WALL DESIGN (NDS)

In accordance with NDS2018 and SDPWS2021 allowable stress design

Tedds calculation version 1.2.11

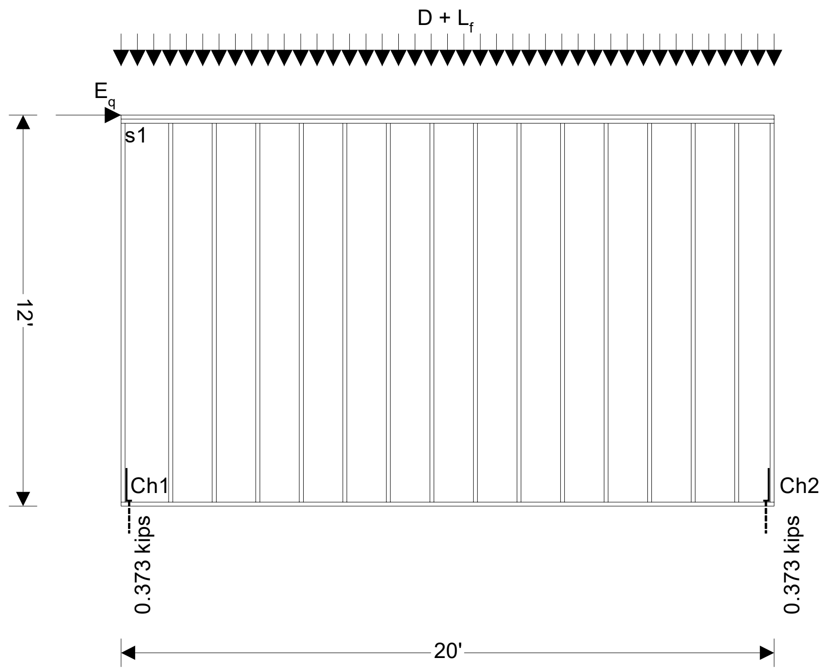
Panel details

Structural wood panel sheathing on one side

Panel height h = **12** ft

Panel length b = **20** ft

Total area of wall $A = h \times b$ = **240** ft^2



Panel construction

Nominal stud size 2" x 8"

Dressed stud size 1.5" x 7.25"

Cross-sectional area of studs A_s = **10.875** in^2

Stud spacing s = **16** in

Nominal end post size 2" x 8"

Dressed end post size 1.5" x 7.25"

Cross-sectional area of end posts A_e = **10.875** in^2

Hole diameter Dia = **1** in

Net cross-sectional area of end posts A_{en} = **9.375** in^2

Nominal collector size 2 x 2" x 8"

Dressed collector size 2 x 1.5" x 7.25"

Service condition Dry

Temperature 100 degF or less

Anchor location Inside face

Anchor offset e_{anchor} = **0** in

Vertical anchor stiffness k_a = **30000** lb/in

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

Species, grade and size classification Southern Pine, stud grade, 8" wide

Specific gravity G = **0.55**

Tension parallel to grain F_t = **325** lb/in^2

Compression parallel to grain F_c = **775** lb/in^2

Compression perpendicular to grain F_{c_perp} = **565** lb/in^2

Modulus of elasticity E = **1300000** lb/in^2

Minimum modulus of elasticity E_{min} = **470000** lb/in^2

Sheathing details

Sheathing material 5/16" wood panel oriented strandboard sheathing

Fastener type 6d common nails at 6"centers

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

Nominal unit shear capacity v_n = **505** lb/ft

Apparent shear wall shear stiffness G_a = **13** kips/in

Loading details

Dead load acting on top of panel D = **21** lb/ft

Floor live load acting on top of panel L_f = **150** lb/ft

Self weight of panel S_{wt} = **12** lb/ft^2

In plane seismic load acting at head of panel E_q = **1000** lbs

Design spectral response accel. par., short periods S_{DS} = **1**

From ASCE 7-22 - cl.2.4.1 and cl. 2.4.5 Basic combinations

Load combination no.1 D + 0.6W

Load combination no.2 D + 0.7E

Load combination no.3 D + 0.75L_f + 0.45W + 0.75L_r

Load combination no.4 D + 0.75L_f + 0.45W + 0.525S

Load combination no.5 D + 0.75L_f + 0.45W + 0.75R

Load combination no.6 D + 0.75L_f + 0.525E + 0.75S

Load combination no.7 0.6D + 0.6W

Load combination no.8 0.6D + 0.7E

Adjustment factors

Load duration factor – Table 2.3.2 C_D = **1.60**

Size factor for tension – Table 4B C_{Ft} = **1.00**

Size factor for compression – Table 4B C_{Fc} = **1.00**

Wet service factor for tension – Table 4B C_{Mt} = **1.00**

Wet service factor for compression – Table 4B C_{Mc} = **1.00**

Wet service factor for modulus of elasticity – Table 4B

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	$E_{min}' = E_{min} \times C_{ME} \times C_{tE} \times C_i \times C_T$ = **470000** psi
Critical buckling design value	$F_{cE} = 0.822 \times E_{min}' / (h / d)^2$ = **979** psi
Reference compression design value	$F_c^* = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i$ = **1240** psi
For sawn lumber	c = **0.8**
Column stability factor – eqn.3.7-1	$C_P = (1 + (F_{cE} / F_c^*)) / (2 \times c) - \sqrt{([(1 + (F_{cE} / F_c^*)) / (2 \times c)]^2 - (F_{cE} / F_c^*) / c)}$ = **0.60**

From SDPWS Table 4.3.3 Maximum Shear Wall Aspect Ratios

Maximum shear wall aspect ratio	3.5
Shear wall length	b = **20** ft
Shear wall aspect ratio	h / b = **0.6**

Segmented shear wall capacity

Maximum shear force under seismic loading	$V_{s\_max} = 0.7 \times E_q$ = **0.7** kips
Shear capacity for seismic loading	$V_s = v_s \times b / 2.8$ = **3.607** kips
	V_{s_max} / V_s = **0.194**

PASS - Shear capacity for seismic load exceeds maximum shear force

Chord capacity for chords 1 and 2

Shear wall aspect ratio	h / b = **0.6**
Effective length for chord forces	$b_{eff} = b - 3 / 2 \times b_{EndPost} - e_{anchor}$ = **19.81** ft
Load combination 8	
Shear force for maximum tension	$V = 0.7 \times E_q$ = **0.7** kips
Axial force for maximum tension	$P = (0.6 \times (D + S_{wt} \times h) - 0.7 \times 0.2 \times S_{DS} \times (D + S_{wt} \times h)) \times s / 2$ = **0.051** kips
Maximum tensile force in chord	$T = V \times h / b_{eff} - P$ = **0.373** kips
Maximum applied tensile stress	$f_t = T / A_{en}$ = **40** lb/in^2
Design tensile stress	$F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i$ = **520** lb/in^2
	f_t / F_t' = **0.077**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 2	
Shear force for maximum compression	$V = 0.7 \times E_q$ = **0.7** kips
Axial force for maximum compression	$P = ((D + S_{wt} \times h) + 0.7 \times 0.2 \times S_{DS} \times (D + S_{wt} \times h)) \times s / 2$ = **0.125** kips
Maximum compressive force in chord	$C = V \times h / b_{eff} + P$ = **0.549** kips
Maximum applied compressive stress	$f_c = C / A_e$ = **51** lb/in^2
Design compressive stress	$F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P$ = **750** lb/in^2
	f_c / F_c' = **0.067**

PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate	$F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b$ = **565** lb/in^2
	f_c / F_{c_perp}' = **0.089**

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

Hold down force

Chord 1	T_1 = **0.373** kips
Chord 2	T_2 = **0.373** kips

Seismic deflection

Design shear force $V_{\delta s} = E_q =$ **1** kips

Deflection limit $\Delta_{s\_allow} = 0.020 \times h =$ **2.88** in

Induced unit shear $v_{\delta s} = V_{\delta s} / b =$ **50** lb/ft

Anchor tension force $T_{\delta} = \max(0 \text{ kips}, v_{\delta s} \times h \times b / b_{eff} - (0.6 - 0.2 \times S_{DS}) \times (D + S_{wt} \times h) \times s / 2) =$ **0.562** kips

Chord compression force $C_{\delta} = \max(0 \text{ kips}, v_{\delta s} \times h \times b / b_{eff} + (0.6 - 0.2 \times S_{DS}) \times (D + S_{wt} \times h) \times s / 2) =$ **0.650** kips

Vertical elongation at anchor $\Delta_T = T_{\delta} / k_a =$ **0.019** in

Vertical compression at chord $\Delta_C = 0.04 \text{ in} \times C_{\delta} / (A_e \times F_{c\_perp}) =$ **0.004** in

Total vertical deflection $\Delta_a = (\Delta_T + \Delta_C) \times (b / b_{eff}) =$ **0.023** in

Shear wall elastic deflection – Eqn. 4.3-1 $\delta_{swse} = 2 \times v_{\delta s} \times h^3 / (3 \times E \times A_e \times b) + v_{\delta s} \times h / (G_a) + h \times \Delta_a / b =$ **0.062** in

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

Seismic importance factor $I_e =$ **1.25**

Amp. seis. deflection – ASCE7 Eqn. 12.8-15 $\delta_{sws} = C_{d\delta} \times \delta_{swse} / I_e =$ **0.2** in

$\delta_{sws} / \Delta_{s\_allow} =$ **0.069**

PASS - Shear wall deflection is less than deflection limit