PCS Structural Solutions

Seattle 1011 Western Avenue, Suite 810 | Seattle, WA 98104 | 206.292.5076
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PRCTI20260468

# STRUCTURAL CALCULATIONS

FOR

## PEM WOODS COFFEE
## 1127 E MAIN AVE
## PUYALLUP, WA 98372

Calculations required to be provided by the Permittee on site for all Inspections

PREPARED BY
PCS STRUCTURAL *SOLUTIONS*


MARCH 30, 2026
26-216

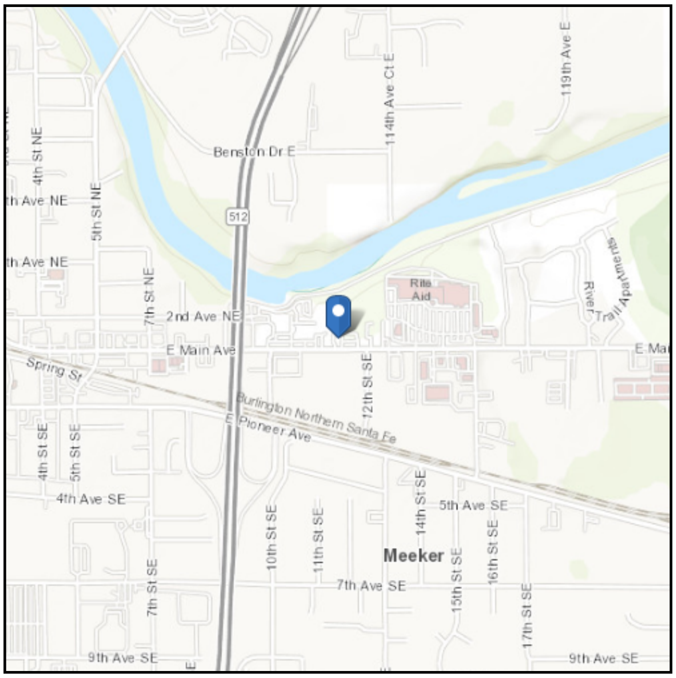
City of Puyallup Building
REVIEWED FOR COMPLIANCE
SKinnear
04/14/2026
9:06:50 AM
CITY OF PUYALLUP
STATE OF WASHINGTON

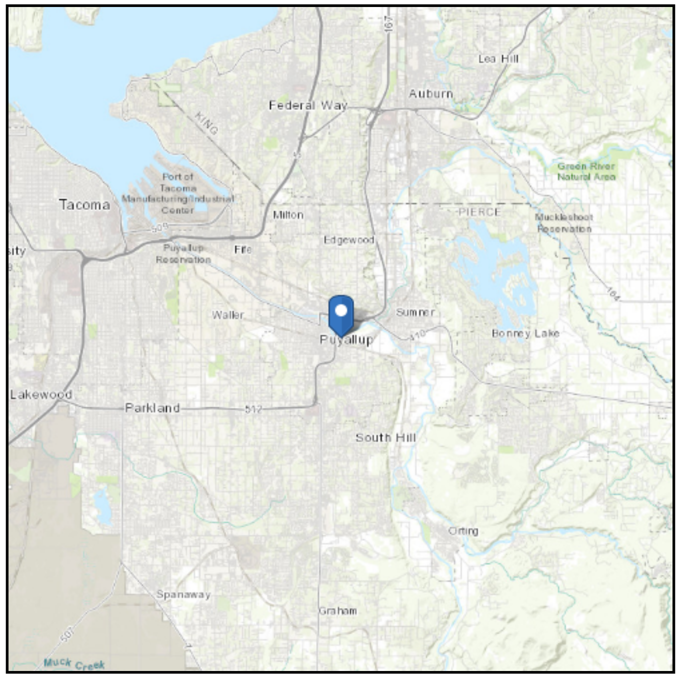
# DESIGN CRITERIA

# ASCE Hazards Report

**Address:**
1115 E Main
Puyallup, Washington
98372

| | | | |
|---|---|---|---|
| **Standard:** | ASCE/SEI 7-16 | **Latitude:** | 47.192103 |
| **Risk Category:** | II | **Longitude:** | -122.27905 |
| **Soil Class:** | D - Default (see Section 11.4.3) | **Elevation:** | 54.273300457218 ft (NAVD 88) |




## Wind

**Results:**

| | |
|---|---|
| Wind Speed | 98 Vmph |
| 10-year MRI | 67 Vmph |
| 25-year MRI | 73 Vmph |
| 50-year MRI | 78 Vmph |
| 100-year MRI | 83 Vmph |

| | |
|---|---|
| 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: | Fri Jul 26 2024 |

Value provided is 3-second gust wind speeds at 33 ft above ground for Exposure C Category, based on linear interpolation between contours. Wind speeds are interpolated in accordance with the 7-16 Standard. Wind speeds correspond to approximately a 7% probability of exceedance in 50 years (annual exceedance probability = 0.00143, MRI = 700 years).

Site is not in a hurricane-prone region as defined in ASCE/SEI 7-16 Section 26.2.

## Seismic

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

**Results:**

| | | | |
|---|---|---|---|
| $S_S$ : | 1.266 | $S_{D1}$ : | N/A |
| $S_1$ : | 0.436 | $T_L$ : | 6 |
| $F_a$ : | 1.2 | PGA : | 0.5 |
| $F_v$ : | N/A | $PGA_M$ : | 0.6 |
| $S_{MS}$ : | 1.519 | $F_{PGA}$ : | 1.2 |
| $S_{M1}$ : | N/A | $I_e$ : | 1 |
| $S_{DS}$ : | 1.013 | $C_v$ : | 1.353 |

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

**Data Accessed:** Fri Jul 26 2024

**Date Source:** [USGS Seismic Design Maps]

AMERICAN SOCIETY OF CIVIL ENGINEERS

# Snow

**Results:**

| | |
|---|---|
| Ground Snow Load, $p_g$ : | 18 lb/ft$^2$ |
| Mapped Elevation: | 54.3 ft |
| Data Source: | |
| Date Accessed: | Fri Jul 26 2024 |

Statutory requirements of the Authority Having Jurisdiction are not included.

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.

Project: PEM Woods Coffee | Job Number: 26-216
Sheet: ____ of ____ | Name: KES
Originating Office: Tacoma | Date: 3/23/2026

# DESIGN CRITERIA CHECKLIST

CODE: IBC 2021, ASCE 7-16 | LOCATION: PUYALLUP, WA

RISK CATEGORY: II (Per ASCE 7-16 Table 1.5-1 & IBC Table 1604.5)

## VERTICAL DESIGN CRITERIA

| | DEAD | LIVE | PARTITION | CONCENTRATED |
|---|---|---|---|---|
| ROOF: | 20 PSF | 25 PSF | | |
| | | | | |
| | | | | |
| | | | | |
| | | | | |
| | | | | |
| | | | | |

## WIND DESIGN CRITERIA

| | | |
|---|---|---|
| BASIC WIND SPEED (V) = | 98 MPH | (Per ASCE 7-16 Sec. 26.5.1, Fig. 26.5-1A; 1B; 1C & 1D, or as required by Bld'g Dept.) |
| EXPOSURE CATEGORY: | B | (Per ACSE 7-16 Section 26.7.3) |
| DIRECTIONALITY FACTOR ($K_d$): | 0.85 | (Per ASCE 7-16 Table 26.6-1) |
| GUST EFFECT FACTOR (G): | 0.85 | (Per ASCE 7-16 Section 26.11) |
| TOPOGRAPHIC FEATURE: | None | (See ASCE 7-16 Figure 26.8-1) |
| HILL HEIGHT (H): | 0 FT | (See ASCE 7-16 Figure 26.8-1) |
| UPWIND DISTANCE TO HALF HILL (Lh): | 0 FT | (See ASCE 7-16 Figure 26.8-1) |
| DISTANCE FROM CREST TO SITE (x): | 0 FT | UPWIND (See ASCE 7-16 Figure 26.8-1) |
| MEAN ROOF HEIGHT: | 0-15 FT | (See ASCE 7-16 Section 26.2 - Definitions) |
| ELEVATION: | 0 FT | (See ASCE 7-16 Section 26.9) |
| ENCLOSURE CLASSIFICATION: | Enclosed | (See ASCE 7-16 Secion 26.2 & Table 26.13-1) |
| ROOF TYPE: | Monoslope | (See ASCE 7-16 Figure 27.3-1) |
| ROOF SLOPE (__:12): | 0.25:12 | (Enter vertical rise in 12 horizontal units) θ (degrees): 1.19 |

## SEISMIC DESIGN CRITERIA

| | | |
|---|---|---|
| SITE CLASS: | D | (Per IBC Section 1613.2.2, Assumed as "D" or per Geotech.) |
| IMPORTANCE FACTOR ($I_E$): | 1 | (Per ASCE 7-16 Table 1.5-2) |
| STRUCTURAL SYSTEM (R): | 6.5 | (Per ASCE 7-16 Table 12.2-1) |
| OVERSTRENGTH FACTOR ($\Omega_o$): | 2.5 | (Per ASCE 7-16 Table 12.2-1) |

INFORMATION BELOW FROM APPLIED TECHNOLOGY COUNCIL (ATC) "HAZARDS BY LOCATION"

| | | | | | |
|---|---|---|---|---|---|
| LATITUDE: | 47.192 | $S_S$ = | 1.266 | $F_a$ = | 1.200 |
| LONGITUDE: | -122.279 | $S_1$ = | 0.436 | $F_v$ = | N/A |

## DEFLECTION CRITERIA

| | | | |
|---|---|---|---|
| FLOOR (LIVE): | L/ 480 | ROOF (LIVE): | L/ 360 |
| FLOOR (TOTAL): | L/ 360 | ROOF (TOTAL): | L/ 240 |
| WALLS: | L/ 360 | SPECIAL: | L/ |

## SOIL DESIGN CRITERIA

REPORT: NO

NO SOILS REPORT PROVIDED. ALL ENGINEERING SOIL PARAMETERS ARE ASSUMED.

| | |
|---|---|
| BEARING: | 1500 PSF |
| ACTIVE: | 35 PCF |
| PASSIVE: | 250 PCF |
| COEFFICIENT OF FRICTION: | 0.35 |

MINIMUM FOOTING DIMENSIONS:

| | |
|---|---|
| CONTINUOUS: | 1'-4" |
| SPREAD: | 1'-6" |
| FROST DEPTH: | 1'-6" |

| | | | |
|---|---|---|---|
| PILE TYPE: | NONE | | |
| VERTICAL CAPACITY : | N/A | LATERAL CAPACITY: | N/A |
| UPLIFT CAPACITY: | N/A | SIZE: | N/A |

| | |
|---|---|
| Project: PEM Woods Coffee | Job Number: 26-216 |
| Sheet: of | Name: KES |
| Originating Office: Tacoma | Date: 03/23/26 |

# MATERIALS

## CONCRETE

| | | | |
|---|---|---|---|
| Footings/Piles: | 3000 PSI | Columns: | 4000 PSI |
| Slabs/Walls: | 4000 PSI | Beams: | 4000 PSI |
| - | - | - | - |

## REINFORCING

| | | | |
|---|---|---|---|
| Steel Grade = | 60 | $f_y$ = | 60 KSI |

## STRUCTURAL STEEL

| | | | |
|---|---|---|---|
| W-Flange Beams | ASTM A992 | $f_y$ = | 50 KSI |
| Shapes & Plates | ASTM A36 | $f_y$ = | 36 KSI |
| Pipes | ASTM A53, Grade B | $f_y$ = | 35 KSI |
| HSS Rect. | ASTM A500, Grade C | $f_y$ = | 50 KSI |
| HSS Round | ASTM A500, Grade C | $f_y$ = | 46 KSI |

## MASONRY

| | | | |
|---|---|---|---|
| ASTM C90 | $f'_m$ = | 2000 PSI | SOLID GROUTED |

## GLULAM BEAMS

| Simple Spans | | Cantilevers |
|---|---|---|
| 24F-V4 | Grade = | 24F-V8 |
| 1.80E+06 PSI | E = | 1.80E+06 PSI |
| 2400 PSI | $F_{b\,(BOTTOM)}$ = | 2400 PSI |
| 1850 PSI | $F_{b\,(TOP)}$ = | 2400 PSI |
| 240 PSI | $F_v$ = | 240 PSI |

## SCL PRODUCTS

| | 2x SCL | 1¾" SCL | 3½, 5¼ SCL |
|---|---|---|---|
| E = | 1.30E+06 PSI | 1.80E+06 PSI | 2.00E+06 PSI |
| $F_b$ = | 1700 PSI | 2600 PSI | 2900 PSI |
| $F_v$ = | 285 PSI | 285 PSI | 285 PSI |
| $F_c$ = | 1400 PSI | 2400 PSI | 2600 PSI |

## FRAMING LUMBER

| | | | | |
|---|---|---|---|---|
| Joists & Studs | | 2x DF #2 | 2x HF #1 | - |
| | E = | 1.60E+06 PSI | 1.50E+06 PSI | - |
| | $F_b$ = | 900 PSI | 975 PSI | - |
| | $F_v$ = | 180 PSI | 150 PSI | - |
| | $F_c$ = | 1350 PSI | 1350 PSI | - |
| Beams & Headers | | 4x DF #2 | 4x HF #1 | 6x DF #1 |
| | E = | 1.60E+06 PSI | 1.50E+06 PSI | 1.60E+06 PSI |
| | $F_b$ = | 900 PSI | 975 PSI | 1350 PSI |
| | $F_v$ = | 180 PSI | 150 PSI | 170 PSI |
| Posts & Timbers | | 6x DF #1 | - | - |
| | E = | 1.60E+06 PSI | - | - |
| | $F_c$ = | 1000 PSI | - | - |

# DESIGN CRITERIA - WIND

| | | | |
|---|---|---|---|
| BASIC WIND SPEED (V): | 98 MPH | MEAN ROOF HEIGHT: | 15 FT |
| RISK CATEGORY: | II | GROUND ELEVATION FACTOR (Ke): | 1.00 |
| EXPOSURE CATEGORY: | B | ENCLOSURE CLASSIFICATION: | Enclosed |
| DIRECTIONALITY FACTOR ($K_d$): | 0.85 | ROOF TYPE: | Monoslope |
| GUST EFFECT FACTOR (G): | 0.85 | ROOF SLOPE (__:12): | 0.3:12 |
| | | θ (degrees): | 1.19 |

**ROOF PRESSURES (Figure 27.3-1)**

| | | External Pressures ($q_h$*($GC_p$)): | | | Internal Pressures (±$q_i$*($GC_{pi}$)) |
|---|---|---|---|---|---|
| Wind Direction: | h/L: | Windward (Positive) | Windward (Negative) | Leeward | All Roofs |
| Normal to Ridge for θ ≥ 10° | ≤0.25 | N/A | N/A | N/A | 2.1 |
| | 0.50 | N/A | N/A | N/A | |
| | ≥1.0 | N/A | N/A | N/A | |

| | | | External Pressures (q*($GC_p$)): | | Internal Pressures (±$q_i$*($GC_{pi}$)) |
|---|---|---|---|---|---|
| | h/L: | Horizontal Distance from Windward Edge | Positive Pressure | Negative Pressure | All Roofs |
| Normal to Ridge for θ < 10° and Parallel to Ridge for All θ | ≤0.5 | 0 to h | -1.8 | -9.1 | 2.1 |
| | | h to 2h | | -5.1 | |
| | | >2h | | -3.0 | |
| | | | -1.8 | -13.2 | |
| | | >h/2 | | -7.1 | |

**ASCE 7-16 CHAPTER 27: WIND LOADS ON BUILDINGS: MWFRS (DIRECTIONAL PROCEDURE)**
**PART 1: ENCLOSED AND PARTIALLY ENCLOSED BUILDINGS OF ALL HEIGHTS**
**HORIZONTAL WALL PRESSURES (Figure 27.3-1)**

| Windward External Pressures ($q_z$*($GC_p$)): | | | Leeward & Sidewall External Pressures ($q_h$*($GC_p$)): | | | Internal Pressures (±$q_i$*($GC_{pi}$)) |
|---|---|---|---|---|---|---|
| Height Above Ground Level, z | $K_{zt}$ | Windward wall | L/B: | Leeward wall | Sidewall | All walls |
| 15 | 1.00 | 8.1 | 0-1 | -5.1 | -7.1 | 2.1 |
| 20 | 1.00 | 8.8 | 2 | -3.0 | | |
| 25 | 1.00 | 9.4 | ≥4 | -2.0 | | |
| 30 | 1.00 | 9.9 | | | | |
| 40 | 1.00 | 10.8 | | | | |
| 50 | 1.00 | 11.5 | | | | |
| 60 | 1.00 | 12.1 | | | | |
| 70 | 1.00 | 12.6 | | | | |
| 80 | 1.00 | 13.2 | | | | |
| 90 | 1.00 | 13.6 | | | | |
| 100 | 1.00 | 14.1 | | | | |
| 120 | 1.00 | 14.8 | | | | |
| 140 | 1.00 | 15.5 | | | | |
| 160 | 1.00 | 16.1 | | | | |
| 180 | 1.00 | 16.6 | | | | |
| 200 | 1.00 | 17.1 | | | | |
| 250 | 1.00 | 18.2 | | | | |
| 300 | 1.00 | 19.2 | | | | |
| 350 | 1.00 | 20.0 | | | | |
| 400 | 1.00 | 20.9 | | | | |
| 450 | 1.00 | 21.6 | | | | |
| 500 | 1.00 | 22.2 | | | | |

**NOTES:**

1) Minimum Design Wind Loads (Per ASCE 7-16 27.1.5): The wind load used for design of the MWFRS shall not be less than 16 PSF multiplied by the wall area of the building, and 8 PSF multiplied by the roof area of the building projected on a vertical plane normal to the assumed wind direction. Wall and roof loads shall be applied simultaneously.

2) $q_i$ has conservatively been taken equal to $q_h$

$K_{ht}$ = 1.00

$q_h$ = 11.9 PSF

# DESIGN CRITERIA - WIND

| | | | |
|---|---|---|---|
| BASIC WIND SPEED (V): | 98 MPH | MEAN ROOF HEIGHT: | 15 FT |
| RISK CATEGORY: | II | GROUND ELEVATION FACTOR (Ke): | 1.00 |
| EXPOSURE CATEGORY: | B | ENCLOSURE CLASSIFICATION: | Enclosed |
| DIRECTIONALITY FACTOR ($K_d$): | 0.85 | ROOF TYPE: | Monoslope |
| GUST EFFECT FACTOR (G): | 0.85 | ROOF SLOPE (__:12): | 0.3:12 |
| | | θ (degrees): | 1.19 |

**ASCE 7-16 CHAPTER 30: WIND LOADS: COMPONENTS AND CLADDING**
**PART 1: LOW-RISE BUILDINGS (h≤60 ft)**

**ROOF SURFACES**

| Effective Wind Area | POSITIVE PRESSURES | NEGATIVE PRESSURES | | | | | |
|---|---|---|---|---|---|---|---|
| | ZONE | | | | | | |
| | ALL ZONES | 1' | 1 | 2 | 3 | N/A | N/A |
| 10 SF | 16.0 | -16.0 | -22.4 | -29.5 | -40.3 | N/A | N/A |
| 20 SF | 16.0 | -16.0 | -20.9 | -27.6 | -36.5 | N/A | N/A |
| 50 SF | 16.0 | -16.0 | -19.0 | -25.1 | -31.4 | N/A | N/A |
| 100 SF | 16.0 | -16.0 | -17.5 | -23.2 | -27.6 | N/A | N/A |

**WALL SURFACES & ROOF OVERHANGS**

| Effective Wind Area | WALL ZONES | | | | ROOF OVERHANG ZONES | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| | POSITIVE PRESSURES | | NEGATIVE PRESSURES | | NEGATIVE PRESSURES | | | | | |
| | 4 | 5 | 4 | 5 | 1' | 1 | 2 | 3 | N/A | N/A |
| 10 SF | 16.0 | 16.0 | -16.0 | -18.8 | -20.3 | -20.3 | -27.4 | -38.1 | N/A | N/A |
| 20 SF | 16.0 | 16.0 | -16.0 | -17.6 | -19.9 | -19.9 | -24.9 | -33.7 | N/A | N/A |
| 50 SF | 16.0 | 16.0 | -16.0 | -16.0 | -19.4 | -19.4 | -21.5 | -27.8 | N/A | N/A |
| 100 SF | 16.0 | 16.0 | -16.0 | -16.0 | -19.1 | -19.1 | -19.0 | -23.4 | N/A | N/A |
| 500 SF | 16.0 | 16.0 | -16.0 | -16.0 | -18.2 | -18.2 | -16.0 | -16.0 | N/A | N/A |

**NOTES:**

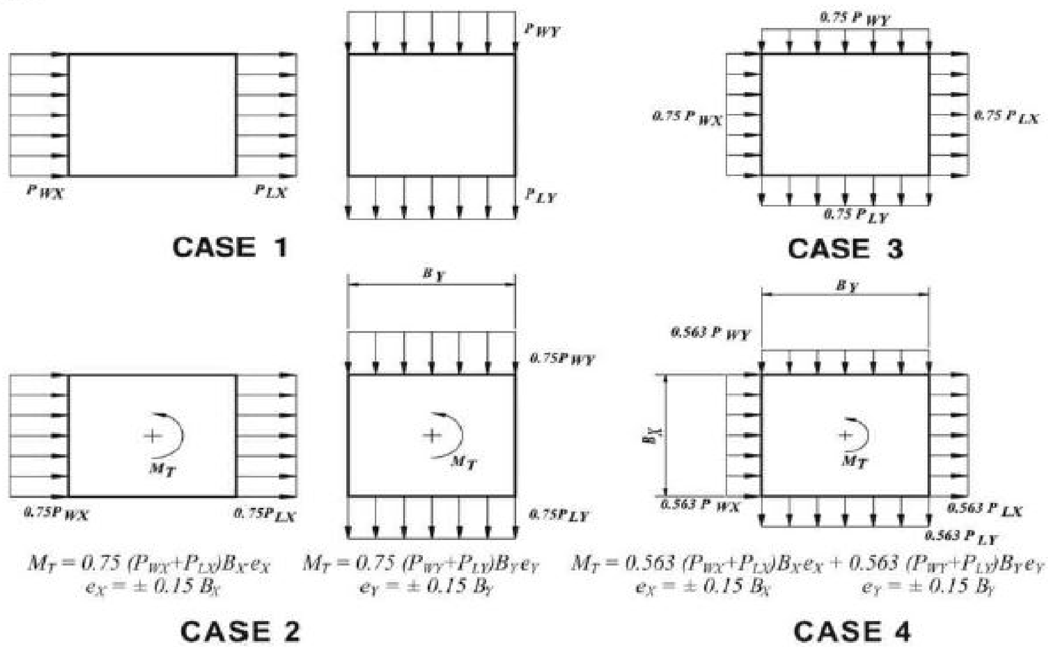
1) ASCE 7-16 30.2.2: Minimum Design Wind Loads: The design wind pressure for C&C of buildings shall not be less than a net pressure of 16 PSF acting in either direction normal to the surface.
2) $q_i$ has conservatively been taken equal to $q_h$

$K_{ht}$ = 1.00
$q_h$ = 11.9 PSF

# DESIGN CRITERIA - WIND

**FIGURE 27.3-8: Main Wind Force Resisting System, Part 1 (All Heights): Design Wind Load Cases per ASCE 7-16**

***Diagrams***




***Notation***

$P_{WX}, P_{WY}$ = Windward face design pressure acting in the $x$, $y$ principal axis, respectively.
$P_{LX}, P_{LY}$ = Leeward face design pressure acting in the $x$, $y$ principal axis, respectively.
$e(e_X, e_Y)$ = Eccentricity for the $x$, $y$ principal axis of the structure, respectively.
$M_T$ = Torsional moment per unit height acting about a vertical axis of the building.

**Case 1.** Full design wind pressure acting on the projected area perpendicular to each principal axis of the structure, considered separately along each principal axis.
**Case 2.** Three-quarters of the design wind pressure acting on the projected area perpendicular to each principal axis of the structure in conjunction with a torsional moment as shown, considered separately for each principal axis.
**Case 3.** Wind loading as defined in Case 1, but considered to act simultaneously at 75% of the specified value.
**Case 4.** Wind loading as defined in Case 2, but considered to act simultaneously at 75% of the specified value.

***Notes***

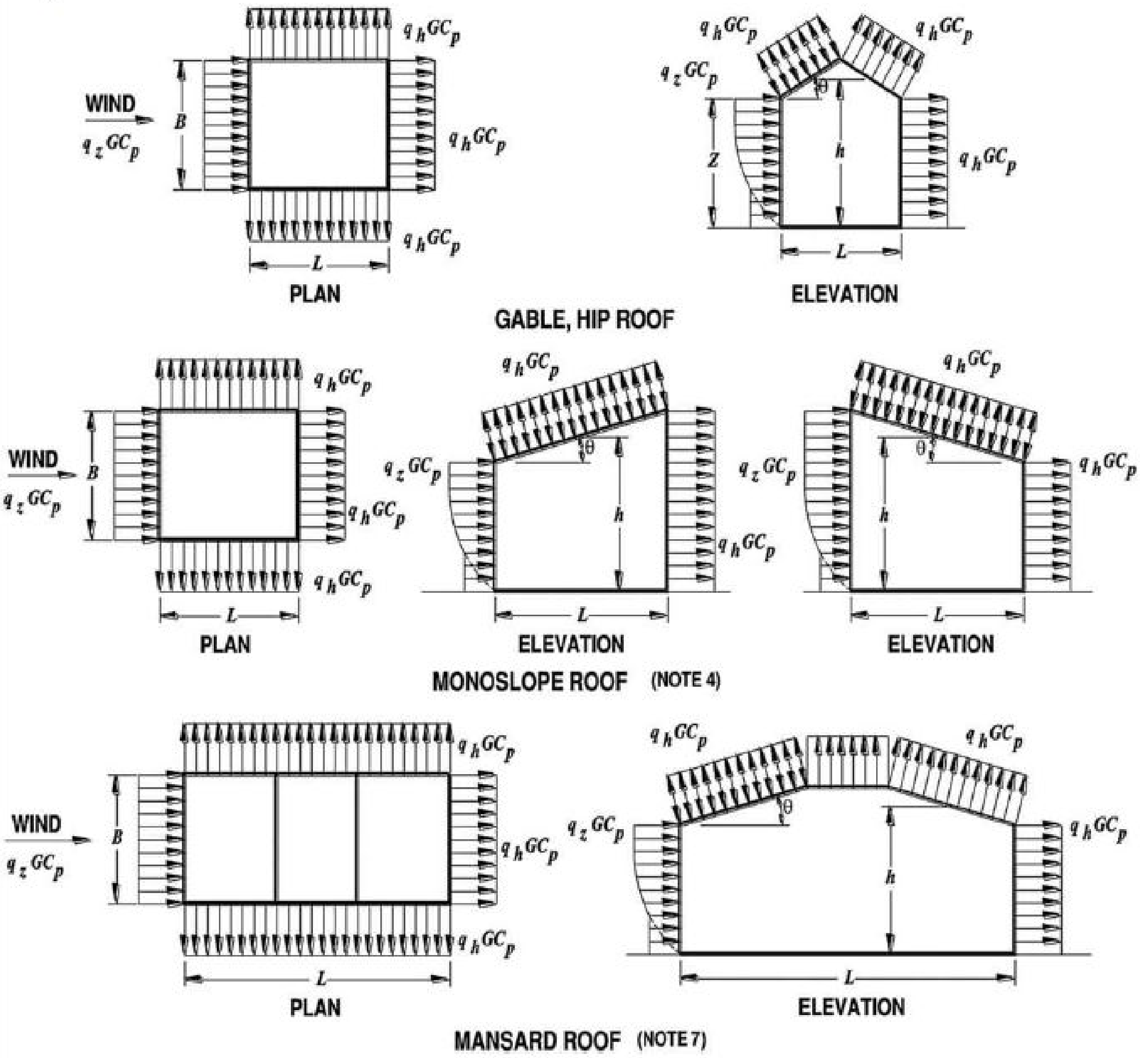
1. Design wind pressures for windward and leeward faces shall be determined in accordance with the provisions of Sections 27.3.1 and 27.3.2 as applicable for buildings of all heights.
2. Diagrams show plan views of buildings.

**FIGURE 27.3-8 Main Wind Force Resisting System, Part 1 (All Heights): Design Wind Load Cases**

# DESIGN CRITERIA - WIND

**FIGURE 27.3-1 Main Wind Force Resisting System, Part 1 (All Heights): External Pressure Coefficients, Cp, for Enclosed and Partially Enclosed Buildings - Walls and Roofs per ASCE 7-16**

***Diagrams***




***Notation***

$B$ = Horizontal dimension of building, in ft (m), measured normal to wind direction.
$L$ = Horizontal dimension of building, in ft (m), measured parallel to wind direction.
$h$ = Mean roof height, in ft (m), except that eave height shall be used for $\theta \leq 10$ degrees.
$z$ = Height above ground, in ft (m).
$G$ = Gust-effect factor.
$q_z$, $q_h$ = Velocity pressure, in lb/ft$^2$ (N/m$^2$), evaluated at respective height.
$\theta$ = Angle of plane of roof from horizontal, in degrees.

**FIGURE 27.3-1 Main Wind Force Resisting System, Part 1 (All Heights): External Pressure Coefficients, $C_p$, for Enclosed and Partially Enclosed Buildings—Walls and Roofs**

Project: PEM Woods Coffee Job Number: 26-216

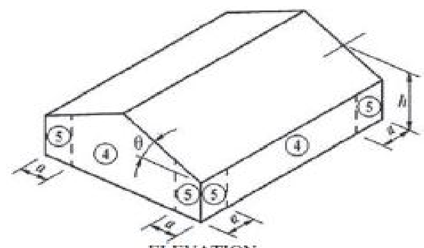
Sheet: of Name: KES

Originating Office: Tacoma Date: 03/23/26

# DESIGN CRITERIA - WIND

**FIGURE 30.3-1: Components and Cladding [h ≤ 60 ft]: External Pressure Coefficients, (GCp), for Enclosed and Partially Enclosed Buildings - Walls**

### *Diagram*




ELEVATION

### *Notation*

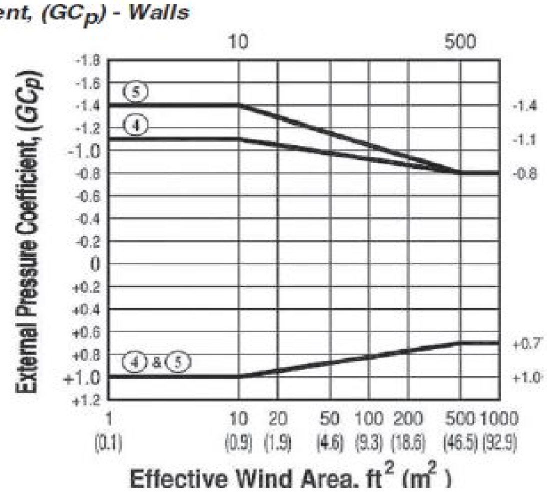
$a$ = 10% of least horizontal dimension or $0.4h$, whichever is smaller, but not less than either 4% of least horizontal dimension or 3 ft (0.9 m).

**Exception:** For buildings with $\theta = 0°$ to $7°$ and a least horizontal dimension greater than 300 ft (90 m), dimension $a$ shall be limited to a maximum of $0.8h$.

$h$ = Mean roof height, in ft (m), except that eave height shall be used for $\theta \leq 10°$.

$\theta$ = Angle of plane of roof from horizontal, in degrees.

### *External Pressure Coefficient, ($GC_p$) - Walls*




### *Notes*

1. Vertical scale denotes ($GCp$) to be used with $qh$.
2. Horizontal scale denotes effective wind area, in ft$^2$ (m$^2$).
3. Plus and minus signs signify pressures acting toward and away from the surfaces, respectively.
4. Each component shall be designed for maximum positive and negative pressures.
5. Values of ($GCp$) for walls shall be reduced by 10% when $\theta \leq 10°$.

**FIGURE 30.3-1 Components and Cladding [$h \leq 60$ ft ($h \leq 18.3$ m)]: External Pressure Coefficients, ($GC_p$), for Enclosed and Partially Enclosed Buildings—Walls**

# DESIGN CRITERIA - SEISMIC

## ASCE 7-16 SECTION 12.8 - EQUIVALENT LATERAL FORCE PROCEDURE

| | | | |
|---|---|---|---|
| RISK CATEGORY: | II | LATITUDE: | 47.192 |
| SITE CLASS: | D | LONGITUDE: | -122.279 |
| IMPORTANCE FACTOR ($I_E$): | 1 | $S_S$ = | 1.266 |
| STRUCTURAL SYSTEM (R): | 6.5 | $S_1$ = | 0.436 |
| OVERSTRENGTH FACTOR ($\Omega_o$): | 2.5 | $F_a$ = | 1.200 |
| | | $F_v$ = | N/A |

## ASCE 7-16 SECTION 11.4 SEISMIC GROUND MOTION VALUES

Section 11.4.4 - Coefficients and Risk-Targeted Maximum Considered Earthquake (MCER) Spectral Response Acceleration Parameters

$S_{MS} = F_a*S_S =$ 1.519 $S_{M1} = F_v*S_1 =$ #VALUE!

Section 11.4.5 - Design Spectral Response Acceleration Parameters

$S_{DS} = 2/3*S_{MS} =$ 1.013 $S_{D1} = 2/3*S_{M1} =$ #VALUE!

## ASCE 7-16 SECTION 11.6 - SEISMIC DESIGN CATEGORY - SECTION 12.8.2 - PERIOD DETERMINATION

ASCE 7-16 TABLE 11.6-1

SEISMIC DESIGN CATEGORY BASED ON $S_{DS}$

| | RISK CATEGORY: | | |
|---|---|---|---|
| | I & II | III | IV |
| < 0.167g | A | A | A |
| < 0.33g | B | B | C |
| < 0.50g | C | C | D |
| >= 0.50g | D | D | D |
| | **D** | | |

ASCE 7-16 TABLE 11.6-2

SEISMIC DESIGN CATEGORY BASED ON $S_{D1}$

| | RISK CATEGORY: | | |
|---|---|---|---|
| | I & II | III | IV |
| < 0.067g | A | A | A |
| < 0.133g | B | B | C |
| < 0.20g | C | C | D |
| >= 0.20g | D | D | D |
| | **#VALUE!** | | |

Each building and structure shall be assigned to the most severe Seismic Design Category in accordance with Table 11.6-1 or Table 11.6-2, irrespective of the fundamental period of vibration of the structure.

PERIOD DETERMINATION:

| | |
|---|---|
| $C_t$ = | 0.02 |
| $h_n$ = | 15 FT |
| x = | 0.75 |
| $T_a = C_t*h_n^x$ = | 0.152 |

## ASCE 7-16 SECTION 12.8.1.1 - SEISMIC RESPONSE COEFFICIENT

| | | | | |
|---|---|---|---|---|
| GENERAL EQUATION: | $C_S = S_{DS}/(R/I)$ = | 0.156 | CONTROLS | EQ. 12.8-2 |
| MAXIMUM: | CS = 1.5*SD1/(T*(R/I)) = | #VALUE! | | EQ. 12.8-3 |
| MINIMUM: | $C_S = 0.044*S_{DS}*I > 0.01$ = | 0.045 | | EQ. 12.8-5 |
| For structures located where S1 > 0.6g | | | | |
| | $C_S = 0.5*S_1/(R/I)$ = | 0.000 | | EQ. 12.8-6 |

## ASCE 7-16 SECTION 12.8.1 - SEISMIC BASE SHEAR

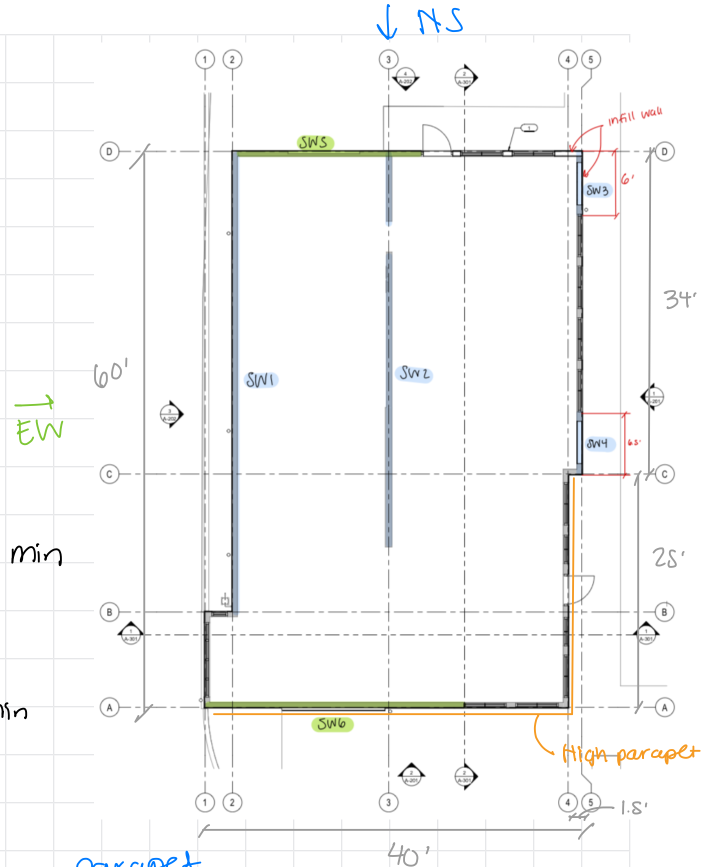
$V = C_S*W =$ **0.156*W**

W = the total dead load and applicable portion of other loads as indicated in Section 12.7.2

LATERAL

PCS Structural Solutions

Project: PEM Woods Coffee Job No: 26.216
Subject: Lateral Sheet ____ Name: KES
Originating Office: ☐ Seattle ☒ Tacoma ☐ Portland Date: 3.17.26




## Seismic

Roof wt: 20psf (2124 ft²) = 42.5k
Walls: 5psf (2124 ft²) = 10.6k
53k

$V = C_s W$  $C_s = 0.156$
$V = .156(53k) = 8.3k$
$0.7V = 0.7(8.3k) = 5.8k$

## Wind

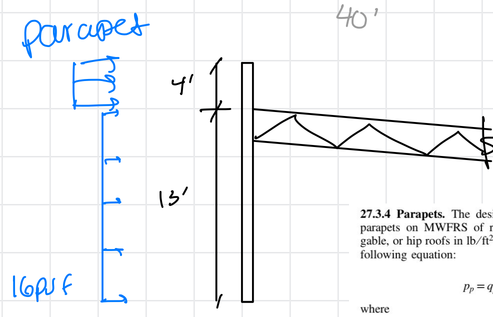
Walls
EW: windward = 8.1psf } 13.2psf → 16psf min
leeward = 5.1psf
L/B = 40'/59' = .67

NS: windward = 8.1psf } 12.7psf → 16psf min
leeward = 4.6psf
L/B = 59/40 = 1.5

Parapet: ASCE 7-16 27.3.4

Windward $p_p = q_p(GC_{pn}) = 8.8psf(1.5) = 13.2psf$ ← windward
@H=20' $q_p = 8.8psf$

Leeward $p_p = q_p(GC_{pn}) = 5psf(1.0) = 5psf$ ← leeward
Σtotal = 18.2psf




**27.3.4 Parapets.** The design wind pressure for the effect of parapets on MWFRS of rigid or flexible buildings with flat, gable, or hip roofs in lb/ft² (N/m²), shall be determined by the following equation:

$$p_p = q_p(GC_{pn})\ (\text{lb/ft}^2) \qquad (27.3\text{-}3)$$

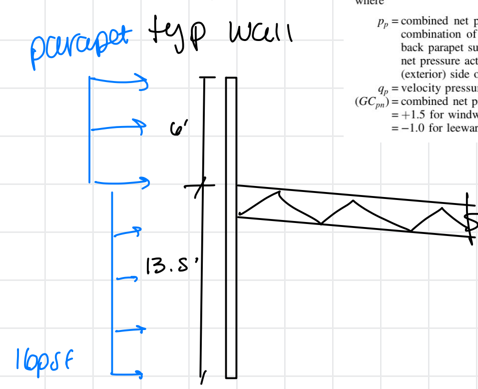
where

$p_p$ = combined net pressure on the parapet caused by the combination of the net pressures from the front and back parapet surfaces. Plus (and minus) signs signify net pressure acting toward (and away from) the front (exterior) side of the parapet.
$q_p$ = velocity pressure evaluated at the top of the parapet.
$(GC_{pn})$ = combined net pressure coefficient:
= +1.5 for windward parapet or
= −1.0 for leeward parapet.

## Total Wind Loading

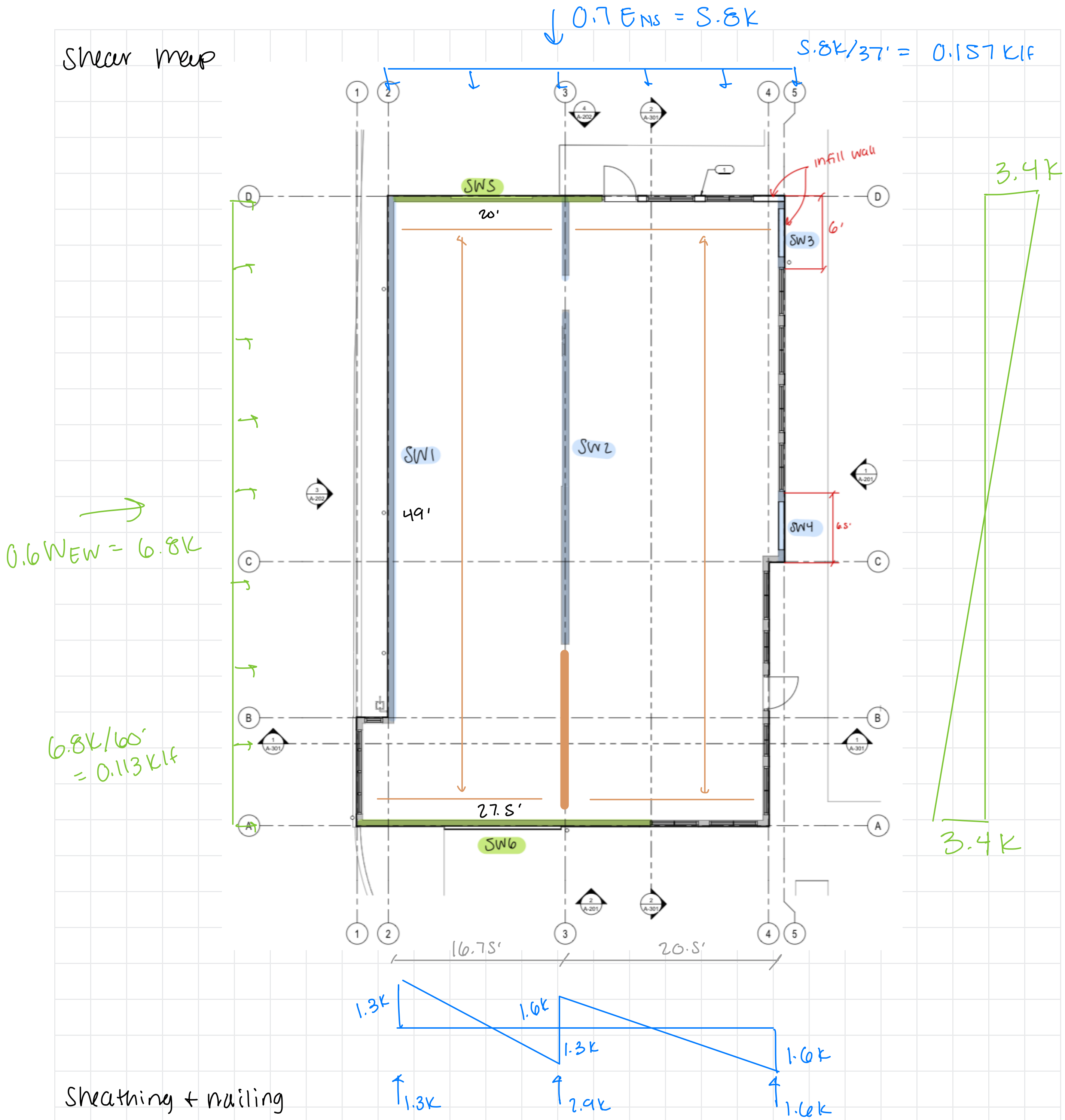
EW: Wall + Parapet
16psf (13'/2)(60') + 18.2psf (4')(34')
+ 18.2psf (6')(25')
= 6.2k + 2.5k + 2.7k
≈ 11.4k
$0.6W_{EW} = 0.6(11.4k) = 6.8k$

NS: Wall + Parapet
16psf (13'/2)(40') + 18.2psf (6')(40')
= 4.2k + 4.4k
= 8.6k
$0.6W_{NS} = 0.6(8.6k) = 5.2k$




| | |
|---|---|
| Seismic controls | NS |
| Wind controls | EW |

Shear map

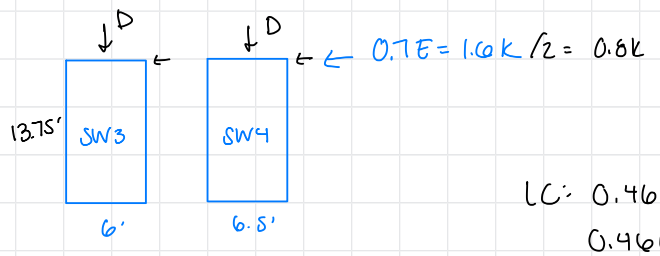



Sheathing + nailing

grid D: 3.4K / 20' = 0.17 klf
grid A: 3.4K / 27.5' = 0.12 klf
grid 2: 1.3K / 49' = .027 klf
grid 3: 2.9K / 31' = .09 klf
grid 4: 1.6K / (6' + 6.5') = .128 klf

10d @ 6" o.c. works for all walls

NS Shear walls




LC: 0.46 SDS D + 0.7Eh

0.46(1.013)D + 0.7Eh

0.466 D

SW3

D roof = 20psf (20.5'/2)(6') = 1230 lb

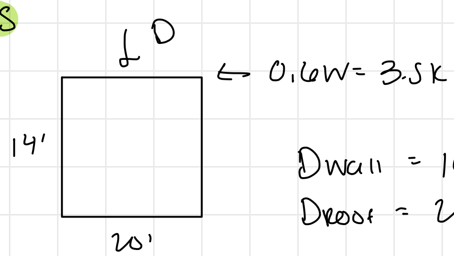
Dwall = 10psf (6')(13.75') = 825 lb } 2k

Mot = 0.7E×H = 0.8k × 13.75' = 11 kft

Mres = 0.466 D×L/2 = 0.466(2k)(6'/2) = 2.8 kft

$T = \frac{M_{OT} - M_{res}}{L} = \frac{11\,kft - 2.8\,kft}{6'} = 1.4\,k$ HDU2

SW5




Dwall = 10psf (20')(14') = 2.8k

Droof = 20psf (1')(20') = .4k

3.2k

Mot = 3.8k × 14' = 49 kft

Mres = 0.6D×L/2 = 0.6(3.2k)(20'/2) = 19.2 kft

$T = \frac{M_{OT} - M_{res}}{L} = \frac{49\,kft - 19.2\,kft}{20'} = 1.5\,k$ HDU2 requ.

By inspection all other shear walls do not need HDU

Structural *Solutions*

| | | | |
|---|---|---|---|
| Project: | Project Name | Job Number: | xx-xxx |
| Sheet: | of | Name: | xxx |
| Originating Office: | Tacoma | Date: | 3/23/2026 |

# WOOD SHEATHED SHEAR WALLS - 2021 SDPWS TABLE 4.3A

| | | | |
|---|---|---|---|
| SEISMIC OR WIND: | SEISMIC | SHEATHING NAILS: | 10d |
| DESIGN METHODOLOGY: | ASD | HOLDOWN SYSTEM: | INSIDE POST |
| SHEAR CAPACITY FACTOR: | 2.8 | 10d NAIL CAPACITY REDUCTION: | 0.92 |
| FRAMING SPECIES: | DFL | SILL PLATE ANCHOR DIAMETER: | 5/8" |
| SHEATHING MATERIAL: | SHEATHING | RIM MATERIAL: | SCL |
| PANEL THICKNESS: | 15/32" | TREATED MATERIAL FACTOR: | 1.00 |
| S.G. ADJUSTMENT FACTOR: | 1.00 | | |

## SHEAR WALL CAPACITIES

| If using LRFD, verify | WALL SHEATHING | SIDES | SHEATHING NAILS | EDGE NAILING | EDGE FRAMING | BOTTOM PLATE | NOMINAL CAPACITY (PLF) | **ADJUSTED CAPACITY (PLF)** |
|---|---|---|---|---|---|---|---|---|
| **A** | 15/32" | (1) | 10d | 6" o.c. | 2x | 2x | 870 | **286** |
| **B** | 15/32" | (1) | 10d | 4" o.c. | 3x | 2x | 1290 | **424** |
| **C** | 15/32" | (1) | 10d | 3" o.c. | 3x | 2x | 1680 | **552** |
| **D** | 15/32" | (1) | 10d | 2" o.c. | 3x | 2x | 2155 | **708** |
| **E** | 15/32" | (2) | 10d | 6" o.c. | 3x | 2x | 1740 | **572** |
| **F** | 15/32" | (2) | 10d | 4" o.c. | 3x | 2x | 2580 | **848** |
| **G** | 15/32" | (2) | 10d | 3" o.c. | 3x | 2x | 3360 | **1104** |
| **H** | 15/32" | (2) | 10d | 2" o.c. | 3x | 2x | 4310 | **1416** |

## CONNECTION CALCULATOR

| SOLE PLATE | | | | | | SILL PLATE | | | RIM CONNECTOR | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| MARK | FASTENER | ROWS | SPACING | RIM | CHECK | ANCHOR | SPACING | CHECK | CLIP | SPACING | CHECK |
| **A** | 16d | (1) | 4" o.c. | 1-1/4" | NG | 5/8" | 48" o.c. | OK | A35 | 16" o.c. | OK |
| **B** | 16d | (2) | 5" o.c. | 1-3/4" | OK | 5/8" | 32" o.c. | OK | A35 | 12" o.c. | OK |
| **C** | 16d | (2) | 4" o.c. | 1-3/4" | NG | 5/8" | 24" o.c. | OK | A35 | 9" o.c. | OK |
| **D** | 16d | (3) | 5" o.c. | 3-1/2" | NG | 5/8" | 16" o.c. | OK | A35 | 8" o.c. | OK |
| **E** | 16d | (2) | 4" o.c. | 1-3/4" | NG | 5/8" | 24" o.c. | OK | A35 | 9" o.c. | OK |
| **F** | SDWS | (2) | 5" o.c. | 3-1/2" | OK | 5/8" | 16" o.c. | OK | A35 | 6" o.c. | OK |
| **G** | SDWS | (2) | 8" o.c. | 3-1/2" | NG | 5/8" | 12" o.c. | OK | A35 | 5" o.c. | OK |
| **H** | SDWS | (2) | 6" o.c. | 3-1/2" | NG | 5/8" | 10" o.c. | OK | N/A | | |

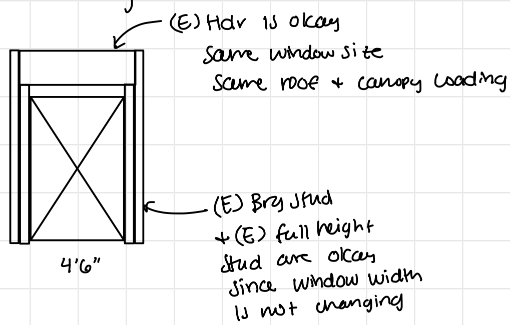
## CONNECTION CALCULATOR - REFERENCE VALUES

| **CONNECTION CAPACITIES** | | | | |
|---|---|---|---|---|
| **SOLE PLATE:** | **16d** | **110 lbs** | **SDWS** | **440 lbs** |
| **5/8" ANCHOR:** | **2x** | **1488 lbs** | **3x** | **1888 lbs** |
| **RIM:** | **A35** | **547 lbs** | **A35/LTP** | **410 lbs** |

| **NDS Table 11.3.1 Adjustment Factors** | | | |
|---|---|---|---|
| **ASD** | **LRFD Adjustment Factors** | | |
| **$C_D$** | **$K_F$** | **Φ** | **λ** |
| **1.6** | **N/A** | **N/A** | **N/A** |

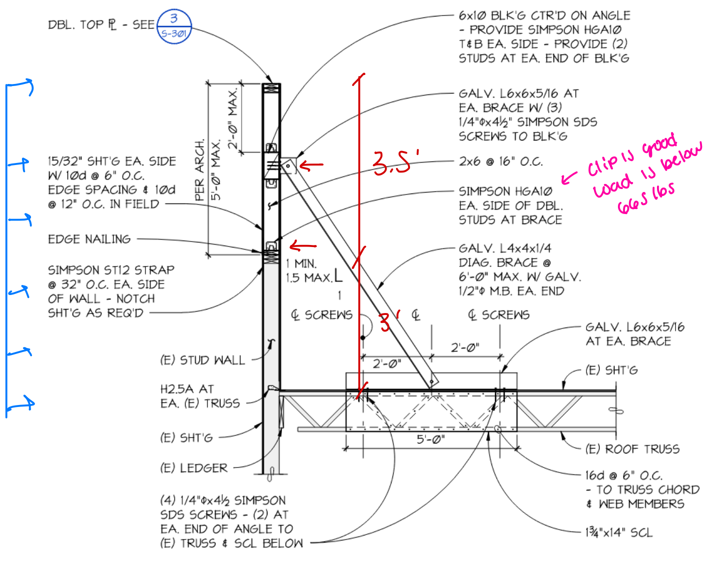
# GRAVITY

Window framing

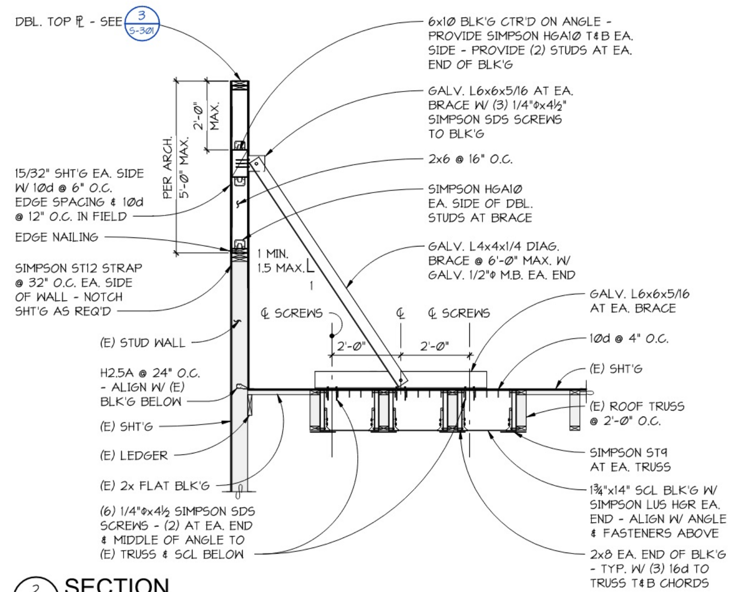


# MISC.

## parapet support




SECTION 1/S-301
1/2" = 1'-0"



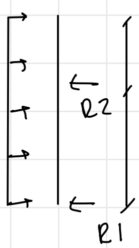

SECTION 2/S-301
1/2" = 1'-0"

## Components + Cladding Wind Load

Case A: $p_1 + p_2 = 45.5$ psf

Case B: $p_3 + p_4 = 34.8$ psf

$p = 45.5$ psf




### Check Studs

Wind loading = 45.5 psf (6')

2x6 works

R1: W = .62 K

R2: W = 1.15 K

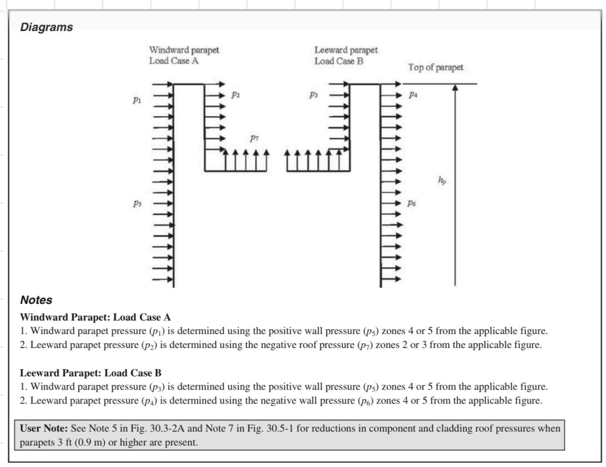



FIGURE 30.8-1 Components and Cladding, Part 6 (All Building Heights): Parapet Wind Loads, All Building Types—Parapet Wind Loads

H2.5A Clip

$$\frac{1.15\,K}{6'}(2') = 0.38\,K < 700\text{lb capacity}$$

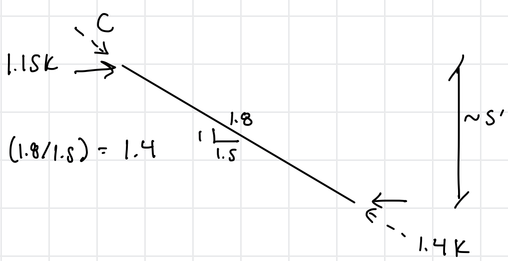
$$\left.\begin{array}{l} P_1 = 16\text{ psf} \\ P_2 = -29.5\text{ psf} \end{array}\right\} 45.5\text{ psf}$$

$$\left.\begin{array}{l} P_3 = 16\text{ psf} \\ P_4 = -18.8\text{ psf} \end{array}\right\} 34.8\text{ psf}$$

## Check bracing @ 6' o.c.

Wind loading = 45.5 psf (6') = 273 plf

R1 : w = 1.15k




C = 1.15 (1.8/1.5) = 1.4

AISC E5

Is flexural torsional buckling a concern

$b/t \le 0.71\sqrt{E/F_y}$

$4''/.25'' \le 0.71\sqrt{29\times10^6/36000}$

$16 \le 20.2$ ✓

buckling is not a concern

$KL/r = 1.0\,(5' \times 12''/ft \times (1.8/1))/1.25'' = 86.4 < 200$ ✓

## AISC E3

$P_n = f_{cr} A_g$

$\frac{KL}{r} \le 4.71\sqrt{E/f_y}$

$86.4 \le 133$ ✓

$f_{cr} = \left(0.658^{F_y/f_e}\right) F_y$

$= \left(0.658^{36ksi/38.3}\right) 36ksi$

$f_{cr} = 24.3\ ksi$

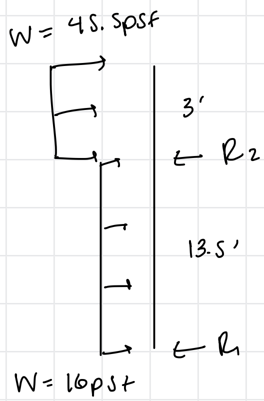
$f_e = \frac{\pi^2 E}{\left(\frac{KL}{r}\right)^2} = \frac{\pi^2\, 29\times10^6 psi}{(86.4)^2}$

$f_e = 38.3\ ksi$

$P_n = 24.3ksi\,(1.93 in^2) = 46.9k$

L4x4x1/4 angle is good

new 2x6 studs @ back wall




$R_1 = 120$ lb
$R_2 = 350$ lb

2x6 full height studs are good

Project Title: PEM Woods Coffee
Engineer: KES
Project ID: 26-216
Project Descr:

# Wood Beam

Project File: 26-216 woods coffee.ec6

LIC# : KW-06014122, Build:20.25.10.02 PCS STRUCTURAL SOLUTIONS 

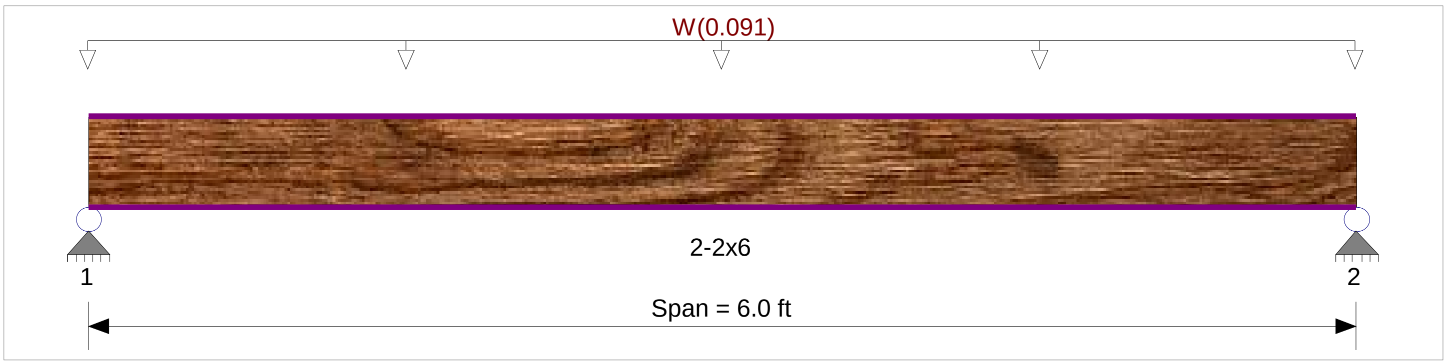
**DESCRIPTION:** PARAPET DBL TOP PL 3-19-26

## *Code References*

Governing Code : IBC 2024
Referenced Design Standard(s) : NDS 2024
Load Combination Set : ASCE 7-16

## Material Properties

| | | | | | |
|---|---|---|---|---|---|
| Analysis Method : | Allowable Stress Design | Fb + | 850.0 psi | *E : Modulus of Elasticity* | |
| Load Combination : | ASCE 7-16 | Fb - | 850.0 psi | Ebend- xx | 1,600.0 ksi |
| | | Fc - Prll | 1,400.0 psi | Eminbend - xx | 580.0 ksi |
| Wood Species : | Douglas Fir-Larch (North) | Fc - Perp | 625.0 psi | | |
| Wood Grade : | No. 1/No. 2 | Fv | 180.0 psi | | |
| | | Ft | 500.0 psi | Density | 30.590 pcf |
| Beam Bracing : | Beam is Fully Braced against lateral-torsional buckling | | | | |




## Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Beam self weight NOT internally calculated and added
Uniform Load : W = 0.04550 ksf, Tributary Width = 2.0 ft

## *DESIGN SUMMARY*

**Design OK**

| | | | | | |
|---|---|---|---|---|---|
| Maximum Bending Stress Ratio | = | **0.110** : 1 | Maximum Shear Stress Ratio | = | **0.044** : 1 |
| Section used for this span | | **2-2x6** | Section used for this span | | **2-2x6** |
| | | NDS2018 | | | NDS2018 |
| fb: Actual | = | 194.94 psi | fv: Actual | = | 12.72 psi |
| F'b | = | 1,768.00 psi | F'v | = | 288.00 psi |
| Load Combination | | +0.60W | Load Combination | | +0.60W |
| Location of maximum on span | = | 3.000 ft | Location of maximum on span | = | 0.000 ft |
| Span # where maximum occurs | = | Span # 1 | Span # where maximum occurs | = | Span # 1 |

### Maximum Deflection

| | | | | | |
|---|---|---|---|---|---|
| Max Downward Transient Deflection | 0.040 in | Ratio = | 1795 | >=360 | Span: 1 : W Only |
| Max Upward Transient Deflection | 0 in | Ratio = | 0 | >=360 | n/a |
| Max Downward Total Deflection | 0.024 in | Ratio = | 2992 | >=240 | Span: 1 : +0.60W |
| Max Upward Total Deflection | 0 in | Ratio = | 0 | >=240 | n/a |

## Maximum Forces & Stresses for Load Combinations

| Load Combination | | Max Stress Ratios | | | | | | | | | | Moment Values | | | Shear Values | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Segment Length | Span # | M | V | CD | CM | $C_t$ | CLx | $C_F$ | Cfu | $C_i$ | $C_r$ | M | fb | F'b | V | fv | F'v |
| | | | | | | | | | | | | | | 0.0 | 0.00 | 0.0 | 0.0 |
| Length = 6.0 ft | 1 | | | 0.90 | 1.00 | 1.00 | 1.00 | 1.300 | 1.00 | 1.00 | 1.00 | | | 994.5 | 0.00 | 0.0 | 162.0 |
| +0.60W | | | | | 1.00 | 1.00 | 1.00 | 1.300 | 1.00 | 1.00 | 1.00 | | | 0.0 | 0.00 | 0.0 | 0.0 |
| Length = 6.0 ft | 1 | 0.110 | 0.044 | 1.60 | 1.00 | 1.00 | 1.00 | 1.300 | 1.00 | 1.00 | 1.00 | 0.25 | 194.9 | 1,768.0 | 0.14 | 12.7 | 288.0 |
| +0.450W | | | | | 1.00 | 1.00 | 1.00 | 1.300 | 1.00 | 1.00 | 1.00 | | | 0.0 | 0.00 | 0.0 | 0.0 |
| Length = 6.0 ft | 1 | 0.083 | 0.033 | 1.60 | 1.00 | 1.00 | 1.00 | 1.300 | 1.00 | 1.00 | 1.00 | 0.18 | 146.2 | 1,768.0 | 0.10 | 9.5 | 288.0 |

# Wood Beam





**DESCRIPTION:** PARAPET DBL TOP PL 3-19-26

## Overall Maximum Deflections

| Span | Load Combination | Max. "-" Defl | Location in Span | Load Combination | Max. "+" Defl | Location in Span |
|---|---|---|---|---|---|---|
| 1 | W Only | 0.0401 | 3.022 | | 0.0000 | 0.000 |

## Vertical Reactions

Support notation : Far left is #1 Values in KIPS

| Load Combination | Support 1 | Support 2 |
|---|---|---|
| Max Upward from all Load Conditions | 0.273 | 0.273 |
| Max Upward from Load Combinations | 0.164 | 0.164 |
| Max Upward from Load Cases | 0.273 | 0.273 |
| +0.60W | 0.164 | 0.164 |
| +0.450W | 0.123 | 0.123 |
| W Only | 0.273 | 0.273 |

# Wood Beam

Project File: 26-216 woods coffee.ec6

LIC# : KW-06014122, Build:20.25.10.02 PCS STRUCTURAL SOLUTIONS 

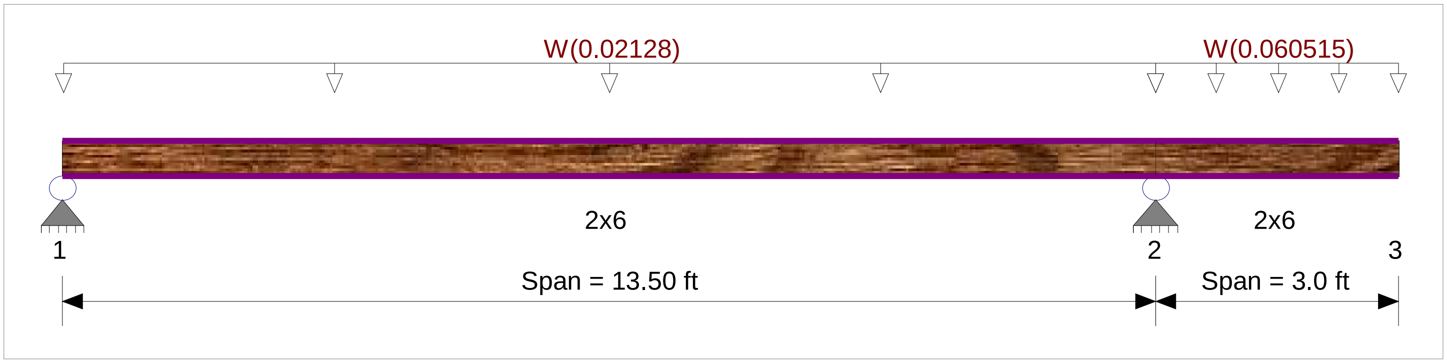
**DESCRIPTION:** new wall at the back (3-26-26)

## *Code References*

Governing Code : IBC 2024
Referenced Design Standard(s) : NDS 2024
Load Combination Set : ASCE 7-16

## Material Properties

| | | | | | |
|---|---|---|---|---|---|
| Analysis Method : | Allowable Stress Design | Fb + | 900.0 psi | *E : Modulus of Elasticity* | |
| Load Combination : | ASCE 7-16 | Fb - | 900.0 psi | Ebend- xx | 1,600.0 ksi |
| | | Fc - Prll | 1,350.0 psi | Eminbend - xx | 580.0 ksi |
| Wood Species : | Douglas Fir-Larch | Fc - Perp | 625.0 psi | | |
| Wood Grade : | No.2 | Fv | 180.0 psi | | |
| | | Ft | 575.0 psi | Density | 31.210 pcf |
| Beam Bracing : | Beam is Fully Braced against lateral-torsional buckling | | | | |




## Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Beam self weight NOT internally calculated and added
Load for Span Number 1
Uniform Load : W = 0.0160 ksf, Tributary Width = 1.330 ft
Load for Span Number 2
Uniform Load : W = 0.04550 ksf, Tributary Width = 1.330 ft

## *DESIGN SUMMARY*

**Design OK**

| | | | | | |
|---|---|---|---|---|---|
| Maximum Bending Stress Ratio | = | **0.182** : 1 | Maximum Shear Stress Ratio | = | **0.058** : 1 |
| Section used for this span | | **2x6** | Section used for this span | | **2x6** |
| | | NDS2018 | | | NDS2018 |
| fb: Actual | = | 341.02 psi | fv: Actual | = | 16.82 psi |
| F'b | = | 1,872.00 psi | F'v | = | 288.00 psi |
| Load Combination | | +0.60W | Load Combination | | +0.60W |
| Location of maximum on span | = | 5.807 ft | Location of maximum on span | = | 13.047 ft |
| Span # where maximum occurs | = | Span # 1 | Span # where maximum occurs | = | Span # 1 |

Maximum Deflection

| | | | | | |
|---|---|---|---|---|---|
| Max Downward Transient Deflection | 0 in | Ratio = | 0 | >=360 | n/a |
| Max Upward Transient Deflection | 0 in | Ratio = | 0 | >=360 | n/a |
| Max Downward Total Deflection | 0.193 in | Ratio = | 839 | >=360 | Span: 1 : +0.60W |
| Max Upward Total Deflection | -0.070 in | Ratio = | 1022 | >=360 | Span: 2 : +0.60W |

## Maximum Forces & Stresses for Load Combinations

| Load Combination | | Max Stress Ratios | | | | | | | | | | Moment Values | | | Shear Values | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Segment Length | Span # | M | V | CD | CM | $C_t$ | CLx | $C_F$ | Cfu | $C_i$ | $C_r$ | M | fb | F'b | V | fv | F'v |
| | | | | | | | | | | | | | | 0.0 | 0.00 | 0.0 | 0.0 |
| Length = 13.50 ft | 1 | | | 0.90 | 1.00 | 1.00 | 1.00 | 1.300 | 1.00 | 1.00 | 1.00 | | | 1,053.0 | 0.00 | 0.0 | 162.0 |
| Length = 3.0 ft | 2 | | | 0.90 | 1.00 | 1.00 | 1.00 | 1.300 | 1.00 | 1.00 | 1.00 | | | 1,053.0 | 0.00 | 0.0 | 162.0 |
| +0.60W | | | | | 1.00 | 1.00 | 1.00 | 1.300 | 1.00 | 1.00 | 1.00 | | | 0.0 | 0.00 | 0.0 | 0.0 |
| Length = 13.50 ft | 1 | 0.182 | 0.058 | 1.60 | 1.00 | 1.00 | 1.00 | 1.300 | 1.00 | 1.00 | 1.00 | 0.21 | 341.0 | 1,872.0 | 0.09 | 16.8 | 288.0 |

# Wood Beam

Project File: 26-216 woods coffee.ec6



**DESCRIPTION:** new wall at the back (3-26-26)

## Maximum Forces & Stresses for Load Combinations

| Load Combination | | Max Stress Ratios | | | | | | | | | | Moment Values | | | Shear Values | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Segment Length | Span # | M | V | CD | CM | $C_t$ | CLx | $C_F$ | Cfu | $C_i$ | $C_r$ | M | fb | F'b | V | fv | F'v |
| Length = 3.0 ft | **2** | 0.138 | 0.058 | 1.60 | 1.00 | 1.00 | 1.00 | 1.300 | 1.00 | 1.00 | 1.00 | 0.16 | 259.3 | 1,872.0 | 0.09 | 16.8 | 288.0 |
| +0.450W | | | | | 1.00 | 1.00 | 1.00 | 1.300 | 1.00 | 1.00 | 1.00 | | | 0.0 | 0.00 | 0.0 | 0.0 |
| Length = 13.50 ft | **1** | 0.137 | 0.044 | 1.60 | 1.00 | 1.00 | 1.00 | 1.300 | 1.00 | 1.00 | 1.00 | 0.16 | 255.8 | 1,872.0 | 0.07 | 12.6 | 288.0 |
| Length = 3.0 ft | **2** | 0.104 | 0.044 | 1.60 | 1.00 | 1.00 | 1.00 | 1.300 | 1.00 | 1.00 | 1.00 | 0.12 | 194.4 | 1,872.0 | 0.07 | 12.6 | 288.0 |

## Overall Maximum Deflections

| Span | Load Combination | Max. "-" Defl | Location in Span | Load Combination | Max. "+" Defl | Location in Span |
|---|---|---|---|---|---|---|
| 1 | +0.60W | 0.1930 | 6.335 | | 0.0000 | 0.000 |
| 2 | | 0.0000 | 6.335 | +0.60W | -0.0704 | 3.000 |

## Vertical Reactions

Support notation : Far left is #1 Values in KIPS

| Load Combination | Support 1 | Support 2 | Support 3 |
|---|---|---|---|
| Max Upward from all Load Conditions | 0.123 | 0.345 | |
| Max Upward from Load Combinations | 0.074 | 0.207 | |
| Max Upward from Load Cases | 0.123 | 0.345 | |
| +0.60W | 0.074 | 0.207 | |
| +0.450W | 0.056 | 0.155 | |
| W Only | 0.123 | 0.345 | |

## Wood Beam

Project File: 26-216 woods coffee.ec6

LIC# : KW-06014122, Build:20.25.10.02 PCS STRUCTURAL SOLUTIONS 

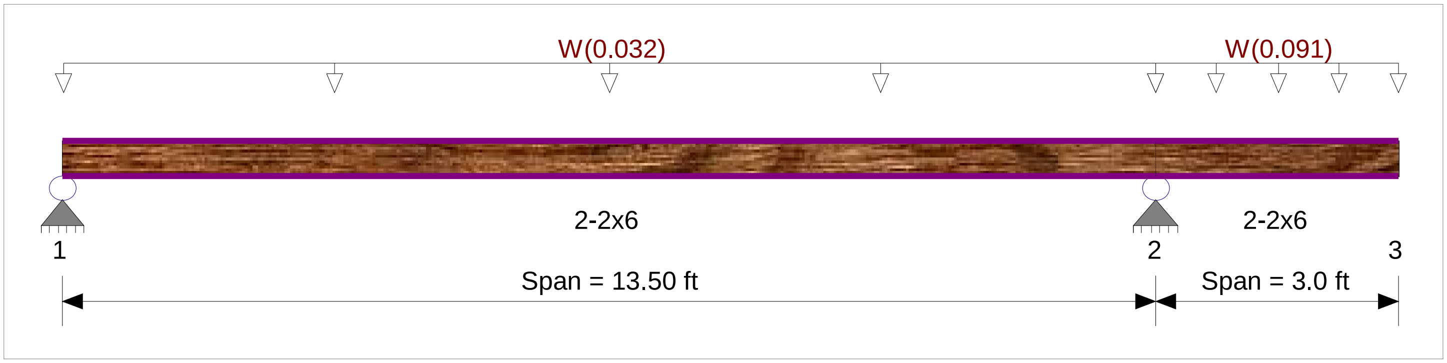
**DESCRIPTION:** new wall at the back (3-26-26) full height studs

### *Code References*

Governing Code : IBC 2024
Referenced Design Standard(s) : NDS 2024
Load Combination Set : ASCE 7-16

### Material Properties

| | | | | | |
|---|---|---|---|---|---|
| Analysis Method : | Allowable Stress Design | Fb + | 900.0 psi | *E : Modulus of Elasticity* | |
| Load Combination : | ASCE 7-16 | Fb - | 900.0 psi | Ebend- xx | 1,600.0 ksi |
| | | Fc - Prll | 1,350.0 psi | Eminbend - xx | 580.0 ksi |
| Wood Species : | Douglas Fir-Larch | Fc - Perp | 625.0 psi | | |
| Wood Grade : | No.2 | Fv | 180.0 psi | | |
| | | Ft | 575.0 psi | Density | 31.210 pcf |
| Beam Bracing : | Beam is Fully Braced against lateral-torsional buckling | | | | |




### Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Beam self weight NOT internally calculated and added
Load for Span Number 1
Uniform Load : W = 0.0160 ksf, Tributary Width = 2.0 ft
Load for Span Number 2
Uniform Load : W = 0.04550 ksf, Tributary Width = 2.0 ft

### *DESIGN SUMMARY*

**Design OK**

| | | | | | |
|---|---|---|---|---|---|
| Maximum Bending Stress Ratio | = | **0.137** : 1 | Maximum Shear Stress Ratio | = | **0.044** : 1 |
| Section used for this span | | **2-2x6** | Section used for this span | | **2-2x6** |
| | | NDS2018 | | | NDS2018 |
| fb: Actual | = | 256.40 psi | fv: Actual | = | 12.65 psi |
| F'b | = | 1,872.00 psi | F'v | = | 288.00 psi |
| Load Combination | | +0.60W | Load Combination | | +0.60W |
| Location of maximum on span | = | 5.807 ft | Location of maximum on span | = | 13.047 ft |
| Span # where maximum occurs | = | Span # 1 | Span # where maximum occurs | = | Span # 1 |

Maximum Deflection

| | | | | | | |
|---|---|---|---|---|---|---|
| Max Downward Transient Deflection | 0 | in | Ratio = | 0 | >=360 | n/a |
| Max Upward Transient Deflection | 0 | in | Ratio = | 0 | >=360 | n/a |
| Max Downward Total Deflection | 0.145 | in | Ratio = | 1116 | >=360 | Span: 1 : +0.60W |
| Max Upward Total Deflection | -0.053 | in | Ratio = | 1360 | >=360 | Span: 2 : +0.60W |

### Maximum Forces & Stresses for Load Combinations

| Load Combination | | Max Stress Ratios | | | | | | | | | | Moment Values | | | Shear Values | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Segment Length | Span # | M | V | CD | CM | $C_t$ | CLx | $C_F$ | Cfu | $C_i$ | $C_r$ | M | fb | F'b | V | fv | F'v |
| | | | | | | | | | | | | | | 0.0 | 0.00 | 0.0 | 0.0 |
| Length = 13.50 ft | **1** | | | 0.90 | 1.00 | 1.00 | 1.00 | 1.300 | 1.00 | 1.00 | 1.00 | | | 1,053.0 | 0.00 | 0.0 | 162.0 |
| Length = 3.0 ft | **2** | | | 0.90 | 1.00 | 1.00 | 1.00 | 1.300 | 1.00 | 1.00 | 1.00 | | | 1,053.0 | 0.00 | 0.0 | 162.0 |
| +0.60W | | | | | 1.00 | 1.00 | 1.00 | 1.300 | 1.00 | 1.00 | 1.00 | | | 0.0 | 0.00 | 0.0 | 0.0 |
| Length = 13.50 ft | **1** | 0.137 | 0.044 | 1.60 | 1.00 | 1.00 | 1.00 | 1.300 | 1.00 | 1.00 | 1.00 | 0.32 | 256.4 | 1,872.0 | 0.14 | 12.6 | 288.0 |

# Wood Beam

Project File: 26-216 woods coffee.ec6



**DESCRIPTION:** new wall at the back (3-26-26) full height studs

## Maximum Forces & Stresses for Load Combinations

| Load Combination | | Max Stress Ratios | | | | | | | | | | Moment Values | | | Shear Values | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Segment Length | Span # | M | V | CD | CM | $C_t$ | CLx | $C_F$ | Cfu | $C_i$ | $C_r$ | M | fb | F'b | V | fv | F'v |
| Length = 3.0 ft | 2 | 0.104 | 0.044 | 1.60 | 1.00 | 1.00 | 1.00 | 1.300 | 1.00 | 1.00 | 1.00 | 0.25 | 194.9 | 1,872.0 | 0.14 | 12.6 | 288.0 |
| +0.450W | | | | | 1.00 | 1.00 | 1.00 | 1.300 | 1.00 | 1.00 | 1.00 | | | 0.0 | 0.00 | 0.0 | 0.0 |
| Length = 13.50 ft | 1 | 0.103 | 0.033 | 1.60 | 1.00 | 1.00 | 1.00 | 1.300 | 1.00 | 1.00 | 1.00 | 0.24 | 192.3 | 1,872.0 | 0.10 | 9.5 | 288.0 |
| Length = 3.0 ft | 2 | 0.078 | 0.033 | 1.60 | 1.00 | 1.00 | 1.00 | 1.300 | 1.00 | 1.00 | 1.00 | 0.18 | 146.2 | 1,872.0 | 0.10 | 9.5 | 288.0 |

## Overall Maximum Deflections

| Span | Load Combination | Max. "-" Defl | Location in Span | Load Combination | Max. "+" Defl | Location in Span |
|---|---|---|---|---|---|---|
| 1 | +0.60W | 0.1451 | 6.335 | | 0.0000 | 0.000 |
| 2 | | 0.0000 | 6.335 | +0.60W | -0.0529 | 3.000 |

## Vertical Reactions

Support notation : Far left is #1 Values in KIPS

| Load Combination | Support 1 | Support 2 | Support 3 |
|---|---|---|---|
| Max Upward from all Load Conditions | 0.186 | 0.519 | |
| Max Upward from Load Combinations | 0.111 | 0.312 | |
| Max Upward from Load Cases | 0.186 | 0.519 | |
| +0.60W | 0.111 | 0.312 | |
| +0.450W | 0.084 | 0.234 | |
| W Only | 0.186 | 0.519 | |

Project Title: PEM Woods Coffee
Engineer: KES
Project ID: 26-216
Project Descr:

## Wood Beam

Project File: 26-216 woods coffee.ec6

LIC# : KW-06014122, Build:20.25.10.02 PCS STRUCTURAL SOLUTIONS (c) ENERCALC, LLC 1982-2026

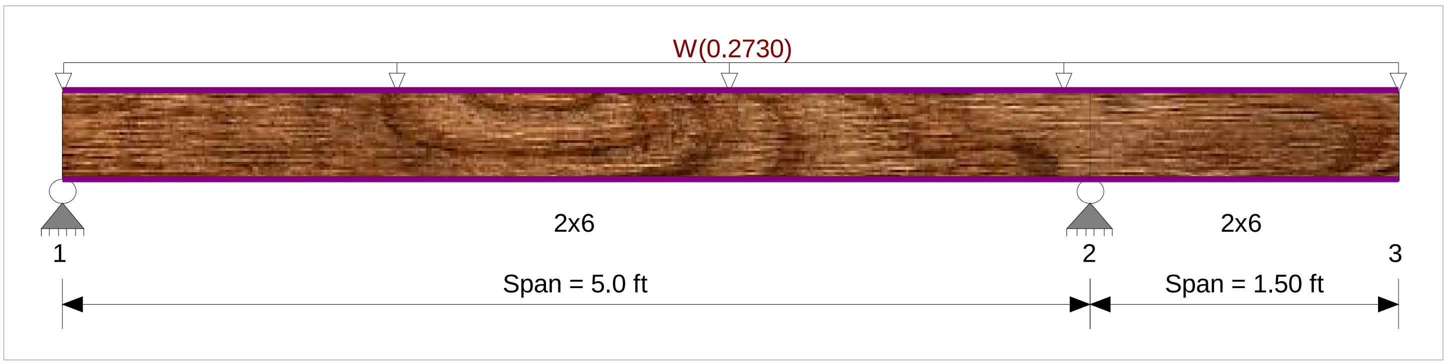
**DESCRIPTION:** Parapet stud 3-18-26

### *Code References*

Governing Code : IBC 2024
Referenced Design Standard(s) : NDS 2024
Load Combination Set : ASCE 7-16

### Material Properties

| | | | | | |
|---|---|---|---|---|---|
| Analysis Method : | Allowable Stress Design | Fb + | 850.0 psi | *E : Modulus of Elasticity* | |
| Load Combination : | ASCE 7-16 | Fb - | 850.0 psi | Ebend- xx | 1,600.0 ksi |
| | | Fc - Prll | 1,400.0 psi | Eminbend - xx | 580.0 ksi |
| Wood Species : | Douglas Fir-Larch (North) | Fc - Perp | 625.0 psi | | |
| Wood Grade : | No. 1/No. 2 | Fv | 180.0 psi | | |
| | | Ft | 500.0 psi | Density | 30.590 pcf |
| Beam Bracing : | Beam is Fully Braced against lateral-torsional buckling | | | | |




### Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Beam self weight NOT internally calculated and added
Loads on all spans...
Uniform Load on ALL spans : W = 0.04550 ksf, Tributary Width = 6.0 ft

### *DESIGN SUMMARY*

**Design OK**

| | | | | | |
|---|---|---|---|---|---|
| Maximum Bending Stress Ratio | = | **0.380** : 1 | Maximum Shear Stress Ratio | = | **0.236** : 1 |
| Section used for this span | | **2x6** | Section used for this span | | **2x6** |
| | | NDS2018 | | | NDS2018 |
| fb: Actual | = | 672.59 psi | fv: Actual | = | 67.85 psi |
| F'b | = | 1,768.00 psi | F'v | = | 288.00 psi |
| Load Combination | | +0.60W | Load Combination | | +0.60W |
| Location of maximum on span | = | 2.263 ft | Location of maximum on span | = | 4.553 ft |
| Span # where maximum occurs | = | Span # 1 | Span # where maximum occurs | = | Span # 1 |

**Maximum Deflection**

| | | | | | |
|---|---|---|---|---|---|
| Max Downward Transient Deflection | 0.091 in | Ratio = | 656 | >=360 | Span: 1 : W Only |
| Max Upward Transient Deflection | -0.062 in | Ratio = | 580 | >=360 | Span: 2 : W Only |
| Max Downward Total Deflection | 0.055 in | Ratio = | 1093 | >=240 | Span: 1 : +0.60W |
| Max Upward Total Deflection | -0.037 in | Ratio = | 968 | >=240 | Span: 2 : +0.60W |

### Maximum Forces & Stresses for Load Combinations

| Load Combination | | Max Stress Ratios | | | | | | | | | | Moment Values | | | Shear Values | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Segment Length | Span # | M | V | CD | CM | $C_t$ | CLx | $C_F$ | Cfu | $C_i$ | $C_r$ | M | fb | F'b | V | fv | F'v |
| | | | | | | | | | | | | | | 0.0 | 0.00 | 0.0 | 0.0 |
| Length = 5.0 ft | 1 | | | 0.90 | 1.00 | 1.00 | 1.00 | 1.300 | 1.00 | 1.00 | 1.00 | | | 994.5 | 0.00 | 0.0 | 162.0 |
| Length = 1.50 ft | 2 | | | 0.90 | 1.00 | 1.00 | 1.00 | 1.300 | 1.00 | 1.00 | 1.00 | | | 994.5 | 0.00 | 0.0 | 162.0 |
| +0.60W | | | | | 1.00 | 1.00 | 1.00 | 1.300 | 1.00 | 1.00 | 1.00 | | | 0.0 | 0.00 | 0.0 | 0.0 |
| Length = 5.0 ft | 1 | 0.380 | 0.236 | 1.60 | 1.00 | 1.00 | 1.00 | 1.300 | 1.00 | 1.00 | 1.00 | 0.42 | 672.6 | 1,768.0 | 0.37 | 67.8 | 288.0 |
| Length = 1.50 ft | 2 | 0.165 | 0.108 | 1.60 | 1.00 | 1.00 | 1.00 | 1.300 | 1.00 | 1.00 | 1.00 | 0.18 | 292.4 | 1,768.0 | 0.17 | 31.2 | 288.0 |

## Wood Beam

Project File: 26-216 woods coffee.ec6

LIC# : KW-06014122, Build:20.25.10.02 PCS STRUCTURAL SOLUTIONS (c) ENERCALC, LLC 1982-2026

**DESCRIPTION:** Parapet stud 3-18-26

### Maximum Forces & Stresses for Load Combinations

| Load Combination | | Max Stress Ratios | | | | | | | | | | Moment Values | | | Shear Values | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Segment Length | Span # | M | V | CD | CM | $C_t$ | CLx | $C_F$ | Cfu | $C_i$ | $C_r$ | M | fb | F'b | V | fv | F'v |
| +0.450W | | | | | 1.00 | 1.00 | 1.00 | 1.300 | 1.00 | 1.00 | 1.00 | | | 0.0 | 0.00 | 0.0 | 0.0 |
| Length = 5.0 ft | **1** | 0.285 | 0.177 | 1.60 | 1.00 | 1.00 | 1.00 | 1.300 | 1.00 | 1.00 | 1.00 | 0.32 | 504.4 | 1,768.0 | 0.28 | 50.9 | 288.0 |
| Length = 1.50 ft | **2** | 0.124 | 0.081 | 1.60 | 1.00 | 1.00 | 1.00 | 1.300 | 1.00 | 1.00 | 1.00 | 0.14 | 219.3 | 1,768.0 | 0.13 | 23.4 | 288.0 |

### Overall Maximum Deflections

| Span | Load Combination | Max. "-" Defl | Location in Span | Load Combination | Max. "+" Defl | Location in Span |
|---|---|---|---|---|---|---|
| 1 | W Only | 0.0915 | 2.430 | | 0.0000 | 0.000 |
| 2 | | 0.0000 | 2.430 | W Only | -0.0619 | 1.500 |

### Vertical Reactions

Support notation : Far left is #1 Values in KIPS

| Load Combination | Support 1 | Support 2 | Support 3 |
|---|---|---|---|
| Max Upward from all Load Conditions | 0.621 | 1.153 | |
| Max Upward from Load Combinations | 0.373 | 0.692 | |
| Max Upward from Load Cases | 0.621 | 1.153 | |
| +0.60W | 0.373 | 0.692 | |
| +0.450W | 0.279 | 0.519 | |
| W Only | 0.621 | 1.153 | |