STRUCTURAL DESIGN AND ANALYSIS
1-STORY BUILDING ADDITION

Location: 4188 Egger Dr. Fremont CA

CODES

1. 2016 California Residential Code
2. 2016 California Building Code
3. 2010 ASCE-7

TABLE OF CONTENTS

Designed By:

Antonio B Prado Jr., P.E.
Civil Engineer

REGISTERED PROFESSIONAL ENGINEER
ANTONIO B. PRADO
No. 066562
Exp. 6/30/20
CIVIL
STATE OF CALIFORNIA

1. Design of Floor Joists (8.33' Span)

A. Roof and Floor Loads

Tributary Width	=	1.33	ft.	
Roof Dead Load	=	0.00	lbs/sq.ft.	
Roof Live Load	=	0.00	lbs/sq.ft.	
Floor Dead Load	=	15.00	lbs/sq.ft.	
Floor Live Load	=	40.00	lbs/sq.ft.	(Future Live Load)
Concentrated Load, P	=	0.00	lbs	
Wall Load, Ww	=	0.00	lbs/ft.	

B. Calculate maximum forces.

Span, L	=	8.33	ft.	Simple Span
Span, L	=	0.00	ft.	Cantilever Span
Distributed Load, w	=	73.33	lbs/ft	
Max. Moment, M	=	636.06	lbs-ft.	($WL^2/8$) - simple span
Max. Moment, M	=	0.00	lbs-ft.	($WL^2/2 + PL$) - cantilever span
Max. Shear, V	=	305.43	lbs.	(WL/2 + P/2) - simple span
Max. Shear, V	=	0.00	lbs.	(WL + P) - cantilver span

C. Calculate Maximum Stresses.

Effective width, b	=	1.50	in.	
Effective depth, d	=	5.50	in.	
Section Modulus, S_x	=	7.56	in^3.	($bd^2/6$)
Max. Bending Stress, f_b	=	1,009.29	psi	(M/S_x)
Max. Shear Stress, f_v	=	37.02	psi	(V/bd)

D. Allowable Bending Stress: $F_b' = F_b \times C_D \times C_M \times C_t \times C_L \times C_F \times C_{fu} \times C_i \times C_r \times C_f \times C_v$

Unsupported Length, L_u	=	99.96	in.
Effective Depth, d	=	5.50	in.
L_u/d	=	18.17	
Effective Span, L_e	=	179.43	in.
Slenderness ratio, R_B	=	5.71	
K_{bE}	=	0.44	
F_{bE}	=	26,896.96	
F_b*	=	1,200.00	
C_L	=	1.00	

From ASB tables,

F_b' = 1,200 x 0.9 x 1.0 x 1.0 x 1.0 x 1.0 x 1.0 x 1.0 x 1.0 =

F_b' = 1,080 psi, > 1,009 psi, ok.

Floor Joists: 2 x 6 DF#2 Joists @ 16" o.c.

E. Check Deflection.

Calculate adjusted modulus of elasticity,

$E' = E \times C_m \times C_t \times C_i$ = 2,000,000 x 1.0 x 1.0 x 1.0 = 2,000,000

I = 20.80 in^4

deflection, $\Delta_L = 5W_L L^4/384E'I$ = 0.10 in.

Allowable deflection, L/240 = 0.42 in. > 0.10 in., ok.

2. Design of Girder (8' Span)

A. Roof and Floor Loads

Tributary Width	=	8.33	ft.	
Roof Dead Load	=	0.00	lbs/sq.ft.	
Roof Live Load	=	0.00	lbs/sq.ft.	
Floor Dead Load	=	15.00	lbs/sq.ft.	
Floor Live Load	=	40.00	lbs/sq.ft.	(Future Live Load)
Concentrated Load, P	=	0.00	lbs	
Wall Load, Ww	=	0.00	lbs/ft.	

B. Calculate maximum forces.

Span, L	=	8.00	ft.	Simple Span
Span, L	=	0.00	ft.	Cantilever Span
Distributed Load, w	=	458.15	lbs/ft	
Max. Moment, M	=	3,665.20	lbs-ft.	$(WL^2/8)$ - simple span
Max. Moment, M	=	0.00	lbs-ft.	$(WL^2/2 + PL)$ - cantilever span
Max. Shear, V	=	1,832.60	lbs.	(WL/2 + P/2) - simple span
Max. Shear, V	=	0.00	lbs.	(WL + P) - cantilver span

C. Calculate Maximum Stresses.

Effective width, b	=	5.50	in.	
Effective depth, d	=	7.50	in.	
Section Modulus, S_x	=	51.56	in^3.	$(bd^2/6)$
Max. Bending Stress, f_b	=	852.99	psi	(M/S_x)
Max. Shear Stress, f_v	=	44.43	psi	(V/bd)

D. Allowable Bending Stress: $F_b' = F_b \times C_D \times C_M \times C_t \times C_L \times C_F \times C_{fu} \times C_i \times C_r \times C_f \times C_v$

Unsupported Length, L_u	=	96.00	in.
Effective Depth, d	=	7.50	in.
L_u/d	=	12.80	
Effective Span, L_e	=	178.98	in.
Slenderness ratio, R_B	=	6.66	
K_{bE}	=	0.44	
F_{bE}	=	19,774.56	
F_b^*	=	1,200.00	
C_L	=	1.00	

From ASB tables,

F_b' = 1,200 x 0.9 x 1.0 x 1.0 x 1.0 x 1.0 x 1.0 x 1.0 x 1.0 =

F_b' = 1,080 psi, > 1,020 psi, ok.

Rafters: 2x6 DF#2 Rafters @ 16" o.c.
Hangers: Simpson LUS26 - sloped

E. Check Deflection.

Calculate adjusted modulus of elasticity,

$E' = E \times C_m \times C_t \times C_i$ = 2,000,000 x 1.0 x 1.0 x 1.0 = 2,000,000

I = 193.36 in^4

deflection, $\Delta_L = 5W_LL^4/384E'I$ = 0.10 in.

Allowable deflection, L/240 = 0.40 in. > 0.10 in., ok.

3. Design of Rafters (8.33' Span)

A. Roof and Floor Loads

Tributary Width	=	2.00	ft.	
Roof Dead Load	=	15.00	lbs/sq.ft.	
Roof Live Load	=	20.00	lbs/sq.ft.	
Floor Dead Load	=	0.00	lbs/sq.ft.	
Floor Live Load	=	0.00	lbs/sq.ft.	(Future Live Load)
Concentrated Load, P	=	0.00	lbs	
Wall Load, Ww	=	0.00	lbs/ft.	

B. Calculate maximum forces.

Span, L	=	8.33	ft.	Simple Span
Span, L	=	0.00	ft.	Cantilever Span
Distributed Load, w	=	70.00	lbs/ft	
Max. Moment, M	=	607.15	lbs-ft.	($WL^2/8$) - simple span
Max. Moment, M	=	0.00	lbs-ft.	($WL^2/2 + PL$) - cantilever span
Max. Shear, V	=	291.55	lbs.	(WL/2 + P/2) - simple span
Max. Shear, V	=	0.00	lbs.	(WL + P) - cantilver span

C. Calculate Maximum Stresses.

Effective width, b	=	1.50	in.	
Effective depth, d	=	5.50	in.	
Section Modulus, S_x	=	7.56	in^3.	($bd^2/6$)
Max. Bending Stress, f_b	=	963.42	psi	(M/S_x)
Max. Shear Stress, f_v	=	35.34	psi	(V/bd)

D. Allowable Bending Stress: $F_b' = F_b \times C_D \times C_M \times C_t \times C_L \times C_F \times C_{fu} \times C_i \times C_r \times C_f \times C_v$

Unsupported Length, L_u	=	99.96	in.
Effective Depth, d	=	5.50	in.
L_u/d	=	18.17	
Effective Span, L_e	=	179.43	in.
Slenderness ratio, R_B	=	5.71	
K_{bE}	=	0.44	
F_{bE}	=	26,896.96	
F_b*	=	1,200.00	
C_L	=	1.00	

From ASB tables,

F_b' = 1,200 x 0.9 x 1.0 x 1.0 x 1.0 x 1.0 x 1.0 x 1.0 x 1.0 =

F_b' = 1,080 psi, > 1,009 psi, ok.

Rafters: 2 x 6 DF#2 Joists @ 24" o.c.

E. Check Deflection.

Calculate adjusted modulus of elasticity,

$E' = E \times C_m \times C_t \times C_i$ = 2,000,000 x 1.0 x 1.0 x 1.0 = 2,000,000

I = 20.80 in^4

deflection, $\Delta_L = 5W_L L^4/384E'I$ = 0.09 in.

Allowable deflection, L/240 = 0.42 in. > 0.10 in., ok.

4. Design of Roof Beam (16.7' Span)

A. Roof and Floor Loads

Tributary Width	=	14.10	ft.	
Roof Dead Load	=	15.00	lbs/sq.ft.	
Roof Live Load	=	20.00	lbs/sq.ft.	
Floor Dead Load	=	0.00	lbs/sq.ft.	
Floor Live Load	=	0.00	lbs/sq.ft.	(Future Live Load)
Concentrated Load, P	=	0.00	lbs	
Wall Load, Ww	=	0.00	lbs/ft.	

B. Calculate maximum forces.

Span, L	=	16.70	ft.	Simple Span
Span, L	=	0.00	ft.	Cantilever Span
Distributed Load, w	=	493.50	lbs/ft	
Max. Moment, M	=	17,204.03	lbs-ft.	($WL^2/8$) - simple span
Max. Moment, M	=	0.00	lbs-ft.	($WL^2/2 + PL$) - cantilever span
Max. Shear, V	=	4,120.73	lbs.	(WL/2 + P/2) - simple span
Max. Shear, V	=	0.00	lbs.	(WL + P) - cantilver span

C. Calculate Maximum Stresses.

Effective width, b	=	5.50	in.	
Effective depth, d	=	11.88	in.	
Section Modulus, S_x	=	129.26	in^3.	($bd^2/6$)
Max. Bending Stress, f_b	=	1,597.10	psi	(M/S_x)
Max. Shear Stress, f_v	=	214.06	psi	(V/bx3.5") - notched beam

D. Allowable Bending Stress: $F_b' = F_b \times C_D \times C_M \times C_t \times C_L \times C_F \times C_{fu} \times C_i \times C_r \times C_f \times C_v$

Unsupported Length, L_u	=	200.40	in.
Effective Depth, d	=	11.88	in.
L_u/d	=	16.88	
Effective Span, L_e	=	362.28	in.
Slenderness ratio, R_B	=	11.93	
K_{bE}	=	0.44	
F_{bE}	=	6,170.18	
F_b*	=	2,900.00	
C_L	=	0.96	

From ASB tables,

F_b' = 2,900 x 0.9 x 1.0 x 1.0 x 1.0 x 1.0 x 1.0 x 1.0 x 0.96 =

F_b' = 2,500 psi, > 1,597 psi, ok.

Roof Beam: 5.5x11-7/8 Parallam Beam 2.2E
Post Cap: CC66, CCE66

E. Check Deflection.

Calculate adjusted modulus of elasticity,

$E' = E \times C_m \times C_t \times C_i$ = 2,000,000 x 1.0 x 1.0 x 1.0 = 2,000,000

I = 767.51 in^4

deflection, $\Delta_L = 5W_LL^4/384E'I$ = 0.29 in.

Allowable deflection, L/240 = 0.84 in. > 0.28 in., ok.

5. Seismic Analysis:

A. Seismic Analysis: Parameters by USGS

Basic Parameters

Name	Value	Description
S_S	2.043	MCE_R ground motion (period=0.2s)
S_1	0.784	MCE_R ground motion (period=1.0s)
S_{MS}	2.043	Site-modified spectral acceleration value
S_{M1}	* null	Site-modified spectral acceleration value
S_{DS}	1.362	Numeric seismic design value at 0.2s SA
S_{D1}	* null	Numeric seismic design value at 1.0s SA

Weight Structure

Roof Deadload, DL_1	=	16,620.00	lbs	(1,108 sq.ft. x 15 lbs/sq.ft.)
Roof Flr. Wall Deadload, DL_2	=	21,525.00	lbs	(205 ft. x 210 lbs/ft. / 2) - half wall height
2nd Flr Deadload, DL_3	=	0.00	lbs	
2nd Flr. Wall Deadload, DL_4	=	0.00	lbs	

Seismic Parameters

Site Class,		=	D	SDC =	E
Site Acceleration,	S_s	=	2.043	g	
Site Acceleration,	S_1	=	0.784	g	
Site Coefficient,	F_a	=	1		
Site Coefficient,	F_v	=	n/a		
Acceleration Coef.	S_{MS}	=	2.043	g	
Acceleration Coef.	S_{M1}	=	n/a	g	
Design Acc. Coef.	S_{DS}	=	1.362	g	
Design Acc. Coef.	S_{D1}	=	n/a	g	

Approximate Fundamental Period (Section 12.8.2)

Height of Structure, ht	=	15.00	ft.
Structure Type	=	All other systems	
Building Period Parameter, Ct	=	0.02	
Building Period Parameter, x	=	0.75	
Approximate Fundamental Period, T_a	=	0.152	sec
Building Fundamental Period, T	=	T_a	
	=	0.152	sec
Long Transition Period, T_L	=	12.00	sec

Seismic Response Coefficient

Seismic Force-Resisting System		A. Bearing Wall System		
Structure Type		12. Light-frame (wood) sheathed with wood structural panels		
Response modification factor, R	=	6.50		
Seismic Importance Factor, I_e	=	1.00		
Seismic Response Coefficients, C_s	=	0.210	- governs	
Seismic Response Coefficients, C_{smax}	=	n/a		
Seismic Response Coefficients, C_{smax}	=	0.599	sec	
Seismic Response Coefficients, C_{smin}	=	0.060	sec	

Seismic Base Shear

Wt. of Structure, W	=	38.15	kips	$(DL_1+DL_2+DL_3+DL_4)$
Seismic Response Coef., C_s	=	0.210		
$V = C_s W \times 0.7$	=	5.59	kips (Seismic Governs)	

Vertical Distribution of Seismic Forces - n/a

6. Design of Shear Wall (SW-1) - new wall between bedroom 4 and dining room

SW-1	Length,	L	=	16.30	ft.	
	Shear,	V_1	=	2,797.50	lbs	(Base shear V/2)
	Shear/Unit Length, s_1		=	171.63	lbs/ft	

Use 15/32 OSB Structural 1 on 2x4 stubs with 10d nails @ 6" o.c. on edges, 12" o.c. in field.
(Capacity = 680 lbs/ft x 0.5 = 340 lbs/ft) - okay

Design of hold down

Overturning Force, F	=	2.80	kips	
Height of shear wall, h	=	9	ft	
Overturning Moment, M	=	25,177	lbs-ft	
Resisting Deadload, DL x 0.6	=	3,227.40	lbs	Roof and wall dead load
Resisting Moment Arm, Lr	=	15.80	ft	(16.3' - 0.5 ft)
Resisting Moment, Mr	=	25,496.46	lbs-ft	
Hold down tension, T	=	-20.19	lbs	no hold down is needed

Hold down: none
Use the same on parallel shear walls to the left and right of this wall

Design of Shear Wall (SW-1) - existing wall between kitchen and laundry

SW-1	Length,	L	=	24.42	ft.	
	Shear,	V_1	=	2,797.50	lbs	(Base shear V/2)
	% Full Ht. Sheating		=	75%		(24.42-6)/24.42)
	Shear Capacity Adjustment Factor, Co		=	0.73		(Opening ht. = 6'-8")
	Shear/Unit Length, s_1		=	156.95	lbs/ft	

Use 15/32 OSB Structural 1 on 2x4 stubs with 10d nails @ 6" o.c. on edges, 12" o.c. in field.
(Capacity = 680 lbs/ft x 0.5 = 340 lbs/ft) - okay

Design of hold down

Overturning Force, F	=	2.80	kips	
Height of shear wall, h	=	9	ft	
Overturning Moment, M	=	25,177	lbs-ft	
Resisting Deadload, DL x 0.6	=	5,383.88	lbs	Roof and wall dead load
Resisting Moment Arm, Lr	=	23.92	ft	(24.2' - 0.5 ft)
Resisting Moment, Mr	=	64,382.17	lbs-ft	
Hold down tension, T	=	-1,639.22	lbs	no hold down is needed

Hold down: none

Design of Shear Wall (SW-2) - new front room exterior wall

SW-1	Length,	L	=	12.00	ft.	
	Shear,	V_1	=	1,398.75	lbs	(Base shear V/4)
	% Full Ht. Sheating		=	33%		(12-8)/12
	Shear Capacity Adjustment Factor, Co		=	0.72		(Opening ht. = 4')
	Shear/Unit Length, s_1		=	161.89	lbs/ft	

Use 15/32 OSB Structural 1 on 2x4 stubs with 10d nails @ 6" o.c. on edges, 12" o.c. in field.
(Capacity = 680 lbs/ft x 0.5 = 340 lbs/ft) - okay

Design of hold down

Overturning Force, F	=	1.40	kips	
Height of shear wall, h	=	9	ft	
Overturning Moment, M	=	12,589	lbs-ft	
Resisting Deadload, DL x 0.6	=	498.96	lbs	Roof and wall dead load
Resisting Moment Arm, Lr	=	11.50	ft	(12' - 0.5 ft)
Resisting Moment, Mr	=	2,869.02	lbs-ft	
Hold down tension, T	=	845.19	lbs	

DTT2Z = 1,825 lbs w/ SSTB16 cap. = 2,550 lbs
Use the same on new dining room front wall