhang

All Zones	Zone 4 Field	Zone 5 Corners Ends
21.6	-23.8	-30.2
21.6	-23.8	-30.2
20.5	-22.6	-27.9
19.8	-21.9	-26.6
19.3	-21.5	-25.6
18.9	-21.1	-24.9
18.6	-20.8	-24.3
18.4	-20.5	-23.8
18.2	-20.3	-23.4
18.0	-20.1	-23.0
17.8	-19.9	-22.6
17.5	-19.6	-22.0
17.2	-19.4	-21.5
17.0	-19.2	-21.1
16.8	-19.0	-20.7
16.6	-18.8	-20.3
15.1	-17.3	-17.3
15.1	-17.3	-17.3

Note:

Add internal pressure per condition q_h GC_{pi} to Values Above

Internal Pressures psf

Enclosed

GCpi = ± 0.18
pi = ± 4.32 psf

Partially Enclosed

GCpi = ± 0.55
pi = ± 13.2 psf

Wind : Components & Cladding **Roof** **θ ≤ 7** **Gable [Fig. 30.3-2A]**

Flat Roof

Wind Velocity Pressure
Based on V = 120 mph
q_h = 24 psf

Roof Pitch
1 :12 Roof Slope
θ = 4.8

Building Dimensions:
Length 95.00 ft Width 45.00 ft Height 12.00 ft

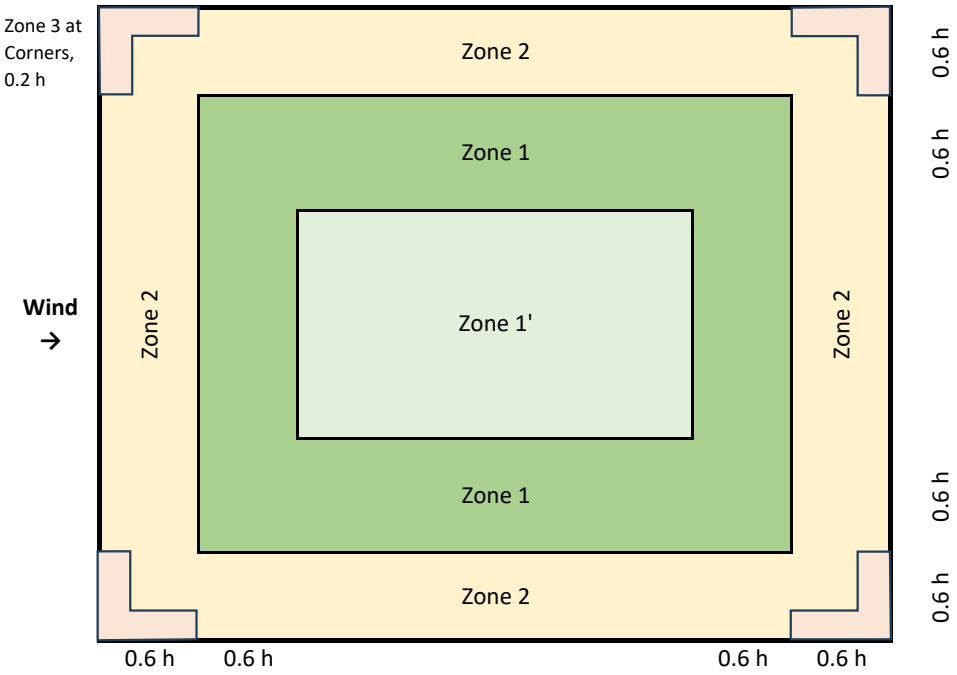
Zone Parameter 0.6 h = 7.20 ft 0.2 h = 2.40 ft

θ ≤ 7

External Pressure Coefficient (GC_p) **Gable [Fig. 30.3-2A]** **ASCE 7-16 p. 336**

Positive with and without Overhang ↓ Negative without Overhang | Negative With Overhang

EWA	All Zones	Zone 1'	Zone 1	Zone 2	Zone 3	Zone 1'	Zone 1	Zone 2	Zone 3
1	0.300	-0.900	-1.700	-2.300	-3.200	-1.700	-1.700	-2.300	-3.200
10	0.300	-0.900	-1.700	-2.300	-3.200	-1.700	-1.700	-2.300	-3.200
20	0.270	-0.900	-1.576	-2.141	-2.881	-1.670	-1.670	-2.087	-2.828
30	0.252	-0.900	-1.503	-2.047	-2.694	-1.652	-1.652	-1.963	-2.610
40	0.240	-0.900	-1.452	-1.981	-2.562	-1.640	-1.640	-1.875	-2.456
50	0.230	-0.900	-1.412	-1.930	-2.459	-1.630	-1.630	-1.806	-2.336
60	0.222	-0.900	-1.379	-1.888	-2.376	-1.622	-1.622	-1.750	-2.238
70	0.215	-0.900	-1.352	-1.852	-2.305	-1.615	-1.615	-1.703	-2.155
80	0.210	-0.900	-1.328	-1.822	-2.243	-1.610	-1.610	-1.662	-2.084
90	0.205	-0.900	-1.307	-1.795	-2.189	-1.605	-1.605	-1.626	-2.021
100	0.200	-0.900	-1.288	-1.770	-2.141	-1.600	-1.600	-1.594	-1.964
150	0.200	-0.812	-1.215	-1.677	-1.954	-1.449	-1.449	-1.469	-1.746
200	0.200	-0.749	-1.164	-1.611	-1.822	-1.342	-1.342	-1.381	-1.592
300	0.200	-0.661	-1.091	-1.518	-1.635	-1.190	-1.190	-1.257	-1.374
400	0.200	-0.599	-1.040	-1.451	-1.503	-1.083	-1.083	-1.168	-1.220
500	0.200	-0.551	-1.000	-1.400	-1.400	-1.000	-1.000	-1.100	-1.100
1000	0.200	-0.400	-1.000	-1.400	-1.400	-1.000	-1.000	-1.100	-1.100



Wind : Components & Cladding **Roof** **θ ≤ 7** **Gable [Fig. 30.3-2A]**

Flat Roof

Internal Pressures psf

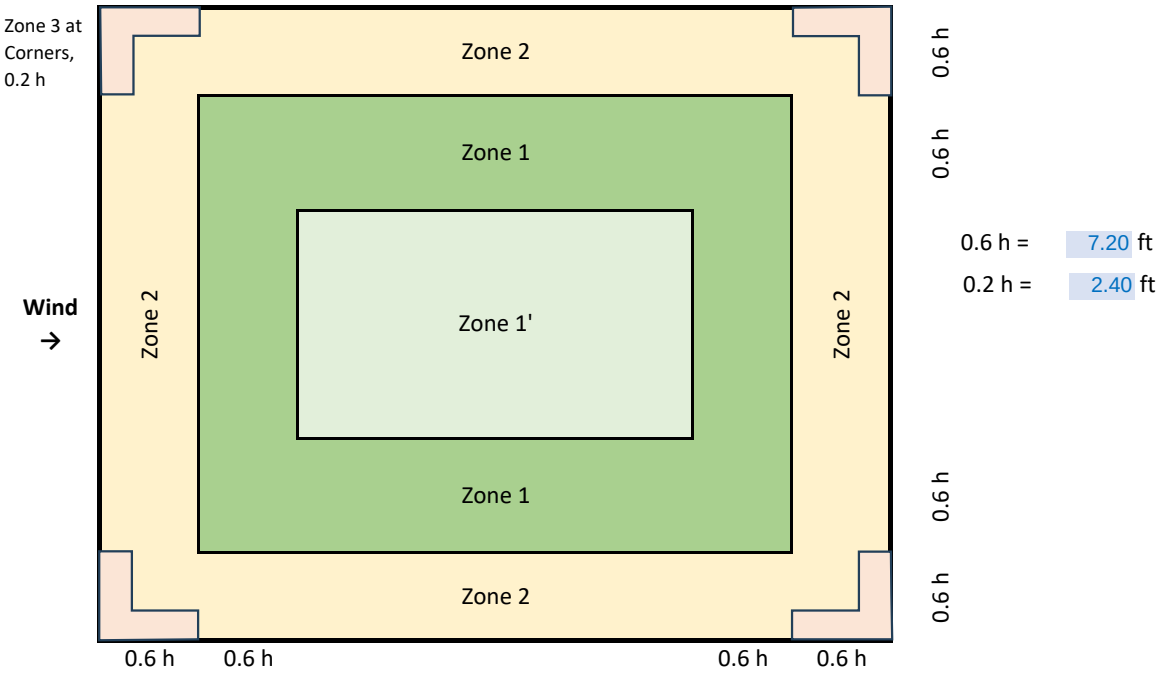
Enclosed				**Partially Enclosed**			
GCpi =	±	0.18		GC_{pi} =	±	0.55	
pi =	±	4.32	psf	p_i =	±	13.2	psf

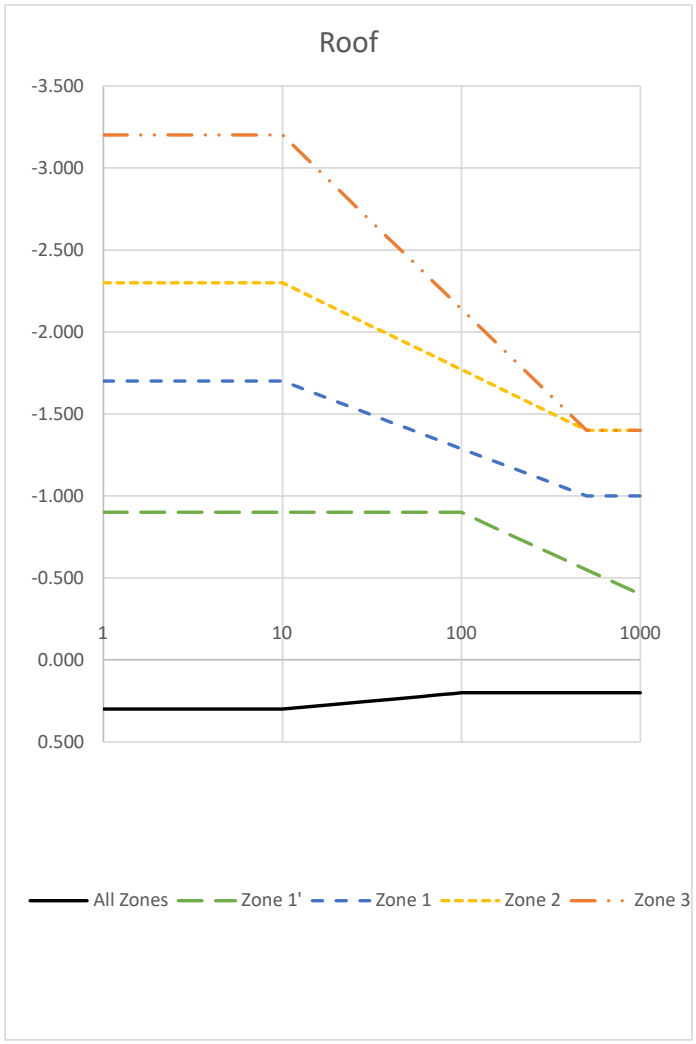
External Pressures psf ***Note: Add internal pressure per condition q_h GC_{pi} to Values Below***

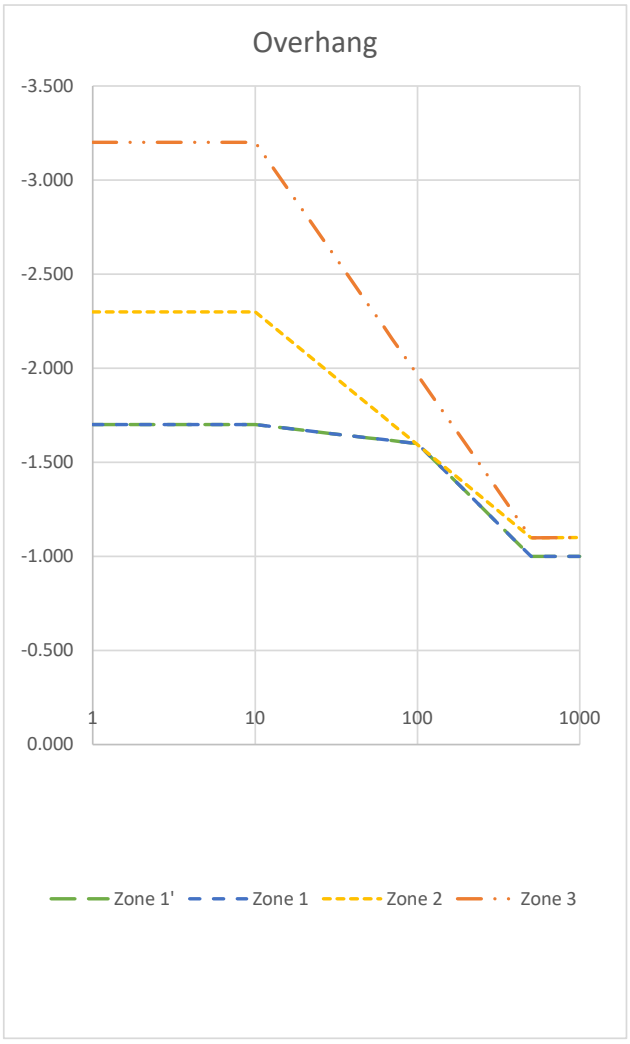
$p = q_h (GC_p)$

Positive with and without Overhang ↓ Negative without Overhang | Negative With Overhang

EWA	All Zones	Zone 1'	Zone 1	Zone 2	Zone 3	Zone 1'	Zone 1	Zone 2	Zone 3
1	7.2	-21.6	-40.8	-55.2	-76.8	-40.8	-40.8	-55.2	-76.8
10	7.2	-21.6	-40.8	-55.2	-76.8	-40.8	-40.8	-55.2	-76.8
20	6.5	-21.6	-37.8	-51.4	-69.1	-40.1	-40.1	-50.1	-67.9
30	6.1	-21.6	-36.1	-49.1	-64.7	-39.7	-39.7	-47.1	-62.6
40	5.8	-21.6	-34.8	-47.5	-61.5	-39.4	-39.4	-45.0	-58.9
50	5.5	-21.6	-33.9	-46.3	-59.0	-39.1	-39.1	-43.4	-56.1
60	5.3	-21.6	-33.1	-45.3	-57.0	-38.9	-38.9	-42.0	-53.7
70	5.2	-21.6	-32.4	-44.5	-55.3	-38.8	-38.8	-40.9	-51.7
80	5.0	-21.6	-31.9	-43.7	-53.8	-38.6	-38.6	-39.9	-50.0
90	4.9	-21.6	-31.4	-43.1	-52.5	-38.5	-38.5	-39.0	-48.5
100	4.8	-21.6	-30.9	-42.5	-51.4	-38.4	-38.4	-38.2	-47.1
150	4.8	-19.5	-29.2	-40.2	-46.9	-34.8	-34.8	-35.3	-41.9
200	4.8	-18.0	-27.9	-38.7	-43.7	-32.2	-32.2	-33.1	-38.2
300	4.8	-15.9	-26.2	-36.4	-39.2	-28.6	-28.6	-30.2	-33.0
400	4.8	-14.4	-25.0	-34.8	-36.1	-26.0	-26.0	-28.0	-29.3
500	4.8	-13.2	-24.0	-33.6	-33.6	-24.0	-24.0	-26.4	-26.4
1000	4.8	-9.6	-24.0	-33.6	-33.6	-24.0	-24.0	-26.4	-26.4




Roof
-3.500
-3.000
-2.500
-2.000
-1.500
-1.000
-0.500
0.000
0.500
1
10
100
1000
All Zones
Zone 1'
Zone 1
Zone 2
Zone 3


Overhang
-3.500
-3.000
-2.500
-2.000
-1.500
-1.000
-0.500
0.000
1
10
100
1000
Zone 1'
Zone 1
Zone 2
Zone 3

Wind : Components & Cladding **Roof** **θ ≤ 7** **Gable [Fig. 30.3-2A]**

Wind Velocity Pressure
Based on V = 120 mph
q_h = 24 psf

Roof Pitch
1 :12 Roof Slope
θ = 4.8

Building Dimensions:

Length 70.00 ft Width 45.00 ft Height 12.00 ft

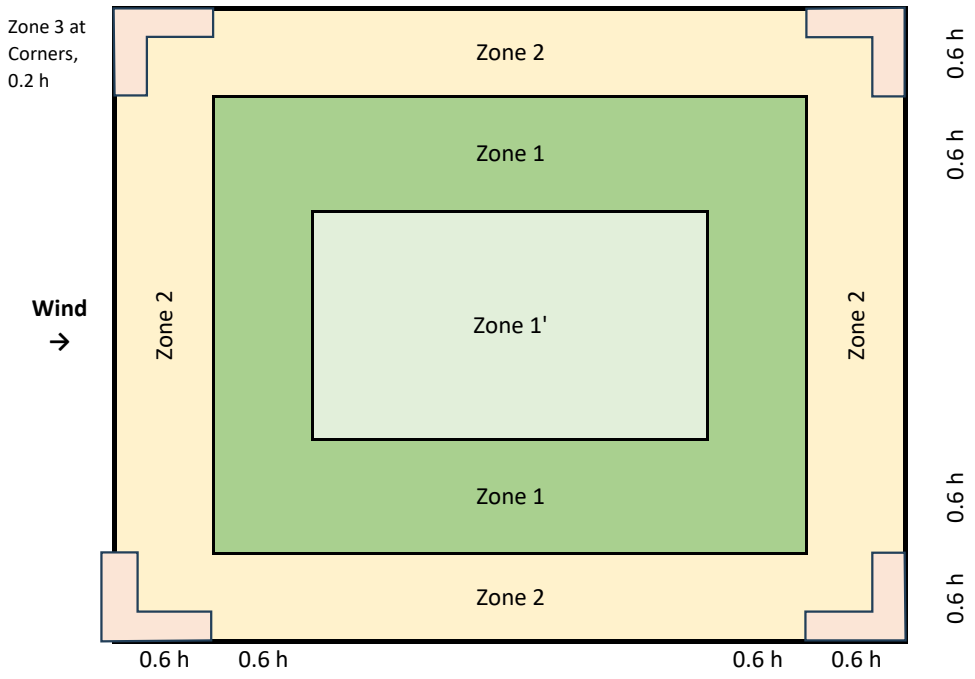
Zone Parameter 0.6 h = 7.2 ft 0.2 h = 2.4 ft

θ ≤ 7

External Pressure Coefficient (GC_p) **Gable [Fig. 30.3-2A]** **ASCE 7-16 p. 336**

Positive with and without Overhang
Negative without Overhang
Negative With Overhang

EWA	All Zones	Zone 1'	Zone 1	Zone 2	Zone 3	Zone 1'	Zone 1	Zone 2	Zone 3	
176	0.200	-0.777	-1.187	-1.640	-1.880	-1.389	-1.389	-1.420	-1.660	Roof Joist Design (Spanning 2--3)
88	0.205	-0.900	-1.311	-1.799	-2.198	-1.605	-1.605	-1.632	-2.032	Roof Joist (Spanning 2--3) Rxn.
208	0.200	-0.741	-1.157	-1.601	-1.803	-1.326	-1.326	-1.369	-1.570	Entrance Roof Joist Design
104	0.200	-0.891	-1.281	-1.761	-2.122	-1.585	-1.585	-1.581	-1.942	Entrance Roof Joist Reaction
115	0.200	-0.870	-1.263	-1.738	-2.076	-1.548	-1.548	-1.551	-1.889	Cont. Beam Span E--F
190	0.200	-0.761	-1.173	-1.623	-1.845	-1.361	-1.361	-1.397	-1.619	Cont. Beam Span D--E
230	0.200	-0.719	-1.139	-1.579	-1.757	-1.289	-1.289	-1.338	-1.517	Cont. Beam Span C--D
235	0.200	-0.714	-1.135	-1.574	-1.747	-1.281	-1.281	-1.332	-1.505	Cont. Beam Span B--C



Wind : Components & Cladding **Roof** **θ ≤ 7** **Gable [Fig. 30.3-2A]**

Internal Pressures psf

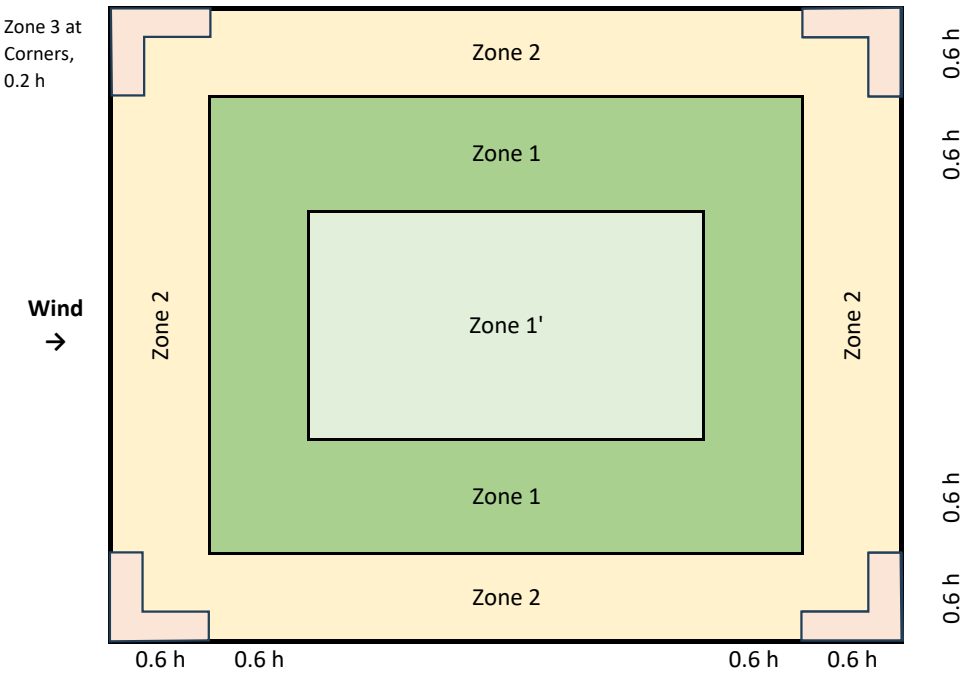
Enclosed				**Partially Enclosed**			
GCpi =	±	0.18		GC_{pi} =	±	0.55	
pi =	±	4.32	psf	p_i =	±	13.2	psf

External Pressures psf ***Note: Add internal pressure per condition q_h GC_{pi} to Values Below***

$p = q_h$ ***(GC_p)***

pi (used) = 4.32 psf

	Positive with and without Overhang w/ internal	Negative without Overhang w/ internal				Negative With Overhang w/ internal				
EWA	All Zones	Zone 1'	Zone 1	Zone 2	Zone 3	Zone 1'	Zone 1	Zone 2	Zone 3	
176	9.1	-23.0	-32.8	-43.7	-49.4	-37.6	-37.6	-38.4	-44.1	Roof Joist Design (Spanning 2--3)
88	9.3	-25.9	-35.8	-47.5	-57.1	-42.9	-42.9	-43.5	-53.1	Roof Joist (Spanning 2--3) Rxn.
208	9.1	-22.1	-32.1	-42.8	-47.6	-36.2	-36.2	-37.2	-42.0	Entrance Roof Joist Design
104	9.1	-25.7	-35.1	-46.6	-55.2	-42.4	-42.4	-42.3	-50.9	Entrance Roof Joist Reaction
115	9.1	-25.2	-34.6	-46.0	-54.1	-41.5	-41.5	-41.5	-49.7	Cont. Beam Span E--F
190	9.1	-22.6	-32.5	-43.3	-48.6	-37.0	-37.0	-37.8	-43.2	Cont. Beam Span D--E
230	9.1	-21.6	-31.7	-42.2	-46.5	-35.3	-35.3	-36.4	-40.7	Cont. Beam Span C--D
235	9.1	-21.5	-31.6	-42.1	-46.3	-35.1	-35.1	-36.3	-40.4	Cont. Beam Span B--C



Cont. Beam Span B--C & C-D **EWA** **230 ft2** **Conservatively**

Roof Joist Sapnning 1--2

22 ft

Σ	z2			
	-42.2			psf
22	22			ft
-928	-928	0	0	plf
-10212	-10212	0	0	plf-ft
Rxn 1	-464	Rxn 2	**-464**	plf Rxn

Σ	z2	z1		
	-42.2	-31.7		psf
22	7.2	14.8		ft
-773	-304	-469	0	plf
-7944	-1094	-6850	0	plf-ft
Rxn 1	-412	Rxn 2	**-361**	plf Rxn

Σ	z2	z1	z1	
	-42.2	-31.7	-21.6	psf
22	7.2	7.2	7.6	ft
-696	-304	-228	-164	plf
-6547	-1094	-2465	-2988	plf-ft
Rxn 1	-399	Rxn 2	**-298**	plf Rxn

Roof Joist Sapnning 2--3

23 ft

Σ	z2			
	-42.2			psf
23	23			ft
-971	-971	0	0	plf
-11162	-11162	0	0	plf-ft
Rxn 1	-485	Rxn 2	**-485**	plf Rxn

Σ	z2	z1		
	-42.2	-31.7		psf
23	7.2	15.8		ft
-805	-304	-501	0	plf
-8657	-1094	-7563	0	plf-ft
Rxn 1	-428	Rxn 2	**-376**	plf Rxn

Σ	z2	z1	z1	
	-42.2	-31.7	-21.6	psf
23	7.2	7.2	8.6	ft
-718	-304	-228	-186	plf
-7033	-1094	-2465	-3474	plf-ft
Rxn 1	-412	Rxn 2	**-306**	plf Rxn

Cont. Beam Span D--E **EWA** **190 ft2** **Conservatively**

Roof Joist Sapnning 1--2

22 ft

Σ	z2	z1		
	-43.3	-32.5		psf
22	7.2	14.8		ft
-793	-312	-481	0	plf
-8145	-1122	-7023	0	plf-ft
Rxn 1	-423	Rxn 2	**-370**	plf Rxn

Σ	z2	z1	z1	
	-43.3	-32.5	-22.6	psf
22	7.2	7.2	7.6	ft
-718	-312	-234	-172	plf
-6776	-1122	-2527	-3126	plf-ft
Rxn 1	-410	Rxn 2	**-308**	plf Rxn

Roof Joist Sapnning 2--3

23 ft

Σ	z2	z1		
	-43.3	-32.5		psf
23	7.2	15.8		ft
-825	-312	-514	0	plf
-8876	-1122	-7754	0	plf-ft
Rxn 1	-439	Rxn 2	**-386**	plf Rxn

Σ	z2	z1	z1	
	-43.3	-32.5	-22.6	psf
23	7.2	7.2	8.6	ft
-740	-312	-234	-194	plf
-7284	-1122	-2527	-3635	plf-ft
Rxn 1	-423	Rxn 2	**-317**	plf Rxn

Cont. Beam Span E-F **EWA** **115 ft2** **Conservatively**

Roof Joist Sapnning 1--2

22 ft

Σ	z2			
	-46.0			psf
22	22			ft
-1012	-1012	0	0	plf
-11132	-11132	0	0	plf-ft
Rxn 1	-506	Rxn 2	**-506**	plf Rxn

Σ	z2	z1		
	-46.0	-34.6		psf
22	7.2	14.8		ft
-843	-331	-512	0	plf
-8669	-1192	-7476	0	plf-ft
Rxn 1	-449	Rxn 2	**-394**	plf Rxn

Roof Joist Sapnning 2--3

23 ft

Σ	z2			
	-46.0			psf
23	23			ft
-1058	-1058	0	0	plf
-12167	-12167	0	0	plf-ft
Rxn 1	-529	Rxn 2	**-529**	plf Rxn

Σ	z2	z1		
	-46.0	-34.6		psf
23	7.2	15.8		ft
-878	-331	-547	0	plf
-9447	-1192	-8255	0	plf-ft
Rxn 1	-467	Rxn 2	**-411**	plf Rxn

C. Shear Wall Design

Forces Per Line Summary

		1.0 E	0.7 ρ E ρ = 1	0.7 ρ E ρ = 1.3		1.0 W	0.6 W
East-West Direction							
	Line A	3,972	2,780	**3,614**	**Line A**	2,072	**1,243**
	Line B	13,186	9,230	**11,999**	**Line B**	8,611	**5,167**
	Line D	12,476	8,733	**11,353**	**Line D**	10,899	**6,539**
	Line F	8,063	5,644	**7,337**	**Line F**	4,360	**2,616**
		37,697	26,388	**34,304**		25,942	**15,565**
North-South Direction							
	Line 1	19,237	13,466	**17,506**	**Line 1**	7,640	**4,584**
	Line 3	19,511	13,658	**17,755**	**Line 3**	7,739	**4,644**
		38,748	27,124	**35,261**		15,379	**9,227**

SHEAR WALL SCHEDULE

SHEAR WALL UNIT SHEAR CAPACITY AT ALLOWABLE STRESS DESIGN ASD LOAD COMBINATIONS. [AWC – SDPWS 2021] (A)		SHEAR WALL NOMINAL UNIT SHEAR CAPACITY	SHEAR WALL SYMBOL Line F	SHEATHING MATERIAL: SEE SCHEDULE BELOW *(SEE REMARKS AND REFER TO STRUCTURAL GENERAL NOTES SGN-1)* Lines 1 & 3	NAILING PATTERN PANEL EDGES AND AT BLOCKING			MIN. FRAMING –OR– BLOCKING SIZE AT ADJOINING PANELS EDGES (12)(13)	MUDSILL TO FOUNDATION & ANCHORAGE			SOLE PLATE / TOP PLATE CONNECTIONS TO RIM (A)		MIN. RIM		RIM TO TOP PLATE CONNECTION W/ SHEAR CLIPS		REMARKS
ASD SEISMIC (LBS/FT) [NOMINAL / 2.8]	ASD WIND (LBS/FT) [NOMINAL / 2]	NOMINAL FOR REFERENCE ONLY (LBS/FT)			EDGE NAILING E.N. (2A.2C,2D) [COMMON]	SPACING	MIN. NAIL BEARING LENGTH IN FRAMING MEMBER OR BLOCKING		MUDSILL	ANCHOR BOLT SIZE (5)	MAXIMUM SPACING	OPTION 1 0.148" x 3 ¼" [16d SINKER] AT 2x SOLES ONLY, SEE (2B)(17)	OPTION 2 SDWS 0.22 x 6" TIMBER SCREW [INTERIOR GRADE] SEE (2B)(17)	SAWN LUMBER (14)	LSL (14)	LTP4 –OR– A35 (15)	RBC (16)	
SHEATHING ON ONE SIDE OF WALL FRAMING																		
260	365	730 (18)	W	3/8" THICK, 3–PLY MIN. PLYWOOD CDX APA RATED SHEATHING	8d x 2½"	6" O.C.	1 ⅜"	2x	2x P.T.	ϕ 5/8"	4'–0" O.C.	6" O.C.	16" O.C.	2x	1 ½"	20" O.C.	12" O.C.	AT PARAPETS & EXTERIOR WALLS WHERE; NO SHEAR–WALL LABEL CALLED OUT ON PLAN(18)
310	435	870	A	15/32" [1/2"] THICK, 4–PLY MIN. PLYWOOD APA RATED SHEATHING	10d x 3"	6" O.C.	1 ½"	2x	3x P.T. 2x P.T. (6)	ϕ 5/8" ϕ 5/8"	4'–0" O.C. 3'–4" O.C. (6)	6" O.C.	16" O.C.	2x	1 ½"	20" O.C.	12" O.C.	
340	475	950	6		10d x 3"	6" O.C.	1 ½"	2x	3x P.T.	ϕ 5/8"	4'–0" O.C.	6" O.C.	16" O.C.	2x	1 ½"	20" O.C.	12" O.C.	
510	715	1,430	4	15/32" [1/2"] THICK, 4–PLY MIN. PLYWOOD APA RATED STRUC–I	10d x 3"	4" O.C.	1 ½"	3x , (2)2x	3x P.T.	ϕ 5/8"	3'–4" O.C.	4" O.C.	12" O.C.	2x	1 ½"	12" O.C.	10" O.C.	
665	930	1,860	3		10d x 3"	3" O.C.	1 ½"	3x , (2)2x	3x P.T.	ϕ 5/8"	2'–8" O.C.	3" O.C.	8" O.C.	2x	1 ½"	10" O.C.	6" O.C.	
870	1,215	2,435	2		10d x 3"	2" O.C.	1 ½"	3x , (2)2x	3x P.T.	ϕ 5/8"	2'–0" O.C.	2 ROWS 4" O.C.	6" O.C.	4x	3 ½"	8" O.C.	6" O.C.	

TABLE 2—ALLOWABLE ASD IN-PLANE SHEAR FOR STANDARD APPLICATION STRONG-WALL HIGH STRENGTH WOOD SHEARWALL (WSWH) ON CONCRETE FOUNDATION

Strong-Wall High Strength Wood Shearwall Model	Panel Evaluation Height, H_e (in.)	Allow. Vertical Load, P (lb.)	2,500 psi Concrete						3,000 psi Concrete					
			Seismic			Wind			Seismic			Wind		
			Allow. ASD Shear Load, V (lb.)	Drift at Allow. Shear, Δ (in.)	Anchor Tension at Allow. Shear, T (lb.)	Allow. ASD Shear Load, V (lb.)	Drift at Allow. Shear, Δ (in.)	Anchor Tension at Allow. Shear, T (lb.)	Allow. ASD Shear Load, V (lb.)	Drift at Allow. Shear, Δ (in.)	Anchor Tension at Allow. Shear, T (lb.)	Allow. ASD Shear Load, V (lb.)	Drift at Allow. Shear, Δ (in.)	Anchor Tension at Allow. Shear, T (lb.)
WSWH12x7	78	1,000	1,300	0.32	13,295	1,670	0.43	17,075	1,300	0.32	13,295	1,670	0.43	17,075
		4,000	1,300	0.32	13,295	1,670	0.43	17,075	1,300	0.32	13,295	1,670	0.43	17,075
		7,500	1,300	0.32	13,295	1,670	0.43	17,075	1,300	0.32	13,295	1,670	0.43	17,075
WSWH18x7	78	1,000	3,795	0.32	23,680	4,470	0.39	27,890	3,795	0.32	23,680	4,470	0.39	27,890
		4,000	3,795	0.32	23,680	4,365	0.38	27,245	3,795	0.32	23,680	4,470	0.39	27,890
		7,500	3,795	0.32	23,680	4,050	0.36	25,285	3,795	0.32	23,680	4,470	0.39	27,890
WSWH24x7	78	1,000	7,450	0.30	33,210	7,795	0.34	34,755	7,450	0.30	33,210	7,795	0.34	34,755
		4,000	7,450	0.30	33,210	7,565	0.33	33,715	7,450	0.30	33,210	7,795	0.34	34,755
		7,500	7,115	0.28	31,715	7,115	0.31	31,715	7,450	0.30	33,210	7,795	0.34	34,755
WSWH12x8	93¼	1,000	1,030	0.40	12,580	1,325	0.53	16,195	1,030	0.40	12,580	1,325	0.53	16,195
		4,000	1,030	0.40	12,580	1,325	0.53	16,195	1,030	0.40	12,580	1,325	0.53	16,195
		7,500	1,030	0.40	12,580	1,325	0.53	16,195	1,030	0.40	12,580	1,325	0.53	16,195
WSWH18x8	93¼	1,000	3,060	0.39	22,835	3,880	0.52	28,925	3,060	0.39	22,835	3,955	0.53	29,490
		4,000	3,060	0.39	22,835	3,650	0.49	27,245	3,060	0.39	22,835	3,955	0.53	29,490
		7,500	3,060	0.39	22,835	3,390	0.46	25,285	3,060	0.39	22,835	3,955	0.53	29,490
WSWH24x8	93¼	1,000	6,240	0.37	33,240	6,650	0.43	35,430	6,240	0.37	33,240	6,910	0.45	36,815
		4,000	6,240	0.37	33,240	6,330	0.41	33,715	6,240	0.37	33,240	6,910	0.45	36,815
		7,500	5,950	0.35	31,715	5,950	0.38	31,715	6,240	0.37	33,240	6,910	0.45	36,815
WSWH12x9	105¼	1,000	850	0.45	11,750	1,095	0.60	15,145	850	0.45	11,750	1,095	0.60	15,145
		4,000	850	0.45	11,750	1,095	0.60	15,145	850	0.45	11,750	1,095	0.60	15,145
		7,500	850	0.45	11,750	1,095	0.60	15,145	850	0.45	11,750	1,095	0.60	15,145
WSWH18x9	105¼	1,000	2,575	0.45	21,680	3,325	0.60	27,975	2,575	0.45	21,680	3,325	0.60	27,975
		4,000	2,575	0.45	21,680	3,235	0.58	27,245	2,575	0.45	21,680	3,325	0.60	27,975
		7,500	2,575	0.45	21,680	3,005	0.54	25,285	2,575	0.45	21,680	3,325	0.60	27,975
WSWH24x9	105¼	1,000	5,150	0.43	30,975	5,890	0.52	35,430	5,150	0.43	30,975	6,120	0.54	36,815
		4,000	5,150	0.43	30,975	5,605	0.50	33,715	5,150	0.43	30,975	6,120	0.54	36,815
		7,500	5,150	0.43	30,975	5,275	0.47	31,715	5,150	0.43	30,975	6,120	0.54	36,815
WSWH12x10	117¼	1,000	700	0.50	10,750	900	0.67	13,855	700	0.50	10,750	900	0.67	13,855
		4,000	700	0.50	10,750	900	0.67	13,855	700	0.50	10,750	900	0.67	13,855
		7,500	700	0.50	10,750	900	0.67	13,855	700	0.50	10,750	900	0.67	13,855
WSWH18x10	117¼	1,000	2,140	0.50	20,055	2,755	0.67	25,840	2,140	0.50	20,055	2,755	0.67	25,840
		4,000	2,140	0.50	20,055	2,755	0.67	25,840	2,140	0.50	20,055	2,755	0.67	25,840
		7,500	2,140	0.50	20,055	2,695	0.65	25,285	2,140	0.50	20,055	2,755	0.67	25,840
WSWH24x10	117¼	1,000	4,010	0.48	26,860	5,215	0.67	34,935	4,010	0.48	26,860	5,215	0.67	34,935
		4,000	4,010	0.48	26,860	5,030	0.64	33,715	4,010	0.48	26,860	5,215	0.67	34,935
		7,500	4,010	0.48	26,860	4,735	0.61	31,715	4,010	0.48	26,860	5,215	0.67	34,935
WSWH12x11	129¼	1,000	595	0.56	10,055	765	0.73	12,930	595	0.56	10,055	765	0.73	12,930
		4,000	595	0.56	10,055	765	0.73	12,930	595	0.56	10,055	765	0.73	12,930
		7,500	595	0.56	10,055	765	0.73	12,930	595	0.56	10,055	765	0.73	12,930
WSWH18x11	129¼	1,000	1,960	0.55	20,240	2,520	0.73	26,060	1,960	0.55	20,240	2,520	0.73	26,060
		4,000	1,960	0.55	20,240	2,520	0.73	26,060	1,960	0.55	20,240	2,520	0.73	26,060
		7,500	1,960	0.55	20,240	2,445	0.71	25,285	1,960	0.55	20,240	2,520	0.73	26,060
WSWH24x11	129¼	1,000	4,000	0.54	29,550	4,795	0.68	35,430	4,000	0.54	29,550	4,985	0.70	36,815
		4,000	4,000	0.54	29,550	4,565	0.64	33,715	4,000	0.54	29,550	4,985	0.70	36,815
		7,500	4,000	0.54	29,550	4,295	0.60	31,715	4,000	0.54	29,550	4,985	0.70	36,815
WSWH12x12	144	1,000	505	0.61	9,495	645	0.80	12,150	505	0.61	9,495	645	0.80	12,150
		4,000	505	0.61	9,495	645	0.80	12,150	505	0.61	9,495	645	0.80	12,150
		7,500	505	0.61	9,495	645	0.80	12,150	505	0.61	9,495	645	0.80	12,150
WSWH18x12	144	1,000	1,705	0.61	19,665	2,195	0.80	25,285	1,705	0.61	19,665	2,195	0.80	25,285
		4,000	1,705	0.61	19,665	2,195	0.80	25,285	1,705	0.61	19,665	2,195	0.80	25,285
		7,500	1,705	0.61	19,665	2,195	0.80	25,285	1,705	0.61	19,665	2,195	0.80	25,285
WSWH24x12	144	1,000	3,525	0.60	29,015	4,305	0.75	35,430	3,525	0.60	29,015	4,475	0.78	36,815
		4,000	3,525	0.60	29,015	4,100	0.72	33,715	3,525	0.60	29,015	4,475	0.78	36,815
		7,500	3,525	0.60	29,015	3,855	0.67	31,715	3,525	0.60	29,015	4,475	0.78	36,815

TABLE 2—ALLOWABLE ASD IN-PLANE SHEAR FOR STANDARD APPLICATION STRONG-WALL HIGH STRENGTH WOOD SHEARWALL (WSWH) ON CONCRETE FOUNDATION (CONTINUED)

Strong-Wall High Strength Wood Shearwall Model	Panel Evaluation Height, H_e (in.)	Allow. Vertical Load, P (lb.)	2,500 psi Concrete						3,000 psi Concrete					
			Seismic			Wind			Seismic			Wind		
			Allow. ASD Shear Load, V (lb.)	Drift at Allow. Shear, Δ (in.)	Anchor Tension at Allow. Shear, T (lb.)	Allow. ASD Shear Load, V (lb.)	Drift at Allow. Shear, Δ (in.)	Anchor Tension at Allow. Shear, T (lb.)	Allow. ASD Shear Load, V (lb.)	Drift at Allow. Shear, Δ (in.)	Anchor Tension at Allow. Shear, T (lb.)	Allow. ASD Shear Load, V (lb.)	Drift at Allow. Shear, Δ (in.)	Anchor Tension at Allow. Shear, T (lb.)
WSWH18x13	156	1,000	1,490	0.66	18,575	1,910	0.87	23,855	1,490	0.66	18,575	1,910	0.87	23,855
		4,000	1,490	0.66	18,575	1,910	0.87	23,855	1,490	0.66	18,575	1,910	0.87	23,855
		7,500	1,490	0.66	18,575	1,910	0.87	23,855	1,490	0.66	18,575	1,910	0.87	23,855
WSWH24x13	156	1,000	3,110	0.65	27,705	3,975	0.86	35,430	3,110	0.65	27,705	4,025	0.87	35,885
		4,000	3,110	0.65	27,705	3,780	0.81	33,715	3,110	0.65	27,705	4,025	0.87	35,885
		7,500	3,110	0.65	27,705	3,560	0.77	31,715	3,110	0.65	27,705	4,025	0.87	35,885
WSWH18x14	168	1,000	1,180	0.72	15,890	1,515	0.93	20,370	1,180	0.72	15,890	1,515	0.93	20,370
		4,000	1,180	0.72	15,890	1,515	0.93	20,370	1,180	0.72	15,890	1,515	0.93	20,370
WSWH24x14	168	1,000	2,620	0.71	25,160	3,365	0.93	32,290	2,620	0.71	25,160	3,365	0.93	32,290
		4,000	2,620	0.71	25,160	3,365	0.93	32,290	2,620	0.71	25,160	3,365	0.93	32,290
WSWH18x16	192	1,000	985	0.82	15,160	1,265	1.07	19,395	985	0.82	15,160	1,265	1.07	19,395
		4,000	985	0.82	15,160	1,265	1.07	19,395	985	0.82	15,160	1,265	1.07	19,395
WSWH24x16	192	1,000	2,130	0.82	23,345	2,735	1.07	29,990	2,130	0.82	23,345	2,735	1.07	29,990
		4,000	2,130	0.82	23,345	2,735	1.07	29,990	2,130	0.82	23,345	2,735	1.07	29,990
WSWH18x18	216	1,000	750	0.93	12,965	960	1.20	16,550	750	0.93	12,965	960	1.20	16,550
		4,000	750	0.93	12,965	960	1.20	16,550	750	0.93	12,965	960	1.20	16,550
WSWH24x18	216	1,000	1,655	0.93	20,400	2,110	1.20	26,060	1,655	0.93	20,400	2,110	1.20	26,060
		4,000	1,655	0.93	20,400	2,110	1.20	26,060	1,655	0.93	20,400	2,110	1.20	26,060
WSWH18x20	240	1,000	605	1.04	11,640	770	1.33	14,825	605	1.04	11,640	770	1.33	14,825
		4,000	605	1.04	11,640	770	1.33	14,825	605	1.04	11,640	770	1.33	14,825
WSWH24x20	240	1,000	1,350	1.04	18,500	1,720	1.33	23,590	1,350	1.04	18,500	1,720	1.33	23,590
		4,000	1,350	1.04	18,500	1,720	1.33	23,590	1,350	1.04	18,500	1,720	1.33	23,590

For **SI**: 1 inch = 25.4 mm, 1 lb. = 4.45 N.

1. Allowable ASD shear loads and anchor tension values are applicable to installations on concrete with specified compressive strengths as listed. No further increase for duration of load is allowed.
2. Allowable vertical load denotes the total maximum vertical load permitted on the panel acting in combination with the allowable shear loads.
3. Allowable shear, drift and anchor tension values may be interpolated for intermediate height or vertical loads.
4. For panels $74^1/_2$"-78" tall, use the values for a 78" tall panel.
5. High strength anchor bolts are required unless a lower strength grade is justified by the registered design professional. Figures 6 and 7 of this report provide WSWH-AB anchor bolt information and anchorage solutions.
6. See Table 6 of this report for allowable out-of-plane values; see Table 7 for allowable axial values.
7. Drifts at lower design shear may be linearly reduced.
8. Angled SDS screws may be omitted from the WSWH-TP top connection for all panels taller than 100"; reduced allowable out-of-plane load loads shall apply.
9. Tabulated anchor tension values assume no resisting vertical load. Anchor tension loads at design shear values and including the effect of vertical load may be determined using the following equation:

   $T = [(V \times H) / B] - P/2$, where:

   T = Anchor tension load (lb.); V = Design shear load (lb.); P = Applied vertical load (lb.); H = Panel height (in.)

   B = Moment arm (in.); 7.625" for WSWH12, 12.50" for WSWH18, 17.50" for WSWH24

C.1 Shear Wall Design

East-West Direction

SEISMIC Line A

hsx (ft) = 10.000 Panel Ht.

Shear Wall Design, Deflection & Compatibility

Allowable Drift Δ = 0.020 hsx (in) = 2.400

Cd = 4

Governing min δsw (in) = 0.500 **Stiff (lb/in) = 8,560** ρ = 1.3

ASD = 0.7

ASD Design	Shear Force =	V (lbs) =	**4,280**	ASD Capacity	**DCR**	**δsw (in)**	**Δsw at 1.0 E**	
	Flexible Diaph. Model :	V (lbs) =	3,614	ASD Demand	**84%**	**0.422**	**1.856**	OK

Simpson Strong Wall

Line 2

	Allowable Shear ASD (lbs)	Drift (Allowable Shear) Allowable (in)	Min. (in)	**Available Shear ASD (lbs)**	T Tension at Allowable ASD Shear (lbs)	Ultimate T,ult (lbs)	A.B. Type	C/C Bolts (in)	Overturming Ultimate Movt,ult (ft-lbs)	Amplified Movt,Ω (ft-lbs)
WSWH 18X10	2,140	0.50	0.50	**2,140**	20,055	28,650	HS	14	33,425	72,900
WSWH 18X10	2,140	0.50	0.50	**2,140**	20,055	28,650	HS	14	33,425	72,900

4,280 **4,280** lbs

0.50 in

8,560 lbs/in

Per ESR 2652

	Allowable Shear ASD (lbs)	Drift (Allowable Shear) Allowable (in)	Ht (in)	T Tension at Allowable ASD Shear (lbs)	A.B. Type	C/C Bolts (in)	Overturming Amplified Movt,Ω (ft-lbs)
WSWH 12X8	1,030	0.40	93.25	12,580	STD	8.125	29,500
WSWH 18X8	3,060	0.39	93.25	22,835	HS	14	72,900
WSWH 24X8	6,240	0.37	93.25	33,240	HS	20	103,500
WSWH 12X9	850	0.45	105.25	11,750	STD	8.125	29,500
WSWH 18X9	2,575	0.45	105.25	21,680	HS	14	72,900
WSWH 24X9	5,150	0.43	105.25	30,975	HS	20	103,500
WSWH 12X10	700	0.50	117.25	10,750	STD	8.125	29,500
WSWH 18X10	2,140	0.50	117.25	20,055	HS	14	72,900
WSWH 24X10	4,010	0.48	117.25	26,860	HS	20	103,500
WSWH 12X11	595	0.56	129.25	10,055	STD	8.125	29,500
WSWH 18X11	1,960	0.55	129.25	20,240	HS	14	72,900

SEISMIC Line B

Shear Wall Design, Deflection & Compatibility

				hsx (ft) =	**10.000**	Panel Ht.
				Allowable Drift Δ = 0.020 hsx (in) =	**2.400**	
				Cd =	**4**	
Governing min δsw (in) =	**0.480**	**Stiff (lb/in) =**	**33,417**	**ρ =**	**1.3**	
				ASD =	**0.7**	

ASD Design	Shear Force =	V (lbs) =	**16,040**	ASD Capacity	**DCR**	**δsw (in)**	**Δsw at 1.0 E**	
	Flexible Diaph. Model :	V (lbs) =	11,999	ASD Demand	**75%**	**0.359**	**1.578**	**OK**

Simpson Strong Wall

Line 2

	Allowable Shear ASD (lbs)	Drift (Allowable Shear) Allowable (in)	Min. (in)	**Available Shear ASD (lbs)**	T Tension at Allowable ASD Shear (lbs)	Ultimate T,ult (lbs)	A.B. Type	C/C Bolts (in)	Overturning Ultimate Movt,ult (ft-lbs)	Amplified Movt,Ω (ft-lbs)
WSWH 24X10	4,010	0.48	0.48	**4,010**	26,860	38,371	HS	20	63,952	103,500
WSWH 24X10	4,010	0.48	0.48	**4,010**	26,860	38,371	HS	20	63,952	103,500
WSWH 24X10	4,010	0.48	0.48	**4,010**	26,860	38,371	HS	20	63,952	103,500
WSWH 24X10	4,010	0.48	0.48	**4,010**	26,860	38,371	HS	20	63,952	103,500

16,040 **16,040** lbs

0.48 in

33,417 lbs/in

Per ESR 2652

	Allowable Shear ASD (lbs)	Drift (Allowable Shear) Allowable (in)	Ht (in)	T Tension at Allowable ASD Shear (lbs)	A.B. Type	C/C Bolts (in)	Overturning Amplified Movt,Ω (ft-lbs)
WSWH 12X8	1,030	0.40	93.25	12,580	STD	8.125	29,500
WSWH 18X8	3,060	0.39	93.25	22,835	HS	14	72,900
WSWH 24X8	6,240	0.37	93.25	33,240	HS	20	103,500
WSWH 12X9	850	0.45	105.25	11,750	STD	8.125	29,500
WSWH 18X9	2,575	0.45	105.25	21,680	HS	14	72,900
WSWH 24X9	5,150	0.43	105.25	30,975	HS	20	103,500
WSWH 12X10	700	0.50	117.25	10,750	STD	8.125	29,500
WSWH 18X10	2,140	0.50	117.25	20,055	HS	14	72,900
WSWH 24X10	4,010	0.48	117.25	26,860	HS	20	103,500
WSWH 12X11	595	0.56	129.25	10,055	STD	8.125	29,500
WSWH 18X11	1,960	0.55	129.25	20,240	HS	14	72,900

SEISMIC Line D

hsx (ft) = 10.000 Panel Ht.

Shear Wall Design, Deflection & Compatibility

Allowable Drift Δ = 0.020 hsx (in) = 2.400

Cd = 4

Governing min δsw (in) = 0.480 **Stiff (lb/in) = 33,417**

ρ = 1.3

ASD = 0.7

ASD Design	Shear Force =	V (lbs) =	**16,040**	ASD Capacity	**DCR**	**δsw (in)**	**Δsw at 1.0 E**	
	Flexible Diaph. Model :	V (lbs) =	11,353	ASD Demand	**71%**	**0.340**	**1.493**	OK

Simpson Strong Wall

Line D

	Allowable Shear ASD (lbs)	Drift (Allowable Shear) Allowable (in)	Min. (in)	**Available Shear ASD (lbs)**	T Tension at Allowable ASD Shear (lbs)	Ultimate T,ult (lbs)	A.B. Type	C/C Bolts (in)	Overturning Ultimate Movt,ult (ft-lbs)	Amplified Movt,Ω (ft-lbs)
WSWH 24X10	4,010	0.48	0.48	**4,010**	26,860	38,371	HS	20	63,952	103,500
WSWH 24X10	4,010	0.48	0.48	**4,010**	26,860	38,371	HS	20	63,952	103,500
WSWH 24X10	4,010	0.48	0.48	**4,010**	26,860	38,371	HS	20	63,952	103,500
WSWH 24X10	4,010	0.48	0.48	**4,010**	26,860	38,371	HS	20	63,952	103,500

16,040 **16,040** lbs

0.48 in

33,417 lbs/in

Per ESR 2652

	Allowable Shear ASD (lbs)	Drift (Allowable Shear) Allowable (in)	Ht (in)	T Tension at Allowable ASD Shear (lbs)	A.B. Type	C/C Bolts (in)	Overturning Amplified Movt,Ω (ft-lbs)
WSWH 12X8	1,030	0.40	93.25	12,580	STD	8.125	29,500
WSWH 18X8	3,060	0.39	93.25	22,835	HS	14	72,900
WSWH 24X8	6,240	0.37	93.25	33,240	HS	20	103,500
WSWH 12X9	850	0.45	105.25	11,750	STD	8.125	29,500
WSWH 18X9	2,575	0.45	105.25	21,680	HS	14	72,900
WSWH 24X9	5,150	0.43	105.25	30,975	HS	20	103,500
WSWH 12X10	700	0.50	117.25	10,750	STD	8.125	29,500
WSWH 18X10	2,140	0.50	117.25	20,055	HS	14	72,900
WSWH 24X10	4,010	0.48	117.25	26,860	HS	20	103,500
WSWH 12X11	595	0.56	129.25	10,055	STD	8.125	29,500
WSWH 18X11	1,960	0.55	129.25	20,240	HS	14	72,900

Shear wall out of plane Line A:

TABLE 6—ALLOWABLE ASD OUT-OF-PLANE SHEAR FOR STRONG-WALL HIGH STRENGTH WOOD SHEARWALL (WSWH) ON CONCRETE FOUNDATION (PSF)

Panel Attachment	WSWH-TP Top Connection Fastening	Strong-Wall High Strength Wood Shearwall Model	Nominal Height of Shearwall (ft.)										
			7	8	9	10	11	12	13	14	16	18	20
Top Plates	Angled SDS Screws Omitted	WSWH12	N/A	N/A	85	75	70	35	N/A	N/A	N/A	N/A	N/A
		WSWH18	N/A	N/A	125	115	105	80	65	50	35	25	15
		WSWH24	N/A	N/A	120	110	100	80	65	50	35	25	15
	Angled SDS Screws Installed	WSWH12	420	290	205	145	95	35	N/A	N/A	N/A	N/A	N/A
		WSWH18	395	290	205	145	110	80	65	50	35	25	15
		WSWH24	370	290	205	145	110	80	65	50	35	25	15
Header	Angled SDS Screws Installed	WSWH12	330	205	150	110	85	45	N/A	N/A	N/A	N/A	N/A
		WSWH18	285	205	150	110	85	65	N/A	N/A	N/A	N/A	N/A
		WSWH24	215	180	150	110	85	65	N/A	N/A	N/A	N/A	N/A

For **SI**: 1 inch = 25.4 mm, 1 ft. = 305 mm, 1 lb. = 4.45 N, 1 psf = 47.88 Pa.

1. Loads shown are at ASD level in pounds per square foot with no further increase allowed.
2. Loads consider a maximum deflection limit of h / 240.
3. Allowable out-of-plane loads can be applied in combination with the allowable vertical loads listed in Tables 2 and 3.
4. Allowable values for header panel attachment assume a maximum header depth of 12". Use a load reduction factor of 0.94, 0.88 and 0.82 for 14", 16" and 18" deep headers respectively.
5. Allowable values shown for header panel attachment require the use of the portal kit to resist header rotation.
6. Angled SDS screws may be omitted for WSWH panels taller than 100" in standard applications; however, SWS16150 screws must be installed for all fastening conditions as shown in Figure 2. When angels SDS screws are omitted, a reduced allowable out-of-plane load may apply.

WSWH18x10, out of plane ar 9.5' = 175 plf

1.5 ft x 9.5 ft x (175 psf) = 2494 lbs. total ASD Horizontal Load

Wind Load: (EWA = 80 ft2, (9ft trib. width x 9.5 ft ht = 86 ft2) WSWH walls located at Zone 4 for wall wind pressure;
20.3 psf + 4.3 psf = 24.6 psf

24.6 psf x 86 ft2 = 2115 lbs < 2,494 lbs **OK** .

- SEISMIC Line F

hsx (ft) = 10.000

Shear Wall Design, Deflection & Compatibility

Allowable Drift Δ = 0.020 hsx (in) = 2.400

Cd = 4 ρ = 1.30

Stiff (lb/in) = 27,770 ASD = 0.7

					DCR		δsw	Δsw at 1.0 E	
ASD Capacity	Shear Force =	V (lbs) =	7,896	lbs			0.284	1.250	At Capacity
ASD Demand	Flexible Diaph. Model :	V (lbs) =	7,337	lbs.	93%	OK	0.263	1.154	OK
Load Input at Load Combination 0.7 ρ E								0.010	*< 0.020*

Shear Wall Label		W-F1	W-F2
Shear Wall Geometry ::			
Shear Wall Length	b (ft) =	16.000	12.000
Shear Wall Height	h (ft) =	10.000	10.000
h/b > 3.5 ? If yes, wall length is inadequate.	h/b =	0.63	0.83
Sheathing Material and Edge Nailing Pattern ::			
Sheathing Grade		Sheathing	Sheathing
Common Nail Size		10d	10d
Min. Nominal Panel Thickness (in.)		15/32	15/32
Panel Edge Nail Spacing (in.)		[6]	[6]
Sheathing Type [Plywood or OSB]		PLY	PLY
If sheathing is Plywood, 4-Ply or 5-Ply ?		No	No
Sheathing Both sides or One Side ?		1-Side	1-Side
End Posts ::			
End-j	End Member =	4x6	4x6
Offset (at Tension), C.L. of Holdown / Strap	CL,j (in) =	5	5
End-k	End Member =	4x6	4x6
Offset (at Tension), C.L. of Holdown / Strap	CL,k (in) =	5	5
**			
Determination of Shear Wall Segment Effective Capacity ::			
	vn (plf) =	870	870
Hold-down inside face of end post, & 10D Edge Nails ?		Yes	Yes
If "Yes", multiply vn x 0.92 =	vn (plf) =	800	800
Seismic v (ASD) = vn / 2.8 , Seismic vs (plf) =		286	286
Aspect Ratio Factor	WSP = 1.25 - 0.125 (h/b) =	1.000	1.000
Unit shear force induced by design load (Wall ASD Capacity)			
Reduced by aspect Ratio Factor, Seismic vs' (plf) =		286	286
Total for Wall Segment, Seismic vs' (plf) =		286	286
	Vs' (lbs) =	4,576	3,432
Shear Wall Segment Capcity Itteration ::			
Tension at End-j *Governing Deflection (in)*	* f =	1.00	0.97
27,770 - 0.284 7,896 lbs	Σ (	4,576	3,320)
% of Contribution to Total Line Capacity of Governing Case		58.0%	42.0%
	v' comp. = f x vs' (plf) =	286	277
Tension at End-k *Governing Deflection (in)*	* f =	1.00	0.97
27,770 - 0.284 7,896 lbs	Σ (	4,576	3,320)
% of Contribution to Total Line Capacity of Governing Case		58.0%	42.0%
	v' comp. = f x vs' (plf) =	286	277

* For Deflection compatibilty:; f = Wall Deflection at ASD capacity per segment / Smallest deflection within the Line

ASD Demand				W-F1	W-F2
Semi-rigid Diaph. Model	Tension at End-j	$V_{,ASD,SR}$	(lbs)	0	0
			(plf)	0	0
	Tension at End-k	$V_{,ASD,SR}$	(lbs)	0	0
			(plf)	0	0
Flexible Diaph. Model	Tension at End-j	$V_{,ASD,F}$	(lbs)	4,252	3,085
			(plf)	266	257
	Tension at End-k	$V_{,ASD,F}$	(lbs)	4,252	3,085
			(plf)	266	257

Deflection Summary ::				W-F1	W-F2
At ASD Wall Capacity::	Tension at	End Post-j ::	δsw (in) =	0.284	0.294
	Tension at	End Post-k ::	δsw (in) =	0.284	0.294
At Compatible Shear::	Tension at	End Post-j ::	δsw (in) =	0.284	0.284
	Tension at	End Post-k ::	δsw (in) =	0.284	0.284

Deflection Calculation at ASD Demand ::

Semi-rigid Diaph Model :					
	Tension at	End Post-j ::	δsw (in) =	0.000	0.000
	Tension at	End Post-k ::	δsw (in) =	0.000	0.000
Flexible Diaph. Model :					
	Tension at	End Post-j ::	δsw (in) =	0.263	0.262
	Tension at	End Post-k ::	δsw (in) =	0.263	0.262

Wall Overturning Stabilty ::			W-F1	W-F2
Loading at End Posts ::				
Uniform Loads ::				
Roof/Floor Weight 1 ::		L (ft) =	16.000	12.000
		Trib (ft) =	2.00	2.00
		Dead (psf) or (plf) =	10.00	10.00
Roof/Floor Weight 2 ::		L (ft) =	16.000	12.000
		Trib (ft) =		
		Dead (psf) or (plf) =		
Wall Self Weight ::		L (ft) =	16.000	12.000
		Height (ft) =	10.000	10.000
		Dead (psf) or (plf) =	14.00	14.00
Total Uniform		(lbs) =	2576	1932
End Post-j ::				
Load 1 ::		(lbs) =	0	0
		Trib. (ft) =		
		Trib. (ft) =		
		Dead (psf) or (plf) pr (lbs) =		
Load 2 ::		(lbs) =	0.000	0.000
		Trib. (ft) =		
		Trib. (ft) =		
		Dead (psf) or (plf) pr (lbs) =		
1/2 Uniform			1288	966
Total End-j		**(lbs) =**	**1288**	**966**
End Post-k ::				
Load 1 ::		(lbs) =	0	0
		Trib. (ft) =		
		Trib. (ft) =		
		Dead (psf) or (plf) pr (lbs) =		
Load 2 ::		(lbs) =	0	0
		Trib. (ft) =		
		Trib. (ft) =		
		Dead (psf) or (plf) pr (lbs) =		
1/2 Uniform			1288	966
Total End-k		**(lbs) =**	**1288**	**966**

- SEISMIC Line F

Determine Holdowns/Straps for Each Wall Segments ASD Capacity

Overturning Moment due to Seismic ::		**W-F1**	**W-F2**
At ASD Capacity of Shear Walls:			
	Vs' (lbs) =	4,576	3,432
	h (ft) =	10.00	10.00
i.e. at [= 0.7 ρ E]	M,overturning = Vs' h (ft-lbs) =	45,760	34,320

Overturning w/ Tension at End Post-j ::			
	b (ft) =	16.00	12.00
	CL,j (in) =	5.00	5.00
	d,k (in) =	3.50	3.50
	Lever Arm,j = b - CL,j - 0.5 d,k (in) =	15.44	11.44
Tension or Compression T,j or C,k =			
	M,overturning / Lever Arm (lbs) =	2,964	3,001
at [ASD LC = 0.7 ρ E - 0.6 D]			
Tnet,k Net Tension at End Post-j = (lbs) =		**2,191**	**2,421**
C,k Compression at End Post-k * = (lbs) =		2,964	3,001

End Post-j ::		**Holdowns**	**Holdowns**
		HDU-2 (2)2x	**HDU-2 (2)2x**
Holdown/Strap ASD Capacity	(lbs) =	3,075	3,075
Deflection at ASD Capacity	(in) =	0.088	0.088
Net Carried Down From Above	(lbs) =		
Holdown/Strap at ASD Demand	(lbs) =	1,982	2,117
		64%	**69%**

Overturning w/ Tension at End Post-k ::			
	b (ft) =	16.00	12.00
	CL,k (in) =	5.00	5.00
	d,j (in) =	3.50	3.50
	Lever Arm,k = b - CL,j - 0.5 d,k (in) =	15.44	11.44
Tension or Compression T,k or C,j =			
	M,overturning / Lever Arm (lbs) =	2,964	3,001
at [ASD LC = 0.7 ρ E - 0.6 D]			
C,j Compression at End Post-j * = (lbs) =		2,964	3,001
Tnet,k Net Tension at End Post-k = (lbs) =		**2,191**	**2,421**

End Post-k ::		**Holdowns**	**Holdowns**
		HDU-2 (2)2x	**HDU-2 (2)2x**
Holdown/Strap ASD Capacity	(lbs) =	3,075	3,075
Deflection at ASD Capacity	(in) =	0.088	0.088
Net Carried Down From Above	(lbs) =		
Holdown/Strap at ASD Demand	(lbs) =	1,982	2,117
		64%	**69%**

** [Exlude Load on Brg. Studs]*

C.1.2 Shear Wall Design

North-South Direction

- SEISMIC Line 1

hsx (ft) = 10.000

Shear Wall Design, Deflection & Compatibility

Allowable Drift Δ = 0.020 hsx (in) = 2.400

Cd = 4 ρ = 1.30

Stiff (lb/in) = 104,695 ASD = 0.7

					DCR		δsw	Δsw at 1.0 E	
ASD Capacity	Shear Force =	V (lbs) =	19,006	lbs			0.182	0.798	At Capacity
ASD Demand	Flexible Diaph. Model :	V (lbs) =	17,506	lbs.	92%	OK	0.167	0.735	OK

Load Input at Load Combination 0.7 ρ E

Shear Wall Label			W-11	W-12
Shear Wall Geometry ::				
Shear Wall Length	b (ft) =		24.000	48.000
Shear Wall Height	h (ft) =		10.000	10.000
h/b > 3.5 ? If yes, wall length is inadequate.	h/b =		0.42	0.21
Sheathing Material and Edge Nailing Pattern ::				
Sheathing Grade			Sheathing	Sheathing
Common Nail Size			10d	10d
Min. Nominal Panel Thickness (in.)			15/32	15/32
Panel Edge Nail Spacing (in.)			[6]	[6]
Sheathing Type [Plywood or OSB]			PLY	PLY
If sheathing is Plywood, 4-Ply or 5-Ply ?			Yes	Yes
Sheathing Both sides or One Side ?			1-Side	1-Side
End Posts ::				
End-j	End Member =		4x6	4x6
Offset (at Tension), C.L. of Holdown / Strap	CL,j (in) =		5	5
End-k	End Member =		4x6	4x6
Offset (at Tension), C.L. of Holdown / Strap	CL,k (in) =		5	5
**				
Determination of Shear Wall Segment Effective Capacity ::				
	vn (plf) =		870	870
Hold-down inside face of end post, & 10D Edge Nails ?			Yes	Yes
If "Yes", multiply vn x 0.92 =	vn (plf) =		800	800
Seismic v (ASD) = vn / 2.8 , Seismic vs (plf) =			286	286
Aspect Ratio Factor	WSP = 1.25 - 0.125 (h/b) =		1.000	1.000
Unit shear force induced by design load (Wall ASD Capacity)				
Reduced by aspect Ratio Factor, Seismic vs' (plf) =			286	286
Total for Wall Segment, Seismic vs' (plf) =			286	286
	Vs' (lbs) =		6,864	13,728
Shear Wall Segment Capcity Itteration ::				
Tension at End-j	*Governing Deflection (in)*	* f =	0.77	1.00
104,695 -	0.182	19,006 lbs Σ (	5,278	13,728)
% of Contribution to Total Line Capacity of Governing Case			27.8%	72.2%
	v' comp. = f x vs' (plf) =		220	286
Tension at End-k	*Governing Deflection (in)*	* f =	0.77	1.00
104,695 -	0.182	19,006 lbs Σ (	5,278	13,728)
% of Contribution to Total Line Capacity of Governing Case			27.8%	72.2%
	v' comp. = f x vs' (plf) =		220	286

* For Deflection compatibilty:; f = Wall Deflection at ASD capacity per segment / Smallest deflection within the Line

ASD Demand				W-11	W-12
Semi-rigid Diaph. Model	Tension at End-j	$V_{,ASD,SR}$	(lbs)	0	0
			(plf)	0	0
	Tension at End-k	$V_{,ASD,SR}$	(lbs)	0	0
			(plf)	0	0
Flexible Diaph. Model	Tension at End-j	$V_{,ASD,F}$	(lbs)	4,862	12,644
			(plf)	203	263
	Tension at End-k	$V_{,ASD,F}$	(lbs)	4,862	12,644
			(plf)	203	263

Deflection Summary ::				W-11	W-12
At ASD Wall Capacity::	Tension at	End Post-j ::	δsw (in) =	0.236	0.182
	Tension at	End Post-k ::	δsw (in) =	0.236	0.182
At Compatible Shear::	Tension at	End Post-j ::	δsw (in) =	0.174	0.182
	Tension at	End Post-k ::	δsw (in) =	0.174	0.182

Deflection Calculation at ASD Demand ::

Semi-rigid Diaph Model :					
	Tension at	End Post-j ::	δsw (in) =	0.000	0.000
	Tension at	End Post-k ::	δsw (in) =	0.000	0.000
Flexible Diaph. Model :					
	Tension at	End Post-j ::	δsw (in) =	0.158	0.167
	Tension at	End Post-k ::	δsw (in) =	0.158	0.167

Wall Overturning Stabilty ::			W-11	W-12
Loading at End Posts ::				
Uniform Loads ::				
Roof/Floor Weight 1 ::		L (ft) =	24.000	48.000
		Trib (ft) =	2.00	10.00
		Dead (psf) or (plf) =	8.00	10.00
Roof/Floor Weight 2 ::		L (ft) =	24.000	48.000
		Trib (ft) =		
		Dead (psf) or (plf) =		
Wall Self Weight ::		L (ft) =	24.000	48.000
		Height (ft) =	10.000	10.000
		Dead (psf) or (plf) =	14.00	14.00
Total Uniform		(lbs) =	3768	11568
End Post-j ::				
Load 1 ::		(lbs) =	0	0
		Trib. (ft) =		
		Trib. (ft) =		
		Dead (psf) or (plf) pr (lbs) =		
Load 2 ::		(lbs) =	0.000	0.000
		Trib. (ft) =		
		Trib. (ft) =		
		Dead (psf) or (plf) pr (lbs) =		
1/2 Uniform			1884	5784
Total End-j		**(lbs) =**	**1884**	**5784**
End Post-k ::				
Load 1 ::		(lbs) =	0	0
		Trib. (ft) =		
		Trib. (ft) =		
		Dead (psf) or (plf) pr (lbs) =		
Load 2 ::		(lbs) =	0	0
		Trib. (ft) =		
		Trib. (ft) =		
		Dead (psf) or (plf) pr (lbs) =		
1/2 Uniform			1884	5784
Total End-k		**(lbs) =**	**1884**	**5784**

- SEISMIC Line 1

Determine Holdowns/Straps for Each Wall Segments ASD Capacity

Overturning Moment due to Seismic ::		**W-11**	**W-12**
At ASD Capacity of Shear Walls:			
	Vs' (lbs) =	6,864	13,728
	h (ft) =	10.00	10.00
i.e. at [= 0.7 ρ E]	M,overturning = Vs' h (ft-lbs) =	68,640	137,280
Overturning w/ Tension at End Post-j ::			
	b (ft) =	24.00	48.00
	CL,j (in) =	5.00	5.00
	d,k (in) =	3.50	3.50
	Lever Arm,j = b - CL,j - 0.5 d,k (in) =	23.44	47.44
	Tension or Compression T,j or C,k = M,overturning / Lever Arm (lbs) =	2,929	2,894
at [ASD LC = 0.7 ρ E - 0.6 D]			
	Tnet,k Net Tension at End Post-j = (lbs) =	**1,798**	**0**
	C,k Compression at End Post-k * = (lbs) =	2,929	2,894
End Post-j ::		**Holdowns HDU-2 (2)2x**	**Holdowns HDU-2 (2)2x**
	Holdown/Strap ASD Capacity (lbs) =	3,075	3,075
	Deflection at ASD Capacity (in) =	0.088	0.088
	Net Carried Down From Above (lbs) =		
	Holdown/Strap at ASD Demand (lbs) =	944	0
		31%	**0%**
Overturning w/ Tension at End Post-k ::			
	b (ft) =	24.00	48.00
	CL,k (in) =	5.00	5.00
	d,j (in) =	3.50	3.50
	Lever Arm,k = b - CL,j - 0.5 d,k (in) =	23.44	47.44
	Tension or Compression T,k or C,j = M,overturning / Lever Arm (lbs) =	2,929	2,894
at [ASD LC = 0.7 ρ E - 0.6 D]			
	C,j Compression at End Post-j * = (lbs) =	2,929	2,894
	Tnet,k Net Tension at End Post-k = (lbs) =	**1,798**	**0**
End Post-k ::		**Holdowns HDU-2 (2)2x**	**Holdowns HDU-2 (2)2x**
	Holdown/Strap ASD Capacity (lbs) =	3,075	3,075
	Deflection at ASD Capacity (in) =	0.088	0.088
	Net Carried Down From Above (lbs) =		
	Holdown/Strap at ASD Demand (lbs) =	944	0
		31%	**0%**

** [Exlude Load on Brg. Studs]*

- SEISMIC Line 3

Shear Wall Design, Deflection & Compatibility

hsx (ft) = 10.000

Allowable Drift Δ = 0.020 hsx (in) = 2.400

Cd = 4 ρ = 1.30

ASD = 0.7

Stiff (lb/in) = 104,695

					DCR		δsw	Δsw at 1.0 E	
ASD Capacity	Shear Force =	V (lbs) =	19,006	lbs			0.182	0.798	At Capacity
ASD Demand	Flexible Diaph. Model :	V (lbs) =	17,755	lbs.	93%	OK	0.170	0.745	OK

Load Input at Load Combination 0.7 ρ E

Shear Wall Label			W-31	W-32
Shear Wall Geometry ::				
Shear Wall Length	b (ft) =		24.000	48.000
Shear Wall Height	h (ft) =		10.000	10.000
h/b > 3.5 ? If yes, wall length is inadequate.	h/b =		0.42	0.21
Sheathing Material and Edge Nailing Pattern ::				
Sheathing Grade			Sheathing	Sheathing
Common Nail Size			10d	10d
Min. Nominal Panel Thickness (in.)			15/32	15/32
Panel Edge Nail Spacing (in.)			[6]	[6]
Sheathing Type [Plywood or OSB]			PLY	PLY
If sheathing is Plywood, 4-Ply or 5-Ply ?			Yes	Yes
Sheathing Both sides or One Side ?			1-Side	1-Side
End Posts ::				
End-j	End Member =		4x6	4x6
Offset (at Tension), C.L. of Holdown / Strap	CL,j (in) =		5	5
End-k	End Member =		4x6	4x6
Offset (at Tension), C.L. of Holdown / Strap	CL,k (in) =		5	5
**				
Determination of Shear Wall Segment Effective Capacity ::				
	vn (plf) =		870	870
Hold-down inside face of end post, & 10D Edge Nails ?			Yes	Yes
If "Yes", multiply vn x 0.92 =	vn (plf) =		800	800
Seismic v (ASD) = vn / 2.8 , Seismic vs (plf) =			286	286
Aspect Ratio Factor	WSP = 1.25 - 0.125 (h/b) =		1.000	1.000
Unit shear force induced by design load (Wall ASD Capacity)				
Reduced by aspect Ratio Factor, Seismic vs' (plf) =			286	286
Total for Wall Segment, Seismic vs' (plf) =			286	286
	Vs' (lbs) =		6,864	13,728
Shear Wall Segment Capcity Itteration ::				
Tension at End-j	*Governing Deflection (in)*	* f =	0.77	1.00
104,695 -	0.182	19,006 lbs Σ (	5,278	13,728)
% of Contribution to Total Line Capacity of Governing Case			27.8%	72.2%
	v' comp. = f x vs' (plf) =		220	286
Tension at End-k	*Governing Deflection (in)*	* f =	0.77	1.00
104,695 -	0.182	19,006 lbs Σ (	5,278	13,728)
% of Contribution to Total Line Capacity of Governing Case			27.8%	72.2%
	v' comp. = f x vs' (plf) =		220	286

* For Deflection compatibilty:; f = Wall Deflection at ASD capacity per segment / Smallest deflection within the Line

ASD Demand				W-31	W-32
Semi-rigid Diaph. Model	Tension at End-j	$V_{,ASD,SR}$	(lbs)	0	0
			(plf)	0	0
	Tension at End-k	$V_{,ASD,SR}$	(lbs)	0	0
			(plf)	0	0
Flexible Diaph. Model	Tension at End-j	$V_{,ASD,F}$	(lbs)	4,931	12,824
			(plf)	205	267
	Tension at End-k	$V_{,ASD,F}$	(lbs)	4,931	12,824
			(plf)	205	267

- SEISMIC Line 3

Deflection Summary ::				W-31	W-32
At ASD Wall Capacity::	Tension at	End Post-j ::	δsw (in) =	0.236	0.182
	Tension at	End Post-k ::	δsw (in) =	0.236	0.182
At Compatible Shear::	Tension at	End Post-j ::	δsw (in) =	0.174	0.182
	Tension at	End Post-k ::	δsw (in) =	0.174	0.182

Deflection Calculation at ASD Demand ::

Semi-rigid Diaph Model :					
	Tension at	End Post-j ::	δsw (in) =	0.000	0.000
	Tension at	End Post-k ::	δsw (in) =	0.000	0.000
Flexible Diaph. Model :					
	Tension at	End Post-j ::	δsw (in) =	0.160	0.170
	Tension at	End Post-k ::	δsw (in) =	0.160	0.170

Wall Overturning Stabilty ::			W-31	W-32
Loading at End Posts ::				
Uniform Loads ::				
Roof/Floor Weight 1 ::		L (ft) =	24.000	48.000
		Trib (ft) =	2.00	10.00
		Dead (psf) or (plf) =	8.00	10.00
Roof/Floor Weight 2 ::		L (ft) =	24.000	48.000
		Trib (ft) =		
		Dead (psf) or (plf) =		
Wall Self Weight ::		L (ft) =	24.000	48.000
		Height (ft) =	10.000	10.000
		Dead (psf) or (plf) =	14.00	14.00
Total Uniform		(lbs) =	3768	11568
End Post-j ::				
Load 1 ::		(lbs) =	0	0
		Trib. (ft) =		
		Trib. (ft) =		
		Dead (psf) or (plf) pr (lbs) =		
Load 2 ::		(lbs) =	0.000	0.000
		Trib. (ft) =		
		Trib. (ft) =		
		Dead (psf) or (plf) pr (lbs) =		
1/2 Uniform			1884	5784
Total End-j		**(lbs) =**	**1884**	**5784**
End Post-k ::				
Load 1 ::		(lbs) =	0	0
		Trib. (ft) =		
		Trib. (ft) =		
		Dead (psf) or (plf) pr (lbs) =		
Load 2 ::		(lbs) =	0	0
		Trib. (ft) =		
		Trib. (ft) =		
		Dead (psf) or (plf) pr (lbs) =		
1/2 Uniform			1884	5784
Total End-k		**(lbs) =**	**1884**	**5784**

- SEISMIC Line 3

Determine Holdowns/Straps for Each Wall Segments ASD Capacity

Overturning Moment due to Seismic ::		**W-31**	**W-32**
At ASD Capacity of Shear Walls:			
	Vs' (lbs) =	6,864	13,728
	h (ft) =	10.00	10.00
i.e. at [= 0.7 ρ E]	M,overturning = Vs' h (ft-lbs) =	68,640	137,280
Overturning w/ Tension at End Post-j ::			
	b (ft) =	24.00	48.00
	CL,j (in) =	5.00	5.00
	d,k (in) =	3.50	3.50
	Lever Arm,j = b - CL,j - 0.5 d,k (in) =	23.44	47.44
	Tension or Compression T,j or C,k = M,overturning / Lever Arm (lbs) =	2,929	2,894
at [ASD LC = 0.7 ρ E - 0.6 D]			
	Tnet,k Net Tension at End Post-j = (lbs) =	**1,798**	**0**
	C,k Compression at End Post-k * = (lbs) =	2,929	2,894
End Post-j ::		**Holdowns HDU-2 (2)2x**	**Holdowns HDU-2 (2)2x**
	Holdown/Strap ASD Capacity (lbs) =	3,075	5,645
	Deflection at ASD Capacity (in) =	0.088	0.115
	Net Carried Down From Above (lbs) =		
	Holdown/Strap at ASD Demand (lbs) =	973	0
		32%	**0%**
Overturning w/ Tension at End Post-k ::			
	b (ft) =	24.00	48.00
	CL,k (in) =	5.00	5.00
	d,j (in) =	3.50	3.50
	Lever Arm,k = b - CL,j - 0.5 d,k (in) =	23.44	47.44
	Tension or Compression T,k or C,j = M,overturning / Lever Arm (lbs) =	2,929	2,894
at [ASD LC = 0.7 ρ E - 0.6 D]			
	C,j Compression at End Post-j * = (lbs) =	2,929	2,894
	Tnet,k Net Tension at End Post-k = (lbs) =	**1,798**	**0**
End Post-k ::		**Holdowns HDU-2 (2)2x**	**Holdowns HDU-2 (2)2x**
	Holdown/Strap ASD Capacity (lbs) =	3,075	5,645
	Deflection at ASD Capacity (in) =	0.088	0.115
	Net Carried Down From Above (lbs) =		
	Holdown/Strap at ASD Demand (lbs) =	973	0
		32%	**0%**

** [Exlude Load on Brg. Studs]*

D. Diaphragm Design

Roof Diaphragm Nailing Pattern

Seismic Loading : East - West Direction

Seismic Forces for Shear Wall per Lines: **0.7 ρ E (lbs)** ρ = **1.30**

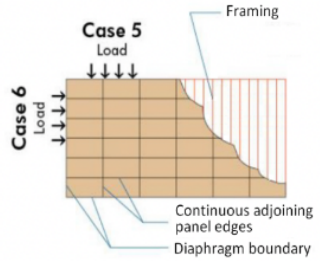
Roof Diaphragm

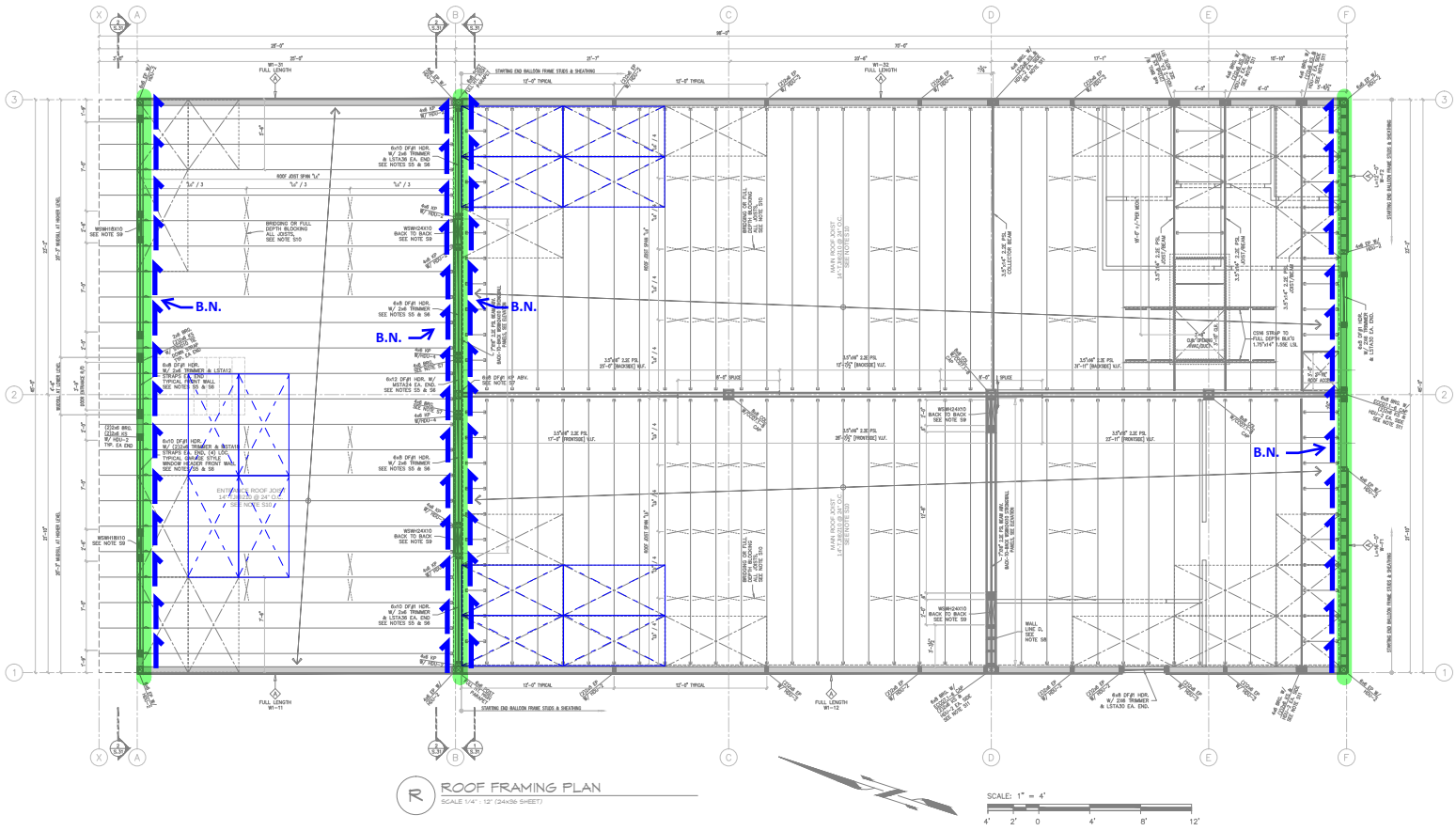
East-West Direction

										Diaph. Shear (lbs)		Diaph. Depth (ft)	Diaph. Shear (plf)
Line A [Wall]	**3,614**	lbs	Line A [Wall]	1,274	lbs								
			Line A[Side Facing B]	2,340	lbs	x	100%	2,340	lbs	44.000	53
			Line B[Side Facing A]	1,964	lbs	x	100%	1,964	lbs	44.000	45
Line B [Wall]	**11,999**	lbs	Line B [Wall]	4,192	lbs						
			Line B[Side Facing D]	5,843	lbs	x	100%	5,843	lbs	44.000	133
			Line D[Side Facing B]	5,843	lbs	x	100%	5,843	lbs	44.000	133
Line D [Wall]	**11,353**	lbs	Line D [Wall]	702	lbs						
			Line D[Side Facing F]	4,808	lbs	x	100%	4,808	lbs	44.000	109
			Line F[Side Facing D]	4,621	lbs	x	100%	4,621	lbs	44.000	105
Line F [Wall]	**7,337**	lbs	Line F [Wall]	2,717	lbs						
Total	**34,304**	**lbs**									

All are under
< 185 plf OK
Unblocked, 6" o.c. B.N.

ESR-1153 TJI Joist Framed Diaphragms: Unblocked
Use Nailing Patterns as follows: Un-Blocked Diaphragm
19/32 min. APA Rated Sheathing, Roof Joists are TJI-210. ,
Diaphragm Boundaries [10dx3" nails at 6" o.c.]
Supported Panel Edges [10dx3" nails at 6" o.c.]
Field Nails [10dx3" nails at 12" o.c.]
Allowable diaphragm shear
Case-5 & 6 is = **185 plf**



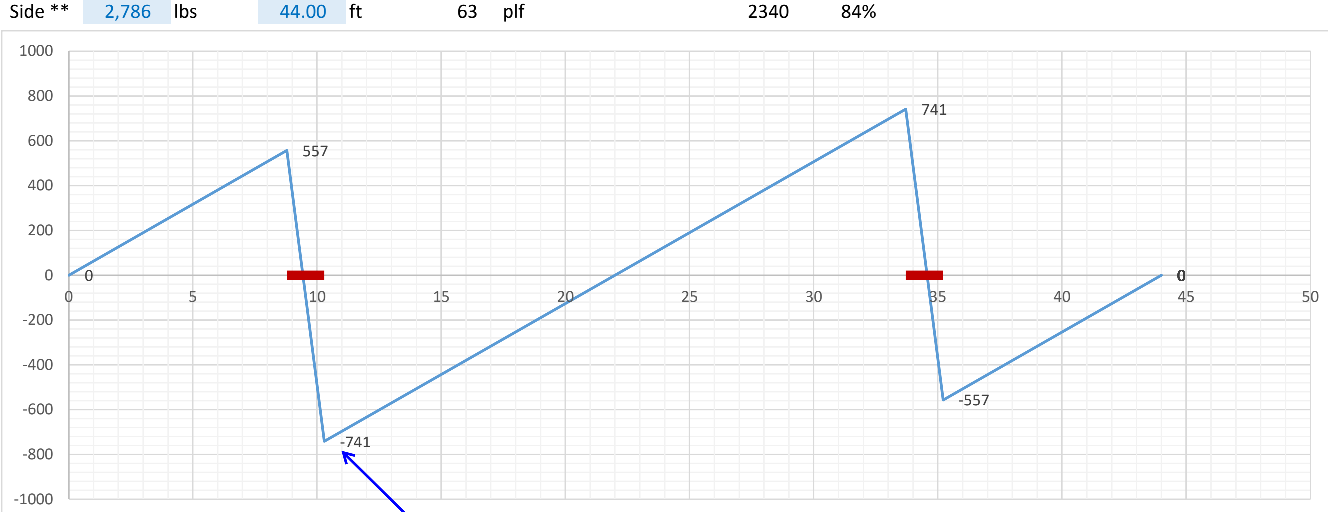


Collectors Forces:

Line A

Length	Location	Type	Label	Wall (ASD Demand)		Diaphragm Portion	A	Side *	B	C Σ B to Loc.	Side **	D	E Σ D to Loc.	Collector Force (D = C + E - A)
(ft)	(ft)			(lbs)		(lbs)	(lbs)		(lbs)	(lbs)		(lbs)	(lbs)	(lbs)
8.79	8.79	Collector			0%	0	0		0	0	Y	557	557	557
1.50	10.29	Wall	W-A1	1,393	50%	1,393	1,393		0	0	Y	95	652	-741
23.42	33.71	Collector			0%	0	1,393		0	0	Y	1,483	2,134	741
1.50	35.21	Wall	W-A2	1,393	50%	1,393	2,786		0	0	Y	95	2,229	-557
8.79	44.00	Collector			0%	0	2,786		0	0	Y	557	2,786	0
	44.00				0%	0	2,786		0	0		0	2,786	0
	44.00				0%	0	2,786		0	0		0	2,786	0
	44.00				0%	0	2,786		0	0		0	2,786	0
	44.00				0%	0	2,786		0	0		0	2,786	0
	44.00				0%	0	2,786		0	0		0	2,786	0
	44.00				0%	0	2,786		0	0		0	2,786	0
	44.00				0%	0	2,786		0	0		0	2,786	0
	44.00				0%	0	2,786		0	0		0	2,786	0
	44.00				0%	0	2,786		0	0		0	2,786	0
	44.00				0%	0	2,786		0	0		0	2,786	0
	44.00				0%	0	2,786		0	0		0	2,786	0
	44.00				0%	0	2,786		0	0		0	2,786	0
	44.00				0%	0	2,786		0	0		0	2,786	0
	44.00				0%	0	2,786		0	0		0	2,786	0
	44.00				0%	0	2,786		0	0		0	2,786	0
44.00				2,786		2,786								

					Diaph. Shear				
Side *	0	lbs	0.00	ft	0	plf			
Wall		lbs						Wall DCR	
Side **	2,786	lbs	44.00	ft	63	plf	2340	84%	



Use Top plate splice
SPLC-A
(12) 16dx3.5" nails
(141 #/nail x 1.6 [CD = 1.6 for E] = 225 #/nail

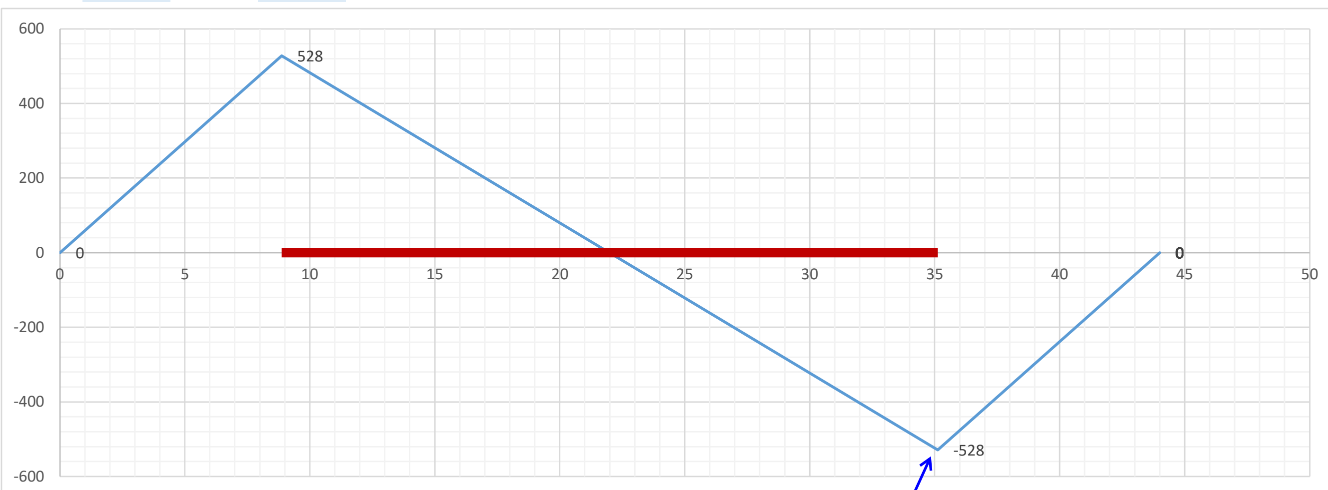
ASD Cap. 2,700 lbs > 557 lbs OK

Collectors Forces:

Line B **Ledger at Side Facing A**

Length	Location	Type	Label	Wall		Diaphragm Portion	A	Side *	B	C Σ B to Loc.	Side **	D	E Σ D to Loc.	Collector Force
				ASD Demand										D = C + E - A
(ft)	(ft)			(lbs)		(lbs)	(lbs)		(lbs)	(lbs)		(lbs)	(lbs)	(lbs)
8.88	8.88	Collector			0%	0	0		0	0	Y	528	528	528
26.25	35.13	Wall	W-B	2,619	100%	2,619	2,619		0	0	Y	1,562	2,090	-528
8.88	44.00	Collector			0%	0	2,619		0	0	Y	528	2,619	0
	44.00				0%	0	2,619		0	0		0	2,619	0
	44.00				0%	0	2,619		0	0		0	2,619	0
	44.00				0%	0	2,619		0	0		0	2,619	0
	44.00				0%	0	2,619		0	0		0	2,619	0
	44.00				0%	0	2,619		0	0		0	2,619	0
	44.00				0%	0	2,619		0	0		0	2,619	0
	44.00				0%	0	2,619		0	0		0	2,619	0
	44.00				0%	0	2,619		0	0		0	2,619	0
	44.00				0%	0	2,619		0	0		0	2,619	0
	44.00				0%	0	2,619		0	0		0	2,619	0
	44.00				0%	0	2,619		0	0		0	2,619	0
	44.00				0%	0	2,619		0	0		0	2,619	0
	44.00				0%	0	2,619		0	0		0	2,619	0
	44.00				0%	0	2,619		0	0		0	2,619	0
	44.00				0%	0	2,619		0	0		0	2,619	0
	44.00				0%	0	2,619		0	0		0	2,619	0
	44.00				0%	0	2,619		0	0		0	2,619	0
	44.00				0%	0	2,619		0	0		0	2,619	0
	44.00				0%	0	2,619		0	0		0	2,619	0
44.00				2,619		2,619								

					Diaph. Shear					
Side *	0	lbs	0.00	ft	0	plf				
Wall		lbs						Wall DCR		
Side **	2,619	lbs	44.00	ft	60	plf		1964	75%	



Use CS16 Strap with 18" MIN end length ea side of break at any point where break in ledger occurs along length of wall line.

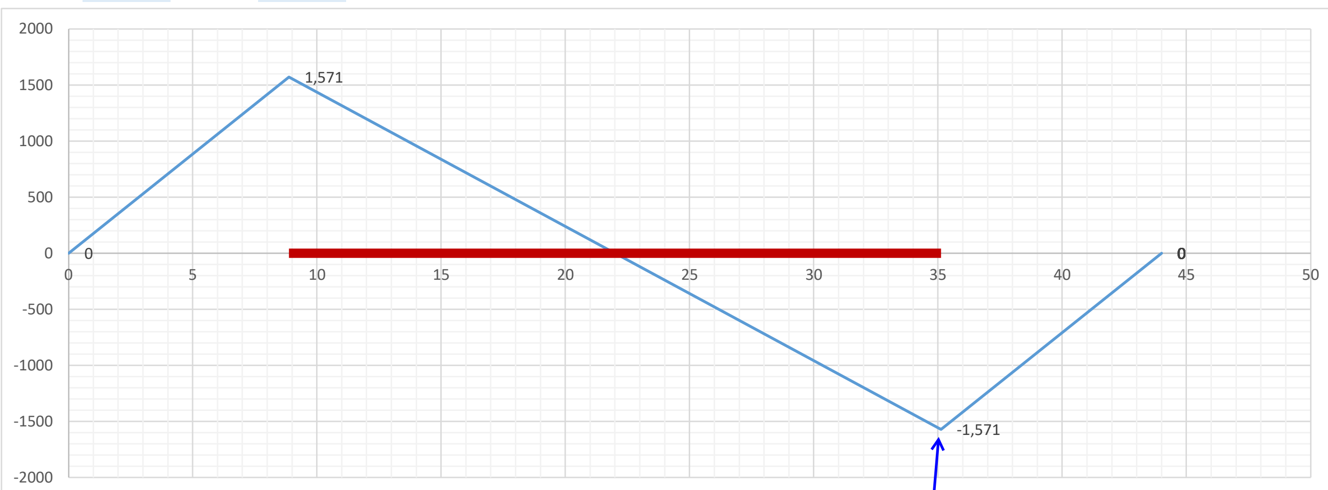
ASD Cap. 1,705 lbs > 1,132 lbs OK

Collectors Forces:

Line B **Ledger at Side Facing D**

Length (ft)	Location (ft)	Type	Label	ASD Demand Wall (lbs)		Diaphragm Portion (lbs)	A (lbs)	Side *	B (lbs)	C Σ B to Loc. (lbs)	Side **	D (lbs)	E Σ D to Loc. (lbs)	D = C + E - A Collector Force (lbs)
8.88	8.88	Collector			0%	0	0		0	0	Y	1,571	1,571	1,571
26.25	35.13	Wall	W-B	7,791	100%	7,791	7,791		0	0	Y	4,648	6,219	-1,571
8.88	44.00	Collector			0%	0	7,791		0	0	Y	1,571	7,791	0
	44.00				0%	0	7,791		0	0		0	7,791	0
	44.00				0%	0	7,791		0	0		0	7,791	0
	44.00				0%	0	7,791		0	0		0	7,791	0
	44.00				0%	0	7,791		0	0		0	7,791	0
	44.00				0%	0	7,791		0	0		0	7,791	0
	44.00				0%	0	7,791		0	0		0	7,791	0
	44.00				0%	0	7,791		0	0		0	7,791	0
	44.00				0%	0	7,791		0	0		0	7,791	0
	44.00				0%	0	7,791		0	0		0	7,791	0
	44.00				0%	0	7,791		0	0		0	7,791	0
	44.00				0%	0	7,791		0	0		0	7,791	0
	44.00				0%	0	7,791		0	0		0	7,791	0
	44.00				0%	0	7,791		0	0		0	7,791	0
	44.00				0%	0	7,791		0	0		0	7,791	0
	44.00				0%	0	7,791		0	0		0	7,791	0
	44.00				0%	0	7,791		0	0		0	7,791	0
	44.00				0%	0	7,791		0	0		0	7,791	0
44.00				7,791		7,791								

						Diaph. Shear			
Side *	0	lbs	0.00	ft	0	plf			
Wall		lbs						Wall DCR	
Side **	7,791	lbs	44.00	ft	177	plf		5843	75%



Use CS14 Strap with 24" MIN end length ea side of break at any point where break in ledger occurs along length of wall line.

ASD Cap. 2,490 lbs > 1,571 lbs OK

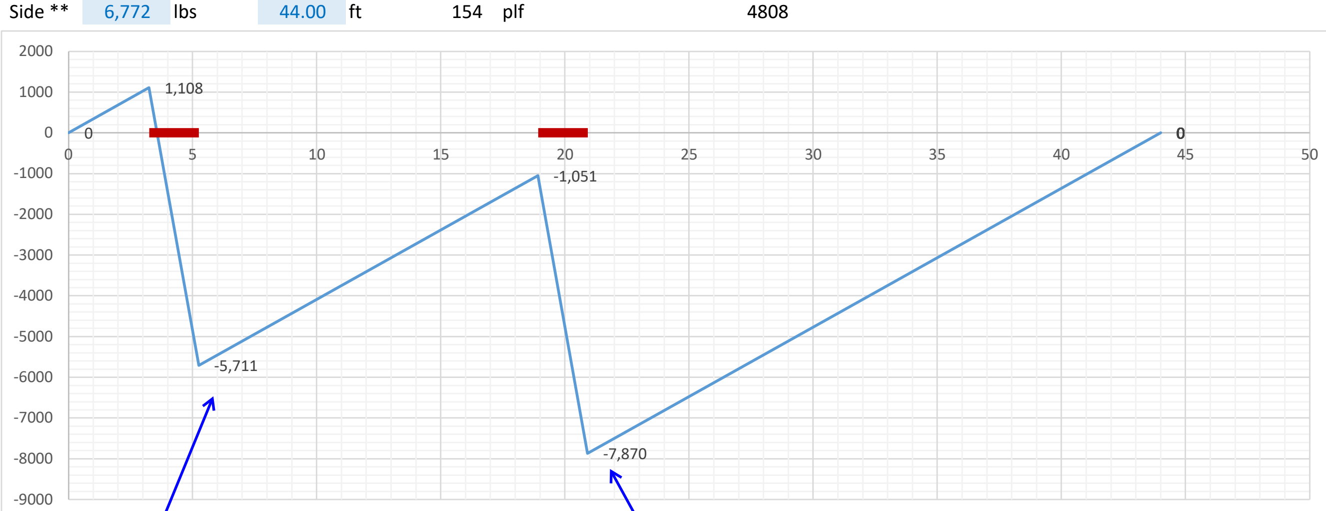
Collectors Forces:

Line D

ASD Demand — D = C + E - A

Length (ft)	Location (ft)	Type	Label	Wall (lbs)		Diaphragm Portion (lbs)	A (lbs)	Side *	B (lbs)	C Σ B to Loc. (lbs)	Side **	D (lbs)	E Σ D to Loc. (lbs)	Collector Force (lbs)
3.25	3.25	Collector			0%	0	0	Y	608	608	Y	500	500	1,108
2.00	5.25	Wall	W-D1	7,501	50%	7,501	7,501	Y	374	982	Y	308	808	-5,711
13.67	18.92	Collector			0%	0	7,501	Y	2,556	3,538	Y	2,103	2,911	-1,051
2.00	20.92	Wall	W-D2	7,501	50%	7,501	15,001	Y	374	3,912	Y	308	3,219	-7,870
23.08	44.00	Collector			0%	0	15,001	Y	4,317	8,230	Y	3,553	6,772	0
	44.00				0%	0	15,001		0	8,230		0	6,772	0
	44.00				0%	0	15,001		0	8,230		0	6,772	0
	44.00				0%	0	15,001		0	8,230		0	6,772	0
	44.00				0%	0	15,001		0	8,230		0	6,772	0
	44.00				0%	0	15,001		0	8,230		0	6,772	0
	44.00				0%	0	15,001		0	8,230		0	6,772	0
	44.00				0%	0	15,001		0	8,230		0	6,772	0
	44.00				0%	0	15,001		0	8,230		0	6,772	0
	44.00				0%	0	15,001		0	8,230		0	6,772	0
	44.00				0%	0	15,001		0	8,230		0	6,772	0
	44.00				0%	0	15,001		0	8,230		0	6,772	0
	44.00				0%	0	15,001		0	8,230		0	6,772	0
	44.00				0%	0	15,001		0	8,230		0	6,772	0
	44.00				0%	0	15,001		0	8,230		0	6,772	0
	44.00				0%	0	15,001		0	8,230		0	6,772	0
44.00				15,001		15,001								

						Diaph. Shear			Wall DCR	
Side *	8,230	lbs	44.00	ft	187	plf		5843	71%	
Wall		lbs								
Side **	6,772	lbs	44.00	ft	154	plf		4808		



continuous PSL Header, no break in shear transfer.

Use CMST12 x 8'-0" STRAP
ABV. SHEATHING TO PSL HEADER & LSL
JOIST w/ (90)16d(0.162)x3.5" NAILS,
1/2 QTY. EA. END

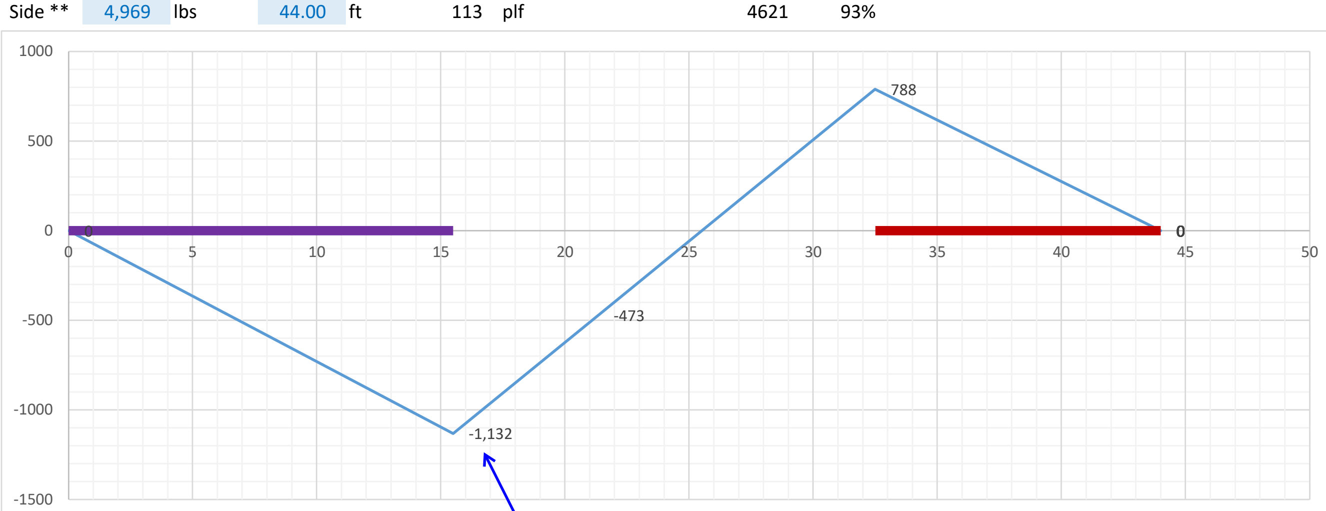
ASD Cap. 9,215 lbs > 7,870 lbs OK

Collectors Forces:

Line F

Length	Location	Type	Label	ASD Demand Wall		Diaphragm Portion	A	Side *	B	C Σ B to Loc.	Side **	D	E Σ D to Loc.	D = C + E - A Collector Force
(ft)	(ft)			(lbs)		(lbs)	(lbs)		(lbs)	(lbs)		(lbs)	(lbs)	(lbs)
15.50	15.50	Wall	W-F1	2,882	58%	2,882	2,882		0	0	Y	1,750	1,750	-1,132
5.83	21.33	Collector			0%	0	2,882		0	0	Y	659	2,409	-473
11.17	32.50	Collector			0%	0	2,882		0	0	Y	1,261	3,670	788
11.50	44.00	Wall	W-F2	2,087	42%	2,087	4,969		0	0	Y	1,299	4,969	0
	44.00				0%	0	4,969		0	0		0	4,969	0
	44.00				0%	0	4,969		0	0		0	4,969	0
	44.00				0%	0	4,969		0	0		0	4,969	0
	44.00				0%	0	4,969		0	0		0	4,969	0
	44.00				0%	0	4,969		0	0		0	4,969	0
	44.00				0%	0	4,969		0	0		0	4,969	0
	44.00				0%	0	4,969		0	0		0	4,969	0
	44.00				0%	0	4,969		0	0		0	4,969	0
	44.00				0%	0	4,969		0	0		0	4,969	0
	44.00				0%	0	4,969		0	0		0	4,969	0
	44.00				0%	0	4,969		0	0		0	4,969	0
	44.00				0%	0	4,969		0	0		0	4,969	0
	44.00				0%	0	4,969		0	0		0	4,969	0
	44.00				0%	0	4,969		0	0		0	4,969	0
	44.00				0%	0	4,969		0	0		0	4,969	0
	44.00				0%	0	4,969		0	0		0	4,969	0
44.00				4,969		4,969								

					Diaph. Shear				
Side *	0	lbs	0.00	ft	0	plf			
Wall		lbs						Wall DCR	
Side **	4,969	lbs	44.00	ft	113	plf	4621		93%



Use CS14 Strap with 24" MIN end length ea side of break at any point where break in ledger occurs along length of wall line.

ASD Cap. 2,490 lbs > 1,132 lbs OK

See Chord Forces Line B and F following pages,

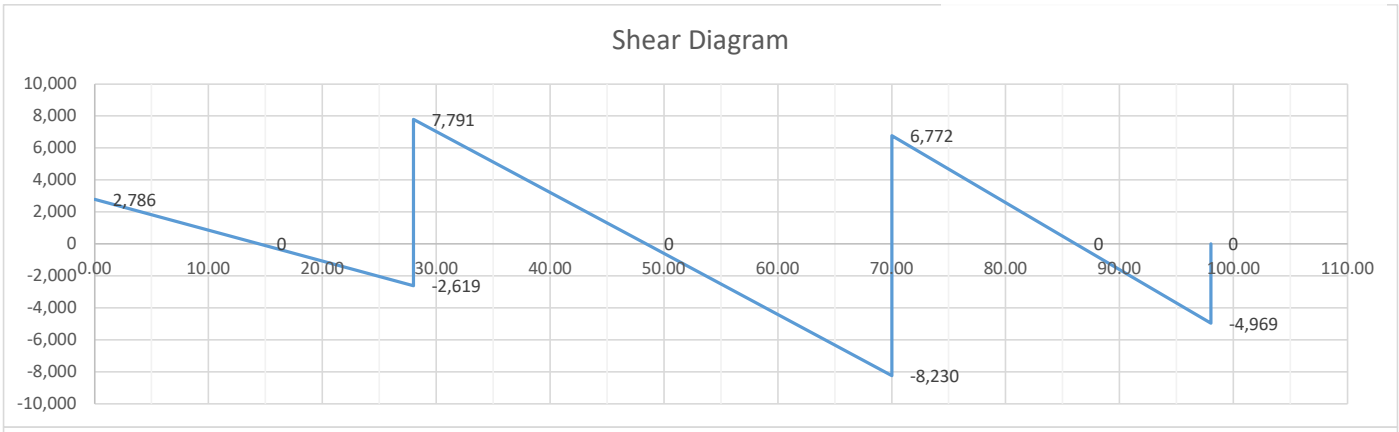
ASD Cap. 2,490 lbs > 2,048 lbs OK

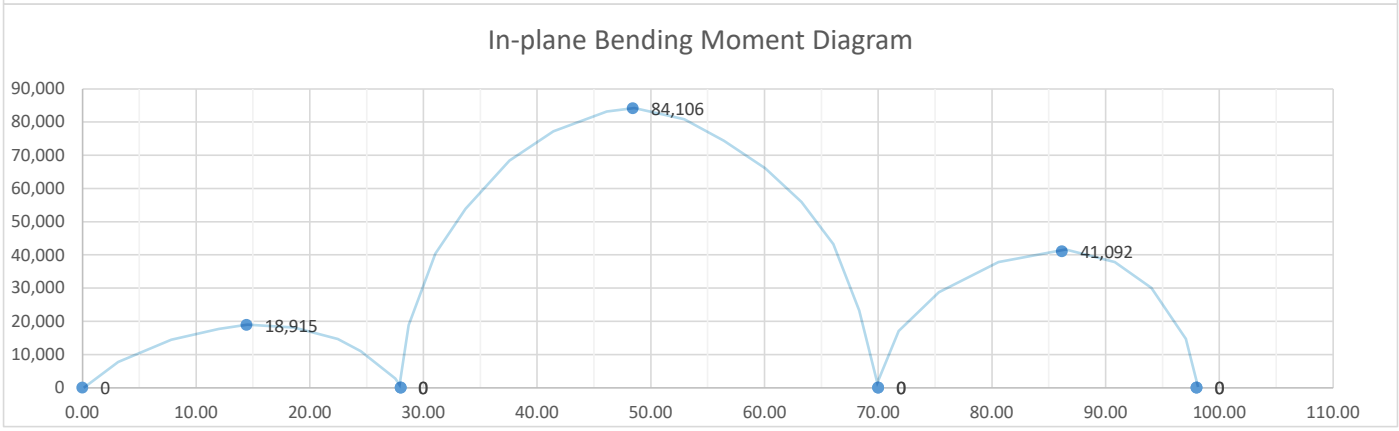
Chord Forces
Lines [1] and [3]

Line	A	28.00 ft	B	42.00 ft	D	28.00 ft	F
Diaphragm Reactions	2,340 lbs	1,964	5,843	5,843	4,808	4,621	
Shear Wall Line DCR	84%	75%	75%	71%	71%	93%	
Diaphragm Reactions at ASD Cap.	2,786 lbs	2,619	7,791	8,230	6,772	4,969	
Max. In-plane Bending Moment							
Mij midspan = (Vi + Vj) x Lij / 8	18,915 ft-lbs		84,106 ft-lbs		41,092 ft-lbs		
Diaphragm Depth d	44.00 ft		44.00 ft		44.00 ft		
Max. Chord Force T or C = M / d	430 lbs		1,912 lbs		934 lbs		

Use CS14 Strap with 24" MIN end length ea side of break at any point where break in ledger occurs along length of wall line.

ASD Cap. 2,490 lbs > 1,912 lbs OK





Roof Diaphragm Nailing Pattern

Seismic Loading : North - South Direction

Seismic Forces for Shear Wall per Lines: **0.7 ρ E (lbs)** ρ = **1.30**

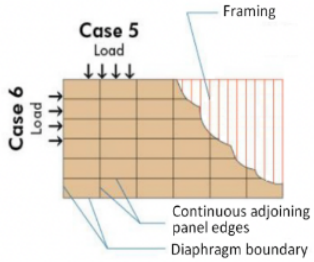
Roof Diaphragm

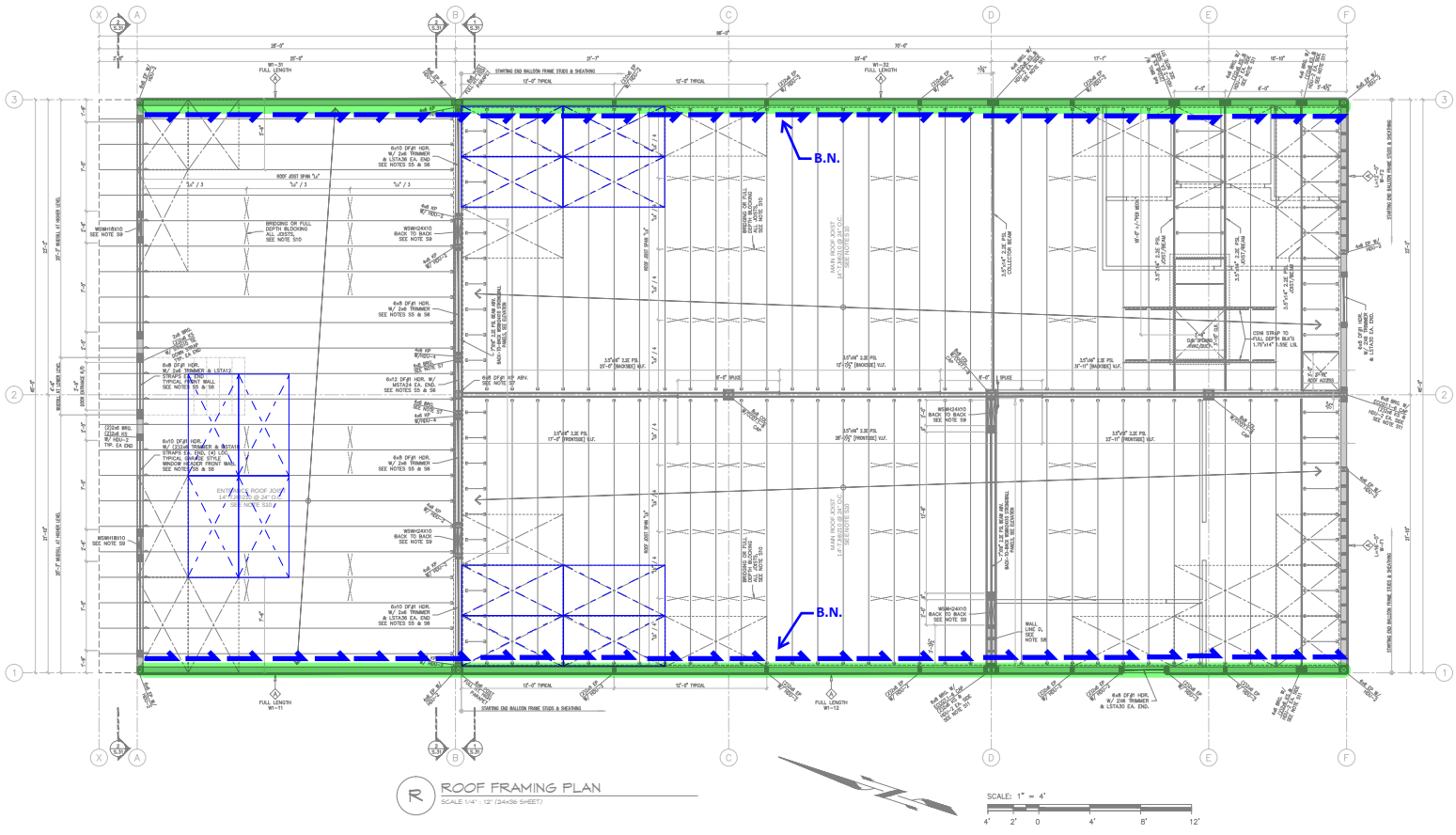
North-South Direction

									Diaph. Shear (lbs)		Diaph. Depth (ft)	Diaph. Shear (plf)
Line 1 [Wall]	**17,506**	lbs	Line 1 [Wall]	5,400	lbs							
			Line 1[Side Facing 3]	12,106	lbs	x	100%		12,106	lbs	95	127
			Line 3[Side Facing 1]	12,355	lbs	x	100%		12,355	lbs	95	130
Line 3 [Wall]	**17,755**	lbs	Line 3 [Wall]	5,400	lbs							
Total	**35,261**	**lbs**										

All are under
< 185 plf OK
Unblocked, 6" o.c. B.N.

ESR-1153 TJI Joist Framed Diaphragms: Unblocked
Use Nailing Patterns as follows: Un-Blocked Diaphragm
19/32 min. APA Rated Sheathing, Roof Joists are TJI-210. ,
Diaphragm Boundaries [10dx3" nails at 6" o.c.]
Supported Panel Edges [10dx3" nails at 6" o.c.]
Field Nails [10dx3" nails at 12" o.c.]
Allowable diaphragm shear
Case-5 & 6 is = **185 plf**



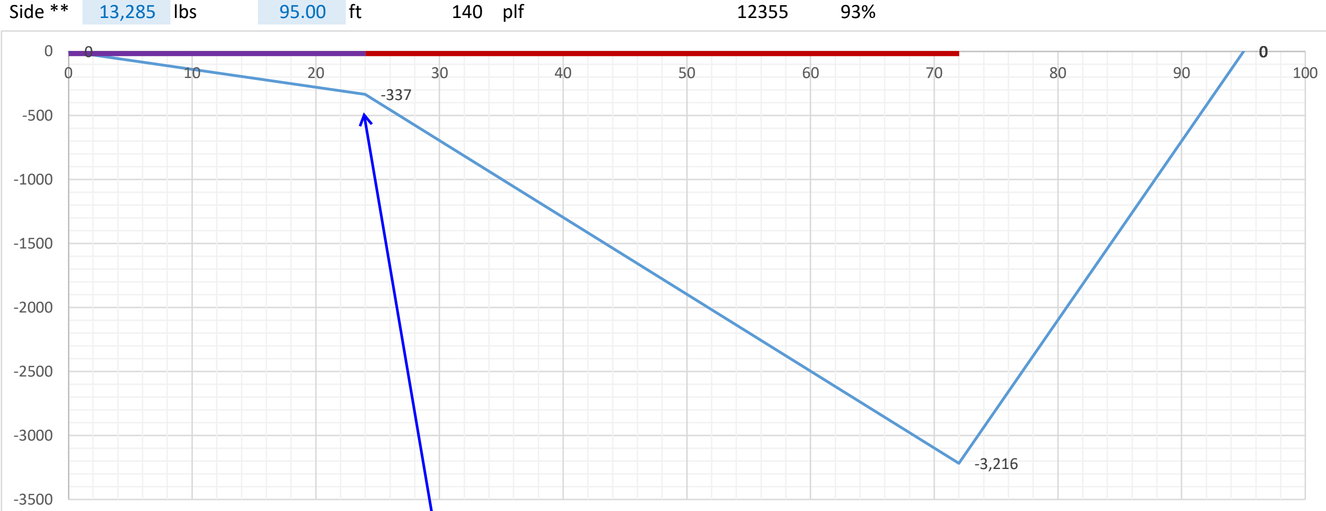


Collectors Forces:

Line 3 Line 1 Sim. / Case1 walls continuous along line

				ASD Demand										D = C + E - A
Length	Location	Type	Label	Wall		Diaphragm Portion	A	Side *	B	C Σ B to Loc.	Side **	D	E Σ D to Loc.	Collector Force
(ft)	(ft)			(lbs)		(lbs)	(lbs)		(lbs)	(lbs)		(lbs)	(lbs)	(lbs)
24.00	24.00	Wall	W-31	3,693	28%	3,693	3,693		0	0	Y	3,356	3,356	-337
48.00	72.00	Wall	W-32	9,592	72%	9,592	13,285		0	0	Y	6,712	10,069	-3,216
23.00	95.00	Collector			0%	0	13,285		0	0	Y	3,216	13,285	0
	95.00				0%	0	13,285		0	0	Y	0	13,285	0
	95.00				0%	0	13,285		0	0		0	13,285	0
	95.00				0%	0	13,285		0	0		0	13,285	0
	95.00				0%	0	13,285		0	0		0	13,285	0
	95.00				0%	0	13,285		0	0		0	13,285	0
	95.00				0%	0	13,285		0	0		0	13,285	0
	95.00				0%	0	13,285		0	0		0	13,285	0
	95.00				0%	0	13,285		0	0		0	13,285	0
	95.00				0%	0	13,285		0	0		0	13,285	0
	95.00				0%	0	13,285		0	0		0	13,285	0
	95.00				0%	0	13,285		0	0		0	13,285	0
	95.00				0%	0	13,285		0	0		0	13,285	0
	95.00				0%	0	13,285		0	0		0	13,285	0
	95.00				0%	0	13,285		0	0		0	13,285	0
	95.00				0%	0	13,285		0	0		0	13,285	0
	95.00				0%	0	13,285		0	0		0	13,285	0
	95.00				0%	0	13,285		0	0		0	13,285	0
95.00				13,285		13,285								

						Diaph. Shear			
Side *	0	lbs	0.00	ft	0	plf			
Wall		lbs						Wall DCR	
Side **	13,285	lbs	95.00	ft	140	plf		12355	93%



Use CS14 Strap with 24" MIN end length ea side of break at any point where break in ledger occurs along length of wall line.

ASD Cap. 2,490 lbs > 337 lbs OK

Collectors Forces:

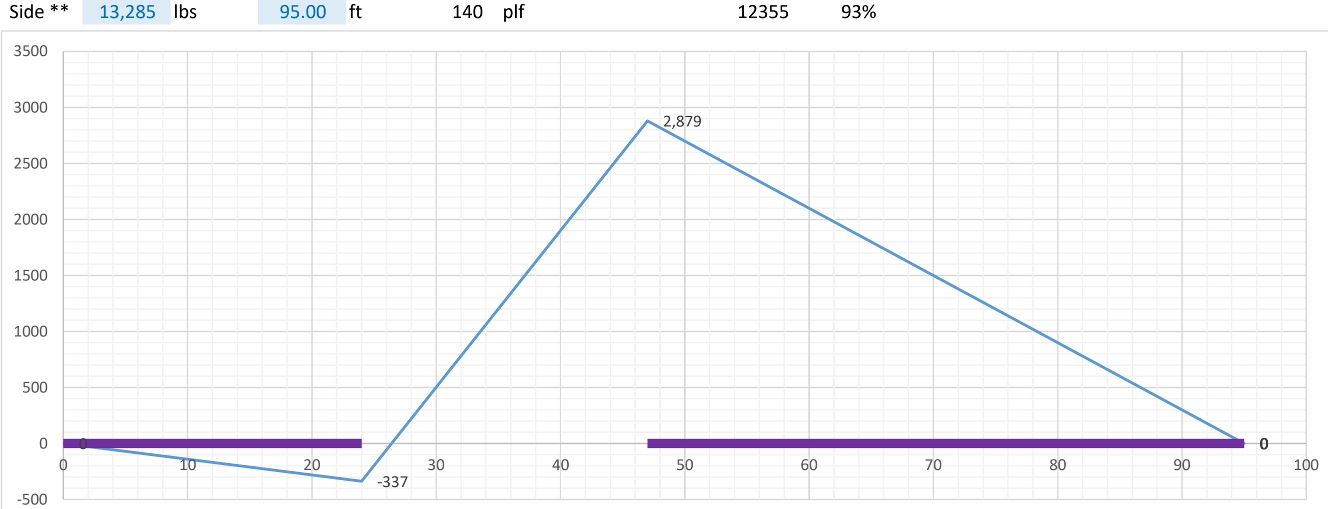
Line 3 Line 1 Sim. / Case2 walls with sheathing gap portion between

ASD Demand D = C + E - A

Length (ft)	Location (ft)	Type	Label	Wall (lbs)		Diaphragm Portion (lbs)	A (lbs)	Side *	B (lbs)	C Σ B to Loc. (lbs)	Side **	D (lbs)	E Σ D to Loc. (lbs)	Collector Force (lbs)
24.00	24.00	Wall	W-31	3,693	28%	3,693	3,693		0	0	Y	3,356	3,356	-337
23.00	47.00	Collector			0%	0	3,693		0	0	Y	3,216	6,573	2,879
48.00	95.00	Wall	W-32	9,592	72%	9,592	13,285		0	0	Y	6,712	13,285	0
	95.00				0%	0	13,285		0	0		0	13,285	0
	95.00				0%	0	13,285		0	0		0	13,285	0
	95.00				0%	0	13,285		0	0		0	13,285	0
	95.00				0%	0	13,285		0	0		0	13,285	0
	95.00				0%	0	13,285		0	0		0	13,285	0
	95.00				0%	0	13,285		0	0		0	13,285	0
	95.00				0%	0	13,285		0	0		0	13,285	0
	95.00				0%	0	13,285		0	0		0	13,285	0
	95.00				0%	0	13,285		0	0		0	13,285	0
	95.00				0%	0	13,285		0	0		0	13,285	0
	95.00				0%	0	13,285		0	0		0	13,285	0
	95.00				0%	0	13,285		0	0		0	13,285	0
	95.00				0%	0	13,285		0	0		0	13,285	0
	95.00				0%	0	13,285		0	0		0	13,285	0
	95.00				0%	0	13,285		0	0		0	13,285	0
	95.00				0%	0	13,285		0	0		0	13,285	0
	95.00				0%	0	13,285		0	0		0	13,285	0
	95.00				0%	0	13,285		0	0		0	13,285	0
	95.00				0%	0	13,285		0	0		0	13,285	0
95.00				13,285		13,285								

Diaph. Shear

Side *	0	lbs	0.00	ft	0	plf			
Wall		lbs						Wall DCR	
Side **	13,285	lbs	95.00	ft	140	plf	12355	93%	



Chord Forces
Lines [B] and [F]

Line	1	44.00	ft	3
Diaphragm Reactions	12,106	lbs	12,355	
Shear Wall Line DCR	92%		93%	
Diaphragm Reactions at ASD Cap.	13,159	lbs	13,285	

Max. In-plane Bending Moment
Mij midspan = (Vi + Vj) x Lij / 8 145,440 ft-lbs

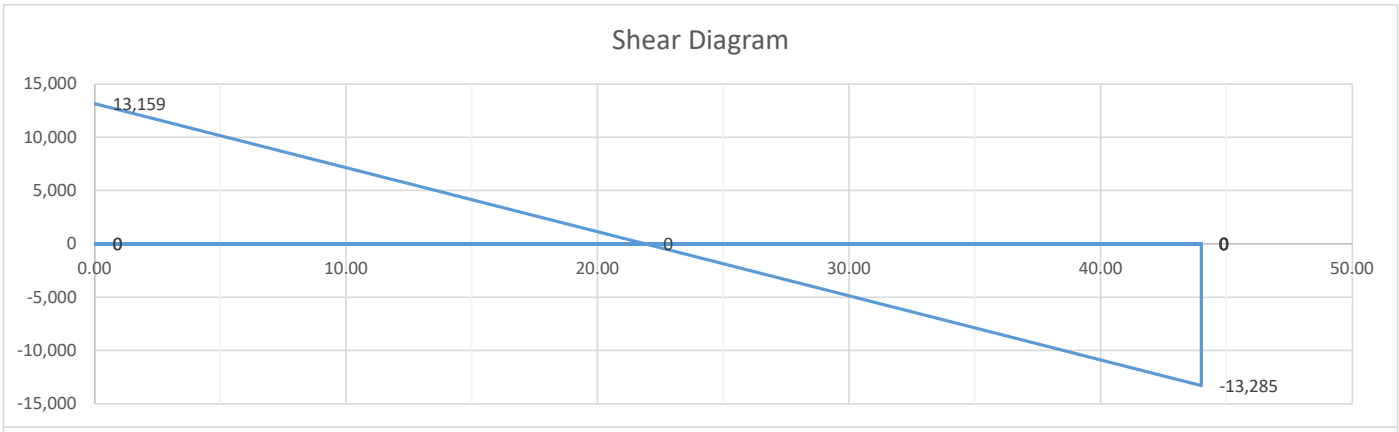
Diaphragm Depth d 71.00 ft Conservative assuming Main Roof Diaph. depth only

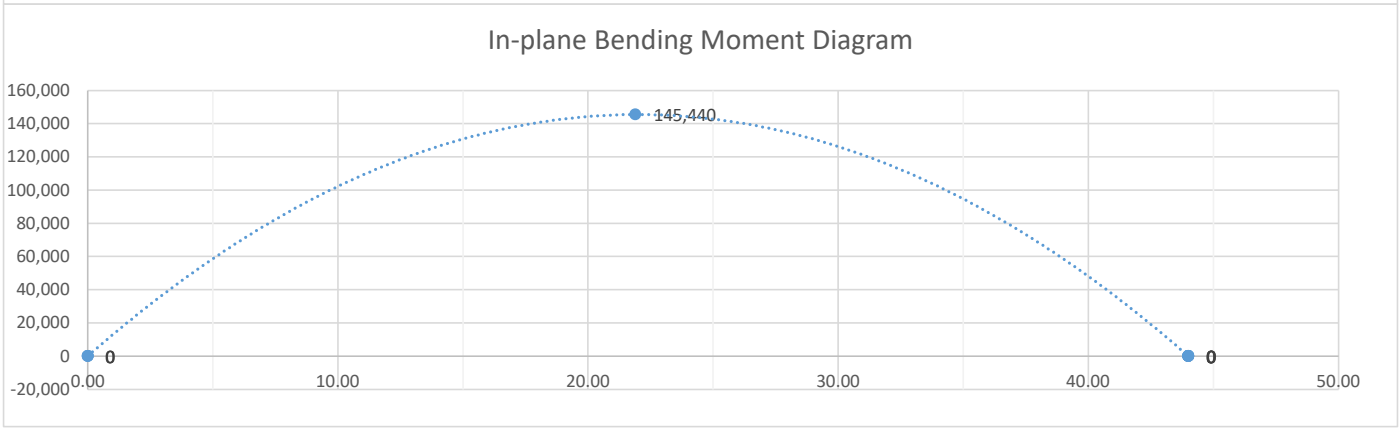
Max. Chord Force T or C = M / d 2,048 lbs

Use CS14 Strap with 24" MIN end length ea side of break at any point where break in ledger occurs along length of wall line.

ASD Cap. 2,490 lbs > 2,048 lbs OK

See Collector Line [F]





Assuming Line 3 and 7 omitted,

Line 4: 5,956 lbs. Diaph. shear
Line 9: 6,869 lbs. Diaph. shear
w average: 5,956 lbs + 6,869 lbs = 12,825 lbs / 66.5 ft = 193 plf

M = 193 plf x 66.5 ft x 66.5 ft / 8 M = 106,690 ft-lbs

T,C = 106,690 ft-lbs / 30.25 ft = 3,527 lbs. x 1.25 Diaph./1.3 Redund = 3,391 lbs
3,391 lbs / 225 lbs [0.162"x3.5" Nail] = 15 nails use Splice Type C (20 Nails)

At Lines D/7: Chord:
T,C = 3,391 lbs.
Use CMSTC16 [ASD 4,690]
Collector Line D/E/G requires a max, collector force of 2251 lbs. < 3,391 lbs OK

TJI Joist Framed Diaphragm Capacity w/ Nailing Pattern ESR-1153

Table 2-Maximum Allowable Unit Shear Design (ASD) (Pounds per foot) for Sheathed Wood-Framed Diaphragms with Trus Joist® TJI® joists for Seismic Loading [a, b, c, d]

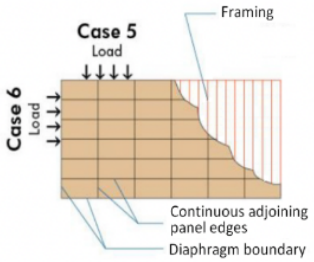
Sheathing Grade	Common Nail SI[e] Length (in.) x Shank diameter (in.) x Head diameter (in.)	Minimum Nominal Panel Thickness (in.)	Minimum Nominal Width of Nailed Face at Adjoining Panel Edges and Boundaries (in.)	TJI® Joist Series with Equivalent Nominal Framing Width	Blocked Diaphragms[f]				Unblocked Diaphragms[f]		
					Nailed 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)				Nails Spaced 6 in. max. at supported edges		
					6	4	2-1/2[g]	2[g]	Case 1	Case 3	All other configurations (Cases 2, 4, 5 & 6)
					Nailed spacing (in.) at other panel edges (Cases 1, 2, 3, & 4)						
					6	6	4	3			
Structural I	8d (2-1/2 x 0.131 x 0.281)	3/8	2	110, 210	270	360	***425***	-	240	180	***155[h]***
			3	230	300	400	***480***	-	265	200	***170[h]***
			3	360, 560, 560D	300	400	600	675	265	200	200
	10d (3 x 0.148 x 0.312)	15/32	2	110, 210	320	425	***425***	-	285	215	***185[h]***
			3	230	360	480	***480***	-	320	240	***205[h]***
			3	360, 560, 560D	360	480	720	-	320	240	240
Sheathing, single floor and other grades covered in DOC PS 1 and PS 2	6d (2 x 0.113 x 0.266)	3/8	2	110, 210	185	250	375	-	165	125	***105[h]***
			3	230	210	280	420	-	185	140	***120[h]***
			3	360, 560, 560D	210	280	420	475	185	140	140
	8d (2-1/2 x 0.131 x 0.281)	3/8	2	110, 210	240	320	***425***	-	215	160	***135[h]***
			3	230	270	360	480	-	240	180	***155[h]***
			3	360, 560, 560D	270	360	540	610	240	180	180
		7/16	2	110, 210	255	340	***425***	-	230	170	***145[h]***
			3	230	285	380	***480***	-	255	190	***160[h]***
			3	360, 560, 560D	285	380	570	645	255	190	190
		15/32	2	110, 210	270	360	***425***	-	240	180	***155[h]***
			3	230	300	400	***480***	-	265	200	***170[h]***
			3	360, 560, 560D	300	400	600	675	265	200	200
	10d (3 x 0.148 x 0.312)	15/32	2	110, 210	290	385	***425***	-	255	190	***160[h]***
			3	230	325	430	***480***	-	290	215	***185[h]***
			3	360, 560, 560D	325	430	650	-	290	215	215
		19/32	2	110, 210	320	425	***425***	-	285	215	***185[h]***
			3	230	360	480	***480***	-	320	240	***205[h]***
			3	360, 560, 560D	360	480	720	-	320	240	240

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

"-" = not permitted

Italicized Bold values indicate values that differ from SDPWS Tables 4.2A and Tables 4.2C.

ESR-1153 TJI Joist Framed Diaphragms: Unblocked
Use Nailing Patterns as follows: Un-Blocked Diaphragm
19/32 min. APA Rated Sheathing, Roof Joists are TJI-210. ,
Diaphragm Boundaries [10dx3" nails at 6" o.c.]
Supported Panel Edges [10dx3" nails at 6" o.c.]
Field Nails [10dx3" nails at 12" o.c.]
Allowable diaphragm shear
Case-5 & 6 is = **185 plf**



Notes for Table 2 – see page 5

(a) For wind load applications, the values in the table below may be increased by a factor of 1.4.
(b) For other than short-term wind and seismic loads, adjustment of the allowable loads for load duration shall be in accordance with the National Design Specification® (NDS®) for Wood Construction.
(c) For Apparent Shear Stiffness Value, Ga, for floor diaphragm deflection, refer to the corresponding table value in Table 4.2A and 4.2C of the Special Design Provisions for Wind and Seismic (SDPWS).
(d) The minimum nail bearing length specified in SDPWS Tables 4.2A and 4.2C does not apply provided the fastener penetrates through the flange.One row of nails is permitted along each sheathing panel end and edge. When nail spacing is less than 6 inches on center, adjacent nails within row must be offset (staggered).
(e) Allowable shear values are applicable for carbon steel smooth shank nails of the specified type and size.
(f) Refer to NDS for definition of Case 1 through 6.
(g) Only suitable for applications where Douglas-fir/Larch or equivalent sawn lumber, glulam or structural composite lumber products that allow $2^1/_2$ inches or 2 inches nail spacings are used for boundary framing, joist framing, or blocking members that receive nail spacings closer than 3 inches.
(h) Value may be multiplied by a factor of 1.18 where sub-floor adhesives, which have been qualified as Class $^1/_8$-inch, Type P/O per ASTM D3498-19, are used in combination with mechanical fasteners attachment. Continuous special inspection is not required for this adhesive application.

Coiled Straps (cont.)

These products are available with additional corrosion protection. For more information, see p. 16.

SS For stainless-steel fasteners, see p. 23.

SD Many of these products are approved for installation with Strong-Drive® SD Connector screws. See pp. 362–366 for more information.

Model No.	Total L (ft.)	Ga.	DF/SP		SPF/HF		Allowable Tension Loads (160)	Code Ref.
			Fasteners (in.)	End Length (in.)	Fasteners (in.)	End Length (in.)		
CMST12	40	12	(74) 0.162 x 2½	33	(84) 0.162 x 2½	38	9,215	IBC®, FL, LA
			(86) 0.148 x 2½	39	(98) 0.148 x 2½	44	9,215	
CMST14	52½	14	(56) 0.162 x 2½	26	(66) 0.162 x 2½	30	6,475	
			(66) 0.148 x 2½	30	(76) 0.148 x 2½	34	6,475	
CMSTC16	54	16	(50) 0.148 x 3¼	20	(50) 0.148 x 3¼	25	4,690	
CS14	100	14	(26) 0.148 x 2½	15	(30) 0.148 x 2½	16	2,490	
			(30) 0.131 x 2½	16	(36) 0.131 x 2½	19	2,490	
SS CS16	150	16	(20) 0.148 x 2½	11	(22) 0.148 x 2½	13	1,705	
			(22) 0.131 x 2½	13	(26) 0.131 x 2½	15	1,705	
CS18	200	18	(16) 0.148 x 2½	9	(18) 0.148 x 2½	11	1,370	
			(18) 0.131 x 2½	11	(22) 0.131 x 2½	12	1,370	
CS20	250	20	(12) 0.148 x 2½	7	(14) 0.148 x 2½	9	1,030	
			(14) 0.131 x 2½	9	(16) 0.131 x 2½	9	1,030	
CS22	300	22	(10) 0.148 x 2½	6	(12) 0.148 x 2½	7	845	
			(12) 0.131 x 2½	7	(14) 0.131 x 2½	8	845	

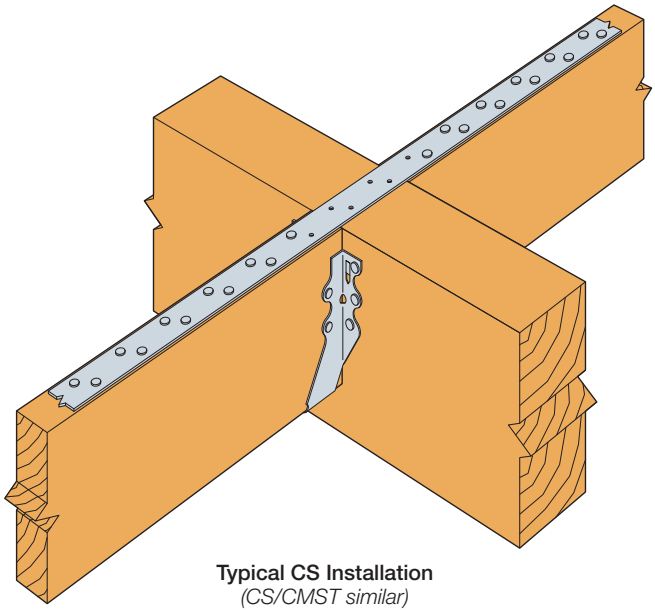
1. See pp. 276–277 for Straps and Ties General Notes.
2. Calculate the connector value for a reduced number of nails as follows:

$$\text{Allowable Load} = \frac{\text{No. of Nails Used}}{\text{No. of Nails in Table}} \times \text{Table Load}$$

Example: CMSTC16 in DF/SP with 40 nails total.
(Half of the nails in each member being connected)

$$\text{Allowable Load} = \frac{\text{40 Nails (Used)}}{\text{50 Nails (Table)}} \times \text{4,690 lb.} = \text{3,752 lb.}$$

3. See p. 285 for alternate nailing and lap splice information.
4. **Fasteners:** Nail dimensions are listed diameter by length. See pp. 23–24 for fastener information.



Typical CS Installation
(CS/CMST similar)

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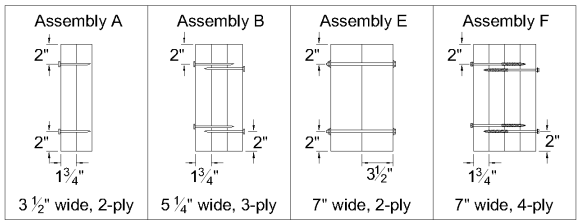
E. Framing Design

Commercial Bldg. - Roof Joists			
Member Name	**Results (Max UTIL %)**	**Current Solution**	**Comments**
Entrance Roof Joist w/ Typical Snow with Drift	Passed (80% M)	1 piece(s) 14" TJI® 210 @ 24" OC	Cantilever Reinforcement (PB1) Required
Entrance Roof Joist w/ Snow min. Balanced	Passed (80% M)	1 piece(s) 14" TJI® 210 @ 24" OC	Cantilever Reinforcement (PB1) Required
at Entrance @ 24" o.c. W/ Dead min. Max. Wind Uplift Zone 2	Passed (100% ΔL)	1 piece(s) 14" TJI® 210	Cantilever Reinforcement (PB1) Required
Spaced @ 24" o.c. [Snow Flat w/ Drift w/ Wind Pressure]	Passed (96% ΔT)	1 piece(s) 14" TJI® 210	Web Stiffeners Required
Spaced @ 24" o.c. [Snow Balanced min. w/ Wind Pressure]	Passed (96% ΔT)	1 piece(s) 14" TJI® 210	Web Stiffeners Required
Spaced @ 24" o.c. [Dead min. w/ Member Wind Uplift Z2]	Passed (45% ΔT)	1 piece(s) 14" TJI® 210	Web Stiffeners Required
Spaced @ 24" o.c. [Dead min. w/ Member Wind Uplift Z1/Z2]	Passed (36% M)	1 piece(s) 14" TJI® 210	Web Stiffeners Required
Spaced @ 24" o.c. [Dead min. w/ Member Wind Uplift Z1'/Z1/Z2]	Passed (36% M)	1 piece(s) 14" TJI® 210	Web Stiffeners Required
Spaced @ 24" o.c. [Dead min. w/ Tie Wind Uplift Z2]	Passed (51% ΔT)	1 piece(s) 14" TJI® 210	Web Stiffeners Required
Spaced @ 24" o.c. [Dead min. w/ Tie Wind Uplift Z1/Z2]	Passed (38% ΔT)	1 piece(s) 14" TJI® 210	Web Stiffeners Required
Spaced @ 24" o.c. [Dead min. w/ Tie Wind Uplift Z1'/Z1/Z2]	Passed (36% M)	1 piece(s) 14" TJI® 210	Web Stiffeners Required
Commercial Bldg. - Roof Beams			
Member Name	**Results (Max UTIL %)**	**Current Solution**	**Comments**
Continuous 1-Ply - Snow balanced min. & Wind pressure	Passed (100% R)	1 piece(s) 3 1/2" x 16" 2.2E Parallam® PSL	
Continuous 2-Ply - Snow balanced min. & Wind pressure	Passed (100% R)	2 piece(s) 3 1/2" x 16" 2.2E Parallam® PSL	
Continuous 1-Ply - Dead min. & Wind Uplift	Passed (72% R)	1 piece(s) 3 1/2" x 16" 2.2E Parallam® PSL	
Continuous 2-Ply - Dead min. & Wind Uplift	Passed (38% R)	2 piece(s) 3 1/2" x 16" 2.2E Parallam® PSL	

Side-Loaded Beam

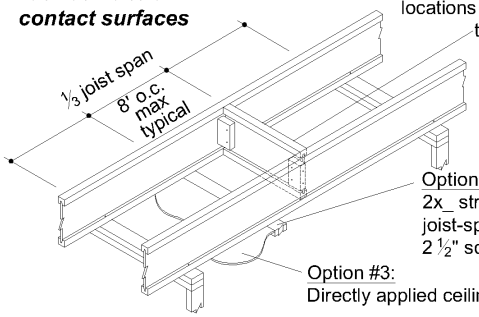
Fastener Pattern



- Washers required for ½" A307 through bolts. Bolt holes to be 9/16" maximum.
- For uniform load connections with fasteners required on both faces, stagger fasteners on the second face so they fall halfway between fasteners in the first face.
- For point load connections with screws required on both faces, refer to screw manufacturer's guidelines for minimum spacing requirements.
- Minimum end distance for bolts and screws is 6".

L17 L18

Apply subfloor adhesive to all contact surfaces



Option #1:
TJI® joist blocking with end blocks installed at ⅓ joist-span locations (8' max typical) using two 8d (0.113" x 2 ½") nails or 2 ½" screws

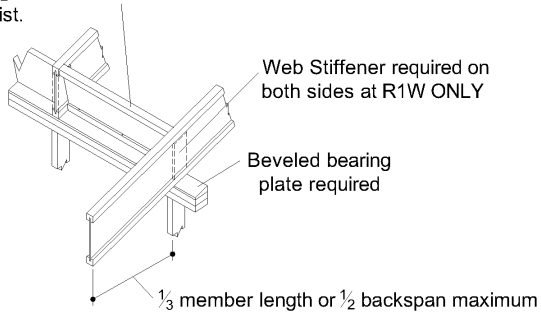
Option #2:
2x_ strapping installed at ⅓ joist-span locations using two 2 ½" screws per joist, typical

Option #3:
Directly applied ceiling

PB1

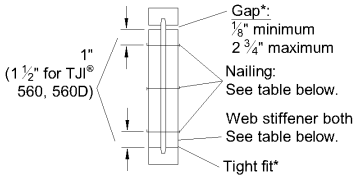
When specified on the layout, one of the bracing options above is required

Shear blocking:
1 1/8" TJ® Rim Board (with depths ≤ 16"),
1 ¼"or 1 ½" TimberStrand® LSL
or TJI® joist.



R1 R1W

WEB STIFFENER ATTACHMENT



* With point load from above, and no support below, install web stiffener tight to top flange (gap at bottom flange)

TJI® Joist Series	Depth (in.)	Minimum Web Stiffener Size	Nailing Requirements		
			Type	Number Nails	
				End	Intermediate
110	All	5/8" x 2 5/16"(1)	8d (0.113" x 2 ½")	3	3
210	All	¾" x 2 5/16"(1)			
230 & 360	All	7/8" x 2 5/16"(1)			
560	All	2x4 (2)	16d (0.135" x 3 ½")		
560D	18"	2x4 (2)	16d (0.135" x 3 ½")	4	4
	20"			5	5
	22"(3)			6	11
	24"(3)			6	13

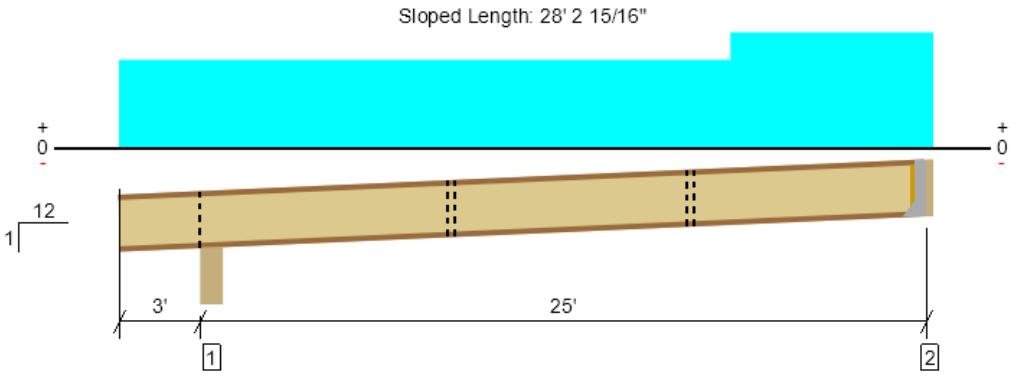
(1) PS1 or PS2 sheathing, face grain vertical
(2) Construction grade or better
(3) Web stiffeners are always required for 22" and 24" TJI® 560D Joists

W

E.1 Framing Design (Roof Joists)

Commercial Bldg. - Roof Joists, Entrance Roof Joist w/ Typical Snow with Drift

1 piece(s) 14" TJI® 210 @ 24" OC



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

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	736 @ 28'	1256 (1.75")	Passed (59%)	1.25	1.0 D + 1.0 Lr (Alt Spans)
Shear (lbs)	736 @ 28'	2431	Passed (30%)	1.25	1.0 D + 1.0 Lr (Alt Spans)
Moment (Ft-lbs)	4503 @ 15' 9 1/16"	5613	Passed (80%)	1.25	1.0 D + 1.0 Lr (Alt Spans)
Live Load Defl. (in)	0.770 @ 15' 7 13/16"	1.243	Passed (L/387)	--	1.0 D + 1.0 Lr (Alt Spans)
Total Load Defl. (in)	1.149 @ 15' 7 15/16"	1.657	Passed (L/260)	--	1.0 D + 1.0 Lr (Alt Spans)

Member Length : 28' 2 5/16"
System : Roof
Member Type : Joist
Building Use : Commercial
Building Code : IBC 2024
Design Methodology : ASD
Member Pitch : 1/12

- Deflection criteria: LL (L/240) and TL (L/180).
- Overhang deflection criteria: LL (2L/240) and TL (2L/180).
- Upward deflection on left cantilever exceeds overhang deflection criteria.
- Allowed moment does not reflect the adjustment for the beam stability factor.
- Upward deflection on left cantilever exceeds 0.4".
- Permanent bracing at third points in the back span or a direct applied ceiling over the entire back span length is required at the left span of the member. See literature detail (PB1) For clarification.

Supports	Bearing Length			Loads to Supports (lbs)				Accessories	Details
	Total	Available	Required	Dead	Roof Live	Snow	Factored		
1 - Beveled Plate - DF	5.50"	5.50"	3.50"	318	633	591	951	Blocking	R1
2 - Hanger on 14" LSL Ledger	1.75"	Hanger[1]	1.75" / - [2]	247	497	620	744	See note [1]	W

- Blocking Panels are assumed to carry no loads applied directly above them and the full load is applied to the member being designed.
- At hanger supports, the Total Bearing dimension is equal to the width of the material that is supporting the hanger
- [1] See Connector grid below for additional information and/or requirements.
- [2] Required Bearing Length / Required Bearing Length with Web Stiffeners

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	3' 8" o/c	
Bottom Edge (Lu)	8' 8" o/c	

- TJI joists are only analyzed using Maximum Allowable bracing solutions.
- Maximum allowable bracing intervals based on applied load.
- Dimensions for lateral bracing intervals are measured along the length of the member for sloped conditions.

Connector: Simpson Strong-Tie						
Support	Model	Seat Length	Top Fasteners	Face Fasteners	Member Fasteners	Accessories
2 - Face Mount Hanger	HU2.1/9X SLD4	2.50"	N/A	14-10dx1.5	6-10dx1.5	Web Stiffeners

- Refer to manufacturer notes and instructions for proper installation and use of all connectors.

Vertical Loads	Location	Spacing	Dead (0.90)	Roof Live (1.25)	Snow (1.15)	Comments
1 - Uniform (PSF)	0 to 28' 1 3/4"	24"	10.0	20.0	--	Roof Loading above Unconditioned spaces.
2 - Uniform (PSF)	0 to 28' 1 3/4"	24"	--	--	17.8	Snow Load Balanced
3 - Uniform (PSF)	21' 3" to 28' 1 3/4"	24"	--	--	14.9	Snow Load (Drift)

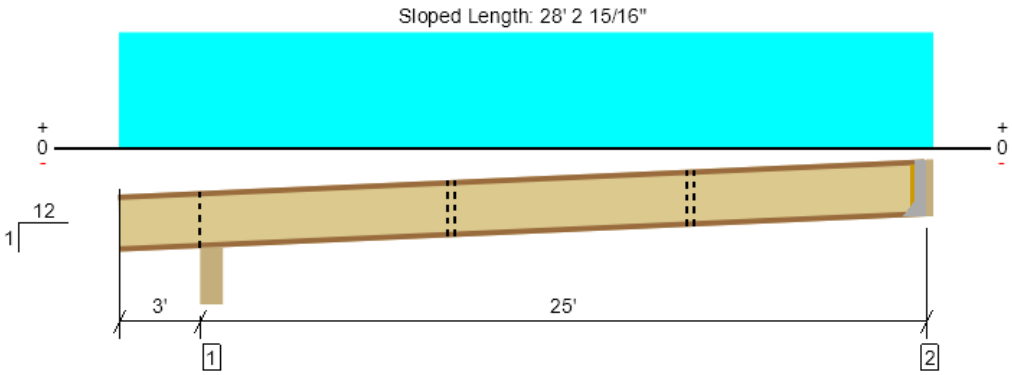
ForteWEB Software Operator	Job Notes
Mohammad Aljazzar California Building Engineers Inc. (510) 674-5270 mohammad@cbestructural.com	

Weyerhaeuser

5/21/2025 6:30:30 PM UTC
ForteWEB v3.9, Engine: V8.4.3.94, Data: V8.1.7.3
File Name: C & S Restaurant 29 Palms Hwy Yucca Valley

Commercial Bldg. - Roof Joists, Entrance Roof Joist w/ Snow min. Balanced

1 piece(s) 14" TJI® 210 @ 24" OC



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

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	736 @ 28'	1256 (1.75")	Passed (59%)	1.25	1.0 D + 1.0 Lr (Alt Spans)
Shear (lbs)	736 @ 28'	2431	Passed (30%)	1.25	1.0 D + 1.0 Lr (Alt Spans)
Moment (Ft-lbs)	4503 @ 15' 9 1/16"	5613	Passed (80%)	1.25	1.0 D + 1.0 Lr (Alt Spans)
Live Load Defl. (in)	0.770 @ 15' 7 13/16"	1.243	Passed (L/387)	--	1.0 D + 1.0 Lr (Alt Spans)
Total Load Defl. (in)	1.149 @ 15' 7 15/16"	1.657	Passed (L/260)	--	1.0 D + 1.0 Lr (Alt Spans)

Member Length : 28' 2 5/16"
System : Roof
Member Type : Joist
Building Use : Commercial
Building Code : IBC 2024
Design Methodology : ASD
Member Pitch : 1/12

- Deflection criteria: LL (L/240) and TL (L/180).
- Overhang deflection criteria: LL (2L/240) and TL (2L/180).
- Upward deflection on left cantilever exceeds overhang deflection criteria.
- Allowed moment does not reflect the adjustment for the beam stability factor.
- Upward deflection on left cantilever exceeds 0.4".
- Permanent bracing at third points in the back span or a direct applied ceiling over the entire back span length is required at the left span of the member. See literature detail (PB1) For clarification.

Supports	Bearing Length			Loads to Supports (lbs)				Accessories	Details
	Total	Available	Required	Dead	Roof Live	Snow	Factored		
1 - Beveled Plate - DF	5.50"	5.50"	3.50"	318	633	760	951	Blocking	R1
2 - Hanger on 14" LSL Ledger	1.75"	Hanger[1]	1.75" / - [2]	247	497	596	744	See note [1]	W

- Blocking Panels are assumed to carry no loads applied directly above them and the full load is applied to the member being designed.
- At hanger supports, the Total Bearing dimension is equal to the width of the material that is supporting the hanger
- [1] See Connector grid below for additional information and/or requirements.
- [2] Required Bearing Length / Required Bearing Length with Web Stiffeners

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	3' 8" o/c	
Bottom Edge (Lu)	8' 8" o/c	

- TJI joists are only analyzed using Maximum Allowable bracing solutions.
- Maximum allowable bracing intervals based on applied load.
- Dimensions for lateral bracing intervals are measured along the length of the member for sloped conditions.

Connector: Simpson Strong-Tie						
Support	Model	Seat Length	Top Fasteners	Face Fasteners	Member Fasteners	Accessories
2 - Face Mount Hanger	HU2.1/9X SLD4	2.50"	N/A	14-10dx1.5	6-10dx1.5	Web Stiffeners

- Refer to manufacturer notes and instructions for proper installation and use of all connectors.

Vertical Loads	Location	Spacing	Dead (0.90)	Roof Live (1.25)	Snow (1.15)	Comments
1 - Uniform (PSF)	0 to 28' 1 3/4"	24"	10.0	20.0	--	Roof Loading above Unconditioned spaces.
2 - Uniform (PSF)	0 to 28' 1 3/4"	24"	--	--	24.0	Snow Load (min.) Balanced

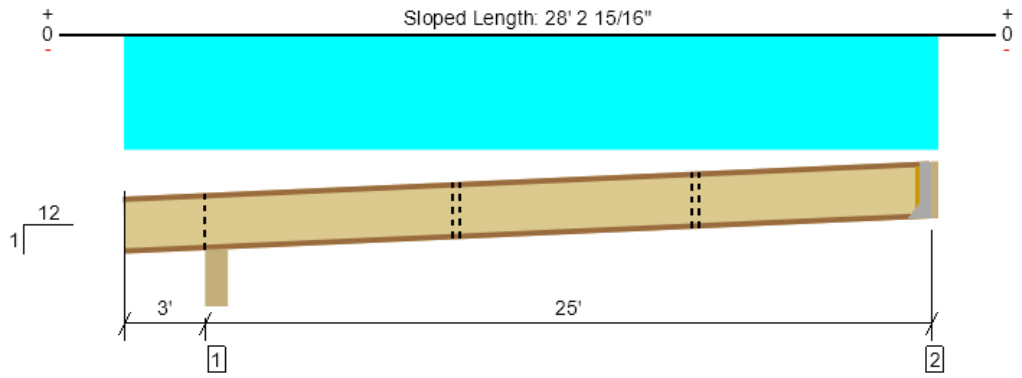
ForteWEB Software Operator	Job Notes
Mohammad Aljazzar California Building Engineers Inc. (510) 674-5270 mohammad@cbestructural.com	

Weyerhaeuser

5/21/2025 6:30:30 PM UTC
ForteWEB v3.9, Engine: V8.4.3.94, Data: V8.1.7.3
File Name: C & S Restaurant 29 Palms Hwy Yucca Valley

Commercial Bldg. - Roof Joists, at Entrance @ 24" o.c. W/ Dead min. Max. Wind Uplift Zone 2

1 piece(s) 14" TJI® 210



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

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	185 @ 28'	905 (1.75")	Passed (20%)	0.90	1.0 D (All Spans)
Shear (lbs)	584 @ 28'	3112	Passed (19%)	1.60	0.6 D + 0.6 W (Alt Spans)
Moment (Ft-lbs)	-3645 @ 15' 6 7/8"	7184	Passed (51%)	1.60	0.6 D + 0.6 W (Alt Spans)
Live Load Defl. (in)	0.434 @ 0	0.432	Passed (2L/180)	--	1.0 D + 0.6 W (Alt Spans)
Total Load Defl. (in)	0.370 @ 0	0.432	Passed (2L/210)	--	0.6 D + 0.6 W (Alt Spans)

Member Length : 28' 2 5/16"
System : Roof
Member Type : Flush Beam
Building Use : Commercial
Building Code : IBC 2024
Design Methodology : ASD
Member Pitch : 1/12

- Deflection criteria: LL (L/180) and TL (L/180).
- Overhang deflection criteria: LL (2L/240) and TL (2L/180).
- Allowed moment does not reflect the adjustment for the beam stability factor.
- -741 lbs uplift at support located at 3' 2 3/4". Strapping or other restraint may be required.
- -589 lbs uplift at support located at 28'. Strapping or other restraint may be required.
- Permanent bracing at third points in the back span or a direct applied ceiling over the entire back span length is required at the left span of the member. See literature detail (PB1) For clarification.

Supports	Bearing Length			Loads to Supports (lbs)			Accessories	Details
	Total	Available	Required	Dead	Wind	Factored		
1 - Beveled Plate - DF	5.50"	5.50"	3.50"	240	-1475	240/-741	Blocking	R1
2 - Hanger on 14" LSL Ledger	1.75"	Hanger[1]	1.75" / - [2]	186	6/-1168	190/-589	See note [1]	W

- Blocking Panels are assumed to carry no loads applied directly above them and the full load is applied to the member being designed.
- At hanger supports, the Total Bearing dimension is equal to the width of the material that is supporting the hanger
- [1] See Connector grid below for additional information and/or requirements.
- [2] Required Bearing Length / Required Bearing Length with Web Stiffeners

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	7' 3" o/c	
Bottom Edge (Lu)	4' 3" o/c	

- TJI joists are only analyzed using Maximum Allowable bracing solutions.
- Maximum allowable bracing intervals based on applied load.
- Dimensions for lateral bracing intervals are measured along the length of the member for sloped conditions.

Connector: Simpson Strong-Tie						
Support	Model	Seat Length	Top Fasteners	Face Fasteners	Member Fasteners	Accessories
2 - Face Mount Hanger	HU2.1/9X SLD4	2.50"	N/A	14-10dx1.5	6-10dx1.5	Web Stiffeners

- Refer to manufacturer notes and instructions for proper installation and use of all connectors.

ForteWEB Software Operator	Job Notes
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Vertical Loads	Location	Tributary Width	Dead (0.90)	Wind (1.60)	Comments
0 - Self Weight (PLF)	0 to 28'	N/A	3.1	--	
1 - Uniform (PSF)	0 to 28' 1 3/4"	2'	6.0	--	Roof Loading above Unconditioned spaces. (Reduced Dead Load, No MEP or Misc.) conservatively for uplift calc.
2 - Uniform (PSF)	0 to 28' 1 3/4"	2'	--	-46.6	Wind Uplift Zone 2 (connector) & Wind Internal Pressure

• Distributed wind loads are applied perpendicular to the slope of the member.

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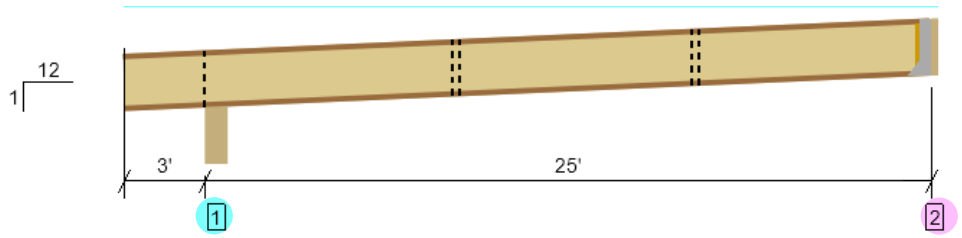
The product application, input design loads, dimensions and support information have been provided by ForteWEB Software Operator

Entrance Joist Reactions:



Commercial Bldg. - Roof Joists, Entrance Roof Joist w/ Typical Snow with Drift

Supports	Bearing Length			Loads to Supports (lbs)				Accessories	Details
	Total	Available	Required	Dead	Roof Live	Snow	Factored		
1 - Beveled Plate - DF	5.50"	5.50"	3.50"	318	633	591	951	Blocking	R1
2 - Hanger on 14" LSL Ledger	1.75"	Hanger[1]	1.75" / - [2]	247	497	620	744	See note [1]	W

Downward 744 < 2,085 (Cd 1.00) **OK**
Downward 867 < 2,350 (Cd 1.15) **OK**

Commercial Bldg. - Roof Joists, Entrance Roof Joist w/ Snow min. Balanced

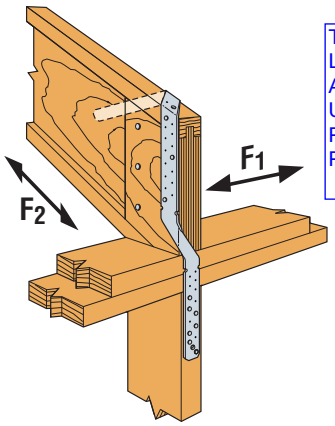
Supports	Bearing Length			Loads to Supports (lbs)				Accessories	Details
	Total	Available	Required	Dead	Roof Live	Snow	Factored		
1 - Beveled Plate - DF	5.50"	5.50"	3.50"	318	633	760	951	Blocking	R1
2 - Hanger on 14" LSL Ledger	1.75"	Hanger[1]	1.75" / - [2]	247	497	596	744	See note [1]	W

Commercial Bldg. - Roof Joists, at Entrance @ 24" o.c. W/ Dead min. Max. Wind Uplift Zone 2

Uplift (Cd=1.6) < 780 **OK**

Supports	Bearing Length			Loads to Supports (lbs)			Accessories	Details
	Total	Available	Required	Dead	Wind	Factored		
1 - Beveled Plate - DF	5.50"	5.50"	3.50"	240	-1475	240/-741	Blocking	R1
2 - Hanger on 14" LSL Ledger	1.75"	Hanger[1]	1.75" / - [2]	186	6/-1168	190/-589	See note [1]	W

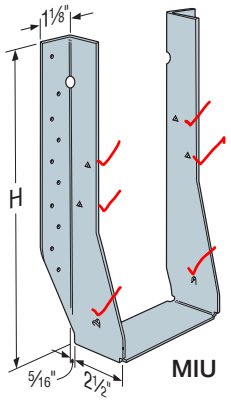
Uplift (Cd=1.6) 488 < 878 lbs **OK**
At Ledgers:
face nails from 16dx3.5 --> #10x1.5 (No reduction)
for 1 3/4" LSL ledger



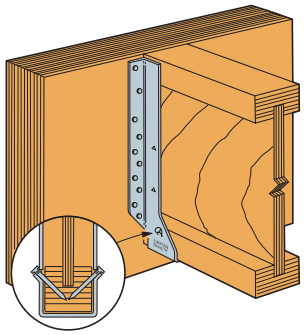
Tie at Joist tot Dbl Top Plates:
LTS20 W/ (6) [0.148] 10dx1 1/2" Nails
ASD Capacity
Uplift = 645 lbs (Cd = 1.6)
F1= 75 lbs
F2= 125 lbs

Face Mount Hanger at Ledger:
MIU2.1/11
ASD Capacity

Uplift = 975 lbs (Cd = 1.6)	x 0.9 =	878 lbs
Down = 2,880 lbs (Cd = 1.0)	x 0.9 =	2,592 lbs
Down = 3,135 lbs (Cd = 1.15)	x 0.9 =	2,822 lbs
Down = 3,135 lbs (Cd = 1.25)	x 0.9 =	2,822 lbs



MIU

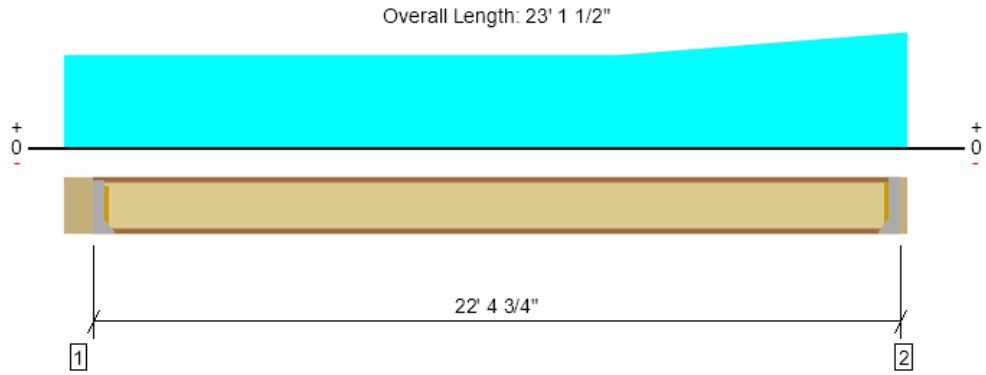

MIU with Correct PAN Installation

Options:

- These hangers cannot be modified. However, these models will normally accommodate a skew of up to 5°. For sloped joists up to ¼:12 there is no reduction; between ¼:12 and up to ½:12, tests show a 10% reduction in ultimate hanger strength. Local crushing of the bottom flange or excessive deflection may be limiting; check with joist manufacturer for specific limitations on bearing of this type.
- MIU — add four additional 0.148" x 1½" joist nails for a total uplift load of 975 lb.

Commercial Bldg. - Roof Joists, Spaced @ 24" o.c. [Snow Flat w/ Drift w/ Wind Pressure]

1 piece(s) 14" TJI® 210



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

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	841 @ 7"	1256 (1.75")	Passed (67%)	1.25	1.0 D + 1.0 Lr (All Spans)
Shear (lbs)	841 @ 7"	2431	Passed (35%)	1.25	1.0 D + 1.0 Lr (All Spans)
Moment (Ft-lbs)	4709 @ 11' 9 3/8"	5613	Passed (84%)	1.25	1.0 D + 1.0 Lr (All Spans)
Live Load Defl. (in)	0.611 @ 11' 9 3/8"	0.747	Passed (L/440)	--	1.0 D + 0.45 W + 0.75 L + 0.75 Lr (All Spans)
Total Load Defl. (in)	1.075 @ 11' 9 3/8"	1.120	Passed (L/250)	--	1.0 D + 0.45 W + 0.75 L + 0.75 Lr (All Spans)

Member Length : 22' 4 3/4"
System : Roof
Member Type : Flush Beam
Building Use : Commercial
Building Code : IBC 2024
Design Methodology : ASD
Member Pitch : 0/12

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

Supports	Bearing Length			Loads to Supports (lbs)					Accessories	Details
	Total	Available	Required	Dead	Roof Live	Snow	Wind	Factored		
1 - Hanger on 14" PSL beam	7.00"	Hanger[1]	1.75" / - [2]	412	471	436	426	957	See note [1]	W
2 - Hanger on 14" LSL Ledger	1.75"	Hanger[1]	1.75" / - [2]	398	454	530	411	923	See note [1]	W

- At hanger supports, the Total Bearing dimension is equal to the width of the material that is supporting the hanger
- [1] See Connector grid below for additional information and/or requirements.
- [2] Required Bearing Length / Required Bearing Length with Web Stiffeners

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	3' 6" o/c	
Bottom Edge (Lu)	22' 5" o/c	

- TJI joists are only analyzed using Maximum Allowable bracing solutions.
- Maximum allowable bracing intervals based on applied load.

Connector: Simpson Strong-Tie						
Support	Model	Seat Length	Top Fasteners	Face Fasteners	Member Fasteners	Accessories
1 - Face Mount Hanger	MIU2.1/11	2.50"	N/A	20-10dx1.5	2-10dx1.5	Web Stiffeners
2 - Face Mount Hanger	MIU2.1/11	2.50"	N/A	20-10dx1.5	2-10dx1.5	Web Stiffeners

- Refer to manufacturer notes and instructions for proper installation and use of all connectors.

Vertical Loads	Location	Tributary Width	Dead (0.90)	Roof Live (1.25)	Snow (1.15)	Wind (1.60)	Comments
0 - Self Weight (PLF)	7" to 22' 11 3/4"	N/A	3.1	--	--	--	
1 - Uniform (PSF)	0 to 23' 1 1/2" (Top)	2'	16.0	20.0	--	--	Roof Loading above Unconditioned spaces.
2 - Uniform (PSF)	0 to 23' 1 1/2" (Top)	2'	--	--	17.8	--	Snow Load Flat Roof
3 - Tapered (PLF)	15' to 23' 1 1/2" (Top)	N/A	--	--	0.0 to 35.2	--	Snow Load (Drift) Added 17.6 psf 8-ft
4 - Uniform (PSF)	0 to 23' 1 1/2" (Top)	2'	--	--	--	4.9	Wind External Pressure Downward All Zones
5 - Uniform (PSF)	0 to 23' 1 1/2" (Top)	2'	--	--	--	13.2	Wind Internal Pressure

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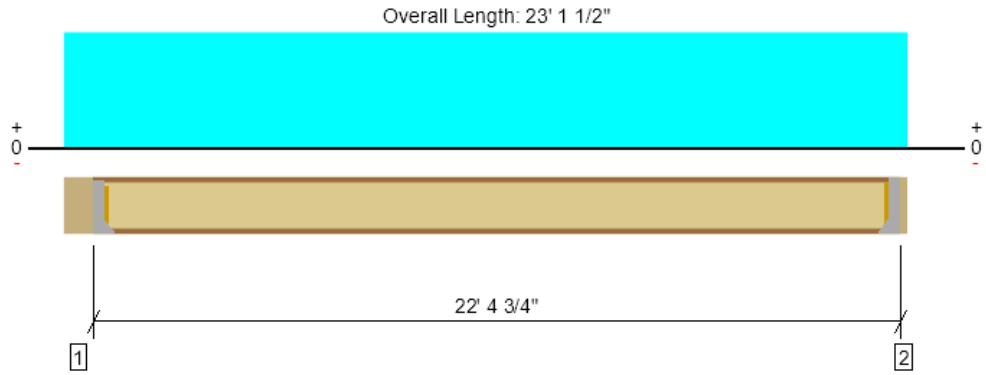
The product application, input design loads, dimensions and support information have been provided by ForteWEB Software Operator

Commercial Bldg. - Roof Joists, Spaced @ 24" o.c. [Snow Balanced min. w/ Wind Pressure]

1 piece(s) 14" TJI® 210



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

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	841 @ 7"	1256 (1.75")	Passed (67%)	1.25	1.0 D + 1.0 Lr (All Spans)
Shear (lbs)	841 @ 7"	2431	Passed (35%)	1.25	1.0 D + 1.0 Lr (All Spans)
Moment (Ft-lbs)	4709 @ 11' 9 3/8"	5613	Passed (84%)	1.25	1.0 D + 1.0 Lr (All Spans)
Live Load Defl. (in)	0.611 @ 11' 9 3/8"	0.747	Passed (L/440)	--	1.0 D + 0.45 W + 0.75 L + 0.75 Lr (All Spans)
Total Load Defl. (in)	1.075 @ 11' 9 3/8"	1.120	Passed (L/250)	--	1.0 D + 0.45 W + 0.75 L + 0.75 Lr (All Spans)

Member Length : 22' 4 3/4"
System : Roof
Member Type : Flush Beam
Building Use : Commercial
Building Code : IBC 2024
Design Methodology : ASD
Member Pitch : 0/12

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

Supports	Bearing Length			Loads to Supports (lbs)					Accessories	Details
	Total	Available	Required	Dead	Roof Live	Snow	Wind	Factored		
1 - Hanger on 14" PSL beam	7.00"	Hanger[1]	1.75" / - [2]	412	471	566	426	957	See note [1]	W
2 - Hanger on 14" LSL Ledger	1.75"	Hanger[1]	1.75" / - [2]	398	454	545	411	923	See note [1]	W

- At hanger supports, the Total Bearing dimension is equal to the width of the material that is supporting the hanger
- [1] See Connector grid below for additional information and/or requirements.
- [2] Required Bearing Length / Required Bearing Length with Web Stiffeners

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	3' 6" o/c	
Bottom Edge (Lu)	22' 5" o/c	

- TJI joists are only analyzed using Maximum Allowable bracing solutions.
- Maximum allowable bracing intervals based on applied load.

Connector: Simpson Strong-Tie						
Support	Model	Seat Length	Top Fasteners	Face Fasteners	Member Fasteners	Accessories
1 - Face Mount Hanger	MIU2.1/11	2.50"	N/A	20-10dx1.5	2-10dx1.5	Web Stiffeners
2 - Face Mount Hanger	MIU2.1/11	2.50"	N/A	20-10dx1.5	2-10dx1.5	Web Stiffeners

- Refer to manufacturer notes and instructions for proper installation and use of all connectors.

Vertical Loads	Location	Tributary Width	Dead (0.90)	Roof Live (1.25)	Snow (1.15)	Wind (1.60)	Comments
0 - Self Weight (PLF)	7" to 22' 11 3/4"	N/A	3.1	--	--	--	
1 - Uniform (PSF)	0 to 23' 1 1/2" (Top)	2'	16.0	20.0	--	--	Roof Loading above Unconditioned spaces.
2 - Uniform (PSF)	0 to 23' 1 1/2" (Top)	2'	--	--	24.0	--	Snow Load (min.) Balanced
3 - Uniform (PSF)	0 to 23' 1 1/2" (Top)	2'	--	--	--	4.9	Wind External Pressure Downward All Zones
4 - Uniform (PSF)	0 to 23' 1 1/2" (Top)	2'	--	--	--	13.2	Wind Internal Pressure

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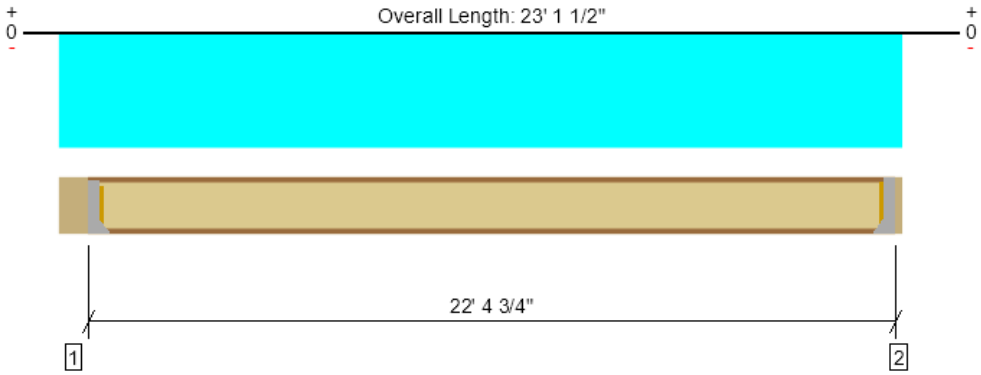
The product application, input design loads, dimensions and support information have been provided by ForteWEB Software Operator

Commercial Bldg. - Roof Joists, Spaced @ 24" o.c. [Dead min. w/ Member Wind Uplift Z2]

1 piece(s) 14" TJI® 210



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

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	259 @ 7"	905 (1.75")	Passed (29%)	0.90	1.0 D (All Spans)
Shear (lbs)	259 @ 7"	1751	Passed (15%)	0.90	1.0 D (All Spans)
Moment (Ft-lbs)	1449 @ 11' 9 3/8"	4041	Passed (36%)	0.90	1.0 D (All Spans)
Live Load Defl. (in)	0.000 @ 7"	0.747	Passed (L/999+)	--	1.0 D (All Spans)
Total Load Defl. (in)	-0.508 @ 11' 9 3/8"	1.120	Passed (L/529)	--	0.6 D + 0.6 W (All Spans)

Member Length : 22' 4 3/4"
System : Roof
Member Type : Flush Beam
Building Use : Commercial
Building Code : IBC 2024
Design Methodology : ASD
Member Pitch : 0/12

- Deflection criteria: LL (L/360) and TL (L/240).
- Allowed moment does not reflect the adjustment for the beam stability factor.
- -454 lbs uplift at support located at 7". Strapping or other restraint may be required.
- -437 lbs uplift at support located at 22' 11 3/4". Strapping or other restraint may be required.

Supports	Bearing Length			Loads to Supports (lbs)			Accessories	Details
	Total	Available	Required	Dead	Wind	Factored		
1 - Hanger on 14" PSL beam	7.00"	Hanger[1]	1.75" / - [2]	270	-1028	270/-454	See note [1]	W
2 - Hanger on 14" LSL Ledger	1.75"	Hanger[1]	1.75" / - [2]	262	-990	262/-437	See note [1]	W

- At hanger supports, the Total Bearing dimension is equal to the width of the material that is supporting the hanger
- [1] See Connector grid below for additional information and/or requirements.
- [2] Required Bearing Length / Required Bearing Length with Web Stiffeners

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	6' 9" o/c	
Bottom Edge (Lu)	5' 3" o/c	

•TJI joists are only analyzed using Maximum Allowable bracing solutions.
•Maximum allowable bracing intervals based on applied load.

Connector: Simpson Strong-Tie						
Support	Model	Seat Length	Top Fasteners	Face Fasteners	Member Fasteners	Accessories
1 - Face Mount Hanger	MIU2.1/11	2.50"	N/A	20-10dx1.5	6-10dx1.5	Web Stiffeners
2 - Face Mount Hanger	MIU2.1/11	2.50"	N/A	20-10dx1.5	6-10dx1.5	Web Stiffeners

- Refer to manufacturer notes and instructions for proper installation and use of all connectors.

ForteWEB Software Operator	Job Notes
Mohammad Aljazzar California Building Engineers Inc. (510) 674-5270 mohammad@cbestructural.com	

Weyerhaeuser

Vertical Loads	Location	Tributary Width	Dead (0.90)	Wind (1.60)	Comments
0 - Self Weight (PLF)	7" to 22' 11 3/4"	N/A	3.1	--	
1 - Uniform (PSF)	0 to 23' 1 1/2" (Top)	2'	10.0	--	Roof Loading above Unconditioned spaces. (Reduced Dead Load, No MEP or Misc.) conservatively for uplift calc.
2 - Uniform (PSF)	0 to 23' 1 1/2" (Top)	2'	--	-39.3	Wind Uplift Zone 2 (Member Design)
3 - Uniform (PSF)	0 to 23' 1 1/2" (Top)	2'	--	-4.3	Wind Internal Pressure

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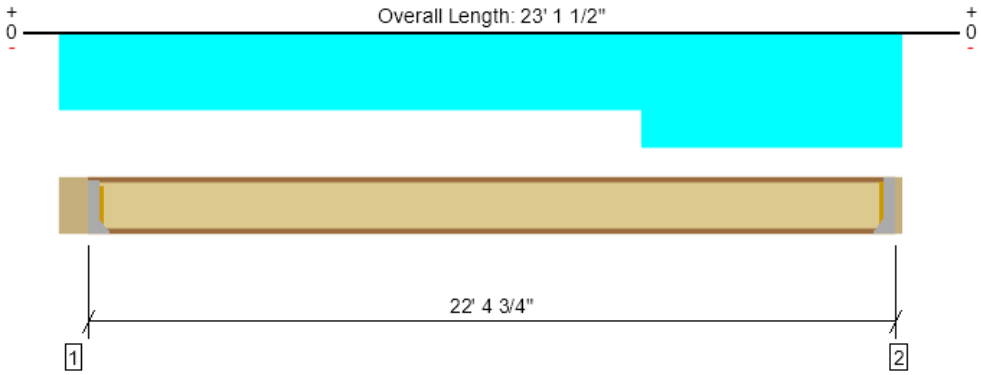
The product application, input design loads, dimensions and support information have been provided by ForteWEB Software Operator

Commercial Bldg. - Roof Joists, Spaced @ 24" o.c. [Dead min. w/ Member Wind Uplift Z1/Z2]

1 piece(s) 14" TJI® 210



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

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	259 @ 7"	905 (1.75")	Passed (29%)	0.90	1.0 D (All Spans)
Shear (lbs)	259 @ 7"	1751	Passed (15%)	0.90	1.0 D (All Spans)
Moment (Ft-lbs)	1449 @ 11' 9 3/8"	4041	Passed (36%)	0.90	1.0 D (All Spans)
Live Load Defl. (in)	0.000 @ 7"	0.747	Passed (L/999+)	--	1.0 D (All Spans)
Total Load Defl. (in)	-0.375 @ 11' 11 1/16"	1.120	Passed (L/717)	--	0.6 D + 0.6 W (All Spans)

Member Length : 22' 4 3/4"
System : Roof
Member Type : Flush Beam
Building Use : Commercial
Building Code : IBC 2024
Design Methodology : ASD
Member Pitch : 0/12

- Deflection criteria: LL (L/360) and TL (L/240).
- Allowed moment does not reflect the adjustment for the beam stability factor.
- -315 lbs uplift at support located at 7". Strapping or other restraint may be required.
- -369 lbs uplift at support located at 22' 11 3/4". Strapping or other restraint may be required.

Supports	Bearing Length			Loads to Supports (lbs)			Accessories	Details
	Total	Available	Required	Dead	Wind	Factored		
1 - Hanger on 14" PSL beam	7.00"	Hanger[1]	1.75" / - [2]	270	-796	270/-315	See note [1]	W
2 - Hanger on 14" LSL Ledger	1.75"	Hanger[1]	1.75" / - [2]	262	-876	262/-369	See note [1]	W

- At hanger supports, the Total Bearing dimension is equal to the width of the material that is supporting the hanger
- [1] See Connector grid below for additional information and/or requirements.
- [2] Required Bearing Length / Required Bearing Length with Web Stiffeners

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	6' 9" o/c	
Bottom Edge (Lu)	6' 2" o/c	

•TJI joists are only analyzed using Maximum Allowable bracing solutions.
•Maximum allowable bracing intervals based on applied load.

Connector: Simpson Strong-Tie						
Support	Model	Seat Length	Top Fasteners	Face Fasteners	Member Fasteners	Accessories
1 - Face Mount Hanger	MIU2.1/11	2.50"	N/A	20-10dx1.5	6-10dx1.5	Web Stiffeners
2 - Face Mount Hanger	MIU2.1/11	2.50"	N/A	20-10dx1.5	6-10dx1.5	Web Stiffeners

- Refer to manufacturer notes and instructions for proper installation and use of all connectors.

Vertical Loads	Location	Tributary Width	Dead (0.90)	Wind (1.60)	Comments
0 - Self Weight (PLF)	7" to 22' 11 3/4"	N/A	3.1	--	
1 - Uniform (PSF)	0 to 23' 1 1/2" (Top)	2'	10.0	--	Roof Loading above Unconditioned spaces. (Reduced Dead Load, No MEP or Misc.) conservatively for uplift calc.
2 - Uniform (PSF)	0 to 15' 9 5/8" (Top)	2'	--	-28.4	Wind Uplift Zone 1 (Member Design)
3 - Uniform (PSF)	15' 9 5/8" to 23' 1 1/2" (Top)	2'	--	-39.3	Wind Uplift Zone 2 (Member Design)
4 - Uniform (PSF)	0 to 23' 1 1/2" (Top)	2'	--	-4.3	Wind Internal Pressure

Weyerhaeuser Notes

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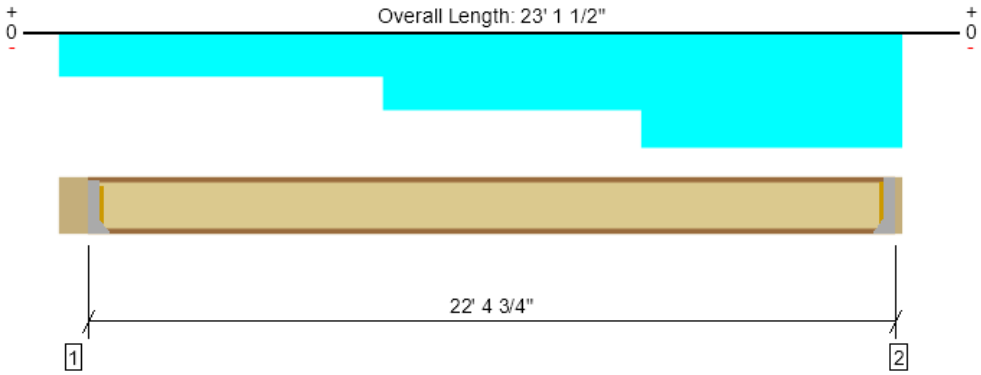
The product application, input design loads, dimensions and support information have been provided by ForteWEB Software Operator

Commercial Bldg. - Roof Joists, Spaced @ 24" o.c. [Dead min. w/ Member Wind Uplift Z1'/Z1/Z2]

1 piece(s) 14" TJI® 210



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

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	259 @ 7"	905 (1.75")	Passed (29%)	0.90	1.0 D (All Spans)
Shear (lbs)	259 @ 7"	1751	Passed (15%)	0.90	1.0 D (All Spans)
Moment (Ft-lbs)	1449 @ 11' 9 3/8"	4041	Passed (36%)	0.90	1.0 D (All Spans)
Live Load Defl. (in)	0.000 @ 7"	0.747	Passed (L/999+)	--	1.0 D (All Spans)
Total Load Defl. (in)	-0.332 @ 12' 1 1/4"	1.120	Passed (L/810)	--	0.6 D + 0.6 W (All Spans)

Member Length : 22' 4 3/4"
System : Roof
Member Type : Flush Beam
Building Use : Commercial
Building Code : IBC 2024
Design Methodology : ASD
Member Pitch : 0/12

- Deflection criteria: LL (L/360) and TL (L/240).
- Allowed moment does not reflect the adjustment for the beam stability factor.
- -231 lbs uplift at support located at 7". Strapping or other restraint may be required.
- -352 lbs uplift at support located at 22' 11 3/4". Strapping or other restraint may be required.

Supports	Bearing Length			Loads to Supports (lbs)			Accessories	Details
	Total	Available	Required	Dead	Wind	Factored		
1 - Hanger on 14" PSL beam	7.00"	Hanger[1]	1.75" / - [2]	270	-655	270/-231	See note [1]	W
2 - Hanger on 14" LSL Ledger	1.75"	Hanger[1]	1.75" / - [2]	262	-848	262/-352	See note [1]	W

- At hanger supports, the Total Bearing dimension is equal to the width of the material that is supporting the hanger
- [1] See Connector grid below for additional information and/or requirements.
- [2] Required Bearing Length / Required Bearing Length with Web Stiffeners

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	6' 9" o/c	
Bottom Edge (Lu)	6' 6" o/c	

- TJI joists are only analyzed using Maximum Allowable bracing solutions.
- Maximum allowable bracing intervals based on applied load.

Connector: Simpson Strong-Tie						
Support	Model	Seat Length	Top Fasteners	Face Fasteners	Member Fasteners	Accessories
1 - Face Mount Hanger	MIU2.1/11	2.50"	N/A	20-10dx1.5	2-10dx1.5	Web Stiffeners
2 - Face Mount Hanger	MIU2.1/11	2.50"	N/A	20-10dx1.5	6-10dx1.5	Web Stiffeners

- Refer to manufacturer notes and instructions for proper installation and use of all connectors.

ForteWEB Software Operator	Job Notes
Mohammad Aljazzar California Building Engineers Inc. (510) 674-5270 mohammad@cbestructural.com	

Weyerhaeuser

Vertical Loads	Location	Tributary Width	Dead (0.90)	Wind (1.60)	Comments
0 - Self Weight (PLF)	7" to 22' 11 3/4"	N/A	3.1	--	
1 - Uniform (PSF)	0 to 23' 1 1/2" (Top)	2'	10.0	--	Roof Loading above Unconditioned spaces. (Reduced Dead Load, No MEP or Misc.) conservatively for uplift calc.
2 - Uniform (PSF)	0 to 8' 7 3/16" (Top)	2'	--	-18.6	Wind Uplift Zone 1' (Member)
3 - Uniform (PSF)	8' 7 3/16" to 15' 9 5/8" (Top)	2'	--	-28.4	Wind Uplift Zone 1 (Member)
4 - Uniform (PSF)	15' 9 5/8" to 23' 1 1/2" (Top)	2'	--	-39.3	Wind Uplift Zone 2 (Member)
5 - Uniform (PSF)	0 to 23' 1 1/2" (Top)	2'	--	-4.3	Wind Internal Pressure

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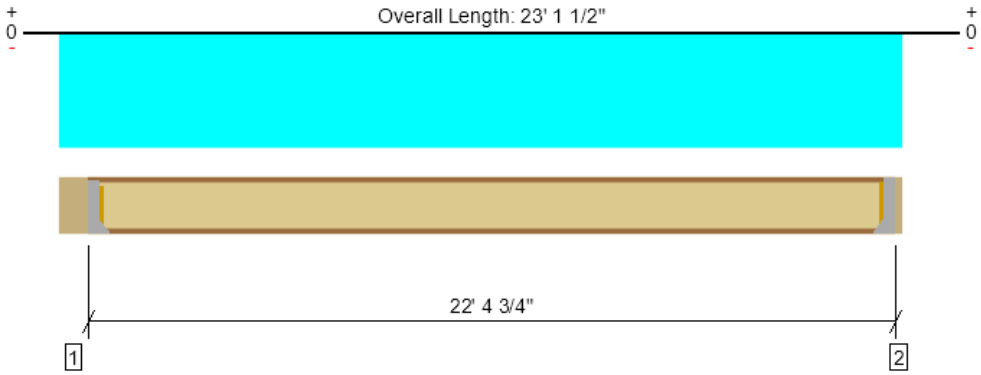
The product application, input design loads, dimensions and support information have been provided by ForteWEB Software Operator

Commercial Bldg. - Roof Joists, Spaced @ 24" o.c. [Dead min. w/ Tie Wind Uplift Z2]

1 piece(s) 14" TJI® 210



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

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	259 @ 7"	905 (1.75")	Passed (29%)	0.90	1.0 D (All Spans)
Shear (lbs)	482 @ 7"	3112	Passed (15%)	1.60	0.6 D + 0.6 W (All Spans)
Moment (Ft-lbs)	-2697 @ 11' 9 3/8"	7184	Passed (38%)	1.60	0.6 D + 0.6 W (All Spans)
Live Load Defl. (in)	0.000 @ 7"	0.747	Passed (L/999+)	--	1.0 D (All Spans)
Total Load Defl. (in)	-0.568 @ 11' 9 3/8"	1.120	Passed (L/473)	--	0.6 D + 0.6 W (All Spans)

Member Length : 22' 4 3/4"
System : Roof
Member Type : Flush Beam
Building Use : Commercial
Building Code : IBC 2024
Design Methodology : ASD
Member Pitch : 0/12

- Deflection criteria: LL (L/360) and TL (L/240).
- Allowed moment does not reflect the adjustment for the beam stability factor.
- -508 lbs uplift at support located at 7". Strapping or other restraint may be required.
- -488 lbs uplift at support located at 22' 11 3/4". Strapping or other restraint may be required.

Supports	Bearing Length			Loads to Supports (lbs)			Accessories	Details
	Total	Available	Required	Dead	Wind	Factored		
1 - Hanger on 14" PSL beam	7.00"	Hanger[1]	1.75" / - [2]	270	-1117	270/-508	See note [1]	W
2 - Hanger on 14" LSL Ledger	1.75"	Hanger[1]	1.75" / - [2]	262	-1075	262/-488	See note [1]	W

- At hanger supports, the Total Bearing dimension is equal to the width of the material that is supporting the hanger
- [1] See Connector grid below for additional information and/or requirements.
- [2] Required Bearing Length / Required Bearing Length with Web Stiffeners

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	6' 9" o/c	
Bottom Edge (Lu)	4' 11" o/c	

•TJI joists are only analyzed using Maximum Allowable bracing solutions.
•Maximum allowable bracing intervals based on applied load.

Connector: Simpson Strong-Tie						
Support	Model	Seat Length	Top Fasteners	Face Fasteners	Member Fasteners	Accessories
1 - Face Mount Hanger	MIU2.1/11	2.50"	N/A	20-10dx1.5	6-10dx1.5	Web Stiffeners
2 - Face Mount Hanger	MIU2.1/11	2.50"	N/A	20-10dx1.5	6-10dx1.5	Web Stiffeners

- Refer to manufacturer notes and instructions for proper installation and use of all connectors.

Vertical Loads	Location	Tributary Width	Dead (0.90)	Wind (1.60)	Comments
0 - Self Weight (PLF)	7" to 22' 11 3/4"	N/A	3.1	--	
1 - Uniform (PSF)	0 to 23' 1 1/2" (Top)	2'	10.0	--	Roof Loading above Unconditioned spaces. (Reduced Dead Load, No MEP or Misc.) conservatively for uplift calc.
2 - Uniform (PSF)	0 to 23' 1 1/2" (Top)	2'	--	-43.1	Wind Uplift Zone 2 (connector)
3 - Uniform (PSF)	0 to 23' 1 1/2" (Top)	2'	--	-4.3	Wind Internal Pressure

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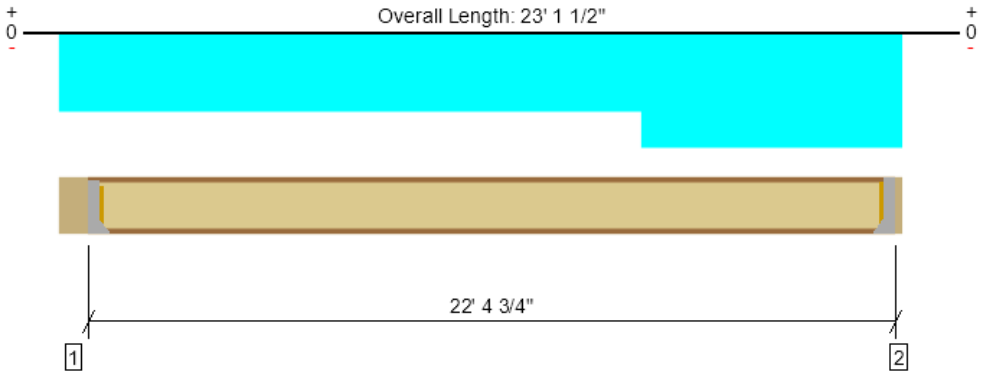
Commercial Bldg. - Roof Joists, Spaced @ 24" o.c. [Dead min. w/ Tie Wind Uplift Z1/Z2]

1 piece(s) 14" TJI® 210



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

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	259 @ 7"	905 (1.75")	Passed (29%)	0.90	1.0 D (All Spans)
Shear (lbs)	259 @ 7"	1751	Passed (15%)	0.90	1.0 D (All Spans)
Moment (Ft-lbs)	1449 @ 11' 9 3/8"	4041	Passed (36%)	0.90	1.0 D (All Spans)
Live Load Defl. (in)	0.000 @ 7"	0.747	Passed (L/999+)	--	1.0 D (All Spans)
Total Load Defl. (in)	-0.425 @ 11' 10 15/16"	1.120	Passed (L/632)	--	0.6 D + 0.6 W (All Spans)

Member Length : 22' 4 3/4"
System : Roof
Member Type : Flush Beam
Building Use : Commercial
Building Code : IBC 2024
Design Methodology : ASD
Member Pitch : 0/12

- Deflection criteria: LL (L/360) and TL (L/240).
- Allowed moment does not reflect the adjustment for the beam stability factor.
- -359 lbs uplift at support located at 7". Strapping or other restraint may be required.
- -416 lbs uplift at support located at 22' 11 3/4". Strapping or other restraint may be required.

Supports	Bearing Length			Loads to Supports (lbs)			Accessories	Details
	Total	Available	Required	Dead	Wind	Factored		
1 - Hanger on 14" PSL beam	7.00"	Hanger[1]	1.75" / - [2]	270	-868	270/-359	See note [1]	W
2 - Hanger on 14" LSL Ledger	1.75"	Hanger[1]	1.75" / - [2]	262	-954	262/-416	See note [1]	W

- At hanger supports, the Total Bearing dimension is equal to the width of the material that is supporting the hanger
- [1] See Connector grid below for additional information and/or requirements.
- [2] Required Bearing Length / Required Bearing Length with Web Stiffeners

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	6' 9" o/c	
Bottom Edge (Lu)	5' 9" o/c	

•TJI joists are only analyzed using Maximum Allowable bracing solutions.
•Maximum allowable bracing intervals based on applied load.

Connector: Simpson Strong-Tie						
Support	Model	Seat Length	Top Fasteners	Face Fasteners	Member Fasteners	Accessories
1 - Face Mount Hanger	MIU2.1/11	2.50"	N/A	20-10dx1.5	6-10dx1.5	Web Stiffeners
2 - Face Mount Hanger	MIU2.1/11	2.50"	N/A	20-10dx1.5	6-10dx1.5	Web Stiffeners

- Refer to manufacturer notes and instructions for proper installation and use of all connectors.

ForteWEB Software Operator	Job Notes
Mohammad Aljazzar California Building Engineers Inc. (510) 674-5270 mohammad@cbestructural.com	

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Vertical Loads	Location	Tributary Width	Dead (0.90)	Wind (1.60)	Comments
0 - Self Weight (PLF)	7" to 22' 11 3/4"	N/A	3.1	--	
1 - Uniform (PSF)	0 to 23' 1 1/2" (Top)	2'	10.0	--	Roof Loading above Unconditioned spaces. (Reduced Dead Load, No MEP or Misc.) conservatively for uplift calc.
2 - Uniform (PSF)	0 to 15' 9 5/8" (Top)	2'	--	-31.4	Wind Uplift Zone 1 (connector)
3 - Uniform (PSF)	15' 9 5/8" to 23' 1 1/2" (Top)	2'	--	-43.1	Wind Uplift Zone 2 (connector)
4 - Uniform (PSF)	0 to 23' 1 1/2" (Top)	2'	--	-4.3	Wind Internal Pressure

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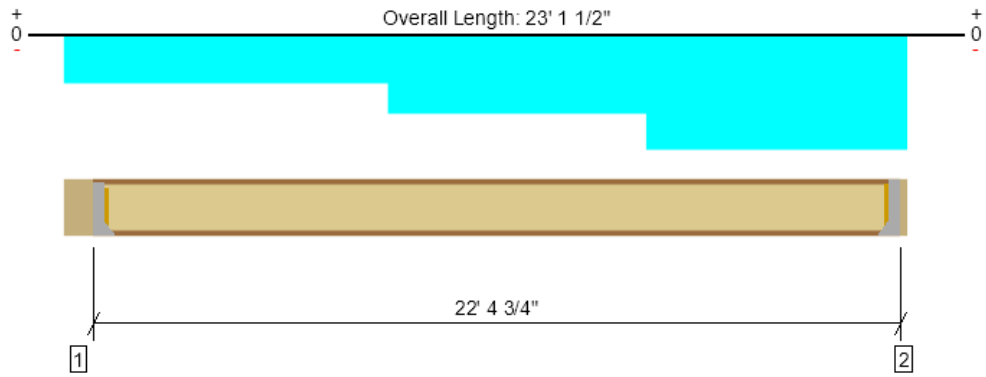
Commercial Bldg. - Roof Joists, Spaced @ 24" o.c. [Dead min. w/ Tie Wind Uplift Z1'/Z1/Z2]

1 piece(s) 14" TJI® 210



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Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	259 @ 7"	905 (1.75")	Passed (29%)	0.90	1.0 D (All Spans)
Shear (lbs)	259 @ 7"	1751	Passed (15%)	0.90	1.0 D (All Spans)
Moment (Ft-lbs)	1449 @ 11' 9 3/8"	4041	Passed (36%)	0.90	1.0 D (All Spans)
Live Load Defl. (in)	0.000 @ 7"	0.747	Passed (L/999+)	--	1.0 D (All Spans)
Total Load Defl. (in)	-0.382 @ 12' 7/8"	1.120	Passed (L/704)	--	0.6 D + 0.6 W (All Spans)

Member Length : 22' 4 3/4"
System : Roof
Member Type : Flush Beam
Building Use : Commercial
Building Code : IBC 2024
Design Methodology : ASD
Member Pitch : 0/12

- Deflection criteria: LL (L/360) and TL (L/240).
- Allowed moment does not reflect the adjustment for the beam stability factor.
- -274 lbs uplift at support located at 7". Strapping or other restraint may be required.
- -399 lbs uplift at support located at 22' 11 3/4". Strapping or other restraint may be required.

Supports	Bearing Length			Loads to Supports (lbs)			Accessories	Details
	Total	Available	Required	Dead	Wind	Factored		
1 - Hanger on 14" PSL beam	7.00"	Hanger[1]	1.75" / - [2]	270	-728	270/-274	See note [1]	W
2 - Hanger on 14" LSL Ledger	1.75"	Hanger[1]	1.75" / - [2]	262	-926	262/-399	See note [1]	W

- At hanger supports, the Total Bearing dimension is equal to the width of the material that is supporting the hanger
- [1] See Connector grid below for additional information and/or requirements.
- [2] Required Bearing Length / Required Bearing Length with Web Stiffeners

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	6' 9" o/c	
Bottom Edge (Lu)	6' o/c	

•TJI joists are only analyzed using Maximum Allowable bracing solutions.
•Maximum allowable bracing intervals based on applied load.

Connector: Simpson Strong-Tie						
Support	Model	Seat Length	Top Fasteners	Face Fasteners	Member Fasteners	Accessories
1 - Face Mount Hanger	MIU2.1/11	2.50"	N/A	20-10dx1.5	6-10dx1.5	Web Stiffeners
2 - Face Mount Hanger	MIU2.1/11	2.50"	N/A	20-10dx1.5	6-10dx1.5	Web Stiffeners

- Refer to manufacturer notes and instructions for proper installation and use of all connectors.

Vertical Loads	Location	Tributary Width	Dead (0.90)	Wind (1.60)	Comments
0 - Self Weight (PLF)	7" to 22' 11 3/4"	N/A	3.1	--	
1 - Uniform (PSF)	0 to 23' 1 1/2" (Top)	2'	10.0	--	Roof Loading above Unconditioned spaces. (Reduced Dead Load, No MEP or Misc.) conservatively for uplift calc.
2 - Uniform (PSF)	0 to 8' 7 3/16" (Top)	2'	--	-21.6	Wind Uplift Zone 1' (connector)
3 - Uniform (PSF)	8' 7 3/16" to 15' 9 5/8" (Top)	2'	--	-31.4	Wind Uplift Zone 1 (connector)
4 - Uniform (PSF)	15' 9 5/8" to 23' 1 1/2" (Top)	2'	--	-43.1	Wind Uplift Zone 2 (connector)
5 - Uniform (PSF)	0 to 23' 1 1/2" (Top)	2'	--	-4.3	Wind Internal Pressure

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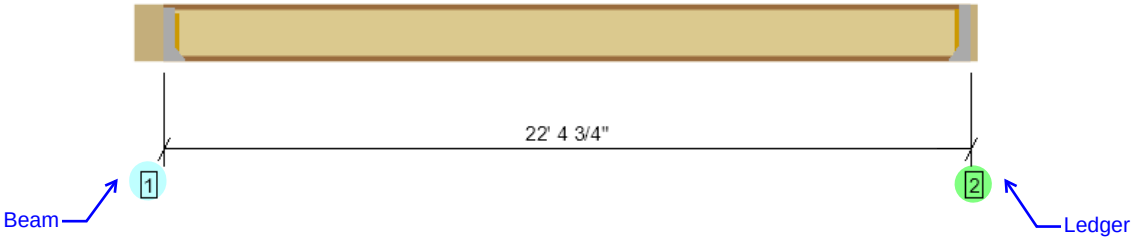
Main Roof Joist Reactions:



Commercial Bldg. - Roof Joists, Spaced @ 24" o.c. [Dead min. w/ Tie Wind Uplift Z2]

Supports	Bearing Length			Loads to Supports (lbs)			Accessories	Details
	Total	Available	Required	Dead	Wind	Factored		
1 - Hanger on 14" PSL beam	7.00"	Hanger[1]	1.75" / - [2]	270	-1117	270/-508	See note [1]	W
2 - Hanger on 14" LSL Ledger	1.75"	Hanger[1]	1.75" / - [2]	262	-1075	262/-488	See note [1]	W

Uplift (Cd=1.6) 508 < 975 lbs **OK**
At Beam: No reduction 16dx3.5" face nails used

Commercial Bldg. - Roof Joists, Spaced @ 24" o.c. [Dead min. w/ Tie Wind Uplift Z1/Z2]

Supports	Bearing Length			Loads to Supports (lbs)			Accessories	Details
	Total	Available	Required	Dead	Wind	Factored		
1 - Hanger on 14" PSL beam	7.00"	Hanger[1]	1.75" / - [2]	270	-868	270/-359	See note [1]	W
2 - Hanger on 14" LSL Ledger	1.75"	Hanger[1]	1.75" / - [2]	262	-954	262/-416	See note [1]	W

Uplift (Cd=1.6) 488 < 975 lbs **OK**
At Ledgers:
face nails from 16dx3.5 --> #10x1.5 (No reduction)
for 1 3/4" LSL ledger

Commercial Bldg. - Roof Joists, Spaced @ 24" o.c. [Dead min. w/ Tie Wind Uplift Z1'/Z1/Z2]

Supports	Bearing Length			Loads to Supports (lbs)			Accessories	Details
	Total	Available	Required	Dead	Wind	Factored		
1 - Hanger on 14" PSL beam	7.00"	Hanger[1]	1.75" / - [2]	270	-728	270/-274	See note [1]	W
2 - Hanger on 14" LSL Ledger	1.75"	Hanger[1]	1.75" / - [2]	262	-926	262/-399	See note [1]	W

Commercial Bldg. - Roof Joists, Spaced @ 24" o.c. [Snow Flat w/ Drift w/ Wind Pressure]

Supports	Bearing Length			Loads to Supports (lbs)					Accessories	Details
	Total	Available	Required	Dead	Roof Live	Snow	Wind	Factored		
1 - Hanger on 14" PSL beam	7.00"	Hanger[1]	1.75" / - [2]	412	471	436	426	957	See note [1]	W
2 - Hanger on 14" LSL Ledger	1.75"	Hanger[1]	1.75" / - [2]	398	454	530	411	923	See note [1]	W

Downward 957 < 2,085 (Cd 1.00) **OK**
Downward 943 < 2,350 x 0.64
= 1504 lbs (Cd 1.15) **OK**

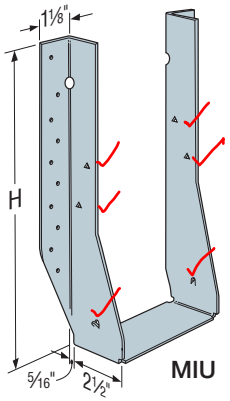
Commercial Bldg. - Roof Joists, Spaced @ 24" o.c. [Snow Balanced min. w/ Wind Pressure]

Supports	Bearing Length			Loads to Supports (lbs)					Accessories	Details
	Total	Available	Required	Dead	Roof Live	Snow	Wind	Factored		
1 - Hanger on 14" PSL beam	7.00"	Hanger[1]	1.75" / - [2]	412	471	566	426	957	See note [1]	W
2 - Hanger on 14" LSL Ledger	1.75"	Hanger[1]	1.75" / - [2]	398	454	545	411	923	See note [1]	W

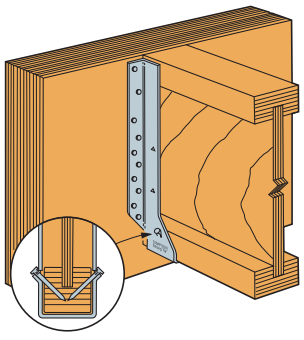
Options:

- These hangers cannot be modified. However, these models will normally accommodate a skew of up to 5°. For sloped joists up to ¼:12 there is no reduction; between ¼:12 and up to ½:12, tests show a 10% reduction in ultimate hanger strength. Local crushing of the bottom flange or excessive deflection may be limiting; check with joist manufacturer for specific limitations on bearing of this type.
- MIU — add four additional 0.148" x 1½" joist nails for a total uplift load of 975 lb.

Face Mount Hanger at Ledger:
MIU2.1/11
ASD Capacity
Uplift = 975 lbs (Cd = 1.6)
Down = 2,880 lbs (Cd = 1.0)
Down = 3,135 lbs (Cd = 1.15)
Down = 3,135 lbs (Cd = 1.25)



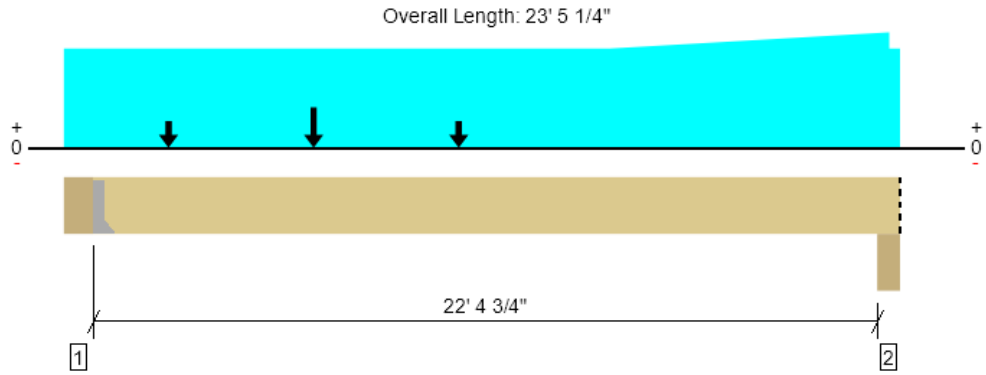
MIU


MIU with Correct PAN Installation

1a. D
2a. $D + L$
3a. $D + (L_r \text{ or } 0.7S \text{ or } R)$
4a. $D + 0.75L + 0.75(L_r \text{ or } 0.7S \text{ or } R)$
5a. $D + 0.6(W \text{ or } W_T)$
6a. $D + 0.75L + 0.75(0.6(W \text{ or } W_T)) + 0.75(L_r \text{ or } 0.7S \text{ or } R)$
7a. $0.6D + 0.6(W \text{ or } W_T)$

Commercial Bldg. - Roof Joists, AC Joist [Snow Flat w/ Drift w/ Wind Pressure]

1 piece(s) 3 1/2" x 14" 1.55E TimberStrand® LSL



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

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	2225 @ 7"	4725 (1.50")	Passed (47%)	--	1.0 D + 0.45 W + 0.75 L + 0.75 Lr (All Spans)
Shear (lbs)	1975 @ 1' 9"	12658	Passed (16%)	1.25	1.0 D + 1.0 Lr (All Spans)
Moment (Ft-lbs)	11093 @ 10' 8"	27300	Passed (41%)	1.25	1.0 D + 1.0 Lr (All Spans)
Live Load Defl. (in)	0.337 @ 11' 10 1/8"	0.751	Passed (L/801)	--	1.0 D + 0.45 W + 0.75 L + 0.75 Lr (All Spans)
Total Load Defl. (in)	0.880 @ 11' 7 1/16"	1.126	Passed (L/307)	--	1.0 D + 0.45 W + 0.75 L + 0.75 Lr (All Spans)

Member Length : 22' 10 1/4"
System : Roof
Member Type : Flush Beam
Building Use : Commercial
Building Code : IBC 2024
Design Methodology : ASD
Member Pitch : 0/12

- Deflection criteria: LL (L/360) and TL (L/240).
- Moment capacity has been adjusted by a factor of 0.99 to account for the beam stability and/or volume/size factors.

Supports	Bearing Length			Loads to Supports (lbs)					Accessories
	Total	Available	Required	Dead	Roof Live	Snow	Wind	Factored	
1 - Hanger on 14" PSL beam	7.00"	Hanger[1]	1.50"	1471	711	650	643	2293	See note [1]
2 - Column - DF	5.50"	5.50"	1.50"	1004	696	745	630	1809	Blocking

- Blocking Panels are assumed to carry no loads applied directly above them and the full load is applied to the member being designed.
- At hanger supports, the Total Bearing dimension is equal to the width of the material that is supporting the hanger
- [1] See Connector grid below for additional information and/or requirements.

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	Continuous	
Bottom Edge (Lu)	End Bearing Points	

Connector: Simpson Strong-Tie						
Support	Model	Seat Length	Top Fasteners	Face Fasteners	Member Fasteners	Accessories
1 - Face Mount Hanger	LUS410	2.00"	N/A	8-16d	6-16d	

- Refer to manufacturer notes and instructions for proper installation and use of all connectors.

ForteWEB Software Operator	Job Notes
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Vertical Loads	Location (Side)	Tributary Width	Dead (0.90)	Roof Live (1.25)	Snow (1.15)	Wind (1.60)	Comments
0 - Self Weight (PLF)	7" to 23' 5 1/4"	N/A	15.3	--	--	--	
1 - Uniform (PSF)	0 to 23' 5 1/4" (Top)	3'	16.0	20.0	--	--	Roof Loading above Unconditioned spaces.
2 - Uniform (PSF)	0 to 23' 5 1/4" (Top)	3'	--	--	17.8	--	Snow Load Flat Roof
3 - Tapered (PLF)	15' to 23' 1 1/2" (Top)	N/A	--	--	0.0 to 35.2	--	Snow Load (Drift) Added 17.6 psf 8-ft
4 - Uniform (PSF)	0 to 23' 5 1/4" (Top)	3'	--	--	--	4.9	Wind External Pressure Downward All Zones
5 - Uniform (PSF)	0 to 23' 5 1/4" (Top)	3'	--	--	--	13.2	Wind Internal Pressure
6 - Point (lb)	2' 8" (Top)	N/A	250	--	--	--	AC Unit
7 - Point (lb)	6' 8" (Top)	N/A	500	--	--	--	AC Unit
8 - Point (lb)	10' 8" (Top)	N/A	250	--	--	--	AC Unit

• Side loads are assumed to not induce cross-grain tension.

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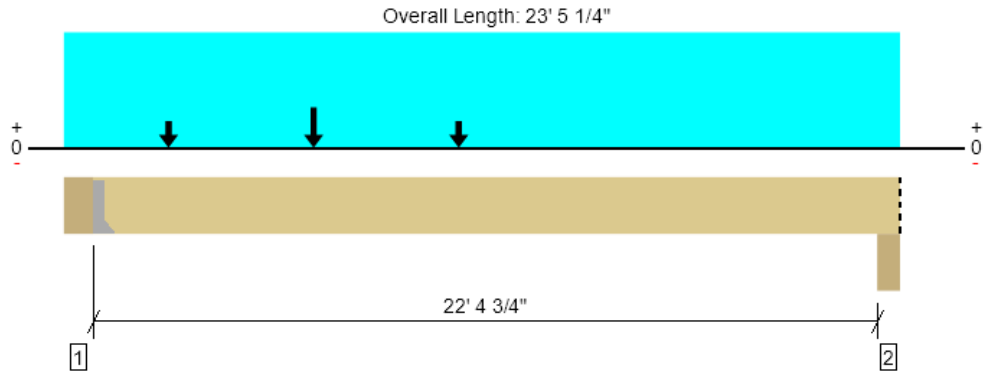
The product application, input design loads, dimensions and support information have been provided by ForteWEB Software Operator

Commercial Bldg. - Roof Joists, AC Joist [Snow Balanced min. w/ Wind Pressure]

1 piece(s) 3 1/2" x 14" 1.55E TimberStrand® LSL



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

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	2225 @ 7"	4725 (1.50")	Passed (47%)	--	1.0 D + 0.45 W + 0.75 L + 0.75 Lr (All Spans)
Shear (lbs)	1878 @ 1' 9"	11646	Passed (16%)	1.15	1.0 D + 0.7 S (All Spans)
Moment (Ft-lbs)	10491 @ 10' 8"	25116	Passed (42%)	1.15	1.0 D + 0.7 S (All Spans)
Live Load Defl. (in)	0.337 @ 11' 10 1/8"	0.751	Passed (L/801)	--	1.0 D + 0.45 W + 0.75 L + 0.75 Lr (All Spans)
Total Load Defl. (in)	0.880 @ 11' 7 1/8"	1.126	Passed (L/307)	--	1.0 D + 0.45 W + 0.75 L + 0.75 Lr (All Spans)

Member Length : 22' 10 1/4"
System : Roof
Member Type : Flush Beam
Building Use : Commercial
Building Code : IBC 2024
Design Methodology : ASD
Member Pitch : 0/12

- Deflection criteria: LL (L/360) and TL (L/240).
- Moment capacity has been adjusted by a factor of 0.99 to account for the beam stability and/or volume/size factors.

Supports	Bearing Length			Loads to Supports (lbs)					Accessories
	Total	Available	Required	Dead	Roof Live	Snow	Wind	Factored	
1 - Hanger on 14" PSL beam	7.00"	Hanger[1]	1.50"	1471	711	853	643	2293	See note [1]
2 - Column - DF	5.50"	5.50"	1.50"	1004	696	835	630	1809	Blocking

- Blocking Panels are assumed to carry no loads applied directly above them and the full load is applied to the member being designed.
- At hanger supports, the Total Bearing dimension is equal to the width of the material that is supporting the hanger
- [1] See Connector grid below for additional information and/or requirements.

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	Continuous	
Bottom Edge (Lu)	End Bearing Points	

Connector: Simpson Strong-Tie						
Support	Model	Seat Length	Top Fasteners	Face Fasteners	Member Fasteners	Accessories
1 - Face Mount Hanger	LUS410	2.00"	N/A	8-16d	6-16d	

- Refer to manufacturer notes and instructions for proper installation and use of all connectors.

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Vertical Loads	Location (Side)	Tributary Width	Dead (0.90)	Roof Live (1.25)	Snow (1.15)	Wind (1.60)	Comments
0 - Self Weight (PLF)	7" to 23' 5 1/4"	N/A	15.3	--	--	--	
1 - Uniform (PSF)	0 to 23' 5 1/4" (Top)	3'	16.0	20.0	--	--	Roof Loading above Unconditioned spaces.
2 - Uniform (PSF)	0 to 23' 5 1/4" (Top)	3'	--	--	24.0	--	Snow Load (min.) Balanced
3 - Uniform (PSF)	0 to 23' 5 1/4" (Top)	3'	--	--	--	4.9	Wind External Pressure Downward All Zones
4 - Uniform (PSF)	0 to 23' 5 1/4" (Top)	3'	--	--	--	13.2	Wind Internal Pressure
5 - Point (lb)	2' 8" (Top)	N/A	250	--	--	--	AC
6 - Point (lb)	6' 8" (Top)	N/A	500	--	--	--	AC
7 - Point (lb)	10' 8" (Top)	N/A	250	--	--	--	AC

• Side loads are assumed to not induce cross-grain tension.

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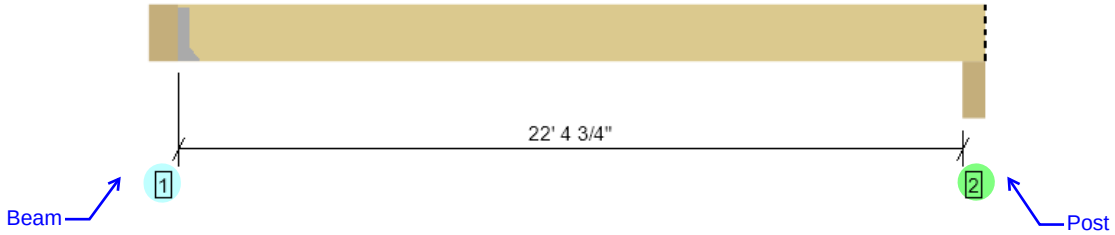
The product application, input design loads, dimensions and support information have been provided by ForteWEB Software Operator

Solid Roof Joist Reactions:



Commercial Bldg. - Roof Joists, Spaced @ 24" o.c. [Dead min. w/ Tie Wind Uplift Z2]

Twice the Tributary Area of Typical Joist

Supports	Bearing Length			Loads to Supports (lbs)			Accessories	Details
	Total	Available	Required	Dead	Wind	Factored		
1 - Hanger on 14" PSL beam	7.00"	Hanger[1]	1.75" / - [2]	270	-1117	270/-508	See note [1]	W
2 - Hanger on 14" LSL Ledger	1.75"	Hanger[1]	1.75" / - [2]	262	-1075	262/-488	See note [1]	W

Uplift (Cd=1.6) 1016 < 1,795 lbs **OK**
HU414 at Beam: No reduction 16dx3.5" face nails used

Commercial Bldg. - Roof Joists, Spaced @ 24" o.c. [Snow Flat w/ Drift w/ Wind Pressure]

Supports	Bearing Length			Loads to Supports (lbs)					Accessories
	Total	Available	Required	Dead	Roof Live	Snow	Wind	Factored	
1 - Hanger on 14" PSL beam	7.00"	Hanger[1]	1.50"	1471	711	650	643	2293	See note [1]
2 - Column - DF	5.50"	5.50"	1.50"	1004	696	745	630	1809	Blocking

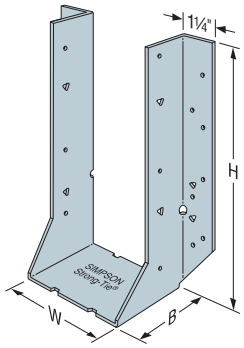
HU414
Downward 2,293 < 3,570 (Cd 1.00) **OK**
Downward 2,857 < 4,030 lbs (Cd 1.15) **OK**

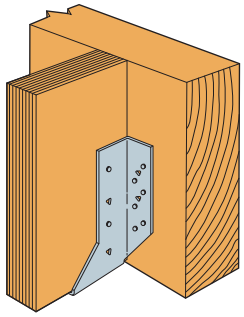
Commercial Bldg. - Roof Joists, Spaced @ 24" o.c. [Snow Balanced min. w/ Wind Pressure]

Supports	Bearing Length			Loads to Supports (lbs)					Accessories
	Total	Available	Required	Dead	Roof Live	Snow	Wind	Factored	
1 - Hanger on 14" PSL beam	7.00"	Hanger[1]	1.50"	1471	711	853	643	2293	See note [1]
2 - Column - DF	5.50"	5.50"	1.50"	1004	696	835	630	1809	Blocking

Face Mount Hanger at beam:
HU414
ASD Capacity
Uplift = 1,795 lbs (Cd = 1.6)
Down = 3,570 lbs (Cd = 1.0)
Down = 4,030 lbs (Cd = 1.15)
Down = 4,335 lbs (Cd = 1.25)

1a. D
2a. $D + L$
3a. $D + (L_r \text{ or } 0.7S \text{ or } R)$
4a. $D + 0.75L + 0.75(L_r \text{ or } 0.7S \text{ or } R)$
5a. $D + 0.6(W \text{ or } W_T)$
6a. $D + 0.75L + 0.75(0.6(W \text{ or } W_T)) + 0.75(L_r \text{ or } 0.7S \text{ or } R)$
7a. $0.6D + 0.6(W \text{ or } W_T)$





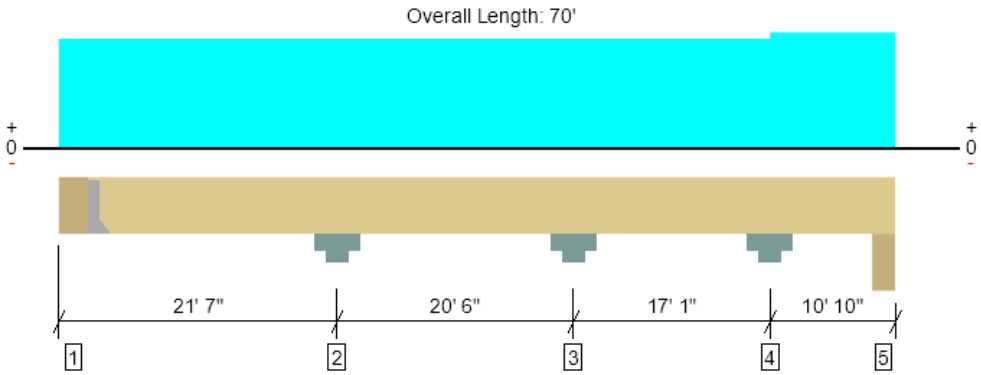
E.2 Framing Design

PSL Built-up Header

2-Ply (2)3.5"X16" Line [2]

Commercial Bldg. - Roof Beams, Continuous 1-Ply - Snow balanced min. & Wind pressure

1 piece(s) 3 1/2" x 16" 2.2E Parallam® PSL



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

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	6534 @ 7"	6534 (2.99")	Passed (100%)	--	1.0 D + 0.45 W + 0.75 L + 0.525 S (Alt Spans)
Shear (lbs)	8296 @ 19' 9 1/2"	12451	Passed (67%)	1.15	1.0 D + 0.7 S (Adj Spans)
Moment (Ft-lbs)	-36057 @ 21' 7"	40198	Passed (90%)	1.15	1.0 D + 0.7 S (Adj Spans)
Live Load Defl. (in)	0.444 @ 10' 3 3/4"	0.525	Passed (L/567)	--	1.0 D + 0.45 W + 0.75 L + 0.525 S (Alt Spans)
Total Load Defl. (in)	0.796 @ 10' 1 3/16"	1.050	Passed (L/316)	--	1.0 D + 0.45 W + 0.75 L + 0.525 S (Alt Spans)

Member Length : 69' 5"
System : Floor
Member Type : Drop Beam
Building Use : Commercial
Building Code : IBC 2024
Design Methodology : ASD

- Deflection criteria: LL (L/480) and TL (L/240).
- Moment capacity has been adjusted by a factor of 0.97 to account for the beam stability and/or volume/size factors.

Supports	Bearing Length			Loads to Supports (lbs)					Accessories
	Total	Available	Required	Dead	Roof Live	Snow	Wind	Factored	
1 - Hanger on 16" DF beam	7.00"	Hanger[1]	2.99"	3331	3358	5037	2207/-84	6968	See note [1]
2 - Column Cap - steel	11.00"	11.00"	8.44"	9023	8787	13167	5595	18453	None
3 - Column Cap - steel	11.00"	11.00"	6.75"	6945	7154	10764	4831	14770	None
4 - Column Cap - steel	11.00"	11.00"	5.64"	5697	6469	8866	3993	12345	None
5 - Column - DF	5.50"	5.50"	1.66"	1420	2171	2576	1301/-455	3633	None

- At hanger supports, the Total Bearing dimension is equal to the width of the material that is supporting the hanger
- [1] See Connector grid below for additional information and/or requirements.

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	Continuous	
Bottom Edge (Lu)	2' o/c	

Connector: Simpson Strong-Tie						
Support	Model	Seat Length	Top Fasteners	Face Fasteners	Member Fasteners	Accessories
1 - Face Mount Hanger	HGUS412	4.00"	N/A	56-10d	20-10d	

- Refer to manufacturer notes and instructions for proper installation and use of all connectors.

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Vertical Loads	Location (Side)	Tributary Width	Dead (0.90)	Roof Live (1.25)	Snow (1.15)	Wind (1.60)	Comments
0 - Self Weight (PLF)	7" to 70'	N/A	17.5	--	--	--	
1 - Uniform (PSF)	0 to 59' 2" (Back)	11'	16.0	16.0	24.0	10.0	Roof Joist (East) [Reduced Live Load At 400 ft2] [B -- E]
2 - Uniform (PSF)	59' 2" to 70' (Back)	11'	16.0	20.0	24.0	10.0	Roof Joist (East) [E -- F]
3 - Uniform (PSF)	0 to 59' 2" (Front)	11' 6"	16.0	16.0	24.0	10.0	Roof Joist (West) [Reduced Live Load At 400 ft2] [B -- E]
4 - Uniform (PSF)	59' 2" to 70' (Front)	11' 6"	16.0	20.0	24.0	10.0	Roof Joist (West) [E -- F]

• Side loads are assumed to not induce cross-grain tension.

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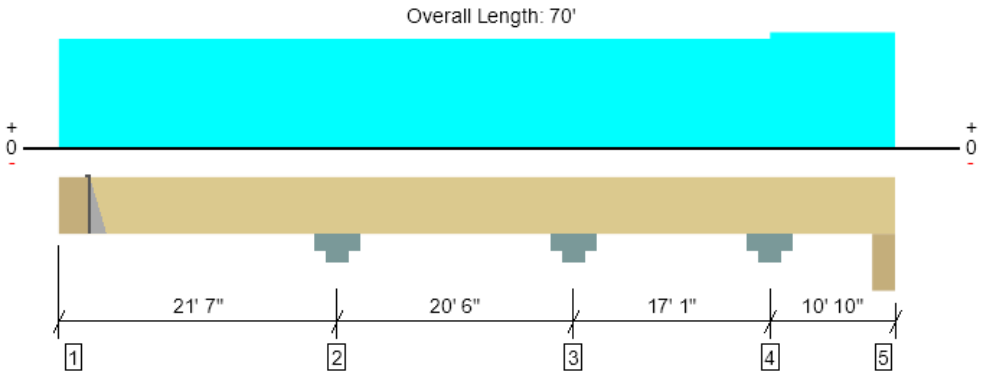
The product application, input design loads, dimensions and support information have been provided by ForteWEB Software Operator

Commercial Bldg. - Roof Beams, Continuous 2-Ply - Snow balanced min. & Wind pressure

2 piece(s) 3 1/2" x 16" 2.2E Parallam® PSL



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

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	6678 @ 7"	6678 (1.53")	Passed (100%)	--	1.0 D + 0.45 W + 0.75 L + 0.525 S (Alt Spans)
Shear (lbs)	8488 @ 19' 9 1/2"	24901	Passed (34%)	1.15	1.0 D + 0.7 S (Adj Spans)
Moment (Ft-lbs)	-36877 @ 21' 7"	80396	Passed (46%)	1.15	1.0 D + 0.7 S (Adj Spans)
Live Load Defl. (in)	0.222 @ 10' 3 3/4"	0.525	Passed (L/999+)	--	1.0 D + 0.45 W + 0.75 L + 0.525 S (Alt Spans)
Total Load Defl. (in)	0.406 @ 10' 1 1/8"	1.050	Passed (L/620)	--	1.0 D + 0.45 W + 0.75 L + 0.525 S (Alt Spans)

Member Length : 69' 5"
System : Floor
Member Type : Drop Beam
Building Use : Commercial
Building Code : IBC 2024
Design Methodology : ASD

- Deflection criteria: LL (L/480) and TL (L/240).
- Moment capacity has been adjusted by a factor of 0.97 to account for the beam stability and/or volume/size factors.
- Member should be side-loaded from both sides of the member or braced to prevent rotation.

Supports	Bearing Length			Loads to Supports (lbs)					Accessories
	Total	Available	Required	Dead	Roof Live	Snow	Wind	Factored	
1 - Hanger on 16" DF beam	7.00"	Hanger[1]	1.53"	3475	3358	5037	2207/-84	7113	See note [1]
2 - Column Cap - steel	11.00"	11.00"	4.31"	9441	8787	13167	5595	18871	None
3 - Column Cap - steel	11.00"	11.00"	3.45"	7267	7154	10764	4831	15092	None
4 - Column Cap - steel	11.00"	11.00"	2.88"	5961	6469	8866	3993	12609	None
5 - Column - DF	5.50"	5.50"	1.50"	1485	2171	2576	1301/-455	3699	None

- At hanger supports, the Total Bearing dimension is equal to the width of the material that is supporting the hanger
- [1] See Connector grid below for additional information and/or requirements.

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	Continuous	
Bottom Edge (Lu)	2' o/c	

Connector: Simpson Strong-Tie						
Support	Model	Seat Length	Top Fasteners	Face Fasteners	Member Fasteners	Accessories
1 - Top Mount Hanger	HGLTV416-2	6.00"	6-16d	12-16d	6-16d	

- Refer to manufacturer notes and instructions for proper installation and use of all connectors.

Multiple Member Connections							
Type	Location	Fastener	Placement	Rows	O.C.	# of Fasteners	Details
Uniform	7" to 21' 7"	Strong-Drive® SDS Screw SDS25600 (6")	Both Faces	2	24"	--	L17
Uniform	21' 7" to 42' 1"	Strong-Drive® SDS Screw SDS25600 (6")	Both Faces	2	24"	--	L17
Uniform	42' 1" to 59' 2"	Strong-Drive® SDS Screw SDS25600 (6")	Both Faces	2	24"	--	L17
Uniform	59' 2" to 70'	Strong-Drive® SDS Screw SDS25600 (6")	Both Faces	2	24"	--	L17

ForteWEB Software Operator	Job Notes
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6/20/2025 6:29:53 AM UTC
ForteWEB v3.9, Engine: V8.4.3.94, Data: V8.1.7.3
File Name: C & S Restaurant 29 Palms Hwy Yucca Valley

Vertical Loads	Location (Side)	Tributary Width	Dead (0.90)	Roof Live (1.25)	Snow (1.15)	Wind (1.60)	Comments
0 - Self Weight (PLF)	7" to 70'	N/A	35.0	--	--	--	
1 - Uniform (PSF)	0 to 59' 2" (Back)	11'	16.0	16.0	24.0	10.0	Roof Joist (East) [Reduced Live Load At 400 ft2] [B -- E]
2 - Uniform (PSF)	59' 2" to 70' (Back)	11'	16.0	20.0	24.0	10.0	Roof Joist (East) [E -- F]
3 - Uniform (PSF)	0 to 59' 2" (Front)	11' 6"	16.0	16.0	24.0	10.0	Roof Joist (West) [Reduced Live Load At 400 ft2] [B -- E]
4 - Uniform (PSF)	59' 2" to 70' (Front)	11' 6"	16.0	20.0	24.0	10.0	Roof Joist (West) [E -- F]

• Side loads are assumed to not induce cross-grain tension.

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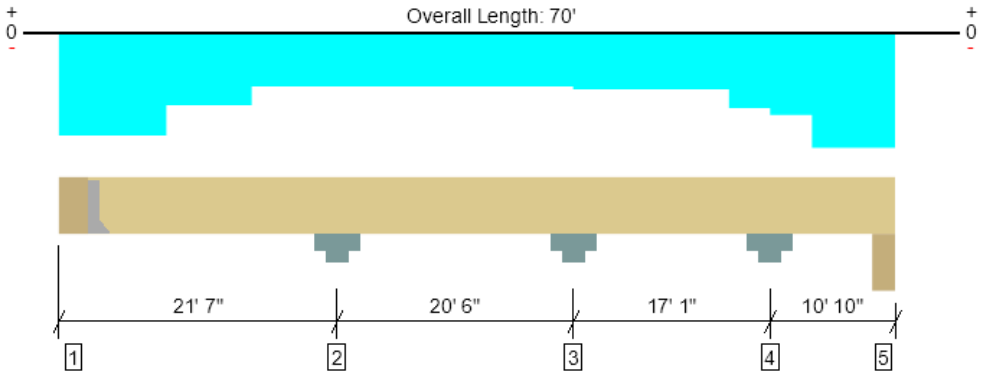
The product application, input design loads, dimensions and support information have been provided by ForteWEB Software Operator

Commercial Bldg. - Roof Beams, Continuous 1-Ply - Dead min. & Wind Uplift

1 piece(s) 3 1/2" x 16" 2.2E Parallam® PSL



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

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	2355 @ 7"	3281 (1.50")	Passed (72%)	--	1.0 D + 0.6 W (Alt Spans)
Shear (lbs)	2653 @ 19' 9 1/2"	9744	Passed (27%)	0.90	1.0 D (All Spans)
Moment (Ft-lbs)	-11374 @ 21' 7"	31459	Passed (36%)	0.90	1.0 D (All Spans)
Live Load Defl. (in)	0.231 @ 30' 6 3/16"	0.512	Passed (L/999+)	--	1.0 D + 0.6 W (Alt Spans)
Total Load Defl. (in)	-0.446 @ 10' 6 7/16"	1.050	Passed (L/565)	--	0.6 D + 0.6 W (Alt Spans)

Member Length : 69' 5"
System : Floor
Member Type : Drop Beam
Building Use : Commercial
Building Code : IBC 2024
Design Methodology : ASD

- Deflection criteria: LL (L/480) and TL (L/240).
- Moment capacity has been adjusted by a factor of 0.97 to account for the beam stability and/or volume/size factors.
- An excessive uplift of -3733 lbs detected at support located at 0".
- An excessive uplift of -6310 lbs detected at support located at 21'.
- An excessive uplift of -5241 lbs detected at support located at 41' 6".
- An excessive uplift of -5872 lbs detected at support located at 58' 7".
- An excessive uplift of -2790 lbs detected at support located at 69' 1".

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Wind	Factored	
1 - Hanger on 16" DF beam	7.00"	Hanger[1]	1.50"	2136	31/-8358	2154/-3733	See note [1]
2 - Column Cap - steel	11.00"	11.00"	2.65"	5796	-16313	5796/-6310	None
3 - Column Cap - steel	11.00"	11.00"	2.04"	4462	-13196	4462/-5241	None
4 - Column Cap - steel	11.00"	11.00"	1.67"	3659	-13445	3659/-5872	None
5 - Column - DF	5.50"	5.50"	1.50"	912	1327/-5561	1708/-2790	None

- At hanger supports, the Total Bearing dimension is equal to the width of the material that is supporting the hanger
- [1] See Connector grid below for additional information and/or requirements.

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	Continuous	
Bottom Edge (Lu)	2' o/c	

Connector: Simpson Strong-Tie						
Support	Model	Seat Length	Top Fasteners	Face Fasteners	Member Fasteners	Accessories
1 - Face Mount Hanger	HGUS412	4.00"	N/A	56-10d	20-10d	

- Refer to manufacturer notes and instructions for proper installation and use of all connectors.

ForteWEB Software Operator	Job Notes
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Vertical Loads	Location (Side)	Tributary Width	Dead (0.90)	Wind (1.60)	Comments
0 - Self Weight (PLF)	7" to 70'	N/A	17.5	--	
1 - Uniform (PSF)	0 to 59' 2" (Front)	11'	10.0	--	Roof Joist (1--2) Dead min.
2 - Uniform (PSF)	59' 2" to 70' (Front)	11'	10.0	--	Roof Joist (1--2) Dead min. [E -- F]
3 - Uniform (PSF)	0 to 59' 2" (Back)	11' 6"	10.0	--	Roof Joist (2--3) Dead min.
4 - Uniform (PSF)	59' 2" to 70' (Back)	11' 6"	10.0	--	Roof Joist (2--3) Dead min. [E -- F]
5 - Uniform (PLF)	0 to 7' 2 3/8" (Front)	N/A	--	-464.0	Wind Uplift (1--2) Wind Zone 2 [B--D] [EWA 230 ft2]
6 - Uniform (PLF)	0 to 7' 2 3/8" (Back)	N/A	--	-485.0	Wind Uplift (2--3) Wind Zone 2 [B--D] [EWA 230 ft2]
7 - Uniform (PLF)	7' 2 3/8" to 14' 4 13/16" (Front)	N/A	--	-361.0	Wind Uplift (1--2) Wind Zone 1/2 [B--D] [EWA 230 ft2]
8 - Uniform (PLF)	7' 2 3/8" to 14' 4 13/16" (Back)	N/A	--	-376.0	Wind Uplift (2--3) Wind Zone 1/2 [B--D] [EWA 230 ft2]
9 - Uniform (PLF)	14' 4 13/16" to 42' 1" (Front)	N/A	--	-298.0	Wind Uplift (1--2) Wind Zone 1'/1/2 [B--D] [EWA 230 ft2]
10 - Uniform (PLF)	14' 4 13/16" to 42' 1" (Back)	N/A	--	-306.0	Wind Uplift (2--3) Wind Zone 1'/1/2 [B--D] [EWA 230 ft2]
11 - Uniform (PLF)	42' 1" to 55' 7 3/16" (Front)	N/A	--	-308.0	Wind Uplift (1--2) Wind Zone 1'/1/2 [D--E] [EWA 190 ft2]
12 - Uniform (PLF)	42' 1" to 55' 7 3/16" (Back)	N/A	--	-317.0	Wind Uplift (2--3) Wind Zone 1'/1/2 [D--E] [EWA 190 ft2]
13 - Uniform (PLF)	55' 7 3/16" to 59' 2" (Front)	N/A	--	-370.0	Wind Uplift (1--2) Wind Zone 1/2 [D--E] [EWA 190 ft2]
14 - Uniform (PLF)	55' 7 3/16" to 59' 2" (Back)	N/A	--	-386.0	Wind Uplift (2--3) Wind Zone 1/2 [D--E] [EWA 190 ft2]
15 - Uniform (PLF)	59' 2" to 62' 9 5/8" (Front)	N/A	--	-394.0	Wind Uplift (1--2) Wind Zone 1/2 [E--F] [EWA 115 ft2]
16 - Uniform (PLF)	59' 2" to 62' 9 5/8" (Back)	N/A	--	-411.0	Wind Uplift (2--3) Wind Zone 1/2 [E--F] [EWA 115 ft2]
17 - Uniform (PLF)	62' 9 5/8" to 70' (Front)	N/A	--	-506.0	Wind Uplift (1--2) Wind Zone 2 [E--F] [EWA 115 ft2]
18 - Uniform (PLF)	62' 9 5/8" to 70' (Back)	N/A	--	-529.0	Wind Uplift (2--3) Wind Zone 2 [E--F] [EWA 115 ft2]

• Side loads are assumed to not induce cross-grain tension.

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

ForteWEB Software Operator	Job Notes
Mohammad Aljazzar California Building Engineers Inc. (510) 674-5270 mohammad@cbestructural.com	

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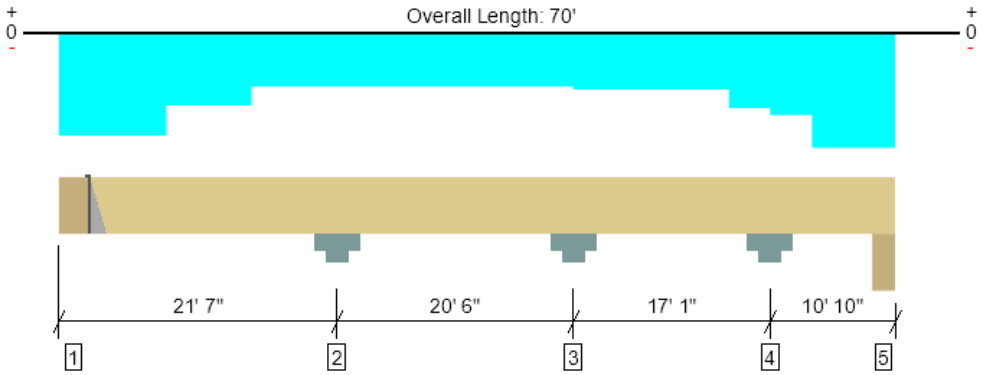
6/20/2025 6:22:39 AM UTC

ForteWEB v3.9, Engine: V8.4.3.94, Data: V8.1.7.3

File Name: C & S Restaurant 29 Palms Hwy Yucca Valley

Commercial Bldg. - Roof Beams, Continuous 2-Ply - Dead min. & Wind Uplift

2 piece(s) 3 1/2" x 16" 2.2E Parallam® PSL



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

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	2500 @ 7"	6563 (1.50")	Passed (38%)	--	1.0 D + 0.6 W (Alt Spans)
Shear (lbs)	2845 @ 19' 9 1/2"	19488	Passed (15%)	0.90	1.0 D (All Spans)
Moment (Ft-lbs)	-12195 @ 21' 7"	62918	Passed (19%)	0.90	1.0 D (All Spans)
Live Load Defl. (in)	0.116 @ 30' 6 3/16"	0.512	Passed (L/999+)	--	1.0 D + 0.6 W (Alt Spans)
Total Load Defl. (in)	-0.218 @ 10' 6 5/8"	1.050	Passed (L/999+)	--	0.6 D + 0.6 W (Alt Spans)

Member Length : 69' 5"
System : Floor
Member Type : Drop Beam
Building Use : Commercial
Building Code : IBC 2024
Design Methodology : ASD

- Deflection criteria: LL (L/480) and TL (L/240).
- Moment capacity has been adjusted by a factor of 0.97 to account for the beam stability and/or volume/size factors.
- An excessive uplift of -3646 lbs detected at support located at 0".
- An excessive uplift of -6059 lbs detected at support located at 21'.
- An excessive uplift of -5048 lbs detected at support located at 41' 6".
- An excessive uplift of -5713 lbs detected at support located at 58' 7".
- An excessive uplift of -2750 lbs detected at support located at 69' 1".
- Member should be side-loaded from both sides of the member or braced to prevent rotation.

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Wind	Factored	
1 - Hanger on 16" DF beam	7.00"	Hanger[1]	1.50"	2281	31/-8358	2299/-3646	See note [1]
2 - Column Cap - steel	11.00"	11.00"	1.50"	6214	-16313	6214/-6059	None
3 - Column Cap - steel	11.00"	11.00"	1.50"	4784	-13196	4784/-5048	None
4 - Column Cap - steel	11.00"	11.00"	1.50"	3924	-13445	3924/-5713	None
5 - Column - DF	5.50"	5.50"	1.50"	978	1327/-5561	1774/-2750	None

- At hanger supports, the Total Bearing dimension is equal to the width of the material that is supporting the hanger
- [1] See Connector grid below for additional information and/or requirements.

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	Continuous	
Bottom Edge (Lu)	2' o/c	

Connector: Simpson Strong-Tie

Support	Model	Seat Length	Top Fasteners	Face Fasteners	Member Fasteners	Accessories
1 - Top Mount Hanger	EGQ7.25-SDS3 H=15.938	6.00"	N/A	28-SDS25300	12-SDS25300	

- Refer to manufacturer notes and instructions for proper installation and use of all connectors.

Multiple Member Connections

Type	Location	Fastener	Placement	Rows	O.C.	# of Fasteners	Details
Uniform	7" to 21' 7"	Strong-Drive® SDS Screw SDS25600 (6")	Both Faces	2	24"	--	L17
Uniform	21' 7" to 42' 1"	Strong-Drive® SDS Screw SDS25600 (6")	Both Faces	2	24"	--	L17
Uniform	42' 1" to 59' 2"	Strong-Drive® SDS Screw SDS25600 (6")	Both Faces	2	24"	--	L17
Uniform	59' 2" to 70'	Strong-Drive® SDS Screw SDS25600 (6")	Both Faces	2	24"	--	L17

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Vertical Loads	Location (Side)	Tributary Width	Dead (0.90)	Wind (1.60)	Comments
0 - Self Weight (PLF)	7" to 70'	N/A	35.0	--	
1 - Uniform (PSF)	0 to 59' 2" (Front)	11'	10.0	--	Roof Joist (1--2) Dead min.
2 - Uniform (PSF)	59' 2" to 70' (Front)	11'	10.0	--	Roof Joist (1--2) Dead min. [E -- F]
3 - Uniform (PSF)	0 to 59' 2" (Back)	11' 6"	10.0	--	Roof Joist (2--3) Dead min.
4 - Uniform (PSF)	59' 2" to 70' (Back)	11' 6"	10.0	--	Roof Joist (2--3) Dead min. [E -- F]
5 - Uniform (PLF)	0 to 7' 2 3/8" (Front)	N/A	--	-464.0	Wind Uplift (1--2) Wind Zone 2 [B--D] [EWA 230 ft2]
6 - Uniform (PLF)	0 to 7' 2 3/8" (Back)	N/A	--	-485.0	Wind Uplift (2--3) Wind Zone 2 [B--D] [EWA 230 ft2]
7 - Uniform (PLF)	7' 2 3/8" to 14' 4 13/16" (Front)	N/A	--	-361.0	Wind Uplift (1--2) Wind Zone 1/2 [B--D] [EWA 230 ft2]
8 - Uniform (PLF)	7' 2 3/8" to 14' 4 13/16" (Back)	N/A	--	-376.0	Wind Uplift (2--3) Wind Zone 1/2 [B--D] [EWA 230 ft2]
9 - Uniform (PLF)	14' 4 13/16" to 42' 1" (Front)	N/A	--	-298.0	Wind Uplift (1--2) Wind Zone 1'/1/2 [B--D] [EWA 230 ft2]
10 - Uniform (PLF)	14' 4 13/16" to 42' 1" (Back)	N/A	--	-306.0	Wind Uplift (2--3) Wind Zone 1'/1/2 [B--D] [EWA 230 ft2]
11 - Uniform (PLF)	42' 1" to 55' 7 3/16" (Front)	N/A	--	-308.0	Wind Uplift (1--2) Wind Zone 1'/1/2 [D--E] [EWA 190 ft2]
12 - Uniform (PLF)	42' 1" to 55' 7 3/16" (Back)	N/A	--	-317.0	Wind Uplift (2--3) Wind Zone 1'/1/2 [D--E] [EWA 190 ft2]
13 - Uniform (PLF)	55' 7 3/16" to 59' 2" (Front)	N/A	--	-370.0	Wind Uplift (1--2) Wind Zone 1/2 [D--E] [EWA 190 ft2]
14 - Uniform (PLF)	55' 7 3/16" to 59' 2" (Back)	N/A	--	-386.0	Wind Uplift (2--3) Wind Zone 1/2 [D--E] [EWA 190 ft2]
15 - Uniform (PLF)	59' 2" to 62' 9 5/8" (Front)	N/A	--	-394.0	Wind Uplift (1--2) Wind Zone 1/2 [E--F] [EWA 115 ft2]
16 - Uniform (PLF)	59' 2" to 62' 9 5/8" (Back)	N/A	--	-411.0	Wind Uplift (2--3) Wind Zone 1/2 [E--F] [EWA 115 ft2]
17 - Uniform (PLF)	62' 9 5/8" to 70' (Front)	N/A	--	-506.0	Wind Uplift (1--2) Wind Zone 2 [E--F] [EWA 115 ft2]
18 - Uniform (PLF)	62' 9 5/8" to 70' (Back)	N/A	--	-529.0	Wind Uplift (2--3) Wind Zone 2 [E--F] [EWA 115 ft2]

• Side loads are assumed to not induce cross-grain tension.

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

ForteWEB Software Operator	Job Notes
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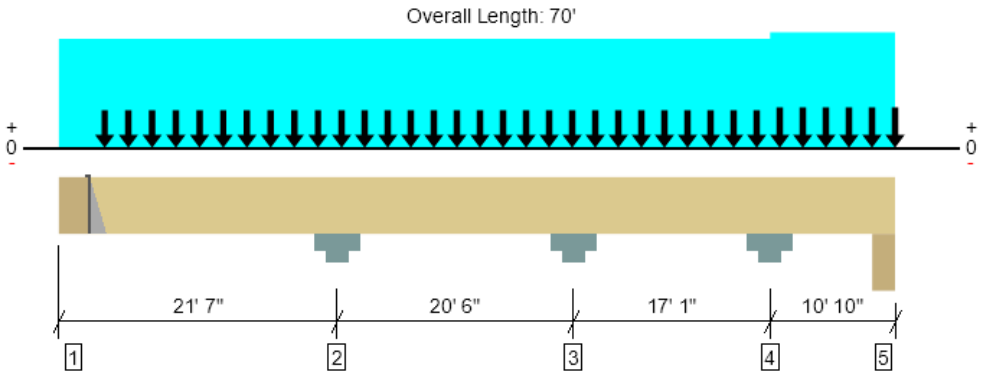
Commercial Bldg. - Roof Beams, Point Continuous 2-Ply - Snow balanced min. & Wind pressure

2 piece(s) 3 1/2" x 16" 2.2E Parallam® PSL



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

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	6523 @ 7"	6563 (1.50")	Passed (99%)	--	1.0 D + 0.45 W + 0.75 L + 0.525 S (Alt Spans)
Shear (lbs)	8825 @ 19' 9 1/2"	24901	Passed (35%)	1.15	1.0 D + 0.7 S (Adj Spans)
Moment (Ft-lbs)	-36737 @ 21' 7"	80396	Passed (46%)	1.15	1.0 D + 0.7 S (Adj Spans)
Live Load Defl. (in)	0.223 @ 10' 3 3/4"	0.525	Passed (L/999+)	--	1.0 D + 0.45 W + 0.75 L + 0.525 S (Alt Spans)
Total Load Defl. (in)	0.408 @ 10' 1 1/16"	1.050	Passed (L/618)	--	1.0 D + 0.45 W + 0.75 L + 0.525 S (Alt Spans)

Member Length : 69' 5"
System : Floor
Member Type : Drop Beam
Building Use : Commercial
Building Code : IBC 2024
Design Methodology : ASD

- Deflection criteria: LL (L/480) and TL (L/240).
- Moment capacity has been adjusted by a factor of 0.97 to account for the beam stability and/or volume/size factors.
- Member should be side-loaded from both sides of the member or braced to prevent rotation.

Supports	Bearing Length			Loads to Supports (lbs)					Accessories
	Total	Available	Required	Dead	Roof Live	Snow	Wind	Factored	
1 - Hanger on 16" DF beam	7.00"	Hanger[1]	1.50"	3294	3174	4762	2091/-149	6735	See note [1]
2 - Column Cap - steel	11.00"	11.00"	4.31"	9443	8789	13169	5595	18875	None
3 - Column Cap - steel	11.00"	11.00"	3.45"	7259	7146	10753	4826	15076	None
4 - Column Cap - steel	11.00"	11.00"	2.88"	5964	6476	8866	3991	12617	None
5 - Column - DF	5.50"	5.50"	1.50"	1669	2402	2853	1416/-455	4108	None

- At hanger supports, the Total Bearing dimension is equal to the width of the material that is supporting the hanger
- [1] See Connector grid below for additional information and/or requirements.

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	Continuous	
Bottom Edge (Lu)	2' o/c	

Connector: Simpson Strong-Tie

Support	Model	Seat Length	Top Fasteners	Face Fasteners	Member Fasteners	Accessories
1 - Top Mount Hanger	HGLTV416-2	6.00"	6-16d	12-16d	6-16d	

- Refer to manufacturer notes and instructions for proper installation and use of all connectors.

Multiple Member Connections

Type	Location	Fastener	Placement	Rows	O.C.	# of Fasteners	Details
Uniform	7" to 21' 7"	Strong-Drive® SDS Screw SDS25600 (6")	Both Faces	2	24"	--	L17
Uniform	21' 7" to 42' 1"	Strong-Drive® SDS Screw SDS25600 (6")	Both Faces	2	24"	--	L17
Uniform	42' 1" to 59' 2"	Strong-Drive® SDS Screw SDS25600 (6")	Both Faces	2	24"	--	L17
Uniform	59' 2" to 70'	Strong-Drive® SDS Screw SDS25600 (6")	Both Faces	2	24"	--	L17

Vertical Loads	Location (Side)	Tributary Width	Dead (0.90)	Roof Live (1.25)	Snow (1.15)	Wind (1.60)	Comments
0 - Self Weight (PLF)	7" to 70'	N/A	35.0	--	--	--	

ForteWEB Software Operator	Job Notes
Mohammad Aljazzar California Building Engineers Inc. (510) 674-5270 mohammad@cbestructural.com	

Weyerhaeuser

6/20/2025 6:22:39 AM UTC
ForteWEB v3.9, Engine: V8.4.3.94, Data: V8.1.7.3
File Name: C & S Restaurant 29 Palms Hwy Yucca Valley

Vertical Loads	Location (Side)	Tributary Width	Dead (0.90)	Roof Live (1.25)	Snow (1.15)	Wind (1.60)	Comments
1 - Uniform (PSF)	0 to 59' 2" (Back)	11'	16.0	16.0	24.0	10.0	Roof Joist (East) [Reduced Live Load At 400 ft2] [B -- E]
2 - Uniform (PSF)	59' 2" to 70' (Back)	11'	16.0	20.0	24.0	10.0	Roof Joist (East) [E -- F]
3 - Point (lb)	2' (Front)	N/A	368	368	552	230	Roof Joist (West) [Reduced Live Load At 400 ft2] [B -- E]
4 - Point (lb)	4' (Front)	N/A	368	368	552	230	Roof Joist (West) [Reduced Live Load At 400 ft2] [B -- E]
5 - Point (lb)	6' (Front)	N/A	368	368	552	230	Roof Joist (West) [Reduced Live Load At 400 ft2] [B -- E]
6 - Point (lb)	8' (Front)	N/A	368	368	552	230	Roof Joist (West) [Reduced Live Load At 400 ft2] [B -- E]
7 - Point (lb)	10' (Front)	N/A	368	368	552	230	Roof Joist (West) [Reduced Live Load At 400 ft2] [B -- E]
8 - Point (lb)	12' (Front)	N/A	368	368	552	230	Roof Joist (West) [Reduced Live Load At 400 ft2] [B -- E]
9 - Point (lb)	14' (Front)	N/A	368	368	552	230	Roof Joist (West) [Reduced Live Load At 400 ft2] [B -- E]
10 - Point (lb)	16' (Front)	N/A	368	368	552	230	Roof Joist (West) [Reduced Live Load At 400 ft2] [B -- E]
11 - Point (lb)	18' (Front)	N/A	368	368	552	230	Roof Joist (West) [Reduced Live Load At 400 ft2] [B -- E]
12 - Point (lb)	20' (Front)	N/A	368	368	552	230	Roof Joist (West) [Reduced Live Load At 400 ft2] [B -- E]
13 - Point (lb)	22' (Front)	N/A	368	368	552	230	Roof Joist (West) [Reduced Live Load At 400 ft2] [B -- E]
14 - Point (lb)	24' (Front)	N/A	368	368	552	230	Roof Joist (West) [Reduced Live Load At 400 ft2] [B -- E]
15 - Point (lb)	26' (Front)	N/A	368	368	552	230	Roof Joist (West) [Reduced Live Load At 400 ft2] [B -- E]
16 - Point (lb)	28' (Front)	N/A	368	368	552	230	Roof Joist (West) [Reduced Live Load At 400 ft2] [B -- E]
17 - Point (lb)	30' (Front)	N/A	368	368	552	230	Roof Joist (West) [Reduced Live Load At 400 ft2] [B -- E]
18 - Point (lb)	32' (Front)	N/A	368	368	552	230	Roof Joist (West) [Reduced Live Load At 400 ft2] [B -- E]
19 - Point (lb)	34' (Front)	N/A	368	368	552	230	Roof Joist (West) [Reduced Live Load At 400 ft2] [B -- E]
20 - Point (lb)	36' (Front)	N/A	368	368	552	230	Roof Joist (West) [Reduced Live Load At 400 ft2] [B -- E]
21 - Point (lb)	38' (Front)	N/A	368	368	552	230	Roof Joist (West) [Reduced Live Load At 400 ft2] [B -- E]

Vertical Loads	Location (Side)	Tributary Width	Dead (0.90)	Roof Live (1.25)	Snow (1.15)	Wind (1.60)	Comments
22 - Point (lb)	40' (Front)	N/A	368	368	552	230	Roof Joist (West) [Reduced Live Load At 400 ft2] [B -- E]
23 - Point (lb)	42' (Front)	N/A	368	368	552	230	Roof Joist (West) [Reduced Live Load At 400 ft2] [B -- E]
24 - Point (lb)	44' (Front)	N/A	368	368	552	230	Roof Joist (West) [Reduced Live Load At 400 ft2] [B -- E]
25 - Point (lb)	46' (Front)	N/A	368	368	552	230	Roof Joist (West) [Reduced Live Load At 400 ft2] [B -- E]
26 - Point (lb)	48' (Front)	N/A	368	368	552	230	Roof Joist (West) [Reduced Live Load At 400 ft2] [B -- E]
27 - Point (lb)	50' (Front)	N/A	368	368	552	230	Roof Joist (West) [Reduced Live Load At 400 ft2] [B -- E]
28 - Point (lb)	52' (Front)	N/A	368	368	552	230	Roof Joist (West) [Reduced Live Load At 400 ft2] [B -- E]
29 - Point (lb)	54' (Front)	N/A	368	368	552	230	Roof Joist (West) [Reduced Live Load At 400 ft2] [B -- E]
30 - Point (lb)	56' (Front)	N/A	368	368	552	230	Roof Joist (West) [Reduced Live Load At 400 ft2] [B -- E]
31 - Point (lb)	58' (Front)	N/A	368	368	552	230	Roof Joist (West) [Reduced Live Load At 400 ft2] [B -- E]
32 - Point (lb)	60' (Front)	N/A	368	460	552	230	Roof Joist (West) [E -- F]
33 - Point (lb)	62' (Front)	N/A	368	460	552	230	Roof Joist (West) [E -- F]
34 - Point (lb)	64' (Front)	N/A	368	460	552	230	Roof Joist (West) [E -- F]
35 - Point (lb)	66' (Front)	N/A	368	460	552	230	Roof Joist (West) [E -- F]
36 - Point (lb)	68' (Front)	N/A	368	460	552	230	Roof Joist (West) [E -- F]
37 - Point (lb)	70' (Front)	N/A	368	460	552	230	Roof Joist (West) [E -- F]

• Side loads are assumed to not induce cross-grain tension.

Weyerhaeuser Notes

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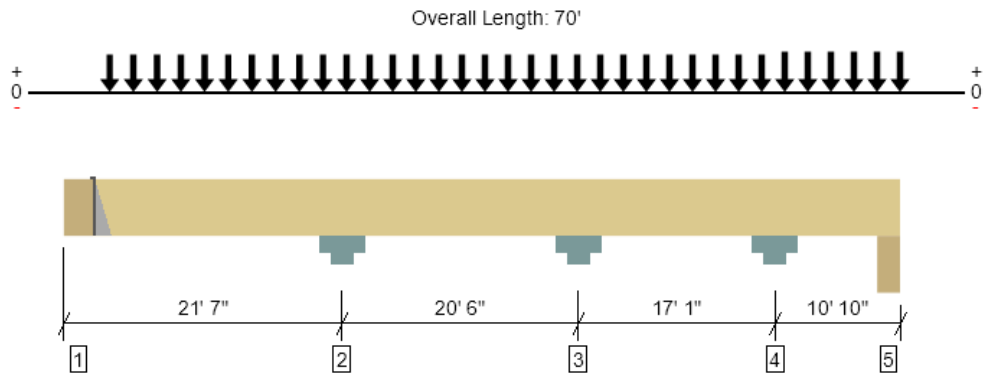
The product application, input design loads, dimensions and support information have been provided by ForteWEB Software Operator

Commercial Bldg. - Roof Beams, 1-sided Point Continuous 2-Ply - Snow balanced min. & Wind pressure

2 piece(s) 3 1/2" x 16" 2.2E Parallam® PSL



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

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	3399 @ 7"	6563 (1.50")	Passed (52%)	--	1.0 D + 0.45 W + 0.75 L + 0.525 S (Alt Spans)
Shear (lbs)	4863 @ 19' 9 1/2"	24901	Passed (20%)	1.15	1.0 D + 0.7 S (Adj Spans)
Moment (Ft-lbs)	-19511 @ 21' 7"	80396	Passed (24%)	1.15	1.0 D + 0.7 S (Adj Spans)
Live Load Defl. (in)	0.114 @ 10' 3 3/4"	0.525	Passed (L/999+)	--	1.0 D + 0.45 W + 0.75 L + 0.525 S (Alt Spans)
Total Load Defl. (in)	0.217 @ 10' 1"	1.050	Passed (L/999+)	--	1.0 D + 0.45 W + 0.75 L + 0.525 S (Alt Spans)

Member Length : 69' 5"
System : Floor
Member Type : Drop Beam
Building Use : Commercial
Building Code : IBC 2024
Design Methodology : ASD

- Deflection criteria: LL (L/480) and TL (L/240).
- Moment capacity has been adjusted by a factor of 0.97 to account for the beam stability and/or volume/size factors.
- Member should be side-loaded from both sides of the member or braced to prevent rotation.

Supports	Bearing Length			Loads to Supports (lbs)					Accessories
	Total	Available	Required	Dead	Roof Live	Snow	Wind	Factored	
1 - Hanger on 16" DF beam	7.00"	Hanger[1]	1.50"	1736	1533	2300	1012/-108	3399	See note [1]
2 - Column Cap - steel	11.00"	11.00"	2.30"	5237	4493	6732	2860	10058	None
3 - Column Cap - steel	11.00"	11.00"	1.83"	4021	3649	5491	2465	8013	None
4 - Column Cap - steel	11.00"	11.00"	1.53"	3308	3313	4531	2038	6710	None
5 - Column - DF	5.50"	5.50"	1.50"	1007	1341	1593	780/-233	2364	None

- At hanger supports, the Total Bearing dimension is equal to the width of the material that is supporting the hanger
- [1] See Connector grid below for additional information and/or requirements.

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	Continuous	
Bottom Edge (Lu)	2' o/c	

Connector: Simpson Strong-Tie

Support	Model	Seat Length	Top Fasteners	Face Fasteners	Member Fasteners	Accessories
1 - Top Mount Hanger	EGQ7.25-SDS3 H=15.938	6.00"	N/A	28-SDS25300	12-SDS25300	

- Refer to manufacturer notes and instructions for proper installation and use of all connectors.

Multiple Member Connections

Type	Location	Fastener	Placement	Rows	O.C.	# of Fasteners	Details
Uniform	7" to 21' 7"	Strong-Drive® SDS Screw SDS25600 (6")	Both Faces	2	24"	--	L17
Uniform	21' 7" to 42' 1"	Strong-Drive® SDS Screw SDS25600 (6")	Both Faces	2	24"	--	L17
Uniform	42' 1" to 59' 2"	Strong-Drive® SDS Screw SDS25600 (6")	Both Faces	2	24"	--	L17
Uniform	59' 2" to 70'	Strong-Drive® SDS Screw SDS25600 (6")	Both Faces	2	24"	--	L17

Vertical Loads	Location (Side)	Tributary Width	Dead (0.90)	Roof Live (1.25)	Snow (1.15)	Wind (1.60)	Comments
0 - Self Weight (PLF)	7" to 70'	N/A	35.0	--	--	--	

ForteWEB Software Operator	Job Notes
Mohammad Aljazzar California Building Engineers Inc. (510) 674-5270 mohammad@cbestructural.com	

Weyerhaeuser

6/20/2025 6:22:39 AM UTC
ForteWEB v3.9, Engine: V8.4.3.94, Data: V8.1.7.3
File Name: C & S Restaurant 29 Palms Hwy Yucca Valley

Vertical Loads	Location (Side)	Tributary Width	Dead (0.90)	Roof Live (1.25)	Snow (1.15)	Wind (1.60)	Comments
1 - Point (lb)	2' (Front)	N/A	368	368	552	230	Roof Joist (West) [Reduced Live Load At 400 ft2] [B -- E]
2 - Point (lb)	4' (Front)	N/A	368	368	552	230	Roof Joist (West) [Reduced Live Load At 400 ft2] [B -- E]
3 - Point (lb)	6' (Front)	N/A	368	368	552	230	Roof Joist (West) [Reduced Live Load At 400 ft2] [B -- E]
4 - Point (lb)	8' (Front)	N/A	368	368	552	230	Roof Joist (West) [Reduced Live Load At 400 ft2] [B -- E]
5 - Point (lb)	10' (Front)	N/A	368	368	552	230	Roof Joist (West) [Reduced Live Load At 400 ft2] [B -- E]
6 - Point (lb)	12' (Front)	N/A	368	368	552	230	Roof Joist (West) [Reduced Live Load At 400 ft2] [B -- E]
7 - Point (lb)	14' (Front)	N/A	368	368	552	230	Roof Joist (West) [Reduced Live Load At 400 ft2] [B -- E]
8 - Point (lb)	16' (Front)	N/A	368	368	552	230	Roof Joist (West) [Reduced Live Load At 400 ft2] [B -- E]
9 - Point (lb)	18' (Front)	N/A	368	368	552	230	Roof Joist (West) [Reduced Live Load At 400 ft2] [B -- E]
10 - Point (lb)	20' (Front)	N/A	368	368	552	230	Roof Joist (West) [Reduced Live Load At 400 ft2] [B -- E]
11 - Point (lb)	22' (Front)	N/A	368	368	552	230	Roof Joist (West) [Reduced Live Load At 400 ft2] [B -- E]
12 - Point (lb)	24' (Front)	N/A	368	368	552	230	Roof Joist (West) [Reduced Live Load At 400 ft2] [B -- E]
13 - Point (lb)	26' (Front)	N/A	368	368	552	230	Roof Joist (West) [Reduced Live Load At 400 ft2] [B -- E]
14 - Point (lb)	28' (Front)	N/A	368	368	552	230	Roof Joist (West) [Reduced Live Load At 400 ft2] [B -- E]
15 - Point (lb)	30' (Front)	N/A	368	368	552	230	Roof Joist (West) [Reduced Live Load At 400 ft2] [B -- E]
16 - Point (lb)	32' (Front)	N/A	368	368	552	230	Roof Joist (West) [Reduced Live Load At 400 ft2] [B -- E]
17 - Point (lb)	34' (Front)	N/A	368	368	552	230	Roof Joist (West) [Reduced Live Load At 400 ft2] [B -- E]
18 - Point (lb)	36' (Front)	N/A	368	368	552	230	Roof Joist (West) [Reduced Live Load At 400 ft2] [B -- E]
19 - Point (lb)	38' (Front)	N/A	368	368	552	230	Roof Joist (West) [Reduced Live Load At 400 ft2] [B -- E]
20 - Point (lb)	40' (Front)	N/A	368	368	552	230	Roof Joist (West) [Reduced Live Load At 400 ft2] [B -- E]

Vertical Loads	Location (Side)	Tributary Width	Dead (0.90)	Roof Live (1.25)	Snow (1.15)	Wind (1.60)	Comments
21 - Point (lb)	42' (Front)	N/A	368	368	552	230	Roof Joist (West) [Reduced Live Load At 400 ft2] [B -- E]
22 - Point (lb)	44' (Front)	N/A	368	368	552	230	Roof Joist (West) [Reduced Live Load At 400 ft2] [B -- E]
23 - Point (lb)	46' (Front)	N/A	368	368	552	230	Roof Joist (West) [Reduced Live Load At 400 ft2] [B -- E]
24 - Point (lb)	48' (Front)	N/A	368	368	552	230	Roof Joist (West) [Reduced Live Load At 400 ft2] [B -- E]
25 - Point (lb)	50' (Front)	N/A	368	368	552	230	Roof Joist (West) [Reduced Live Load At 400 ft2] [B -- E]
26 - Point (lb)	52' (Front)	N/A	368	368	552	230	Roof Joist (West) [Reduced Live Load At 400 ft2] [B -- E]
27 - Point (lb)	54' (Front)	N/A	368	368	552	230	Roof Joist (West) [Reduced Live Load At 400 ft2] [B -- E]
28 - Point (lb)	56' (Front)	N/A	368	368	552	230	Roof Joist (West) [Reduced Live Load At 400 ft2] [B -- E]
29 - Point (lb)	58' (Front)	N/A	368	368	552	230	Roof Joist (West) [Reduced Live Load At 400 ft2] [B -- E]
30 - Point (lb)	60' (Front)	N/A	368	460	552	230	Roof Joist (West) [E -- F]
31 - Point (lb)	62' (Front)	N/A	368	460	552	230	Roof Joist (West) [E -- F]
32 - Point (lb)	64' (Front)	N/A	368	460	552	230	Roof Joist (West) [E -- F]
33 - Point (lb)	66' (Front)	N/A	368	460	552	230	Roof Joist (West) [E -- F]
34 - Point (lb)	68' (Front)	N/A	368	460	552	230	Roof Joist (West) [E -- F]
35 - Point (lb)	70' (Front)	N/A	368	460	552	230	Roof Joist (West) [E -- F]

• Side loads are assumed to not induce cross-grain tension.

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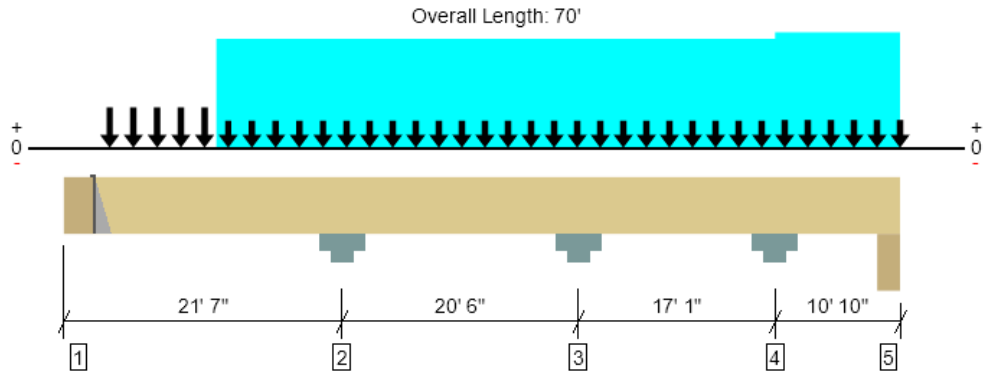
The product application, input design loads, dimensions and support information have been provided by ForteWEB Software Operator

Commercial Bldg. - Roof Beams, 2-sided Point Continuous 2-Ply - Snow balanced min. & Wind pressure

2 piece(s) 3 1/2" x 16" 2.2E Parallam® PSL



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

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	6372 @ 7"	6563 (1.50")	Passed (97%)	--	1.0 D + 0.45 W + 0.75 L + 0.525 S (Alt Spans)
Shear (lbs)	8824 @ 19' 9 1/2"	24901	Passed (35%)	1.15	1.0 D + 0.7 S (Adj Spans)
Moment (Ft-lbs)	-36740 @ 21' 7"	80396	Passed (46%)	1.15	1.0 D + 0.7 S (Adj Spans)
Live Load Defl. (in)	0.223 @ 10' 3 5/8"	0.525	Passed (L/999+)	--	1.0 D + 0.45 W + 0.75 L + 0.525 S (Alt Spans)
Total Load Defl. (in)	0.408 @ 10' 1"	1.050	Passed (L/618)	--	1.0 D + 0.45 W + 0.75 L + 0.525 S (Alt Spans)

Member Length : 69' 5"
System : Floor
Member Type : Drop Beam
Building Use : Commercial
Building Code : IBC 2024
Design Methodology : ASD

- Deflection criteria: LL (L/480) and TL (L/240).
- Moment capacity has been adjusted by a factor of 0.97 to account for the beam stability and/or volume/size factors.
- Member should be side-loaded from both sides of the member or braced to prevent rotation.

Supports	Bearing Length			Loads to Supports (lbs)					Accessories
	Total	Available	Required	Dead	Roof Live	Snow	Wind	Factored	
1 - Hanger on 16" DF beam	7.00"	Hanger[1]	1.50"	3118	2999	4499	1981/-213	6372	See note [1]
2 - Column Cap - steel	11.00"	11.00"	4.31"	9443	8788	13169	5595	18874	None
3 - Column Cap - steel	11.00"	11.00"	3.45"	7259	7146	10753	4826	15076	None
4 - Column Cap - steel	11.00"	11.00"	2.88"	5964	6476	8866	3991	12617	None
5 - Column - DF	5.50"	5.50"	1.50"	1669	2402	2853	1416/-455	4108	None

- At hanger supports, the Total Bearing dimension is equal to the width of the material that is supporting the hanger
- [1] See Connector grid below for additional information and/or requirements.

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	Continuous	
Bottom Edge (Lu)	2' o/c	

Connector: Simpson Strong-Tie

Support	Model	Seat Length	Top Fasteners	Face Fasteners	Member Fasteners	Accessories
1 - Top Mount Hanger	HGLTV416-2	6.00"	6-16d	12-16d	6-16d	

- Refer to manufacturer notes and instructions for proper installation and use of all connectors.

Multiple Member Connections

Type	Location	Fastener	Placement	Rows	O.C.	# of Fasteners	Details
Uniform	7" to 21' 7"	Strong-Drive® SDS Screw SDS25600 (6")	Both Faces	2	24"	--	L17
Uniform	21' 7" to 42' 1"	Strong-Drive® SDS Screw SDS25600 (6")	Both Faces	2	24"	--	L17
Uniform	42' 1" to 59' 2"	Strong-Drive® SDS Screw SDS25600 (6")	Both Faces	2	24"	--	L17
Uniform	59' 2" to 70'	Strong-Drive® SDS Screw SDS25600 (6")	Both Faces	2	24"	--	L17

Vertical Loads	Location (Side)	Tributary Width	Dead (0.90)	Roof Live (1.25)	Snow (1.15)	Wind (1.60)	Comments
0 - Self Weight (PLF)	7" to 70'	N/A	35.0	--	--	--	

ForteWEB Software Operator	Job Notes
Mohammad Aljazzar California Building Engineers Inc. (510) 674-5270 mohammad@cbestructural.com	

Weyerhaeuser

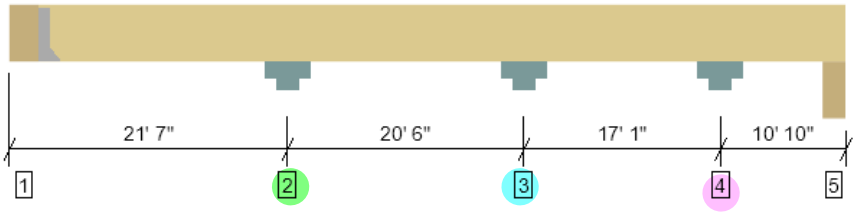
6/20/2025 6:40:12 AM UTC
ForteWEB v3.9, Engine: V8.4.3.94, Data: V8.1.7.3
File Name: C & S Restaurant 29 Palms Hwy Yucca Valley

Vertical Loads	Location (Side)	Tributary Width	Dead (0.90)	Roof Live (1.25)	Snow (1.15)	Wind (1.60)	Comments
1 - Uniform (PSF)	11' to 59' 2" (Back)	11'	16.0	16.0	24.0	10.0	Roof Joist (East) [Reduced Live Load At 400 ft2] [B -- E]
2 - Uniform (PSF)	59' 2" to 70' (Back)	11'	16.0	20.0	24.0	10.0	Roof Joist (East) [E -- F]
3 - Point (lb)	2' (Front)	N/A	368	368	552	230	Roof Joist (West) [Reduced Live Load At 400 ft2] [B -- E]
4 - Point (lb)	4' (Front)	N/A	368	368	552	230	Roof Joist (West) [Reduced Live Load At 400 ft2] [B -- E]
5 - Point (lb)	6' (Front)	N/A	368	368	552	230	Roof Joist (West) [Reduced Live Load At 400 ft2] [B -- E]
6 - Point (lb)	8' (Front)	N/A	368	368	552	230	Roof Joist (West) [Reduced Live Load At 400 ft2] [B -- E]
7 - Point (lb)	10' (Front)	N/A	368	368	552	230	Roof Joist (West) [Reduced Live Load At 400 ft2] [B -- E]
8 - Point (lb)	12' (Front)	N/A	368	368	552	230	Roof Joist (West) [Reduced Live Load At 400 ft2] [B -- E]
9 - Point (lb)	14' (Front)	N/A	368	368	552	230	Roof Joist (West) [Reduced Live Load At 400 ft2] [B -- E]
10 - Point (lb)	16' (Front)	N/A	368	368	552	230	Roof Joist (West) [Reduced Live Load At 400 ft2] [B -- E]
11 - Point (lb)	18' (Front)	N/A	368	368	552	230	Roof Joist (West) [Reduced Live Load At 400 ft2] [B -- E]
12 - Point (lb)	20' (Front)	N/A	368	368	552	230	Roof Joist (West) [Reduced Live Load At 400 ft2] [B -- E]
13 - Point (lb)	22' (Front)	N/A	368	368	552	230	Roof Joist (West) [Reduced Live Load At 400 ft2] [B -- E]
14 - Point (lb)	24' (Front)	N/A	368	368	552	230	Roof Joist (West) [Reduced Live Load At 400 ft2] [B -- E]
15 - Point (lb)	26' (Front)	N/A	368	368	552	230	Roof Joist (West) [Reduced Live Load At 400 ft2] [B -- E]
16 - Point (lb)	28' (Front)	N/A	368	368	552	230	Roof Joist (West) [Reduced Live Load At 400 ft2] [B -- E]
17 - Point (lb)	30' (Front)	N/A	368	368	552	230	Roof Joist (West) [Reduced Live Load At 400 ft2] [B -- E]
18 - Point (lb)	32' (Front)	N/A	368	368	552	230	Roof Joist (West) [Reduced Live Load At 400 ft2] [B -- E]
19 - Point (lb)	34' (Front)	N/A	368	368	552	230	Roof Joist (West) [Reduced Live Load At 400 ft2] [B -- E]
20 - Point (lb)	36' (Front)	N/A	368	368	552	230	Roof Joist (West) [Reduced Live Load At 400 ft2] [B -- E]
21 - Point (lb)	38' (Front)	N/A	368	368	552	230	Roof Joist (West) [Reduced Live Load At 400 ft2] [B -- E]

Vertical Loads	Location (Side)	Tributary Width	Dead (0.90)	Roof Live (1.25)	Snow (1.15)	Wind (1.60)	Comments
22 - Point (lb)	40' (Front)	N/A	368	368	552	230	Roof Joist (West) [Reduced Live Load At 400 ft2] [B -- E]
23 - Point (lb)	42' (Front)	N/A	368	368	552	230	Roof Joist (West) [Reduced Live Load At 400 ft2] [B -- E]
24 - Point (lb)	44' (Front)	N/A	368	368	552	230	Roof Joist (West) [Reduced Live Load At 400 ft2] [B -- E]
25 - Point (lb)	46' (Front)	N/A	368	368	552	230	Roof Joist (West) [Reduced Live Load At 400 ft2] [B -- E]
26 - Point (lb)	48' (Front)	N/A	368	368	552	230	Roof Joist (West) [Reduced Live Load At 400 ft2] [B -- E]
27 - Point (lb)	50' (Front)	N/A	368	368	552	230	Roof Joist (West) [Reduced Live Load At 400 ft2] [B -- E]
28 - Point (lb)	52' (Front)	N/A	368	368	552	230	Roof Joist (West) [Reduced Live Load At 400 ft2] [B -- E]
29 - Point (lb)	54' (Front)	N/A	368	368	552	230	Roof Joist (West) [Reduced Live Load At 400 ft2] [B -- E]
30 - Point (lb)	56' (Front)	N/A	368	368	552	230	Roof Joist (West) [Reduced Live Load At 400 ft2] [B -- E]
31 - Point (lb)	58' (Front)	N/A	368	368	552	230	Roof Joist (West) [Reduced Live Load At 400 ft2] [B -- E]
32 - Point (lb)	60' (Front)	N/A	368	460	552	230	Roof Joist (West) [E -- F]
33 - Point (lb)	62' (Front)	N/A	368	460	552	230	Roof Joist (West) [E -- F]
34 - Point (lb)	64' (Front)	N/A	368	460	552	230	Roof Joist (West) [E -- F]
35 - Point (lb)	66' (Front)	N/A	368	460	552	230	Roof Joist (West) [E -- F]
36 - Point (lb)	68' (Front)	N/A	368	460	552	230	Roof Joist (West) [E -- F]
37 - Point (lb)	70' (Front)	N/A	368	460	552	230	Roof Joist (West) [E -- F]
38 - Point (lb)	2' (Back)	N/A	352	352	528	220	Roof Joist (East) [Reduced Live Load At 400 ft2] [B -- E]
39 - Point (lb)	4' (Back)	N/A	352	352	528	220	Roof Joist (East) [Reduced Live Load At 400 ft2] [B -- E]
40 - Point (lb)	6' (Back)	N/A	352	352	528	220	Roof Joist (East) [Reduced Live Load At 400 ft2] [B -- E]
41 - Point (lb)	8' (Back)	N/A	352	352	528	220	Roof Joist (East) [Reduced Live Load At 400 ft2] [B -- E]
42 - Point (lb)	10' (Back)	N/A	352	352	528	220	Roof Joist (East) [Reduced Live Load At 400 ft2] [B -- E]

• Side loads are assumed to not induce cross-grain tension.

Built-up Beam Reactions Summary:



Commercial Bldg. - Roof Beams, Continuous 1-Ply - Snow balanced min. & Wind pressure

Supports	Bearing Length			Loads to Supports (lbs)					Accessories
	Total	Available	Required	Dead	Roof Live	Snow	Wind	Factored	
1 - Hanger on 16" DF beam	7.00"	Hanger[1]	2.99"	3331	3358	5037	2207/-84	6968	See note [1]
2 - Column Cap - steel	11.00"	11.00"	8.44"	9023	8787	13167	5595	18453	None
3 - Column Cap - steel	11.00"	11.00"	6.75"	6945	7154	10764	4831	14770	None
4 - Column Cap - steel	11.00"	11.00"	5.64"	5697	6469	8866	3993	12345	None
5 - Column - DF	5.50"	5.50"	1.66"	1420	2171	2576	1301/-455	3633	None

Commercial Bldg. - Roof Beams, Continuous 2-Ply - Snow balanced min. & Wind pressure

Supports	Bearing Length			Loads to Supports (lbs)					Accessories
	Total	Available	Required	Dead	Roof Live	Snow	Wind	Factored	
1 - Hanger on 16" DF beam	7.00"	Hanger[1]	1.53"	3475	3358	5037	2207/-84	7113	See note [1]
2 - Column Cap - steel	11.00"	11.00"	4.31"	9441	8787	13167	5595	18871	None
3 - Column Cap - steel	11.00"	11.00"	3.45"	7267	7154	10764	4831	15092	None
4 - Column Cap - steel	11.00"	11.00"	2.88"	5961	6469	8866	3993	12609	None
5 - Column - DF	5.50"	5.50"	1.50"	1485	2171	2576	1301/-455	3699	None

EGQ7.25-SDS3 Down < ASD Down Capacity 18,600 **OK**

Downward < 48,265 **OK**

ECCQ.1-6 Down < ASD Down Capacity 38,500 **OK**

Commercial Bldg. - Roof Beams, Continuous 1-Ply - Dead min. & Wind Uplift

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Wind	Factored	
1 - Hanger on 16" DF beam	7.00"	Hanger[1]	1.50"	2136	31/-8358	2154/-3733	See note [1]
2 - Column Cap - steel	11.00"	11.00"	2.65"	5796	-16313	5796/-6310	None
3 - Column Cap - steel	11.00"	11.00"	2.04"	4462	-13196	4462/-5241	None
4 - Column Cap - steel	11.00"	11.00"	1.67"	3659	-13445	3659/-5872	None
5 - Column - DF	5.50"	5.50"	1.50"	912	1327/-5561	1708/-2790	None

EGQ7.25-SDS3 Uplift < ASD Uplift Capacity 7,670 **OK**

Column Cap:
CCQ7.1-8 SDS 2.5
ASD Capacity
Uplift = 6,785 lbs (Cd = 1.6)
Down = 48,265 (Cd = 1.0)

Uplift < 6,785 **OK**

ECCQ.1-6 Uplift < ASD Uplift Capacity 6,785 **OK**

Commercial Bldg. - Roof Beams, Continuous 2-Ply - Dead min. & Wind Uplift

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Wind	Factored	
1 - Hanger on 16" DF beam	7.00"	Hanger[1]	1.50"	2281	30/-8358	2299/-3646	See note [1]
2 - Column Cap - steel	11.00"	11.00"	1.50"	6214	-16310	6214/-6058	None
3 - Column Cap - steel	11.00"	11.00"	1.50"	4786	-13193	4786/-5044	None
4 - Column Cap - steel	11.00"	11.00"	1.50"	3908	-13387	3908/-5687	None
5 - Hanger on 16" DF beam	5.50"	Hanger[1]	1.50"	975	876/-5642	1500/-2800	See note [1]

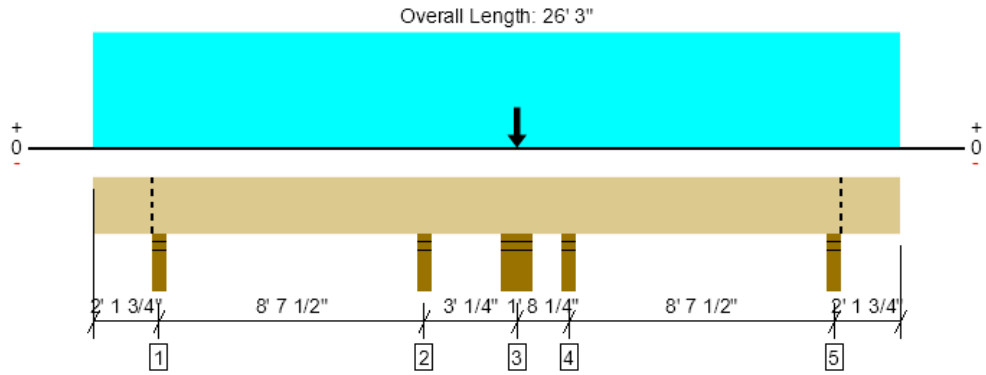
Uplift < 6,785 **OK**

E.3 Framing Design

PSL Header 7"X16" Line [B]

Commercial Bldg. - PSL Header Line B, with Post under Roof Beam

1 piece(s) 7" x 16" 2.2E Parallam® PSL



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

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	7163 @ 15' 5 3/4"	15313 (3.50")	Passed (47%)	--	1.0 D + 1.0 Lr (Adj Spans)
Shear (lbs)	4000 @ 15' 5 1/4"	24901	Passed (16%)	1.15	1.0 D + 0.7 S (Adj Spans)
Moment (Ft-lbs)	-4304 @ 15' 5 3/4"	80396	Passed (5%)	1.15	1.0 D + 0.7 S (Adj Spans)
Live Load Defl. (in)	0.006 @ 6' 1 1/4"	0.216	Passed (L/999+)	--	1.0 D + 1.0 Lr (Alt Spans)
Total Load Defl. (in)	0.012 @ 6' 1 5/8"	0.431	Passed (L/999+)	--	1.0 D + 1.0 Lr (Alt Spans)

Member Length : 26' 3"
System : Floor
Member Type : Flush Beam
Building Use : Commercial
Building Code : IBC 2024
Design Methodology : ASD

- Deflection criteria: LL (L/480) and TL (L/240).
- Overhang deflection criteria: LL (2L/480) and TL (2L/240).
- Moment capacity has been adjusted by a factor of 0.97 to account for the beam stability and/or volume/size factors.
- Member should be side-loaded from both sides of the member or braced to prevent rotation.

Supports	Bearing Length			Loads to Supports (lbs)					Accessories
	Total	Available	Required	Dead	Roof Live	Snow	Wind	Factored	
1 - Stud wall - DF	3.50"	3.50"	1.50"	1887	1727	2310	750	3614	Blocking
2 - Stud wall - DF	3.50"	3.50"	1.50"	2972	2837	3795	1282	5809	None
3 - Stud wall - DF	7.50"	7.50"	1.50"	1447	3064/-498	4644/-393	2753/-1083	5123	None
4 - Stud wall - DF	3.50"	3.50"	1.64"	3619	3544	4741	1639/-111	7163	None
5 - Stud wall - DF	3.50"	3.50"	1.50"	1872	1710	2288	742	3582	Blocking

• Blocking Panels are assumed to carry no loads applied directly above them and the full load is applied to the member being designed.

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	Continuous	
Bottom Edge (Lu)	All Bearing Points	

Vertical Loads	Location (Side)	Tributary Width	Dead (0.90)	Roof Live (1.25)	Snow (1.15)	Wind (1.60)	Comments
0 - Self Weight (PLF)	0 to 26' 3"	N/A	35.0	--	--	--	
1 - Uniform (PSF)	0 to 26' 3" (Top)	4' 6"	28.0	--	--	--	Parapet Wall
2 - Uniform (PSF)	0 to 26' 3" (Back)	2'	16.0	20.0	17.8	--	Main Roof Loading 2-ft trib.
3 - Uniform (PSF)	0 to 26' 3" (Back)	2'	--	--	20.5	--	Added Snow Drift 2-ft trib.
4 - Uniform (PLF)	0 to 26' 3" (Front)	N/A	124.0	249.0	310.0	125.0	Entrance Roof Joists Rxn [2] / 2ft
5 - Point (lb)	13' 9 1/2" (Back)	N/A	3475	3358	5037	2207	Built-up Roof Beam Rxn [1]

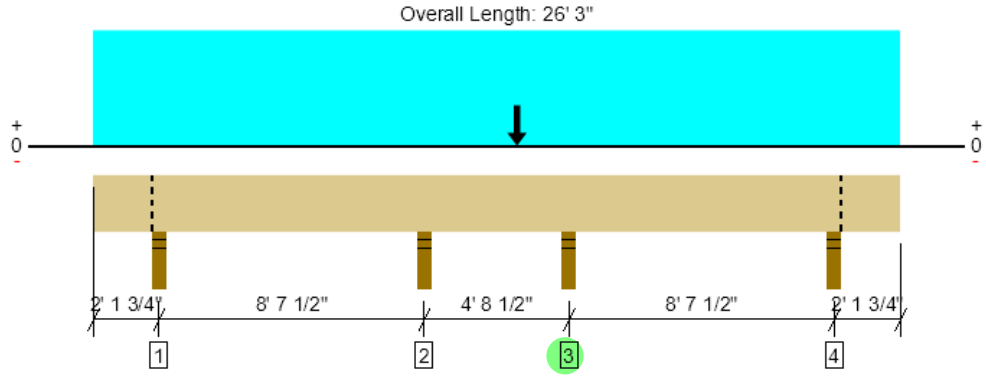
• Side loads are assumed to not induce cross-grain tension.

ForteWEB Software Operator	Job Notes
Mohammad Aljazzar California Building Engineers Inc. (510) 674-5270 mohammad@cbestructural.com	

Weyerhaeuser

Commercial Bldg. - PSL Header Line B, NO Post under Roof Beam

1 piece(s) 7" x 16" 2.2E Parallam® PSL



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

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	9310 @ 15' 5 3/4"	15313 (3.50")	Passed (61%)	--	1.0 D + 0.45 W + 0.75 L + 0.75 Lr (Adj Spans)
Shear (lbs)	5340 @ 14'	24901	Passed (21%)	1.15	1.0 D + 0.7 S (Adj Spans)
Moment (Ft-lbs)	-5310 @ 15' 5 3/4"	80396	Passed (7%)	1.15	1.0 D + 0.7 S (Adj Spans)
Live Load Defl. (in)	0.006 @ 13' 9 1/2"	0.118	Passed (L/999+)	--	1.0 D + 0.45 W + 0.75 L + 0.525 S (Alt Spans)
Total Load Defl. (in)	0.011 @ 13' 9 1/2"	0.235	Passed (L/999+)	--	1.0 D + 0.45 W + 0.75 L + 0.525 S (Alt Spans)

Member Length : 26' 3"
System : Floor
Member Type : Flush Beam
Building Use : Commercial
Building Code : IBC 2024
Design Methodology : ASD

- Deflection criteria: LL (L/480) and TL (L/240).
- Overhang deflection criteria: LL (2L/480) and TL (2L/240).
- Moment capacity has been adjusted by a factor of 0.97 to account for the beam stability and/or volume/size factors.
- Member should be side-loaded from both sides of the member or braced to prevent rotation.

Maximum Vertical Loads

Supports	Bearing Length			Loads to Supports (lbs)					Accessories
	Total	Available	Required	Dead	Roof Live	Snow	Wind	Factored	
1 - Stud wall - DF	3.50"	3.50"	1.50"	1852	1741	2322	778	3593	Blocking
2 - Stud wall - DF	3.50"	3.50"	1.62"	3506	3588	5001	1969	7094	None
3 - Stud wall - DF	3.50"	3.50"	2.13"	4613	4658	6606	2673	9310	None
4 - Stud wall - DF	3.50"	3.50"	1.50"	1825	1728	2303	778	3553	Blocking

• Blocking Panels are assumed to carry no loads applied directly above them and the full load is applied to the member being designed.

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	Continuous	
Bottom Edge (Lu)	All Bearing Points	

Vertical Loads	Location (Side)	Tributary Width	Dead (0.90)	Roof Live (1.25)	Snow (1.15)	Wind (1.60)	Comments
0 - Self Weight (PLF)	0 to 26' 3"	N/A	35.0	--	--	--	
1 - Uniform (PSF)	0 to 26' 3" (Top)	4' 6"	28.0	--	--	--	Parapet Wall
2 - Uniform (PSF)	0 to 26' 3" (Back)	2'	16.0	20.0	17.8	--	Main Roof Loading 2-ft trib.
3 - Uniform (PSF)	0 to 26' 3" (Back)	2'	--	--	20.5	--	Added Snow Drift 2-ft trib.
4 - Uniform (PLF)	0 to 26' 3" (Front)	N/A	124.0	249.0	310.0	125.0	Entrance Roof Joists Rxn [2] / 2ft
5 - Point (lb)	13' 9 1/2" (Back)	N/A	3475	3358	5037	2207	Built-up Roof Beam Rxn [1]

• Side loads are assumed to not induce cross-grain tension.

ForteWEB Software Operator	Job Notes
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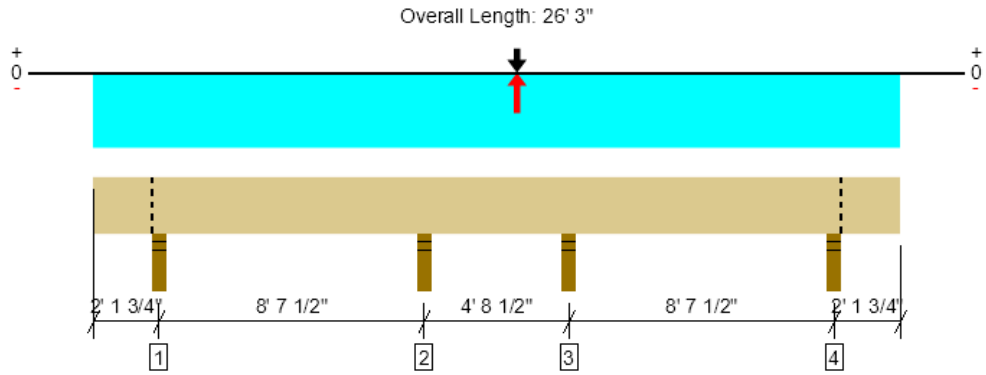
Weyerhaeuser

Commercial Bldg. - PSL Header Line B, Uplift - NO Post under Roof Beam

1 piece(s) 7" x 16" 2.2E Parallam® PSL



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

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	3238 @ 15' 5 3/4"	15313 (3.50")	Passed (21%)	--	1.0 D (All Spans)
Shear (lbs)	1704 @ 14'	19488	Passed (9%)	0.90	1.0 D (All Spans)
Moment (Ft-lbs)	-3793 @ 13' 9 1/2"	111855	Passed (3%)	1.60	0.6 D + 0.6 W (Alt Spans)
Live Load Defl. (in)	0.004 @ 0	0.200	Passed (2L/999+)	--	1.0 D + 0.6 W (Alt Spans)
Total Load Defl. (in)	-0.006 @ 13' 9 1/2"	0.235	Passed (L/999+)	--	0.6 D + 0.6 W (Alt Spans)

Member Length : 26' 3"
System : Floor
Member Type : Flush Beam
Building Use : Commercial
Building Code : IBC 2024
Design Methodology : ASD

- Deflection criteria: LL (L/480) and TL (L/240).
- Overhang deflection criteria: LL (0.2") and TL (2L/240).
- Moment capacity has been adjusted by a factor of 0.97 to account for the beam stability and/or volume/size factors.
- An excessive uplift of -1143 lbs detected at support located at 2' 1 3/4".
- An excessive uplift of -3121 lbs detected at support located at 10' 9 1/4".
- An excessive uplift of -4139 lbs detected at support located at 15' 5 3/4".
- An excessive uplift of -1154 lbs detected at support located at 24' 1 1/4".
- Member should be side-loaded from both sides of the member or braced to prevent rotation.

Use LSTA30 W/
reduced design value for LSTA24 [ASD 1,235 lbs Uplift] min (14) 10dx2.5" nails

Use Inverted MSTC66B3Z [ASD 4,490 lbs Uplift]

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Wind	Factored	
1 - Stud wall - DF	3.50"	3.50"	1.50"	1398	-3304	1398/-1143	Blocking
2 - Stud wall - DF	3.50"	3.50"	1.50"	2511	-7713	2511/-3121	None
3 - Stud wall - DF	3.50"	3.50"	1.50"	3238	-10137	3238/-4139	None
4 - Stud wall - DF	3.50"	3.50"	1.50"	1381	-3304	1381/-1154	Blocking

• Blocking Panels are assumed to carry no loads applied directly above them and the full load is applied to the member being designed.

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	Continuous	
Bottom Edge (Lu)	All Bearing Points	

Vertical Loads	Location (Side)	Tributary Width	Dead (0.90)	Wind (1.60)	Comments
0 - Self Weight (PLF)	0 to 26' 3"	N/A	35.0	--	
1 - Uniform (PSF)	0 to 26' 3" (Top)	4' 6"	20.0	--	Parapet Wall
2 - Uniform (PSF)	0 to 26' 3" (Back)	2'	10.0	--	Main Roof Loading 2-ft trib. Min. Dead Load
3 - Uniform (PLF)	0 to 26' 3" (Front)	N/A	93.0	-531.0	Entrance Roof Joists Rxn [2] / 2ft min. Dead + w uplift 9% Decrease for EWA
4 - Point (lb)	13' 9 1/2" (Back)	N/A	2281	-7605	Built-up Roof Beam Rxn [1]

• Side loads are assumed to not induce cross-grain tension.

ForteWEB Software Operator	Job Notes
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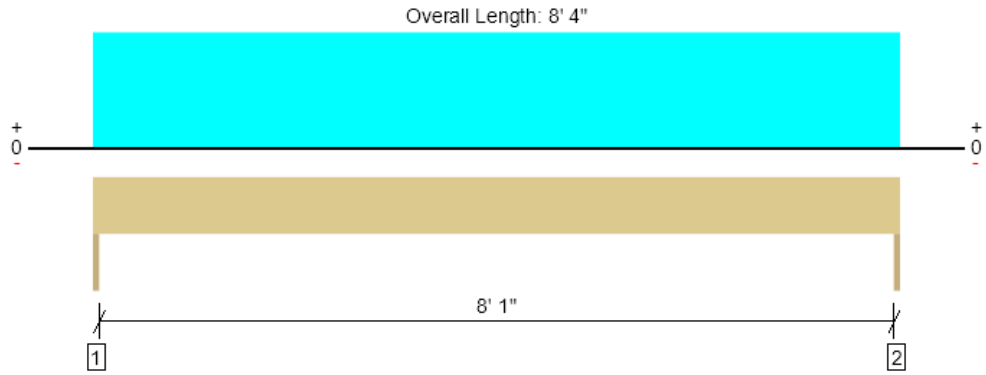
Weyerhaeuser

5/23/2025 2:42:27 AM UTC
ForteWEB v3.9, Engine: V8.4.3.94, Data: V8.1.7.3
File Name: C & S Restaurant 29 Palms Hwy Yucca Valley

E.4 Framing Design (Headers)

Commercial Bldg. - Headers, Wall: Header 8'-0" Span [Line B]

1 piece(s) 6 x 8 DF No.2



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

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	269 @ 0	5156 (1.50")	Passed (5%)	--	1.0 D (All Spans)
Shear (lbs)	220 @ 9"	4208	Passed (5%)	0.90	1.0 D (All Spans)
Moment (Ft-lbs)	559 @ 4' 2"	2892	Passed (19%)	0.90	1.0 D (All Spans)
Vert Live Load Defl. (in)	0.000 @ 0	0.208	Passed (2L/999+)	--	1.0 D (All Spans)
Vert Total Load Defl. (in)	0.028 @ 4' 2"	0.417	Passed (L/999+)	--	1.0 D (All Spans)
Lat Member Reaction (lbs)	322 @ 8' 4"	N/A	Passed (N/A)	1.60	1.0 D + 0.6 W
Lat Shear (lbs)	277 @ 7"	7480	Passed (4%)	1.60	1.0 D + 0.6 W
Lat Moment (Ft-lbs)	671 @ mid-span	3781	Passed (18%)	1.60	1.0 D + 0.6 W
Lat Deflection (in)	0.043 @ mid-span	0.278	Passed (L/999+)	--	1.0 D + 0.6 W
Bi-Axial Bending	0.29	1.00	Passed (29%)	1.60	1.0 D + 0.6 W

Member Length : 8' 4"
System : Wall
Member Type : Header
Building Use : Commercial
Building Code : IBC 2024
Design Methodology : ASD

- Deflection criteria: LL (L/480) and TL (L/240).
- Wall deflection criteria: TL (L/360)
- Moment capacity has been adjusted by a factor of 0.85 to account for the beam stability and/or volume/size factors.
- Applicable calculations are based on NDS.

Supports	Bearing Length			Loads to Supports (lbs)		Accessories
	Total	Available	Required	Dead	Factored	
1 - Trimmer - DF	1.50"	1.50"	1.50"	269	269	None
2 - Trimmer - DF	1.50"	1.50"	1.50"	269	269	None

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	End Bearing Points	
Bottom Edge (Lu)	End Bearing Points	

Lateral Connections						
Supports	Stud Size	Stud Material	Connector	Type/Model	Quantity	Nailing
Left	2X	Douglas Fir-Larch	Nails	8d (0.113" x 2 1/2") (Toe)	4	
Right	2X	Douglas Fir-Larch	Nails	8d (0.113" x 2 1/2") (Toe)	4	

Vertical Loads	Location	Tributary Width	Dead (0.90)	Comments
0 - Self Weight (PLF)	0 to 8' 4"	N/A	10.4	
1 - Uniform (PSF)	0 to 8' 4"	3'	18.0	Wall Above Header (Note PSL Full Length Header Abv.)

ForteWEB Software Operator	Job Notes
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Weyerhaeuser

6/3/2025 5:55:22 AM UTC
ForteWEB v3.9, Engine: V8.4.3.94, Data: V8.1.7.3
File Name: C & S Restaurant 29 Palms Hwy Yucca Valley

Lateral Load	Location	Tributary Width	Wind (1.60)	Comments
1 - Uniform (PSF)	Full Length	4'	32.2	Zone 5 + Internal [27.9 + 4.3]

• IBC Table 1604.3, footnote f: Deflection checks are performed using 42% of this lateral wind load.

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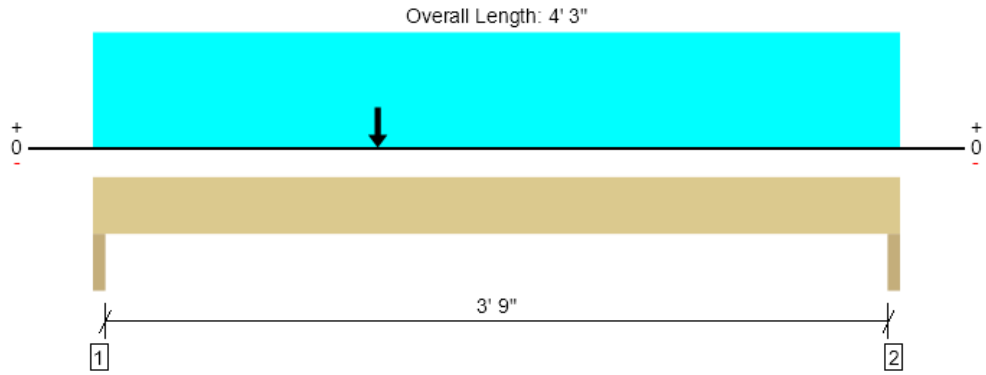
The product application, input design loads, dimensions and support information have been provided by ForteWEB Software Operator

Commercial Bldg. - Headers, Wall: Header 3'-8" Span [Line B]

1 piece(s) 6 x 12 DF No.2



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

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	7592 @ 1 1/2"	10313 (3.00")	Passed (74%)	--	1.0 D + 1.0 Lr (All Spans)
Shear (lbs)	6336 @ 1' 2 1/2"	8244	Passed (77%)	1.15	1.0 D + 0.7 S (All Spans)
Moment (Ft-lbs)	9023 @ 1' 6"	10129	Passed (89%)	1.15	1.0 D + 0.7 S (All Spans)
Vert Live Load Defl. (in)	0.013 @ 2' 13/16"	0.100	Passed (L/999+)	--	1.0 D + 1.0 Lr (All Spans)
Vert Total Load Defl. (in)	0.025 @ 2' 13/16"	0.200	Passed (L/999+)	--	1.0 D + 1.0 Lr (All Spans)
Lat Member Reaction (lbs)	232 @ 4' 1 1/2"	N/A	Passed (N/A)	1.60	1.0 D + 0.6 W
Lat Shear (lbs)	164 @ 8 1/2"	11469	Passed (1%)	1.60	1.0 D + 0.6 W
Lat Moment (Ft-lbs)	232 @ mid-span	6764	Passed (3%)	1.60	1.0 D + 0.6 W
Lat Deflection (in)	0.002 @ mid-span	0.133	Passed (L/999+)	--	1.0 D + 0.6 W
Bi-Axial Bending	0.63	1.00	Passed (63%)	1.60	1.0 D + 0.45 W + 0.75 L + 0.75 Lr

Member Length : 4' 3"
System : Wall
Member Type : Header
Building Use : Commercial
Building Code : IBC 2024
Design Methodology : ASD

- Deflection criteria: LL (L/480) and TL (L/240).
- Wall deflection criteria: TL (L/360)
- Moment capacity has been adjusted by a factor of 1 to account for the beam stability and/or volume/size factors.
- Lumber grading provisions must be extended over the length of the member per NDS 4.2.5.5.
- Applicable calculations are based on NDS.

Supports	Bearing Length			Loads to Supports (lbs)				Accessories
	Total	Available	Required	Dead	Roof Live	Snow	Factored	
1 - Trimmer - DF	3.00"	3.00"	2.21"	3815	3776	4590	7592	None
2 - Trimmer - DF	3.00"	3.00"	1.50"	2329	2251	2760	4581	None

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	End Bearing Points	
Bottom Edge (Lu)	End Bearing Points	

Lateral Connections						
Supports	Stud Size	Stud Material	Connector	Type/Model	Quantity	Nailing
Left	2X	Douglas Fir-Larch	Nails	8d (0.113" x 2 1/2") (Toe)	3	
Right	2X	Douglas Fir-Larch	Nails	8d (0.113" x 2 1/2") (Toe)	3	

ForteWEB Software Operator	Job Notes
Mohammad Aljazzar California Building Engineers Inc. (510) 674-5270 mohammad@cbestructural.com	

Weyerhaeuser

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ForteWEB v3.9, Engine: V8.4.3.94, Data: V8.1.7.3
File Name: C & S Restaurant 29 Palms Hwy Yucca Valley

Vertical Loads	Location	Tributary Width	Dead (0.90)	Roof Live (1.25)	Snow (1.15)	Comments
0 - Self Weight (PLF)	0 to 4' 3"	N/A	16.0	--	--	
1 - Uniform (PSF)	0 to 4' 3"	4'	18.0	--	--	Wall Above (Internal)
2 - Uniform (PSF)	0 to 4' 3"	3'	28.0	--	--	Parapet Above (Stucco Both Sides)
3 - Uniform (PLF)	0 to 4' 3"	N/A	137.0	250.0	313.0	Entrance Roof
4 - Uniform (PLF)	0 to 4' 3"	N/A	18.0	20.0	38.3	Main Roof
5 - Point (lb)	1' 6"	N/A	4755	4880	5856	Roof Beam Reaction

Lateral Load	Location	Tributary Width	Wind (1.60)	Comments
1 - Uniform (PSF)	Full Length	6'	32.2	Zone 5 + Internal [27.9 + 4.3]

• IBC Table 1604.3, footnote f: Deflection checks are performed using 42% of this lateral wind load.

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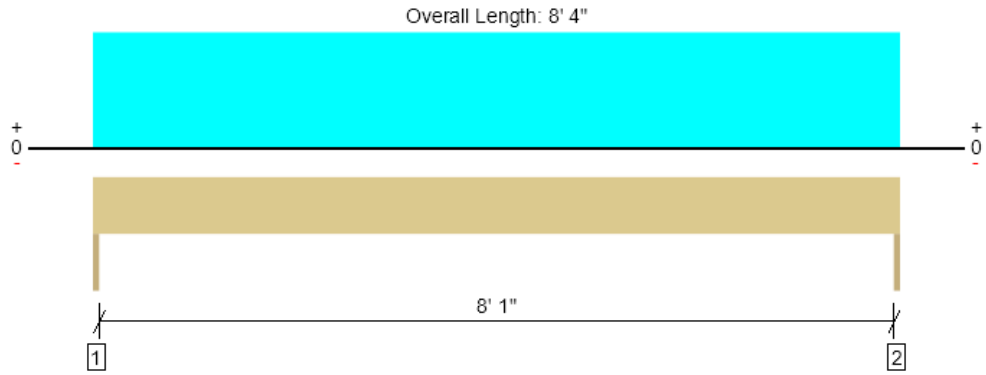
The product application, input design loads, dimensions and support information have been provided by ForteWEB Software Operator

Commercial Bldg. - Headers, Wall: Header 8'-0" Span [Line B] Balloon Frame

1 piece(s) 6 x 10 DF No.2



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

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	2734 @ 0	5156 (1.50")	Passed (53%)	--	1.0 D + 1.0 Lr (All Spans)
Shear (lbs)	2073 @ 11"	6810	Passed (30%)	1.15	1.0 D + 0.7 S (All Spans)
Moment (Ft-lbs)	5537 @ 4' 2"	6896	Passed (80%)	1.15	1.0 D + 0.7 S (All Spans)
Vert Live Load Defl. (in)	0.061 @ 4' 2"	0.208	Passed (L/999+)	--	1.0 D + 1.0 Lr (All Spans)
Vert Total Load Defl. (in)	0.139 @ 4' 2"	0.417	Passed (L/717)	--	1.0 D + 1.0 Lr (All Spans)
Lat Member Reaction (lbs)	322 @ 8' 4"	N/A	Passed (N/A)	1.60	1.0 D + 0.6 W
Lat Shear (lbs)	277 @ 7"	9475	Passed (3%)	1.60	1.0 D + 0.6 W
Lat Moment (Ft-lbs)	671 @ mid-span	5588	Passed (12%)	1.60	1.0 D + 0.6 W
Lat Deflection (in)	0.034 @ mid-span	0.278	Passed (L/999+)	--	1.0 D + 0.6 W
Bi-Axial Bending	0.68	1.00	Passed (68%)	1.60	1.0 D + 0.45 W + 0.75 L + 0.75 Lr

Member Length : 8' 4"
System : Wall
Member Type : Header
Building Use : Commercial
Building Code : IBC 2024
Design Methodology : ASD

- Deflection criteria: LL (L/480) and TL (L/240).
- Wall deflection criteria: TL (L/360)
- Moment capacity has been adjusted by a factor of 0.99 to account for the beam stability and/or volume/size factors.
- Lumber grading provisions must be extended over the length of the member per NDS 4.2.5.5.
- Applicable calculations are based on NDS.

Supports	Bearing Length			Loads to Supports (lbs)					Accessories
	Total	Available	Required	Dead	Roof Live	Snow	Wind	Factored	
1 - Trimmer - DF	1.50"	1.50"	1.50"	1530	1204	1611	604	2734	None
2 - Trimmer - DF	1.50"	1.50"	1.50"	1530	1204	1611	604	2734	None

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	End Bearing Points	
Bottom Edge (Lu)	End Bearing Points	

Lateral Connections						
Supports	Stud Size	Stud Material	Connector	Type/Model	Quantity	Nailing
Left	2X	Douglas Fir-Larch	Nails	8d (0.113" x 2 1/2") (Toe)	4	
Right	2X	Douglas Fir-Larch	Nails	8d (0.113" x 2 1/2") (Toe)	4	

ForteWEB Software Operator	Job Notes
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File Name: C & S Restaurant 29 Palms Hwy Yucca Valley

Vertical Loads	Location	Tributary Width	Dead (0.90)	Roof Live (1.25)	Snow (1.15)	Wind (1.60)	Comments
0 - Self Weight (PLF)	0 to 8' 4"	N/A	13.2	--	--	--	
1 - Uniform (PSF)	0 to 8' 4"	4'	18.0	--	--	--	Wall Above Header (Note PSL Full Length Header Abv.)
2 - Uniform (PSF)	0 to 8' 4"	4' 6"	28.0	--	--	--	Parapet Portion Abv. Stucco Both Sides
3 - Uniform (PLF)	0 to 8' 4"	N/A	124.0	249.0	310.0	125.0	Entrance Rafters
4 - Uniform (PSF)	0 to 8' 4"	2'	16.0	20.0	17.8	10.0	Main Roof Trib. 2-ft
5 - Uniform (PSF)	0 to 8' 4"	2'	--	--	20.5	--	Snow Drift Added 2-ft Trib.

Lateral Load	Location	Tributary Width	Wind (1.60)	Comments
1 - Uniform (PSF)	Full Length	4'	32.2	Zone 5 + Internal [27.9 + 4.3]

• IBC Table 1604.3, footnote f: Deflection checks are performed using 42% of this lateral wind load.

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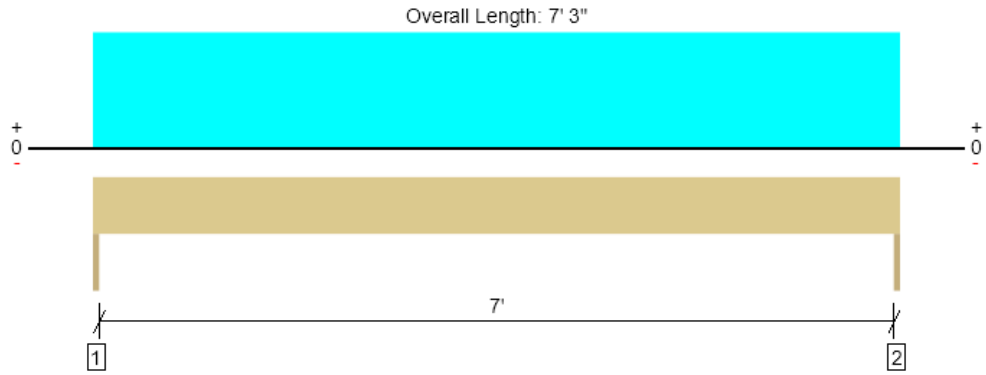
The product application, input design loads, dimensions and support information have been provided by ForteWEB Software Operator

Commercial Bldg. - Headers, Wall: Header 7'-0" Span [Line A]

1 piece(s) 6 x 8 DF No.2



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

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	1894 @ 0	5156 (1.50")	Passed (37%)	--	1.0 D + 1.0 Lr (All Spans)
Shear (lbs)	1502 @ 9"	5844	Passed (26%)	1.25	1.0 D + 1.0 Lr (All Spans)
Moment (Ft-lbs)	3433 @ 3' 7 1/2"	4014	Passed (86%)	1.25	1.0 D + 1.0 Lr (All Spans)
Vert Live Load Defl. (in)	0.078 @ 3' 7 1/2"	0.181	Passed (L/999+)	--	1.0 D + 1.0 Lr (All Spans)
Vert Total Load Defl. (in)	0.129 @ 3' 7 1/2"	0.363	Passed (L/673)	--	1.0 D + 1.0 Lr (All Spans)
Lat Member Reaction (lbs)	315 @ 7' 3"	N/A	Passed (N/A)	1.60	1.0 D + 0.6 W
Lat Shear (lbs)	264 @ 7"	7480	Passed (4%)	1.60	1.0 D + 0.6 W
Lat Moment (Ft-lbs)	571 @ mid-span	3781	Passed (15%)	1.60	1.0 D + 0.6 W
Lat Deflection (in)	0.028 @ mid-span	0.242	Passed (L/999+)	--	1.0 D + 0.6 W
Bi-Axial Bending	0.76	1.00	Passed (76%)	1.60	1.0 D + 0.45 W + 0.75 L + 0.75 Lr

Member Length : 7' 3"
System : Wall
Member Type : Header
Building Use : Commercial
Building Code : IBC 2024
Design Methodology : ASD

- Deflection criteria: LL (L/480) and TL (L/240).
- Wall deflection criteria: TL (L/360)
- Moment capacity has been adjusted by a factor of 0.85 to account for the beam stability and/or volume/size factors.
- Applicable calculations are based on NDS.

Supports	Bearing Length			Loads to Supports (lbs)					Accessories
	Total	Available	Required	Dead	Roof Live	Snow	Wind	Factored	
1 - Trimmer - DF	1.50"	1.50"	1.50"	745	1149	1378	508	1894	None
2 - Trimmer - DF	1.50"	1.50"	1.50"	745	1149	1378	508	1894	None

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	End Bearing Points	
Bottom Edge (Lu)	End Bearing Points	

Lateral Connections						
Supports	Stud Size	Stud Material	Connector	Type/Model	Quantity	Nailing
Left	2X	Douglas Fir-Larch	Nails	8d (0.113" x 2 1/2") (Toe)	4	
Right	2X	Douglas Fir-Larch	Nails	8d (0.113" x 2 1/2") (Toe)	4	

Vertical Loads	Location	Tributary Width	Dead (0.90)	Roof Live (1.25)	Snow (1.15)	Wind (1.60)	Comments
0 - Self Weight (PLF)	0 to 7' 3"	N/A	10.4	--	--	--	
1 - Uniform (PSF)	0 to 7' 3"	2'	18.0	--	--	--	Wall Above Header (Note PSL Full Length Header Abv.)
2 - Uniform (PLF)	0 to 7' 3"	N/A	159.0	317.0	380.0	140.0	Entrance Roof Joist Rxn (1)

ForteWEB Software Operator	Job Notes
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File Name: C & S Restaurant 29 Palms Hwy Yucca Valley

Lateral Load	Location	Tributary Width	Wind (1.60)	Comments
1 - Uniform (PSF)	Full Length	4' 6"	32.2	Zone 5 + Internal [27.9 + 4.3]

• IBC Table 1604.3, footnote f: Deflection checks are performed using 42% of this lateral wind load.

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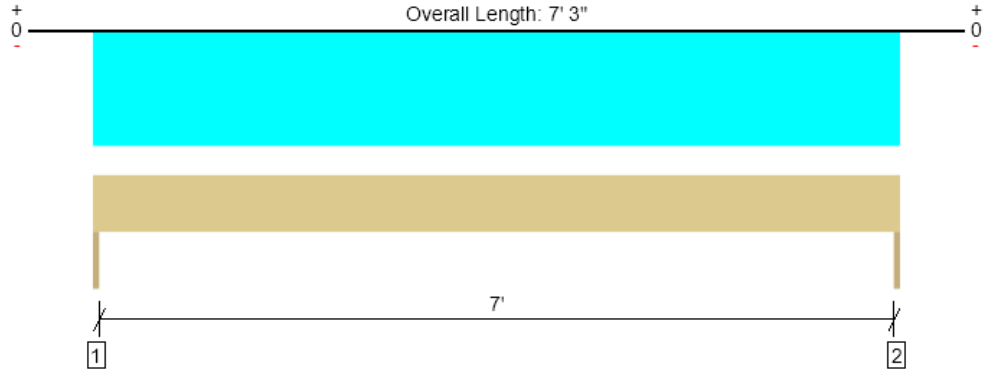
Commercial Bldg. - Headers, Wall: Header 7'-0" Span [Line A] Uplift

1 piece(s) 6 x 8 DF No.2

An excessive uplift of -1276 lbs at support located at 0"

An excessive uplift of -1276 lbs at support located at 7' 3"

MSTA18 [ASD Uplift capacity = 1315 lbs]
Header to Trimmers, see plan and elevations
Trimmers to Footing with HDU-2 [ASD Uplift 3,075 lbs]



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

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	545 @ 0	5156 (1.50")	Passed (11%)	--	1.0 D (All Spans)
Shear (lbs)	1012 @ 9"	7480	Passed (14%)	1.60	0.6 D + 0.6 W (All Spans)
Moment (Ft-lbs)	-2312 @ 3' 7 1/2"	5132	Passed (45%)	1.60	0.6 D + 0.6 W (All Spans)
Vert Live Load Defl. (in)	0.000 @ 0	0.181	Passed (2L/999+)	--	1.0 D (All Spans)
Vert Total Load Defl. (in)	-0.087 @ 3' 7 1/2"	0.363	Passed (L/999+)	--	0.6 D + 0.6 W (All Spans)
Lat Member Reaction (lbs)	315 @ 7' 3"	N/A	Passed (N/A)	1.60	1.0 D + 0.6 W
Lat Shear (lbs)	264 @ 7"	7480	Passed (4%)	1.60	1.0 D + 0.6 W
Lat Moment (Ft-lbs)	571 @ mid-span	3781	Passed (15%)	1.60	1.0 D + 0.6 W
Lat Deflection (in)	0.028 @ mid-span	0.242	Passed (L/999+)	--	1.0 D + 0.6 W
Bi-Axial Bending	0.60	1.00	Passed (60%)	1.60	0.6 D + 0.6 W

Member Length : 7' 3"
System : Wall
Member Type : Header
Building Use : Commercial
Building Code : IBC 2024
Design Methodology : ASD

- Deflection criteria: LL (L/480) and TL (L/240).
- Wall deflection criteria: TL (L/360)
- Moment capacity has been adjusted by a factor of 0.85 to account for the beam stability and/or volume/size factors.
- Applicable calculations are based on NDS.

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Wind	Factored	
1 - Trimmer - DF	1.50"	1.50"	1.50"	545	-2672	545/-1276	None
2 - Trimmer - DF	1.50"	1.50"	1.50"	545	-2672	545/-1276	None

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	End Bearing Points	
Bottom Edge (Lu)	End Bearing Points	

Lateral Connections						
Supports	Stud Size	Stud Material	Connector	Type/Model	Quantity	Nailing
Left	2X	Douglas Fir-Larch	Nails	8d (0.113" x 2 1/2") (Toe)	4	
Right	2X	Douglas Fir-Larch	Nails	8d (0.113" x 2 1/2") (Toe)	4	

Vertical Loads	Location	Tributary Width	Dead (0.90)	Wind (1.60)	Comments
0 - Self Weight (PLF)	0 to 7' 3"	N/A	10.4	--	
1 - Uniform (PSF)	0 to 7' 3"	2'	10.0	--	Wall Above Header (Note PSL Full Length Header Abv.)
2 - Uniform (PLF)	0 to 7' 3"	N/A	120.0	-737.0	Entrance Roof Joist Rxn (1)

ForteWEB Software Operator	Job Notes
Mohammad Aljazzar California Building Engineers Inc. (510) 674-5270 mohammad@cbestructural.com	

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File Name: C & S Restaurant 29 Palms Hwy Yucca Valley

Lateral Load	Location	Tributary Width	Wind (1.60)	Comments
1 - Uniform (PSF)	Full Length	4' 6"	32.2	Zone 5 + Internal [27.9 + 4.3]

• IBC Table 1604.3, footnote f: Deflection checks are performed using 42% of this lateral wind load.

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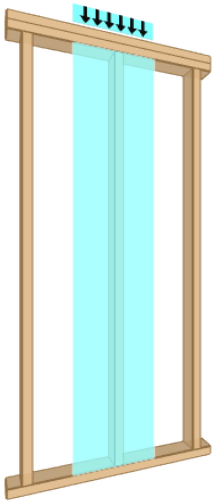
The product application, input design loads, dimensions and support information have been provided by ForteWEB Software Operator

E.5 Framing Design (Wall Studs)

Commercial Bldg. - Walls, Wall: Stud Typical Bearing Wall Lines (1) and (3)

1 piece(s) 2 x 6 DF No.2 @ 16" OC

Wall Height: 12' Member Height: 11' 7 1/2" O. C. Spacing: 16.00"



Drawing is Conceptual

Design Results	Actual	Allowed	Result	LDF	Load: Combination
Slenderness	32	50	Passed (64%)	--	--
Compression (lbs)	952	3672	Passed (26%)	1.60	1.0 D + 0.45 W + 0.75 L + 0.75 Lr
Plate Bearing (lbs)	952	6445	Passed (15%)	--	1.0 D + 0.45 W + 0.75 L + 0.75 Lr
Lateral Reaction (lbs)	177	--	--	1.60	1.0 D + 0.6 W
Lateral Shear (lbs)	163	1584	Passed (10%)	1.60	1.0 D + 0.6 W
Lateral Moment (ft-lbs)	515 @ mid-span	1239	Passed (42%)	1.60	1.0 D + 0.6 W
Total Deflection (in)	0.34 @ mid-span	0.39	Passed (L/407)	--	1.0 D + 0.6 W
Bending/Compression	0.69	1	Passed (69%)	1.60	1.0 D + 0.6 W

- Wall deflection criteria: TL (L/360)
- Input axial load eccentricity for this design is 50% of applicable member side dimension.
- Applicable calculations are based on NDS.
- A bearing area factor of 1.25 has been applied to base plate bearing capacity.
- A 15% increase in the moment capacity has been added to account for repetitive member usage.

Supports	Type	Material
Top	Dbl 2X	Douglas Fir-Larch
Base	2X	Douglas Fir-Larch

System : Wall
Member Type : Stud
Building Code : IBC 2024
Design Methodology : ASD

Max Unbraced Length	Comments
4'	Blocking at Sheathing Panel Edges Hung Horizontally

Lateral Connections

Supports	Connector	Type/Model	Quantity	Connector Nailing
Top	Nails	8d (0.113" x 2 1/2") (Toe)	2	N/A
Base	Nails	8d (0.113" x 2 1/2") (Toe)	2	N/A

- Nailed connection at the top of the member is assumed to be nailed through the bottom 2x plate prior to placement of the top 2x of the double top plate assembly.

Vertical Loads	Spacing	Dead (0.90)	Roof Live (1.25)	Snow (1.15)	Wind (1.60)	Comments
1 - Point (PLF)	16.00"	199.0	227.0	272.5	205.5	Roof Joist Rxn [2] / 2
2 - Point (PLF)	16.00"	252.0	--	--	--	Wall Weight 18 psf x 14 ft

Lateral Load	Location	Spacing	Wind (1.60)	Comments
1 - Uniform (PSF)	Full Length	16.00"	38.1	Zone 5 + Internal [24.9 + 13.2]

- IBC Table 1604.3, footnote f: Deflection checks are performed using 42% of this lateral wind load.

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

ForteWEB Software Operator	Job Notes
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Weyerhaeuser

6/3/2025 6:11:25 AM UTC
ForteWEB v3.9, Engine: V8.4.3.94, Data: V8.1.7.3
File Name: C & S Restaurant 29 Palms Hwy Yucca Valley

FORTEWEB

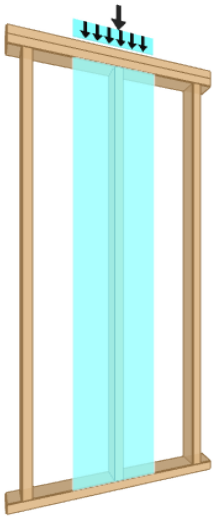
MEMBER REPORT

PASSED

Commercial Bldg. - Walls, Wall: Stud ALIGN W/ JOIST Bearing Wall Lines (1) and (3)

1 piece(s) 2 x 6 DF No.2 @ 16" OC

Wall Height: 12' Member Height: 11' 7 1/2" O. C. Spacing: 16.00"



Drawing is Conceptual

Design Results	Actual	Allowed	Result	LDF	Load: Combination
Slenderness	32	50	Passed (64%)	--	--
Compression (lbs)	1227	3672	Passed (33%)	1.60	1.0 D + 0.45 W + 0.75 L + 0.75 Lr
Plate Bearing (lbs)	1227	6445	Passed (19%)	--	1.0 D + 0.45 W + 0.75 L + 0.75 Lr
Lateral Reaction (lbs)	177	--	--	1.60	1.0 D + 0.6 W
Lateral Shear (lbs)	163	1584	Passed (10%)	1.60	1.0 D + 0.6 W
Lateral Moment (ft-lbs)	515 @ mid-span	1239	Passed (42%)	1.60	1.0 D + 0.6 W
Total Deflection (in)	0.36 @ mid-span	0.39	Passed (L/385)	--	1.0 D + 0.6 W
Bending/Compression	0.80	1	Passed (80%)	1.60	1.0 D + 0.45 W + 0.75 L + 0.75 Lr

- Wall deflection criteria: TL (L/360)
- Input axial load eccentricity for this design is 50% of applicable member side dimension.
- Applicable calculations are based on NDS.
- A bearing area factor of 1.25 has been applied to base plate bearing capacity.
- A 15% increase in the moment capacity has been added to account for repetitive member usage.

Supports	Type	Material
Top	Dbl 2X	Douglas Fir-Larch
Base	2X	Douglas Fir-Larch

System : Wall
Member Type : Stud
Building Code : IBC 2024
Design Methodology : ASD

Max Unbraced Length	Comments
4'	Blocking at Sheathing Panel Edges Hung Horizontally

Lateral Connections

Supports	Connector	Type/Model	Quantity	Connector Nailing
Top	Nails	8d (0.113" x 2 1/2") (Toe)	2	N/A
Base	Nails	8d (0.113" x 2 1/2") (Toe)	2	N/A

- Nailed connection at the top of the member is assumed to be nailed through the bottom 2x plate prior to placement of the top 2x of the double top plate assembly.

Vertical Loads	Spacing	Dead (0.90)	Roof Live (1.25)	Snow (1.15)	Wind (1.60)	Comments
1 - Point (lb)	N/A	376	439	527	412	Roof Joist Rxn, SEE LEDGER CALC
2 - Point (PLF)	16.00"	252.0	--	--	--	Wall Weight 18 psf x 14 ft

Lateral Load	Location	Spacing	Wind (1.60)	Comments
1 - Uniform (PSF)	Full Length	16.00"	38.1	Zone 5 + Internal [24.9 + 13.2]

- IBC Table 1604.3, footnote f: Deflection checks are performed using 42% of this lateral wind load.

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

ForteWEB Software Operator	Job Notes
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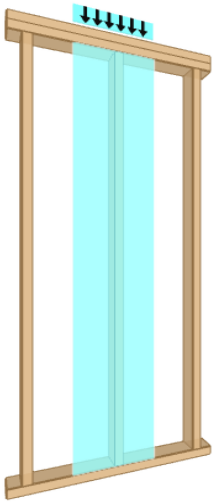
Weyerhaeuser

6/4/2025 4:03:08 AM UTC
ForteWEB v3.9, Engine: V8.4.3.94, Data: V8.1.7.3
File Name: C & S Restaurant 29 Palms Hwy Yucca Valley

Commercial Bldg. - Walls, Wall: Stud Typical Bearing Wall Line (F)

1 piece(s) 2 x 6 DF No.2 @ 16" OC

Wall Height: 12' Member Height: 11' 7 1/2" O. C. Spacing: 16.00"



Drawing is Conceptual

Design Results	Actual	Allowed	Result	LDF	Load: Combination
Slenderness	32	50	Passed (64%)	--	--
Compression (lbs)	450	3595	Passed (13%)	1.15	1.0 D + 0.7 S
Plate Bearing (lbs)	450	6445	Passed (7%)	--	1.0 D + 0.7 S
Lateral Reaction (lbs)	177	--	--	1.60	1.0 D + 0.6 W
Lateral Shear (lbs)	163	1584	Passed (10%)	1.60	1.0 D + 0.6 W
Lateral Moment (ft-lbs)	515 @ mid-span	1239	Passed (42%)	1.60	1.0 D + 0.6 W
Total Deflection (in)	0.30 @ mid-span	0.39	Passed (L/461)	--	1.0 D + 0.6 W
Bending/Compression	0.53	1	Passed (53%)	1.60	1.0 D + 0.6 W

- Wall deflection criteria: TL (L/360)
- Input axial load eccentricity for this design is 50% of applicable member side dimension.
- Applicable calculations are based on NDS.
- A bearing area factor of 1.25 has been applied to base plate bearing capacity.
- A 15% increase in the moment capacity has been added to account for repetitive member usage.

Supports	Type	Material
Top	Dbl 2X	Douglas Fir-Larch
Base	2X	Douglas Fir-Larch

System : Wall
Member Type : Stud
Building Code : IBC 2024
Design Methodology : ASD

Max Unbraced Length	Comments
4'	Blocking at Sheathing Panel Edges Hung Horizontally

Lateral Connections

Supports	Connector	Type/Model	Quantity	Connector Nailing
Top	Nails	8d (0.113" x 2 1/2") (Toe)	2	N/A
Base	Nails	8d (0.113" x 2 1/2") (Toe)	2	N/A

- Nailed connection at the top of the member is assumed to be nailed through the bottom 2x plate prior to placement of the top 2x of the double top plate assembly.

Vertical Loads	Spacing	Dead (0.90)	Roof Live (1.25)	Snow (1.15)	Comments
1 - Point (PLF)	16.00"	32.0	40.0	35.6	Roof Load 2-ft Trib.
2 - Point (PLF)	16.00"	252.0	--	--	Wall Weight 18 psf x 14 ft
3 - Point (PLF)	16.00"	--	--	41.0	Snow Drift 2-ft Trib.

Lateral Load	Location	Spacing	Wind (1.60)	Comments
1 - Uniform (PSF)	Full Length	16.00"	38.1	Zone 5 + Internal [24.9 + 13.2]

- IBC Table 1604.3, footnote f: Deflection checks are performed using 42% of this lateral wind load.

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

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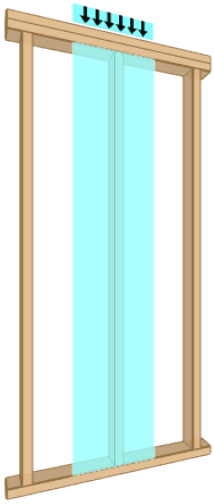
Weyerhaeuser

Commercial Bldg. - Walls, Wall: Stud Typical Bearing Wall Line (B)

1 piece(s) 2 x 6 DF No.2 @ 16" OC

Wall Height: 12' 6" | Member Height: 12' 1 1/2" | O. C. Spacing: 16.00"



Drawing is Conceptual

Design Results	Actual	Allowed	Result	LDF	Load: Combination
Slenderness	32	50	Passed (64%)	--	--
Compression (lbs)	929	3617	Passed (26%)	1.25	1.0 D + 1.0 Lr
Plate Bearing (lbs)	929	6445	Passed (14%)	--	1.0 D + 1.0 Lr
Lateral Reaction (lbs)	185	--	--	1.60	1.0 D + 0.6 W
Lateral Shear (lbs)	171	1584	Passed (11%)	1.60	1.0 D + 0.6 W
Lateral Moment (ft-lbs)	560 @ mid-span	1239	Passed (45%)	1.60	1.0 D + 0.6 W
Total Deflection (in)	0.39 @ mid-span	0.40	Passed (L/377)	--	1.0 D + 0.6 W
Bending/Compression	0.69	1	Passed (69%)	1.60	1.0 D + 0.6 W

- Wall deflection criteria: TL (L/360)
- Input axial load eccentricity for this design is 50% of applicable member side dimension.
- Applicable calculations are based on NDS.
- A bearing area factor of 1.25 has been applied to base plate bearing capacity.
- A 15% increase in the moment capacity has been added to account for repetitive member usage.

Supports	Type	Material
Top	Dbl 2X	Douglas Fir-Larch
Base	2X	Douglas Fir-Larch

System : Wall
Member Type : Stud
Building Code : IBC 2024
Design Methodology : ASD

Max Unbraced Length	Comments
4'	Blocking at Sheathing Panel Edges Hung Horizontally

Lateral Connections

Supports	Connector	Type/Model	Quantity	Connector Nailing
Top	Nails	8d (0.113" x 2 1/2") (Toe)	2	N/A
Base	Nails	8d (0.113" x 2 1/2") (Toe)	2	N/A

- Nailed connection at the top of the member is assumed to be nailed through the bottom 2x plate prior to placement of the top 2x of the double top plate assembly.

Vertical Loads	Spacing	Dead (0.90)	Roof Live (1.25)	Snow (1.15)	Wind (1.60)	Comments
1 - Point (PLF)	16.00"	32.0	40.0	35.6	20.0	Roof Load 2-ft Trib.
2 - Point (PLF)	16.00"	252.0	--	--	--	Wall Weight 18 psf x 14 ft
3 - Point (PLF)	16.00"	--	--	41.0	--	Snow Drift 2-ft Trib.
4 - Point (PLF)	16.00"	124.0	249.0	310.0	125.0	Entrance Roof Stud Reactions

Lateral Load	Location	Spacing	Wind (1.60)	Comments
1 - Uniform (PSF)	Full Length	16.00"	38.1	Zone 5 + Internal [24.9 + 13.2]

- IBC Table 1604.3, footnote f: Deflection checks are performed using 42% of this lateral wind load.

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

ForteWEB Software Operator	Job Notes
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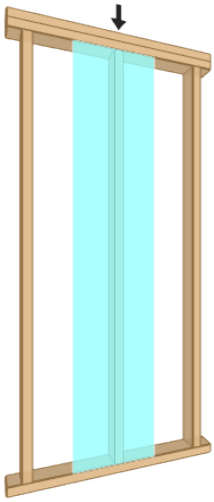
Weyerhaeuser

Commercial Bldg. - Walls, Wall: Typical Bearing Studs Wall Lines (A)

2 piece(s) 2 x 6 DF No.2 @ 16" OC

Wall Height: 9' 6" | Member Height: 9' 1 1/2" | O. C. Spacing: 16.00"



Drawing is Conceptual

Design Results	Actual	Allowed	Result	LDF	Load: Combination
Slenderness	20	50	Passed (40%)	--	--
Compression (lbs)	2756	12719	Passed (22%)	1.25	1.0 D + 1.0 Lr
Plate Bearing (lbs)	2756	11602	Passed (24%)	--	1.0 D + 1.0 Lr
Lateral Reaction (lbs)	139	--	--	1.60	1.0 D + 0.6 W
Lateral Shear (lbs)	125	3168	Passed (4%)	1.60	1.0 D + 0.6 W
Lateral Moment (ft-lbs)	317 @ mid-span	2685	Passed (12%)	1.60	1.0 D + 0.6 W
Total Deflection (in)	0.12 @ mid-span	0.30	Passed (L/900)	--	1.0 D + 0.45 W + 0.75 L + 0.75 Lr
Bending/Compression	0.41	1	Passed (41%)	1.25	1.0 D + 1.0 Lr

- Wall deflection criteria: TL (L/360)
- Input axial load eccentricity for this design is 50% of applicable member side dimension.
- Applicable calculations are based on NDS.
- A bearing area factor of 1.125 has been applied to base plate bearing capacity.
- The column stability factor (Kf = 0.6) applied to this design assumes nailed built-up columns per NDS section 15.3.3. For Weyerhaeuser ELP products refer to the U.S. Wall Guide for multiple-member connection requirements.
- A 15% increase in the moment capacity has been added to account for repetitive member usage.

Supports	Type	Material
Top	Dbl 2X	Douglas Fir-Larch
Base	2X	Douglas Fir-Larch

System : Wall
Member Type : Stud
Building Code : IBC 2024
Design Methodology : ASD

Max Unbraced Length	Comments
4'	Blocking at Sills and Headers

Lateral Connections

Supports	Connector	Type/Model	Quantity	Connector Nailing
Top	Nails	8d (0.113" x 2 1/2") (Toe)	2	N/A
Base	Nails	8d (0.113" x 2 1/2") (Toe)	2	N/A

- Nailed connection at the top of the member is assumed to be nailed through the bottom 2x plate prior to placement of the top 2x of the double top plate assembly.

Vertical Loads	Spacing	Dead (0.90)	Roof Live (1.25)	Snow (1.15)	Wind (1.60)	Comments
1 - Point (lb)	N/A	212	420	506	120	Entrance Roof Joist Rxn [2] / 2 x 1.33 ft Trib.
2 - Point (lb)	N/A	230	--	--	--	Wall Weight 18 psf x 1.33 ft x 9.5 ft
3 - Point (lb)	N/A	745	1149	1378	508	Header Rxns

Lateral Load	Location	Spacing	Wind (1.60)	Comments
1 - Uniform (PSF)	Full Length	16.00"	38.1	Zone 5 + Internal [24.9 + 13.2]

- IBC Table 1604.3, footnote f: Deflection checks are performed using 42% of this lateral wind load.

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Weyerhaeuser

6/3/2025 6:11:25 AM UTC
ForteWEB v3.9, Engine: V8.4.3.94, Data: V8.1.7.3
File Name: C & S Restaurant 29 Palms Hwy Yucca Valley

E.6 Framing Design (Posts)

Commercial Bldg. - Posts , Free Standing Post Typical Post Line C

1 piece(s) 8 x 8 DF No.2

Post Height: 10'

Drawing is Conceptual

Design Results	Actual	Allowed	Result	LDF	Load: Combination
Slenderness	16	50	Passed (32%)	--	--
Compression (lbs)	18656	38766	Passed (48%)	1.15	1.0 D + 0.7 S
Base Bearing (lbs)	18869	1670625	Passed (1%)	--	1.0 D + 0.45 W + 0.75 L + 0.525 S
Bending/Compression	0.75	1	Passed (75%)	1.15	1.0 D + 0.7 S

- Input axial load eccentricity for this design is 16.67% of applicable member side dimension.
- Applicable calculations are based on NDS.

Supports	Type	Material
Base	Plate	Steel

Max Unbraced Length	Comments
Full Member Length	No bracing assumed.

Member Type : Free Standing Post
Building Code : IBC 2024
Design Methodology : ASD

Vertical Load	Dead (0.90)	Roof Live (1.25)	Snow (1.15)	Wind (1.60)	Comments
1 - Point (lb)	9440	8786	13165	5594	Roof Beam Reaction [2]

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Weyerhaeuser

Commercial Bldg. - Posts , Free Standing Post Typical Post Line D

1 piece(s) 8 x 8 DF No.2

Post Height: 10'

Drawing is Conceptual

Design Results	Actual	Allowed	Result	LDF	Load: Combination
Slenderness	16	50	Passed (32%)	--	--
Compression (lbs)	14807	38766	Passed (38%)	1.15	1.0 D + 0.7 S
Base Bearing (lbs)	15096	1670625	Passed (1%)	--	1.0 D + 0.45 W + 0.75 L + 0.525 S
Bending/Compression	0.53	1	Passed (53%)	1.15	1.0 D + 0.7 S

- Input axial load eccentricity for this design is 16.67% of applicable member side dimension.
- Applicable calculations are based on NDS.

Supports	Type	Material
Base	Plate	Steel

Max Unbraced Length	Comments
Full Member Length	No bracing assumed.

Member Type : Free Standing Post
Building Code : IBC 2024
Design Methodology : ASD

Vertical Load	Dead (0.90)	Roof Live (1.25)	Snow (1.15)	Wind (1.60)	Comments
1 - Point (lb)	7271	7155	10765	4829	Roof Beam Reaction [3]

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Weyerhaeuser

Commercial Bldg. - Posts , Free Standing Post Typical Post Line E

1 piece(s) 8 x 8 DF No.2

Post Height: 10'

Drawing is Conceptual

Design Results	Actual	Allowed	Result	LDF	Load: Combination
Slenderness	16	50	Passed (32%)	--	--
Compression (lbs)	12125	38766	Passed (31%)	1.15	1.0 D + 0.7 S
Base Bearing (lbs)	12563	1670625	Passed (1%)	--	1.0 D + 0.45 W + 0.75 L + 0.75 Lr
Bending/Compression	0.40	1	Passed (40%)	1.15	1.0 D + 0.7 S

- Input axial load eccentricity for this design is 16.67% of applicable member side dimension.
- Applicable calculations are based on NDS.

Supports	Type	Material
Base	Plate	Steel

Max Unbraced Length	Comments
Full Member Length	No bracing assumed.

Member Type : Free Standing Post
Building Code : IBC 2024
Design Methodology : ASD

Vertical Load	Dead (0.90)	Roof Live (1.25)	Snow (1.15)	Wind (1.60)	Comments
1 - Point (lb)	5938	6443	8839	3983	Roof Beam Reaction [4]

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ForteWEB Software Operator	Job Notes
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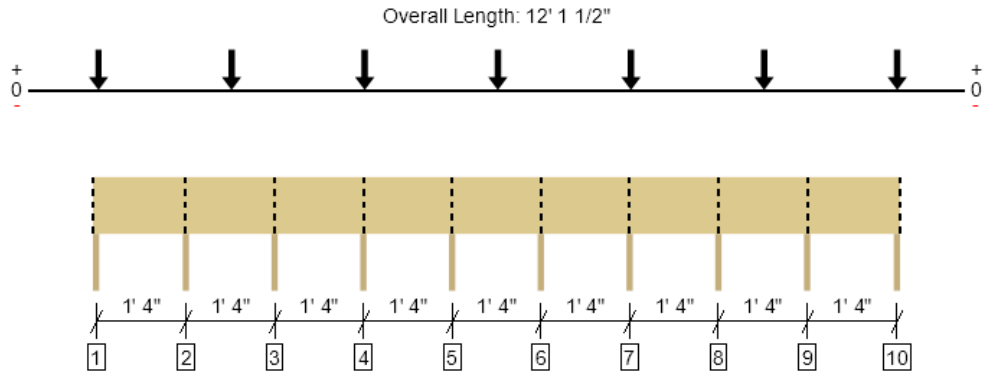
Weyerhaeuser

E.7 (Ledgers)

Commercial Bldg. - Ledgers, Line 3 - Line 1 Similar LSL Ledger

1 piece(s) 1 3/4" x 14" 1.55E TimberStrand® LSL



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

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	891 @ 0	1641 (1.50")	Passed (54%)	--	1.0 D + 0.45 W + 0.75 L + 0.75 Lr (Alt Spans)
Shear (lbs)	226 @ 1' 6"	6329	Passed (4%)	1.25	1.0 D + 1.0 Lr (Adj Spans)
Moment (Ft-lbs)	196 @ 10' 1"	13311	Passed (1%)	1.25	1.0 D + 1.0 Lr (Alt Spans)
Live Load Defl. (in)	0.000 @ 0	0.033	Passed (2L/999+)	--	1.0 D + 0.45 W + 0.75 L + 0.75 Lr (Alt Spans)
Total Load Defl. (in)	0.000 @ 0	0.067	Passed (2L/999+)	--	1.0 D + 0.45 W + 0.75 L + 0.75 Lr (Alt Spans)

Member Length : 12' 1 1/2"
System : Floor
Member Type : Drop Beam
Building Use : Commercial
Building Code : IBC 2024
Design Methodology : ASD

- Deflection criteria: LL (L/480) and TL (L/240).
- Moment capacity has been adjusted by a factor of 0.96 to account for the beam stability and/or volume/size factors.

Supports	Bearing Length			Loads to Supports (lbs)					Accessories
	Total	Available	Required	Dead	Roof Live	Snow	Wind	Factored	
1 - Beam - DF	1.50"	1.50"	1.50"	376	439	527	412/-28	891	Blocking
2 - Beam - DF	1.50"	1.50"	1.50"	223	243	292	222/-3	506	Blocking
3 - Beam - DF	1.50"	1.50"	1.50"	274	302	363	274	624	Blocking
4 - Beam - DF	1.50"	1.50"	1.50"	307	397	476	412	790	Blocking
5 - Beam - DF	1.50"	1.50"	1.50"	252	277	333	253/-4	573	Blocking
6 - Beam - DF	1.50"	1.50"	1.50"	267	295	354	269/-4	610	Blocking
7 - Beam - DF	1.50"	1.50"	1.50"	307	397	476	412	790	Blocking
8 - Beam - DF	1.50"	1.50"	1.50"	259	284	341	258	588	Blocking
9 - Beam - DF	1.50"	1.50"	1.50"	239	261	313	238/-3	541	Blocking
10 - Beam - DF	1.50"	1.50"	1.50"	375	439	527	412/-29	889	Blocking

• Blocking Panels are assumed to carry no loads applied directly above them and the full load is applied to the member being designed.

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	All Bearing Points	
Bottom Edge (Lu)	All Bearing Points	

Use (3) SDWS 0.22" x 5" Screw Ledger to Studs

Wood Side Plate 1.75" SCL Shear = 890 lbs ASD Cd = 1.0
The Following extreme load combination vs Cd (duration factor)

1.0 D = 376 lbs
Cd = 0.9 --> ASD capacity = 0.9 (890) = 801 lbs OK 47% DCR

1.0 D + 1.0 S = 376 + 527 = 903 lbs
Cd = 1.15 --> ASD capacity = 1.15 (890) = 1,023 lbs OK 88% DCR

SDWS timber screw
SDWS22400DB [Use SDWS22500 DB]
890 lbs. Cd = 1.0
1,023 lbs Cd = 1.15 Snow
1,112 lbs Cd = 1.25 Live Roof
1,424 lbs Cd = 1.6 E/W

ForteWEB Software Operator	Job Notes
Mohammad Aljazzar California Building Engineers Inc. (510) 674-5270 mohammad@cbestructural.com	

Weyerhaeuser

6/3/2025 7:10:44 AM UTC
ForteWEB v3.9, Engine: V8.4.3.94, Data: V8.1.7.3
File Name: C & S Restaurant 29 Palms Hwy Yucca Valley

Vertical Loads	Location (Side)	Tributary Width	Dead (0.90)	Roof Live (1.25)	Snow (1.15)	Wind (1.60)	Comments
0 - Self Weight (PLF)	0 to 12' 1 1/2"	N/A	7.7	--	--	--	
1 - Point (lb)	1" (Front)	N/A	398	454	545	411	Max. Downward Reaction
2 - Point (lb)	2' 1" (Front)	N/A	398	454	545	411	Max. Downward Reaction
3 - Point (lb)	4' 1" (Front)	N/A	398	454	545	411	Max. Downward Reaction
4 - Point (lb)	6' 1" (Front)	N/A	398	454	545	411	Max. Downward Reaction
5 - Point (lb)	8' 1" (Front)	N/A	398	454	545	411	Max. Downward Reaction
6 - Point (lb)	10' 1" (Front)	N/A	398	454	545	411	Max. Downward Reaction
7 - Point (lb)	12' 1" (Front)	N/A	398	454	545	411	Max. Downward Reaction

• Side loads are assumed to not induce cross-grain tension.

Weyerhaeuser Notes

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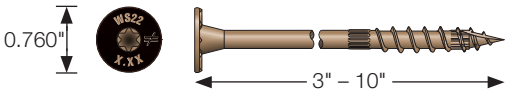
The product application, input design loads, dimensions and support information have been provided by ForteWEB Software Operator

Strong-Drive®
SDWS™ TIMBER Screw (Exterior Grade) in Ledger-to-Stud Applications

Strong-Drive SDWS Timber screws may be used to attach a ledger to the narrow face of nominal 2x lumber studs according to the following table. Model numbers with SDWS22xxxxDB indicate tan double-barrier coating; SDWS22xxxxDBB indicate black double-barrier coating.

For more information: see p. 59, C-F-2025 *Fastening Systems* catalog

SDWS timber screw
SDWS22400DB [Use SDWS22500 DB]
890 lbs. Cd = 1.0
1,023 lbs Cd = 1.15 Snow
1,112 lbs Cd = 1.25 Live Roof
1,424 lbs Cd = 1.6 E/W

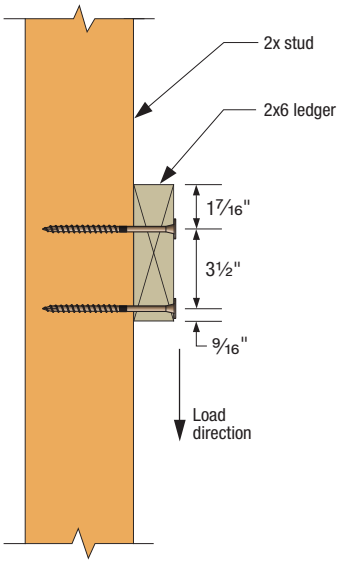


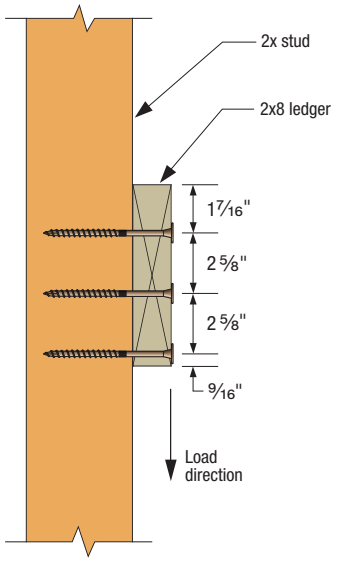
Use 5" length fastener

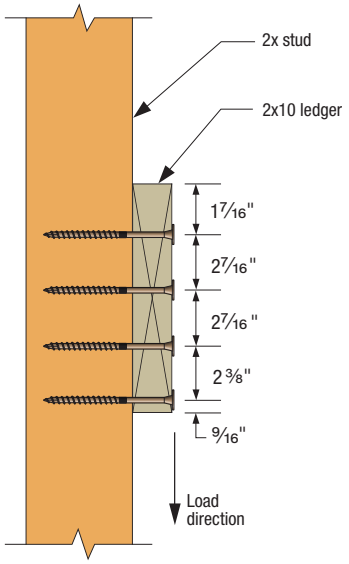
SDWS Timber Screw (Exterior Grade) — Allowable Shear Loads for Ledger to Studs

Fastener Length (in.)	Model No.	Ledger Nominal Size (in.)	Number of Screws per Stud	Reference Allowable Shear Loads (lb.)		
				SP	DFL	SPF/HF
4	SDWS22400DB SDWS22400DBB	2x6	2	785	630	565
		2x8	3	1,060	890	855
		2x10	4	—	1,040	1,040

1. Allowable loads shall be limited to parallel-to-grain loaded solid sawn main members (minimum 2" nominal). Wood side members shall be loaded perpendicular to grain.
2. Allowable loads are based on DFL, SPF/HF, and SP wood members having a minimum specific gravity of 0.50, 0.42, and 0.55, respectively. Where the side and main members have different specific gravities, the lower values shall be used.
3. Allowable loads are shown at the wood load duration factor of C_D = 1.00. Loads may be increased for load duration as permitted by the building code up to a C_D = 1.60. All adjustment factors shall be applied per NDS-2024. For in-service moisture content greater than 19%, use C_M = 0.70.
4. Fasteners shall be centered in the stud and spaced as shown in the figure. The stud minimum end distance is 6" when loaded toward the end and 2½" when loaded away from the end. The ledger end distance is 6" for full values. For ledger end distances between 2" and 6" use 50% of the table loads. For end distances between 2" and 4", predrill using a 5⁄32" bit for SDWS.
5. Screws may be installed with an intermediate layer of wood structural panel between the side and main member provided the wood structural panel is fastened to the main member per code and the minimum screw penetration of 2½" into the main member (excluding the wood structural panel) is met. Longer lengths of the screw series may be used.
6. For LRFD values, the reference connection design values shall be adjusted in accordance with the NDS-2024, section 11.3.
7. For 2x10 SP ledgers, use the number of screws and allowable loads of the 2x8 SP ledger.
8. For 2x8 ledgers with two screws, use 2x6 values. For 2x10 ledgers with three screws, use 2x8 values. Spacings and edge distances shown in the figure are minimum dimensions.
9. For loads in the opposite direction from that shown in the figure, use the table values multiplied by: 0.50 for two-screw connections, 0.67 for three-screw connections, and 0.75 for four-screw connections.
10. Visit **strongtie.com/drawings** and search for SD1-L for additional ledger fastening detail sheets and load tables in DWG, PDF or DXF format.
11. Fastener loads are based on the lesser of single fastener ICC-ES AC233 testing with a safety factor of 5.0 or ICC-ES AC13 assembly testing with a factor of safety of 5.0.







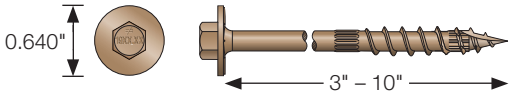
Ledger Structural Fastening Applications

Strong-Drive® SDWH™ TIMBER-HEX Screw in Ledger-to-Stud Applications

FOR REFERENCE NOT USED

Strong-Drive SDWH Timber-Hex screws may be used to attach a ledger to the narrow face of nominal 2x lumber studs according to the following table.

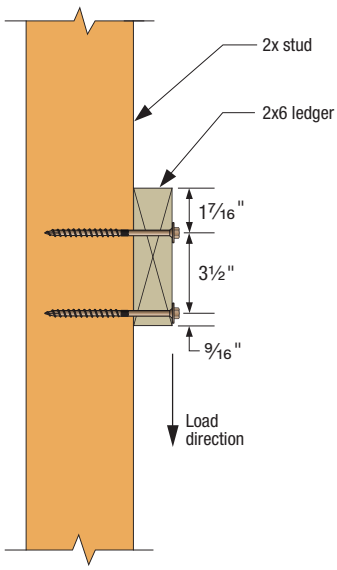
For more information: see p. 62, C-F-2025 *Fastening Systems* catalog

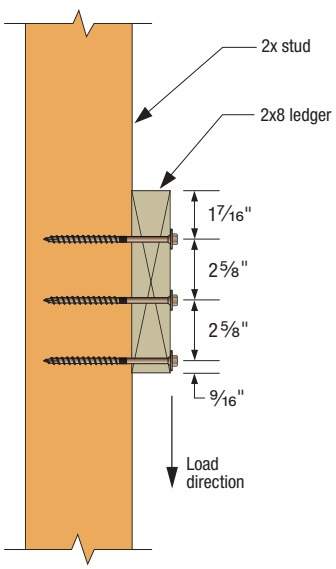


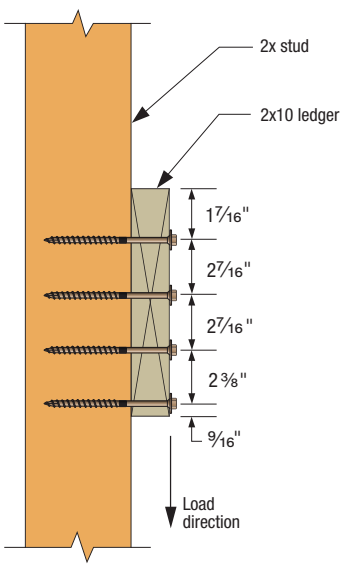
SDWH Timber-Hex Screw — Allowable Shear Loads for Ledger to Studs

Fastener Length (in.)	Model No.	Nominal Ledger Size (in.)	Number of Screws per Stud	Reference Allowable Shear Loads (lb.)		
				SP	DFL	SPF/HF
4	SDWH19400DB	2x6	2	630	630	540
		2x8	3	630	815	815
		2x10	4	—	1,170	975

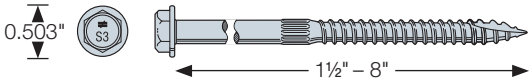
1. Allowable loads shall be limited to parallel-to-grain loaded solid sawn main members (minimum 2" nominal). Wood side members shall be loaded perpendicular to grain.
2. Allowable loads are based on DFL, SPF/HF, and SP wood members having a minimum specific gravity of 0.50, 0.42, and 0.55, respectively. Where the side and main members have different specific gravities, the lower values shall be used.
3. Allowable loads are shown at the wood load duration factor of C_D = 1.00. Loads may be increased for load duration as permitted by the building code up to a C_D = 1.60. For in-service moisture content greater than 19%, use C_M = 0.70.
4. Fasteners shall be centered in the stud and spaced as shown in the figure. The stud minimum end distance is 6" when loaded toward the end and 2½" when loaded away from the end. The ledger end distance is 6" for full values. For ledger end distanced between 2" and 6" use 50% of the table loads. For end distances between 2" and 4", predrill using a ⅛" bit for the SDWH.
5. Screws shall be placed 1.5" to 2" from the top and bottom of the ledger or rim board with 3" minimum and 6" maximum vertical distance between fasteners with horizontal on-center spacing per the table. End screws shall be located 6" from the end and at 1.5" to 2" from the bottom of the ledger. For screws located at least 2" but less than 6" from the end, use 50% of the load per screw and 50% of the table spacing between the end screw and the adjacent screw, and for screws located between 2" and 4" from the end, predrill using a ⅛" drill bit.
6. For LRFD values, the reference connection design values shall be adjusted in accordance with the NDS-2024, section 11.3.
7. For 2x10 SP ledgers, use the number of screws and allowable loads of the 2x8 SP ledger.
8. For 2x8 ledgers with two screws, use 2x6 values. For 2x10 ledgers with three screws, use 2x8 values. Spacings and edge distances shown in the figure are minimum dimensions.
9. For loads in the opposite direction from that shown in the figure, use the table values multiplied by: 0.50 for two-screw connections, 0.67 for three-screw connections, and 0.75 for four-screw connections.
10. Visit **strongtie.com/drawings** and search for SD1-L for additional ledger fastening detail sheets and load tables in DWG, PDF or DXF format.
11. Fastener loads are based on the lesser of single fastener ICC-ES AC233 testing with a safety factor of 5.0 or ICC-ES AC13 assembly testing with a factor of safety of 5.0.







Strong-Drive® SDS HEAVY-DUTY CONNECTOR Screw

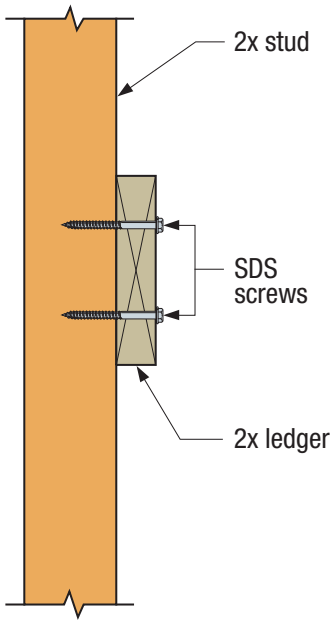


SDS Heavy-Duty Connector Screw — Allowable Shear Loads for Ledger to Studs

Fastener Length (in.)	Model No.	Nominal Ledger Size (in.)	Number of Screws per Stud	Allowable Shear Loads (lb.)	
				DFL/SP	SPF/HF
3½	SDS25312	2x8	2	500	380
4½	SDS25412	2x10	3	750	570

1. Allowable loads are limited to parallel-to-grain loaded solid-sawn main members (2" nominal). Wood side members may be loaded parallel or perpendicular to grain (see footnote 4).
2. DFL/SP allowable loads are based on wood members having a minimum specific gravity of 0.50, and SPF/HF allowable loads are based on wood members having a minimum specific gravity of 0.42. Where the side and main members have different specific gravities, the lower values shall be used.
3. Allowable loads are shown at the wood load duration factor of C_D = 1.00. Loads may be increased for load duration by the building code up to a C_D = 1.60.
4. Minimum spacing of fasteners is 3" o.c., minimum end distance is 3" for all parallel-to-grain loaded members, or 4" for all perpendicular-to-grain loaded members, and minimum edge distance is ¾" for all parallel-to-grain loaded members, or 1½" for perpendicular-to-grain loaded side members.
5. Screws may be installed with a maximum ½" thick intermediate layer of wood structural panel between the side and main member provided the wood structural panel is fastened to the main member per code and the minimum penetration of the screw into the main member (excluding the wood structural panel) is met.
6. Visit **strongtie.com/drawings** and search for SD1-L for additional ledger fastening detail sheets and load tables in DWG, PDF or DXF format.
7. Fastener loads are based on the lesser of single fastener ICC-ES AC233 testing with a safety factor of 5.0 or ICC-ES AC13 assembly testing with a factor of safety of 5.0.

FOR REFERENCE NOT USED



Ledger-to-Stud Assembly

Ledger Structural Fastening Applications

FOR REFERENCE
NOT USED

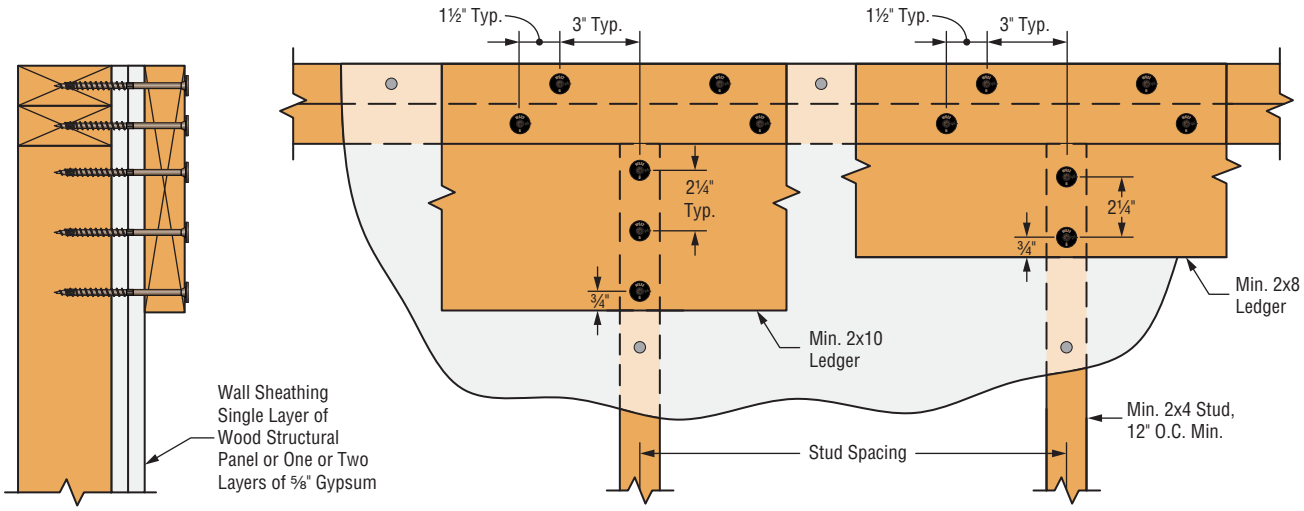
Strong-Drive® SDWS™ TIMBER Screw Ledger Attachment for Top-of-Wall Alignment

The 5" Strong-Drive SDWS Timber Screw (Exterior Grade), model SDWS22500DB, has been tested for use in attaching a ledger to minimum 2x4 wall framing when the ledger is flush with the top of the wall plates and installed over: a) one layer of ½" maximum wood structural panel sheathing, b) one or two layers of ⅝" maximum gypsum board, or c) one layer of ⅝" maximum gypsum over one layer of ½" maximum WSP. The allowable loads are the lesser of single-fastener testing (in accordance with ICC-ES AC233) or full-scale testing of the assemblies shown (in accordance with ICC-ES AC13) with a safety factor of 5.0.

SDWS Timber Screw (Exterior Grade) — Allowable Downloads for Ledger to Top-of-Wall over WSP or Gypsum Wall Sheathing

Number of SDWS22500DB Ledger Screws at Each Stud Connection			Min. Ledger Size	Allowable Download at Each Stud Connection[1,2] (lb.)	Allowable Unit Load Based on Stud Spacing[3] (plf)			
Total	Stud	Top Plates			24" o.c.	19.2" o.c.	16" o.c.	12" o.c.
6	2	4	2x8	855	430	535	640	855
7	3	4	2x10	1,430	715	895	1,075	1,430

1. Allowable loads are applicable to DFL/SP/SPF stud and top plate species and DFL/SP ledger species. For SPF ledger, allowable load per stud is 855 pounds for assembly with six total screws and 1,230 pounds for assembly with seven total screws.
2. Allowable loads are shown at the wood load duration factor of C_D = 1.00. Loads may be increased for load duration as permitted by the building code up to a C_D = 1.60. All adjustment factors shall be applied per the National Design Specification (NDS). For in-service moisture content greater than 19%, use C_M = 0.70.
3. Minimum stud spacing is 12" on center. Allowable unit loads listed based on specified fastening at every stud.
4. Fasteners shall be centered in the stud and wall plates and spaced as shown in the figures below. The minimum distance from a fastener to the end of a ledger is 6" for full values. For connections where fastener to ledger end distances are between 1½" and 6", use 50% of the table loads. For end distances between 1½" and 4", predrill using a ⅝₂" bit.
5. Design of wall assembly and ledger is the responsibility of the designer. Wall sheathing must be attached to wall framing as required per the building code.



Section View

Assembly Detail for Seven Total Screws

Assembly Detail for Six Total Screws

E.8 (Wall Studs: Out of Plane)

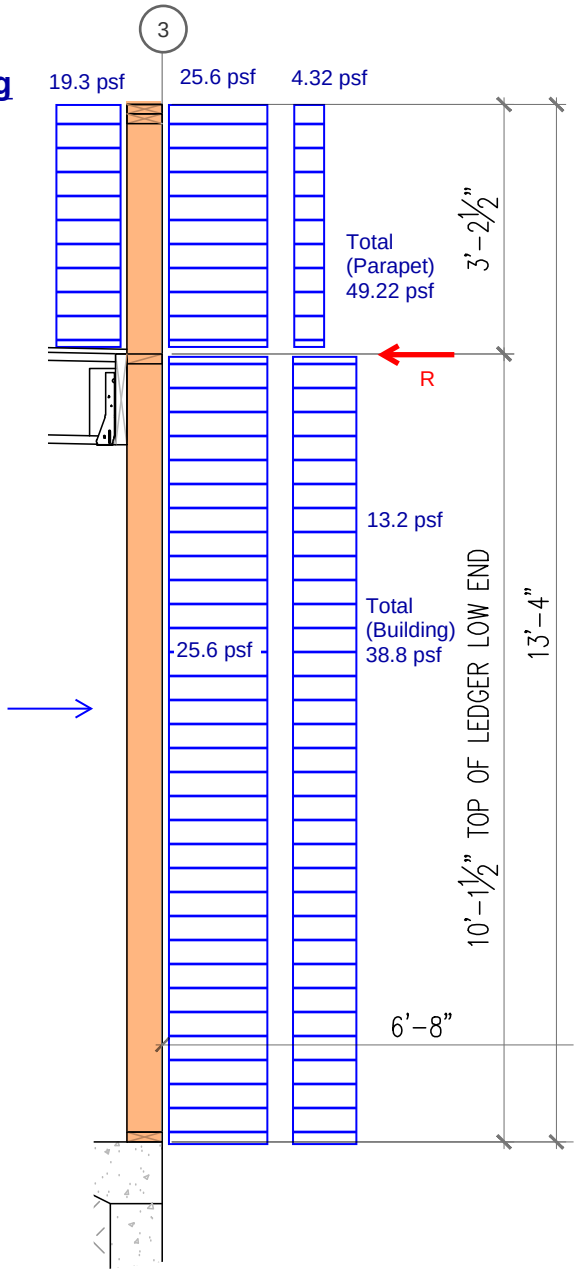
Wind Load: Out of Plane Components and Cladding

Balloon Frame Studs: Line B; Ht.= 13'-4"
EWA = 13.33 x 13.33/3 = 59 ft2
for Connection: 13.33 ft x (0.5 x 13.33 ft) = R x 10.125 ft --> R = 8.78 ft
Hence Reaction is 8.78 / 13.333 = 66% Tributary to top connection.
EWA Reaction = 66% x 59 = 39 ft say 40 ft

Windward Loading: C & C
Zone 4 and 5 = 19.3 psf

Leeward Loading: C & C
Zone 4 = -21.5 psf
Zone 5 = -25.6 psf

Internal Pressure:
Conditioned space = 13.2 psf
Parapet: (Blocked internally, enclosed) = 4.32 psf



Seismic Load: Wall out-of-plane (Anchorage)

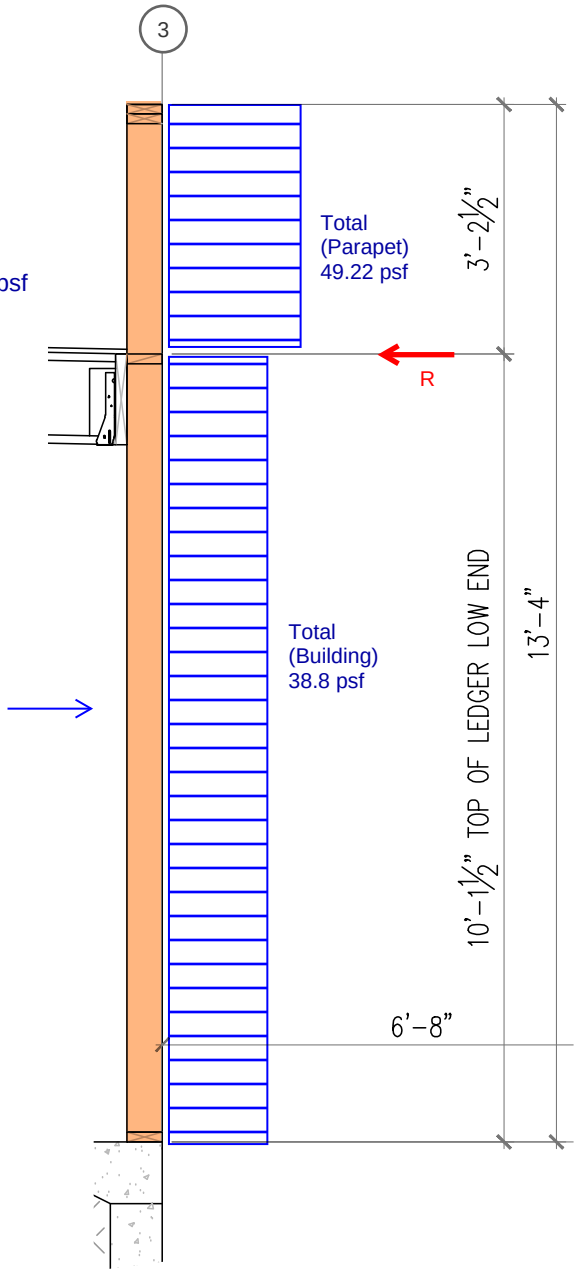
Per ASCE 7-16 Ch. 12.11.2.1

Fp = 0.4 Sds ka Ie Wp
Fp min = 0.2 ka Ie Wp

Fp = 0.4 (1.777) (1.5) (1.0) (18) Wall at conditioned Space Wp = 18 psf
Fp = 19.2 psf say 20 psf

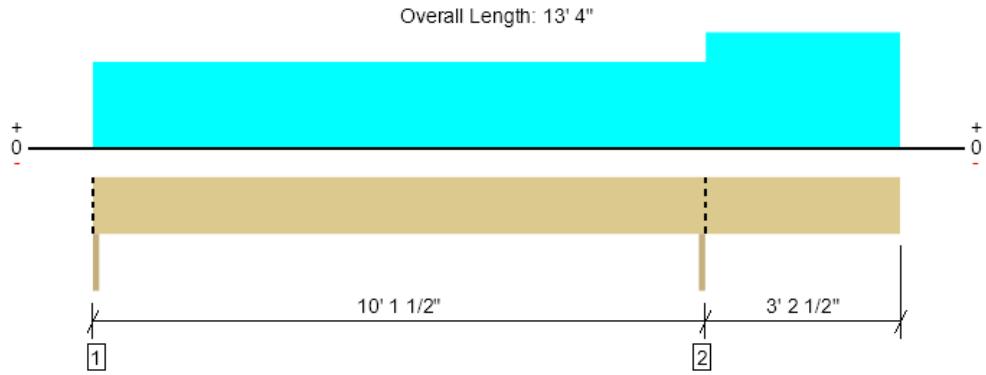
Fp = 0.4 (1.777) (1.5) (1.0) (28) Wall at Parapet Wp = 28 psf
Fp = 29.9 psf say 30 psf

ka = 1.00 for flexible diaphragm, however if assumed to be calculated (semi-rigid) then,
ka = 1.0 + Lf / 100
Lf = 45 ft max. per plans, however use ka = 1.5 conservatively



Commercial Bldg. - Out of Plane, Wall Studs Typical Line [3]

1 piece(s) 2 x 6 DF No.2 @ 16" O.C.



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

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	324 @ 10' 3/4"	1406 (1.50")	Passed (23%)	--	1.0 D + 0.6 W (All Spans)
Shear (lbs)	171 @ 9' 6 1/2"	1584	Passed (11%)	1.60	1.0 D + 0.6 W (All Spans)
Moment (Ft-lbs)	414 @ 5'	1103	Passed (38%)	1.60	1.0 D + 0.6 W (Alt Spans)
Live Load Defl. (in)	0.149 @ 13' 4"	0.218	Passed (2L/526)	--	1.0 D + 0.6 W (Alt Spans)
Total Load Defl. (in)	0.226 @ 5' 1/4"	0.503	Passed (L/534)	--	1.0 D + 0.6 W (Alt Spans)

Member Length : 13' 4"
System : Roof
Member Type : Flush Beam
Building Use : Commercial
Building Code : IBC 2024
Design Methodology : ASD
Member Pitch : 0/12

- Deflection criteria: LL (L/360) and TL (L/240).
- Overhang deflection criteria: LL (2L/360) and TL (2L/240).
- Upward deflection on right cantilever exceeds overhang deflection criteria.
- Applicable calculations are based on NDS.

324 lbs. / 1.333ft x 4 ft = 972 lbs. per twist strap at stud aligned with roof joist.

Supports	Bearing Length			Loads to Supports (lbs)				Accessories
	Total	Available	Required	Dead	Wind	Seismic	Factored	
1 - Beam - HF	1.50"	1.50"	1.50"	9	260/-35	134/-134	166/-88	Blocking
2 - Beam - DF	1.50"	1.50"	1.50"	18	510	286/-286	324/-189	Blocking

- Blocking Panels are assumed to carry no loads applied directly above them and the full load is applied to the member being designed.

Per connection Stud to Ledger or aligned Roof Joist

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	4' o/c	Wall Blk'g at 4' max. for wall sheathing.
Bottom Edge (Lu)	4' o/c	Wall Blk'g at 4' max. for wall sheathing.

Vertical Loads	Location (Side)	Tributary Width	Dead (0.90)	Wind (1.60)	Seismic (1.60)	Comments
0 - Self Weight (PLF)	0 to 13' 4"	N/A	2.1	--	--	
1 - Uniform (PSF)	0 to 10' 1 1/2" (Top)	1' 4"	--	38.8	--	Wind (Building) Zone 5 + Internal [25.6 + 13.2]
2 - Uniform (PSF)	10' 1 1/2" to 13' 4" (Top)	1' 4"	--	49.2	--	Wind (Parapet) Zone 5 + Internal [19.3 + 25.6 + 4.32]
3 - Uniform (PSF)	0 to 10' 1 1/2" (Top)	1' 4"	--	--	20.0	Wall Seismic OOP Anchorage Force Conditioned space
4 - Uniform (PSF)	10' 1 1/2" to 13' 4" (Top)	1' 4"	--	--	30.0	Wall Seismic OOP Anchorage Force Parapet

- Side loads are assumed to not induce cross-grain tension.

Use (3) SDS 1/4" x 5" Screw Ledger to Studs

Wood Side Plate 1.75" SCL Withdrawal = 475 lbs
ASD Cd = 1.0
The Following extreme load combination vs Cd (duration factor)
172 lbs / in [threaded portion 2 3/4"]

0.6 W = 306 lbs
Cd = 1.6 --> ASD capacity
= 1.6 (475) (3) = 2,280 lbs OK 14% DCR

0.7 E = 200 lbs
Cd = 1.6 --> ASD capacity
= 1.6 (475) (3) = 2,280 lbs OK 9% DCR

Combined Withdrawal and Lateral Loading to Screws:
Per NDS 12.4.1 Lag screws:
$Z'\alpha = W'p\ Z' / (W'p \cos^2 \alpha + Z' \sin^2 \alpha)$

Withdrawal [Cd = 1.6] = 306 lbs [0.6W]
Lateral [Cd = 1.15] = 903 lbs [D + S] see ledger calcs.

$\alpha = \tan\text{-}1(306/903) = 18.72$ deg.
$\cos \alpha = 0.947$ $\cos^2 \alpha = 0.896$
$\sin \alpha = 0.321$ $\sin^2 \alpha = 0.103$
Calculated for Cd = 1.0;
$Z'a = (475)(340) / (475x0.896 + 340x0.103) = 350$ lbs

Resultant Force = $(306^2 + 903^2)^{1/2}$ = 953 lbs

Resultant Force = $(306^2 + 903^2)^{1/2}$ = 953 lbs

Cd = 1.6

350 lbs x 1.6 x 3 screws = 1,680 lbs

953 / 1680 = 57% DCR OK

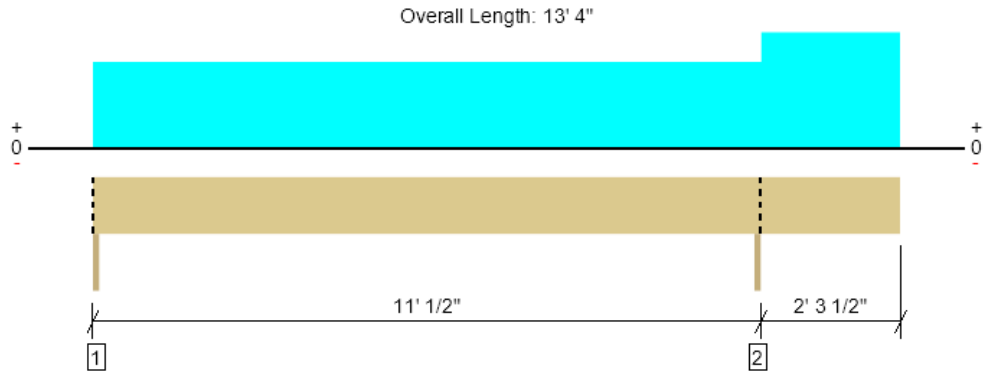
ForteWEB Software Operator	Job Notes
Mohammad Aljazzar California Building Engineers Inc. (510) 674-5270 mohammad@cbestructural.com	

Weyerhaeuser

6/3/2025 7:59:06 AM UTC
ForteWEB v3.9, Engine: V8.4.3.94, Data: V8.1.7.3
File Name: C & S Restaurant 29 Palms Hwy Yucca Valley

Commercial Bldg. - Out of Plane, Wall Studs Typical Line [1]

1 piece(s) 2 x 6 DF No.2 @ 16" O.C.



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

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	290 @ 10' 11 3/4"	1406 (1.50")	Passed (21%)	--	1.0 D + 0.6 W (All Spans)
Shear (lbs)	175 @ 10' 5 1/2"	1584	Passed (11%)	1.60	1.0 D + 0.6 W (All Spans)
Moment (Ft-lbs)	496 @ 5' 5 11/16"	1103	Passed (45%)	1.60	1.0 D + 0.6 W (Alt Spans)
Live Load Defl. (in)	0.305 @ 5' 5 7/8"	0.366	Passed (L/432)	--	1.0 D + 0.6 W (Alt Spans)
Total Load Defl. (in)	0.323 @ 5' 5 13/16"	0.549	Passed (L/408)	--	1.0 D + 0.6 W (Alt Spans)

Member Length : 13' 4"
System : Roof
Member Type : Flush Beam
Building Use : Commercial
Building Code : IBC 2024
Design Methodology : ASD
Member Pitch : 0/12

- Deflection criteria: LL (L/360) and TL (L/240).
- Overhang deflection criteria: LL (2L/360) and TL (2L/240).
- Upward deflection on right cantilever exceeds overhang deflection criteria.
- Applicable calculations are based on NDS.

Supports	Bearing Length			Loads to Supports (lbs)				Accessories
	Total	Available	Required	Dead	Wind	Seismic	Factored	
1 - Beam - HF	1.50"	1.50"	1.50"	11	284/-17	146/-146	181/-96	Blocking
2 - Beam - DF	1.50"	1.50"	1.50"	17	455	251/-251	290/-165	Blocking

- Blocking Panels are assumed to carry no loads applied directly above them and the full load is applied to the member being designed.

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	4' o/c	Wall Blk'g at 4' max. for wall sheathing.
Bottom Edge (Lu)	4' o/c	Wall Blk'g at 4' max. for wall sheathing.

Vertical Loads	Location (Side)	Tributary Width	Dead (0.90)	Wind (1.60)	Seismic (1.60)	Comments
0 - Self Weight (PLF)	0 to 13' 4"	N/A	2.1	--	--	
1 - Uniform (PSF)	0 to 11' 1/2" (Top)	1' 4"	--	38.8	--	Wind (Building) Zone 5 + Internal [25.6 + 13.2]
2 - Uniform (PSF)	11' 1/2" to 13' 4" (Top)	1' 4"	--	49.2	--	Wind (Parapet) Zone 5 + Internal [19.3 + 25.6 + 4.32]
3 - Uniform (PSF)	0 to 11' 1/2" (Top)	1' 4"	--	--	20.0	Wall Seismic OOP Anchorage Force Conditioned space
4 - Uniform (PSF)	11' 1/2" to 13' 4" (Top)	1' 4"	--	--	30.0	Wall Seismic OOP Anchorage Force Parapet

- Side loads are assumed to not induce cross-grain tension.

See previous page for critical loading condition.

ForteWEB Software Operator	Job Notes
Mohammad Aljazzar California Building Engineers Inc. (510) 674-5270 mohammad@cbestructural.com	

Weyerhaeuser

6/3/2025 7:59:06 AM UTC
ForteWEB v3.9, Engine: V8.4.3.94, Data: V8.1.7.3
File Name: C & S Restaurant 29 Palms Hwy Yucca Valley

E.9 Connector Capacities

Strong-Drive®
SDWS™ TIMBER Screw (Exterior Grade)

Structural Wood-to-Wood Connections Including Ledgers, Indoor/Outdoor Projects

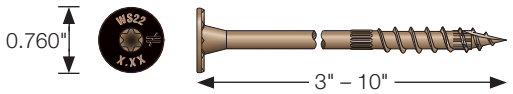
Designed to provide an easy-to-install, high-strength alternative to through-bolting and traditional lag screws. The Strong-Drive SDWS Timber screws are ideal for the contractor and do-it-yourselfer alike. Model numbers with SDWS22xxxxDB indicate tan double-barrier coating; SDWS22xxxxDBB indicate black double-barrier coating.

Codes/Standards: IAPMO UES ER-192, State of Florida FL13975

US Patent: 9,523,383

For more information: see p. 59, C-F-2025 *Fastening Systems* catalog

Double-barrier coating provides corrosion resistance equivalent to hot-dip galvanization, making it suitable for certain exterior and preservative-treated wood applications, as described in the evaluation report.



SDWS Timber Screw — Allowable Shear Loads — Douglas Fir–Larch and Southern Pine Lumber

Fastener Length (in.)	Model No.	Thread Length (in.)	Reference DFL/SP Allowable Shear Loads (lb.)								
			Wood Side Member Thickness (in.)								
			1.5	2	2.5	3	3.5	4	4.5	6	8
3	SDWS22300DB SDWS22300DBB	1½	255	—	—	—	—	—	—	—	—
4	SDWS22400DB SDWS22400DBB	2⅜	405	405	305	—	—	—	—	—	—
5	SDWS22500DB SDWS22500DBB	3	405	405	360	360	325	—	—	—	—
6	SDWS22600DB SDWS22600DBB	3	405	405	405	405	365	365	355	—	—
8	SDWS22800DB SDWS22800DBB	3	405	405	405	405	395	395	395	395	—
10	SDWS221000DB SDWS221000DBB	3	405	405	405	405	395	395	395	395	395

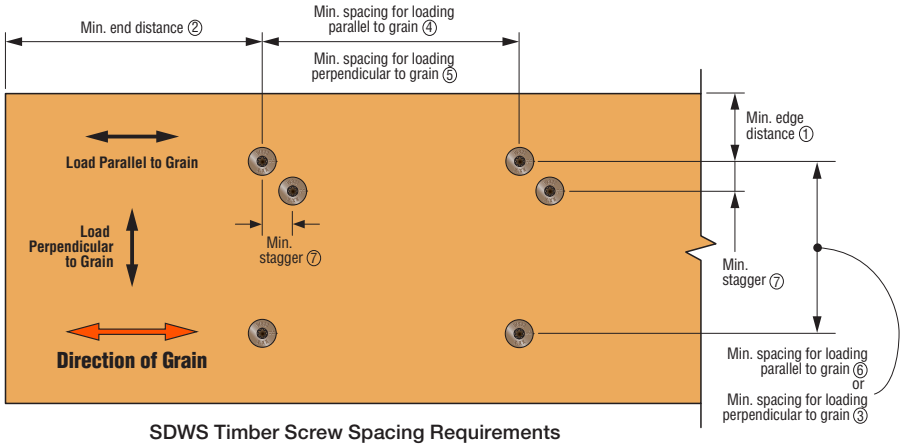
See footnotes below.

SDWS Timber Screw — Allowable Shear Loads — Spruce-Pine-Fir and Hem-Fir Lumber

Fastener Length (in.)	Model No.	Thread Length (in.)	Reference SPF/HF Allowable Shear Loads (lb.)								
			Wood Side Member Thickness (in.)								
			1.5	2	2.5	3	3.5	4	4.5	6	8
3	SDWS22300DB SDWS22300DBB	1½	190	—	—	—	—	—	—	—	—
4	SDWS22400DB SDWS22400DBB	2⅜	385	285	215	—	—	—	—	—	—
5	SDWS22500DB SDWS22500DBB	3	405	290	290	290	195	—	—	—	—
6	SDWS22600DB SDWS22600DBB	3	405	365	365	365	310	310	210	—	—
8	SDWS22800DB SDWS22800DBB	3	405	365	365	365	310	310	280	280	—
10	SDWS221000DB SDWS221000DBB	3	405	365	365	365	310	310	280	280	280

1. All applications are based on full penetration into the main member. Full penetration is the screw length minus the side member thickness.
2. Allowable loads are shown at the wood load duration factor of C_D = 1.0. Loads may be increased for load duration per the building code up to a C_D = 1.6. Tabulated values must be multiplied by all applicable adjustment factors per the NDS.
3. For minimum fastener spacing requirements for both side and main members, see the Spacing Requirements Figure and Table on the next page.
4. For in-service moisture content greater than 19%, use C_M = 0.7.
5. Loads are based on installation into the side grain of the wood with the screw axis perpendicular to the face of the member.

Strong-Drive® SDWS™ **TIMBER** Screw (Exterior Grade) (cont.)



SDWS Timber Screw Spacing Requirements

SDWS Timber Screw Spacing Requirements

Condition	Direction of Load to Grain	ID	Minimum Distance or Spacing (in.)
Edge Distance	Perpendicular	①	1 7/16
	Parallel	①	1 7/16
End Distance	Perpendicular	②	6
	Parallel	②	6
Spacing Between Fasteners in a Row	Perpendicular	③	4
	Parallel	④	8
Spacing Between Rows of Fasteners	Perpendicular	⑤	4
	Parallel	⑥	4
Spacing Between Staggered Rows	Perpendicular or Parallel	⑦	5/8

1. For axial loading only, use the following minimum dimensions: end distance = 3¼", edge distance = 1⅜", spacing parallel to grain = 2¼", spacing perpendicular to grain = 1⅜".

SDWS Timber Screw — Reference Allowable Withdrawal Loads — Douglas Fir–Larch, Southern Pine, Spruce-Pine-Fir and Hem-Fir Lumber

Model No.	Fastener Length (in.)	Thread Length (in.)	Reference Allowable Withdrawal Loads, W (lb./in.)		Max. Reference Allowable Withdrawal Loads, W_{max} (lb.)	
			DFL and SP Main Member	HF and SPF Main Member	DFL and SP Main Member	HF and SPF Main Member
SDWS22300DB SDWS22300DBB	3	1½	164	151	245	225
SDWS22400DB SDWS22400DBB	4	2⅜	179	160	425	380
SDWS22500DB SDWS22500DBB	5	3	214	187	590	495
SDWS22600DB SDWS22600DBB	6	3	214	187	590	495
SDWS22800DB SDWS22800DBB	8	3	214	187	590	495
SDWS221000DB SDWS221000DBB	10	3	214	187	590	495

1. The tabulated reference withdrawal design value, W, is in pounds per inch of the thread penetration into the side grain of the main member.
2. The tabulated reference withdrawal design value, W_{max}, is in pounds where the entire thread length must penetrate into the side grain of the main member.
3. The tabulated reference withdrawal design values, W and W_{max}, are shown at a C_D = 1.0. For end-grain withdrawal, C_{eg} = 0.65. Tabulated values must be multiplied by all applicable adjustment factors from the NDS as referenced in the IBC or IRC.
4. Embedded thread length is that portion held in the main member including the screw tip.
5. Values are based on the lesser of withdrawal from the main member or pull-through of a 1½" side member.
6. For in-service moisture content greater than 19%, use C_M = 0.7.

Twist Straps

Twist straps provide a tension connection between two wood members. They resist uplift at the heel of a truss economically. LTS/MTS have a 2"-bend section and HTS has a 3¾"-bend section that eliminates interference at the transition points between the two members.

Material: LTS — 18 gauge; MTS — 16 gauge; HTS — 14 gauge

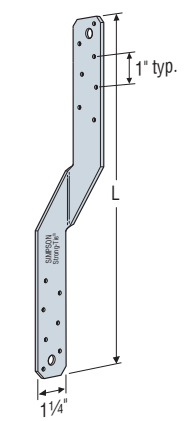
Finish: Galvanized. Some products available in stainless steel or ZMAX® coating.

Installation:

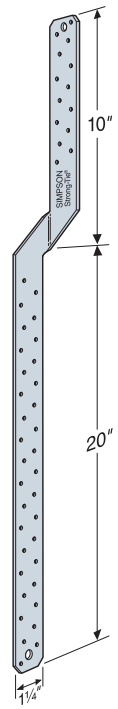
- Use all specified fasteners, with half into each member being connected, to achieve the listed loads; see General Notes.
- LTS, MTS and HTS are available with the bend reversed. Specify "-REV" after the model number, such as MTS16-REV.

Codes: See p. 13 for Code Reference Key Chart

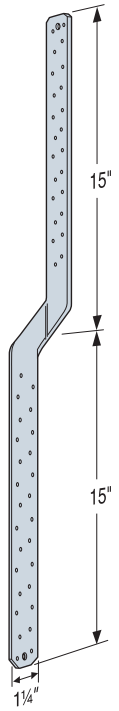
Web Applications: Visit **app.strongtie.com/rws** to access our Roof-to-Wall Selector web application.



LTS12
(MTS and HTS similar)



MTS30
(HTS30 similar)



HTS30C
(MTS30C similar)

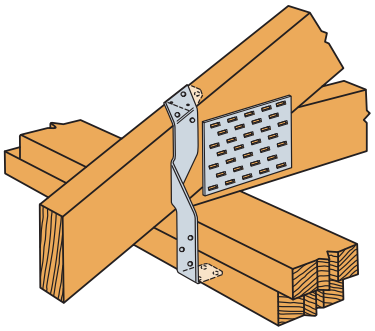
These products are available with additional corrosion protection. For more information, see p. 16.

SS For stainless-steel fasteners, see p. 23.

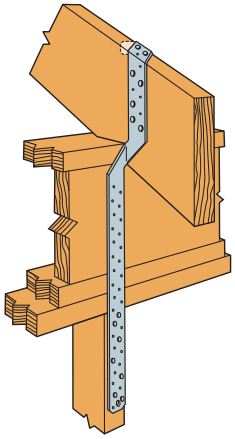
SD Many of these products are approved for installation with Strong-Drive® SD Connector screws. See pp. 362–366 for more information.

	Model No.	Strap Length (in.)	Total Quantity of Fasteners		DF/SP Allowable Uplift Loads (160)		SPF/HF Allowable Uplift Loads (160)		Code Ref.
			0.148" x 3" Nails	0.148" x 1½" Nails	0.148" x 3" Nails	0.148" x 1½" Nails	0.148" x 3" Nails	0.148" x 1½" Nails	
SS	LTS12	12	12	12	660	645	570	515	IBC®, FL, LA
	LTS16	16							
	LTS20	20							
SS	MTS12	12	14	14	990	990	850	850	
	MTS16	16							
SS	MTS20	20							
	MTS24C	24							
	MTS30	30							
	MTS30C	30							
	HTS16	16	16	16	1,415	1,415	1,215	1,215	
	HTS20	20							
	HTS24	24							
	HTS30	30							
	HTS30C	30							

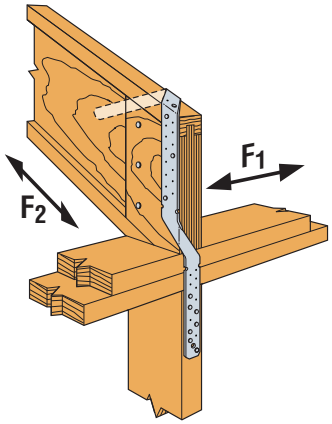
1. See pp. 276–277 for Straps and Ties General Notes.
2. All LTS, MTS and HTS models (except for MTS12 and HTS16) have additional nail holes.
3. All straps except the MTS30 and HTS30 have the twist in the center of the strap.
4. Twist straps do not need to be wrapped over the truss to achieve the allowable load.
5. May be installed on the inside face of the stud.
6. Allowable lateral loads are F_1 = 75 lb. and F_2 = 125 lb. when the following installation requirements are met. The first seven nail holes on each side of the bend must be filled with 0.148" x 1½" minimum nails. All additional fasteners may be installed in any remaining strap holes.
7. For simultaneous loads in more than one direction, the connector must be evaluated using either the Unity Equation or the 75% Rule, as described in Straps and Ties General Notes on p. 277.
8. **Fasteners:** Nail dimensions are listed diameter by length. See pp. 23–24 for fastener information.
9. Using Strong-Drive® SD Connector (SD9112) screws for 0.148" x 1½" and 0.148" x 3" nails will get the same loads as using nails.



MTS Installation as a Truss-to-Top Plate Tie



Typical MTS30 Installation



MTS30 Installation with I-Joist Rafter

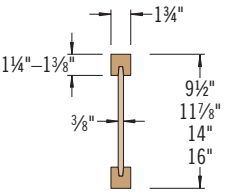
SECTION 1: 9½"–16" TJI® JOISTS

This section contains design information for 9½"–16" deep Trus Joist® TJI® joists.
These standard-size TJI® joists are readily available through your local Weyerhaeuser dealer or distributor.
Offered with the flange sizes shown below, they come in lengths up to 60' (in 1' increments).

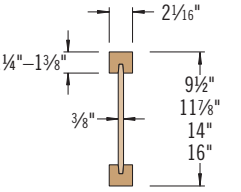
Design Properties (100% Load Duration)

Depth	TJI®	Basic Properties				Reaction Properties					
		Joist Weight (lbs/ft)	Maximum Resistive Moment(1) (ft-lbs)	Joist Only EI x 10^6 ($in.^2$-lbs)	Maximum Vertical Shear (lbs)	1¾" End Reaction (lbs)	3½" End Reaction (lbs)	3½" Intermediate Reaction (lbs)		5¼" Intermediate Reaction (lbs)	
								No Web Stiffeners	With Web Stiffeners(2)	No Web Stiffeners	With Web Stiffeners(2)
9½"	110	2.3	2,500	157	1,220	910	1,220	1,935	N.A.	2,350	N.A.
	210	2.6	3,000	186	1,330	1,005	1,330	2,145	N.A.	2,565	N.A.
	230	2.7	3,330	206	1,330	1,060	1,330	2,410	N.A.	2,790	N.A.
11⅞"	110	2.5	3,160	267	1,560	910	1,375	1,935	2,295	2,350	2,705
	210	2.8	3,795	315	1,655	1,005	1,460	2,145	2,505	2,565	2,925
	230	3.0	4,215	347	1,655	1,060	1,485	2,410	2,765	2,790	3,150
	360	3.0	6,180	419	1,705	1,080	1,505	2,460	2,815	3,000	3,360
	560	4.0	9,500	636	2,050	1,265	1,725	3,000	3,475	3,455	3,930
14"	110	2.8	3,740	392	1,860	910	1,375	1,935	2,295	2,350	2,705
	210	3.1	4,490	462	1,945	1,005	1,460	2,145	2,505	2,565	2,925
	230	3.3	4,990	509	1,945	1,060	1,485	2,410	2,765	2,790	3,150
	360	3.3	7,335	612	1,955	1,080	1,505	2,460	2,815	3,000	3,360
	560	4.2	11,275	926	2,390	1,265	1,725	3,000	3,475	3,455	3,930
16"	110	3.0	4,280	535	2,145	910	1,375	1,935	2,295	2,350	2,705
	210	3.3	5,140	629	2,190	1,005	1,460	2,145	2,505	2,565	2,925
	230	3.5	5,710	691	2,190	1,060	1,485	2,410	2,765	2,790	3,150
	360	3.5	8,405	830	2,190	1,080	1,505	2,460	2,815	3,000	3,360
	560	4.5	12,925	1,252	2,710	1,265	1,725	3,000	3,475	3,455	3,930

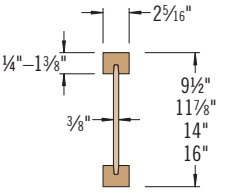
(1) **Caution: Do not** increase joist moment design properties by a repetitive member use factor.
(2) See detail W on page 29 for web stiffener requirements and nailing information.



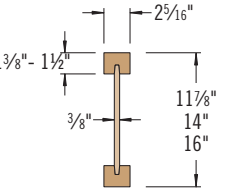
TJI® 110 joists



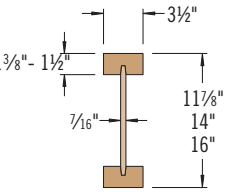
TJI® 210 joists



TJI® 230 joists



TJI® 360 joists



TJI® 560 joists

General Notes

- Design reaction includes all loads on the joist. Design shear is computed at the inside face of supports and includes all loads on the span(s). Allowable shear may sometimes be increased at interior supports in accordance with ICC-ES ESR-1153, and these increases are reflected in span tables.
- The formulas at right approximate the uniform load deflection of Δ (inches).

TJI® joists are intended for dry-use applications

Some TJI® joist series may not be available in your region. Contact your Weyerhaeuser representative for information.

For TJI® 110, 210, 230, and 360 Joists

$$\Delta = \frac{22.5\ wL^4}{EI} + \frac{2.67\ wL^2}{d \times 10^5}$$

For TJI® 560 Joists

$$\Delta = \frac{22.5\ wL^4}{EI} + \frac{2.29\ wL^2}{d \times 10^5}$$

w = uniform load in pounds per linear foot
L = span in feet
d = out-to-out depth of the joist in inches
EI = value from table above


DO NOT walk on joists until braced. INJURY MAY RESULT.


DO NOT stack building materials on unsheathed joists. Stack only over beams or walls.


DO NOT walk on joists that are lying flat.

WARNING

Joists are unstable until braced laterally

Bracing Includes:
- **Blocking**
- **Hangers**
- **Rim Board**
- **Sheathing**
- **Rim Joist**
- **Strut Lines**

WARNING NOTES: *Lack of proper bracing during construction can result in serious accidents. Observe the following guidelines:*

1. All blocking, hangers, rim boards, and rim joists at the end supports of the TJI® joists must be completely installed and properly nailed.
2. Lateral strength, like a braced end wall or an existing deck, must be established at the ends of the bay. This can also be accomplished by a temporary or permanent deck (sheathing) fastened to the first 4 feet of joists at the end of the bay.
3. Safety bracing of 1x4 (minimum) must be nailed to a braced end wall or sheathed area (as in note 2) and to each joist. Without this bracing, buckling sideways or rollover is highly probable under light construction loads—such as a worker or one layer of unnailed sheathing.
4. Sheathing must be completely attached to each TJI® joist before additional loads can be placed on the system.
5. Ends of cantilevers require safety bracing on both the top and bottom flanges.
6. The flanges must remain straight within a tolerance of ½" from true alignment.

IUS/MIU

I-Joist Hangers

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.

The IUS is a hybrid hanger that incorporates the advantages of the face-mount and top-mount hanger. Installation is fast with the Strong-Grip™ seat, easy-to-reach face nails and self-jigging locator tabs.

The MIU series hangers are designed for commercial and high-load I-joist applications without requiring web stiffeners. The MIU features Positive Angle Nailing (PAN), which minimizes splitting of the flanges while permitting time-saving nailing from a better angle.

Material: IUS — 18 gauge; MIU — 16 gauge

Finish: Galvanized

Uplift Loads:

- Models have optional triangle joist nail holes for additional uplift. Properly attached web stiffeners are required.
- MIU — add four additional 0.148" x 1½" joist nails for a total uplift load of 975 lb.
- IUS — add web stiffeners and two 0.148" x 1½" joist nails in the triangle holes for a total uplift of 365 lb.

With Web Stiffeners

Installation:

- Use all specified fasteners. Verify that the header can take the required fasteners specified in the table. Do not overspread hanger. Side flanges should be in contact with top chord of I-Joist. See pp. 99–100 for more installation information.
- IUS — fasten hanger to header. Position I-joist into hanger and snap into place. No joist nailing required. Some IUS models have triangle and round header nail holes. To achieve max. download, fill both round and triangle holes.
- IUS — Locator tabs are not structural. They may be bent back to adjust for hanger placement.
- IUS — for rimboard applications see technical bulletin T-C-RIMBDHGR at **strongtie.com**.
- IUS — I-joists with web stiffeners or rectangular sections can be used with the installation of (2) 0.148" x 1½" nails into the optional triangle joist nails.
- IUS — web stiffeners are not required with I-joists when the top flange is laterally supported by the sides of the hanger unless required by I-joist manufacturer.

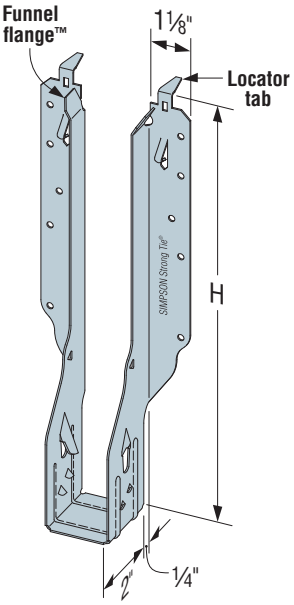
Options:

- These hangers cannot be modified. However, these models will normally accommodate a skew of up to 5°. For sloped joists up to ¼:12 there is no reduction; between ¼:12 and up to ½:12, tests show a 10% reduction in ultimate hanger strength. Local crushing of the bottom flange or excessive deflection may be limiting; check with joist manufacturer for specific limitations on bearing of this type.

Roof Slope is 1/4":12

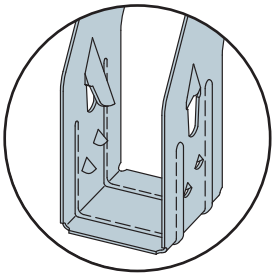
Codes: See p. 13 for Code Reference Key Chart

Web Applications: Visit **app.strongtie.com/hs** to access our Hanger Selector web application.



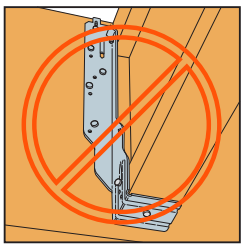
IUS

(some IUS models have triangle holes in header flanges for min./max. nailing)



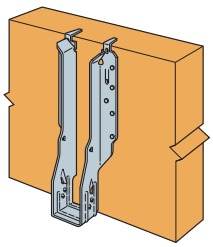
The Strong-Grip™ seat secures I-joists in position without joist nails.

Avoid a Misinstallation

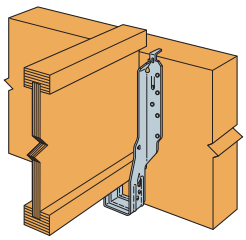


Do not make your own holes. Do not nail the bottom flange.

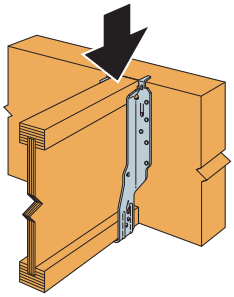
IUS Installation Sequence



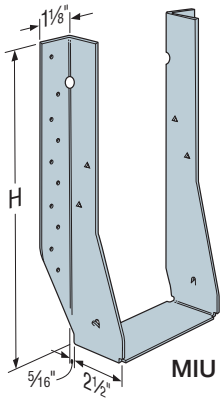
Step 1
Attach the IUS to the header.



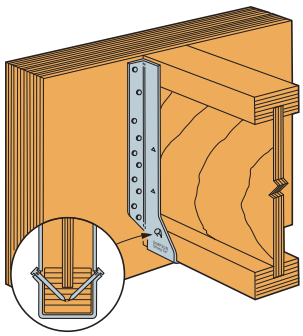
Step 2
Slide the I-joist downward into the IUS until it rests above the large teardrop.



Step 3
Firmly push or snap I-joist fully into the seat of the IUS.



MIU



MIU with Correct PAN Installation

U/HU/HUC/HUCQ

Face-Mount Hangers

See hanger tables on pp. 156–162.

U — The standard U hanger provides flexibility of joist to header installation. Versatile fastener selection with tested allowable loads.

HU/HUC — Most models have triangle and round holes. To achieve maximum loads, fill both round and triangle holes with common nails.

HUCQ — Features concealed flanges so it can be installed close to the end of the supporting beam or on a post. They install with Strong-Drive® SDS Heavy-Duty Connector screws (supplied with the hanger) for high capacity and ease of installation.

Feature:

- HUCQ only — Fire-resistant F (flame) and T (temperature) rated in Intertek Design No. SST/WPCF 120-01.

Material: U— 16 gauge; HU/HUC/HUCQ — 14 gauge

Finish: Galvanized

Installation:

- Use all specified fasteners; see General Notes.
- HU/HUC — Can be installed filling round holes only, or filling round and triangle holes for maximum values.
- HUCQ — When using structural composite lumber columns, the capacities shown in the tables are for fasteners applied to the wide face of the column.
- Web stiffeners are required for all I-joists used with these hangers.
- For installation to masonry or concrete, see pp. 253–255.
- HU/HUC/HUCQ hangers can be welded to a steel member. For HU/HUC allowable loads, refer to technical bulletin T-C-HUHUC-W at **strongtie.com**. HUCQ allowable loads listed in hanger tables on pp. 156–162 apply when installed with minimum (6) 1" welds.

Options:

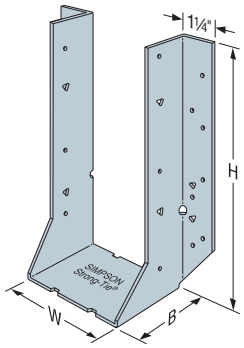
- Order HUC_X hanger. For both flanges concealed, order HUC.

Sloped, Skewed and Sloped/Skewed:

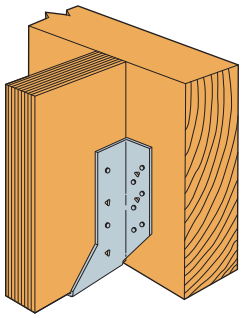
- For low-cost, code-approved 45° skewed hangers, see SUR/SUL on pp. 164–165.
- For field-adjustable hangers, see LSSR on pp. 166–167.
- See modification table for available options and associated load capacities for U and HU hangers.
- HUCQ cannot be modified.

Codes: See p. 13 for Code Reference Key Chart

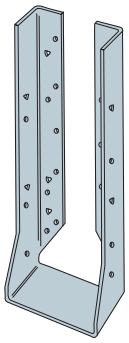
Web Applications:
Visit **app.strongtie.com/hs** to access our Hanger Selector web application.



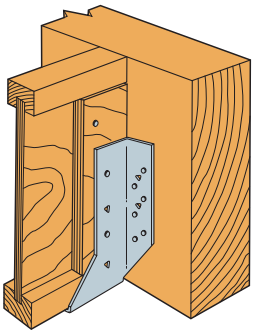
HU410



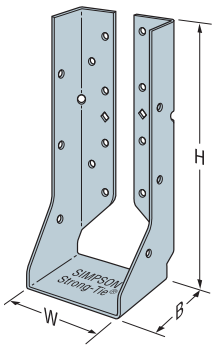
Typical HU7 Installation



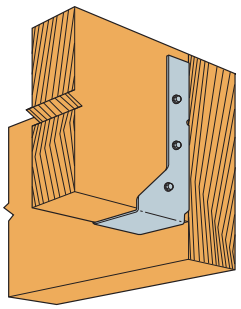
HUC412
Concealed Flanges



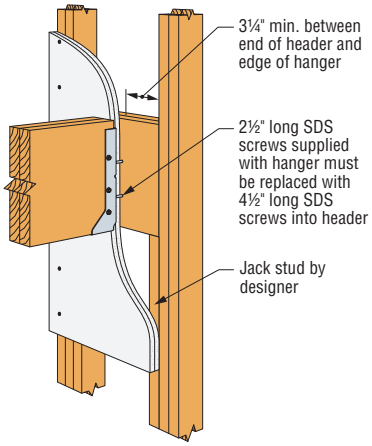
Typical HU7 Installation



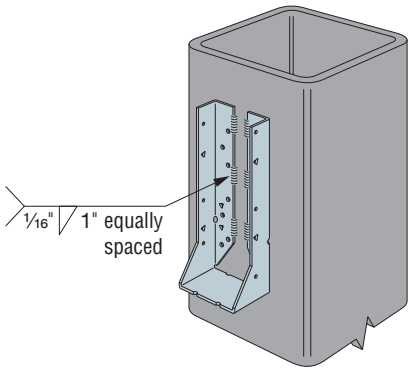
HUCQ



Typical HUCQ Installed on End of a Beam



HUCQ Over Two Layers of ⅝" Drywall



HUC Welded to Steel Column
(HUCQ similar)

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

Codes: See p. 13 for Code Reference Key Chart.

Web Applications: Visit **app.strongtie.com/hs** to access our Hanger Selector web application.

Actual Joist Size (in.)	Model No.	Carried Member				Dimensions (in.)			Min./ Max.	Fasteners (in.)		Allowable Loads							Code Ref.
		Glulam	SCL	I-joist	Web Stiff Reqd.	W	H	B		Face	Joist	DF/SP Species Header				SPF/HF Species Header			
												Uplift (160)	Floor (100)	Snow (115)	Roof (125)	Floor (100)	Snow (115)	Roof (125)	
1½ x 9½	U210			•	✓	1⅜	7 13/16	2	—	(6) 0.148 x 3	(6) 0.148 x 1½	990	1,220	1,380	1,480	1,050	1,185	1,275	IBC®, FL, LA
	MIU1.56/9			•	—	1⅜	8 15/16	2½	—	(16) 0.162 x 3½	(2) 0.148 x 1½	230	2,305	2,615	2,820	1,980	2,245	2,425	
1½ x 11⅞	U210			•	✓	1⅜	7 13/16	2	—	(6) 0.148 x 3	(6) 0.148 x 1½	990	1,220	1,380	1,480	1,050	1,185	1,275	
	MIU1.56/11			•	—	1⅜	11 1/16	2½	—	(20) 0.162 x 3½	(2) 0.148 x 1½	230	2,880	3,135	3,135	2,475	2,695	2,695	
1¾ x 5½	HU1.81/5		•		—	1 13/16	5⅜	2½	Min.	(12) 0.162 x 3½	(4) 0.148 x 1½	610	1,785	2,015	2,165	1,540	1,735	1,865	
									Max.	(16) 0.162 x 3½	(6) 0.148 x 1½	915	2,380	2,685	2,890	2,050	2,315	2,490	
1¾ x 7¼	HU7		•		—	1 13/16	6 1/16	2½	Min.	(12) 0.162 x 3½	(4) 0.148 x 1½	610	1,785	2,015	2,165	1,540	1,735	1,865	
									Max.	(16) 0.162 x 3½	(8) 0.148 x 1½	1515	2,380	2,685	2,890	2,050	2,315	2,490	
1¾ x 9½	IUS1.81/9.5			•	—	1⅞	9½	2	—	(8) 0.148 x 3	—	70	950	1,080	1,165	815	925	1,000	
	HU9		•	•	✓	1 13/16	9 5/16	2½	Min.	(18) 0.162 x 3½	(6) 0.148 x 1½	915	2,680	3,020	3,250	2,305	2,605	2,800	
									Max.	(24) 0.162 x 3½	(10) 0.148 x 1½	1,795	3,570	4,030	4,335	3,075	3,470	3,735	
	HUS1.81/10		•		—	1 13/16	8⅞	3	—	(30) 0.162 x 3½	(10) 0.162 x 3½	2,675	5,510	5,830	5,830	4,360	4,675	4,885	
	HUCQ1.81/9-SDS		•		—	1 13/16	9	3	—	(8) ¼ x 1¾ SDS	(4) ¼ x 1¾ SDS	1,310	2,000	2,300	2,500	1,440	1,655	1,800	
	MIU1.81/9		•	•	—	1 13/16	8 13/16	2½	—	(16) 0.162 x 3½	(2) 0.148 x 1½	230	2,305	2,615	2,820	1,980	2,245	2,425	
1¾ x 11⅞	IUS1.81/11.88			•	—	1⅞	11⅞	2	—	(10) 0.148 x 3	—	70	1,185	1,345	1,455	1,020	1,160	1,250	
	MIU1.81/11		•	•	—	1 13/16	11 1/16	2½	—	(20) 0.162 x 3½	(2) 0.148 x 1½	230	2,880	3,135	3,135	2,475	2,695	2,695	
	HUS1.81/10		•		—	1 13/16	8⅞	3	—	(30) 0.162 x 3½	(10) 0.162 x 3½	2,675	5,510	5,830	5,830	4,360	4,675	4,885	
	HU11		•	•	✓	1 13/16	11 1/16	2½	Min.	(22) 0.162 x 3½	(6) 0.148 x 1½	915	3,275	3,695	3,970	2,820	3,180	3,425	
									Max.	(30) 0.162 x 3½	(10) 0.148 x 1½	1,795	4,465	4,705	4,810	3,845	4,340	4,600	
	HUCQ1.81/11-SDS		•		—	1 13/16	11	3	—	(10) ¼ x 1¾ SDS	(4) ¼ x 1¾ SDS	1,310	2,500	2,875	3,125	1,800	2,070	2,250	
1¾ x 14	IUS1.81/14		•	•	—	1⅞	14	2	Min.	(12) 0.148 x 3	—	70	1,420	1,615	1,745	1,220	1,390	1,500	
									Max.	(14) 0.148 x 3	—	70	1,660	1,805	1,805	1,425	1,550	1,550	
	MIU1.81/14		•	•	—	1 13/16	13 5/16	2½	—	(22) 0.162 x 3½	(2) 0.148 x 1½	230	3,170	3,595	3,875	2,725	3,090	3,335	
	HUS1.81/10		•		—	1 13/16	8⅞	3	—	(30) 0.162 x 3½	(10) 0.162 x 3½	2,675	5,510	5,830	5,830	4,360	4,675	4,885	
	U14		•	•	✓	1 13/16	10¼	2	—	(14) 0.162 x 3½	(6) 0.148 x 1½	970	2,015	2,285	2,465	1,735	1,965	2,120	
	HU14		•	•	✓	1 13/16	13 11/16	2½	Min.	(28) 0.162 x 3½	(8) 0.148 x 1½	1,515	4,165	4,420	4,505	3,590	4,050	4,335	
									Max.	(36) 0.162 x 3½	(14) 0.148 x 1½	1,795	5,055	5,275	5,420	4,615	5,000	5,130	
	HUCQ1.81/11-SDS		•		—	1 13/16	11	3	—	(10) ¼ x 1¾ SDS	(4) ¼ x 1¾ SDS	1,310	2,500	2,875	3,125	1,800	2,070	2,250	
1¾ x 16	IUS1.81/16			•	—	1⅞	16	2	Min.	(14) 0.148 x 3	—	70	1,660	1,805	1,805	1,425	1,555	1,555	
									Max.	(16) 0.148 x 3	—	70	1,805	1,805	1,805	1,555	1,555	1,555	
	MIU1.81/16			•	—	1 13/16	15 5/16	2½	—	(24) 0.162 x 3½	(2) 0.148 x 1½	230	3,455	3,920	4,045	2,970	3,370	3,480	
1¾ x 18	MIU1.81/18			•	—	1 13/16	17 5/16	2½	—	(26) 0.162 x 3½	(2) 0.148 x 1½	230	3,745	4,020	4,045	3,220	3,460	3,480	
2 x 9½	IUS2.06/9.5			•	—	2⅛	9½	2	—	(8) 0.148 x 3	—	70	950	1,080	1,165	815	925	1,000	
	HU2.1/9			•	✓	2⅛	9 3/16	2½	—	(14) 0.162 x 3½	(6) 0.148 x 1½	915	2,085	2,350	2,530	1,795	2,025	2,180	
2 x 11⅞	IUS2.06/11.88			•	—	2⅛	11⅞	2	—	(10) 0.148 x 3	—	70	1,185	1,345	1,455	1,020	1,160	1,250	
	MIU2.1/11			•	✓	2⅛	11 1/16	2½	—	(20) 0.162 x 3½	(2) 0.148 x 1½	230	2,880	3,135	3,135	2,475	2,695	2,695	
	HU2.1/11			•	✓	2⅛	11	2½	—	(16) 0.162 x 3½	(6) 0.148 x 1½	915	2,380	2,685	2,890	2,050	2,315	2,490	
2 x 14	IUS2.06/14			•	—	2⅛	14	2	Min.	(12) 0.148 x 3	—	70	1,420	1,615	1,745	1,220	1,390	1,500	
									Max.	(14) 0.148 x 3	—	70	1,660	1,805	1,805	1,425	1,555	1,555	
	MIU2.1/11			•	✓	2⅛	11 1/16	2½	—	(20) 0.162 x 3½	(2) 0.148 x 1½	230	2,880	3,135	3,135	2,475	2,695	2,695	
	HU2.1/11			•	✓	2⅛	11	2½	—	(16) 0.162 x 3½	(6) 0.148 x 1½	915	2,380	2,685	2,890	2,050	2,315	2,490	
2 x 16	IUS2.06/16			•	—	2⅛	16	2	Min.	(14) 0.148 x 3	—	70	1,660	1,805	1,805	1,425	1,555	1,555	
									Max.	(16) 0.148 x 3	—	70	1,805	1,805	1,805	1,555	1,555	1,555	
	HU2.1/11			•	✓	2⅛	11	2½	—	(16) 0.162 x 3½	(6) 0.148 x 1½	915	2,380	2,685	2,890	2,050	2,315	2,490	
2⅛ x 9½	IUS2.06/9.5			•	—	2⅛	9½	2	—	(8) 0.148 x 3	—	70	950	1,080	1,165	815	925	1,000	
	HU2.1/9			•	✓	2⅛	9 3/16	2½	—	(14) 0.162 x 3½	(6) 0.148 x 1½	915	2,085	2,350	2,530	1,795	2,025	2,180	
2⅛ x 11⅞	IUS2.06/11.88			•	—	2⅛	11⅞	2	—	(10) 0.148 x 3	—	70	1,185	1,345	1,455	1,020	1,160	1,250	
	MIU2.1/11			•	✓	2⅛	11 1/16	2½	—	(20) 0.162 x 3½	(2) 0.148 x 1½	230	2,880	3,135	3,135	2,475	2,695	2,695	
	HU2.1/11			•	✓	2⅛	11	2½	—	(16) 0.162 x 3½	(6) 0.148 x 1½	915	2,380	2,685	2,890	2,050	2,315	2,490	

See footnotes on p. 162.

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

Codes: See p. 13 for Code Reference Key Chart.

Actual Joist Size (in.)	Model No.	Carried Member				Dimensions (in.)			Min./ Max.	Fasteners (in.)		Allowable Loads							Code Ref.
		Glulam	SCL	I-joist	Web Stiff Reqd.	W	H	B		Face	Joist	DF/SP Species Header				SPF/HF Species Header			
												Uplift (160)	Floor (100)	Snow (115)	Roof (125)	Floor (100)	Snow (115)	Roof (125)	
2⅛6 x 14	IUS2.06/14			•	—	2⅛	14	2	—	(12) 0.148 x 3	—	70	1,420	1,615	1,745	1,220	1,390	1,500	IBC®, FL, LA
	MIU2.1/11			•	✓	2⅛	11 1⅛6	2½	—	(20) 0.162 x 3½	(2) 0.148 x 1½	230	2,880	3,135	3,135	2,475	2,695	2,695	
	HU2.1/11			•	✓	2⅛	11	2½	—	(16) 0.162 x 3½	(6) 0.148 x 1½	915	2,380	2,685	2,890	2,050	2,315	2,490	
2⅛6 x 16	IUS2.06/16			•	—	2⅛	16	2	—	(14) 0.148 x 3	—	70	1,660	1,805	1,805	1,425	1,555	1,555	
	MIU2.1/11			•	✓	2⅛	11 1⅛6	2½	—	(20) 0.162 x 3½	(2) 0.148 x 1½	230	2,880	3,135	3,135	2,475	2,695	2,695	
	HU2.1/11			•	✓	2⅛	11	2½	—	(16) 0.162 x 3½	(6) 0.148 x 1½	915	2,380	2,685	2,890	2,050	2,315	2,490	
2¼ x 9½ to 20	2¼"-wide joists use the same hangers as 2⅝"-wide joists with the following load adjustments to the table loads: IUS download is the lesser of the table load or 1,400 lb.; IUS uplift is 55 lb.; MIU and U downloads are the lesser of the table load or 2,140 lb.																		
2⅝6 x 9½	IUS2.37/9.5			•	—	2⅞6	9½	2	—	(8) 0.148 x 3	—	70	950	1,080	1,165	815	925	1,000	IBC, FL, LA
	MIU2.37/9			•	—	2⅜	9	2½	—	(16) 0.162 x 3½	(2) 0.148 x 1½	230	2,305	2,615	2,820	1,980	2,245	2,425	
	U3510/14			•	✓	2⅝6	9	2	—	(14) 0.162 x 3½	(6) 0.148 x 1½	970	2,015	2,285	2,465	1,735	1,965	2,120	
	HU359 / HUC359			•	✓	2⅜	9	2½	Min.	(14) 0.162 x 3½	(6) 0.148 x 1½	915	2,085	2,350	2,530	1,795	2,025	2,180	
									Max.	(18) 0.162 x 3½	(10) 0.148 x 1½	1,795	2,680	3,020	3,250	2,305	2,605	2,800	
2⅝6 x 11⅞	IUS2.37/11.88			•	—	2⅞6	11⅞	2	—	(10) 0.148 x 3	—	70	1,185	1,345	1,455	1,020	1,160	1,250	
	MIU2.37/11			•	—	2⅜	11 1⅛6	2½	—	(20) 0.162 x 3½	(2) 0.148 x 1½	230	2,880	3,135	3,135	2,475	2,695	2,695	
	U3516/20			•	✓	2⅝6	10⅞6	2	—	(16) 0.162 x 3½	(6) 0.148 x 1½	970	2,305	2,615	2,820	1,980	2,245	2,425	
	HU3511 / HUC3511			•	✓	2⅜	10 15⅛6	2½	Min.	(16) 0.162 x 3½	(6) 0.148 x 1½	915	2,380	2,685	2,890	2,050	2,315	2,490	
									Max.	(22) 0.162 x 3½	(10) 0.148 x 1½	1,795	3,275	3,695	3,970	2,820	3,180	3,425	
2⅝6 x 14	IUS2.37/14			•	—	2⅞6	14	2	Min.	(12) 0.148 x 3	—	70	1,420	1,615	1,745	1,220	1,390	1,500	
									Max.	(14) 0.148 x 3	—	70	1,660	1,805	1,805	1,425	1,555	1,555	
	MIU2.37/14			•	—	2⅜	13½	2½	—	(22) 0.162 x 3½	(2) 0.148 x 1½	230	3,170	3,595	3,875	2,725	3,090	3,335	
	U3516/20			•	✓	2⅝6	10⅞6	2	—	(16) 0.162 x 3½	(6) 0.148 x 1½	970	2,305	2,615	2,820	1,980	2,245	2,425	
	HU3514 / HUC3514			•	✓	2⅜	12½	2½	Min.	(18) 0.162 x 3½	(8) 0.148 x 1½	1,515	2,680	3,020	3,250	2,305	2,605	2,800	
									Max.	(24) 0.162 x 3½	(12) 0.148 x 1½	1,795	3,570	4,030	4,335	3,075	3,470	3,735	
2⅝6 x 16	IUS2.37/16			•	—	2⅞6	16	2	Min.	(14) 0.148 x 3	—	70	1,660	1,805	1,805	1,425	1,555	1,555	
									Max.	(16) 0.148 x 3	—	70	1,805	1,805	1,805	1,555	1,555	1,555	
	MIU2.37/16			•	—	2⅜	15½	2½	—	(24) 0.162 x 3½	(2) 0.148 x 1½	230	3,455	3,920	4,045	2,970	3,370	3,480	
	U3516/20			•	✓	2⅝6	10⅞6	2	—	(16) 0.162 x 3½	(6) 0.148 x 1½	970	2,305	2,615	2,820	1,980	2,245	2,425	
	HU3516/22 / HUC3516/22			•	✓	2⅜	14¼	2½	—	(20) 0.162 x 3½	(8) 0.148 x 1½	1,515	2,975	3,360	3,610	2,565	2,895	3,110	
2⅝6 x 18	MIU2.37/18			•	—	2⅜	17½	2½	—	(26) 0.162 x 3½	(2) 0.148 x 1½	230	3,745	4,045	4,045	3,220	3,480	3,480	
	HU3524/30			•	✓	2⅜	18	2½	Min.	(18) 0.162 x 3½	(8) 0.148 x 1½	1,515	2,680	3,020	3,250	2,305	2,605	2,800	
									Max.	(24) 0.162 x 3½	(14) 0.148 x 1½	1,795	3,570	4,030	4,335	3,075	3,470	3,735	
2⅝6 x 20	MIU2.37/20			•	—	2⅜	19½	2½	—	(28) 0.162 x 3½	(2) 0.148 x 1½	230	4,030	4,060	4,060	3,465	3,495	3,495	
2⅝6 x 22 to 30	MIU2.37/20			•	✓	2⅜	19½	2½	—	(28) 0.162 x 3½	(2) 0.148 x 1½	230	4,030	4,060	4,060	3,465	3,495	3,495	
	HU3524/30			•	✓	2⅜	18	2½	Min.	(18) 0.162 x 3½	(8) 0.148 x 1½	1,515	2,680	3,020	3,250	2,305	2,605	2,800	
									Max.	(24) 0.162 x 3½	(14) 0.148 x 1½	1,795	3,570	4,030	4,335	3,075	3,470	3,735	
2½ x 9½	IUS2.56/9.5			•	—	2⅝	9½	2	—	(8) 0.148 x 3	—	70	950	1,080	1,165	815	925	1,000	IBC, FL, LA
	MIU2.56/9			•	—	2⅝6	8 15⅛6	2½	—	(16) 0.162 x 3½	(2) 0.148 x 1½	230	2,305	2,615	2,820	1,980	2,245	2,425	
	U310			•	✓	2⅝6	8⅞	2	—	(14) 0.162 x 3½	(6) 0.148 x 1½	970	1,705	1,930	2,075	1,465	1,660	1,785	
	HU310 / HUC310			•	✓	2⅝6	8⅞	2½	—	(14) 0.162 x 3½	(6) 0.148 x 1½	915	2,085	2,350	2,520	1,795	2,025	2,170	
2½ x 11⅞	IUS2.56/11.88			•	—	2⅝	11⅞	2	—	(10) 0.148 x 3	—	70	1,185	1,345	1,455	1,020	1,160	1,250	
	MIU2.56/11			•	—	2⅝6	11 1⅛6	2½	—	(20) 0.162 x 3½	(2) 0.148 x 1½	230	2,880	3,135	3,135	2,475	2,695	2,695	
	U314			•	✓	2⅝6	10½	2	—	(16) 0.162 x 3½	(6) 0.148 x 1½	970	1,945	2,205	2,375	1,675	1,895	2,045	
	HU312 / HUC312			•	✓	2⅝6	10⅝	2½	—	(16) 0.162 x 3½	(6) 0.148 x 1½	915	2,380	2,685	2,890	2,050	2,315	2,490	
2½ x 14	IUS2.56/14			•	—	2⅝	14	2	Min.	(12) 0.148 x 3	—	70	1,420	1,615	1,745	1,220	1,390	1,500	
									Max.	(14) 0.148 x 3	—	70	1,660	1,805	1,805	1,425	1,555	1,555	
	MIU2.56/14			•	—	2⅝6	13⅞6	2½	—	(22) 0.162 x 3½	(2) 0.148 x 1½	230	3,170	3,595	3,875	2,725	3,090	3,335	
	U314			•	✓	2⅝6	10½	2	—	(16) 0.162 x 3½	(6) 0.148 x 1½	970	1,945	2,205	2,375	1,675	1,895	2,045	
	HU314 / HUC314			•	✓	2⅝6	12⅜	2½	—	(18) 0.162 x 3½	(8) 0.148 x 1½	1,515	2,680	3,020	3,250	2,305	2,605	2,800	

See footnotes on p. 162.

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

These products are available with additional corrosion protection. For more information, see p. 16.

SS For stainless-steel fasteners, see p. 23.

Codes: See p. 13 for Code Reference Key Chart.

Actual Joist Size (in.)	Model No.	Carried Member: Glulam	SCL	I-joist	Web Stiff Reqd.	Dimensions (in.): W	H	B	Min./ Max.	Fasteners (in.): Face	Joist	Allowable Loads: DF/SP Species Header: Uplift (160)	Floor (100)	Snow (115)	Roof (125)	SPF/HF Species Header: Floor (100)	Snow (115)	Roof (125)	Code Ref.
3½ x 14	IUS3.56/14			•	—	3⅝	14	2	Min.	(12) 0.148 x 3	—	70	1,420	1,615	1,745	1,220	1,390	1,500	IBC®, FL, LA
									Max.	(14) 0.148 x 3	—	70	1,660	1,805	1,805	1,425	1,555	1,555	
	MIU3.56/14			•		3⅜6	13⅝6	2½	—	(22) 0.162 x 3½	(2) 0.148 x 1½	210	3,170	3,595	3,875	2,725	3,090	3,335	
	U414	•	•	•	✓	3⅜6	10	2	—	(16) 0.162 x 3½	(6) 0.148 x 3	970	2,305	2,615	2,820	1,980	2,245	2,425	
	HHUS410	•	•		—	3⅝	9	3	—	(30) 0.162 x 3½	(10) 0.162 x 3½	3,565	5,635	6,380	6,445	4,845	5,486	5,545	
	HUS412	•	•		—	3⅜6	10½	2	—	(10) 0.162 x 3½	(10) 0.162 x 3½	3,635	2,660	3,025	3,265	2,275	2,590	2,795	
	HU414	•	•	•	✓	3⅜6	12⅝	2½	Max.	(24) 0.162 x 3½	(12) 0.148 x 3	1,795	3,570	4,030	4,335	3,075	3,470	3,735	
	HU416 / HUC416	•	•	•	✓	3⅜6	13⅝	2½	Min.	(20) 0.162 x 3½	(8) 0.148 x 3	1,515	2,975	3,360	3,610	2,565	2,895	3,110	
									Max.	(26) 0.162 x 3½	(12) 0.148 x 3	1,795	3,870	4,365	4,695	3,330	3,760	4,045	
SS	HUCQ412-SDS	•	•		—	3⅜6	11	3	—	(14) ¼ x 2½ SDS	(6) ¼ x 2½ SDS	2,265	5,045	5,045	5,045	3,630	3,630	3,630	
	HGUS414	•	•		—	3⅝	12⅞6	4	—	(66) 0.162 x 3½	(22) 0.162 x 3½	5,360	13,860	14,350	14,350	11,115	12,420	12,420	
	LGU3.63-SDS	•	•		—	3⅝	8 to 30	4½	—	(16) ¼ x 2½ SDS	(12) ¼ x 2½ SDS	5,555	6,720	6,720	6,720	4,840	4,840	4,840	
	MGU3.63-SDS	•	•		—	3⅝	9¼ to 30	4½	—	(24) ¼ x 2½ SDS	(16) ¼ x 2½ SDS	7,260	9,450	9,450	9,450	6,805	6,805	6,805	
	HGU3.63-SDS	•	•		—	3⅝	11 to 30	5¼	—	(36) ¼ x 2½ SDS	(24) ¼ x 2½ SDS	9,460	13,160	13,160	13,160	9,475	9,475	9,475	
3½ x 16	IUS3.56/16			•	—	3⅝	16	2	Min.	(14) 0.148 x 3	—	70	1,660	1,805	1,805	1,425	1,555	1,555	
									Max.	(16) 0.148 x 3	—	70	1,805	1,805	1,805	1,555	1,555	1,555	
	MIU3.56/16	•	•	•	—	3⅜6	15⅝6	2½	—	(24) 0.162 x 3½	(2) 0.148 x 1½	210	3,455	3,920	4,045	2,970	3,370	3,480	
	HU416 / HUC416	•	•	•	✓	3⅜6	13⅝	2½	Min.	(20) 0.162 x 3½	(8) 0.148 x 3	1,515	2,975	3,360	3,610	2,565	2,895	3,110	
									Max.	(26) 0.162 x 3½	(12) 0.148 x 3	1,795	3,870	4,365	4,695	3,330	3,760	4,045	
	HGUS414	•	•		—	3⅝	12⅞6	4	—	(66) 0.162 x 3½	(22) 0.162 x 3½	5,360	13,860	14,350	14,350	11,115	12,420	12,420	
	HUCQ412-SDS	•	•		—	3⅜6	11	3	—	(14) ¼ x 2½ SDS	(6) ¼ x 2½ SDS	2,265	5,045	5,045	5,045	3,630	3,630	3,630	
	LGU3.63-SDS	•	•		—	3⅝	8 to 30	4½	—	(16) ¼ x 2½ SDS	(12) ¼ x 2½ SDS	5,555	6,720	6,720	6,720	4,840	4,840	4,840	
	MGU3.63-SDS	•	•		—	3⅝	9¼ to 30	4½	—	(24) ¼ x 2½ SDS	(16) ¼ x 2½ SDS	7,260	9,450	9,450	9,450	6,805	6,805	6,805	
	HGU3.63-SDS	•	•		—	3⅝	11 to 30	5¼	—	(36) ¼ x 2½ SDS	(24) ¼ x 2½ SDS	9,895	14,145	14,145	14,145	10,185	10,185	10,185	
3½ x 18	MIU3.56/18			•	—	3⅜6	17⅝6	2½	—	(26) 0.162 x 3½	(2) 0.148 x 1½	210	3,745	4,045	4,045	3,220	3,480	3,480	
	HU416 / HUC416	•	•	•	✓	3⅜6	13⅝	2½	Min.	(20) 0.162 x 3½	(8) 0.148 x 3	1,515	2,975	3,360	3,610	2,565	2,895	3,110	
	HGUS414	•	•		—	3⅝	12⅞6	4	—	(66) 0.162 x 3½	(22) 0.162 x 3½	5,360	13,860	14,350	14,350	11,115	12,420	12,420	
SS	HUCQ412-SDS	•	•		—	3⅜6	11	3	—	(14) ¼ x 2½ SDS	(6) ¼ x 2½ SDS	2,265	5,045	5,045	5,045	3,630	3,630	3,630	
	LGU3.63-SDS	•	•		—	3⅝	8 to 30	4½	—	(16) ¼ x 2½ SDS	(12) ¼ x 2½ SDS	5,555	6,720	6,720	6,720	4,840	4,840	4,840	
	MGU3.63-SDS	•	•		—	3⅝	9¼ to 30	4½	—	(24) ¼ x 2½ SDS	(16) ¼ x 2½ SDS	7,260	9,450	9,450	9,450	6,805	6,805	6,805	
	HGU3.63-SDS	•	•		—	3⅝	11 to 30	5¼	—	(36) ¼ x 2½ SDS	(24) ¼ x 2½ SDS	9,460	13,160	13,160	13,160	9,475	9,475	9,475	
3½ x 20	MIU3.56/20			•	—	3⅜6	19⅝6	2½	—	(28) 0.162 x 3½	(2) 0.148 x 1½	210	4,030	4,060	4,060	3,465	3,495	3,495	
3½ x 21 to 30	MIU3.56/20			•	✓	3⅜6	19⅝6	2½	—	(28) 0.162 x 3½	(2) 0.148 x 1½	210	4,030	4,060	4,060	3,465	3,495	3,495	
	LGU3.63-SDS	•	•		—	3⅝	8 to 30	4½	—	(16) ¼ x 2½ SDS	(12) ¼ x 2½ SDS	5,555	6,720	6,720	6,720	4,840	4,840	4,840	
	MGU3.63-SDS	•	•		—	3⅝	9¼ to 30	4½	—	(24) ¼ x 2½ SDS	(16) ¼ x 2½ SDS	7,260	9,450	9,450	9,450	6,805	6,805	6,805	
	HGU3.63-SDS	•	•		—	3⅝	11 to 30	5¼	—	(36) ¼ x 2½ SDS	(24) ¼ x 2½ SDS	9,460	13,160	13,160	13,160	9,475	9,475	9,475	
4 x 9½	MIU4.12/9	•		•	—	4⅛	9⅛6	2½	—	(16) 0.162 x 3½	(2) 0.148 x 1½	210	2,305	2,615	2,820	1,980	2,245	2,425	
	HU4.12/9 / HUC4.12/9			•	✓	4⅛	8⅝	2½	Max.	(18) 0.162 x 3½	(10) 0.148 x 3	1,795	2,680	3,020	3,250	2,305	2,605	2,800	
4 x 11⅞	MIU4.12/11			•	—	4⅛	11⅛	2½	—	(20) 0.162 x 3½	(2) 0.148 x 1½	210	2,880	3,135	3,135	2,475	2,695	2,695	
	HU4.12/11 / HUC4.12/11			•	✓	4⅛	10⅝6	2½	Max.	(22) 0.162 x 3½	(10) 0.148 x 3	1,795	3,275	3,695	3,970	2,820	3,180	3,425	
4 x 14	MIU4.12/14			•	—	4⅛	13⅜6	2½	—	(22) 0.162 x 3½	(2) 0.148 x 1½	210	3,170	3,595	3,875	2,725	3,090	3,335	
	HU4.12/11 / HUC4.12/11			•	✓	4⅛	10⅝6	2½	Max.	(22) 0.162 x 3½	(10) 0.148 x 3	1,795	3,275	3,695	3,970	2,820	3,180	3,425	
4 x 16	MIU4.12/16			•	—	4⅛	15⅜6	2½	—	(24) 0.162 x 3½	(2) 0.148 x 1½	210	3,455	3,920	4,045	2,970	3,370	3,480	
	HU4.12/11 / HUC4.12/11			•	✓	4⅛	10⅝6	2½	Max.	(22) 0.162 x 3½	(10) 0.148 x 3	1,795	3,275	3,695	3,970	2,820	3,180	3,425	
4⅛ x 9½	MIU4.28/9			•	—	4⅝32	9	2½	—	(16) 0.162 x 3½	(2) 0.148 x 1½	210	2,305	2,615	2,820	1,980	2,245	2,425	
	HU4.28/9 / HUC4.28/9			•	✓	4⅝32	9	2½	—	(18) 0.162 x 3½	(8) 0.148 x 3	1,515	2,680	3,020	3,250	2,305	2,605	2,800	
4⅛ x 11⅞	MIU4.28/11			•	—	4⅝32	11⅛	2½	—	(20) 0.162 x 3½	(2) 0.148 x 1½	210	2,880	3,135	3,135	2,475	2,695	2,695	
	HU4.28/11 / HUC4.28/11			•	✓	4⅝32	11	2½	—	(22) 0.162 x 3½	(8) 0.148 x 3	1,515	3,275	3,695	3,970	2,820	3,180	3,425	
4⅛ x 14	MIU4.28/14			•	—	4⅝32	13½	2½	—	(22) 0.162 x 3½	(2) 0.148 x 1½	210	3,170	3,595	3,875	2,725	3,090	3,335	
4⅛ x 16	MIU4.28/16			•	—	4⅝32	15½	2½	—	(24) 0.162 x 3½	(2) 0.148 x 1½	210	3,455	3,920	4,045	2,970	3,370	3,480	

See footnotes on p. 162.

F. Foundation Design

Foundation Bearing Design:

- Dimensions/Embedment Depths:

Number of Stories (Floors supported)	Mínimum Width (ft)	Mínimum Footing Thickness (ft)	Minimum Embedment Below Lowest Finished Surface (ft.)	
1	1.0	6	Peri meter	2.0
			Interior	1.5
2	1.25	7	1.5	
Square Column Footings to 50 Kip	-	-	2.0	

- Allowable Bearing Capacity:

Embedment Depth (ft.)	Allowable Bearing Capacity (lb/ft^2)
1,5	2,400
2,0	2,800
3,0	3,200

- All continuous footings should be reinforced with at least four (2) # 4 reinforcing bars, one placed at the near top and one at the bottom, consistent with the recommendations from the structural engineer or design architect and in compliance with guidelines as found in the California Building Code.

- Following site preparation, the use of shallow spread and/or continuous footings is feasible. An allowable bearing value of 2400 psf is recommended. This bearing pressure has been established based on the assumption that the footings will be embedded at least 18-inches (1.5 ft) below lowest adjacent firm grade and into compacted soil mat, and measure at least 15 inches (1.25ft) in width. Isolated column footings should be at least 24 inches (2 ft) wide and embedded at least 18 inches (1.5 ft) below lowest adjacent firm grade.

The net allowable bearing values indicated above are for the dead and frequently applied live loads and are obtained by applying a factor of safety of 3.0 to the net ultimate bearing capacity. If normal code requirements are applied for design, the above vertical bearing value may be increased by 33 percent.

For short-duration loadings, such as those induced by wind or seismic forces. The settlement of structures supported in strip and/or spread footings founded on compacted fill will depend on the actual footing dimensions, type, and density of soil and on the foundation load influence zone (Normally up to twice the width of footing).

F.1 Spread Footing Design

Project Information

CBC 2022

Project Title : C & S Restaurant

Description : Commercial Bldg.

I.D. : CBE#: 25-304

Address : 55795 Twenty-Nine Palms Hwy, Yucca Valley, CA 92284

Project Leader : Mohammad Aljazzar P.E.

Phone : 714-745-0564 Fax : eMail : mohammad@cbestructural.com

Project Notes

Project Title: C & S Restaurant
Engineer: Mohammad Aljazzar P.E.
Project ID: CBE#: 25-304
Project Descr: Commercial Bldg.

General Footing

Project File: C and S Restaurant_29 Palms Hwy.ec6

DESCRIPTION: Footing at Lines [C/2]

CBC 2022

Code References

Calculations per ACI 318-19, IBC 2021
Load Combinations Used : IBC 2021

General Information

Material Properties

f'c : Concrete 28 day strength	=	3.0 ksi
fy : Rebar Yield	=	60.0 ksi
Ec : Concrete Elastic Modulus	=	3,122.02 ksi
Concrete Density	=	145.0 pcf
φ Values Flexure	=	0.90
Shear	=	0.750

Analysis Settings

Min Steel % Bending Reinf.	=	
Min Allow % Temp Reinf.	=	0.00180
Min. Overturning Safety Factor	=	1.0 : 1
Min. Sliding Safety Factor	=	1.0 : 1
Add Ftg Wt for Soil Pressure	:	Yes
Use ftg wt for stability, moments & shears	:	Yes
Add Pedestal Wt for Soil Pressure	:	No
Use Pedestal wt for stability, mom & shear	:	No

Soil Design Values

Allowable Soil Bearing	=	2.40 ksf
Soil Density	=	134.0 pcf
Increase Bearing By Footing Weight	=	No
Soil Passive Resistance (for Sliding)	=	300.0 pcf
Soil/Concrete Friction Coeff.	=	0.30

Increases based on footing Depth

Footing base depth below soil surface	=	1.50 ft
Allow press. increase per foot of depth	=	0.40 ksf
when footing base is below	=	2.0 ft

Increases based on footing plan dimension

Allowable pressure increase per foot of depth	=	ksf
when max. length or width is greater than	=	ft

Dimensions

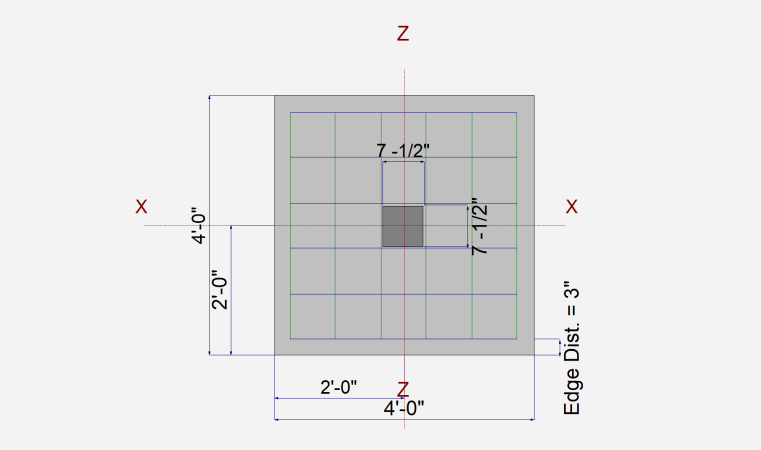
Width parallel to X-X Axis	=	4.0 ft
Length parallel to Z-Z Axis	=	4.0 ft
Footing Thickness	=	18 in

Pedestal dimensions...

px : parallel to X-X Axis	=	7.50 in
pz : parallel to Z-Z Axis	=	7.50 in
Height	=	0.250 in

Bandwidth Distribution Check (ACI 15.4.4.2)

Direction Requiring Closer Separation	n/a
# Bars required within zone	n/a
# Bars required on each side of zone	n/a



Bottom Reinforcing

Bars parallel to X-X Axis (resisting Z Flexure)

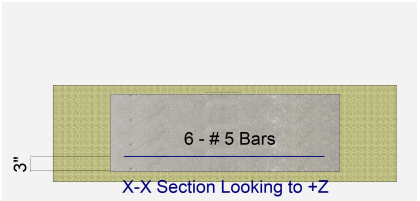
Number of Bars	=	6
Reinforcing Bar Size	=	# 5

Bars parallel to Z-Z Axis (resisting X Flexure)

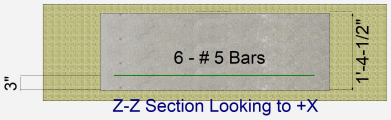
Number of Bars	=	6.0
Reinforcing Bar Size	=	# 5

Rebar Centerline to Edge of Concrete...

at Bottom of footing	=	3.0 in



X-X Section Looking to +Z



Z-Z Section Looking to +X

Project Title: C & S Restaurant
Engineer: Mohammad Aljazzar P.E.
Project ID: CBE#: 25-304
Project Descr: Commercial Bldg.

General Footing

Project File: C and S Restaurant_29 Palms Hwy.ec6

DESCRIPTION: Footing at Lines [C/2]

CBC 2022

Applied Loads

		D	Lr	L	S	W	E	H	
P : Column Load	=	9.440	8.786		13.165	5.594			k
OB : Overburden	=								ksf
M-xx	=								k-ft
M-zz	=								k-ft
V-x	=								k
V-z	=								k

DESIGN SUMMARY

Design OK

	Min. Ratio	Item	Applied	Capacity	Governing Load Combination
PASS	0.6788	Soil Bearing	1.629 ksf	2.40 ksf	+D+S about Z-Z axis
PASS	n/a	Overturning - X-X	0.0 k-ft	0.0 k-ft	No Overturning
PASS	n/a	Overturning - Z-Z	0.0 k-ft	0.0 k-ft	No Overturning
PASS	n/a	Sliding - X-X	0.0 k	0.0 k	No Sliding
PASS	n/a	Sliding - Z-Z	0.0 k	0.0 k	No Sliding
PASS	n/a	Uplift	0.0 k	0.0 k	No Uplift
PASS	0.1147	Z Flexure (+X) Bot Tens	3.131 k-ft/ft	27.295 k-ft/ft	+1.20D+1.60S+0.50W
PASS	0.1147	Z Flexure (-X) Bot Tens	3.131 k-ft/ft	27.295 k-ft/ft	+1.20D+1.60S+0.50W
PASS	0.766	Min Steel X Flexure Bottom	0.356 in2/ft	0.465 in2/ft	n/a
PASS	0.1147	X Flexure (+Z) Bot Tens	3.131 k-ft/ft	27.295 k-ft/ft	+1.20D+1.60S+0.50W
PASS	0.1147	X Flexure (-Z) Bot Tens	3.131 k-ft/ft	27.295 k-ft/ft	+1.20D+1.60S+0.50W
PASS	0.766	Min Steel Z Flexure Bottom	0.356 in2/ft	0.465 in2/ft	n/a
PASS	0.0	Z Flexure (+X) Top Tens	0 k-ft/ft	0.0 k-ft/ft	
PASS	0.0	Z Flexure (-X) Top Tens	0 k-ft/ft	0.0 k-ft/ft	
PASS	0.000	Min Steel X Flexure Top	0.000 in2/ft	0.000 in2/ft	n/a
PASS	0.0	X Flexure (+Z) Top Tens	0 k-ft/ft	0.0 k-ft/ft	
PASS	0.0	X Flexure (-Z) Top Tens	0 k-ft/ft	0.0 k-ft/ft	
PASS	0.000	Min Steel Z Flexure Top	0.000 in2/ft	0.000 in2/ft	n/a
PASS	0.1627	1-way Shear (+X)	7.601 psi	46.704 psi	+1.20D+1.60S+0.50W
PASS	0.1627	1-way Shear (-X)	7.601 psi	46.704 psi	+1.20D+1.60S+0.50W
PASS	0.1627	1-way Shear (+Z)	7.601 psi	46.704 psi	+1.20D+1.60S+0.50W
PASS	0.1627	1-way Shear (-Z)	7.601 psi	46.704 psi	+1.20D+1.60S+0.50W
PASS	0.1523	2-way Punching	25.018 psi	164.317 psi	+1.20D+1.60S+0.50W

Detailed Results

Soil Bearing

Rotation Axis & Load Combination...	Gross Allowable	Xecc (in)	Zecc (in)	Actual Soil Bearing Stress @ Location Bottom, -Z	Top, +Z	Left, -X	Right, +X	Actual / Allow Ratio
X-X, D Only	2.40	n/a	0.0	0.8057	0.8057	n/a	n/a	0.336
X-X, +D+Lr	2.40	n/a	0.0	1.355	1.355	n/a	n/a	0.565
X-X, +D+S	2.40	n/a	0.0	1.629	1.629	n/a	n/a	0.679
X-X, +D+0.750Lr	2.40	n/a	0.0	1.218	1.218	n/a	n/a	0.508
X-X, +D+0.750S	2.40	n/a	0.0	1.423	1.423	n/a	n/a	0.593
X-X, +D+0.60W	2.40	n/a	0.0	1.015	1.015	n/a	n/a	0.423
X-X, +D+0.750Lr+0.450W	2.40	n/a	0.0	1.375	1.375	n/a	n/a	0.573
X-X, +D+0.750S+0.450W	2.40	n/a	0.0	1.580	1.580	n/a	n/a	0.658
X-X, +0.60D+0.60W	2.40	n/a	0.0	0.6932	0.6932	n/a	n/a	0.289
X-X, +0.60D	2.40	n/a	0.0	0.4834	0.4834	n/a	n/a	0.201
Z-Z, D Only	2.40	0.0	n/a	n/a	n/a	0.8057	0.8057	0.336
Z-Z, +D+Lr	2.40	0.0	n/a	n/a	n/a	1.355	1.355	0.565
Z-Z, +D+S	2.40	0.0	n/a	n/a	n/a	1.629	1.629	0.679
Z-Z, +D+0.750Lr	2.40	0.0	n/a	n/a	n/a	1.218	1.218	0.508
Z-Z, +D+0.750S	2.40	0.0	n/a	n/a	n/a	1.423	1.423	0.593
Z-Z, +D+0.60W	2.40	0.0	n/a	n/a	n/a	1.015	1.015	0.423
Z-Z, +D+0.750Lr+0.450W	2.40	0.0	n/a	n/a	n/a	1.375	1.375	0.573
Z-Z, +D+0.750S+0.450W	2.40	0.0	n/a	n/a	n/a	1.580	1.580	0.658
Z-Z, +0.60D+0.60W	2.40	0.0	n/a	n/a	n/a	0.6932	0.6932	0.289
Z-Z, +0.60D	2.40	0.0	n/a	n/a	n/a	0.4834	0.4834	0.201

General Footing

Project File: C and S Restaurant_29 Palms Hwy.ec6

DESCRIPTION: Footing at Lines [C/2]

CBC 2022

Overturning Stability

Rotation Axis & Load Combination...	Overturning Moment	Resisting Moment	Stability Ratio	Status
Footing Has NO Overturning				

All units k

Sliding Stability

Force Application Axis Load Combination...	Sliding Force	Resisting Force	Stability Ratio	Status
Footing Has NO Sliding				

Footing Tension on Bottom

Flexure Axis & Load Combination	Mu k-ft	Side	Tension Surface	As Req'd in^2	Gvrn. As in^2	Actual As in^2	Phi*Mn k-ft	Status
X-X, +1.40D	1.175	+Z	Bottom	0.3564	ACI 7.6.1.1	0.4650	27.295	OK
X-X, +1.40D	1.175	-Z	Bottom	0.3564	ACI 7.6.1.1	0.4650	27.295	OK
X-X, +1.20D+0.50Lr	1.398	+Z	Bottom	0.3564	ACI 7.6.1.1	0.4650	27.295	OK
X-X, +1.20D+0.50Lr	1.398	-Z	Bottom	0.3564	ACI 7.6.1.1	0.4650	27.295	OK
X-X, +1.20D+0.50S	1.593	+Z	Bottom	0.3564	ACI 7.6.1.1	0.4650	27.295	OK
X-X, +1.20D+0.50S	1.593	-Z	Bottom	0.3564	ACI 7.6.1.1	0.4650	27.295	OK
X-X, +1.20D+1.60Lr	2.258	+Z	Bottom	0.3564	ACI 7.6.1.1	0.4650	27.295	OK
X-X, +1.20D+1.60Lr	2.258	-Z	Bottom	0.3564	ACI 7.6.1.1	0.4650	27.295	OK
X-X, +1.20D+1.60Lr+0.50W	2.507	+Z	Bottom	0.3564	ACI 7.6.1.1	0.4650	27.295	OK
X-X, +1.20D+1.60Lr+0.50W	2.507	-Z	Bottom	0.3564	ACI 7.6.1.1	0.4650	27.295	OK
X-X, +1.20D+1.60S	2.882	+Z	Bottom	0.3564	ACI 7.6.1.1	0.4650	27.295	OK
X-X, +1.20D+1.60S	2.882	-Z	Bottom	0.3564	ACI 7.6.1.1	0.4650	27.295	OK
X-X, +1.20D+1.60S+0.50W	3.131	+Z	Bottom	0.3564	ACI 7.6.1.1	0.4650	27.295	OK
X-X, +1.20D+1.60S+0.50W	3.131	-Z	Bottom	0.3564	ACI 7.6.1.1	0.4650	27.295	OK
X-X, +1.20D+0.50Lr+W	1.896	+Z	Bottom	0.3564	ACI 7.6.1.1	0.4650	27.295	OK
X-X, +1.20D+0.50Lr+W	1.896	-Z	Bottom	0.3564	ACI 7.6.1.1	0.4650	27.295	OK
X-X, +1.20D+0.50S+W	2.091	+Z	Bottom	0.3564	ACI 7.6.1.1	0.4650	27.295	OK
X-X, +1.20D+0.50S+W	2.091	-Z	Bottom	0.3564	ACI 7.6.1.1	0.4650	27.295	OK
X-X, +1.20D+0.70S	1.827	+Z	Bottom	0.3564	ACI 7.6.1.1	0.4650	27.295	OK
X-X, +1.20D+0.70S	1.827	-Z	Bottom	0.3564	ACI 7.6.1.1	0.4650	27.295	OK
X-X, +0.90D+W	1.253	+Z	Bottom	0.3564	ACI 7.6.1.1	0.4650	27.295	OK
X-X, +0.90D+W	1.253	-Z	Bottom	0.3564	ACI 7.6.1.1	0.4650	27.295	OK
X-X, +0.90D	0.7555	+Z	Bottom	0.3564	ACI 7.6.1.1	0.4650	27.295	OK
X-X, +0.90D	0.7555	-Z	Bottom	0.3564	ACI 7.6.1.1	0.4650	27.295	OK
Z-Z, +1.40D	1.175	-X	Bottom	0.3564	ACI 7.6.1.1	0.4650	27.295	OK
Z-Z, +1.40D	1.175	+X	Bottom	0.3564	ACI 7.6.1.1	0.4650	27.295	OK
Z-Z, +1.20D+0.50Lr	1.398	-X	Bottom	0.3564	ACI 7.6.1.1	0.4650	27.295	OK
Z-Z, +1.20D+0.50Lr	1.398	+X	Bottom	0.3564	ACI 7.6.1.1	0.4650	27.295	OK
Z-Z, +1.20D+0.50S	1.593	-X	Bottom	0.3564	ACI 7.6.1.1	0.4650	27.295	OK
Z-Z, +1.20D+0.50S	1.593	+X	Bottom	0.3564	ACI 7.6.1.1	0.4650	27.295	OK
Z-Z, +1.20D+1.60Lr	2.258	-X	Bottom	0.3564	ACI 7.6.1.1	0.4650	27.295	OK
Z-Z, +1.20D+1.60Lr	2.258	+X	Bottom	0.3564	ACI 7.6.1.1	0.4650	27.295	OK
Z-Z, +1.20D+1.60Lr+0.50W	2.507	-X	Bottom	0.3564	ACI 7.6.1.1	0.4650	27.295	OK
Z-Z, +1.20D+1.60Lr+0.50W	2.507	+X	Bottom	0.3564	ACI 7.6.1.1	0.4650	27.295	OK
Z-Z, +1.20D+1.60S	2.882	-X	Bottom	0.3564	ACI 7.6.1.1	0.4650	27.295	OK
Z-Z, +1.20D+1.60S	2.882	+X	Bottom	0.3564	ACI 7.6.1.1	0.4650	27.295	OK
Z-Z, +1.20D+1.60S+0.50W	3.131	-X	Bottom	0.3564	ACI 7.6.1.1	0.4650	27.295	OK
Z-Z, +1.20D+1.60S+0.50W	3.131	+X	Bottom	0.3564	ACI 7.6.1.1	0.4650	27.295	OK
Z-Z, +1.20D+0.50Lr+W	1.896	-X	Bottom	0.3564	ACI 7.6.1.1	0.4650	27.295	OK
Z-Z, +1.20D+0.50Lr+W	1.896	+X	Bottom	0.3564	ACI 7.6.1.1	0.4650	27.295	OK
Z-Z, +1.20D+0.50S+W	2.091	-X	Bottom	0.3564	ACI 7.6.1.1	0.4650	27.295	OK
Z-Z, +1.20D+0.50S+W	2.091	+X	Bottom	0.3564	ACI 7.6.1.1	0.4650	27.295	OK
Z-Z, +1.20D+0.70S	1.827	-X	Bottom	0.3564	ACI 7.6.1.1	0.4650	27.295	OK
Z-Z, +1.20D+0.70S	1.827	+X	Bottom	0.3564	ACI 7.6.1.1	0.4650	27.295	OK
Z-Z, +0.90D+W	1.253	-X	Bottom	0.3564	ACI 7.6.1.1	0.4650	27.295	OK
Z-Z, +0.90D+W	1.253	+X	Bottom	0.3564	ACI 7.6.1.1	0.4650	27.295	OK
Z-Z, +0.90D	0.7555	-X	Bottom	0.3564	ACI 7.6.1.1	0.4650	27.295	OK
Z-Z, +0.90D	0.7555	+X	Bottom	0.3564	ACI 7.6.1.1	0.4650	27.295	OK

One Way Shear X

Load Combination...	Vu @ -X	Vu @ +X	Vu:Max	Phi Vn	Vu / Phi*Vn	Status
+1.40D	2.85 psi	2.85 psi	2.85 psi	46.70 psi	0.06	OK

General Footing

DESCRIPTION: Footing at Lines [C/2]

CBC 2022

One Way Shear X

Load Combination...	Vu @ -X	Vu @ +X	Vu:Max	Phi Vn	Vu / Phi*Vn	Status
+1.20D+0.50Lr	3.40 psi	3.40 psi	3.40 psi	46.70 psi	0.07	OK
+1.20D+0.50S	3.87 psi	3.87 psi	3.87 psi	46.70 psi	0.08	OK
+1.20D+1.60Lr	5.48 psi	5.48 psi	5.48 psi	46.70 psi	0.12	OK
+1.20D+1.60Lr+0.50W	6.09 psi	6.09 psi	6.09 psi	46.70 psi	0.13	OK
+1.20D+1.60S	7.00 psi	7.00 psi	7.00 psi	46.70 psi	0.15	OK
+1.20D+1.60S+0.50W	7.60 psi	7.60 psi	7.60 psi	46.70 psi	0.16	OK
+1.20D+0.50Lr+W	4.60 psi	4.60 psi	4.60 psi	46.70 psi	0.10	OK
+1.20D+0.50S+W	5.08 psi	5.08 psi	5.08 psi	46.70 psi	0.11	OK
+1.20D+0.70S	4.44 psi	4.44 psi	4.44 psi	46.70 psi	0.10	OK
+0.90D+W	3.04 psi	3.04 psi	3.04 psi	46.70 psi	0.07	OK
+0.90D	1.83 psi	1.83 psi	1.83 psi	46.70 psi	0.04	OK

One Way Shear Z

Load Combination...	Vu @ -Z	Vu @ +Z	Vu:Max	Phi Vn	Vu / Phi*Vn	Status
+1.40D	2.85 psi	2.85 psi	2.85 psi	46.70 psi	0.06	OK
+1.20D+0.50Lr	3.40 psi	3.40 psi	3.40 psi	46.70 psi	0.07	OK
+1.20D+0.50S	3.87 psi	3.87 psi	3.87 psi	46.70 psi	0.08	OK
+1.20D+1.60Lr	5.48 psi	5.48 psi	5.48 psi	46.70 psi	0.12	OK
+1.20D+1.60Lr+0.50W	6.09 psi	6.09 psi	6.09 psi	46.70 psi	0.13	OK
+1.20D+1.60S	7.00 psi	7.00 psi	7.00 psi	46.70 psi	0.15	OK
+1.20D+1.60S+0.50W	7.60 psi	7.60 psi	7.60 psi	46.70 psi	0.16	OK
+1.20D+0.50Lr+W	4.60 psi	4.60 psi	4.60 psi	46.70 psi	0.10	OK
+1.20D+0.50S+W	5.08 psi	5.08 psi	5.08 psi	46.70 psi	0.11	OK
+1.20D+0.70S	4.44 psi	4.44 psi	4.44 psi	46.70 psi	0.10	OK
+0.90D+W	3.04 psi	3.04 psi	3.04 psi	46.70 psi	0.07	OK
+0.90D	1.83 psi	1.83 psi	1.83 psi	46.70 psi	0.04	OK

Two-Way "Punching" Shear

All units k

Load Combination...	Vu	Phi*Vn	Vu / Phi*Vn	Status
+1.40D	9.39 psi	164.32 psi	0.06	OK
+1.20D+0.50Lr	11.17 psi	164.32 psi	0.07	OK
+1.20D+0.50S	12.73 psi	164.32 psi	0.08	OK
+1.20D+1.60Lr	18.05 psi	164.32 psi	0.11	OK
+1.20D+1.60Lr+0.50W	20.04 psi	164.32 psi	0.12	OK
+1.20D+1.60S	23.03 psi	164.32 psi	0.14	OK
+1.20D+1.60S+0.50W	25.02 psi	164.32 psi	0.15	OK
+1.20D+0.50Lr+W	15.15 psi	164.32 psi	0.09	OK
+1.20D+0.50S+W	16.71 psi	164.32 psi	0.10	OK
+1.20D+0.70S	14.60 psi	164.32 psi	0.09	OK
+0.90D+W	10.02 psi	164.32 psi	0.06	OK
+0.90D	6.04 psi	164.32 psi	0.04	OK

Project Title: C & S Restaurant
Engineer: Mohammad Aljazzar P.E.
Project ID: CBE#: 25-304
Project Descr: Commercial Bldg.

General Footing

Project File: C and S Restaurant_29 Palms Hwy.ec6

DESCRIPTION: Footing at Lines [F/2]

CBC 2022

Code References

Calculations per ACI 318-19, IBC 2021
Load Combinations Used : IBC 2021

General Information

Material Properties

f'c : Concrete 28 day strength	=	3.0	ksi
fy : Rebar Yield	=	60.0	ksi
Ec : Concrete Elastic Modulus	=	3,122.02	ksi
Concrete Density	=	145.0	pcf
φ Values Flexure	=	0.90	
Shear	=	0.750	

Analysis Settings

Min Steel % Bending Reinf.	=	
Min Allow % Temp Reinf.	=	0.00180
Min. Overturning Safety Factor	=	1.0 : 1
Min. Sliding Safety Factor	=	1.0 : 1
Add Ftg Wt for Soil Pressure	:	Yes
Use ftg wt for stability, moments & shears	:	Yes
Add Pedestal Wt for Soil Pressure	:	Yes
Use Pedestal wt for stability, mom & shear	:	Yes

Soil Design Values

Allowable Soil Bearing	=	2.40	ksf
Soil Density	=	134.0	pcf
Increase Bearing By Footing Weight	=	No	
Soil Passive Resistance (for Sliding)	=	300.0	pcf
Soil/Concrete Friction Coeff.	=	0.30	

Increases based on footing Depth

Footing base depth below soil surface	=	1.50	ft
Allow press. increase per foot of depth	=	0.40	ksf
when footing base is below	=	2.0	ft

Increases based on footing plan dimension

Allowable pressure increase per foot of depth	=		ksf
when max. length or width is greater than	=		ft

Dimensions

Width parallel to X-X Axis	=	3.50 ft
Length parallel to Z-Z Axis	=	3.0 ft
Footing Thickness	=	12.0 in

Load location offset from footing center...

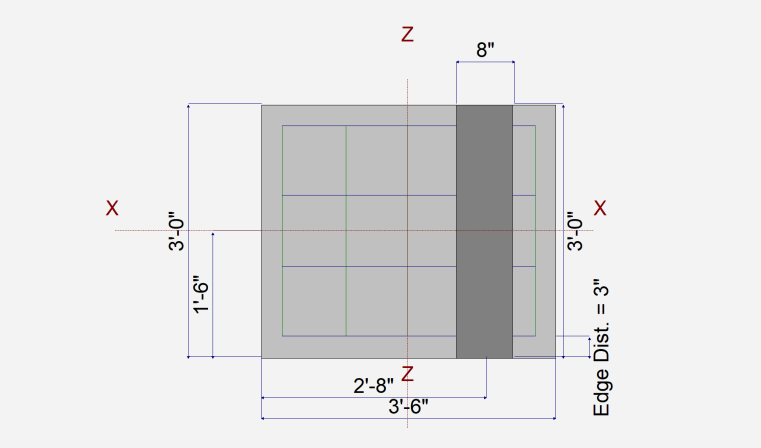
ex : Prll to X-X Axis	=	11 in
	=	in

Pedestal dimensions...

px : parallel to X-X Axis	=	8.0 in
pz : parallel to Z-Z Axis	=	36.0 in
Height	=	24.0 in

Bandwidth Distribution Check (ACI 15.4.4.2)

Direction Requiring Closer Separation Bars along Z-Z Axis
Bars required within zone 92.3 %
Bars required on each side of zone 7.7 %



Bottom Reinforcing

Bars parallel to X-X Axis (resisting Z Flexure)

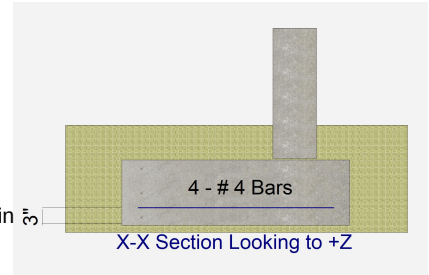
Number of Bars	=		4
Reinforcing Bar Size	=	#	4

Bars parallel to Z-Z Axis (resisting X Flexure)

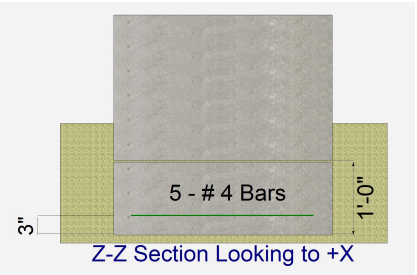
Number of Bars	=		5
Reinforcing Bar Size	=	#	4

Rebar Centerline to Edge of Concrete...

at Bottom of footing	=	3.0 in



X-X Section Looking to +Z



Z-Z Section Looking to +X

Project Title: C & S Restaurant
Engineer: Mohammad Aljazzar P.E.
Project ID: CBE#: 25-304
Project Descr: Commercial Bldg.

General Footing

Project File: C and S Restaurant_29 Palms Hwy.ec6

DESCRIPTION: Footing at Lines [F/2]

CBC 2022

Applied Loads

		D	Lr	L	S	W	E	H	
P : Column Load	=	2.371	2.262		2.613	1.320			k
OB : Overburden	=								ksf
M-xx	=								k-ft
M-zz	=								k-ft
V-x	=								k
V-z	=								k

DESIGN SUMMARY

Design OK

	Min. Ratio	Item	Applied	Capacity	Governing Load Combination
PASS	0.6413	Soil Bearing	1.539 ksf	2.40 ksf	+D+S about Z-Z axis
PASS	n/a	Overturning - X-X	0.0 k-ft	0.0 k-ft	No Overturning
PASS	n/a	Overturning - Z-Z	0.0 k-ft	0.0 k-ft	No Overturning
PASS	n/a	Sliding - X-X	0.0 k	0.0 k	No Sliding
PASS	n/a	Sliding - Z-Z	0.0 k	0.0 k	No Sliding
PASS	n/a	Uplift	0.0 k	0.0 k	No Uplift
PASS	0.02257	Z Flexure (+X) Bot Tens	0.2367 k-ft/ft	10.486 k-ft/ft	+1.20D+1.60S+0.50W
PASS	0.0290	Z Flexure (-X) Bot Tens	0.3040 k-ft/ft	10.486 k-ft/ft	+1.20D+1.60S+0.50W
PASS	0.972	Min Steel X Flexure Bottom	0.259 in2/ft	0.267 in2/ft	n/a
PASS	0.0	X Flexure (+Z) Bot Tens	0.0 k-ft/ft	11.211 k-ft/ft	+1.40D
PASS	0.0	X Flexure (-Z) Bot Tens	0.0 k-ft/ft	11.211 k-ft/ft	+1.40D
PASS	0.907	Min Steel Z Flexure Bottom	0.259 in2/ft	0.286 in2/ft	n/a
PASS	0.0	Z Flexure (+X) Top Tens	0 k-ft/ft	0.0 k-ft/ft	
PASS	0.0	Z Flexure (-X) Top Tens	0 k-ft/ft	0.0 k-ft/ft	
PASS	0.000	Min Steel X Flexure Top	0.000 in2/ft	0.000 in2/ft	n/a
PASS	0.0	X Flexure (+Z) Top Tens	0 k-ft/ft	0.0 k-ft/ft	
PASS	0.0	X Flexure (-Z) Top Tens	0 k-ft/ft	0.0 k-ft/ft	
PASS	0.000	Min Steel Z Flexure Top	0.000 in2/ft	0.000 in2/ft	n/a
PASS	n/a	1-way Shear (+X)	0.0 psi	44.418 psi	n/a
PASS	0.03703	1-way Shear (-X)	1.645 psi	44.418 psi	+1.20D+1.60S+0.50W
PASS	n/a	1-way Shear (+Z)	0.0 psi	45.451 psi	n/a
PASS	n/a	1-way Shear (-Z)	0.0 psi	45.451 psi	n/a
PASS	0.01059	2-way Punching	1.256 psi	118.673 psi	+1.40D

Detailed Results

Soil Bearing

Rotation Axis & Load Combination...	Gross Allowable	Xecc (in)	Zecc (in)	Actual Soil Bearing Stress @ Location Bottom, -Z	Top, +Z	Left, -X	Right, +X	Actual / Allow Ratio
X-X, D Only	2.40	n/a	0.0	0.4803	0.4803	n/a	n/a	0.200
X-X, +D+Lr	2.40	n/a	0.0	0.6957	0.6957	n/a	n/a	0.290
X-X, +D+S	2.40	n/a	0.0	0.7291	0.7291	n/a	n/a	0.304
X-X, +D+0.750Lr	2.40	n/a	0.0	0.6419	0.6419	n/a	n/a	0.268
X-X, +D+0.750S	2.40	n/a	0.0	0.6669	0.6669	n/a	n/a	0.278
X-X, +D+0.60W	2.40	n/a	0.0	0.5557	0.5557	n/a	n/a	0.232
X-X, +D+0.750Lr+0.450W	2.40	n/a	0.0	0.6984	0.6984	n/a	n/a	0.291
X-X, +D+0.750S+0.450W	2.40	n/a	0.0	0.7235	0.7235	n/a	n/a	0.302
X-X, +0.60D+0.60W	2.40	n/a	0.0	0.3636	0.3636	n/a	n/a	0.152
X-X, +0.60D	2.40	n/a	0.0	0.2882	0.2882	n/a	n/a	0.120
Z-Z, D Only	2.40	6.145	n/a	n/a	n/a	0.06291	0.8977	0.374
Z-Z, +D+Lr	2.40	7.648	n/a	n/a	n/a	0.0	1.451	0.605
Z-Z, +D+S	2.40	7.802	n/a	n/a	n/a	0.0	1.539	0.641
Z-Z, +D+0.750Lr	2.40	7.367	n/a	n/a	n/a	0.0	1.311	0.546
Z-Z, +D+0.750S	2.40	7.503	n/a	n/a	n/a	0.0	1.376	0.573
Z-Z, +D+0.60W	2.40	6.804	n/a	n/a	n/a	0.02099	1.090	0.454
Z-Z, +D+0.750Lr+0.450W	2.40	7.661	n/a	n/a	n/a	0.0	1.458	0.608
Z-Z, +D+0.750S+0.450W	2.40	7.777	n/a	n/a	n/a	0.0	1.524	0.635
Z-Z, +0.60D+0.60W	2.40	7.820	n/a	n/a	n/a	0.0	0.7684	0.320
Z-Z, +0.60D	2.40	6.988	n/a	n/a	n/a	0.003372	0.5730	0.239

General Footing

Project File: C and S Restaurant_29 Palms Hwy.ec6

DESCRIPTION: Footing at Lines [F/2]

CBC 2022

Overturning Stability

Rotation Axis & Load Combination...	Overturning Moment	Resisting Moment	Stability Ratio	Status
Footing Has NO Overturning				

All units k

Sliding Stability

Force Application Axis Load Combination...	Sliding Force	Resisting Force	Stability Ratio	Status
Footing Has NO Sliding				

Footing Tension on Bottom

Flexure Axis & Load Combination	Mu k-ft	Side	Tension Surface	As Req'd in^2	Gvrn. As in^2	Actual As in^2	Phi*Mn k-ft	Status
X-X, +1.40D	0.0	+Z	Bottom	0.2592	ACI 7.6.1.1	0.2857	11.211	OK
X-X, +1.40D	0.0	-Z	Bottom	0.2592	ACI 7.6.1.1	0.2857	11.211	OK
X-X, +1.20D+0.50Lr	0.0	+Z	Bottom	0.2592	ACI 7.6.1.1	0.2857	11.211	OK
X-X, +1.20D+0.50Lr	0.0	-Z	Bottom	0.2592	ACI 7.6.1.1	0.2857	11.211	OK
X-X, +1.20D+0.50S	0.0	+Z	Bottom	0.2592	ACI 7.6.1.1	0.2857	11.211	OK
X-X, +1.20D+0.50S	0.0	-Z	Bottom	0.2592	ACI 7.6.1.1	0.2857	11.211	OK
X-X, +1.20D+1.60Lr	0.0	+Z	Bottom	0.2592	ACI 7.6.1.1	0.2857	11.211	OK
X-X, +1.20D+1.60Lr	0.0	-Z	Bottom	0.2592	ACI 7.6.1.1	0.2857	11.211	OK
X-X, +1.20D+1.60Lr+0.50W	0.0	+Z	Bottom	0.2592	ACI 7.6.1.1	0.2857	11.211	OK
X-X, +1.20D+1.60Lr+0.50W	0.0	-Z	Bottom	0.2592	ACI 7.6.1.1	0.2857	11.211	OK
X-X, +1.20D+1.60S	0.0	+Z	Bottom	0.2592	ACI 7.6.1.1	0.2857	11.211	OK
X-X, +1.20D+1.60S	0.0	-Z	Bottom	0.2592	ACI 7.6.1.1	0.2857	11.211	OK
X-X, +1.20D+1.60S+0.50W	0.0	+Z	Bottom	0.2592	ACI 7.6.1.1	0.2857	11.211	OK
X-X, +1.20D+1.60S+0.50W	0.0	-Z	Bottom	0.2592	ACI 7.6.1.1	0.2857	11.211	OK
X-X, +1.20D+0.50Lr+W	0.0	+Z	Bottom	0.2592	ACI 7.6.1.1	0.2857	11.211	OK
X-X, +1.20D+0.50Lr+W	0.0	-Z	Bottom	0.2592	ACI 7.6.1.1	0.2857	11.211	OK
X-X, +1.20D+0.50S+W	0.0	+Z	Bottom	0.2592	ACI 7.6.1.1	0.2857	11.211	OK
X-X, +1.20D+0.50S+W	0.0	-Z	Bottom	0.2592	ACI 7.6.1.1	0.2857	11.211	OK
X-X, +1.20D+0.70S	0.0	+Z	Bottom	0.2592	ACI 7.6.1.1	0.2857	11.211	OK
X-X, +1.20D+0.70S	0.0	-Z	Bottom	0.2592	ACI 7.6.1.1	0.2857	11.211	OK
X-X, +0.90D+W	0.0	+Z	Bottom	0.2592	ACI 7.6.1.1	0.2857	11.211	OK
X-X, +0.90D+W	0.0	-Z	Bottom	0.2592	ACI 7.6.1.1	0.2857	11.211	OK
X-X, +0.90D	0.0	+Z	Bottom	0.2592	ACI 7.6.1.1	0.2857	11.211	OK
X-X, +0.90D	0.0	-Z	Bottom	0.2592	ACI 7.6.1.1	0.2857	11.211	OK
Z-Z, +1.40D	0.1824	-X	Bottom	0.2592	ACI 7.6.1.1	0.2667	10.486	OK
Z-Z, +1.40D	0.1097	+X	Bottom	0.2592	ACI 7.6.1.1	0.2667	10.486	OK
Z-Z, +1.20D+0.50Lr	0.1748	-X	Bottom	0.2592	ACI 7.6.1.1	0.2667	10.486	OK
Z-Z, +1.20D+0.50Lr	0.1280	+X	Bottom	0.2592	ACI 7.6.1.1	0.2667	10.486	OK
Z-Z, +1.20D+0.50S	0.1806	-X	Bottom	0.2592	ACI 7.6.1.1	0.2667	10.486	OK
Z-Z, +1.20D+0.50S	0.1331	+X	Bottom	0.2592	ACI 7.6.1.1	0.2667	10.486	OK
Z-Z, +1.20D+1.60Lr	0.2598	-X	Bottom	0.2592	ACI 7.6.1.1	0.2667	10.486	OK
Z-Z, +1.20D+1.60Lr	0.2005	+X	Bottom	0.2592	ACI 7.6.1.1	0.2667	10.486	OK
Z-Z, +1.20D+1.60Lr+0.50W	0.2835	-X	Bottom	0.2592	ACI 7.6.1.1	0.2667	10.486	OK
Z-Z, +1.20D+1.60Lr+0.50W	0.220	+X	Bottom	0.2592	ACI 7.6.1.1	0.2667	10.486	OK
Z-Z, +1.20D+1.60S	0.2801	-X	Bottom	0.2592	ACI 7.6.1.1	0.2667	10.486	OK
Z-Z, +1.20D+1.60S	0.2171	+X	Bottom	0.2592	ACI 7.6.1.1	0.2667	10.486	OK
Z-Z, +1.20D+1.60S+0.50W	0.3040	-X	Bottom	0.2592	ACI 7.6.1.1	0.2667	10.486	OK
Z-Z, +1.20D+1.60S+0.50W	0.2367	+X	Bottom	0.2592	ACI 7.6.1.1	0.2667	10.486	OK
Z-Z, +1.20D+0.50Lr+W	0.2189	-X	Bottom	0.2592	ACI 7.6.1.1	0.2667	10.486	OK
Z-Z, +1.20D+0.50Lr+W	0.1662	+X	Bottom	0.2592	ACI 7.6.1.1	0.2667	10.486	OK
Z-Z, +1.20D+0.50S+W	0.2249	-X	Bottom	0.2592	ACI 7.6.1.1	0.2667	10.486	OK
Z-Z, +1.20D+0.50S+W	0.1713	+X	Bottom	0.2592	ACI 7.6.1.1	0.2667	10.486	OK
Z-Z, +1.20D+0.70S	0.1978	-X	Bottom	0.2592	ACI 7.6.1.1	0.2667	10.486	OK
Z-Z, +1.20D+0.70S	0.1482	+X	Bottom	0.2592	ACI 7.6.1.1	0.2667	10.486	OK
Z-Z, +0.90D+W	0.1140	-X	Bottom	0.2592	ACI 7.6.1.1	0.2667	10.486	OK
Z-Z, +0.90D+W	0.1122	+X	Bottom	0.2592	ACI 7.6.1.1	0.2667	10.486	OK
Z-Z, +0.90D	0.07037	-X	Bottom	0.2592	ACI 7.6.1.1	0.2667	10.486	OK
Z-Z, +0.90D	0.07404	+X	Bottom	0.2592	ACI 7.6.1.1	0.2667	10.486	OK

One Way Shear X

Load Combination...	Vu @ -X	Vu @ +X	Vu:Max	Phi Vn	Vu / Phi*Vn	Status
+1.40D	1.02 psi	0.00 psi	1.02 psi	44.42 psi	0.02	OK

General Footing

DESCRIPTION: Footing at Lines [F/2]

CBC 2022

One Way Shear X

Load Combination...	Vu @ -X	Vu @ +X	Vu:Max	Phi Vn	Vu / Phi*Vn	Status
+1.20D+0.50Lr	0.99 psi	0.00 psi	0.99 psi	44.42 psi	0.02	OK
+1.20D+0.50S	1.02 psi	0.00 psi	1.02 psi	44.42 psi	0.02	OK
+1.20D+1.60Lr	1.44 psi	0.00 psi	1.44 psi	44.42 psi	0.03	OK
+1.20D+1.60Lr+0.50W	1.55 psi	0.00 psi	1.55 psi	44.42 psi	0.03	OK
+1.20D+1.60S	1.54 psi	0.00 psi	1.54 psi	44.42 psi	0.03	OK
+1.20D+1.60S+0.50W	1.65 psi	0.00 psi	1.65 psi	44.42 psi	0.04	OK
+1.20D+0.50Lr+W	1.23 psi	0.00 psi	1.23 psi	44.42 psi	0.03	OK
+1.20D+0.50S+W	1.27 psi	0.00 psi	1.27 psi	44.42 psi	0.03	OK
+1.20D+0.70S	1.12 psi	0.00 psi	1.12 psi	44.42 psi	0.03	OK
+0.90D+W	0.65 psi	0.00 psi	0.65 psi	44.42 psi	0.01	OK
+0.90D	0.41 psi	0.00 psi	0.41 psi	44.42 psi	0.01	OK

One Way Shear Z

Load Combination...	Vu @ -Z	Vu @ +Z	Vu:Max	Phi Vn	Vu / Phi*Vn	Status
+1.40D	0.00 psi	0.00 psi	0.00 psi	45.45 psi	0.00	OK
+1.20D+0.50Lr	0.00 psi	0.00 psi	0.00 psi	45.45 psi	0.00	OK
+1.20D+0.50S	0.00 psi	0.00 psi	0.00 psi	45.45 psi	0.00	OK
+1.20D+1.60Lr	0.00 psi	0.00 psi	0.00 psi	45.45 psi	0.00	OK
+1.20D+1.60Lr+0.50W	0.00 psi	0.00 psi	0.00 psi	45.45 psi	0.00	OK
+1.20D+1.60S	0.00 psi	0.00 psi	0.00 psi	45.45 psi	0.00	OK
+1.20D+1.60S+0.50W	0.00 psi	0.00 psi	0.00 psi	45.45 psi	0.00	OK
+1.20D+0.50Lr+W	0.00 psi	0.00 psi	0.00 psi	45.45 psi	0.00	OK
+1.20D+0.50S+W	0.00 psi	0.00 psi	0.00 psi	45.45 psi	0.00	OK
+1.20D+0.70S	0.00 psi	0.00 psi	0.00 psi	45.45 psi	0.00	OK
+0.90D+W	0.00 psi	0.00 psi	0.00 psi	45.45 psi	0.00	OK
+0.90D	0.00 psi	0.00 psi	0.00 psi	45.45 psi	0.00	OK

Two-Way "Punching" Shear

All units k

Load Combination...	Vu	Phi*Vn	Vu / Phi*Vn	Status
+1.40D	1.26 psi	118.67 psi	0.01	OK
+1.20D+0.50Lr	1.26 psi	118.67 psi	0.01	OK
+1.20D+0.50S	1.26 psi	118.67 psi	0.01	OK
+1.20D+1.60Lr	1.26 psi	118.67 psi	0.01	OK
+1.20D+1.60Lr+0.50W	1.26 psi	118.67 psi	0.01	OK
+1.20D+1.60S	1.26 psi	118.67 psi	0.01	OK
+1.20D+1.60S+0.50W	1.26 psi	118.67 psi	0.01	OK
+1.20D+0.50Lr+W	1.26 psi	118.67 psi	0.01	OK
+1.20D+0.50S+W	1.26 psi	118.67 psi	0.01	OK
+1.20D+0.70S	1.26 psi	118.67 psi	0.01	OK
+0.90D+W	1.26 psi	118.67 psi	0.01	OK
+0.90D	1.26 psi	118.67 psi	0.01	OK

Project Title: C & S Restaurant
Engineer: Mohammad Aljazzar P.E.
Project ID: CBE#: 25-304
Project Descr: Commercial Bldg.

General Footing

Project File: C and S Restaurant_29 Palms Hwy.ec6

DESCRIPTION: Footing at Posts (Door Jambs) Supporting Header Lines [B]

CBC 2022

Code References

Calculations per ACI 318-19, IBC 2021
Load Combinations Used : IBC 2021

General Information

Material Properties

f'c : Concrete 28 day strength	=	3.0 ksi
fy : Rebar Yield	=	60.0 ksi
Ec : Concrete Elastic Modulus	=	3,122.02 ksi
Concrete Density	=	145.0 pcf
φ Values Flexure	=	0.90
Shear	=	0.750

Analysis Settings

Min Steel % Bending Reinf.	=	
Min Allow % Temp Reinf.	=	0.00180
Min. Overturning Safety Factor	=	1.0 : 1
Min. Sliding Safety Factor	=	1.0 : 1
Add Ftg Wt for Soil Pressure	:	Yes
Use ftg wt for stability, moments & shears	:	Yes
Add Pedestal Wt for Soil Pressure	:	No
Use Pedestal wt for stability, mom & shear	:	No

Soil Design Values

Allowable Soil Bearing	=	2.40 ksf
Soil Density	=	134.0 pcf
Increase Bearing By Footing Weight	=	No
Soil Passive Resistance (for Sliding)	=	300.0 pcf
Soil/Concrete Friction Coeff.	=	0.30

Increases based on footing Depth

Footing base depth below soil surface	=	1.50 ft
Allow press. increase per foot of depth	=	0.40 ksf
when footing base is below	=	2.0 ft

Increases based on footing plan dimension

Allowable pressure increase per foot of depth	=	ksf
when max. length or width is greater than	=	ft

Dimensions

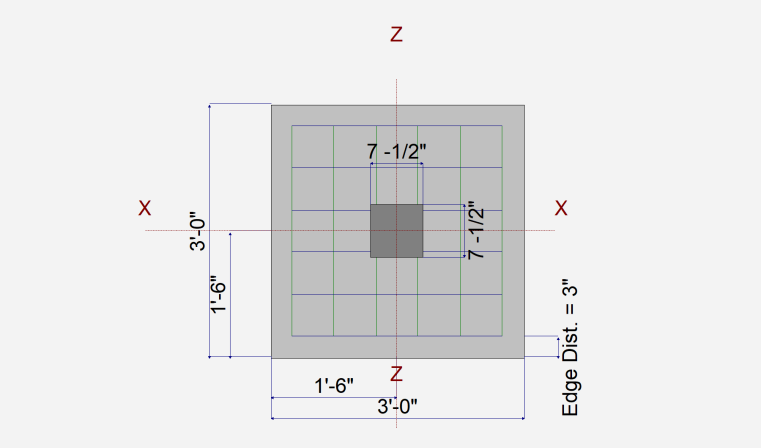
Width parallel to X-X Axis	=	3.0 ft
Length parallel to Z-Z Axis	=	3.0 ft
Footing Thickness	=	18.0 in

Pedestal dimensions...

px : parallel to X-X Axis	=	7.50 in
pz : parallel to Z-Z Axis	=	7.50 in
Height	=	0.250 in

Bandwidth Distribution Check (ACI 15.4.4.2)

Direction Requiring Closer Separation	n/a
# Bars required within zone	n/a
# Bars required on each side of zone	n/a



Bottom Reinforcing

Bars parallel to X-X Axis (resisting Z Flexure)

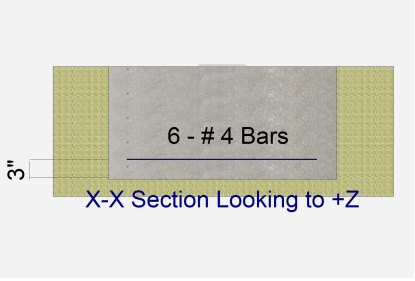
Number of Bars	=	6.0
Reinforcing Bar Size	=	# 4

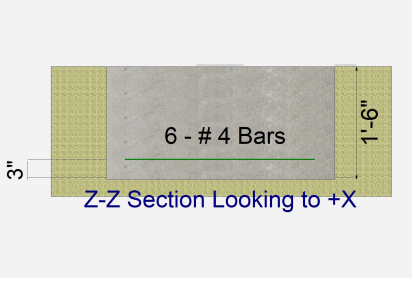
Bars parallel to Z-Z Axis (resisting X Flexure)

Number of Bars	=	6.0
Reinforcing Bar Size	=	# 4

Rebar Centerline to Edge of Concrete...

at Bottom of footing	=	3.0 in





Project Title: C & S Restaurant
Engineer: Mohammad Aljazzar P.E.
Project ID: CBE#: 25-304
Project Descr: Commercial Bldg.

General Footing

Project File: C and S Restaurant_29 Palms Hwy.ec6

DESCRIPTION: Footing at Posts (Door Jambs) Supporting Header Lines [B]

CBC 2022

Applied Loads

		D	Lr	L	S	W	E	H	
P : Column Load	=	4.613	4.658		6.606	2.673			k
OB : Overburden	=								ksf
M-xx	=								k-ft
M-zz	=								k-ft
V-x	=								k
V-z	=								k

DESIGN SUMMARY

Design OK

	Min. Ratio	Item	Applied	Capacity	Governing Load Combination
PASS	0.610	Soil Bearing	1.464 ksf	2.40 ksf	+D+S about Z-Z axis
PASS	n/a	Overturning - X-X	0.0 k-ft	0.0 k-ft	No Overturning
PASS	n/a	Overturning - Z-Z	0.0 k-ft	0.0 k-ft	No Overturning
PASS	n/a	Sliding - X-X	0.0 k	0.0 k	No Sliding
PASS	n/a	Sliding - Z-Z	0.0 k	0.0 k	No Sliding
PASS	n/a	Uplift	0.0 k	0.0 k	No Uplift
PASS	0.05196	Z Flexure (+X) Bot Tens	1.366 k-ft/ft	26.294 k-ft/ft	+1.20D+1.60S+0.50W
PASS	0.05196	Z Flexure (-X) Bot Tens	1.366 k-ft/ft	26.294 k-ft/ft	+1.20D+1.60S+0.50W
PASS	0.972	Min Steel X Flexure Bottom	0.389 in2/ft	0.400 in2/ft	n/a
PASS	0.05196	X Flexure (+Z) Bot Tens	1.366 k-ft/ft	26.294 k-ft/ft	+1.20D+1.60S+0.50W
PASS	0.05196	X Flexure (-Z) Bot Tens	1.366 k-ft/ft	26.294 k-ft/ft	+1.20D+1.60S+0.50W
PASS	0.972	Min Steel Z Flexure Bottom	0.389 in2/ft	0.400 in2/ft	n/a
PASS	0.0	Z Flexure (+X) Top Tens	0 k-ft/ft	0.0 k-ft/ft	
PASS	0.0	Z Flexure (-X) Top Tens	0 k-ft/ft	0.0 k-ft/ft	
PASS	0.000	Min Steel X Flexure Top	0.000 in2/ft	0.000 in2/ft	n/a
PASS	0.0	X Flexure (+Z) Top Tens	0 k-ft/ft	0.0 k-ft/ft	
PASS	0.0	X Flexure (-Z) Top Tens	0 k-ft/ft	0.0 k-ft/ft	
PASS	0.000	Min Steel Z Flexure Top	0.000 in2/ft	0.000 in2/ft	n/a
PASS	n/a	1-way Shear (+X)	0.0 psi	42.885 psi	n/a
PASS	0.0	1-way Shear (-X)	0.0 psi	0.0 psi	n/a
PASS	n/a	1-way Shear (+Z)	0.0 psi	42.885 psi	n/a
PASS	n/a	1-way Shear (-Z)	0.0 psi	42.885 psi	n/a
PASS	n/a	2-way Punching	7.953 psi	42.885 psi	+1.20D+1.60S+0.50W

Detailed Results

Soil Bearing

Rotation Axis & Load Combination...	Gross Allowable	Xecc (in)	Zecc (in)	Actual Soil Bearing Stress @ Location Bottom, -Z	Top, +Z	Left, -X	Right, +X	Actual / Allow Ratio
X-X, D Only	2.40	n/a	0.0	0.7301	0.7301	n/a	n/a	0.304
X-X, +D+Lr	2.40	n/a	0.0	1.248	1.248	n/a	n/a	0.520
X-X, +D+S	2.40	n/a	0.0	1.464	1.464	n/a	n/a	0.610
X-X, +D+0.750Lr	2.40	n/a	0.0	1.118	1.118	n/a	n/a	0.466
X-X, +D+0.750S	2.40	n/a	0.0	1.281	1.281	n/a	n/a	0.534
X-X, +D+0.60W	2.40	n/a	0.0	0.9083	0.9083	n/a	n/a	0.379
X-X, +D+0.750Lr+0.450W	2.40	n/a	0.0	1.252	1.252	n/a	n/a	0.522
X-X, +D+0.750S+0.450W	2.40	n/a	0.0	1.414	1.414	n/a	n/a	0.589
X-X, +0.60D+0.60W	2.40	n/a	0.0	0.6162	0.6162	n/a	n/a	0.257
X-X, +0.60D	2.40	n/a	0.0	0.4380	0.4380	n/a	n/a	0.183
Z-Z, D Only	2.40	0.0	n/a	n/a	n/a	0.7301	0.7301	0.304
Z-Z, +D+Lr	2.40	0.0	n/a	n/a	n/a	1.248	1.248	0.520
Z-Z, +D+S	2.40	0.0	n/a	n/a	n/a	1.464	1.464	0.610
Z-Z, +D+0.750Lr	2.40	0.0	n/a	n/a	n/a	1.118	1.118	0.466
Z-Z, +D+0.750S	2.40	0.0	n/a	n/a	n/a	1.281	1.281	0.534
Z-Z, +D+0.60W	2.40	0.0	n/a	n/a	n/a	0.9083	0.9083	0.379
Z-Z, +D+0.750Lr+0.450W	2.40	0.0	n/a	n/a	n/a	1.252	1.252	0.522
Z-Z, +D+0.750S+0.450W	2.40	0.0	n/a	n/a	n/a	1.414	1.414	0.589
Z-Z, +0.60D+0.60W	2.40	0.0	n/a	n/a	n/a	0.6162	0.6162	0.257
Z-Z, +0.60D	2.40	0.0	n/a	n/a	n/a	0.4380	0.4380	0.183

General Footing

DESCRIPTION: Footing at Posts (Door Jambs) Supporting Header Lines [B]

CBC 2022

Overturning Stability

Rotation Axis & Load Combination...	Overturning Moment	Resisting Moment	Stability Ratio	Status
Footing Has NO Overturning				

All units k

Sliding Stability

Force Application Axis Load Combination...	Sliding Force	Resisting Force	Stability Ratio	Status
Footing Has NO Sliding				

Footing Tension on Bottom

Flexure Axis & Load Combination	Mu k-ft	Side	Tension Surface	As Req'd in^2	Gvrn. As in^2	Actual As in^2	Phi*Mn k-ft	Status
X-X, +1.40D	0.5059	+Z	Bottom	0.3888	ACI 7.6.1.1	0.40	26.294	OK
X-X, +1.40D	0.5059	-Z	Bottom	0.3888	ACI 7.6.1.1	0.40	26.294	OK
X-X, +1.20D+0.50Lr	0.6161	+Z	Bottom	0.3888	ACI 7.6.1.1	0.40	26.294	OK
X-X, +1.20D+0.50Lr	0.6161	-Z	Bottom	0.3888	ACI 7.6.1.1	0.40	26.294	OK
X-X, +1.20D+0.50S	0.6924	+Z	Bottom	0.3888	ACI 7.6.1.1	0.40	26.294	OK
X-X, +1.20D+0.50S	0.6924	-Z	Bottom	0.3888	ACI 7.6.1.1	0.40	26.294	OK
X-X, +1.20D+1.60Lr	1.017	+Z	Bottom	0.3888	ACI 7.6.1.1	0.40	26.294	OK
X-X, +1.20D+1.60Lr	1.017	-Z	Bottom	0.3888	ACI 7.6.1.1	0.40	26.294	OK
X-X, +1.20D+1.60Lr+0.50W	1.122	+Z	Bottom	0.3888	ACI 7.6.1.1	0.40	26.294	OK
X-X, +1.20D+1.60Lr+0.50W	1.122	-Z	Bottom	0.3888	ACI 7.6.1.1	0.40	26.294	OK
X-X, +1.20D+1.60S	1.262	+Z	Bottom	0.3888	ACI 7.6.1.1	0.40	26.294	OK
X-X, +1.20D+1.60S	1.262	-Z	Bottom	0.3888	ACI 7.6.1.1	0.40	26.294	OK
X-X, +1.20D+1.60S+0.50W	1.366	+Z	Bottom	0.3888	ACI 7.6.1.1	0.40	26.294	OK
X-X, +1.20D+1.60S+0.50W	1.366	-Z	Bottom	0.3888	ACI 7.6.1.1	0.40	26.294	OK
X-X, +1.20D+0.50Lr+W	0.8254	+Z	Bottom	0.3888	ACI 7.6.1.1	0.40	26.294	OK
X-X, +1.20D+0.50Lr+W	0.8254	-Z	Bottom	0.3888	ACI 7.6.1.1	0.40	26.294	OK
X-X, +1.20D+0.50S+W	0.9017	+Z	Bottom	0.3888	ACI 7.6.1.1	0.40	26.294	OK
X-X, +1.20D+0.50S+W	0.9017	-Z	Bottom	0.3888	ACI 7.6.1.1	0.40	26.294	OK
X-X, +1.20D+0.70S	0.7959	+Z	Bottom	0.3888	ACI 7.6.1.1	0.40	26.294	OK
X-X, +1.20D+0.70S	0.7959	-Z	Bottom	0.3888	ACI 7.6.1.1	0.40	26.294	OK
X-X, +0.90D+W	0.5346	+Z	Bottom	0.3888	ACI 7.6.1.1	0.40	26.294	OK
X-X, +0.90D+W	0.5346	-Z	Bottom	0.3888	ACI 7.6.1.1	0.40	26.294	OK
X-X, +0.90D	0.3252	+Z	Bottom	0.3888	ACI 7.6.1.1	0.40	26.294	OK
X-X, +0.90D	0.3252	-Z	Bottom	0.3888	ACI 7.6.1.1	0.40	26.294	OK
Z-Z, +1.40D	0.5059	-X	Bottom	0.3888	ACI 7.6.1.1	0.40	26.294	OK
Z-Z, +1.40D	0.5059	+X	Bottom	0.3888	ACI 7.6.1.1	0.40	26.294	OK
Z-Z, +1.20D+0.50Lr	0.6161	-X	Bottom	0.3888	ACI 7.6.1.1	0.40	26.294	OK
Z-Z, +1.20D+0.50Lr	0.6161	+X	Bottom	0.3888	ACI 7.6.1.1	0.40	26.294	OK
Z-Z, +1.20D+0.50S	0.6924	-X	Bottom	0.3888	ACI 7.6.1.1	0.40	26.294	OK
Z-Z, +1.20D+0.50S	0.6924	+X	Bottom	0.3888	ACI 7.6.1.1	0.40	26.294	OK
Z-Z, +1.20D+1.60Lr	1.017	-X	Bottom	0.3888	ACI 7.6.1.1	0.40	26.294	OK
Z-Z, +1.20D+1.60Lr	1.017	+X	Bottom	0.3888	ACI 7.6.1.1	0.40	26.294	OK
Z-Z, +1.20D+1.60Lr+0.50W	1.122	-X	Bottom	0.3888	ACI 7.6.1.1	0.40	26.294	OK
Z-Z, +1.20D+1.60Lr+0.50W	1.122	+X	Bottom	0.3888	ACI 7.6.1.1	0.40	26.294	OK
Z-Z, +1.20D+1.60S	1.262	-X	Bottom	0.3888	ACI 7.6.1.1	0.40	26.294	OK
Z-Z, +1.20D+1.60S	1.262	+X	Bottom	0.3888	ACI 7.6.1.1	0.40	26.294	OK
Z-Z, +1.20D+1.60S+0.50W	1.366	-X	Bottom	0.3888	ACI 7.6.1.1	0.40	26.294	OK
Z-Z, +1.20D+1.60S+0.50W	1.366	+X	Bottom	0.3888	ACI 7.6.1.1	0.40	26.294	OK
Z-Z, +1.20D+0.50Lr+W	0.8254	-X	Bottom	0.3888	ACI 7.6.1.1	0.40	26.294	OK
Z-Z, +1.20D+0.50Lr+W	0.8254	+X	Bottom	0.3888	ACI 7.6.1.1	0.40	26.294	OK
Z-Z, +1.20D+0.50S+W	0.9017	-X	Bottom	0.3888	ACI 7.6.1.1	0.40	26.294	OK
Z-Z, +1.20D+0.50S+W	0.9017	+X	Bottom	0.3888	ACI 7.6.1.1	0.40	26.294	OK
Z-Z, +1.20D+0.70S	0.7959	-X	Bottom	0.3888	ACI 7.6.1.1	0.40	26.294	OK
Z-Z, +1.20D+0.70S	0.7959	+X	Bottom	0.3888	ACI 7.6.1.1	0.40	26.294	OK
Z-Z, +0.90D+W	0.5346	-X	Bottom	0.3888	ACI 7.6.1.1	0.40	26.294	OK
Z-Z, +0.90D+W	0.5346	+X	Bottom	0.3888	ACI 7.6.1.1	0.40	26.294	OK
Z-Z, +0.90D	0.3252	-X	Bottom	0.3888	ACI 7.6.1.1	0.40	26.294	OK
Z-Z, +0.90D	0.3252	+X	Bottom	0.3888	ACI 7.6.1.1	0.40	26.294	OK

One Way Shear X

Load Combination...	Vu @ -X	Vu @ +X	Vu:Max	Phi Vn	Vu / Phi*Vn	Status
+1.40D	0.00 psi	0.00 psi	0.00 psi	42.89 psi	0.00	OK

Project Title: C & S Restaurant
Engineer: Mohammad Aljazzar P.E.
Project ID: CBE#: 25-304
Project Descr: Commercial Bldg.

General Footing

Project File: C and S Restaurant_29 Palms Hwy.ec6

DESCRIPTION: Footing at Posts (Door Jambs) Supporting Header Lines [B]

CBC 2022

One Way Shear X

Load Combination...	Vu @ -X	Vu @ +X	Vu:Max	Phi Vn	Vu / Phi*Vn	Status
+1.20D+0.50Lr	0.00 psi	0.00 psi	0.00 psi	42.89 psi	0.00	OK
+1.20D+0.50S	0.00 psi	0.00 psi	0.00 psi	42.89 psi	0.00	OK
+1.20D+1.60Lr	0.00 psi	0.00 psi	0.00 psi	42.89 psi	0.00	OK
+1.20D+1.60Lr+0.50W	0.00 psi	0.00 psi	0.00 psi	42.89 psi	0.00	OK
+1.20D+1.60S	0.00 psi	0.00 psi	0.00 psi	42.89 psi	0.00	OK
+1.20D+1.60S+0.50W	0.00 psi	0.00 psi	0.00 psi	42.89 psi	0.00	OK
+1.20D+0.50Lr+W	0.00 psi	0.00 psi	0.00 psi	42.89 psi	0.00	OK
+1.20D+0.50S+W	0.00 psi	0.00 psi	0.00 psi	42.89 psi	0.00	OK
+1.20D+0.70S	0.00 psi	0.00 psi	0.00 psi	42.89 psi	0.00	OK
+0.90D+W	0.00 psi	0.00 psi	0.00 psi	42.89 psi	0.00	OK
+0.90D	0.00 psi	0.00 psi	0.00 psi	42.89 psi	0.00	OK

One Way Shear Z

Load Combination...	Vu @ -Z	Vu @ +Z	Vu:Max	Phi Vn	Vu / Phi*Vn	Status
+1.40D	0.00 psi	0.00 psi	0.00 psi	42.89 psi	0.00	OK
+1.20D+0.50Lr	0.00 psi	0.00 psi	0.00 psi	42.89 psi	0.00	OK
+1.20D+0.50S	0.00 psi	0.00 psi	0.00 psi	42.89 psi	0.00	OK
+1.20D+1.60Lr	0.00 psi	0.00 psi	0.00 psi	42.89 psi	0.00	OK
+1.20D+1.60Lr+0.50W	0.00 psi	0.00 psi	0.00 psi	42.89 psi	0.00	OK
+1.20D+1.60S	0.00 psi	0.00 psi	0.00 psi	42.89 psi	0.00	OK
+1.20D+1.60S+0.50W	0.00 psi	0.00 psi	0.00 psi	42.89 psi	0.00	OK
+1.20D+0.50Lr+W	0.00 psi	0.00 psi	0.00 psi	42.89 psi	0.00	OK
+1.20D+0.50S+W	0.00 psi	0.00 psi	0.00 psi	42.89 psi	0.00	OK
+1.20D+0.70S	0.00 psi	0.00 psi	0.00 psi	42.89 psi	0.00	OK
+0.90D+W	0.00 psi	0.00 psi	0.00 psi	42.89 psi	0.00	OK
+0.90D	0.00 psi	0.00 psi	0.00 psi	42.89 psi	0.00	OK

Two-Way "Punching" Shear

All units k

Load Combination...	Vu	Phi*Vn	Vu / Phi*Vn	Status
+1.40D	2.95 psi	164.32 psi	0.02	OK
+1.20D+0.50Lr	3.59 psi	164.32 psi	0.02	OK
+1.20D+0.50S	4.03 psi	164.32 psi	0.02	OK
+1.20D+1.60Lr	5.92 psi	164.32 psi	0.04	OK
+1.20D+1.60Lr+0.50W	6.53 psi	164.32 psi	0.04	OK
+1.20D+1.60S	7.34 psi	164.32 psi	0.04	OK
+1.20D+1.60S+0.50W	7.95 psi	164.32 psi	0.05	OK
+1.20D+0.50Lr+W	4.81 psi	164.32 psi	0.03	OK
+1.20D+0.50S+W	5.25 psi	164.32 psi	0.03	OK
+1.20D+0.70S	4.63 psi	164.32 psi	0.03	OK
+0.90D+W	3.11 psi	164.32 psi	0.02	OK
+0.90D	1.89 psi	164.32 psi	0.01	OK

F.2 Concrete Wall & Footing Design

Plinth Wall Design:

Design Parameter		Soil Type	
		On-Site Sand and Silty Sands	
Internal Friction Angle (ϕ)		30°	
Unit Weight		134 lbs/ft^3	
Equivalent Fluid Pressure:	Active Condition (Level backfill)	40 lbs/ft^3 (Up to 6 feet)	45 lbs/ft^3 (>6 feet to 10 feet)
	Active Condition (2h:1v) backfill)	65 lbs/ft^3 (Up to 6 feet)	70 lbs/ft^3 (>6 feet to 10 feet)
	At-Rest Condition	60 lbs/ft^3 (Up to 6 feet)	100 lbs/ft^3 (>6 feet to 10 feet)
	Seismic (Level/2h:1v backfill)	--	125 lbs/ft^3 (>6 feet to 10 feet)

Notes:

1. Applicable to retaining walls only.
2. Active force applied a 1/3 wall height.
3. Seismic force applied to at ½ to 6/10 wall height.
4. Lateral pressure acts normally on the vertical stem.

The walls should be designed using a soil-footing coefficient of friction of 0.30 and an equivalent passive pressure of 300 lbs/ft3. The structural engineer should incorporate appropriate factors of safety in the design of the retaining walls.

4.9.1 Retaining Wall Foundation Design

Conventional footings for retaining walls founded in property compacted fill should be embedded at least 18 inches below the lowest adjacent grade. At this depth, an allowable bearing capacity of 2,800 psf may assumed for retaining walls founded in competent compacted fill.

CASE - A

Wall Footing, Line F & Lines [1] & [3] Between [D] & [F]

Project Title: C & S Restaurant
Engineer: Mohammad Aljazzar P.E.
Project ID: CBE#: 25-304
Project Descr: Commercial Bldg.

Restrained Retaining Wall

Project File: C and S Restaurant_29 Palms Hwy.ec6

DESCRIPTION: Wall Footing Line [1]D-F Post SOG Construction

CBC 2022

Code References

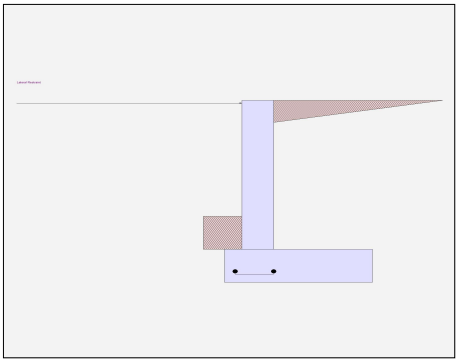
Calculations per IBC 2021, ACI 318-14

Criteria

Retained Height	=	4.50 ft
Wall height above soil	=	ft
Total Wall Height	=	4.50 ft
Top Support Height	=	4.250 ft
Slope Behind Wall	=	0
Height of Soil over Toe	=	12.0 in

Soil Data

Allow Soil Bearing	=	2,400.0 psf
Equivalent Fluid Pressure Method		
At-Rest Heel Pressure	=	60.0 psf/ft
	=	0.0 psf/ft
Passive Pressure	=	300.0 psf/ft
Soil Density	=	134.0 pcf
Footing || Soil Frictic	=	0.30
Soil height to ignore for passive pressure	=	12 in



Surcharge Loads

Surcharge Over Heel = 200.0 psf
>>>Used To Resist Sliding & Overturning
Surcharge Over Toe = psf
Used for Sliding & Overturning

Axial Load Applied to Stem

Axial Dead Load	=	480.0 lbs
Axial Live Load	=	275.0 lbs
Axial Load Eccentricity	=	4.0 in

Earth Pressure Seismic Load

Uniform Lateral Load Applied to Stem

Lateral Load	=	#/ft
...Height to Top	=	ft
...Height to Bottom	=	ft
Load Type	=	Wind (W) (Service Level)
Wind on Exposed Stem	=	0.00 psf (Strength Level)

Wind acts left-to-right toward retention side.

Adjacent Footing Load

Adjacent Footing Load	=	lbs
Footing Width	=	ft
Eccentricity	=	in
Wall to Ftg CL Dist	=	ft
Footing Type		Line Load
Base Above/Below Soil at Back of Wall	=	ft
Poisson's Ratio	=	0.3

K_h Soil Density Multiplier = 0.2 g Added seismic per unit area = 0.0 psf

Design Summary

Total Bearing Load	=	3,670.67 lbs	
...resultant ecc.	=	0.0 in	
Soil Pressure @ Toe	=	1,048.76 psf	OK
Soil Pressure @ Heel	=	1,048.76 psf	OK
Allowable	=	psf	
Soil Pressure Less Than Allowable			
ACI Factored @ Toe	=	1,289.94 psf	
ACI Factored @ Heel	=	1,289.94 psf	
Footing Shear @ Toe	=	0.7438 psi	OK
Footing Shear @ Heel	=	1.290 psi	OK
Allowable	=	82.158 psi	
Reaction at Top	=	387.868 lbs	
Reaction at Bottom	=	1,010.90 lbs	

Sliding Calcs
Lateral Sliding Force = 1,010.90 lbs

Vertical component of active lateral soil pressure IS NOT considered in the calculation of soil bearing pressures.

Load Factors

Building Code	
Dead Load	1.200
Live Load	1.600
Earth, H	1.600
Wind, W	1.000
Seismic, E	1.000

Concrete Stem Construction

Thickness = 8.00 in Fy = 60000 psi
Wall Weight = 100.0 psf f'c = 3000 psi
Stem is FIXED to top of footing

		@ Top Support	Mmax Between Top & Base	@ Base of Wall
		Stem OK	Stem OK	Stem OK
Design Height Above Ftg	=	4.250 ft	2.168 ft	0.00 ft
Rebar Size	=	# 4	# 4	# 4
Rebar Spacing	=	12.00 in	12.00 in	12.00 in
Rebar Placed at	=	Center	Center	Center
Rebar Depth 'd'	=	4.0 in	4.0 in	4.0 in
Design Data				
fb/FB + fa/Fa	=	0.100	0.091	0.204
Mu....Actual	=	343.394 ft-#	312.865 ft-#	697.31 ft-#
Mn * Phi.....Allowable	=	3,423.0 ft-#	3,423.0 ft-#	3,423.0 ft-#
Shear Force @ this height	=	563.65 lbs		1,016.75 lbs
Shear.....Actual	=	11.743 psi		21.182 psi
Shear.....Allowable	=	52.882 psi		52.882 psi

Project Title: C & S Restaurant
Engineer: Mohammad Aljazzar P.E.
Project ID: CBE#: 25-304
Project Descr: Commercial Bldg.

Restrained Retaining Wall

Project File: C and S Restaurant_29 Palms Hwy.ec6

DESCRIPTION: Wall Footing Line [1]D-F Post SOG Construction

CBC 2022

Footing Strengths & Dimensions

Toe Width	=	0.50 ft
Heel Width	=	3.0
Total Footing Width	=	3.50
Footing Thickness	=	12.0 in

f'c = 3,000 psi Fy = 60000 psi
Footing Concrete Density = 150 pcf
Min. As % = 0.0018
Cover @ Top = 2 in @ Btm.= 3 in

Footing Design Results

		Toe	Heel	
Factored Pressure	=	1,289.94	1,289.94	psf
Mu' : Upward	=	161.243		ft-#
Mu' : Downward	=	42.60		ft-#
Mu: Design	=	119	-181	ft-#
Actual 1-Way Shear	=	0.7438	1.290	psi
Allow 1-Way Shear	=	43.818	43.818	psi

Other Acceptable Sizes & Spacings:

Toe: # 4 @ 18.00 in -or- phiMn = phi * 5 * lambda * sqrt(fc) * Sm
Heel: None Spec'd -or- phiMn = phi * 5 * lambda * sqrt(fc) * Sm

Min footing T&S reinf Area 0.91 in2
Min footing T&S reinf Area per foot 0.26 in2 /ft

#4@ 9" o.c. Provided

If one layer of horizontal bars:	If two layers of horizontal bars:
#4@ 9.26 in	#4@ 18.52 in
#5@ 14.35 in	#5@ 28.70 in
#6@ 20.37 in	#6@ 40.74 in

Summary of Forces on Footing : Slab is NOT providing sliding, stem is FIXED at footing

Forces acting on footing for sliding & soil pressure....

Sliding Forces

Stem Shear @ Top of Footing	=	621.35 lbs
Heel Active Pressure	=	389.552
Key Active Pressure	=	0.0
Sliding Force	**=**	**1,010.90 lbs**

Stem is specified to be fixed to footing, and top restraint is assumed to react out any tendency for moment at the footing/soil interface, so uniform soil pressure is assumed.

Load & Moment Summary For Footing : For Soil Pressure Calcs

Moment @ Top of Footing Applied from Stem			=	-415.818 ft-#
Surcharge Over Heel		466.667	2.333	1,088.89
Adjacent Footing Load	=	0.0 lbs	0.0 ft	0.0 ft-#
Axial Dead Load on Stem	=	480.0 lbs	0.50 ft	240.0 ft-#
Axial Live Load on Stem	=	275.0 lbs	0.50 ft	137.50 ft-#
Soil Over Toe	=	67.0 lbs	0.250 ft	16.750 ft-#
Surcharge Over Toe	=	0.0 lbs	0.0 ft	0.0 ft-#
Stem Weight	=	450.0 lbs	0.8333 ft	375.0 ft-#
Soil Over Heel	=	1,407.0 lbs	2.333 ft	3,283.0 ft-#
Footing Weight	=	525.0 lbs	1.750 ft	918.75 ft-#
Total Vertical Force	=	3,670.67 lbs	Base Moment =	5,644.07 ft-#

Summary of Sliding Forces

	FS = 1.0	FS = 1.5
Lateral Force @ Base of Footing	1,010.90 lbs	1,516.36 lbs
less Passive Pressure Force	- 450.0 lbs	- 450.0 lbs
less Friction Force	- 1,018.70 lbs	- 1,018.70 lbs
Added Resisting Force Required	0.0 lbs	
Added Resisting Force Required for 1.5 Factor of Safety		47.66 lbs

Sliding Factor of Safety = 1.534: 1.00

Vertical component of active lateral soil pressure IS NOT considered in the calculation of Sliding Resistance.

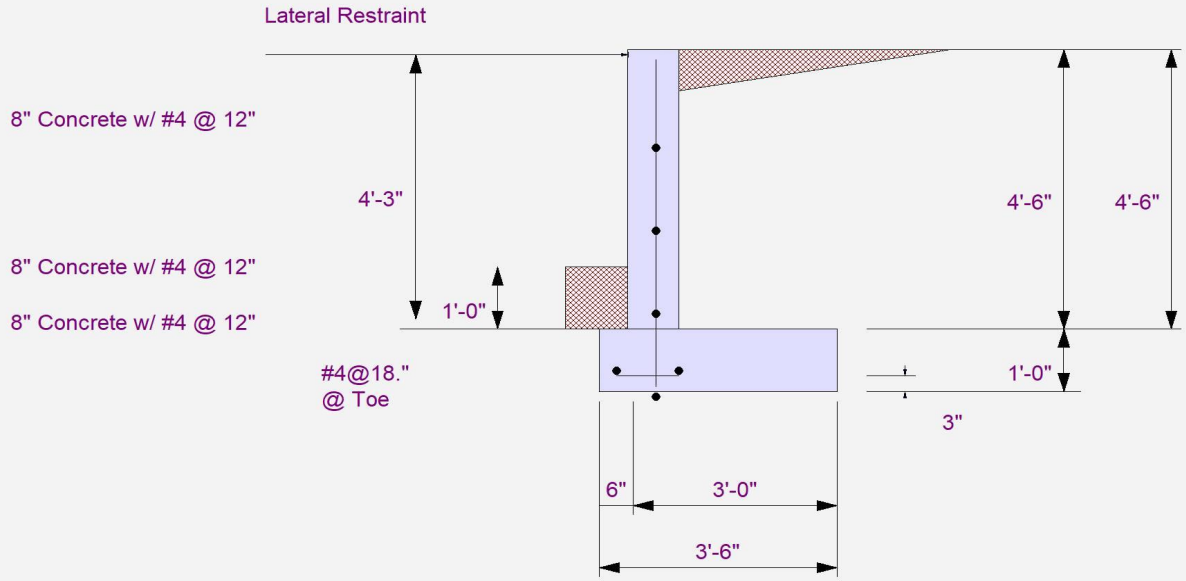
Project Title: C & S Restaurant
Engineer: Mohammad Aljazzar P.E.
Project ID: CBE#: 25-304
Project Descr: Commercial Bldg.

Restrained Retaining Wall

Project File: C and S Restaurant_29 Palms Hwy.ec6

DESCRIPTION: Wall Footing Line [1]D-F Post SOG Construction

CBC 2022



Restrained Retaining Wall

DESCRIPTION: Wall Footing Line [1]D-F Post SOG Construction

CBC 2022

Project Title: C & S Restaurant
Engineer: Mohammad Aljazzar P.E.
Project ID: CBE#: 25-304
Project Descr: Commercial Bldg.

Cantilevered Retaining Wall

Project File: C and S Restaurant_29 Palms Hwy.ec6

DESCRIPTION: Wall Footing Line [1]D-F Prior SOG Construction

CBC 2022

Code Reference

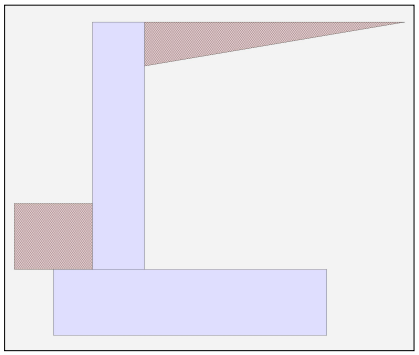
Calculations per IBC 2021, ACI 318-19, TMS 402-16

Criteria

Retained Height	=	3.75 ft
Wall height above soil	=	0.00 ft
Slope Behind Wall	=	0.00
Height of Soil over Toe	=	12.00 in
Water table above bottom of footing	=	0.0 ft

Soil Data

Allow Soil Bearing	=	2,400.0 psf
Equivalent Fluid Pressure Method		
Active Heel Pressure	=	40.0 psf/ft
	=	
Passive Pressure	=	300.0 psf/ft
Soil Density, Heel	=	134.00 pcf
Soil Density, Toe	=	110.00 pcf
Footing||Soil Friction	=	0.300
Soil height to ignore for passive pressure	=	12.00 in



Surcharge Loads

Surcharge Over Heel	=	100.0 psf
Used To Resist Sliding & Overturning		
Surcharge Over Toe	=	0.0
Used for Sliding & Overturning		

Axial Load Applied to Stem

Axial Dead Load	=	0.0 lbs
Axial Live Load	=	0.0 lbs
Axial Load Eccentricity	=	0.0 in

Lateral Load Applied to Stem

Lateral Load	=	0.0 #/ft
...Height to Top	=	0.00 ft
...Height to Bottom	=	0.00 ft
Load Type	=	Wind (W) (Service Level)
Wind on Exposed Stem (Strength Level)	=	0.0 psf

Adjacent Footing Load

Adjacent Footing Load	=	0.0 lbs
Footing Width	=	0.00 ft
Eccentricity	=	0.00 in
Wall to Ftg CL Dist	=	0.00 ft
Footing Type		Line Load
Base Above/Below Soil at Back of Wall	=	0.0 ft
Poisson's Ratio	=	0.300

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DESCRIPTION: Wall Footing Line [1]D-F Prior SOG Construction

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Design Summary

Wall Stability Ratios

Overturning	=	5.05	OK
Sliding	=	1.53	OK
Global Stability	=	3.50	

Total Bearing Load	=	2,584	lbs
...resultant ecc.	=	1.24	in

Eccentricity within middle third

Soil Pressure @ Toe	=	869 psf	OK
Soil Pressure @ Heel	=	607 psf	OK
Allowable	=	2,400 psf	

Soil Pressure Less Than Allowable

ACI Factored @ Toe	=	1,112 psf	
ACI Factored @ Heel	=	777 psf	
Footing Shear @ Toe	=	0.7 psi	OK
Footing Shear @ Heel	=	4.0 psi	OK
Allowable	=	82.2 psi	

Sliding Calcs

Lateral Sliding Force	=	593.0	lbs

Vertical component of active lateral soil pressure IS considered in the calculation of soil bearing pressures.

Load Factors

Building Code	
Dead Load	1.200
Live Load	1.600
Earth, H	1.600
Wind, W	1.600
Seismic, E	1.000

Stem Construction

		Bottom
		Stem OK
Design Height Above Ftg	ft =	0.00
Wall Material Above "Ht"	=	Concrete
Design Method	=	SD SD SD
Thickness	=	8.00
Rebar Size	=	# 4
Rebar Spacing	=	12.00
Rebar Placed at	=	Center
Design Data		
fb/FB + fa/Fa	=	**0.262**
Total Force @ Section		
Service Level	lbs =	
Strength Level	lbs =	629.1
Moment....Actual		
Service Level	ft-# =	
Strength Level	ft-# =	898.3
Moment.....Allowable	=	3,423.0
Shear.....Actual		
Service Level	psi =	
Strength Level	psi =	13.1
Shear.....Allowable	psi =	52.9
Anet (Masonry)	in2 =	
Wall Weight	psf =	100.0
Rebar Depth 'd'	in =	4.00

Masonry Data

f'm	psi =	
Fs	psi =	
Solid Grouting	=	
Modular Ratio 'n'	=	
Equiv. Solid Thick.	=	
Masonry Block Type	=	
Masonry Design Method	=	ASD

Concrete Data

f'c	psi =	3,000.0
Fy	psi =	60,000.0

Summary of Sliding Forces

	FS = 1.0	FS = 1.5
Lateral Force @ Base of Footing	593.04 lbs	889.56 lbs
less 30% Passive Force	- 135.0 lbs	- 135.0 lbs
less 100% Friction Force	- 775.06 lbs	- 775.06 lbs
Added Resisting Force Required	0.0 lbs	
Added Resisting Force Required for 1.5 Factor of Safety		0.00 lbs

Sliding Factor of Safety = 1.535: 1.00

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DESCRIPTION: Wall Footing Line [1]D-F Prior SOG Construction

CBC 2022

Concrete Stem Rebar Area Details

Bottom Stem	Vertical Reinforcing	Horizontal Reinforcing	
As (based on applied moment) :	0.0542 in2/ft		
0.0018bh : 0.0018(12)(8) :	0.1728 in2/ft	Horizontal Reinforcing Options :	
	==========	One layer of :	Two layers of :
Required Area :	0.1728 in2/ft	#4@ 13.89 in	#4@ 27.78 in
Provided Area :	0.2 in2/ft	#5@ 21.53 in	#5@ 43.06 in
Maximum Area :	0.6503 in2/ft	#6@ 30.56 in	#6@ 61.11 in

Footing Data

Toe Width	=	0.50 ft
Heel Width	=	3.00
Total Footing Width	=	3.50
Footing Thickness	=	12.00 in

f'c = 3,000 psi Fy = 60,000 psi
Footing Concrete Density = 150.00 pcf
Min. As % = 0.0018
Cover @ Top 2.00 @ Btm.= 3.00 in

Footing Design Results

		Toe	Heel	
Factored Pressure	=	1,112	777	psf
Mu' : Upward	=	137	2,317	ft-#
Mu' : Downward	=	43	3,398	ft-#
Mu: Design	=	94	1,081	ft-#
φ Mn	=	2,739	2,739	ft-#
Actual 1-Way Shear	=	0.68	4.03	psi
Allow 1-Way Shear	=	43.82	43.82	psi

Toe Reinforcing = None Spec'd
Heel Reinforcing = None Spec'd
Key Reinforcing = None Spec'd
Footing Torsion, Tu = 0.00 ft-lbs
Footing Allow. Torsion, φ Tn = 0.00 ft-lbs

If torsion exceeds allowable, provide supplemental design for footing torsion.

Other Acceptable Sizes & Spacings

Toe: phiMn = phi*5*lambda*sqrt(fc)*Sm

Heel: phiMn = phi*5*lambda*sqrt(fc)*Sm

Key: No key defined

Min footing T&S reinf Area 0.91 in2
Min footing T&S reinf Area per foot 0.26 in2 /ft

If one layer of horizontal bars:	If two layers of horizontal bars:
#4@ 9.26 in	#4@ 18.52 in
#5@ 14.35 in	#5@ 28.70 in
#6@ 20.37 in	#6@ 40.74 in

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DESCRIPTION: Wall Footing Line [1]D-F Prior SOG Construction

CBC 2022

Summary of Overturning & Resisting Forces & Moments

Item		.....OVERTURNING..... Force lbs	Distance ft	Moment ft-#		.....RESISTING..... Force lbs	Distance ft	Moment ft-#
HL Act Pres (ab water tbl)		451.3	1.58	714.5	Soil Over HL (ab. water tbl)	1,172.5	2.33	2,735.8
HL Act Pres (be water tbl)					Soil Over HL (bel. water tbl)		2.33	2,735.8
Hydrostatic Force					Water Table			
Buoyant Force	=				Sloped Soil Over Heel	=		
Surcharge over Heel	=	141.8	2.38	336.8	Surcharge Over Heel	= 233.3	2.33	544.4
Surcharge Over Toe	=				Adjacent Footing Load	=		
Adjacent Footing Load	=				Axial Dead Load on Stem	=		
Added Lateral Load	=				* Axial Live Load on Stem	=		
Load @ Stem Above Soil	=				Soil Over Toe	= 55.0	0.25	13.8
	=				Surcharge Over Toe	=		
					Stem Weight(s)	= 375.0	0.83	312.5
					Earth @ Stem Transitions	=		
Total	=	593.0	**O.T.M.** =	1,051.2	Footing Weight	= 525.0	1.75	918.8
					Key Weight	=		
					Vert. Component	= 222.7	3.50	779.5
					Total =	2,583.5 lbs	**R.M.=**	5,304.8

Resisting/Overturning Ratio = **5.05**
Vertical Loads used for Soil Pressure = 2,583.5 lbs

* Axial live load NOT included in total displayed, or used for overturning resistance, but is included for soil pressure calculation.

Vertical component of active lateral soil pressure IS considered in the calculation of Sliding Resistance.

Vertical component of active lateral soil pressure IS considered in the calculation of Overturning Resistance.

Tilt

Horizontal Deflection at Top of Wall due to settlement of soil

(Deflection due to wall bending not considered)

Soil Spring Reaction Modulus	250.0	pci
Horizontal Defl @ Top of Wall (approximate only)	0.026	in

The above calculation is not valid if the heel soil bearing pressure exceeds that of the toe, because the wall would then tend to rotate into the retained soil.

Cantilevered Retaining Wall

DESCRIPTION: Wall Footing Line [1]D-F Prior SOG Construction

CBC 2022

Rebar Lap & Embedment Lengths Information

Stem Design Segment: Bottom

Stem Design Height: 0.00 ft above top of footing

Lap Splice length for #4 bar specified in this stem design segment (25.4.2.4a) =	17.09 in
Development length for #4 bar specified in this stem design segment =	13.15 in
Hooked embedment length into footing for #4 bar specified in this stem design segment =	6.00 in
As Provided =	0.2000 in2/ft
As Required =	0.1728 in2/ft

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Cantilevered Retaining Wall

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DESCRIPTION: Wall Footing Line [1]D-F Prior SOG Construction

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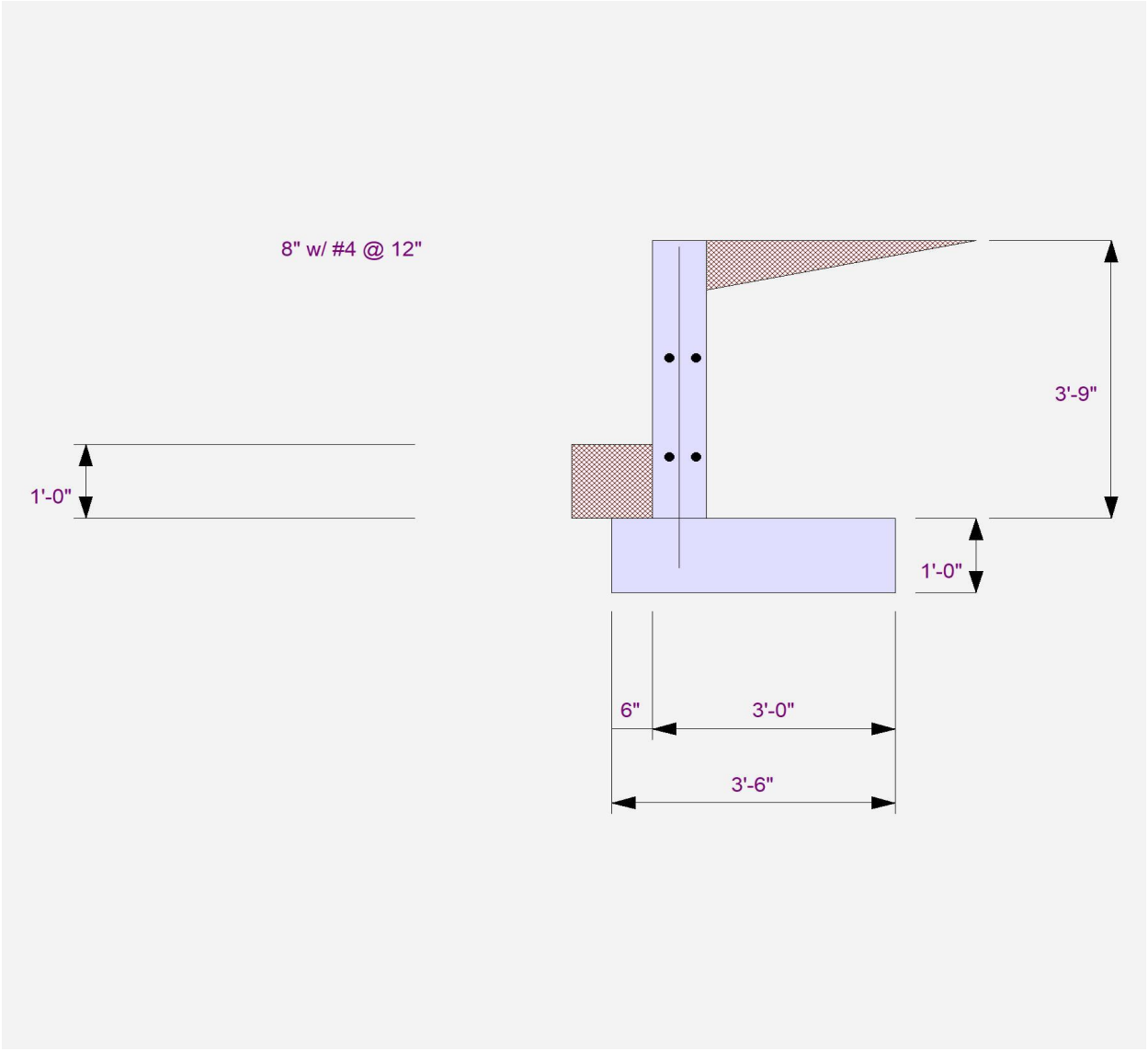

Cantilevered Retaining Wall

DESCRIPTION: Wall Footing Line [1]D-F Prior SOG Construction

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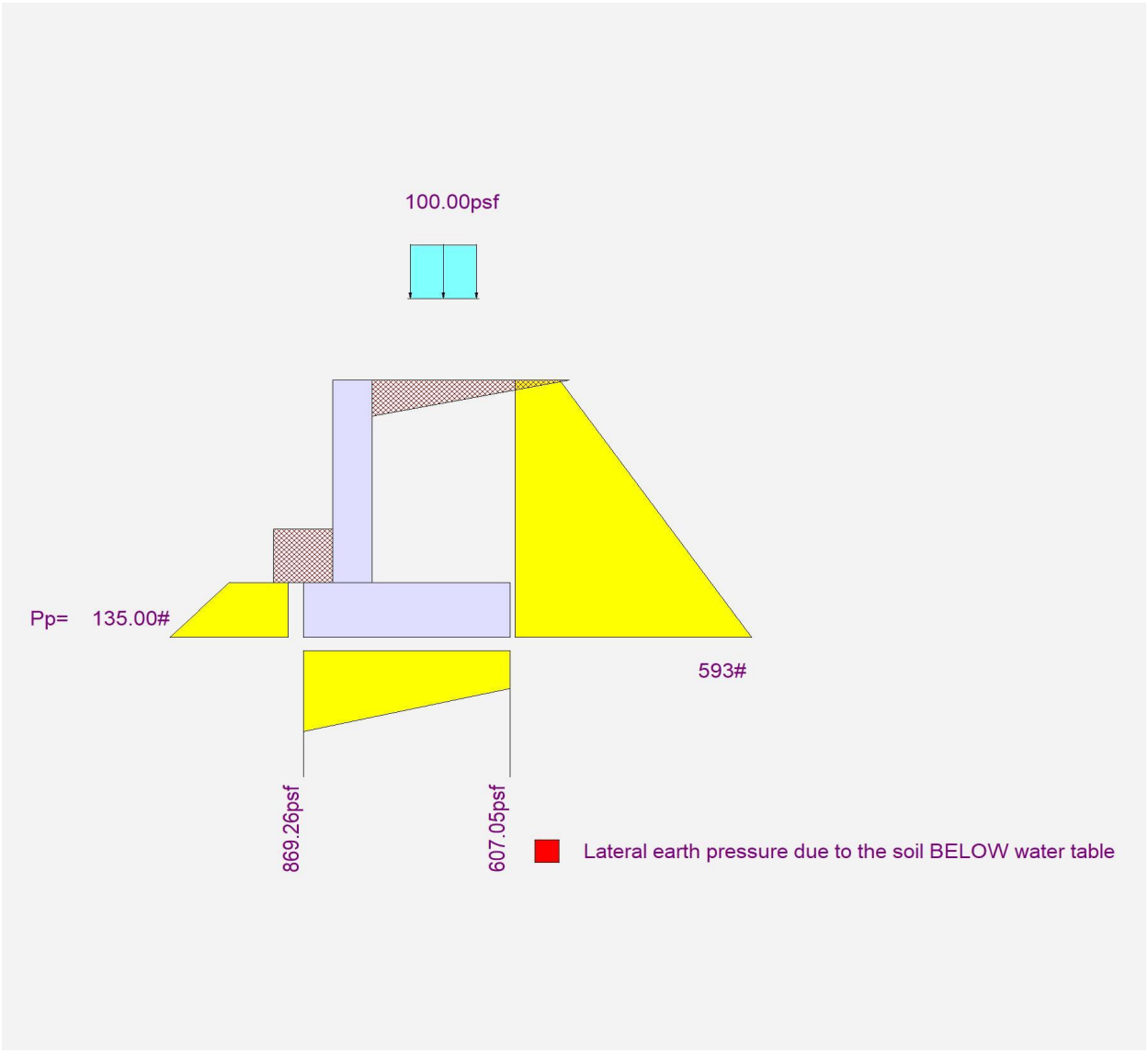


CASE - B

Wall Footing, Lines [1] & [3] Between [B] & [D]

Project Title: C & S Restaurant
Engineer: Mohammad Aljazzar P.E.
Project ID: CBE#: 25-304
Project Descr: Commercial Bldg.

Restrained Retaining Wall

Project File: C and S Restaurant_29 Palms Hwy.ec6

DESCRIPTION: Wall Footing Line [1]B-D Post SOG Construction

CBC 2022

Code References

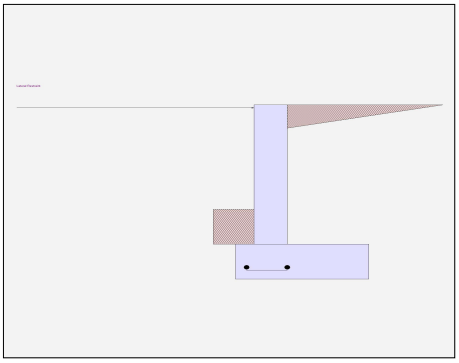
Calculations per IBC 2021, ACI 318-14

Criteria

Retained Height	=	4.0 ft
Wall height above soil	=	ft
Total Wall Height	=	4.0 ft
Top Support Height	=	3.750 ft
Slope Behind Wall	=	0
Height of Soil over Toe	=	12.0 in

Soil Data

Allow Soil Bearing	=	2,400.0 psf
Equivalent Fluid Pressure Method		
At-Rest Heel Pressure	=	60.0 psf/ft
	=	0.0 psf/ft
Passive Pressure	=	300.0 psf/ft
Soil Density	=	134.0 pcf
Footing || Soil Friction	=	0.30
Soil height to ignore for passive pressure	=	12 in



Surcharge Loads

Surcharge Over Heel	=	200.0 psf
>>>Used To Resist Sliding & Overturning		
Surcharge Over Toe	=	psf
Used for Sliding & Overturning		

Axial Load Applied to Stem

Axial Dead Load	=	480.0 lbs
Axial Live Load	=	275.0 lbs
Axial Load Eccentricity	=	4.0 in

Earth Pressure Seismic Load

Uniform Lateral Load Applied to Stem

Lateral Load	=	#/ft
...Height to Top	=	ft
...Height to Bottom	=	ft
Load Type	=	Wind (W) (Service Level)
Wind on Exposed Stem	=	0.00 psf (Strength Level)

Wind acts left-to-right toward retention side.

Adjacent Footing Load

Adjacent Footing Load	=	lbs
Footing Width	=	ft
Eccentricity	=	in
Wall to Ftg CL Dist	=	ft
Footing Type		Line Load
Base Above/Below Soil at Back of Wall	=	ft
Poisson's Ratio	=	0.3

K_h Soil Density Multiplier = 0.2 g Added seismic per unit area = 0.0 psf

Design Summary

Total Bearing Load	=	3,021.33 lbs	
...resultant ecc.	=	0.0 in	
Soil Pressure @ Toe	=	1,007.11 psf	OK
Soil Pressure @ Heel	=	1,007.11 psf	OK
Allowable	=	psf	
Soil Pressure Less Than Allowable			
ACI Factored @ Toe	=	1,245.20 psf	
ACI Factored @ Heel	=	1,245.20 psf	
Footing Shear @ Toe	=	0.7438 psi	OK
Footing Shear @ Heel	=	1.558 psi	OK
Allowable	=	82.158 psi	
Reaction at Top	=	356.279 lbs	
Reaction at Bottom	=	840.25 lbs	

Sliding Calcs
Lateral Sliding Force = 840.25 lbs

Vertical component of active lateral soil pressure IS NOT considered in the calculation of soil bearing pressures.

Load Factors

Building Code	
Dead Load	1.200
Live Load	1.600
Earth, H	1.600
Wind, W	1.000
Seismic, E	1.000

Concrete Stem Construction

Thickness = 8.00 in Fy = 60000 psi
Wall Weight = 100.0 psf f'c = 3000 psi
Stem is FIXED to top of footing

		@ Top Support	Mmax Between Top & Base	@ Base of Wall
		Stem OK	Stem OK	Stem OK
Design Height Above Ftg	=	3.750 ft	1.807 ft	0.00 ft
Rebar Size	=	# 4	# 4	# 4
Rebar Spacing	=	12.00 in	12.00 in	12.00 in
Rebar Placed at	=	Center	Center	Center
Rebar Depth 'd'	=	4.0 in	4.0 in	4.0 in
Design Data				
fb/FB + fa/Fa	=	0.100	0.060	0.134
Mu....Actual	=	343.394 ft-#	205.869 ft-#	459.863 ft-#
Mn * Phi.....Allowable	=	3,423.0 ft-#	3,423.0 ft-#	3,423.0 ft-#
Shear Force @ this height	=	509.76 lbs		794.72 lbs
Shear.....Actual	=	10.620 psi		16.557 psi
Shear.....Allowable	=	52.882 psi		52.882 psi

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Restrained Retaining Wall

Project File: C and S Restaurant_29 Palms Hwy.ec6

DESCRIPTION: Wall Footing Line [1]B-D Post SOG Construction

CBC 2022

Footing Strengths & Dimensions

Toe Width	=	0.50 ft
Heel Width	=	2.50
Total Footing Width	=	3.0
Footing Thickness	=	12.0 in

f'c = 3,000 psi Fy = 60000 psi
Footing Concrete Density = 150 pcf
Min. As % = 0.0018
Cover @ Top = 2 in @ Btm.= 3 in

Footing Design Results

		Toe	Heel	
Factored Pressure	=	1,245.20	1,245.20	psf
Mu' : Upward	=	155.650		ft-#
Mu' : Downward	=	42.60		ft-#
Mu: Design	=	113	-171	ft-#
Actual 1-Way Shear	=	0.7438	1.558	psi
Allow 1-Way Shear	=	43.818	43.818	psi

Other Acceptable Sizes & Spacings:

Toe: # 4 @ 18.00 in -or- phiMn = phi * 5 * lambda * sqrt(fc) * Sm
Heel: None Spec'd -or- phiMn = phi * 5 * lambda * sqrt(fc) * Sm

Min footing T&S reinf Area 0.78 in2
Min footing T&S reinf Area per foot 0.26 in2 /ft

If one layer of horizontal bars:	If two layers of horizontal bars:
#4@ 9.26 in	#4@ 18.52 in
#5@ 14.35 in	#5@ 28.70 in
#6@ 20.37 in	#6@ 40.74 in

Summary of Forces on Footing : Slab is NOT providing sliding, stem is FIXED at footing

Forces acting on footing for sliding & soil pressure....

Sliding Forces

Stem Shear @ Top of Footing	=	480.697 lbs
Heel Active Pressure	=	359.552
Key Active Pressure	=	0.0
Sliding Force	**=**	**840.25 lbs**

Stem is specified to be fixed to footing, and top restraint is assumed to react out any tendency for moment at the footing/soil interface, so uniform soil pressure is assumed.

Load & Moment Summary For Footing : For Soil Pressure Calcs

Moment @ Top of Footing Applied from Stem			=	-267.414 ft-#
Surcharge Over Heel		366.667	2.083	763.89
Adjacent Footing Load	=	0.0 lbs	0.0 ft	0.0 ft-#
Axial Dead Load on Stem	=	480.0 lbs	0.50 ft	240.0 ft-#
Axial Live Load on Stem	=	275.0 lbs	0.50 ft	137.50 ft-#
Soil Over Toe	=	67.0 lbs	0.250 ft	16.750 ft-#
Surcharge Over Toe	=	0.0 lbs	0.0 ft	0.0 ft-#
Stem Weight	=	400.0 lbs	0.8333 ft	333.333 ft-#
Soil Over Heel	=	982.67 lbs	2.083 ft	2,047.22 ft-#
Footing Weight	=	450.0 lbs	1.50 ft	675.0 ft-#
Total Vertical Force	=	3,021.33 lbs	Base Moment =	3,946.28 ft-#

Summary of Sliding Forces

	FS = 1.0	FS = 1.5
Lateral Force @ Base of Footing	840.25 lbs	1,260.37 lbs
less Passive Pressure Force	- 450.0 lbs	- 450.0 lbs
less Friction Force	- 823.90 lbs	- 823.90 lbs
Added Resisting Force Required	0.0 lbs	
Added Resisting Force Required for 1.5 Factor of Safety		0.00 lbs

Sliding Factor of Safety = 1.614: 1.00

Vertical component of active lateral soil pressure IS NOT considered in the calculation of Sliding Resistance.

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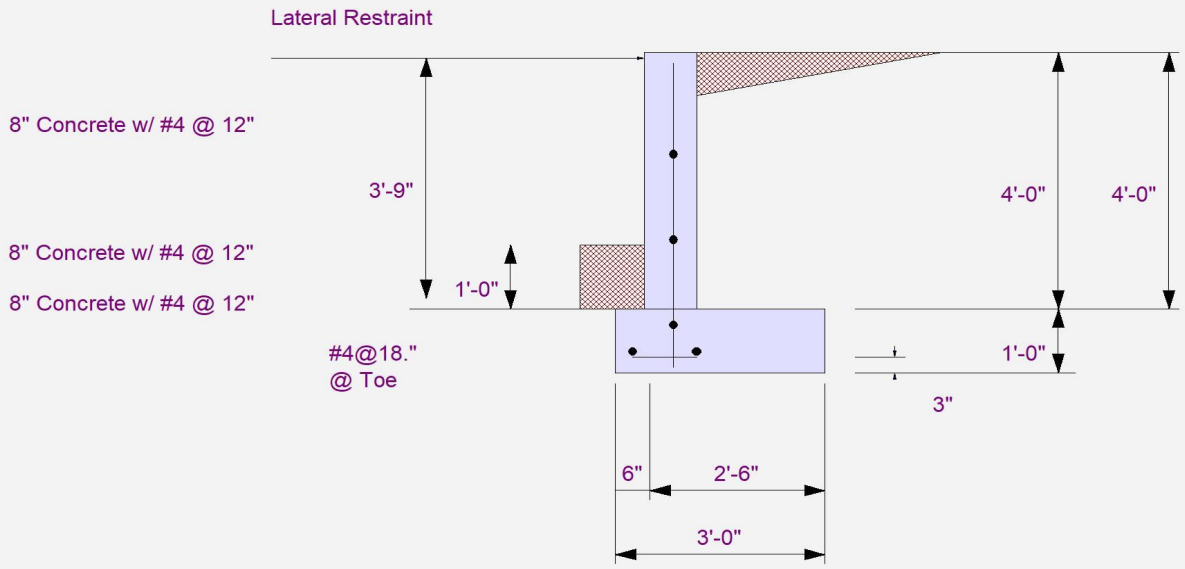

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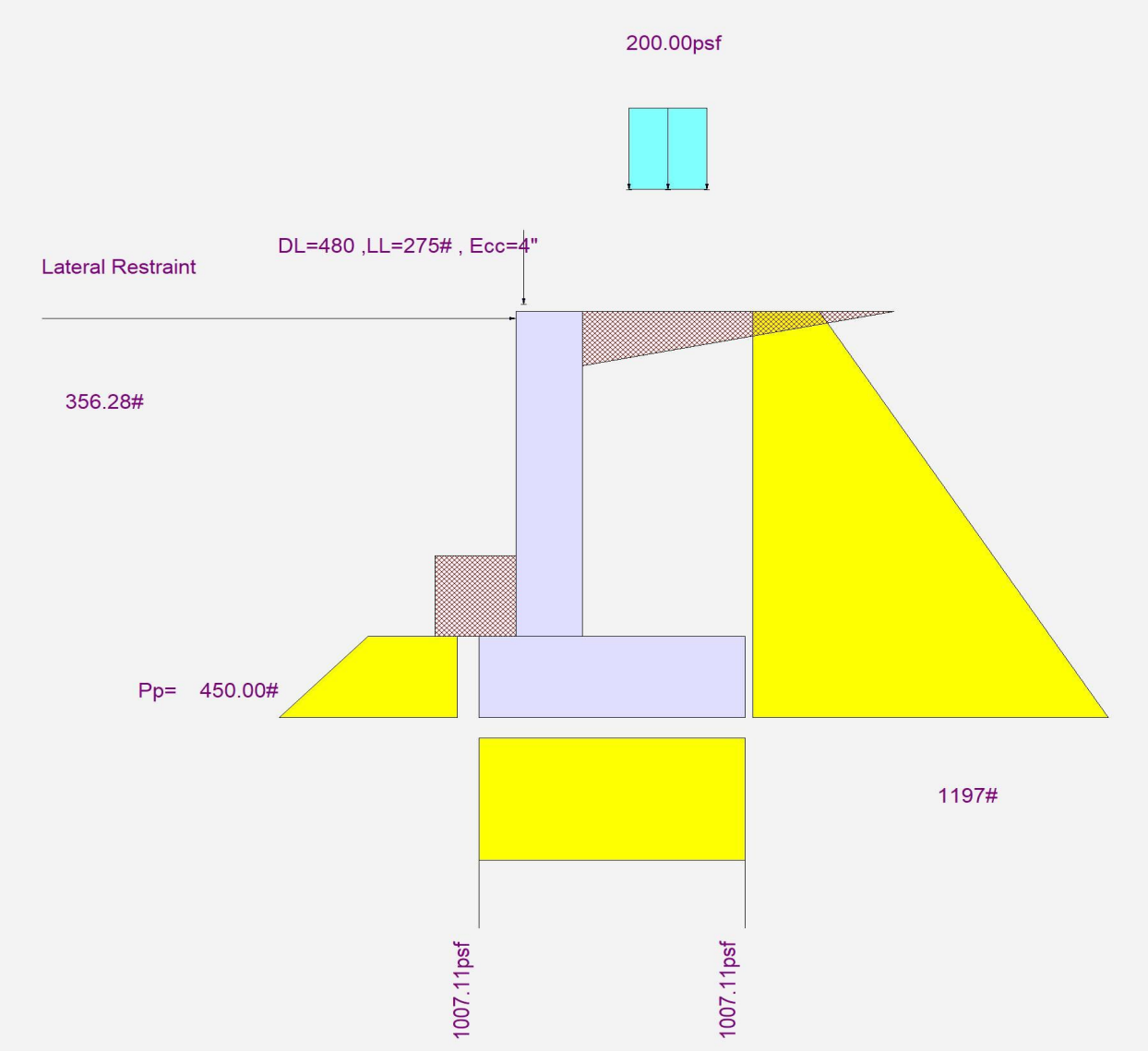

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Engineer: Mohammad Aljazzar P.E.
Project ID: CBE#: 25-304
Project Descr: Commercial Bldg.

Cantilevered Retaining Wall

Project File: C and S Restaurant_29 Palms Hwy.ec6

DESCRIPTION: Wall Footing Line [1]B-D Prior SOG Construction

CBC 2022

Code Reference

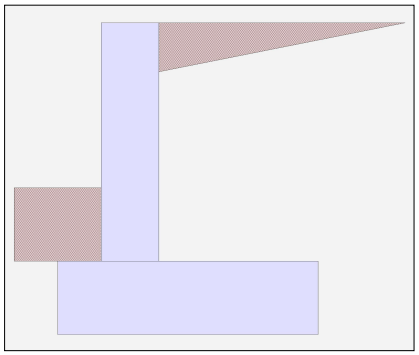
Calculations per IBC 2021, ACI 318-19, TMS 402-16

Criteria

Retained Height	=	3.25 ft
Wall height above soil	=	0.00 ft
Slope Behind Wall	=	0.00
Height of Soil over Toe	=	12.00 in
Water table above bottom of footing	=	0.0 ft

Soil Data

Allow Soil Bearing	=	2,400.0 psf
Equivalent Fluid Pressure Method		
Active Heel Pressure	=	40.0 psf/ft
	=	
Passive Pressure	=	300.0 psf/ft
Soil Density, Heel	=	134.00 pcf
Soil Density, Toe	=	110.00 pcf
Footing||Soil Friction	=	0.300
Soil height to ignore for passive pressure	=	12.00 in



Surcharge Loads

Surcharge Over Heel	=	100.0 psf
Used To Resist Sliding & Overturning		
Surcharge Over Toe	=	0.0
Used for Sliding & Overturning		

Axial Load Applied to Stem

Axial Dead Load	=	0.0 lbs
Axial Live Load	=	0.0 lbs
Axial Load Eccentricity	=	0.0 in

Lateral Load Applied to Stem

Lateral Load	=	0.0 #/ft
...Height to Top	=	0.00 ft
...Height to Bottom	=	0.00 ft
Load Type	=	Wind (W) (Service Level)
Wind on Exposed Stem (Strength Level)	=	0.0 psf

Adjacent Footing Load

Adjacent Footing Load	=	0.0 lbs
Footing Width	=	0.00 ft
Eccentricity	=	0.00 in
Wall to Ftg CL Dist	=	0.00 ft
Footing Type		Line Load
Base Above/Below Soil at Back of Wall	=	0.0 ft
Poisson's Ratio	=	0.300

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CBC 2022

Design Summary

Wall Stability Ratios

Overturning	=	4.53	OK
Sliding	=	1.50	OK
Global Stability	=	3.83	

Total Bearing Load	=	1,991	lbs
...resultant ecc.	=	1.36	in

Eccentricity within middle third

Soil Pressure @ Toe	=	814	psf	OK
Soil Pressure @ Heel	=	512	psf	OK
Allowable	=	2,400	psf	

Soil Pressure Less Than Allowable

ACI Factored @ Toe	=	1,038	psf	
ACI Factored @ Heel	=	653	psf	
Footing Shear @ Toe	=	0.7	psi	OK
Footing Shear @ Heel	=	3.8	psi	OK
Allowable	=	82.2	psi	

Sliding Calcs

Lateral Sliding Force	=	488.1	lbs

Vertical component of active lateral soil pressure IS considered in the calculation of soil bearing pressures.

Load Factors

Building Code	
Dead Load	1.200
Live Load	1.600
Earth, H	1.600
Wind, W	1.600
Seismic, E	1.000

Stem Construction

		Bottom	
		Stem OK	
Design Height Above Ftg	ft =	0.00	
Wall Material Above "Ht"	=	Concrete	
Design Method	=	SD	SD SD
Thickness	=	8.00	
Rebar Size	=	# 4	
Rebar Spacing	=	12.00	
Rebar Placed at	=	Center	
Design Data			
fb/FB + fa/Fa	=	**0.180**	
Total Force @ Section			
Service Level	lbs =		
Strength Level	lbs =	493.2	
Moment....Actual			
Service Level	ft-# =		
Strength Level	ft-# =	618.4	
Moment.....Allowable	=	3,423.0	
Shear.....Actual			
Service Level	psi =		
Strength Level	psi =	10.3	
Shear.....Allowable	psi =	52.9	
Anet (Masonry)	in2 =		
Wall Weight	psf =	100.0	
Rebar Depth 'd'	in =	4.00	

Masonry Data

f'm	psi =	
Fs	psi =	
Solid Grouting	=	
Modular Ratio 'n'	=	
Equiv. Solid Thick.	=	
Masonry Block Type	=	
Masonry Design Method	=	ASD

Concrete Data

f'c	psi =	3,000.0
Fy	psi =	60,000.0

Summary of Sliding Forces

	FS = 1.0	FS = 1.5
Lateral Force @ Base of Footing	488.12 lbs	732.17 lbs
less 30% Passive Force	- 135.0 lbs	- 135.0 lbs
less 100% Friction Force	- 597.22 lbs	- 597.22 lbs
Added Resisting Force Required	0.0 lbs	
Added Resisting Force Required for 1.5 Factor of Safety		0.00 lbs

Sliding Factor of Safety = 1.50: 1.00

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DESCRIPTION: Wall Footing Line [1]B-D Prior SOG Construction

CBC 2022

Concrete Stem Rebar Area Details

Bottom Stem	Vertical Reinforcing	Horizontal Reinforcing	
As (based on applied moment) :	0.0373 in2/ft		
0.0018bh : 0.0018(12)(8) :	0.1728 in2/ft	Horizontal Reinforcing Options :	
	==========	One layer of :	Two layers of :
Required Area :	0.1728 in2/ft	#4@ 13.89 in	#4@ 27.78 in
Provided Area :	0.2 in2/ft	#5@ 21.53 in	#5@ 43.06 in
Maximum Area :	0.6503 in2/ft	#6@ 30.56 in	#6@ 61.11 in

Footing Data

Toe Width	=	0.50 ft
Heel Width	=	2.50
Total Footing Width	=	3.00
Footing Thickness	=	12.00 in

f'c = 3,000 psi Fy = 60,000 psi
Footing Concrete Density = 150.00 pcf
Min. As % = 0.0018
Cover @ Top 2.00 @ Btm.= 3.00 in

Footing Design Results

		Toe	Heel	
Factored Pressure	=	1,038	653	psf
Mu' : Upward	=	127	1,231	ft-#
Mu' : Downward	=	43	1,974	ft-#
Mu: Design	=	84	744	ft-#
φ Mn	=	2,739	2,739	ft-#
Actual 1-Way Shear	=	0.68	3.78	psi
Allow 1-Way Shear	=	43.82	43.82	psi
Toe Reinforcing	=	None Spec'd		
Heel Reinforcing	=	None Spec'd		
Key Reinforcing	=	None Spec'd		
Footing Torsion, Tu	=	0.00		ft-lbs
Footing Allow. Torsion, φ Tn	=	0.00		ft-lbs

If torsion exceeds allowable, provide supplemental design for footing torsion.

Other Acceptable Sizes & Spacings

Toe: phiMn = phi*5*lambda*sqrt(fc)*Sm

Heel: phiMn = phi*5*lambda*sqrt(fc)*Sm

Key: No key defined

Min footing T&S reinf Area	0.78	in2
Min footing T&S reinf Area per foot	0.26	in2 /ft

If one layer of horizontal bars:	If two layers of horizontal bars:
#4@ 9.26 in	#4@ 18.52 in
#5@ 14.35 in	#5@ 28.70 in
#6@ 20.37 in	#6@ 40.74 in

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Cantilevered Retaining Wall

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DESCRIPTION: Wall Footing Line [1]B-D Prior SOG Construction

CBC 2022

Summary of Overturning & Resisting Forces & Moments

Item		.....OVERTURNING..... Force lbs	Distance ft	Moment ft-#		.....RESISTING..... Force lbs	Distance ft	Moment ft-#
HL Act Pres (ab water tbl)		361.3	1.42	511.8	Soil Over HL (ab. water tbl)	798.9	2.08	1,664.7
HL Act Pres (be water tbl)					Soil Over HL (bel. water tbl)		2.08	1,664.7
Hydrostatic Force					Water Table			
Buoyant Force	=				Sloped Soil Over Heel	=		
Surcharge over Heel	=	126.9	2.13	269.6	Surcharge Over Heel	= 183.4	2.08	382.2
Surcharge Over Toe	=				Adjacent Footing Load	=		
Adjacent Footing Load	=				Axial Dead Load on Stem	=		
Added Lateral Load	=				* Axial Live Load on Stem	=		
Load @ Stem Above Soil	=				Soil Over Toe	= 55.0	0.25	13.8
	=				Surcharge Over Toe	=		
					Stem Weight(s)	= 325.0	0.83	270.8
					Earth @ Stem Transitions	=		
					Footing Weight	= 450.2	1.50	675.5
					Key Weight	=		
					Vert. Component	= 178.3	3.00	535.0
Total	=	488.1	**O.T.M.** =	781.4	**Total =**	1,990.7 lbs	**R.M.=**	3,542.0

Resisting/Overturning Ratio = **4.53**
Vertical Loads used for Soil Pressure = 1,990.7 lbs

* Axial live load NOT included in total displayed, or used for overturning resistance, but is included for soil pressure calculation.

Vertical component of active lateral soil pressure IS considered in the calculation of Sliding Resistance.

Vertical component of active lateral soil pressure IS considered in the calculation of Overturning Resistance.

Tilt

Horizontal Deflection at Top of Wall due to settlement of soil

(Deflection due to wall bending not considered)

Soil Spring Reaction Modulus	250.0	pci
Horizontal Defl @ Top of Wall (approximate only)	0.024	in

The above calculation is not valid if the heel soil bearing pressure exceeds that of the toe, because the wall would then tend to rotate into the retained soil.

Cantilevered Retaining Wall

DESCRIPTION: Wall Footing Line [1]B-D Prior SOG Construction

CBC 2022

Rebar Lap & Embedment Lengths Information

Stem Design Segment: Bottom

Stem Design Height: 0.00 ft above top of footing

Lap Splice length for #4 bar specified in this stem design segment (25.4.2.4a) =	17.09 in
Development length for #4 bar specified in this stem design segment =	13.15 in
Hooked embedment length into footing for #4 bar specified in this stem design segment =	6.00 in
As Provided =	0.2000 in2/ft
As Required =	0.1728 in2/ft

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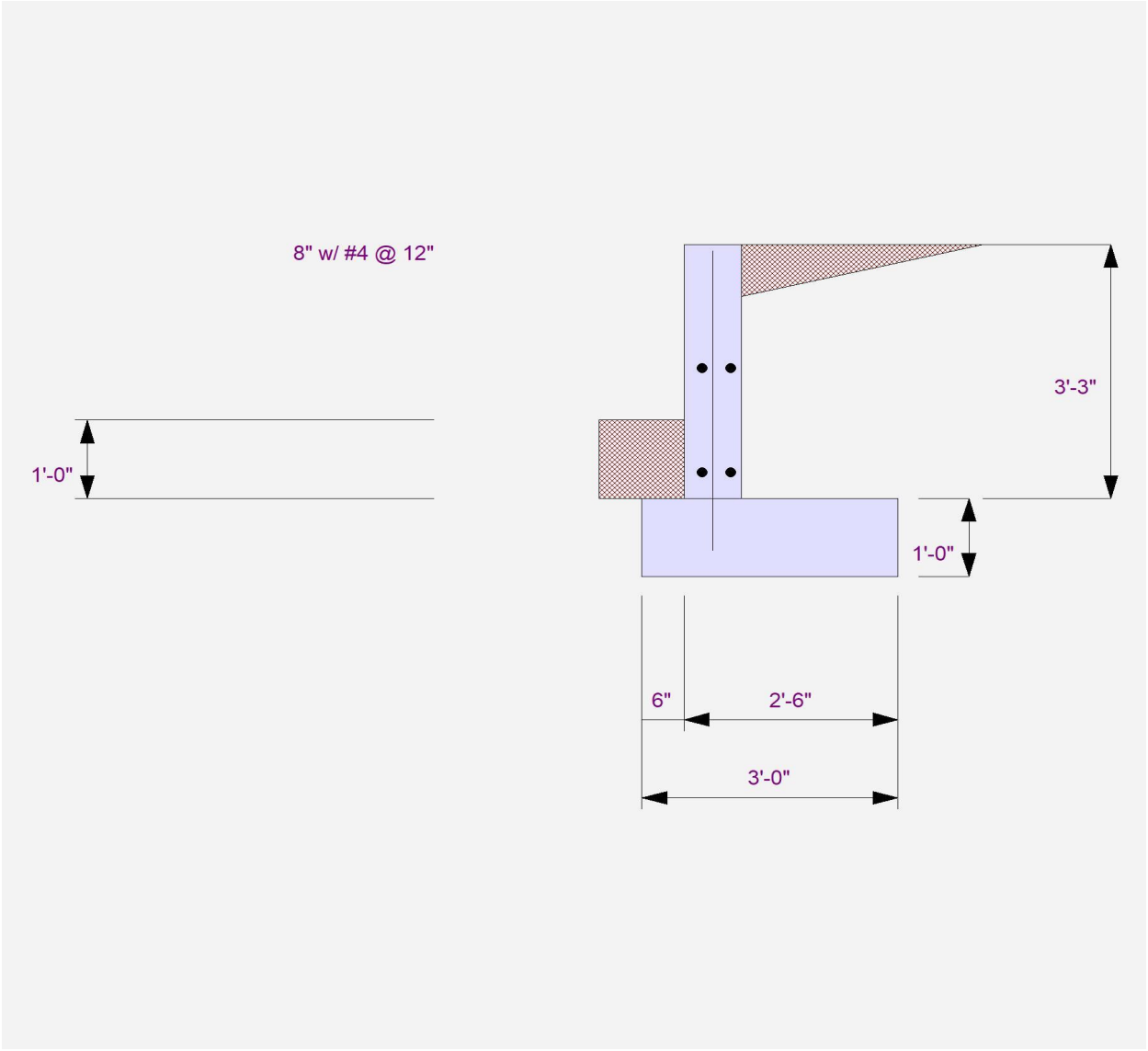
Cantilevered Retaining Wall

Project File: C and S Restaurant_29 Palms Hwy.ec6

DESCRIPTION: Wall Footing Line [1]B-D Prior SOG Construction

CBC 2022



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DESCRIPTION: Wall Footing Line [1]B-D Prior SOG Construction

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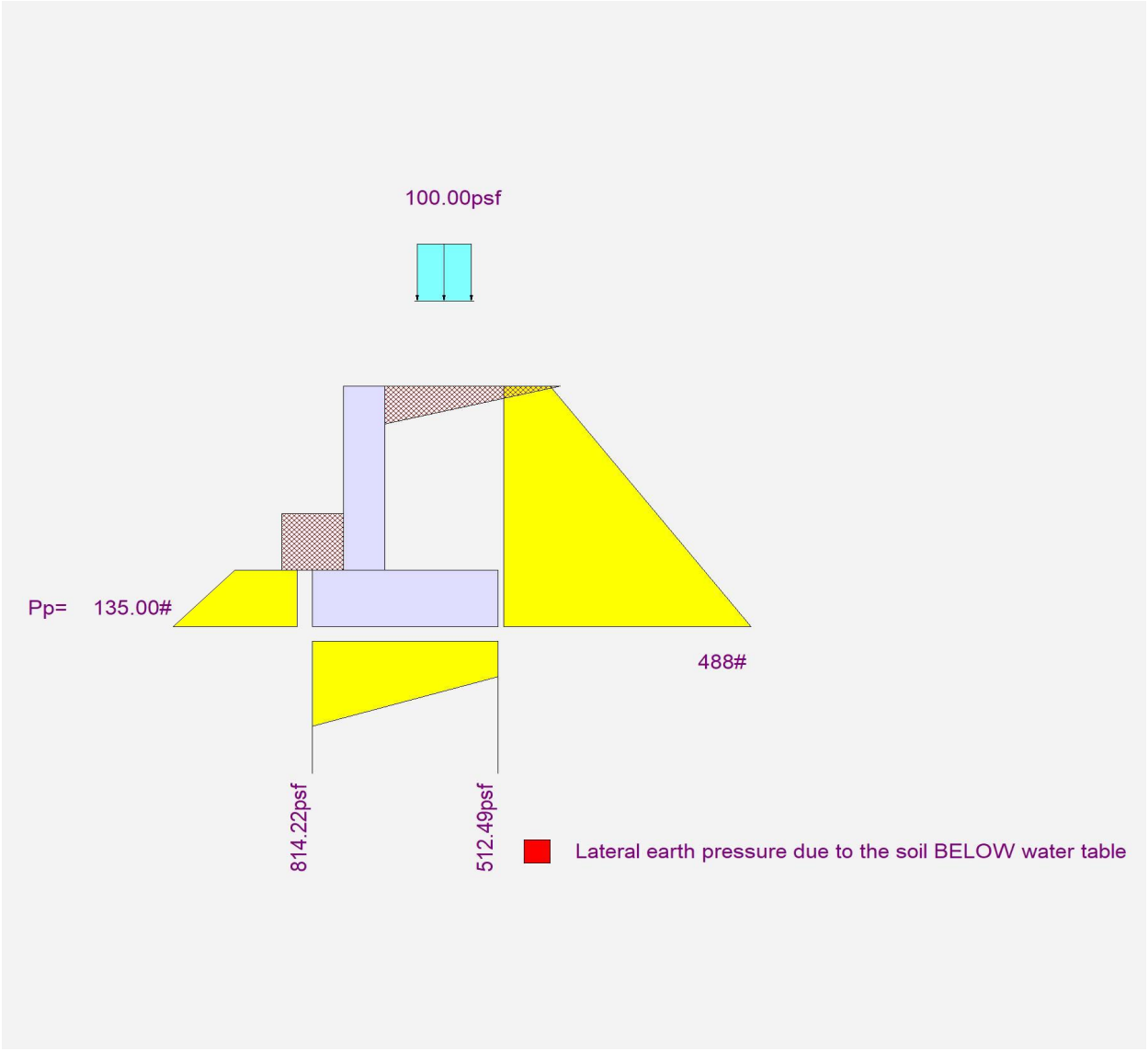


CASE - C

Wall Footing, Line A & Lines [1] & [3] Between [A] & [B]

Project Title: C & S Restaurant
Engineer: Mohammad Aljazzar P.E.
Project ID: CBE#: 25-304
Project Descr: Commercial Bldg.

Restrained Retaining Wall

Project File: C and S Restaurant_29 Palms Hwy.ec6

DESCRIPTION: Wall Footing Line [1]A-B Post SOG Construction

CBC 2022

Code References

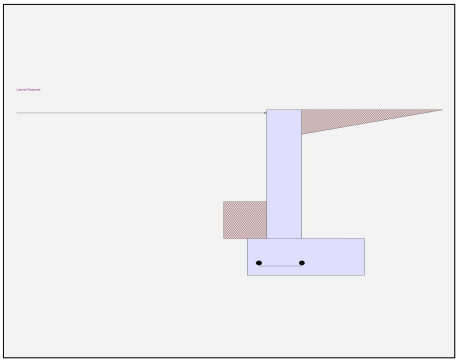
Calculations per IBC 2021, ACI 318-14

Criteria

Retained Height	=	3.50 ft
Wall height above soil	=	ft
Total Wall Height	=	3.50 ft
Top Support Height	=	3.250 ft
Slope Behind Wall	=	0
Height of Soil over Toe	=	12.0 in

Soil Data

Allow Soil Bearing	=	2,400.0 psf
Equivalent Fluid Pressure Method		
At-Rest Heel Pressure	=	60.0 psf/ft
	=	0.0 psf/ft
Passive Pressure	=	300.0 psf/ft
Soil Density	=	134.0 pcf
Footing || Soil Frictic	=	0.30
Soil height to ignore for passive pressure	=	12 in



Surcharge Loads

Surcharge Over Heel = 200.0 psf
>>>Used To Resist Sliding & Overturning
Surcharge Over Toe = psf
Used for Sliding & Overturning

Axial Load Applied to Stem

Axial Dead Load	=	480.0 lbs
Axial Live Load	=	275.0 lbs
Axial Load Eccentricity	=	4.0 in

Earth Pressure Seismic Load

Uniform Lateral Load Applied to Stem

Lateral Load	=	#/ft
...Height to Top	=	ft
...Height to Bottom	=	ft
Load Type	=	Wind (W) (Service Level)
Wind on Exposed Stem	=	0.00 psf (Strength Level)

Wind acts left-to-right toward retention side.

Adjacent Footing Load

Adjacent Footing Load	=	lbs
Footing Width	=	ft
Eccentricity	=	in
Wall to Ftg CL Dist	=	ft
Footing Type		Line Load
Base Above/Below Soil at Back of Wall	=	ft
Poisson's Ratio	=	0.3

K_h Soil Density Multiplier = 0.2 g Added seismic per unit area = 0.0 psf

Design Summary

Total Bearing Load	=	2,439.0 lbs	
...resultant ecc.	=	0.0 in	
Soil Pressure @ Toe	=	975.60 psf	OK
Soil Pressure @ Heel	=	975.60 psf	OK
Allowable	=	psf	
Soil Pressure Less Than Allowable			
ACI Factored @ Toe	=	1,214.72 psf	
ACI Factored @ Heel	=	1,214.72 psf	
Footing Shear @ Toe	=	0.7438 psi	OK
Footing Shear @ Heel	=	1.688 psi	OK
Allowable	=	82.158 psi	
Reaction at Top	=	331.377 lbs	
Reaction at Bottom	=	677.91 lbs	

Sliding Calcs
Lateral Sliding Force = 677.91 lbs

Vertical component of active lateral soil pressure IS NOT considered in the calculation of soil bearing pressures.

Load Factors

Building Code	
Dead Load	1.200
Live Load	1.600
Earth, H	1.600
Wind, W	1.000
Seismic, E	1.000

Concrete Stem Construction

Thickness = 8.00 in Fy = 60000 psi
Wall Weight = 100.0 psf f'c = 3000 psi
Stem is FIXED to top of footing

	@ Top Support	Mmax Between Top & Base	@ Base of Wall
	Stem OK	Stem OK	Stem OK
Design Height Above Ftg =	3.250 ft	1.436 ft	0.00 ft
Rebar Size =	# 4	# 4	# 4
Rebar Spacing =	12.00 in	12.00 in	12.00 in
Rebar Placed at =	Center	Center	Center
Rebar Depth 'd' =	4.0 in	4.0 in	4.0 in
Design Data			
fb/FB + fa/Fa =	0.100	0.036	0.079
Mu....Actual =	343.394 ft-#	124.493 ft-#	268.874 ft-#
Mn * Phi.....Allowable =	3,423.0 ft-#	3,423.0 ft-#	3,423.0 ft-#
Shear Force @ this height =	465.635 lbs		586.91 lbs
Shear.....Actual =	9.701 psi		12.227 psi
Shear.....Allowable =	52.882 psi		52.882 psi

Project Title: C & S Restaurant
Engineer: Mohammad Aljazzar P.E.
Project ID: CBE#: 25-304
Project Descr: Commercial Bldg.

Restrained Retaining Wall

Project File: C and S Restaurant_29 Palms Hwy.ec6

DESCRIPTION: Wall Footing Line [1]A-B Post SOG Construction

CBC 2022

Footing Strengths & Dimensions

Toe Width	=	0.50 ft
Heel Width	=	2.0
Total Footing Width	=	2.50
Footing Thickness	=	12.0 in

f'c = 3,000 psi Fy = 60000 psi
Footing Concrete Density = 150 pcf
Min. As % = 0.0018
Cover @ Top = 2 in @ Btm.= 3 in

Footing Design Results

		Toe	Heel	
Factored Pressure	=	1,214.72	1,214.72	psf
Mu' : Upward	=	151.840		ft-#
Mu' : Downward	=	42.60		ft-#
Mu: Design	=	109	-135	ft-#
Actual 1-Way Shear	=	0.7438	1.688	psi
Allow 1-Way Shear	=	43.818	43.818	psi

Other Acceptable Sizes & Spacings:

Toe: # 4 @ 18.00 in -or- phiMn = phi * 5 * lambda * sqrt(fc) * Sm
Heel: None Spec'd -or- phiMn = phi * 5 * lambda * sqrt(fc) * Sm

Min footing T&S reinf Area 0.65 in2
Min footing T&S reinf Area per foot 0.26 in2 /ft

If one layer of horizontal bars:	If two layers of horizontal bars:
#4@ 9.26 in	#4@ 18.52 in
#5@ 14.35 in	#5@ 28.70 in
#6@ 20.37 in	#6@ 40.74 in

Summary of Forces on Footing : Slab is NOT providing sliding, stem is FIXED at footing

Forces acting on footing for sliding & soil pressure....

Sliding Forces

Stem Shear @ Top of Footing	=	348.355 lbs
Heel Active Pressure	=	329.552
Key Active Pressure	=	0.0
Sliding Force	**=**	**677.91 lbs**

Stem is specified to be fixed to footing, and top restraint is assumed to react out any tendency for moment at the footing/soil interface, so uniform soil pressure is assumed.

Load & Moment Summary For Footing : For Soil Pressure Calcs

Moment @ Top of Footing Applied from Stem			=	-148.046 ft-#
Surcharge Over Heel		266.667	1.833	488.889
Adjacent Footing Load	=	0.0 lbs	0.0 ft	0.0 ft-#
Axial Dead Load on Stem	=	480.0 lbs	0.50 ft	240.0 ft-#
Axial Live Load on Stem	=	275.0 lbs	0.50 ft	137.50 ft-#
Soil Over Toe	=	67.0 lbs	0.250 ft	16.750 ft-#
Surcharge Over Toe	=	0.0 lbs	0.0 ft	0.0 ft-#
Stem Weight	=	350.0 lbs	0.8333 ft	291.667 ft-#
Soil Over Heel	=	625.33 lbs	1.833 ft	1,146.44 ft-#
Footing Weight	=	375.0 lbs	1.250 ft	468.750 ft-#
Total Vertical Force	=	2,439.0 lbs	Base Moment =	2,641.95 ft-#

Summary of Sliding Forces

	FS = 1.0	FS = 1.5
Lateral Force @ Base of Footing	677.91 lbs	1,016.86 lbs
less Passive Pressure Force	- 450.0 lbs	- 450.0 lbs
less Friction Force	- 649.20 lbs	- 649.20 lbs
Added Resisting Force Required	0.0 lbs	
Added Resisting Force Required for 1.5 Factor of Safety		0.00 lbs

Sliding Factor of Safety = 1.743: 1.00

Vertical component of active lateral soil pressure IS NOT considered in the calculation of Sliding Resistance.

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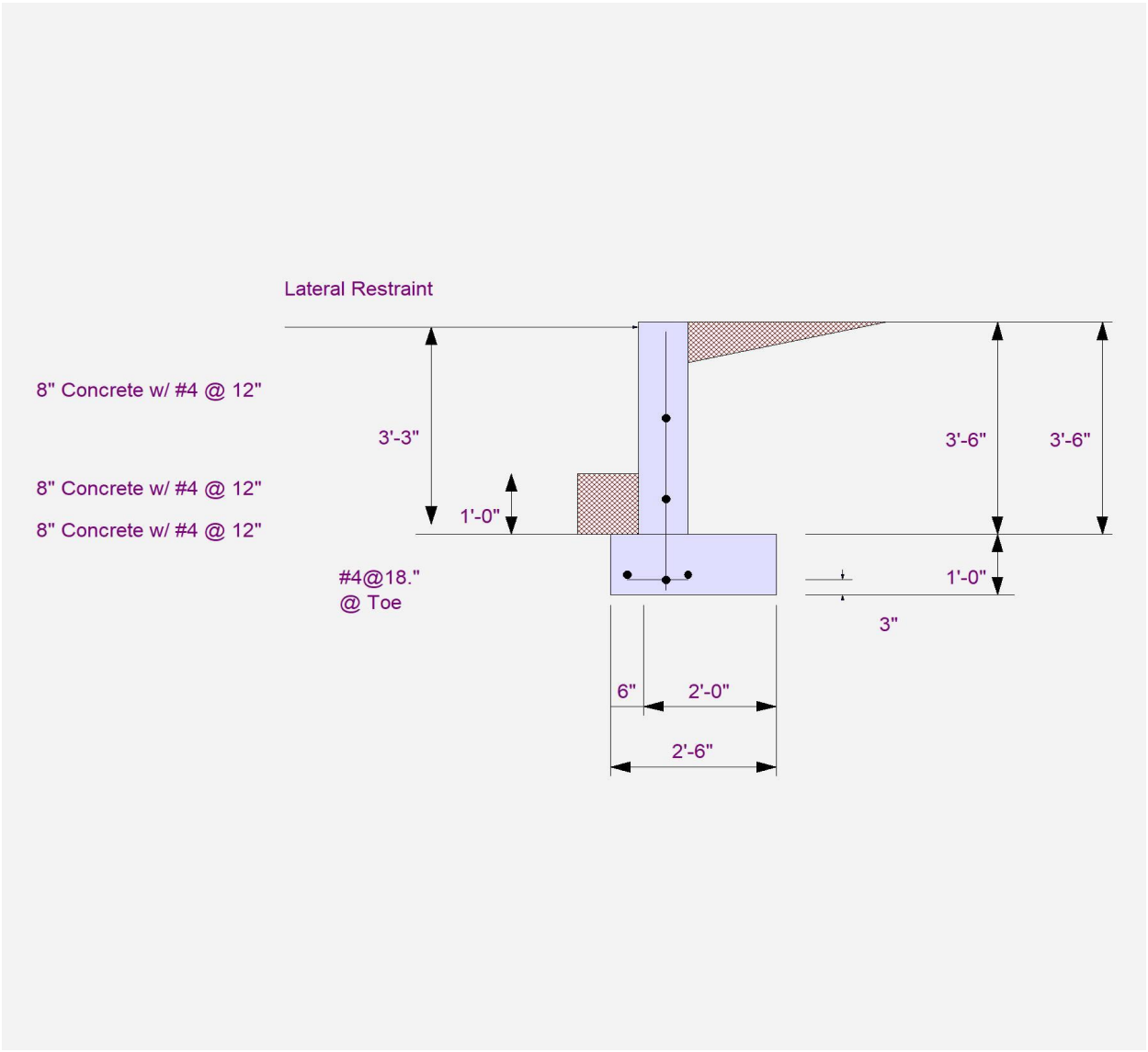
DESCRIPTION: Wall Footing Line [1]A-B Post SOG Construction

CBC 2022



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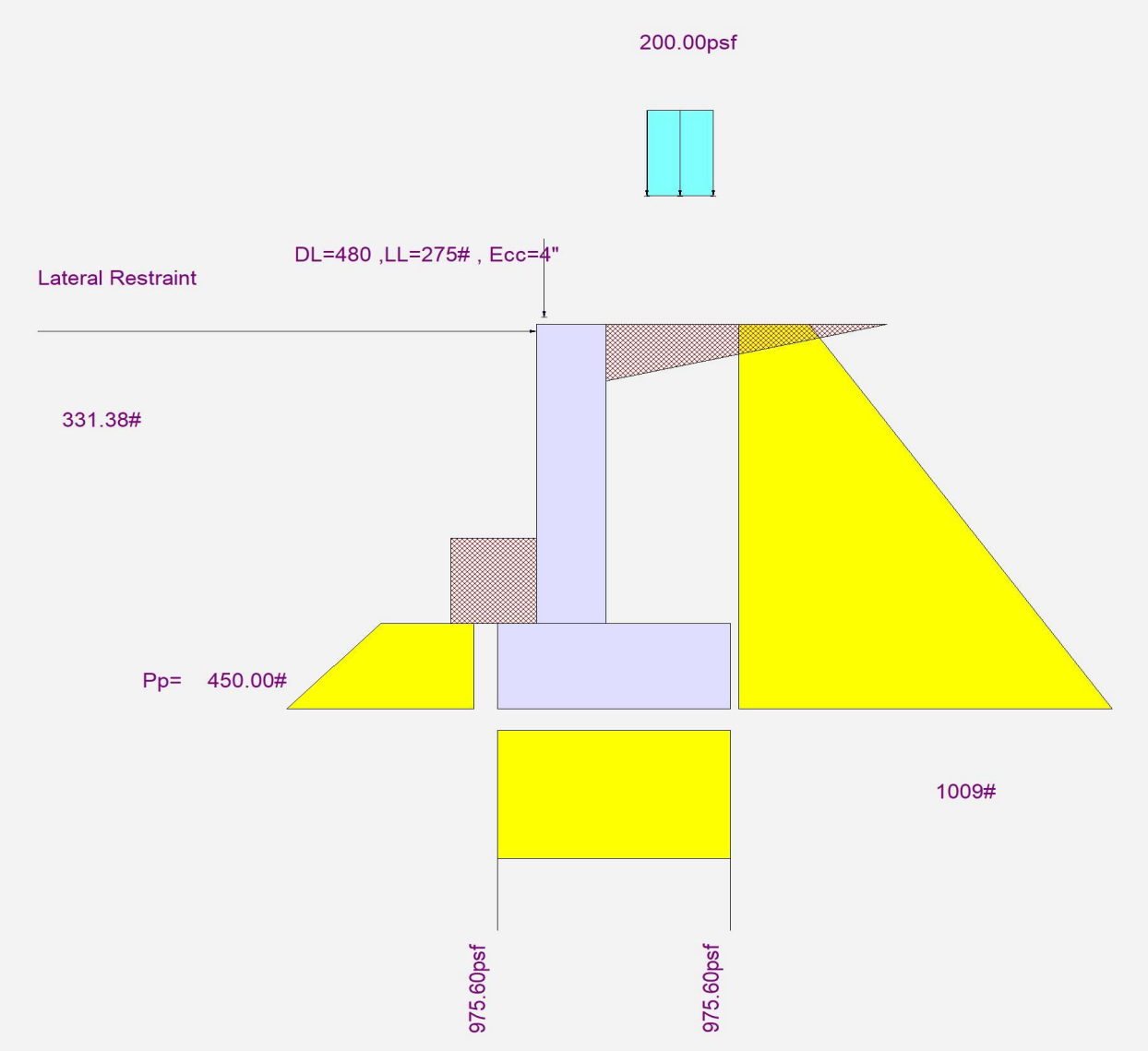

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DESCRIPTION: Wall Footing Line [1]A-B Prior SOG Construction

CBC 2022

Code Reference

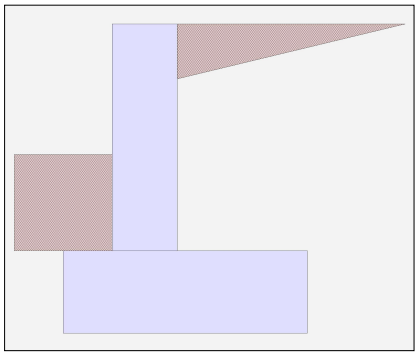
Calculations per IBC 2021, ACI 318-19, TMS 402-16

Criteria

Retained Height	=	2.75 ft
Wall height above soil	=	0.00 ft
Slope Behind Wall	=	0.00
Height of Soil over Toe	=	14.00 in
Water table above bottom of footing	=	0.0 ft

Soil Data

Allow Soil Bearing	=	2,400.0 psf
Equivalent Fluid Pressure Method		
Active Heel Pressure	=	40.0 psf/ft
	=	
Passive Pressure	=	300.0 psf/ft
Soil Density, Heel	=	134.00 pcf
Soil Density, Toe	=	110.00 pcf
Footing||Soil Friction	=	0.300
Soil height to ignore for passive pressure	=	12.00 in



Surcharge Loads

Surcharge Over Heel	=	100.0 psf
Used To Resist Sliding & Overturning		
Surcharge Over Toe	=	0.0
Used for Sliding & Overturning		

Lateral Load Applied to Stem

Lateral Load	=	0.0 #/ft
...Height to Top	=	0.00 ft
...Height to Bottom	=	0.00 ft
Load Type	=	Wind (W) (Service Level)
Wind on Exposed Stem (Strength Level)	=	0.0 psf

Adjacent Footing Load

Adjacent Footing Load	=	0.0 lbs
Footing Width	=	0.00 ft
Eccentricity	=	0.00 in
Wall to Ftg CL Dist	=	0.00 ft
Footing Type		Line Load
Base Above/Below Soil at Back of Wall	=	0.0 ft
Poisson's Ratio	=	0.300

Axial Load Applied to Stem

Axial Dead Load	=	0.0 lbs
Axial Live Load	=	0.0 lbs
Axial Load Eccentricity	=	0.0 in

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DESCRIPTION: Wall Footing Line [1]A-B Prior SOG Construction

CBC 2022

Design Summary

Wall Stability Ratios

Overturning	=	3.93	OK
Sliding	=	1.55	OK
Global Stability	=	4.62	

Total Bearing Load	=	1,478	lbs
...resultant ecc.	=	1.64	in

Eccentricity within middle third

Soil Pressure @ Toe	=	785 psf	OK
Soil Pressure @ Heel	=	397 psf	OK
Allowable	=	2,400 psf	

Soil Pressure Less Than Allowable

ACI Factored @ Toe	=	996 psf	
ACI Factored @ Heel	=	503 psf	
Footing Shear @ Toe	=	0.7 psi	OK
Footing Shear @ Heel	=	3.5 psi	OK
Allowable	=	82.2 psi	

Sliding Calcs

Lateral Sliding Force	=	393.2 lbs

Vertical component of active lateral soil pressure IS considered in the calculation of soil bearing pressures.

Load Factors

Building Code	
Dead Load	1.200
Live Load	1.600
Earth, H	1.600
Wind, W	1.600
Seismic, E	1.000

Stem Construction

		Bottom		
		Stem OK		
Design Height Above Ftg	ft =	0.00		
Wall Material Above "Ht"	=	Concrete		
Design Method	=	SD	SD	SD
Thickness	=	8.00		
Rebar Size	=	# 4		
Rebar Spacing	=	12.00		
Rebar Placed at	=	Center		
Design Data				
fb/FB + fa/Fa	=	**0.117**		
Total Force @ Section				
Service Level	lbs =			
Strength Level	lbs =	373.3		
Moment....Actual				
Service Level	ft-# =			
Strength Level	ft-# =	402.4		
Moment.....Allowable	=	3,423.0		
Shear.....Actual				
Service Level	psi =			
Strength Level	psi =	7.8		
Shear.....Allowable	psi =	52.9		
Anet (Masonry)	in2 =			
Wall Weight	psf =	100.0		
Rebar Depth 'd'	in =	4.00		

Masonry Data

f'm	psi =	
Fs	psi =	
Solid Grouting	=	
Modular Ratio 'n'	=	
Equiv. Solid Thick.	=	
Masonry Block Type	=	
Masonry Design Method	=	ASD

Concrete Data

f'c	psi =	3,000.0
Fy	psi =	60,000.0

Summary of Sliding Forces

	FS = 1.0	FS = 1.5
Lateral Force @ Base of Footing	393.19 lbs	589.79 lbs
less 30% Passive Force	- 166.250 lbs	- 166.250 lbs
less 100% Friction Force	- 443.29 lbs	- 443.29 lbs
Added Resisting Force Required	0.0 lbs	
Added Resisting Force Required for 1.5 Factor of Safety		0.00 lbs

Sliding Factor of Safety = 1.550: 1.00

Project Title: C & S Restaurant
Engineer: Mohammad Aljazzar P.E.
Project ID: CBE#: 25-304
Project Descr: Commercial Bldg.

Cantilevered Retaining Wall

Project File: C and S Restaurant_29 Palms Hwy.ec6

DESCRIPTION: Wall Footing Line [1]A-B Prior SOG Construction

CBC 2022

Concrete Stem Rebar Area Details

Bottom Stem	Vertical Reinforcing	Horizontal Reinforcing	
As (based on applied moment) :	0.0243 in2/ft		
0.0018bh : 0.0018(12)(8) :	0.1728 in2/ft	Horizontal Reinforcing Options :	
	==========	One layer of :	Two layers of :
Required Area :	0.1728 in2/ft	#4@ 13.89 in	#4@ 27.78 in
Provided Area :	0.2 in2/ft	#5@ 21.53 in	#5@ 43.06 in
Maximum Area :	0.6503 in2/ft	#6@ 30.56 in	#6@ 61.11 in

Footing Data

Toe Width	=	0.50 ft
Heel Width	=	2.00
Total Footing Width	=	2.50
Footing Thickness	=	12.00 in

f'c = 3,000 psi Fy = 60,000 psi
Footing Concrete Density = 150.00 pcf
Min. As % = 0.0018
Cover @ Top 2.00 @ Btm.= 3.00 in

Footing Design Results

		Toe	Heel	
Factored Pressure	=	996	503	psf
Mu' : Upward	=	120	525	ft-#
Mu' : Downward	=	46	991	ft-#
Mu: Design	=	74	466	ft-#
φ Mn	=	2,739	2,739	ft-#
Actual 1-Way Shear	=	0.75	3.49	psi
Allow 1-Way Shear	=	43.82	43.82	psi

Toe Reinforcing = None Spec'd
Heel Reinforcing = None Spec'd
Key Reinforcing = None Spec'd
Footing Torsion, Tu = 0.00 ft-lbs
Footing Allow. Torsion, φ Tn = 0.00 ft-lbs

If torsion exceeds allowable, provide supplemental design for footing torsion.

Other Acceptable Sizes & Spacings

Toe: phiMn = phi*5*lambda*sqrt(fc)*Sm

Heel: phiMn = phi*5*lambda*sqrt(fc)*Sm

Key: No key defined

Min footing T&S reinf Area 0.65 in2
Min footing T&S reinf Area per foot 0.26 in2 /ft

If one layer of horizontal bars:	If two layers of horizontal bars:
#4@ 9.26 in	#4@ 18.52 in
#5@ 14.35 in	#5@ 28.70 in
#6@ 20.37 in	#6@ 40.74 in

Project Title: C & S Restaurant
Engineer: Mohammad Aljazzar P.E.
Project ID: CBE#: 25-304
Project Descr: Commercial Bldg.

Cantilevered Retaining Wall

Project File: C and S Restaurant_29 Palms Hwy.ec6

DESCRIPTION: Wall Footing Line [1]A-B Prior SOG Construction

CBC 2022

Summary of Overturning & Resisting Forces & Moments

Item		.....OVERTURNING..... Force lbs	Distance ft	Moment ft-#		.....RESISTING..... Force lbs	Distance ft	Moment ft-#
HL Act Pres (ab water tbl)		281.3	1.25	351.6	Soil Over HL (ab. water tbl)	491.3	1.83	900.8
HL Act Pres (be water tbl)					Soil Over HL (bel. water tbl)		1.83	900.8
Hydrostatic Force					Water Table			
Buoyant Force	=				Sloped Soil Over Heel	=		
Surcharge over Heel	=	111.9	1.88	209.9	Surcharge Over Heel	= 133.3	1.83	244.4
Surcharge Over Toe	=				Adjacent Footing Load	=		
Adjacent Footing Load	=				Axial Dead Load on Stem	=		
Added Lateral Load	=				* Axial Live Load on Stem	=		
Load @ Stem Above Soil	=				Soil Over Toe	= 64.2	0.25	16.0
	=				Surcharge Over Toe	=		
					Stem Weight(s)	= 275.0	0.83	229.2
					Earth @ Stem Transitions	=		
					Footing Weight	= 375.0	1.25	468.8
					Key Weight	=		
					Vert. Component	= 138.8	2.50	347.0
Total	=	393.2	**O.T.M.** =	561.5	**Total =**	1,477.6 lbs	**R.M.=**	2,206.2

Resisting/Overturning Ratio = **3.93**
Vertical Loads used for Soil Pressure = 1,477.6 lbs

* Axial live load NOT included in total displayed, or used for overturning resistance, but is included for soil pressure calculation.

Vertical component of active lateral soil pressure IS considered in the calculation of Sliding Resistance.

Vertical component of active lateral soil pressure IS considered in the calculation of Overturning Resistance.

Tilt

Horizontal Deflection at Top of Wall due to settlement of soil

(Deflection due to wall bending not considered)

Soil Spring Reaction Modulus	250.0	pci
Horizontal Defl @ Top of Wall (approximate only)	0.024	in

The above calculation is not valid if the heel soil bearing pressure exceeds that of the toe, because the wall would then tend to rotate into the retained soil.

Cantilevered Retaining Wall

DESCRIPTION: Wall Footing Line [1]A-B Prior SOG Construction

CBC 2022

Rebar Lap & Embedment Lengths Information

Stem Design Segment: Bottom

Stem Design Height: 0.00 ft above top of footing

Lap Splice length for #4 bar specified in this stem design segment (25.4.2.4a) =	17.09 in
Development length for #4 bar specified in this stem design segment =	13.15 in
Hooked embedment length into footing for #4 bar specified in this stem design segment =	6.00 in
As Provided =	0.2000 in2/ft
As Required =	0.1728 in2/ft

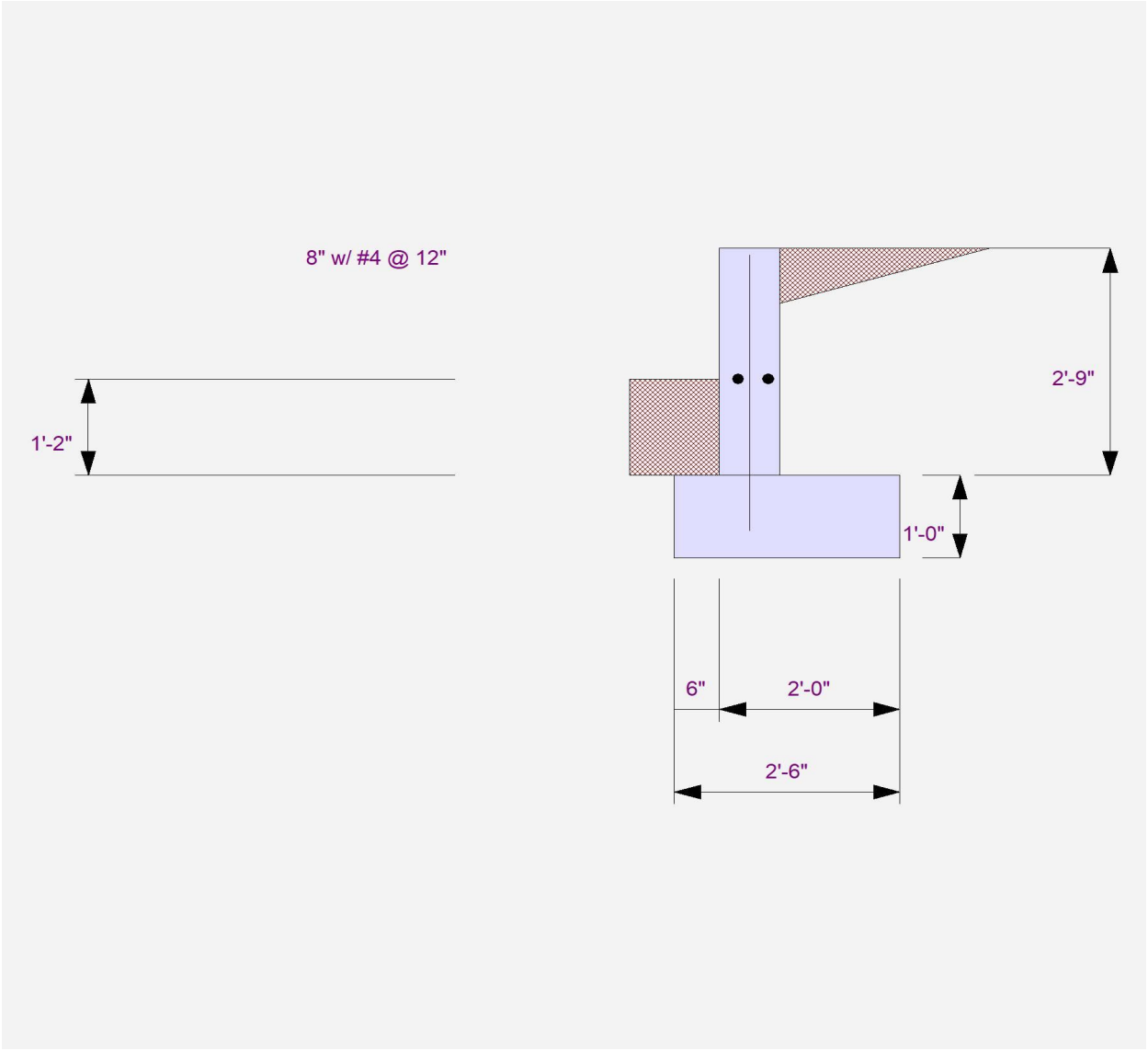
Project Title: C & S Restaurant
Engineer: Mohammad Aljazzar P.E.
Project ID: CBE#: 25-304
Project Descr: Commercial Bldg.

Cantilevered Retaining Wall

Project File: C and S Restaurant_29 Palms Hwy.ec6

DESCRIPTION: Wall Footing Line [1]A-B Prior SOG Construction

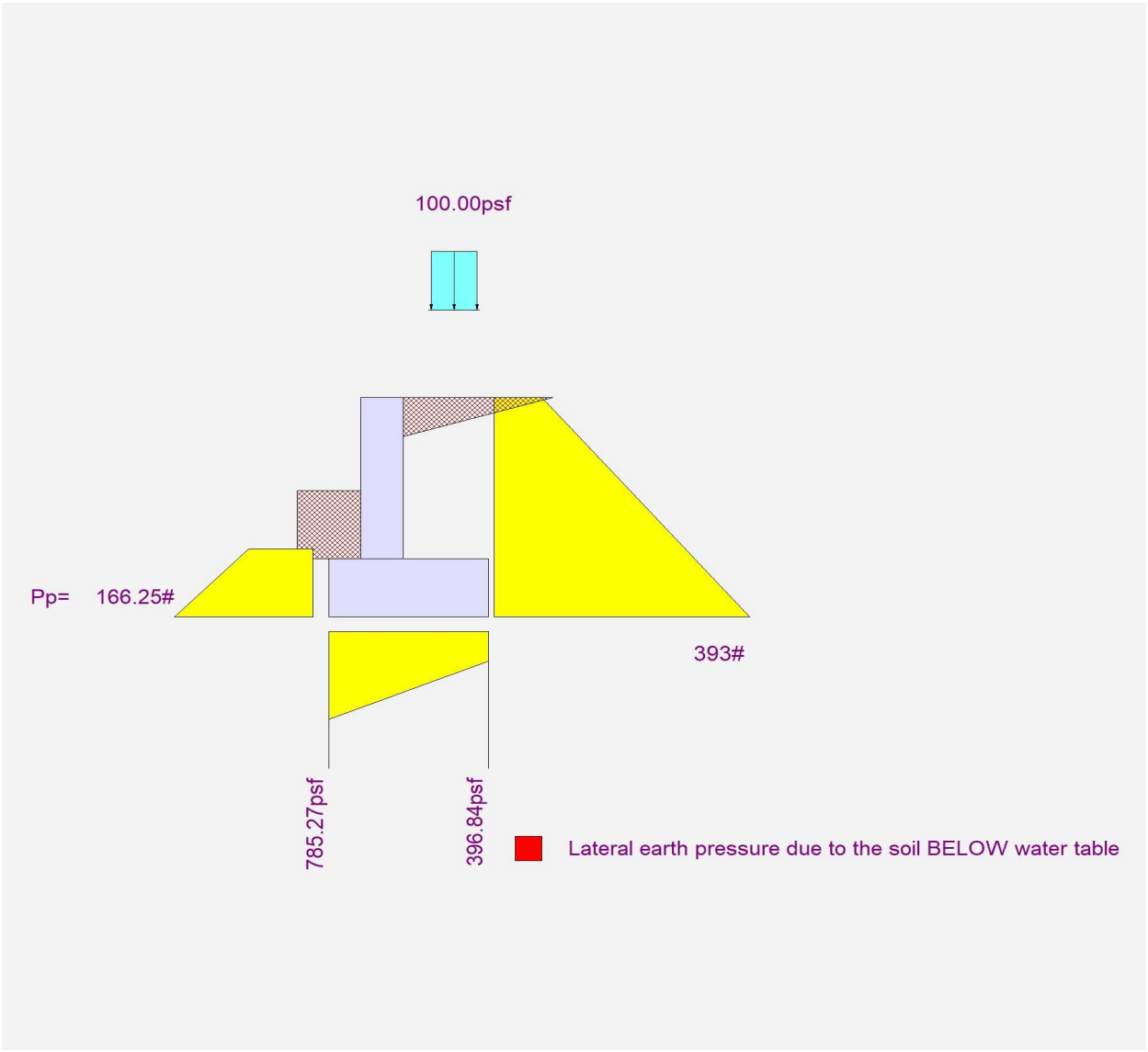
CBC 2022



Cantilevered Retaining Wall

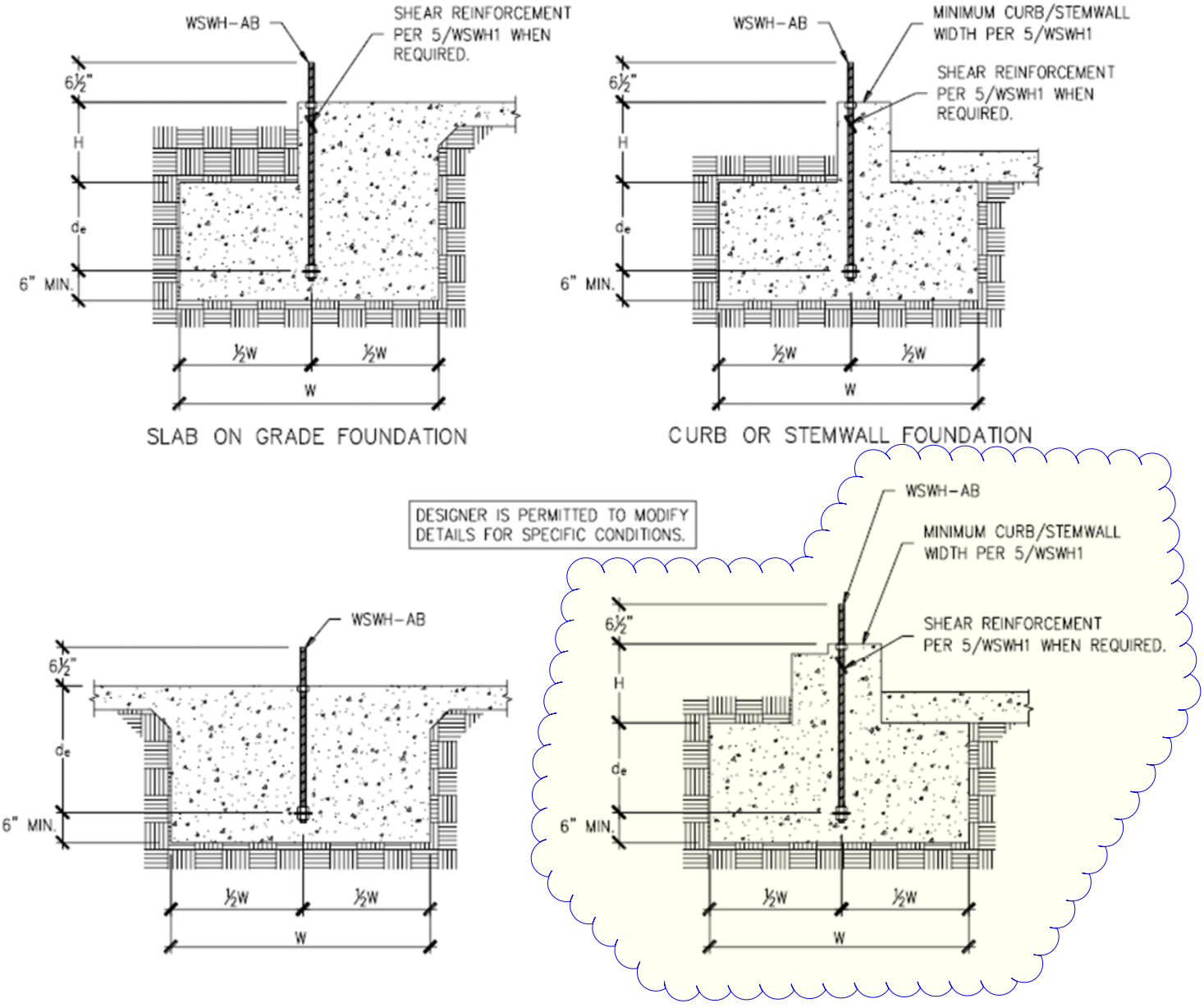
DESCRIPTION: Wall Footing Line [1]A-B Prior SOG Construction

CBC 2022



F.3 Strong Wall WSWH Footing Design

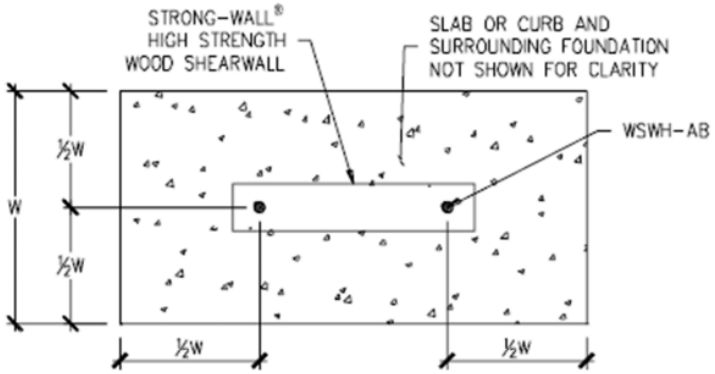
Line [A] Strong Wall Foundations



NOTES :
1. SEE 2/WSWH1 FOR DIMENSIONS AND ADDITIONAL NOTES.
2. SEE 5/WSWH1 FOR SHEAR REINFORCEMENT WHEN REQUIRED.
3. MAXIMUM H = $l_e - d_e$. SEE 3/WSWH1 AND 4/WSWH1 FOR l_e.

1/WSWH1 – STRONG-WALL HIGH STRENGTH WOOD SHEARWALL ANCHORAGE – TYPICAL SECTIONS

FIGURE 6 – STRONG-WALL HIGH STRENGTH WOOD SHEARWALL ANCHORAGE DETAILS (1/WSWH1)



FOUNDATION PLAN VIEW

WSWH ANCHORAGE SOLUTIONS FOR 3000 PSI CONCRETE

DESIGN CRITERIA	CONCRETE CONDITION	ANCHOR STRENGTH	WSWH-AB1 ANCHOR BOLT		
			ASD ALLOWABLE UPLIFT (lbs)	W (in)	d_e (in)
SEISMIC	CRACKED	STANDARD	16,000	31	11
			17,100	33	11
		HIGH STRENGTH	33,900	49	17
			36,800	52	18
	UNCRACKED	STANDARD	16,300	27	9
			17,100	28	10
		HIGH STRENGTH	34,000	43	15
			36,800	46	16
WIND	CRACKED	STANDARD	5,600	14	6
			10,200	21	7
			17,100	30	10
		HIGH STRENGTH	20,000	33	11
			26,500	39	13
			33,600	45	15
			36,800	48	16
	UNCRACKED	STANDARD	6,200	13	6
			12,800	21	7
			17,100	26	9
		HIGH STRENGTH	21,800	30	10
			28,900	36	12
			33,100	39	13
			36,800	42	14

NOTES :
1. ANCHORAGE DESIGNS CONFORM TO ACI 318-11 APPENDIX D, ACI 318-14 CHAPTER 17 AND ACI 318-19 CHAPTER 17 WITH NO SUPPLEMENTARY REINFORCEMENT FOR CRACKED OR UNCRACKED CONCRETE AS NOTED.
2. ANCHOR STRENGTH INDICATES REQUIRED GRADE OF WSWH-AB ANCHOR BOLT. STANDARD (ASTM F1554 GRADE 36) OR HIGH STRENGTH (HS) (ASTM A193 GRADE B7).
3. SEISMIC INDICATES SEISMIC DESIGN CATEGORY C-F. DETACHED 1 AND 2 FAMILY DWELLINGS IN SDC C MAY USE WIND ANCHORAGE SOLUTIONS. SEISMIC ANCHORAGE DESIGNS CONFORM TO ACI 318-11 SECTION D.3.3.4.3, ACI 318-14 SECTION 17.2.3.4.3 AND ACI 318-19 SECTION 17.10.5.3.
4. WIND INCLUDES SEISMIC DESIGN CATEGORY A AND B AND DETACHED 1 AND 2 FAMILY DWELLINGS IN SDC C.
5. FOUNDATION DIMENSIONS ARE FOR ANCHORAGE ONLY. FOUNDATION DESIGN (SIZE AND REINFORCEMENT) BY OTHERS. THE DESIGNER MAY SPECIFY ALTERNATE EMBEDMENT, FOOTING SIZE OR ANCHOR BOLT.
6. REFER TO 1/WSWH1 FOR d_e.

2/WSWH1 – STRONG-WALL HIGH STRENGTH WOOD SHEARWALL TENSION ANCHORAGE SCHEDULE

FIGURE 6 – STRONG-WALL HIGH STRENGTH WOOD SHEARWALL ANCHORAGE DETAILS (Continued) (2/WSWH1)

Line [A] WSWH Strong Walls:

Overturning:

Maximum Lateral Force [Line A] = 3,614 lbs ASD [0.7ρE ρ=1.3]

(1) Walls Loading = 1,807 lbs.

Overturning Moment:
M,ovt. = V x h
1,807 lbs x 9.33' = 16,860 ft-lbs at Base , h = 9.33' ht. of panel

Stabilizing/Resisting Moment:
Assuming weight of footing only, with no additional roof dead loads (conservatively)
M,res. = 0.9 (P,footing) x 1/2 L
= 0.9 (L x W x D x 150 pcf) x 1/2 L
= 0.9 (5.83 x 4.33 x D x 150 pcf) x 0.5 (5.83)
= 11,037 D,min.

16,860 < 11,037 Dmin. Dmin > 1.53 ft

D provided = 2'-0" (2.0 > 1.53 OK)

CBC-2022:

1605.2 Alternative Allowable Stress Design Load Combinations Diagram

In lieu of the load combinations in ASCE 7, Section 2.4, structures and portions thereof shall be permitted to be designed for the most critical effects resulting from the following combinations. Where using these alternative allowable stress load combinations that include wind or seismic loads, allowable stresses are permitted to be increased or load combinations reduced where permitted by the material chapter of this code or the referenced standards. For load combinations that include the counteracting effects of dead and wind loads, only two-thirds of the minimum dead load likely to be in place during a design wind event shall be used. Where using these alternative load combinations to evaluate sliding, overturning and soil bearing at the soil-structure interface, the reduction of foundation overturning from Section 12.13.4 in ASCE 7 shall not be used. Where using these alternative basic load combinations for proportioning foundations for loadings, which include seismic loads, the vertical seismic load effect, E_v, in Equation 12.4-4 of ASCE 7 is permitted to be taken equal to zero. Where required by ASCE 7, Chapters 12, 13 and 15, the load combinations including overstrength of ASCE 7, Section 2.3.6 shall be used. ***[OSHPD 1R, 2 & 5]*** *Each load combination shall be investigated with one or more of the variable loads set to zero.*

$D + L + (L_r \text{ or } S \text{ or } R)$ **(Equation 16-1)**

$D + L + 0.6W$ **(Equation 16-2)**

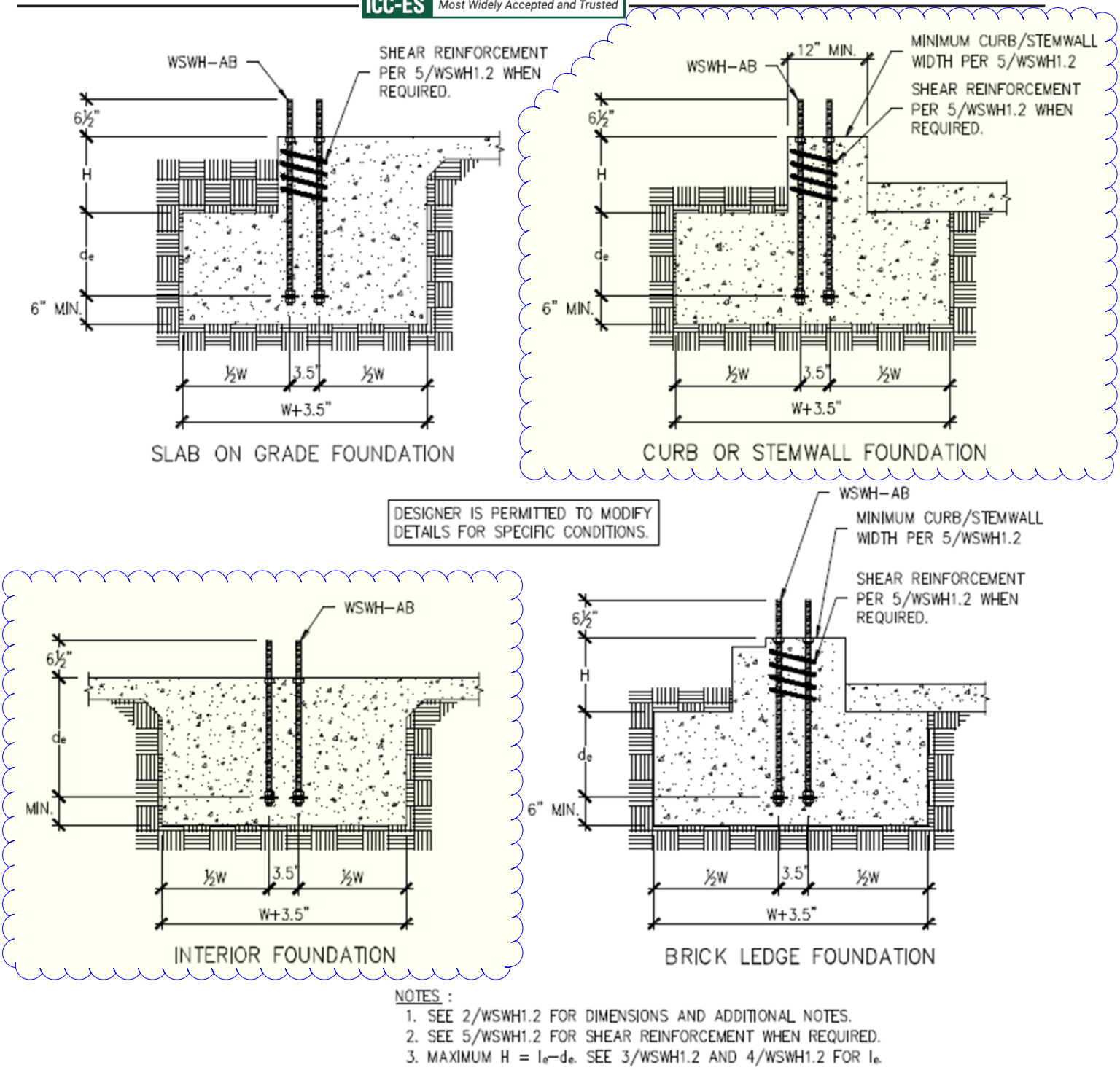
$D + L + 0.6W + S/2$ **(Equation 16-3)**

$D + L + S + 0.6W/2$ **(Equation 16-4)**

$D + L + S + E/1.4$ **(Equation 16-5)**

$0.9D + E/1.4$ **(Equation 16-6)**

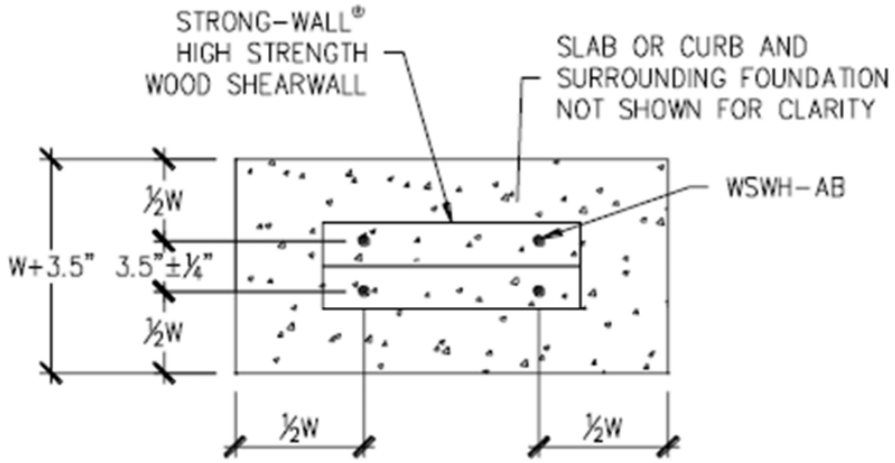
Lines [B] and [D]
Back to Back Strong Wall Foundation



NOTES :
1. SEE 2/WSWH1.2 FOR DIMENSIONS AND ADDITIONAL NOTES.
2. SEE 5/WSWH1.2 FOR SHEAR REINFORCEMENT WHEN REQUIRED.
3. MAXIMUM H = l_e–d_e. SEE 3/WSWH1.2 AND 4/WSWH1.2 FOR l_e.

1/WSWH1.2 – STRONG-WALL WSWH BACK-TO-BACK ANCHORAGE – TYPICAL SECTIONS

FIGURE 7 – STRONG-WALL HIGH STRENGTH WOOD SHEARWALL BACK-TO-BACK ANCHORAGE DETAILS (1/WSWH1.2)



FOUNDATION PLAN VIEW

WSWH BACK-TO-BACK ANCHORAGE SOLUTIONS FOR 3000 PSI CONCRETE

DESIGN CRITERIA	CONCRETE CONDITION	ANCHOR STRENGTH	WSWH-AB1 ANCHOR BOLT		
			ASD ALLOWABLE UPLIFT (lbs)	W (in)	d_e (in)
SEISMIC	CRACKED	STANDARD	34,200	48	16
		HIGH STRENGTH	73,600	76	26
	UNCRACKED	STANDARD	34,200	42	14
		HIGH STRENGTH	73,600	67	23
WIND	CRACKED	STANDARD	9700	18	6
			19,500	30	10
			34,200	44	15
		HIGH STRENGTH	48,500	54	18
			57,400	60	20
			66,900	66	22
			73,600	70	24
	UNCRACKED	STANDARD	12,100	18	6
			24,400	30	10
			34,200	38	13
		HIGH STRENGTH	45,200	45	15
			60,600	54	18
			71,800	60	20
			73,600	61	21

NOTES :

1. ANCHORAGE DESIGNS CONFORM TO ACI 318-11 APPENDIX D, ACI 318-14 CHAPTER 17 AND ACI 318-19 CHAPTER 17 WITH NO SUPPLEMENTARY REINFORCEMENT FOR CRACKED OR UNCRACKED CONCRETE AS NOTED.
2. ANCHOR STRENGTH INDICATES REQUIRED GRADE OF WSWH-AB ANCHOR BOLT. STANDARD (ASTM F1554 GRADE 36) OR HIGH STRENGTH (HS) (ASTM A193 GRADE B7).
3. SEISMIC INDICATES SEISMIC DESIGN CATEGORY C-F. DETACHED 1 AND 2 FAMILY DWELLINGS IN SDC C MAY USE WIND ANCHORAGE SOLUTIONS. SEISMIC ANCHORAGE DESIGNS CONFORM TO ACI 318-11 SECTION D.3.3.4.3, ACI 318-14 SECTION 17.2.3.4.3, AND ACI 318-19 SECTION 17.10.5.3.
4. WIND INCLUDES SEISMIC DESIGN CATEGORY A AND B AND DETACHED 1 AND 2 FAMILY DWELLINGS IN SDC C.
5. SOLUTIONS ASSUME THAT BACK-TO-BACK PANEL ARE IN CONTACT WITH EACH OTHER.
6. FOUNDATION DIMENSIONS ARE FOR ANCHORAGE ONLY. FOUNDATION DESIGN (SIZE AND REINFORCEMENT) BY OTHERS. THE DESIGNER MAY SPECIFY ALTERNATE EMBEDMENT, FOOTING SIZE OR ANCHOR BOLT.
7. REFER TO 1/WSWH1.2 FOR d_e.

2/WSWH1.2 – STRONG-WALL WSWH BACK-TO-BACK ANCHORAGE TENSION ANCHORAGE SCHEDULE

FIGURE 7 – STRONG-WALL HIGH STRENGTH WOOD SHEARWALL BACK-TO-BACK ANCHORAGE DETAILS (Continued) (2/WSWH1.2)

Tension in Anchor Bolts:
Back to Back WSWH24x10

At ASD Wall Demand;

Line [D] = 12,000 lbs. Total Line ASD Demand

Per pair [Each End] = 6,000 lbs ASD [0.7ρE, ρ=1.3]

Movt, Demand: (1) pair Back to Back
Movt = 6,000 x 9.33 ft = 56,000 ft-lbs Demand ASD
Movt = 56,000 / 0.7 = 80,000 ft-lbs Demand LRFD

T = [V x H / B] - P/2
B = 17.5" for WSWH24 walls

ASD Tension: [0.7pE, p=1.3]
T = 56,000 x 12" / 17.5"
= 38,400 lbs. Per Pair
= 19,200 lbs Per Wall Segment

LRFD Tension: [1.0pE, p=1.3]
T = 38,400 / 0.7
= 54,860 lbs Per Pair
= 27,430 lbs Per Wall Segment

At ASD Wall Capacity: per Table 2 ESR-2652;

ASD [0.7 E]
T = 4010 lbs x 117.5"/ 17.5"
= 26,860 lbs Per Wall Segment
= 53,720 lbs Per Pair

Movt. Back to Back; (for bearing design)
M = T x B = (53,720) (17.5) /12
= 78,400 ft-lbs ASD
See Stability Calculations following page.

LRFD [1.0 E];
T = 26,860 / 0.7
= 38,400 lbs. Per Wall Segment
= 76,800 lbs Per Pair.

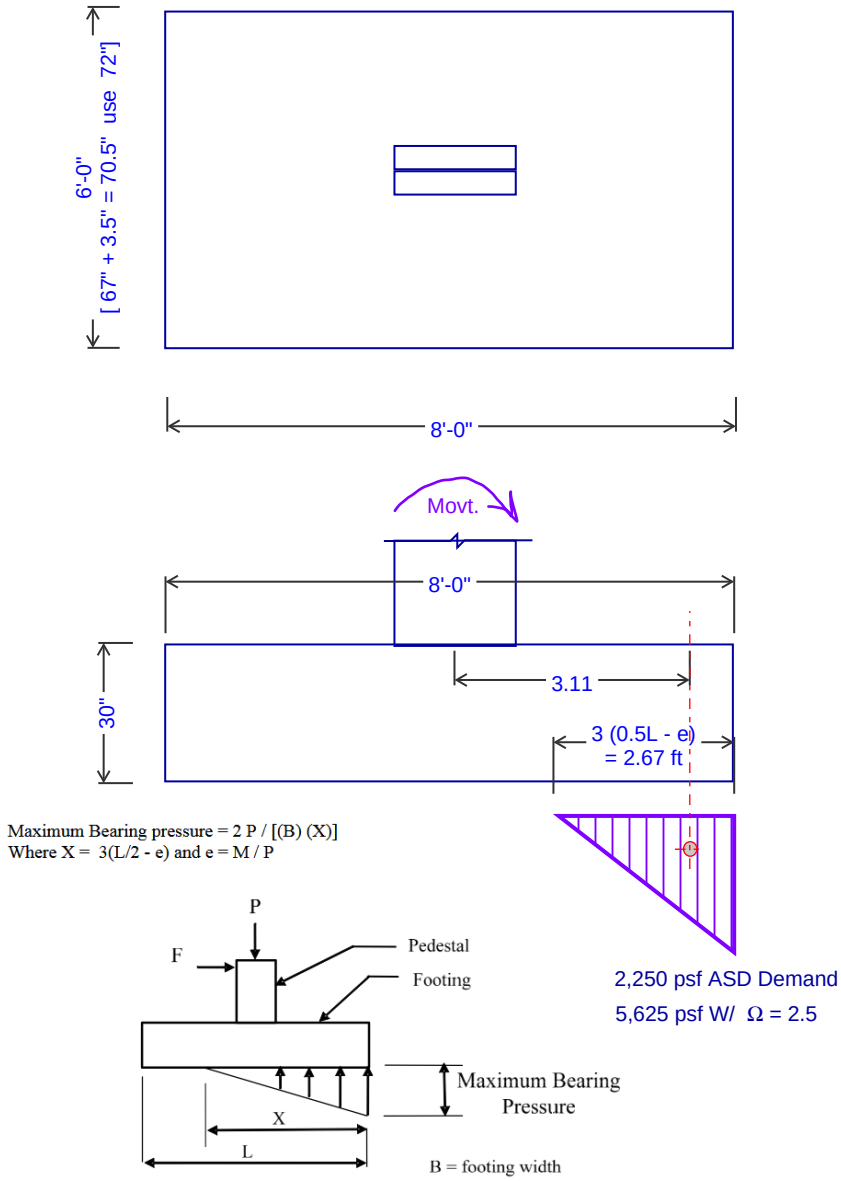
Movt. Back to Back; (for concrete design)
M = T x B = (76,800) (17.5) /12
= 112,000 ft-lbs LRFD

M LRFD / M ASD Demand =
112,000 ft-lbs / 56,00 ft-lbs = 2
Multiply ASD Demand stress by factor of 2.

ACI 318-19: 17.10.5.3:(d) ;

HS; High Strength Anchor Bolts:
fya = 105 ksi
futa= 125 ksi
Nominal Strength: ACI 17.6.1.2;
Nsa = Ase,N futa
futa = 1.9 fya 125 ksi max use 125 ksi
Ase,N = 0.606 in2 1" dia.
Nsa = 75,750 lbs
Tensile Strength per AISC J.3.6;
Rn = Fnt Ab
Fnt = 0.75 Fu = 0.75 (125) = 93.75 ksi
Ab = 0.606 in2
Rn = 56,800 lbs.
ϕRn = 0.75 (56,800) = 42,600 lbs.

Section uncracked, as the bending moment is less than the cracking moment of the concrete section.



For Concrete Design:

Weight of Footing :
2.5 ft x 150 pcf = 375 psf
P = 6ft x 8 ft x 375 psf = 18,000 lbs
L = 8 ft B = 6 ft.

M,ovt = 56,000 ft-lbs at ASD Demand

e = M/P = 56,000 ft-lbs / 18,000 lbs = 3.11
L / 6 = 8 ft / 6 = 1.333 ft
e > L/6;
Stress = 2P / (3 x (L/2 - e) x B)
= 2 (18000) / ((3)((8/2)-3.11)(6)) = 2250 psf

to Scale to Wall LRFD Capacity
multiply by 2 , see left column calc.
stress = 2 x 2250 = 4,500 psf

to Scale of Overstrength;
multiply by 2.5 (Ω for flexible diaphragm)
stress = 2.5x 2250 = 5,625 psf

Use 5,625 psf Linear Varying Load.

M max = 5,625 psf x 6 ft x 2.67 ft x 0.5 x [4 - 2.67/3] ft = 140,124 ft-lbs.

V max = 5,625 psf x 6 ft x 2.67 ft x 0.5 = 45,056 lbs

Lines [B] and [D], Back to Back WSWH Strong Walls:

Overturning:

Maximum Lateral Force [Line B] = 12,000 lbs ASD [0.7ρE ρ=1.3]

(1) set of Back to Back Walls Loading = 6,000 lbs.

Overturning Moment:
M,ovt. = V x h
6,000 lbs x 9.33' = 56,000 ft-lbs at Base , h = 9.33' ht. of panel

Stabilizing/Resisting Moment:
Assuming weight of footing only, with no additional roof dead loads (conservatively)
M,res. = 0.9 (P,footing) x 1/2 L
= 0.9 (L x W x D x 150 pcf) x 1/2 L
= 0.9 (8.00 x 6.00 x D x 150 pcf) x 0.5 (8.00)
= 25,920 D,min.

56,600 < 25,920 Dmin. Dmin > 2.18 ft

D provided = 2'-6" (2.50 > 2.18 OK)

CBC-2022:

1605.2 Alternative Allowable Stress Design Load Combinations Diagram

In lieu of the load combinations in ASCE 7, Section 2.4, structures and portions thereof shall be permitted to be designed for the most critical effects resulting from the following combinations. Where using these alternative allowable stress load combinations that include wind or seismic loads, allowable stresses are permitted to be increased or load combinations reduced where permitted by the material chapter of this code or the referenced standards. For load combinations that include the counteracting effects of dead and wind loads, only two-thirds of the minimum dead load likely to be in place during a design wind event shall be used. Where using these alternative load combinations to evaluate sliding, overturning and soil bearing at the soil-structure interface, the reduction of foundation overturning from Section 12.13.4 in ASCE 7 shall not be used. Where using these alternative basic load combinations for proportioning foundations for loadings, which include seismic loads, the vertical seismic load effect, E_v, in Equation 12.4-4 of ASCE 7 is permitted to be taken equal to zero. Where required by ASCE 7, Chapters 12, 13 and 15, the load combinations including overstrength of ASCE 7, Section 2.3.6 shall be used. ***[OSHPD 1R, 2 & 5]*** *Each load combination shall be investigated with one or more of the variable loads set to zero.*

$D + L + (L_r \text{ or } S \text{ or } R)$ **(Equation 16-1)**

$D + L + 0.6W$ **(Equation 16-2)**

$D + L + 0.6W + S/2$ **(Equation 16-3)**

$D + L + S + 0.6W/2$ **(Equation 16-4)**

$D + L + S + E/1.4$ **(Equation 16-5)**

$0.9D + E/1.4$ **(Equation 16-6)**

Detail # 2, 5 & 6 / S5.3 WSWH Foundation

California Building Engineers, Inc.

Material Strength:

fc' =	3,000	psi	εcu =	0.00300		λ =	1.00
√fc' =	54.8	psi	β1 =	0.85	*per ACI 22.2.2.4.3*		
fy =	60,000	psi	εy =	0.00207		fy,ties =	60,000 psi

Beam Geometry:

Total Depth, h = 30 in

Beam Width, b *(Beam Top)* = 72 in — *Bottom Tension* — *Section Height, h1 =* 30 in

b *(Beam Bottom)* = 72 in — *Top Tension* — *Section Height, h2 =* in

Reinforcement:

Cover to Stirrup: Top = 3 in, Side = 3 in

Bottom = 3 in, Spacer between Reinf. Layers = 2 in

Flexural Reinf. Top Tension

Top Reinf.	Qty.	Bar Size	As,bar (in²)	As,provided (in²)		d,eff (in)	*Dist to Center of Bar from Extreme Tension Fiber (in)*
Upper Layer	7	#5	0.31	2.17	d,TU =	26.50	3.50
Lower Layer			0.00	0.00	d,TL =	0.00	
Inner Layer			0.00	0.00	d,TI =	0.00	
		Provided	As,top =	2.17	d,top =	26.50	

Section uncracked under soil pressure with seismic overstrength

As,temp		As,req'd	1.33 As,req'd	As,min	As,max
0.0018	*0.0009*				
3.89	1.94	1.18	1.58	6.36	25.63

Max. Factored Bending Moment , Mu =	140,124	ft-lbs	DCR
Cracking Bending Moment , ϕ Mn =	147,885	ft-lbs	95%
Flex. Capacity w/ Provided Reinf. , ϕ Mn =	255,310	ft-lbs	55%

Flexural Reinf. Bottom Tension

Bottom Reinf.	Qty.	Bar Size	As,bar (in²)	As,provided (in²)		d,eff (in)	*Dist to Center of Bar from Extreme Tension Fiber (in)*
Inner Layer			0.00	0.00	d,BI =	0.00	
Upper Layer			0.00	0.00	d,BU =	0.00	
Lower Layer	7	#5	0.31	2.17	d,BL =	26.50	3.50
			As,bottom =	2.17	d,bot =	26.50	

As,temp		As,req'd	1.33 As,req'd	As,min	As,max
0.0018	*0.0009*				
3.89	1.94	1.18	1.58	6.36	25.63

Max. Factored Bending Moment , Mu =	140,124	ft-lbs	DCR
Cracking Bending Moment , ϕ Mn =	147,885	ft-lbs	95%
Flex. Capacity w/ Provided Reinf. , ϕ Mn =	255,310	ft-lbs	55%

Shear Reinforcement

Spacing Along Length;

s,max = 26.50 in *(default)*

Spacing Across Width;

s' max = 24.00 in

s' provided = 66.00 in

Concurrent Axial Force (+) for Compression , Nu = lbs

Maximum Factored Shear , Vu =	45,056	lbs	DCR
Shear Strength by Concrete , ϕ Vc =	48,449	lbs	93%
Shear Strength provided by Transverse Reinf. , ϕ Vs =	0	lbs	
Shear Strength Combined , ϕ Vn =	48,449	lbs	93%

Provide additional legs of shear reinforcement, spacing across width exceeds allowed.

Section Dimesnions are OK for Shear

	# of Legs	Size	Av,leg	Av, total	s,user *(if other than s,max)*	
Outer			0.00	0.00		0
+ Inner			0.00	0.00		0

Av,provided / s = 0.0000

Av,min / s = Not Req'd

Av ≥ Av,min ? No

Flexural Capacity:

Cracking Moment: *ACI 318-19 ; Ch. 14.5.2.1*

ACI 14.5.2.1

ϕ Mn = 147,885 ft-lbs ***Top Tension***

ϕ Mn = 147,885 ft-lbs ***Bottom Tension***

Section Modulus, S_m;

Width (in)	Depth (in)		A (in^2)	y (in) (from top)	y A (in^3)		I (in^4) = $bh^3/12$	d (in)	A d^2 (in^4)
72	30		2160	15.00	32,400		162,000	0.0	0
72	0		0	30.00	0		0	15.0	0
	30	Σ A =	2160	Σ y A =	32,400	Σ I =	162,000	Σ A d^2 =	0
				y' = Σ y A / Σ A =	15.00			I = Σ I + Σ Ad^2 =	162,000

Moment of Inertia, I = 162,000 in^4

$c_{,t}$ = 15.00 in $S_{,m} = I / c_{,t}$ = 10,800 in^3 *Top Tension*

ϕ = 0.60 $c_{,b}$ = 15.00 in $S_{,m} = I / c_{,b}$ = 10,800 in^3 *Bottom Tension*

	(a) $M_n = 5 \lambda \sqrt{f'_c} S_m$	(b) $M_n = 0.85 f'_c S_m$	$\phi M_n = \phi$ min {(a), (b)}
Top Tension	246,475 ft-lbs	2,295,000 ft-lbs	147,885 ft-lbs
Bottom Tension	246,475 ft-lbs	2,295,000 ft-lbs	147,885 ft-lbs

Tension in Bottom Fiber:

Strain Calculations:

For Singly Reinforced Section:

Compression Block Area, A_c ;

$A_c = A_s f_y / 0.85 f'_c$ $A_s =$ 2.17 in² $A_c =$ 51.06 in²

Whitney Stress Block Depth, a ;

$a = A_c / b$ $b =$ 72.00 in $a =$ 0.71 in

Neutral Axis Depth, c ;

$c = a / \beta_1$ $c =$ 0.83 in

Calculate Strain in Bars;

Bottom Reinforcement; $\varepsilon_s = \varepsilon_{cu} (d-c) / c$

Upper Layer	$d_{,BU} =$	0.00	in	$\varepsilon_s =$	0.00000 -
Lower Layer	$d_{,BL} =$	0.00	in	$\varepsilon_s =$	0.00000 -
Inner Layer	$d_{,BI} =$	26.50	in	$\varepsilon_s =$	0.09229 > εy, Reinf. yielding assumption correct

Flexural Capacity with Provided Reinforcement:

$A_{s,provided} =$ 2.17 in² b = 72.00 in

$a = A_s f_y / 0.85 f_c' b_f =$ 0.709 in d = 26.50 in

$\phi M_n = \phi f_y A_s (d - a/2) =$ 3,063,721 in-lb $\phi =$ 0.90

$\phi M_n =$ 255,310 ft-lbs

Design Required Flexural Reinforcement;

Maximum Factored Bending Moment; b = 72.00 in

$M_u =$ 140,124 ft-lbs d = 26.50 in

$M_u =$ 1,681,488 lb-in $\phi =$ 0.90

$R_n = M_u / \phi b d^2$ $R_n =$ 36.95 psi

$\rho_{,required} = [1 - \sqrt{\{1 - (2R_n / 0.85 f_c')\}}] 0.85 f_c' / f_y$ $\rho_{,required} =$ 0.00062

$A_{s,required} = \rho_{,required} b d$ **$A_{s,required} =$ 1.184 in²**

Minimum Flexural Reinforcement;

Per ACI 9.6.1.2;

$A_{s,min} =$ max {

(a) $3 (\sqrt{f_c'}/f_y) b_w d =$ 5.23 in² $\rho_{,min} =$ 0.00274 $b_w =$ 72.00 in

(b) $(200/f_y) b_w d =$ 6.36 in² $\rho_{,min} =$ 0.00333 d = 26.50 in

$A_{s,min} =$ 6.36 in² **$\rho_{,min} =$ 0.00333**

ACI 9.6.1.3

If $A_{s,provided}$ is at least 1/3 greater than $A_{s,required}$ by design at each section, ACI 9.6.1.2 need not to be satisfied;

$A_{s,provided} \geq 1.333 \times A_{s,required}$

$A_{s,provided} \geq$ 1.333 x 1.184 in² = 1.58 in² **$A_{s,provided} \geq$ 1.58 in²**

Maximum Flexural Reinforcement;

Reinforcement Ratio and Strain;

$\rho = (0.85 \beta_1 f'_c / f_y) [\varepsilon_{cu} / (\varepsilon_{cu} + \varepsilon_t)]$

Balanced Reinforcement Ratio;

$\rho_{,balance} = (0.85 \beta_1 f'_c / f_y) [\varepsilon_{cu} / (\varepsilon_{cu} + \varepsilon_t)]$; When $\varepsilon_t = \varepsilon_y$ $\rho_{,balance} =$ 0.02138

Maximum Reinforcement Ratio;

$\rho_{,max} = (0.85 \beta_1 f'_c / f_y) [\varepsilon_{cu} / (\varepsilon_{cu} + \varepsilon_t)]$; When $\varepsilon_t = \varepsilon_y + 0.003$ **$\rho_{,max} =$ 0.01343**

Maximum Flexural Reinforcement;

$A_{s,max} = \rho_{,max} b d$ $b_w =$ 72.00 in

$A_{s,max} =$ 25.63 in² d = 26.50 in

Tension in Top Fiber:

Strain Calculations:

For Singly Reinforced Section:

Compression Block Area, A_c ;

$A_c = A_s f_y / 0.85 f'_c$ $A_s =$ 2.17 in² $A_c =$ 51.06 in²

Whitney Stress Block Depth, a ;

$a = A_c / b$ $b =$ 72.00 in $a =$ 0.71 in

Neutral Axis Depth, c ;

$c = a / \beta_1$ $c =$ 0.83 in

Calculate Strain in Bars;

Top Reinforcement; $\varepsilon_s = \varepsilon_{cu} (d-c) / c$

Upper Layer	$d_{,TU} =$	26.50	in	$\varepsilon_s =$	0.09229 > εy, Reinf. yielding assumption correct
Lower Layer	$d_{,TL} =$	0.00	in	$\varepsilon_s =$	0.00000 -
Inner Layer	$d_{,TI} =$	0.00	in	$\varepsilon_s =$	0.00000 -

Flexural Capacity with Provided Reinforcement:

$A_{s,provided} =$ 2.17 in² b = 72.00 in

$a = A_s f_y / 0.85 f_c' b_f =$ 0.709 in d = 26.50 in

$\phi M_n = \phi f_y A_s (d - a/2) =$ 3,063,721 in-lb $\phi =$ 0.90

$\phi M_n =$ 255,310 ft-lbs

Design Required Flexural Reinforcement;

Maximum Factored Bending Moment; b = 72.00 in

$M_u =$ 140,124 ft-lbs d = 26.50 in

$M_u =$ 1,681,488 lb-in $\phi =$ 0.90

$R_n = M_u / \phi b d^2$ $R_n =$ 36.95 psi

$\rho_{,required} = [1 - \sqrt{\{1 - (2R_n / 0.85 f_c')\}}] 0.85 f_c' / f_y$ $\rho_{,required} =$ 0.00062

$A_{s,required} = \rho_{,required} b d$ **$A_{s,required} =$ 1.184 in²**

Minimum Flexural Reinforcement;

Per ACI 9.6.1.2;

$A_{s,min} =$ max { (a) $3 (\sqrt{f_c'}/f_y) b_w d =$ 5.23 in² $\rho_{,min} =$ 0.00274 $b_w =$ 72.00 in

(b) $(200/f_y) b_w d =$ 6.36 in² $\rho_{,min} =$ 0.00333 d = 26.50 in

$A_{s,min} =$ 6.36 in² **$\rho_{,min} =$ 0.00333**

ACI 9.6.1.3

If $A_{s,provided}$ is at least 1/3 greater than $A_{s,required}$ by design at each section, ACI 9.6.1.2 need not to be satisfied;

$A_{s,provided} \geq 1.333 \times A_{s,required}$

$A_{s,provided} \geq$ 1.333 x 1.184 in² = 1.58 in² **$A_{s,provided} \geq$ 1.58 in²**

Maximum Flexural Reinforcement;

Reinforcement Ratio and Strain;

$\rho = (0.85 \beta_1 f'_c / f_y) [\varepsilon_{cu} / (\varepsilon_{cu} + \varepsilon_t)]$

Balanced Reinforcement Ratio;

$\rho_{,balance} = (0.85 \beta_1 f'_c / f_y) [\varepsilon_{cu} / (\varepsilon_{cu} + \varepsilon_t)]$; When $\varepsilon_t = \varepsilon_y$ $\rho_{,balance} =$ 0.02138

Maximum Reinforcement Ratio;

$\rho_{,max} = (0.85 \beta_1 f'_c / f_y) [\varepsilon_{cu} / (\varepsilon_{cu} + \varepsilon_t)]$; When $\varepsilon_t = \varepsilon_y + 0.003$ **$\rho_{,max} =$ 0.01343**

Maximum Flexural Reinforcement;

$A_{s,max} = \rho_{,max} b d$ **$A_{s,max} =$ 25.63 in²** $b_w =$ 72.00 in

d = 26.50 in

Shear Capacity:

Shear Capacity: *ACI 318-19 ; Ch. 22.5*

V_u = **45,056** lbs h = 30.00 in

d,min = 26.50 in $A_{s,\ provided,\ min}$ = 2.17 in²

ϕ = 0.75 $b_{w,min}$ = 72.00 in ρ_w = 0.0011

Cross-Sectional Dimensions shall satisfy ;

$V_u \le \phi\ (V_c + 8\ \sqrt{f'_c}\ b_w\ d)$ (Eq. 22.5.1.2) *cl. 22.5.1.2*

$V_u \le$ 675,482 lbs *Vu = 45056 lbs < V,section limit = 675482 lbs , Section Dimesnions are OK for Shear*

Concrete Shear Strength, V_c ; *Table 22.5.2.1*

$A_v \ge A_{v,min}$		$[(2\ \lambda\ \sqrt{f'_c}) + (N_u / 6\ A_g)]\ b_w\ d$	(a)	209,011 lbs	Utilize the greater value.
	or	$[(8\ \lambda\ (\rho_w)^{1/3}\ \sqrt{f'_c}) + (N_u / 6\ A_g)]\ b_w\ d$	(b)	87,268 lbs	Either may be used
$A_v < A_{v,min}$		$[(8\ \lambda_s\ \lambda\ (\rho_w)^{1/3}\ \sqrt{f'_c}) + (N_u / 6\ A_g)]\ b_w\ d$	(c)	64,599 lbs	

$A_v \ge A_{v,min}$? No $V_{c,\ calc}$ = 64,599 lbs

Limit on V_c ; *cl. 22.5.5.1.1*

$V_{c\ max} \le 5\ \lambda\ \sqrt{f'_c}\ b_w\ d$

$V_{c\ max} \le$ 522,527

$V_{c,\ used}$ = 64,599 lbs

Limit on $N_u / 6\ A_g \le 0.05\ f'_c$; *cl. 22.5.5.1.2*

N_u = 0 lbs + for Compression

A_g = 2160 in²

$N_u / 6\ A_g$ = 0 psi

$0.05\ f'_c$ = 150 psi

$N_u / 6\ A_g$, used = 0 psi

Size Effect modification factor, λs ; *cl. 22.5.5.1.3*

$\lambda s = \sqrt{[2 / (1 + d/10)]} \le 1$

λs = 0.74

λs , used = 0.74

Shear Reinforcement Capacity , V_s :

$V_s \ge V_u / \phi - V_c$ *Eq (22.5.8.1)* $V_{s,required}$ = 0 lbs

No Shear Reinforcement Required, see Av,min requirements

$V_s = (A_v / s)\ f_{yt}\ d$ *Eq (22.5.8.5.3)* f_{yt} = 60,000 psi d = 26.50 in

	# of Legs	Size	$A_{v,leg}$	$A_{v,\ total}$	s	A_v / s	$V_s = (A_v / s)\ f_{yt}\ d$		Utilized Shear		*
Outer	0	0	0.00	0.00	26.50	0.0000	#DIV/0!	0 lbs	0	#####	#####
+ Inner	0	0	0.00	0.00	26.50	0.0000	#DIV/0!	0 lbs	0	#####	

$A_{v,min}$ / s = Not Req'd **$A_{v,provided}$ / s = 0.0000** **V_s = 0 lbs**

** Av / s Remaining for Torsion - (Only outer legs) =* #DIV/0!

Combined Shear Capacity , ϕV_n :

$\phi V_n = V_c + V_s$ *Eq (22.5.1.1)* ϕV_n = (0.75) (64599 lbs + 0 lbs) **ϕV_n = 48,449 lbs**

Minimum Shear Reinf. Requirements:

Minimum Area of Shear Reinforcement:

Provide $A_{v,min}$ where $V_u > \phi\ \lambda\ \sqrt{f'_c}\ b_w\ d$ = 78,379 lbs *cl. 9.6.3.1* *$A_{v,min}$ required ? No*

Vu = 45056 lbs < φ λ √f'c bw d = 78379 lbs , Av,min not required

$A_{v,min}$; *Table 9.6.3.4*

$A_{v,min}$ / s =	greater	$0.75\ \sqrt{f'_c}\ b_w / f_{yt}$	(a)	0.0493	**$A_{v,min}$ / s = Not Req'd**
		$50\ b_w / f_{yt}$	(b)	0.0600	**$A_{v,provided}$ / s No min req'd**

Maximum Spacing of Legs: *Table 9.7.6.2.2*

$4\ \sqrt{f'_c}\ b_w\ d$ = 418,022 lbs	Along Length		Across Width		max
$V_{s,req} \le 4\ \sqrt{f'_c}\ b_w\ d$	d/2 =	13.25 in	d =	26.50 in	24 in
$V_{s,req} > 4\ \sqrt{f'_c}\ b_w\ d$	d/4 =	6.50 in	d/2 =	13.25 in	12 in

Maximum Spacing per Design Shear Load;

Along Length, s_{max} = 26.5 in

Along Width, s'_{max} = 24 in s' = Distance Between Legs in one transverse reinf. set

s' provided = 66.00 in *Provide additional legs of shear reinforcement, spacing across width e*

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