STRUCTURAL CALCULATIONS
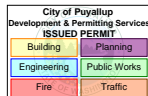
FOR

Puyallup Fair Barn M
Puyallup, WA

PREPARED BY
PCS STRUCTURAL *SOLUTIONS*
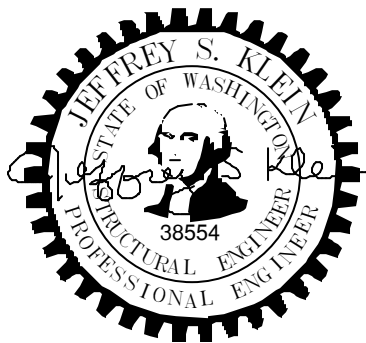
JEFFREY S. KLEIN
STATE OF WASHINGTON
STRUCTURAL ENGINEER
38554
PROFESSIONAL ENGINEER

July 16, 2025
25-037

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:	15 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 ASCE 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:	Escarpment	(See ASCE 7-16 Figure 26.8-1)
HILL HEIGHT (H):	37 FT	(See ASCE 7-16 Figure 26.8-1)
UPWIND DISTANCE TO HALF HILL (L_h):	155 FT	(See ASCE 7-16 Figure 26.8-1)
DISTANCE FROM CREST TO SITE (x):	500 FT	UPWIND (See ASCE 7-16 Figure 26.8-1)
MEAN ROOF HEIGHT:	20 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:	Gable	(See ASCE 7-16 Figure 27.3-1)
ROOF SLOPE (__:12):	8.00:12	(Enter vertical rise in 12 horizontal units) θ (degrees): 33.69

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 (Ω_o):	3.0	(Per ASCE 7-16 Table 12.2-1)

INFORMATION BELOW FROM "ASCE HAZARD TOOL"

LATITUDE:	47.183	S_S =	1.271	F_a =	1.200
LONGITUDE:	-122.296	S_1 =	0.438	F_v =	1.900

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	SEE SOILS REPORT FOR ACTIVE, PASSIVE PRESSURES AND FRICTION COEFFICIENT
BEARING:	1500 PSF	
ACTIVE:	35 PCF	MINIMUM FOOTING DIMENSIONS:
PASSIVE:	200 PCF	CONTINUOUS: 1'-4"
COEFFICIENT OF FRICTION:	0.35	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

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DESIGN CRITERIA - WIND

BASIC WIND SPEED (V): 98 MPH	MEAN ROOF HEIGHT: 20 FT
RISK CATEGORY: II	GROUND ELEVATION FACTOR (K_e): 1.00
EXPOSURE CATEGORY: B	ENCLOSURE CLASSIFICATION: Enclosed
DIRECTIONALITY FACTOR (K_d): 0.85	ROOF TYPE: Gable
GUST EFFECT FACTOR (G): 0.85	ROOF SLOPE (__:12): 8.0:12
	θ (degrees): 33.69

ROOF PRESSURES (Figure 27.3-1)

Wind Direction:	h/L:	External Pressures (q_h*(GC_p)): Windward (Positive)	Windward (Negative)	Leeward	Internal Pressures (±q_i*(GC_{pi})) All Roofs
Normal to Ridge for $\theta \geq 10°$	≤0.25	3.0	-0.4	-4.8	1.7
	0.50	2.2	-1.6	-4.8	
	≥1.0	1.6	-1.8	-4.8	

Wind Direction:	h/L:	Horizontal Distance from Windward Edge	External Pressures (q*(GC_p)): Positive Pressure	Negative Pressure	Internal Pressures (±q_i*(GC_{pi})) All Roofs
Normal to Ridge for $\theta < 10°$ and Parallel to Ridge for All θ	≤0.5	0 to h	-1.4	-7.2	1.7
		h to 2h		-4.0	
		>2h		-2.4	
		>h/2	-1.4	-10.4	
				-5.6	

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)): Height Above Ground Level, z	K_{zt}	Windward wall	Leeward & Sidewall External Pressures (q_h*(GC_p)): L/B:	Leeward wall	Sidewall	Internal Pressures (±q_i*(GC_{pi})) All walls
15	0.70	5.7	0-1	-4.0	-5.6	1.7
20	0.72	6.4	2	-2.4		
25	0.74	7.0	≥4	-1.6		
30	0.76	7.6				
40	0.80	8.6				
50	0.82	9.5				
60	0.85	10.3				
70	0.87	11.0				
80	0.89	11.8				
90	0.91	12.4				
100	0.92	12.9				
120	0.94	13.9				
140	0.96	14.8				
160	0.97	15.6				
180	0.98	16.3				
200	0.98	16.8				
250	0.99	18.1				
300	1.00	19.1				
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

$K_{ht} = 0.72$

$q_h = 9.4$ PSF

Project:	Puyallup Fair Grounds Barn M Ph1	Job Number:	25-037
Sheet:	of	Name:	KJH
Originating Office:	Tacoma	Date:	07/16/25

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DESIGN CRITERIA - WIND

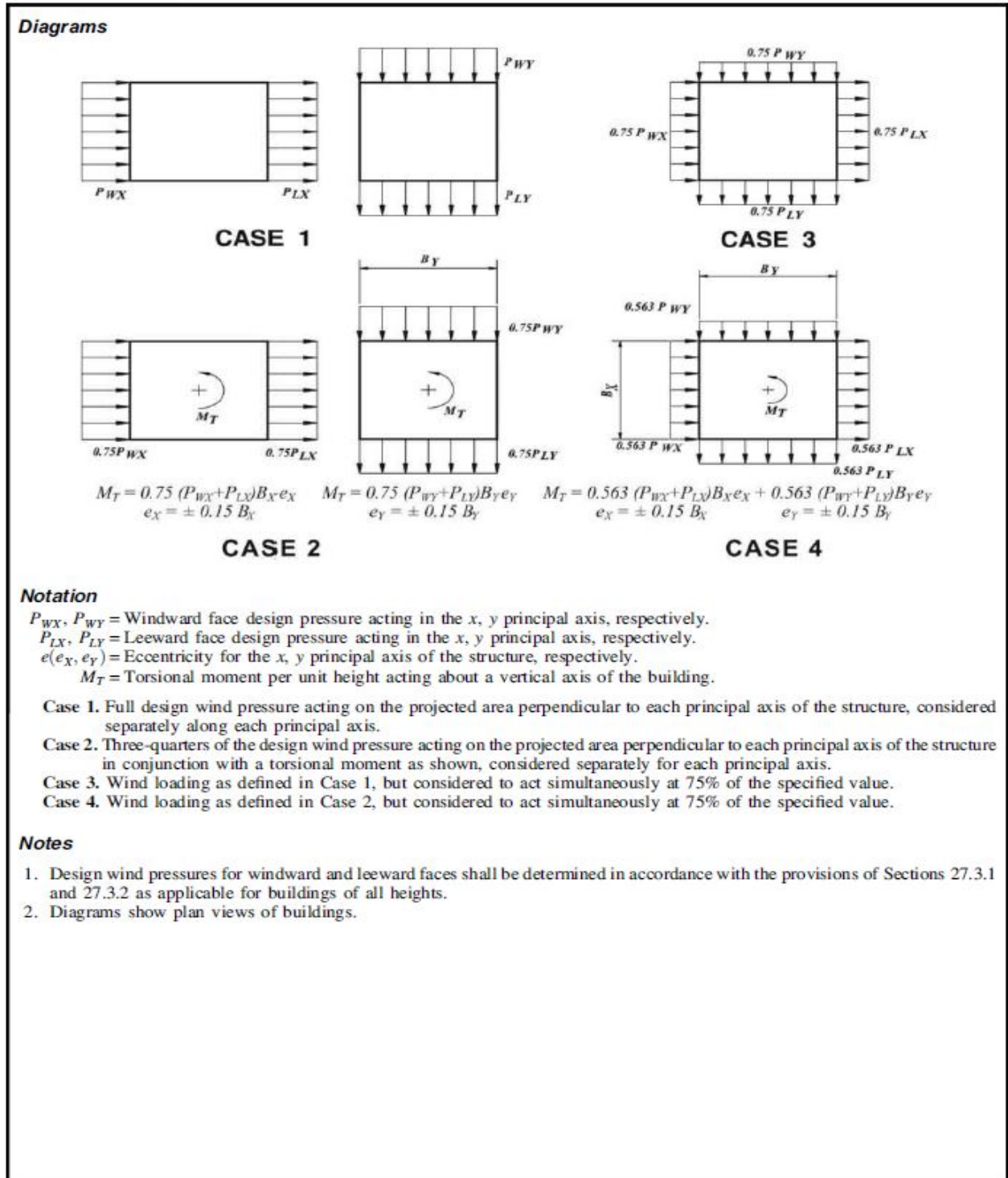### FIGURE 27.3-8: Main Wind Force Resisting System, Part 1 (All Heights): Design Wind Load Cases per ASCE 7-16

Diagrams

CASE 1

CASE 3

$$M_T = 0.75\,(P_{WX}+P_{LX})B_X \cdot e_X$$
$$e_X = \pm\,0.15\,B_X$$

$$M_T = 0.75\,(P_{WY}+P_{LY})B_Y e_Y$$
$$e_Y = \pm\,0.15\,B_Y$$

$$M_T = 0.563\,(P_{WX}+P_{LX})B_X e_X + 0.563\,(P_{WY}+P_{LY})B_Y e_Y$$
$$e_X = \pm\,0.15\,B_X \qquad e_Y = \pm\,0.15\,B_Y$$

CASE 2

CASE 4

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 - SEISMIC

ASCE 7-16 SECTION 12.8 - EQUIVALENT LATERAL FORCE PROCEDURE

RISK CATEGORY:	II	LATITUDE:	47.183
SITE CLASS:	D	LONGITUDE:	-122.296
IMPORTANCE FACTOR (I_E):	1	S_S =	1.271
STRUCTURAL SYSTEM (R):	6.5	S_1 =	0.438
OVERSTRENGTH FACTOR (Ω_o):	3	F_a =	1.200
		F_v =	1.900

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

$$S_{MS} = F_a*S_S = \quad 1.525 \qquad S_{M1} = 1.5*F_v*S_1 = \quad 1.248$$

Section 11.4.5 - Design Spectral Response Acceleration Parameters

$$S_{DS} = 2/3*S_{MS} = \quad 1.017 \qquad S_{D1} = 2/3*S_{M1} = \quad 0.832$$

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		

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.

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
	D		

PERIOD DETERMINATION:	
C_t =	0.02
h_n =	40 FT
x =	0.75
$T_a = C_t*h_n{}^x$ =	0.318

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:	$C_S = S_{D1}/(T*(R/I))$ =	0.402		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	$C_S = 0.5*S_1/(R/I)$ =			EQ. 12.8-6

ASCE 7-16 SECTION 12.8.1 - SEISMIC BASE SHEAR

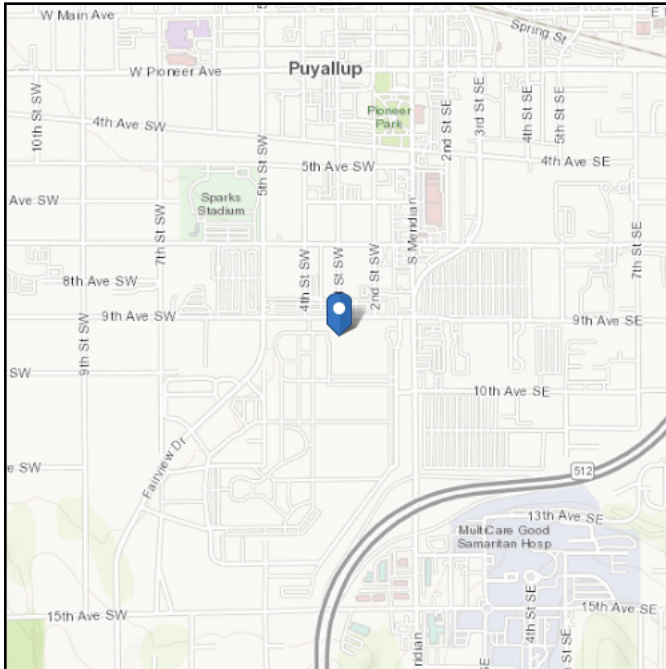$$V = C_S*W = \boxed{\textbf{0.156*W}}$$

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

ASCE Hazards Report

Address:
110 9th Ave SW
Puyallup, Washington
98371

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

Latitude: 47.183993
Longitude: -122.296295
Elevation: 44.94265765207576 ft (NAVD 88)
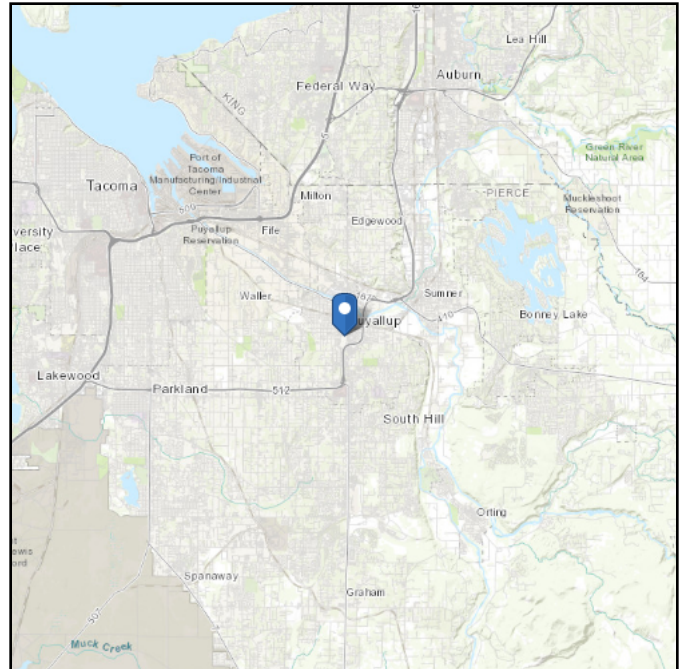
Wind

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:	Mon Jun 23 2025

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 :	1.271	S_{D1} :	N/A
S_1 :	0.438	T_L :	6
F_a :	1.2	PGA :	0.5
F_v :	N/A	PGA_M :	0.6
S_{MS} :	1.525	F_{PGA} :	1.2
S_{M1} :	N/A	I_e :	1
S_{DS} :	1.017	C_v :	1.354

Ground motion hazard analysis may be required. See ASCE/SEI 7-16 Section 11.4.8.
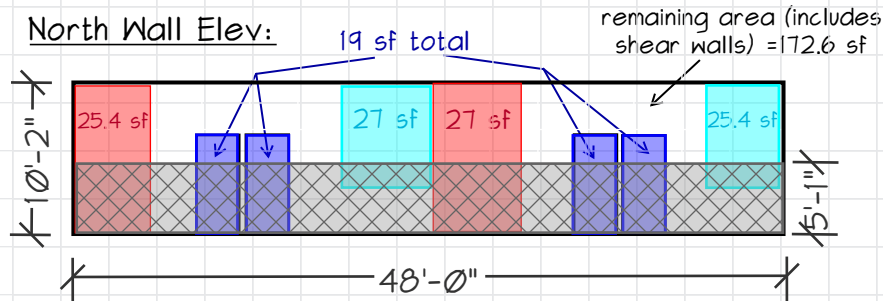
Data Accessed: Mon Jun 23 2025

Date Source: [USGS Seismic Design Maps](https://ascehazardtool.org/)

Seismic Weight of Walls:

South Wall Elev:

remaining area = 183.7 sf

25.4 sf   9.5 sf total   25.4 sf

10'-2"   48'-0"   5'-1"
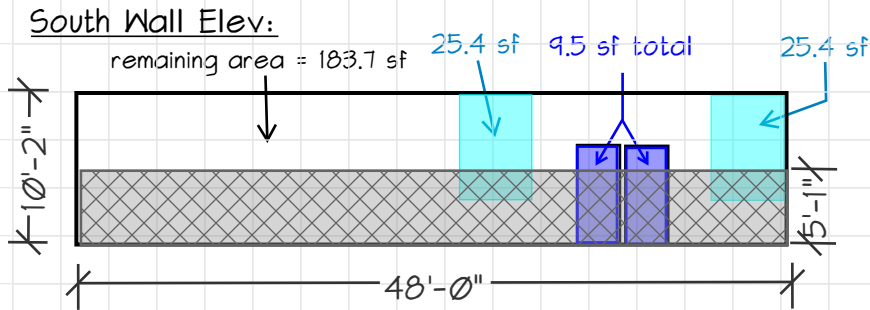
⬛ = not included in seismic weight, lower half of building

🟦 = existing solid birch door = 6.5 psf x 9.5 sf = 61.8 lb

🟦 = plate glass = 8 psf x 50.8 sf = 406.4 lb

⬜ = 2x4 studs @16" oc, 1/2" plywood sheathing, 1" cedar siding
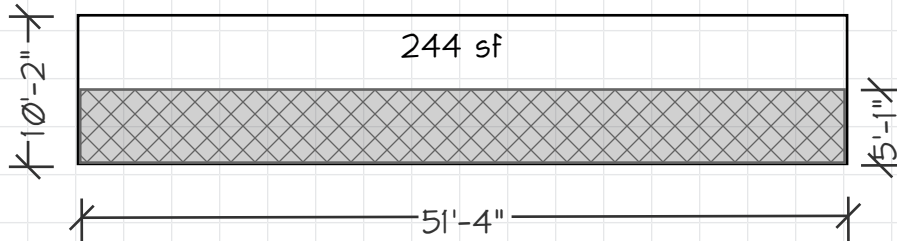
= 12 psf x 183.7 sf = 2204.4 lb

South Wall W = 2672.6 lb

North Wall Elev:

19 sf total

remaining area (includes shear walls) =172.6 sf

25.4 sf   27 sf   27 sf   25.4 sf

10'-2"   48'-0"   5'-1"

⬛ = not included in seismic weight, lower half of building

🟦 = existing solid birch door = 6.5 psf x 19 sf = 123.5 lb

🟦 = plate glass = 8 psf x 52.4 sf = 419.2 lb

⬜ = 2x4 studs @16" oc, 1/2" plywood sheathing, 1" cedar siding

🟥 = new shear wall locations 2x4 studs @16" oc, 1/2" plywood sheathing, 1" cedar siding

= 12 psf x 172.6 sf = 2071.2 lb

North Wall W = 2613.9 lb

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Tacoma   1250 Pacific Avenue, Suite 701 | Tacoma, WA 98402 | 253.383.2797
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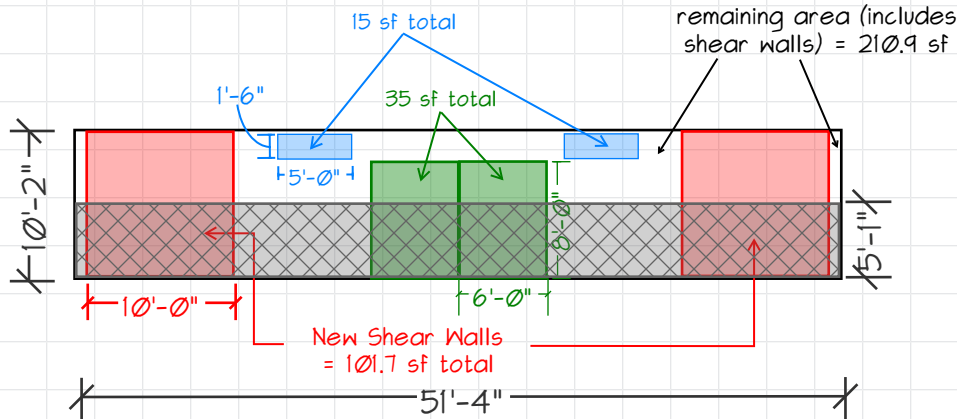## West Wall Elev:

10'-2"   5'-1"

244 sf

51'-4"

▨ = not included in seismic weight; lower half of building

☐ = 2x4 studs @16" oc, 1/2" plywood sheathing, 1" cedar siding

= 12 psf x 244 sf = 2928 lb

West Wall W = 2928 lb

East Wall Elev:

15 sf total

remaining area (includes shear walls) = 210.9 sf

1'-6"

35 sf total

5'-0"

8'-0"

10'-2"   5'-1"

10'-0"   6'-0"

New Shear Walls
= 101.7 sf total

51'-4"

▨ = not included in seismic weight; lower half of building

▧ (green) = sliding barn doors = 6.5 psf x 35 sf = 227.5 lb

▧ (blue) = windows = 8 psf x 15 sf = 120 lb

▧ (red) = shear wall locations; 2x4 studs @16" oc, 1/2"
plywood sheathing, 1" cedar siding

☐ = 2x4 studs @16" oc, 1/2" plywood sheathing, 1" cedar siding

= 12 psf x 210.9 sf = 2530.8 lb

East Wall W = 2878.3 lb

Roof weight = (15 psf)(57.3' × 48') = 41256 lb

weight for V_{NS} = 41256 lb + 2928 lb + 2878 lb = 47062 lb = W_{NS}

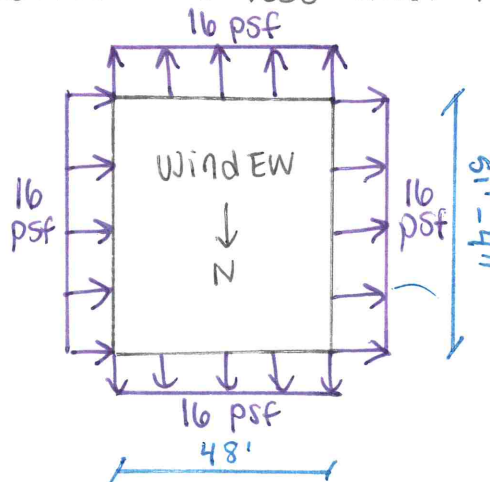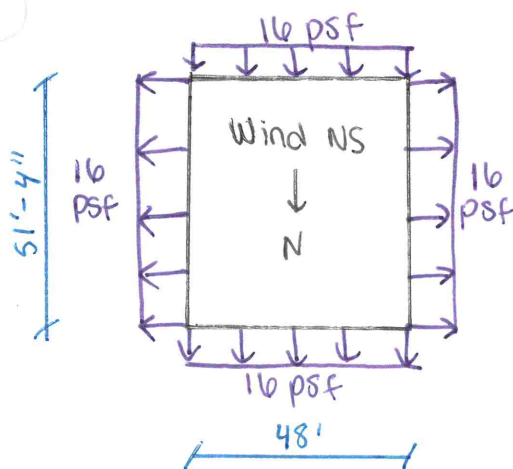weight for V_{EW} = 41256 lb + 2673 lb + 2614 lb = 46543 lb = W_{EW}

Base shear, V:
↳ ASCE 7-16 Ch 12.8.1

$V_{NS} = C_s W_{NS} = (0.156)(47062 lb) = 7.34 k \rightarrow 0.7E = 5.2 k$

$V_{EW} = C_s W_{EW} = (0.156)(46543 lb) = 7.26 k \rightarrow 0.7E = 5.1 k$

Wind
↳ ASCE 7-16 Ch 27.3.1

All calculated wind pressures were less than 16 psf ∴ use 16 psf for all walls

Wind NS diagram: 16 psf top, 16 psf left, 16 psf right, 16 psf bottom. 51'-4" height, 48' width.

Wind EW diagram: 16 psf top, 16 psf left, 16 psf right, 16 psf bottom. 51'-4" height, 48' width.

Resultant forces:
On EW face: 16 psf (51.3' × 10.16') = 8.34 k → 0.6W = 5.0 k
on NS face: 16 psf (48' × 10.16') = 7.81 k → 0.6W = 4.7 k

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Windward: Ø.6W/L = 97.9 plf

Side Wall:
Ø.6W/L = 97.4 plf

Wind Load in NS Direction

NORTH

Side Wall:
Ø.6W/L = 97.4 plf

51'-4"

48'-Ø"

Leeward: Ø.6W/L = 97.9 plf

Side Wall: Ø.6W/L = 97.9 plf

Windward:
Ø.6W/L = 97.4 plf

Wind Load in EW Direction

NORTH

Leeward:
Ø.6W/L = 97.4 plf

51'-4"

48'-Ø"

Side Wall: Ø.6W/L = 97.9 plf

Ø.7E/L = 108.3 plf

Seismic Load in
Each Direction

NORTH

Ø.7E/L = 99.4 plf

Ø.7E/L = 99.4 plf

51'-4"

48'-Ø"

Ø.7E/L = 108.3 plf

Lateral Load Summary:

	Seismic	Wind
NS	5.2 k	4.7 k
EW	5.1 k	5.Ø k

Seismic governs
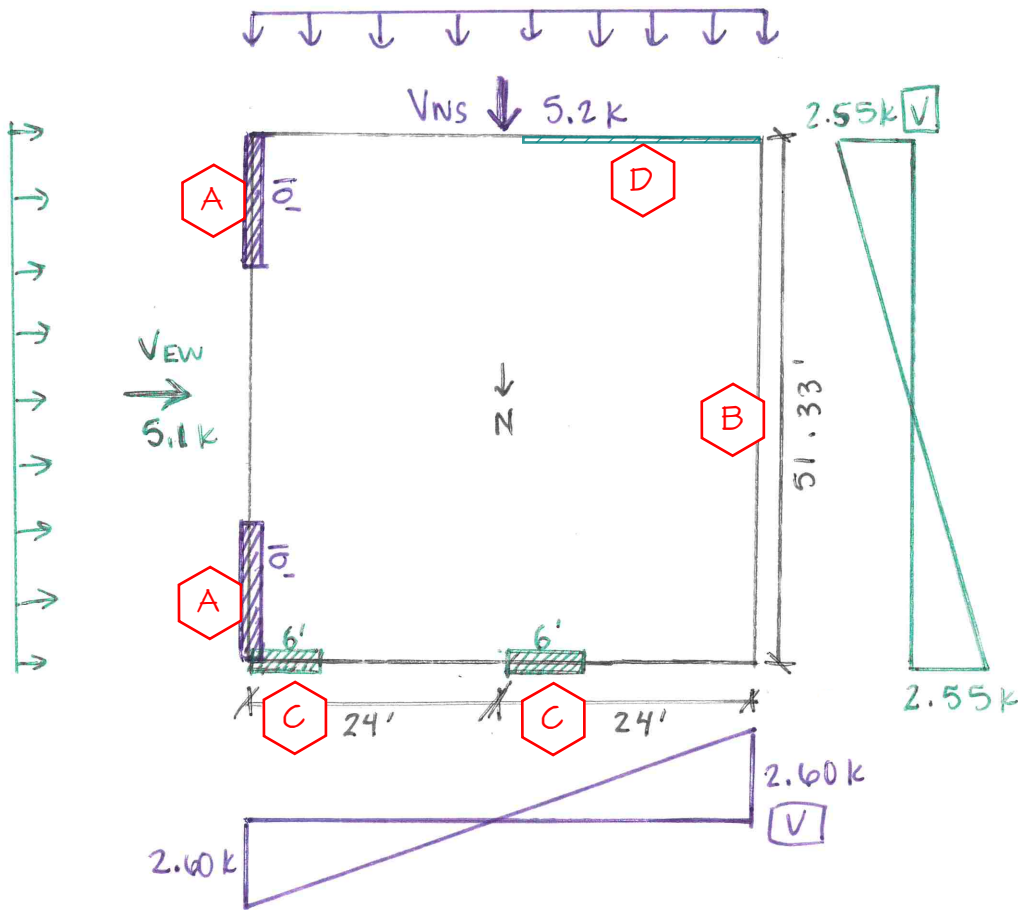
Size Sheathing for New Shear Walls

All sheathing is blocked, uses min. of 2x4 studs @ 16" oc, and is rated for exterior use

Ref: 2021 SDPWS Table 4.3A

NS walls: $\dfrac{2.6k}{20'} = 0.13$ klf = 130 plf

EW walls: $\dfrac{2.55k}{12'} = 0.2125$ klf = 212.5 plf

use **3/8"** Structural 1 sht'g 8d @ 6" o.c.

$V_{cap,\ ASD} = \dfrac{645 \text{ plf}}{2.8} = 230.4 \text{ plf}$

130 plf ≤ Vcap ✓

212.5 plf ≤ Vcap ✓

3/8" struct. 1 sht'g 8d 6"oc
w/ blk'g & 2x4 studs @ 16"oc
↳ for all shear walls (minimum)

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Tacoma   1250 Pacific Avenue, Suite 701 · Tacoma, WA 98402 · tel: 253.383.2797
Portland   101 SW Main Street, Suite 280 · Portland, OR 97204 · tel: 503.232.3746

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PCS Structural Solutions

Project: _Puyallup Fairgrounds Barn M_ Job No: _25-037_
Subject: _Lateral_ Sheet _1965 Plans_ Name: _KJH_
Originating Office: ☐ Seattle ☑ Tacoma ☐ Portland Date: _6/30/25_

Shear Walls - Overturning

NS - New Shear Walls:

$(0.13 \text{ klf})(10') = 1.3 \text{ k}$

Dead Load

1x26 GL

20'-8" trib. width

ⒶCol ⒶCol

10'-0" 10'-0" 10'-2"

51'-4"
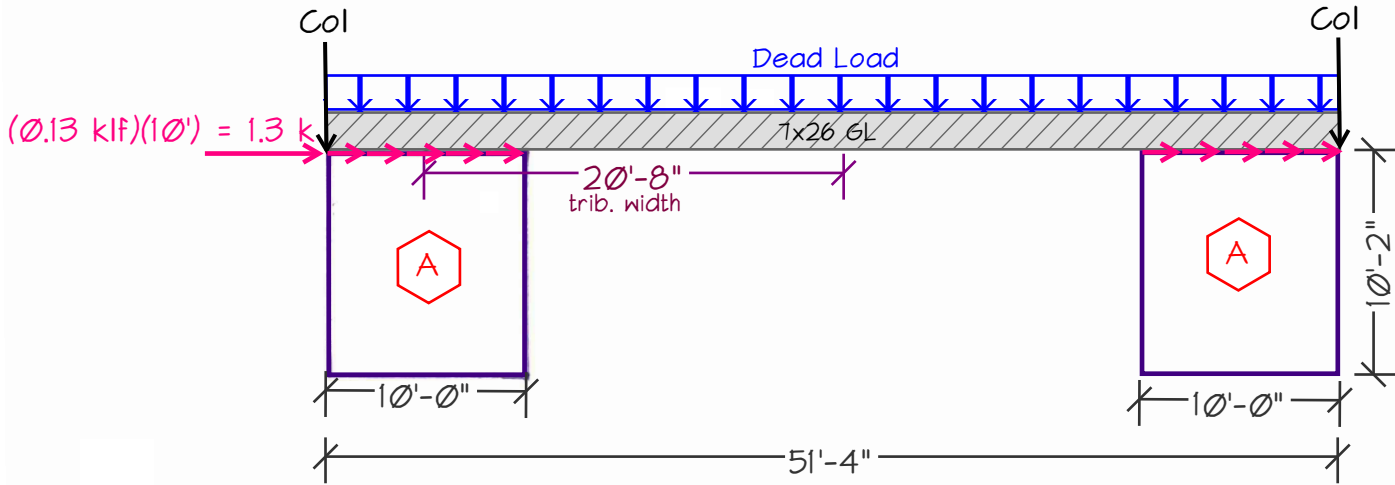
Ⓐ New NS Shear Wall (conservative w/out col)

Roof: $(15 \text{ psf})(20.67')(12') = 3.73 \text{ k}$

Wall: $(12 \text{ psf})(10')(10.16') = \underline{1.22 \text{ k}}$

$\Sigma = 4.95 \text{ k}$

factored $= (0.46)(4.95 \text{ k}) = 2.3 \text{ k}$
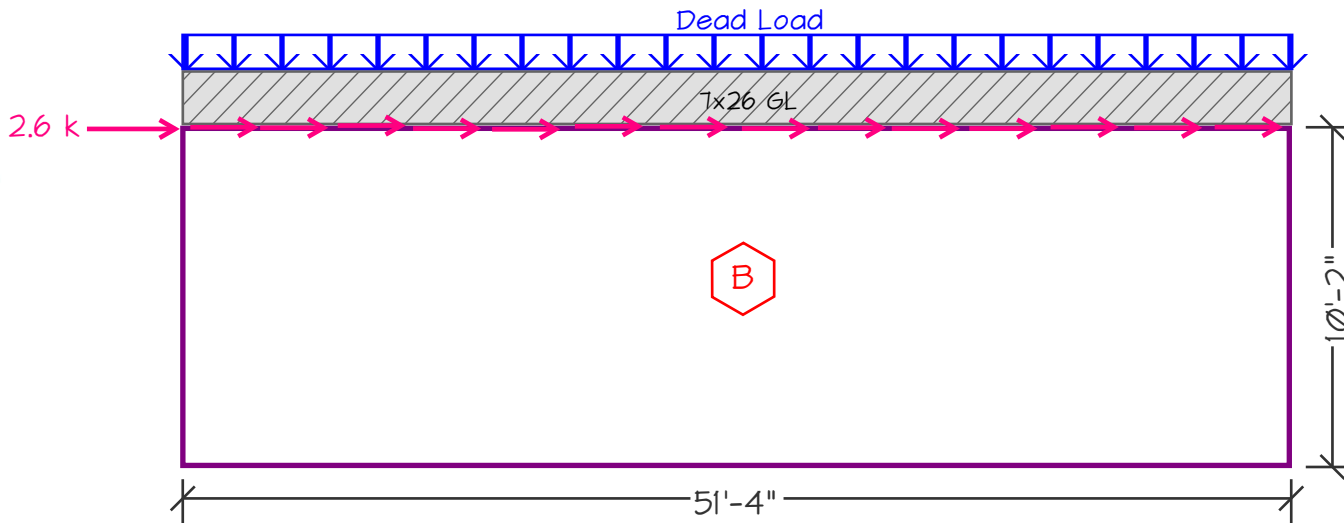
$M_{RES} = 2.3 \text{ k} \times \dfrac{10'}{2} = 11.5 \text{ k} \cdot \text{ft}$

$M_{OT} = 1.3 \text{ k} \times 10.16' = 13.2 \text{ k} \cdot \text{ft}$

$T = \dfrac{13.2 \text{ k} \cdot \text{ft} - 11.5 \text{ k} \cdot \text{ft}}{10'} = 0.17 \text{ k} = 170 \text{ lb} < 200 \text{ lb}$

$\therefore$ no holdown needed

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Tacoma 1250 Pacific Avenue, Suite 701 · Tacoma, WA 98402 · tel: 253.383.2797
Portland 101 SW Main Street, Suite 280 · Portland, OR 97204 · tel: 503.232.3746

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Project: Puyallup Fairgrounds Barn M   Job No: 25-037
Subject: Lateral   Sheet 1965 plans   Name: KJH
Originating Office: ☐ Seattle ☑ Tacoma ☐ Portland   Date: 7/2/25

NS – Existing Wall:

Dead Load

7x26 GL

2.6 k

10'-2"

51'-4"

B

B Existing Wall (conservative w/out col)

Roof: $(15 \text{ psf})(51.3')(12') = 9.2 \text{ k}$

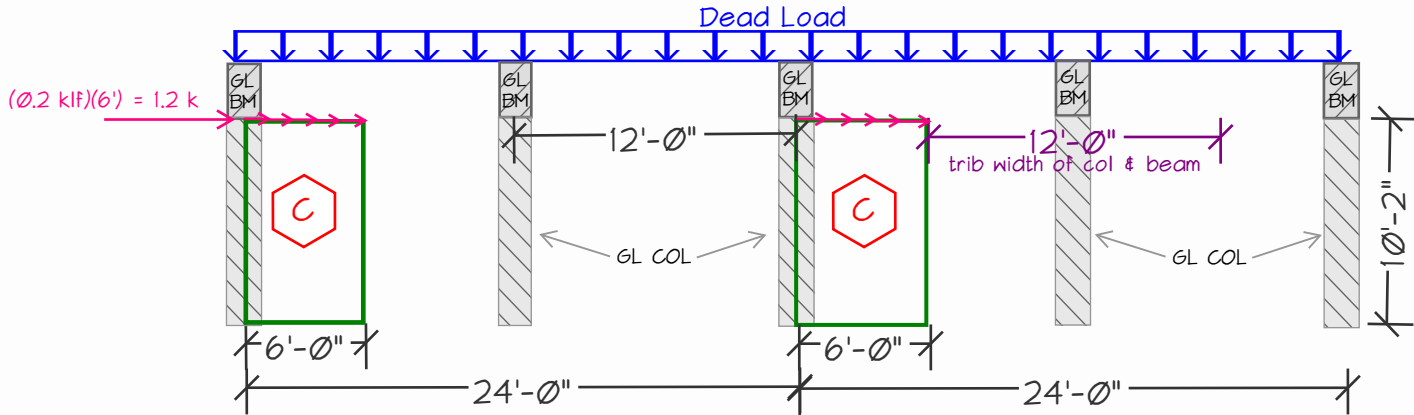Wall: $(12 \text{ psf})(51.3')(10.16') = 6.3 \text{ k}$

$$\Sigma = 15.5 \text{ k}$$

$$\text{factored} = (0.46)(15.5 \text{ k}) = 7.1 \text{ k}$$

$$M_{RES} = 7.1 \text{ k} \times \frac{51.3'}{2} = 366 \text{ k·ft}$$

$$M_{OT} = 2.6 \text{ k} \times 10.61' = 26.4 \text{ k·ft}$$

$$T = \frac{26.4 \text{ k·ft} - 366 \text{ k·ft}}{51.3'} = -6.6 \text{ k} \quad \therefore \underline{\text{no holdown needed}}$$

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EW – New Shear Walls:

Ⓒ New EW shear wall (conservative w/out col)

Roof: $(15 \text{ psf})\left(\frac{51.3'}{2}\right)(6') = 2.34 \text{ k}$

Wall: $(12 \text{ psf})(6')(10.16') = 0.74 \text{ k}$

$\Sigma = 3.1 \text{ k}$

factored $= (0.46)(3.1 \text{ k}) = 1.43 \text{ k}$

$M_{RES} = 1.43 \text{ k} \times \dfrac{6'}{2} = 4.3 \text{ k} \cdot \text{ft}$

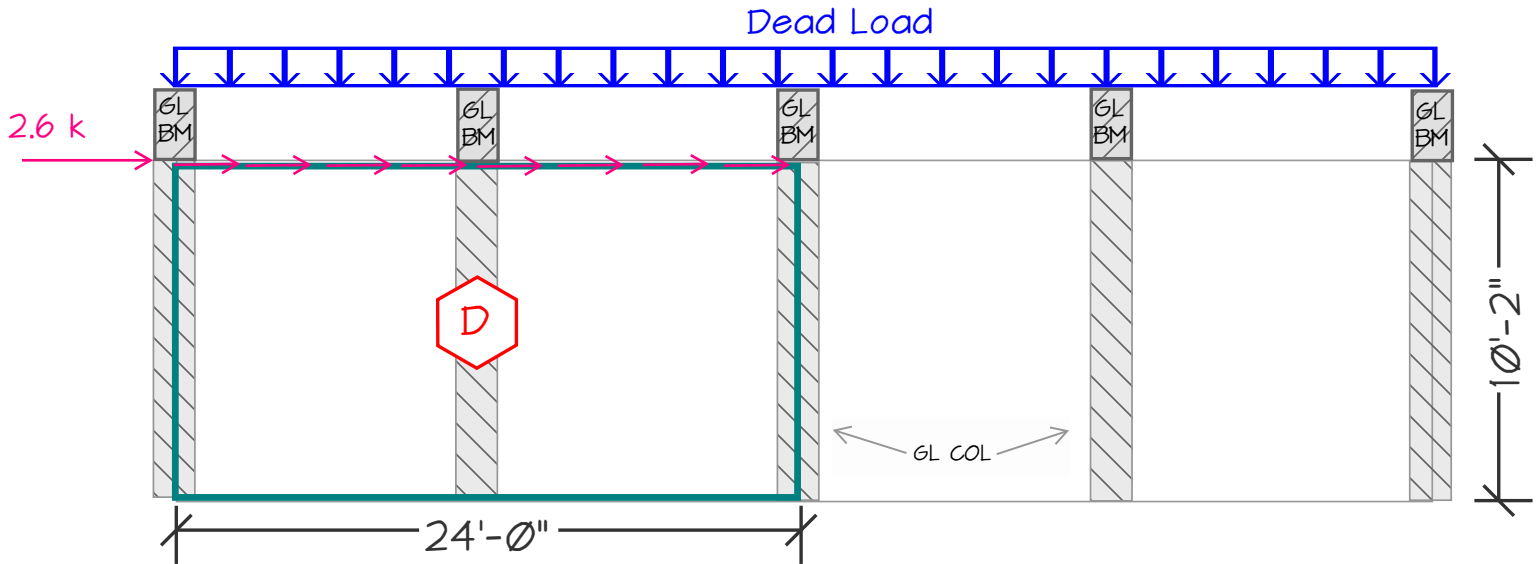$M_{OT} = 1.2 \text{ k} \times 10.16' = 12.2 \text{ k} \cdot \text{ft}$

$T = \dfrac{12.2 \text{ k} \cdot \text{ft} - 4.3 \text{ k} \cdot \text{ft}}{6'} = 1.3 \text{ k} \geq 0.2 \text{ k}$

∴ Need holdown

↳ use **HD3B** for 2×4 members

$T_{allow} = 1.895 \text{ k} \geq 1.3 \text{ k}$ ✓

EW – Existing Shear Wall:

Dead Load

2.6 k

GL BM   GL BM   GL BM   GL BM   GL BM

D

GL COL

10'-2"

24'-0"

D Existing wall (conservative w/out col)

Roof: $(15 \text{ psf})\left(\dfrac{51.3'}{2}\right)(24') = 9.23 \text{ k}$

Wall: $(12 \text{ psf})(24')(10.16') = 2.93 \text{ k}$

$$\Sigma = 12.2 \text{ k}$$

$$\text{factored} = (0.46)(12.2\text{k}) = 5.6 \text{ k}$$

$$M_{RES} = 5.6\text{k} \times \frac{24'}{2} = 67.2 \text{ k·ft}$$

$$M_{OT} = 1.2\text{k} \times 10.16' = 12.2 \text{ k·ft}$$

$$T = \frac{12.2 \text{ k·ft} - 67.2 \text{ k·ft}}{24'} = -2.3\text{k} \quad \therefore \text{ no holdown needed}$$