

STRUCTURAL CALCULATION REPORT

DESIGN CALCULATIONS FOR THE Shear Walls of 84265 Edwards Rd Milton Free water, Oregon 97862

Revision	Title
00	Design Calculations for the shear walls of 84265 Edwards Rd Milton Free water, Oregon 97862

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1. Executive Summary

This document provides the shear wall calculations for 84265 Edwards Rd Milton Free water, Oregon 97862

2. Design Data

2.1. Codes and Standards

2019 Oregon Structural Specialty Code (OSSC) or 2018 International Building Code (IBC)
ASCE 7-16
AWC NDS 2018
AWC SDPWS 2015
ACI 318-14

2.2. Material Data

Structural Lumber is Douglas-Fir Larch
Douglas-Fir #1 U.N.O. for Posts
Douglas-Fir #1 U.N.O. for Beams
Douglas-Fir #1 U.N.O. for Joists & Rafters
Douglas-Fir #2 U.N.O. for Studs
Machine Bolt = ASTM A307
Anchor Bolt = ASTM F1554
Concrete compressive strength = 2,500 psi
qa (soil allowable bearing pressure) = 2,000 psf

3. Load Estimation

3.1. Roof Loads:

Roof finishes = 8 psf
Plywood Sheathing (1/2") = 1.5 psf
Roof Framing = 2.5 psf
Insulation = 2 psf
Stucco Ceiling = 8 psf
Solar = 5 psf
Miscellaneous (Lighting, Mechanical, etc.) = 3 psf.

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Total Dead Load = 30 psf
 Roof Live Load = 20 psf

3.2. Exterior Walls Load

Stucco Siding = 8 psf.
 Plywood Sheathing (1/2") = 1.5 psf.
 2x Studs @ 16" O.C. = 1.5 psf
 R-19 Batt Insulation = 2 psf.
 Gyp. Board (1/2") = 2.5 psf.

Total Dead Load = 15.5 ~ 16 psf

3.3. Interior Walls Load

2x Studs @ 16" O.C. = 1.5 psf
 Gyp. Board (1/2") x2 = 5 psf.
 Miscellaneous (Lighting, Mechanical, etc.) = 2 psf.
 Total Dead Load = 8.5 ~ 10 psf

3.4. Seismic Weight

Exterior Walls to Roof Level = $260' * \frac{14'}{2} * 16 \text{ psf} = \underline{29,120 \text{ lbs}}$
 Interior Walls to Roof Level = $217' * \frac{14'}{2} * 10 \text{ psf} = \underline{15,190 \text{ lbs}}$
 Roof = $5,500 \text{ sf} * 30 \text{ psf} = \underline{165,000 \text{ lbs}}$
 $W_{\text{Total}} = 29,120 \text{ lbs} + 15,190 \text{ lbs} + 165,000 \text{ lbs} = \underline{209,310 \text{ lbs (approximately)}}$

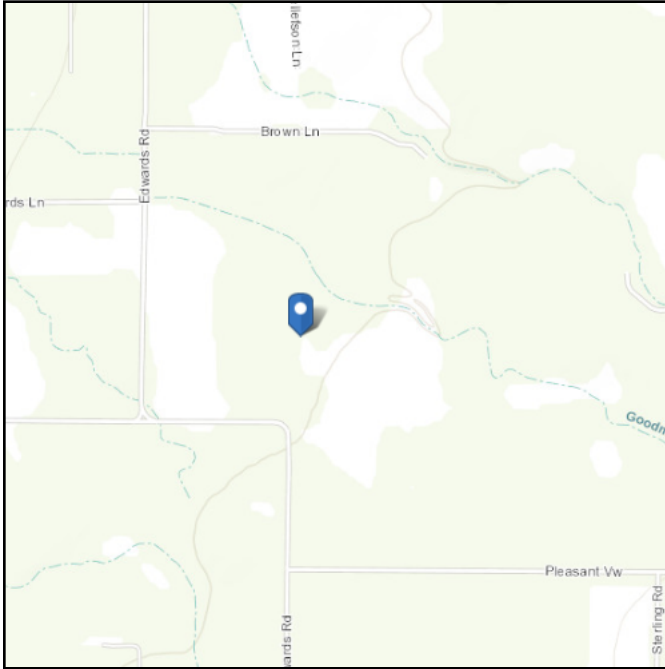
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ASCE Hazards Report

Address:
84265 Edwards Rd
Milton Freewater, Oregon
97862

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

Latitude: 45.967725
Longitude: -118.454608
Elevation: 797.4768065297557 ft
(NAVD 88)



Wind

Results:

Wind Speed	101 Vmph
10-year MRI	70 Vmph
25-year MRI	76 Vmph
50-year MRI	81 Vmph
100-year MRI	86 Vmph
300-year MRI	94 Vmph
700-year MRI	101 Vmph
1,700-year MRI	108 Vmph
3,000-year MRI	112 Vmph
10,000-year MRI	122 Vmph
100,000-year MRI	140 Vmph
1,000,000-year MRI	158 Vmph

Data Source: ASCE/SEI 7-22, Fig. 26.5-1B and Figs. CC.2-1–CC.2-4, and Section 26.5.2
Date Accessed: Mon Feb 17 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 7-22 Standard. Wind speeds correspond to approximately a 7% probability of exceedance in 50 years (annual exceedance probability = 0.00143, MRI = 700 years). Values for 10-year MRI, 25-year MRI, 50-year MRI and 100-year MRI are Service Level wind speeds, all other wind speeds are Ultimate wind speeds.

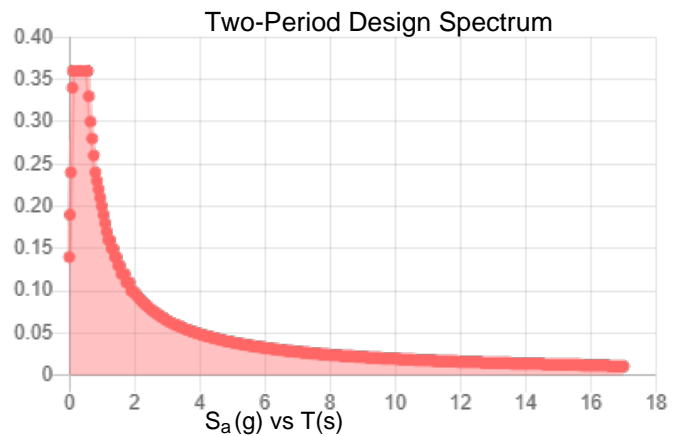
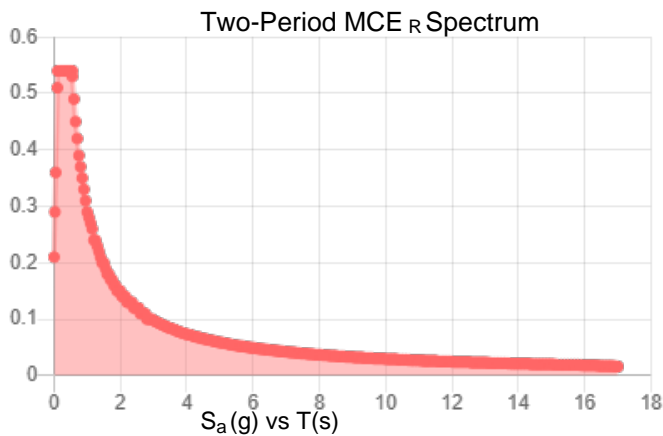
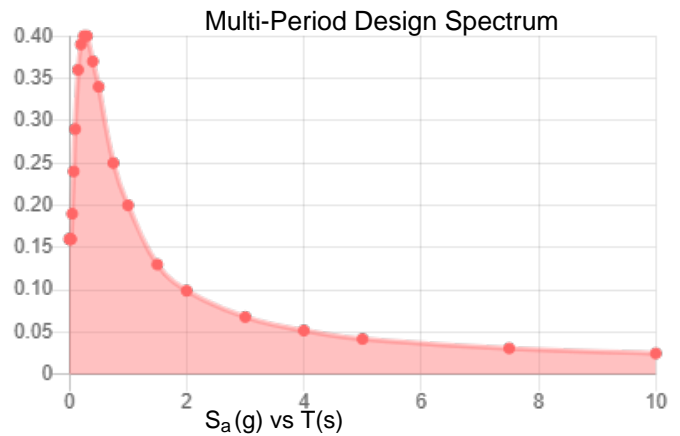
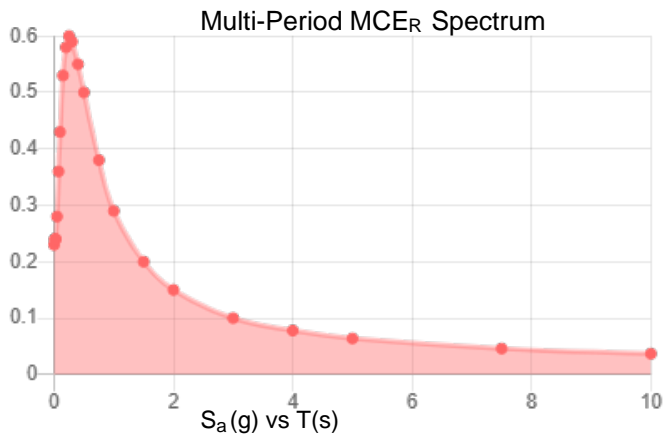
Site is not in a hurricane-prone region as defined in ASCE/SEI 7-22 Section 26.2.

Site Soil Class: Default

Results:

PGA _M :	0.22	T _L :	16
S _{MS} :	0.54	S _S :	0.39
S _{M1} :	0.29	S ₁ :	0.11
S _{DS} :	0.36	V _{S30} :	260
S _{D1} :	0.2		

Seismic Design Category: C



MCE_R Vertical Response Spectrum
Vertical ground motion data has not yet been made available by USGS.

Design Vertical Response Spectrum
Vertical ground motion data has not yet been made available by USGS.

Data Accessed: Mon Feb 17 2025

Date Source:

USGS Seismic Design Maps based on ASCE/SEI 7-22 and ASCE/SEI 7-22 Table 1.5-2. Additional data for site-specific ground motion procedures in accordance with ASCE/SEI 7-22 Ch. 21 are available from USGS.

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SEISMIC FORCES

In accordance with ASCE 7-16

Tedds calculation version 3.1.04

Site parameters

Site class	D, utilizing exception per 11.4.8(2)
Mapped acceleration parameters (Section 11.4.2)	
at short period	$S_S = 1.87$
at 1 sec period	$S_1 = 0.658$
Site coefficient at short period (Table 11.4-1)	$F_a = 1.000$
at 1 sec period (Table 11.4-2)	$F_v = 1.700$

Spectral response acceleration parameters

at short period (Eq. 11.4-1)	$S_{MS} = F_a \times S_S = 1.870$
at 1 sec period (Eq. 11.4-2)	$S_{M1} = F_v \times S_1 = 1.119$

Design spectral acceleration parameters (Sect 11.4.4)

at short period (Eq. 11.4-3)	$S_{DS} = 2 / 3 \times S_{MS} = 1.247$
at 1 sec period (Eq. 11.4-4)	$S_{D1} = 2 / 3 \times S_{M1} = 0.746$

Seismic design category

Occupancy category (Table 1-1)	III
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Seismic design category based on short period response acceleration (Table 11.6-1)

D

Seismic design category based on 1 sec period response acceleration (Table 11.6-2)

D

Seismic design category

D

Approximate fundamental period

Height above base to highest level of building	$h_n = 16.83$ ft
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From Table 12.8-2:

Structure type	All other systems
Building period parameter C_t	$C_t = 0.02$
Building period parameter x	$x = 0.75$

Approximate fundamental period (Eq 12.8-7)	$T_a = C_t \times (h_n)^x \times 1 \text{ sec} / (1 \text{ ft})^x = 0.166$ sec
Building fundamental period (Sect 12.8.2)	$T = T_a = 0.166$ sec
Long-period transition period	$T_L = 12$ sec
Limiting period	$T_S = S_{D1} / S_{DS} \times 1 \text{ sec} = 0.598$ sec

Seismic response coefficient

Seismic force-resisting system (Table 12.2-1)	A. Bearing_Wall_Systems 15. Light-frame (wood) walls sheathed with wood structural panels
Response modification factor (Table 12.2-1)	$R = 6.5$
Seismic importance factor (Table 1.5-2)	$I_e = 1.000$
Seismic response coefficient (Sect 11.4.8)	
Calculated (Eq 12.8-2)	$C_{s_calc} = S_{DS} / (R / I_e) = 0.1918$

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Minimum:

Eq. 12.8-5

$$C_{s_min1} = \max(0.044 \times S_{DS} \times I_e, 0.01) = \mathbf{0.0549}$$

Eq 12.8-6 (where $S_1 \geq 0.6$)

$$C_{s_min2} = (0.5 \times S_1) / (R / I_e) = \mathbf{0.0506}$$

$$C_{s_min} = \mathbf{0.0549}$$

Seismic response coefficient

$$C_s = \mathbf{0.1918}$$

Seismic base shear (Sect 12.8.1)

Effective seismic weight of the structure


$$W = \mathbf{210.0 \text{ kips}}$$

Seismic response coefficient

$$C_s = \mathbf{0.1918}$$

Seismic base shear (Eq 12.8-1)

$$V = C_s \times W = \mathbf{40.3 \text{ kips}}$$

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SHEAR WALL DESIGN SW1

In accordance with NDS2015 and SDPWS2015 allowable stress design and the segmented shear wall method

Tedds calculation version 1.2.11

Design summary

Description	Unit	Provided	Required	Utilization	Result
Shear capacity	lbs	2359	1299	0.551	PASS
Chord capacity	lb/in ²	521	448	0.860	PASS
Deflection	in	3.360	3.331	0.991	PASS

Panel details

Structural I wood panel sheathing on one side

Panel height

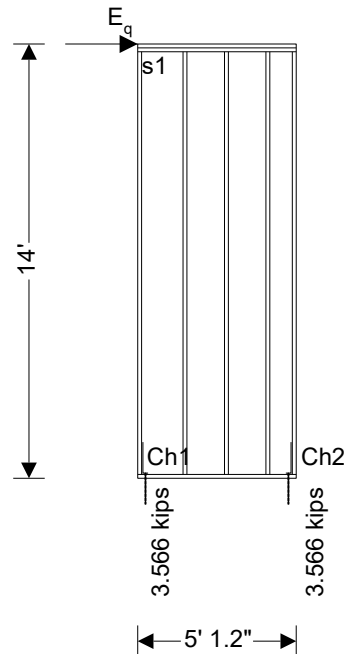
$$h = 14 \text{ ft}$$

Panel length

$$b = 5.1 \text{ ft}$$

Total area of wall

$$A = h \times b = 71.4 \text{ ft}^2$$



Panel construction

Nominal stud size

$$2" \times 6"$$

Dressed stud size

$$1.5" \times 5.5"$$

Cross-sectional area of studs

$$A_s = 8.25 \text{ in}^2$$

Stud spacing

$$s = 16 \text{ in}$$

Nominal end post size

$$2" \times 6"$$

Dressed end post size

$$1.5" \times 5.5"$$

Cross-sectional area of end posts

$$A_e = 8.25 \text{ in}^2$$

Hole diameter

$$\text{Dia} = 1 \text{ in}$$

Net cross-sectional area of end posts

$$A_{en} = 6.75 \text{ in}^2$$

Nominal collector size

$$2 \times 2" \times 6"$$

Dressed collector size

$$2 \times 1.5" \times 5.5"$$

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Service condition Dry
Temperature 100 degF or less
Vertical anchor stiffness $k_a = 30000$ lb/in

From NDS Supplement Table 4A - Reference design values for visually graded dimension lumber (2" - 4" thick)

Species, grade and size classification Douglas Fir-Larch, no.1 grade, 2" & wider
Specific gravity $G = 0.50$
Tension parallel to grain $F_t = 675$ lb/in²
Compression parallel to grain $F_c = 1500$ lb/in²
Compression perpendicular to grain $F_{c_perp} = 625$ lb/in²
Modulus of elasticity $E = 1700000$ lb/in²
Minimum modulus of elasticity $E_{min} = 620000$ lb/in²

Sheathing details

Sheathing material 15/32" wood panel structural I 5-ply plywood sheathing
Fastener type 10d common nails at 4"centers

From SDPWS Table 4.3A Nominal Unit Shear Capacities for Wood-Frame Shear Walls - Wood-based Panels

Nominal unit shear capacity for seismic design $v_s = 1020$ lb/ft
Nominal unit shear capacity for wind design $v_w = 1430$ lb/ft
Apparent shear wall shear stiffness $G_a = 20$ kips/in

Loading details


Self weight of panel $S_{wt} = 12$ lb/ft²
In plane seismic load acting at head of panel $E_q = 1856$ lbs
Design spectral response accel. par., short periods $S_{DS} = 1$

From IBC 2015 cl.1605.3.1 Basic load combinations

Load combination no.1 $D + 0.6W$
Load combination no.2 $D + 0.7E$
Load combination no.3 $D + 0.45W + 0.75L_f + 0.75(L_r \text{ or } S \text{ or } R)$
Load combination no.4 $D + 0.525E + 0.75L_f + 0.75S$
Load combination no.5 $0.6D + 0.6W$
Load combination no.6 $0.6D + 0.7E$

Adjustment factors

Load duration factor – Table 2.3.2 $C_D = 1.60$
Size factor for tension – Table 4A $C_{Ft} = 1.30$
Size factor for compression – Table 4A $C_{Fc} = 1.10$
Wet service factor for tension – Table 4A $C_{Mt} = 1.00$
Wet service factor for compression – Table 4A $C_{Mc} = 1.00$
Wet service factor for modulus of elasticity – Table 4A
 $C_{ME} = 1.00$
Temperature factor for tension – Table 2.3.3 $C_{tt} = 1.00$
Temperature factor for compression – Table 2.3.3
 $C_{tc} = 1.00$
Temperature factor for modulus of elasticity – Table 2.3.3
 $C_{tE} = 1.00$
Incising factor – cl.4.3.8 $C_i = 1.00$
Buckling stiffness factor – cl.4.4.2 $C_T = 1.00$

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Bearing area factor - cl. 3.10.4

$$C_b = 1.0$$

Adjusted modulus of elasticity

$$E_{min}' = E_{min} \times C_{ME} \times C_{IE} \times C_i \times C_T = 620000 \text{ psi}$$

Critical buckling design value

$$F_{cE} = 0.822 \times E_{min}' / (h / d)^2 = 546 \text{ psi}$$

Reference compression design value

$$F_c^* = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i = 2640 \text{ psi}$$

For sawn lumber

$$c = 0.8$$

Column stability factor – eqn.3.7-1

$$C_P = (1 + (F_{cE} / F_c^*)) / (2 \times c) - \sqrt{[(1 + (F_{cE} / F_c^*)) / (2 \times c)]^2 - (F_{cE} / F_c^*) / c} = 0.20$$

From SDPWS Table 4.3.4 Maximum Shear Wall Aspect Ratios

Maximum shear wall aspect ratio

$$3.5$$

Shear wall length

$$b = 5.1 \text{ ft}$$

Shear wall aspect ratio

$$h / b = 2.745$$

Segmented shear wall capacity

Maximum shear force under seismic loading

$$V_{s,max} = 0.7 \times E_q = 1.299 \text{ kips}$$

Shear capacity for seismic loading

$$V_s = v_s \times b \times (1.25 - 0.125 \times h / b_s) / 2 = 2.359 \text{ kips}$$

$$V_{s,max} / V_s = 0.551$$

PASS - Shear capacity for seismic load exceeds maximum shear force

Chord capacity for chords 1 and 2

Shear wall aspect ratio

$$h / b = 2.745$$

Load combination 6

Shear force for maximum tension

$$V = 0.7 \times E_q = 1.299 \text{ kips}$$

Axial force for maximum tension

$$P = 0 \text{ kips} = 0 \text{ kips}$$

Maximum tensile force in chord

$$T = V \times h / b - P = 3.566 \text{ kips}$$

Maximum applied tensile stress

$$f_t = T / A_{en} = 528 \text{ lb/in}^2$$

Design tensile stress

$$F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i = 1404 \text{ lb/in}^2$$

$$f_t / F_t' = 0.376$$

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 2

Shear force for maximum compression

$$V = 0.7 \times E_q = 1.299 \text{ kips}$$

Axial force for maximum compression

$$P = (S_{wt} \times h + 0.7 \times 0.2 \times S_{DS} \times S_{wt} \times h) \times s / 2 = 0.128 \text{ kips}$$

Maximum compressive force in chord

$$C = V \times h / b + P = 3.694 \text{ kips}$$

Maximum applied compressive stress

$$f_c = C / A_e = 448 \text{ lb/in}^2$$

Design compressive stress

$$F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P = 521 \text{ lb/in}^2$$

$$f_c / F_c' = 0.860$$

PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate

$$F_{c_perp}' = F_{c_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b = 625 \text{ lb/in}^2$$

$$f_c / F_{c_perp}' = 0.716$$

PASS - Design bearing compressive stress exceeds maximum applied bearing compressive stress

Hold down force

Chord 1

$$T_1 = 3.566 \text{ kips}$$

Chord 2

$$T_2 = 3.566 \text{ kips}$$

Seismic deflection

Design shear force

$$V_{\delta s} = E_q = 1.856 \text{ kips}$$



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Deflection limit

$$\Delta_{s_allow} = 0.020 \times h = \mathbf{3.36 \text{ in}}$$

Induced unit shear

$$v_{\delta s} = V_{\delta s} / b = \mathbf{363.92 \text{ lb/ft}}$$

Anchor tension force

$$T_{\delta} = \max(0 \text{ kips}, v_{\delta s} \times h) = \mathbf{5.095 \text{ kips}}$$

Vertical elongation at anchor

$$\Delta_a = T_{\delta} / k_a = \mathbf{0.170 \text{ in}}$$

Shear wall elastic deflection – Eqn. 4.3-1

$$\delta_{swse} = 2 \times v_{\delta s} \times h^3 / (3 \times E \times A_e \times b) + v_{\delta s} \times h / (G_a) + h \times \Delta_a / b = \mathbf{0.833 \text{ in}}$$

Deflection ampification factor

$$C_{d\delta} = \mathbf{4}$$

Seismic importance factor

$$I_e = \mathbf{1}$$

Amp. seis. deflection – ASCE7 Eqn. 12.8-15

$$\delta_{sws} = C_{d\delta} \times \delta_{swse} / I_e = \mathbf{3.331 \text{ in}}$$

$$\delta_{sws} / \Delta_{s_allow} = \mathbf{0.991}$$

PASS - Shear wall deflection is less than deflection limit