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

Contents

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.

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 = $260' * \frac{14'}{2} * 16\ psf$ = 29,120 lbs

Interior Walls to Roof Level = $217' * \frac{14'}{2} * 10\ psf$ = 15,190 lbs

Roof = $5{,}500\ sf * 30\ psf$ = 165,000 lbs

W_{Total} = $29{,}120\ lbs + 15{,}190\ lbs + 165{,}000\ lbs$ = 209,310 lbs (approximately)

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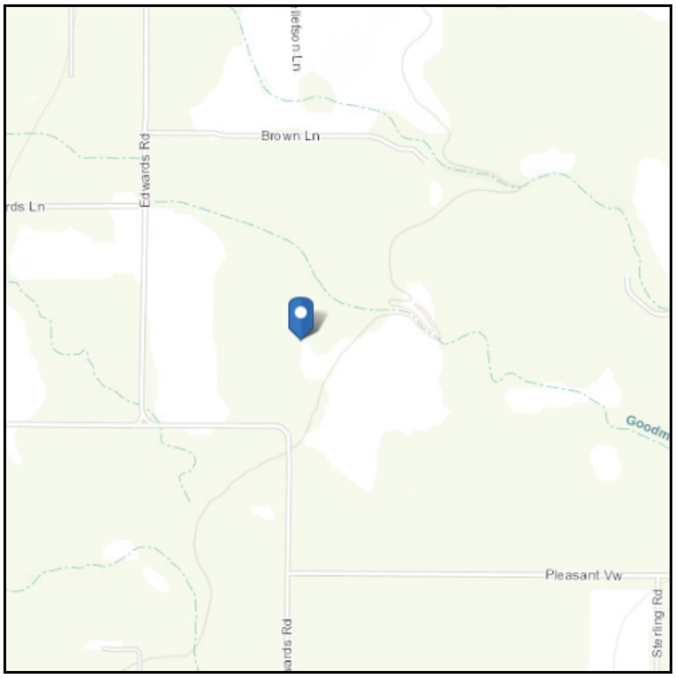
ASCE Hazards Report

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.

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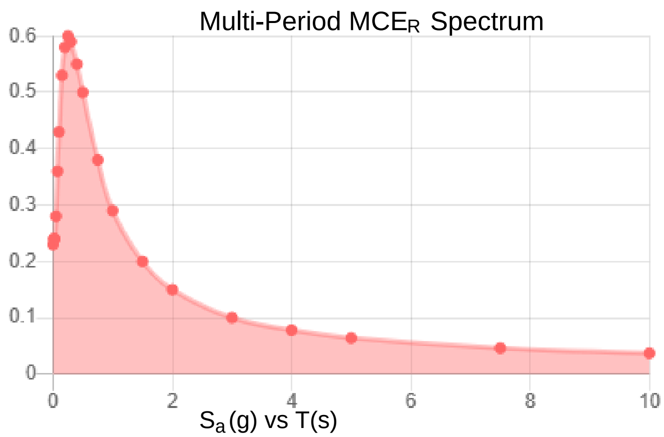
Seismic

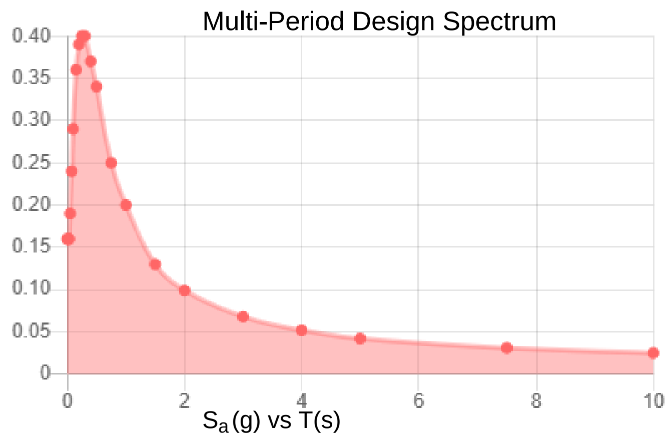
Site Soil Class: Default

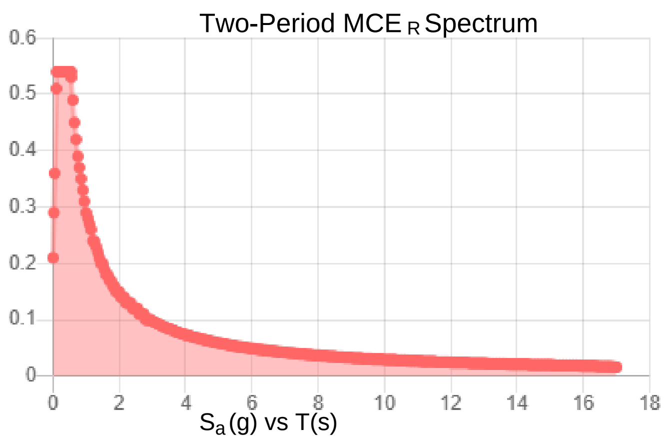
Results:

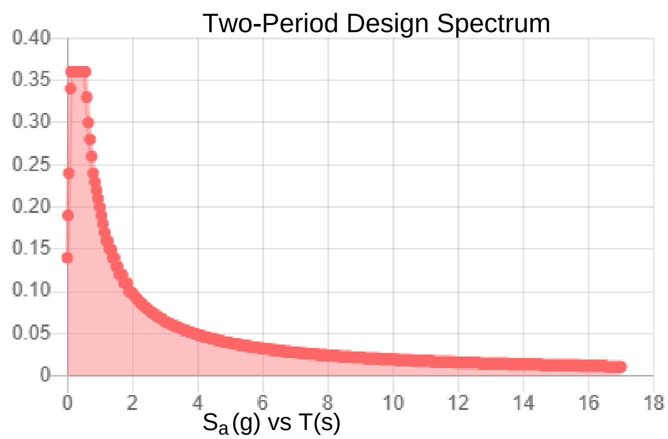
PGA_M :	0.22	T_L :	16
S_{MS} :	0.54	S_S :	0.39
S_{M1} :	0.29	S_1 :	0.11
S_{DS} :	0.36	V_{S30} :	260
S_{D1} :	0.2		

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 coefficientat 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

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

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) $T_a = C_t \times (h_n)^x \times 1sec / (1ft)^x =$ **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 $T_S = S_{D1} / S_{DS} \times 1$ 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**

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 >= 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 = **210.0** kips

Seismic response coefficient $C_s = \mathbf{0.1918}$

Seismic base shear (Eq 12.8-1) $V = C_s \times W = \mathbf{40.3}$ kips

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

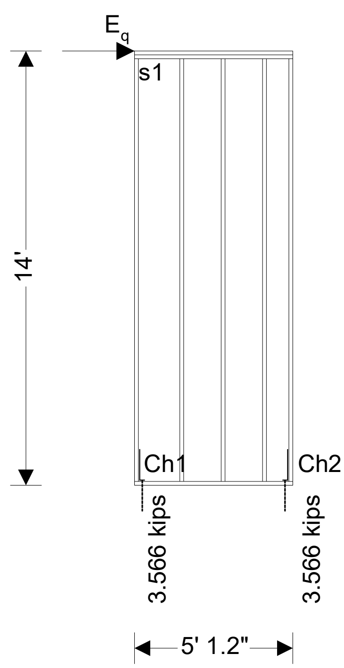
Panel details

Structural I wood panel sheathing on one side

Panel height h = **14** ft

Panel length b = **5.1** ft

Total area of wall A = h × b = **71.4** ft^2



Panel construction

Nominal stud size 2" x 6"

Dressed stud size 1.5" x 5.5"

Cross-sectional area of studs A_s = **8.25** in^2

Stud spacing s = **16** in

Nominal end post size 2" x 6"

Dressed end post size 1.5" x 5.5"

Cross-sectional area of end posts A_e = **8.25** in^2

Hole diameter Dia = **1** in

Net cross-sectional area of end posts A_{en} = **6.75** in^2

Nominal collector size 2 x 2" x 6"

Dressed collector size 2 x 1.5" x 5.5"

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/in^2
Compression parallel to grain	F_c = **1500** lb/in^2
Compression perpendicular to grain	F_{c_perp} = **625** lb/in^2
Modulus of elasticity	E = **1700000** lb/in^2
Minimum modulus of elasticity	E_{min} = **620000** lb/in^2

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/ft^2
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 or S 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**

Bearing area factor - cl. 3.10.4 C_b = **1.0**

Adjusted modulus of elasticity $E_{min}' = E_{min} \times C_{ME} \times C_{tE} \times C_i \times C_T$ = **620000** psi

Critical buckling design value $F_{cE} = 0.822 \times E_{min}' / (h / d)^2$ = **546** 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** 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** 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** kips

Shear capacity for seismic loading $V_s = v_s \times b \times (1.25 - 0.125 \times h / b_s) / 2$ = **2.359** 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** kips

Axial force for maximum tension P = 0 kips = **0** kips

Maximum tensile force in chord $T = V \times h / b - P$ = **3.566** kips

Maximum applied tensile stress $f_t = T / A_{en}$ = **528** 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** 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** 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** kips

Maximum compressive force in chord $C = V \times h / b + P$ = **3.694** kips

Maximum applied compressive stress $f_c = C / A_e$ = **448** 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** 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** 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** kips

Chord 2 T_2 = **3.566** kips

Seismic deflection

Design shear force $V_{\delta s} = E_q$ = **1.856** kips

Deflection limit $\Delta_{s\_allow} = 0.020 \times h =$ **3.36** in

Induced unit shear $v_{\delta s} = V_{\delta s} / b =$ **363.92** lb/ft

Anchor tension force $T_\delta = \max(0 \text{ kips}, v_{\delta s} \times h) =$ **5.095** kips

Vertical elongation at anchor $\Delta_a = T_\delta / k_a =$ **0.170** in

Shear wall elastic deflection – Eqn. 4.3-1 $\delta_{swse} = 2 \times v_{\delta s} \times h^3 / (3 \times E \times A_e \times b) + v_{\delta s} \times h / (G_a) + h \times \Delta_a / b =$ **0.833** in

Deflection ampification factor $C_{d\delta} =$ **4**

Seismic importance factor $I_e =$ **1**

Amp. seis. deflection – ASCE7 Eqn. 12.8-15 $\delta_{sws} = C_{d\delta} \times \delta_{swse} / I_e =$ **3.331** in

$\delta_{sws} / \Delta_{s\_allow} =$ **0.991**

PASS - Shear wall deflection is less than deflection limit

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	2933	1299	0.443	PASS
Chord capacity	lb/in^2	521	383	0.735	PASS
Deflection	in	3.360	2.536	0.755	PASS

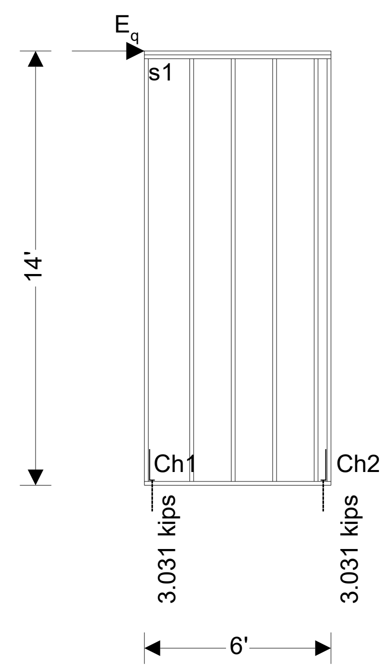
Panel details

Structural I wood panel sheathing on one side

Panel height h = **14** ft

Panel length b = **6** ft

Total area of wall A = h × b = **84** ft^2



Panel construction

Nominal stud size 2" x 6"

Dressed stud size 1.5" x 5.5"

Cross-sectional area of studs A_s = **8.25** in^2

Stud spacing s = **16** in

Nominal end post size 2" x 6"

Dressed end post size 1.5" x 5.5"

Cross-sectional area of end posts A_e = **8.25** in^2

Hole diameter Dia = **1** in

Net cross-sectional area of end posts A_{en} = **6.75** in^2

Nominal collector size 2 x 2" x 6"

Dressed collector size 2 x 1.5" x 5.5"

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/in^2
Compression parallel to grain	F_c = **1500** lb/in^2
Compression perpendicular to grain	F_{c_perp} = **625** lb/in^2
Modulus of elasticity	E = **1700000** lb/in^2
Minimum modulus of elasticity	E_{min} = **620000** lb/in^2

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/ft^2
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 or S 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**

Bearing area factor - cl. 3.10.4 C_b = **1.0**

Adjusted modulus of elasticity $E_{min}' = E_{min} \times C_{ME} \times C_{tE} \times C_i \times C_T$ = **620000** psi

Critical buckling design value $F_{cE} = 0.822 \times E_{min}' / (h / d)^2$ = **546** 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** 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 = **6** ft

Shear wall aspect ratio h / b = **2.333**

Segmented shear wall capacity

Maximum shear force under seismic loading $V_{s\_max} = 0.7 \times E_q$ = **1.299** kips

Shear capacity for seismic loading $V_s = v_s \times b \times (1.25 - 0.125 \times h / b_s) / 2$ = **2.933** kips

V_{s_max} / V_s = **0.443**

PASS - Shear capacity for seismic load exceeds maximum shear force

Chord capacity for chords 1 and 2

Shear wall aspect ratio h / b = **2.333**

Load combination 6

Shear force for maximum tension $V = 0.7 \times E_q$ = **1.299** kips

Axial force for maximum tension P = 0 kips = **0** kips

Maximum tensile force in chord $T = V \times h / b - P$ = **3.031** kips

Maximum applied tensile stress $f_t = T / A_{en}$ = **449** 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** lb/in^2

f_t / F_t' = **0.320**

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** 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** kips

Maximum compressive force in chord $C = V \times h / b + P$ = **3.159** kips

Maximum applied compressive stress $f_c = C / A_e$ = **383** 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** lb/in^2

f_c / F_c' = **0.735**

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** lb/in^2

f_c / F_{c_perp}' = **0.613**

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

Hold down force

Chord 1 T_1 = **3.031** kips

Chord 2 T_2 = **3.031** kips

Seismic deflection

Design shear force $V_{\delta s} = E_q$ = **1.856** kips

Deflection limit	Δ_{s_allow}= 0.020 × h = **3.36** in
Induced unit shear	$v_{\delta s}$ = $V_{\delta s}$ / b = **309.33** lb/ft
Anchor tension force	T_{δ} = max(0 kips,$v_{\delta s}$ × h) = **4.331** kips
Vertical elongation at anchor	Δ_a = T_{δ} / k_a = **0.144** in
Shear wall elastic deflection – Eqn. 4.3-1	δ_{swse} = 2 × $v_{\delta s}$ × h^3 / (3 × E × A_e × b) + $v_{\delta s}$ × h / (G_a) + h × Δ_a / b = **0.634** in
Deflection ampification factor	$C_{d\delta}$ = **4**
Seismic importance factor	I_e = **1**
Amp. seis. deflection – ASCE7 Eqn. 12.8-15	δ_{sws} = $C_{d\delta}$ × δ_{swse} / I_e = **2.536** in
	δ_{sws} / Δ_{s_allow} = **0.755**

PASS - Shear wall deflection is less than deflection limit