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LAGUNA NIGUEL ADUs

PLAN 3

LAGUNA NIGUEL, CA

RRM Project No. 2889-01-CU22

STRUCTURAL CALCULATIONS

January 10, 2025

Prepared by: Ian Kelly EIT

Under Direction of: Jessica Meadows, SE

REGISTERED PROFESSIONAL ENGINEER
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LAGUNA NIGUEL ADUs
PLAN 2
LAGUNA NIGUEL, CA
RRM Project No. 2889-01-CU22

Index of Structural Calculations

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		PROJECT NO.:	2889-01-CU22
		DATE:	01/2025
PROJECT:	LAGUNA NIGUEL ADUs	BY:	IPK
SUBJECT:	PROJECT DESIGN CRITERIA	CHECKED BY:	JMM

PROJECT DESIGN CRITERIA

PROJECT LOCATION:

Address: PLAN 1
LAGUNA NIGUEL, CA

Latitude: N 33.498759°
Longitude: W 117.733465°

DESIGN CODES:

Notation	Code/ Standard Reference	Title
A	ASCE 7-16	Minimum Design Loads for Buildings and Other Structures
C	2022 CBC	2022 California Building Code
I	ACI 318-19	2019 Building Code Requirements for Structural Concrete
M	TMS 402/ 602-16	2016 Building Code Requirements and Specification for Masonry Structures
S	AISC 360-16	2016 Steel Construction Manual 15th Edition
W	2018 NDS	2018 National Design Specification for Wood Construction
SDP	2021 SDPWS	2021 Seismic Deisgn Provisions for Wind and Seismic (American Wood Council)

MATERIAL REFERENCES:

Notation	Material Reference	Title
B		Design of Wood Structures ASD/LRFD, Breyer, 6th Edition
T	TJ-4000	Weyerhaeuser Specifier's Guide

PROJECT DESCRIPTION:

The calculations herewith represent the following structures as described:

Pre-Approved Accessory Dwelling Units:

Configuration: Accessory Dwelling Units (ADUs)

Gravity System: Plywood sheathing spans to pre-manufactured wood trusses at roof. Wood framing bears on wood beams, wood stud bearing walls and wood headers. Wood beams & headers span to wood posts or king studs.

Lateral System: Plywood sheathed diaphragm distributes lateral load to collectors. Continuous chords resist diaphragm tension and compression loads. Collectors drag lateal loads to plywood sheathed wood stud shear walls. Shear walls distribute shear lateral load to foundation thru sill anchor bolts and overturning lateral load thru end columns and uplift connectors.

Foundation System: Shallow continuous concrete strip footings support bearing walls.

PROJECT: LAGUNA NIGUEL ADUs
SUBJECT: PROJECT DESIGN CRITERIA

PROJECT DESIGN CRITERIA

GEOTECHNICAL PARAMETERS:

Geotechnical Report:
Prepared by: N/A

Project No: N/A
Dated: N/A

Bearing Pressure:

Allowable Bearing Pressure:	1500	psf	Presumptive load bearing values
Passive Earth Pressure:	100	psf/ft	per 2022 CBC Table 1806.2
Friction Coefficient:	0.25		
Friction + Passive:	100% + 100%		

Retaining Walls:

Allowable Bearing Pressure:	N/A	psf
Passive Earth Pressure:	N/A	psf/ft
Friction Coefficient:	0.25	
Friction + Passive:	100% + 100%	

Active Lateral Earth Pressure:	N/A	psf/ft for Level Backfill
Increase for Slope:	N/A	psf/ft per degree of incline
At Rest Lateral Earth Pressure:	N/A	psf/ft for Level Backfill
Increase for Slope:	N/A	psf/ft per degree of incline
Seismic Active Earth Pressure:	N/A	psf/ft for Level Backfill
Seismic At-Rest Earth Pressure:	N/A	psf/ft for Level Backfill

PROJECT NO.: 2889-01-CU22
DATE: 12/2024
BY: IPK
CHECKED BY: JMM

PROJECT: LAGUNA NIGUEL ADUs
SUBJECT: PROJECT DEAD LOADS - ROOF

PROJECT DEAD LOADS - ROOF

ROOF: Clay Tile Over Trusses w/ Gyp Ceiling & PV Panels — 5/12 *Roof Slope*

Dead Load Occurs Yes

Thick	Spacing	Description	Base Weight (psf)	Slope (deg)	Factor	Layers	Total Weight (psf)	Code/Standard Reference
		Clay Tile	10	22.6	1.08	1	10.8	A Table C3-1
		PV Panels	4	22.6	1.08	1	4.3	
		Miscellaneous	1	22.6	1.08	1	1.1	
					Roof & Re-Roof Total =		16.3	Some Jurisdictions
15/32 "		Sheathing	3	22.6	1.08	1	1.5	B Appendix B
10.0 "		Batt Insulation	0.075	22.6	1.08	1	0.8	B Appendix B
5/8 "		Gyp. Ceiling	4.48	0.0	1.00	1	2.8	B Appendix B
		Lights/Ducts/Sprinklers	2	0.0	1.00	1	2.0	B Appendix B
					Joist Super Load =		23.4	
		Roof Trusses @ 24" OC						
	24 "	2x6 Top Chord	0.95	22.6	1.08	1	1.0	B Appendix A
	24 "	2x4 Web	0.6	22.6	1.08	1	0.7	B Appendix A
	24 "	2x6 Bottom Chord	0.95	0	1.00	1	1.0	B Appendix A
					Total Dead Load =		26.0	27.0 rounded

Live Load

		26. Roofs	20.0	C Table 1607.1
		Live Load =	20.0	

ROOF: Clay Tile Over Trusses w/ Stucco Ceiling — 5/12 *Roof Slope*

Dead Load Occurs Yes

Thick	Spacing	Description	Base Weight (psf)	Slope (deg)	Factor	Layers	Total Weight (psf)	Code/Standard Reference
		Clay Tile	10	22.6	1.08	1	10.8	A Table C3-1
		PV Panels	4	22.6	1.08	1	4.3	
		Miscellaneous	1	22.6	1.08	1	1.1	
					Roofing and Re-Roofing Total =		16.3	Some Jurisdictions
15/32 "		Sheathing	3	22.6	1.08	1	1.5	B Appendix B
		Stucco Ceiling	10.4	0	1.00	1	10.4	B Appendix B
		Lights/Ducts/Sprinklers	2	0.0	1.00	1	2.0	B Appendix B
					Joist Super Load =		30.2	
		Roof Trusses @ 24" OC						
	24 "	2x6 Top Chord	0.95	22.6	1.08	1	1.0	B Appendix A
	24 "	2x4 Web	0.6	22.6	1.08	1	0.7	B Appendix A
	24 "	2x6 Bottom Chord	0.95	0	1.00	1	1.0	B Appendix A
					Total Dead Load =		32.8	33.0 rounded

Live Load

		26. Roofs	20.0	C Table 1607.1
		Live Load =	20.0	

PROJECT: LAGUNA NIGUEL ADUs
SUBJECT: PROJECT DEAD LOADS - WALLS

PROJECT DEAD LOADS - WALLS

WALL: Plaster/Stucco Exterior Wall (2x6) Occurs?: Yes

Thick	Spacing	Description	Base Weight (psf)		Layers	Total Weight (psf)	Code/Standard Reference
		Stucco	10.4		1	10.4	B Appendix B
15/32 "		Sheathing	3		1	1.4	B Appendix B
	16 "	2x6 Stud Wall	1.425		1	1.4	B Appendix B
5 1/2 "		Insulation	0.04		1	0.2	B Appendix B
5/8 "		Gyp. Board	5		1	3.1	B Appendix B
		Miscellaneous	1		1	1.0	B Appendix B
					Total Wall Load =	17.6	17.6 rounded

WALL: Plaster/Stucco Exterior Wall (2x4) Occurs?: Yes

Thick	Spacing	Description	Base Weight (psf)		Layers	Total Weight (psf)	Code/Standard Reference
		Stucco	10.4		1	10.4	B Appendix B
15/32 "		Sheathing	3		1	1.4	B Appendix B
	16 "	2x4 Stud Wall	0.9		1	0.9	B Appendix B
3 1/2 "		Insulation	0.04		1	0.1	B Appendix B
5/8 "		Gyp. Board	5		1	3.1	B Appendix B
		Miscellaneous	0.5		1	0.5	B Appendix B
					Total Wall Load =	16.4	17.0 rounded

WALL: Non-Bearing Partition Wall (2x4) Occurs?: Yes

Thick	Spacing	Description	Base Weight (psf)		Layers	Total Weight (psf)	Code/Standard Reference
	16 "	2x4 Stud Wall	0.9		1	0.9	B Appendix B
5/8 "		Gyp. Board	5		2	6.3	B Appendix B
		Miscellaneous	5		1	1.0	B Appendix B
					Total Wall Load =	8.2	8.2 rounded

PROJECT: LAGUNA NIGUEL ADUs

SUBJECT: TYPICAL CHORD SPLICE CALCULATIONS

TYPICAL CALCULATIONS

TYPICAL CHORD SPLICE @ 2x6 STUDS

NAILED SPLICES
ALLOWABLE TENSION

(1) 2x6 : T_{2x6} = 1(1.5)(5.5)(575psi)(1.6)(1.3) = 9867 lbs

MARK	No. 16d NAILS	CAPACITY (lbs)		
A	8	8(130lbs)(1.60) =	1671 lbs	
B	12	12(130lbs)(1.60) =	2507 lbs	
C	16	16(130lbs)(1.60) =	3342 lbs	
D	20	20(130lbs)(1.60) =	4178 lbs	
E	24	24(130lbs)(1.60) =	5013 lbs	
F	28	28(130lbs)(1.60) =	5849 lbs	
G	32	32(130lbs)(1.60) =	6684 lbs	< 9867 lbs

NOTE: NAIL VALUES GOVERN ALL CASES

16d COMMON NAIL CAPACITY

PER 2018 NDS TBL 12N

ts = 1.5 IN
tm = 1.5 IN
G = 0.5 (DF-L)
Z = 141 LBS
16d NAIL DIA = 0.162 IN
PENETRATION, p = 1.5" / 1.62" = 0.926
p * Z = 130 LBS

PER 2018 NDS TBL 11.3.1

$Z' = Z \times C_D \times C_M \times Ct \times Cg \times C\Delta \times Ceg \times Cdi \times Ctn \times p$

C_D = 1.6 [2018 NDS TBL 2.3.2]

$C_M = Ct = Cg = C\Delta = Ceg = Cdi = Ctn = 1.0$

PROJECT NO.: 2889-01-CU22
DATE: 12/2024
BY: IPK
CHECKED BY: JMM

PROJECT: LAGUNA NIGUEL ADUs
SUBJECT: SHEAR WALL SCHEDULE

TYPICAL CALCULATIONS

SHEAR WALL SCHEDULE

CONVENTIONAL HOLDOWNS[1]: Yes — 8% Reduction SDPWS Table 4.3A, Footnote 10
SHORT NAILS[2]: Yes — SDPWS C4.3.5.2, 5% Reduction Selected

SPECIFIC GRAVITY, G (FRAMING): 0.5 — per NDS Supplement Table 4A / 4B
FRAMING ADJUSTMENT FACTOR: 1.00 — Capacity Reduction factor per SDPWS Table 4.3A Footnote 2, for species and grades of framing other than DF-L and SP

MARK	SHEATHING THICKNESS*	EDGE NAILING	SILL PLATE SHEAR ANCHOR BOLTS	ALLOWABLE SHEAR[1 2] (plf)
A	15/32"	8d @ 6"	5/8" @ 48" o.c.	266
B	15/32"	10d @ 6"	5/8" @ 48" o.c.	294
C	15/32"	10d @ 4"	5/8" @ 32" o.c.	443
D	15/32"	10d @ 3"	5/8" @ 24" o.c.	577
E	15/32"	10d @ 2"	5/8" @ 24" o.c.**	756
F	15/32" EA. SIDE	10d @ 4"	5/8" @ 16" o.c.**	888
G	15/32" EA. SIDE	10d @ 3"	5/8" @ 16" o.c.**	1,155
H	15/32" EA. SIDE	10d @ 2"	5/8" @ 12" o.c.**	1,512

*Structural 1 Plywood
**Requires 3x sole plate to achieve prescribed anchor bolt spacing

BOUNDARY ELEMENT HOLDOWNS

SIMP. HOLDOWN	BOUNDARY ELEMENT	FASTENERS	ALLOW UPLIFT (lbs)
DTT2Z	2x MIN.	(8) ¼"x1½" SDS	1825
DTT2Z-2.5	(2) 2x MIN.	(8) ¼"x2½" SDS	2145
HDU2	(2) 2x MIN.	(6) ¼"x2½" SDS	3075
HDU4	(2) 2x MIN.	(10) ¼"x2½" SDS	4565
HDU5	(2) 2x MIN.	(14) ¼"x2½" SDS	5645
HDU8	4x MIN.	(20) ¼"x2½" SDS	6970
HDU11	6x MIN.	(30) ¼"x2½" SDS	9535
HDU14	6x MIN.	(36) ¼"x2½" SDS	14445
HDQ8	4x MIN.	(20) ¼"x3" SDS	7630
HHDQ11	6x MIN.	(24) ¼"x2½" SDS	9535
HHDQ14	6x MIN.	(30) ¼"x2½" SDS	13710
HD19	6x MIN.	(5) 1" BOLTS	19070

		PROJECT NO.:	2889-01-CU22
		DATE:	12/2024
PROJECT:	LAGUNA NIGUEL ADUs	BY:	IPK
SUBJECT:	TYPICAL PAD FOOTING CALCULATIONS	CHECKED BY:	JMM

TYPICAL CALCULATIONS

TYPICAL PAD FOOTING

PAD FOOTING 'F2' (2'-0" SQUARE x 1'-6" THICK)

b =	2.00	ft	f_c' =	2500	psi
t =	1.50	ft	f_y =	60	ksi
q_{ALLOW} =	1500	psf	Reinf. =	#5	

BEARING CAPACITY

MAX DL+LL = $q_{ALLOW}(b)^2/1000$ = 6.00 kips

MAX UPLIFT = $(0.150\text{kcf})b^2t$ = 0.90 kips

BENDING CAPACITY

FOR fy = 60 ksi, ρ ≥ (0.0018)bt; $A_{s\_req'd}$ = 0.78 in^2

FOR #5 EA. WAY; As = 0.31 in2 #5 = 6 bars; 3 Bottom Bars

FOR (3) #5 BARS @ 24" FTG. w/ 3" CLR.; b_w = 8.69 in

FOR #5 BARS @ 18" THICK FTG.; d = 14.06 in

$$\emptyset M_N = \emptyset A_s f_Y\left(d - \frac{A_s f_Y}{1.7 f_C' b_w}\right) = \quad 226.97 \text{ kip-in}$$

AT ASD: MN_ASD = 11822 ft-lbs

$$q_{U\_ftg} = \frac{2M_{N\_ASD}}{\left(\frac{1}{2}(b-1')\right)^2} = \quad 94572.1669 \text{ psf/ft}$$

> qallow, FLEXURE OK

TWO WAY (PUNCHING) SHEAR

$$\varphi V_C = 0.75\left[4\sqrt{f_c'}(12" + d)(4d)\right] = \quad 219.9 \text{ kips}$$

$$V_U = 1.6\left[q_{ALLOW}\left[b^2 - \left(\frac{6" + d}{12"/ft}\right)^2\right]\right] = \quad 2.9 \text{ kips}$$

< ΦVc, PUNCHING SHEAR OK

ONE WAY (BEAM ACTION) SHEAR

$$\varphi V_C = 0.75\left[2\sqrt{f_c'}(bd(12"/ft))\right] = \quad 25.3 \text{ kips}$$

$$V_U = 1.6\left[q_{ALLOW}\left[\frac{b}{2} - \frac{6" + d}{12"/ft}\right]b\right] = \quad -3.225 \text{ kips}$$

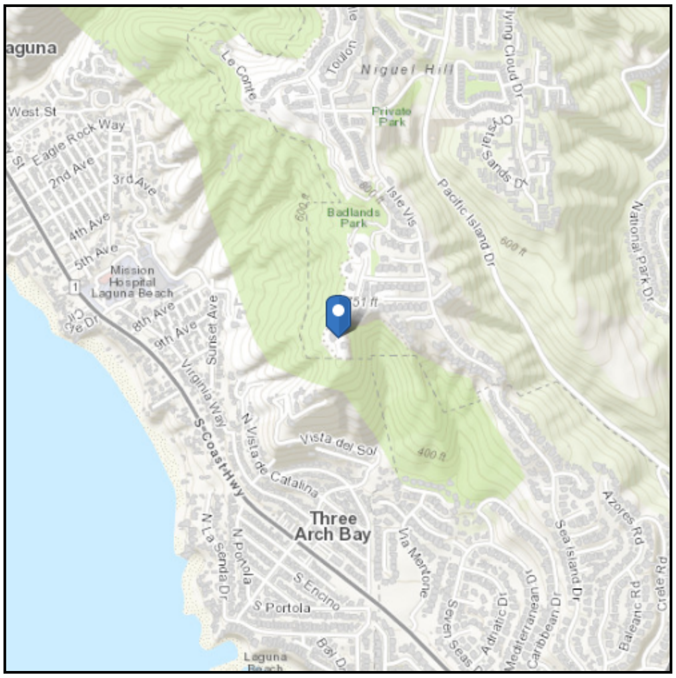
< ΦVc, BEAM ACTION SHEAR OK

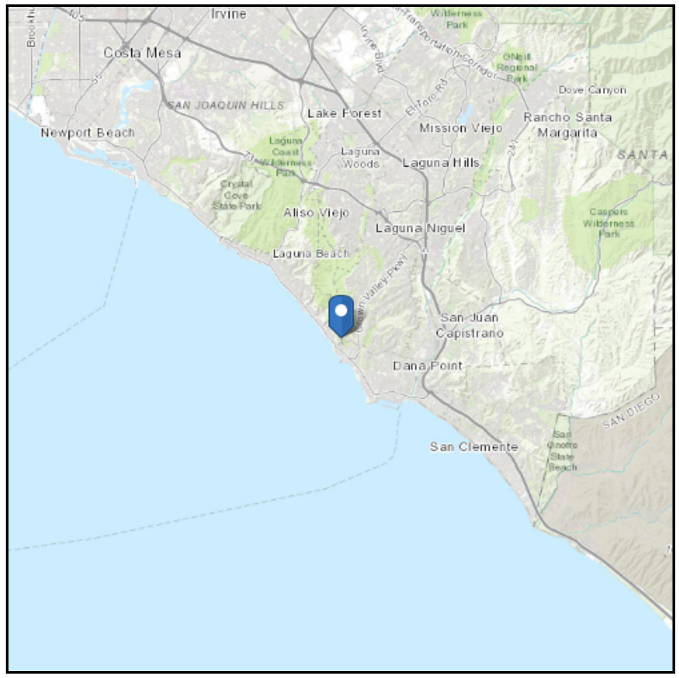
PROVIDE 2'-0" SQUARE x 1'-6" THICK FOOTING
WITH (3) #5 T&B EA. WAY @ PAD FOOTING 'F-2'

AMERICAN SOCIETY OF CIVIL ENGINEERS

ASCE Hazards Report

Address:	**Standard:**	ASCE/SEI 7-16	**Latitude:**	33.498759
No Address at This Location	**Risk Category:**	II	**Longitude:**	-117.733465
	Soil Class:	D - Default (see Section 11.4.3)	**Elevation:**	733.402305703125 ft (NAVD 88)





Wind

Results:

Wind Speed	95 Vmph
10-year MRI	66 Vmph
25-year MRI	72 Vmph
50-year MRI	77 Vmph
100-year MRI	81 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:	Fri Dec 27 2024

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.

ASCE
AMERICAN SOCIETY OF CIVIL ENGINEERS

Seismic

Site Soil Class: D - Default (see Section 11.4.3)

Results:

S_S :	1.314	S_{D1} :	N/A
S_1 :	0.467	T_L :	8
F_a :	1.2	PGA :	0.577
F_v :	N/A	PGA_M :	0.692
S_{MS} :	1.576	F_{PGA} :	1.2
S_{M1} :	N/A	I_e :	1
S_{DS} :	1.051	C_v :	1.363

Ground motion hazard analysis may be required. See ASCE/SEI 7-16 Section 11.4.8.

Data Accessed: Fri Dec 27 2024

Date Source: [USGS Seismic Design Maps]

The ASCE Hazard Tool is provided for your convenience, for informational purposes only, and is provided "as is" and without warranties of any kind. The location data included herein has been obtained from information developed, produced, and maintained by third party providers; or has been extrapolated from maps incorporated in the ASCE standard. While ASCE has made every effort to use data obtained from reliable sources or methodologies, ASCE does not make any representations or warranties as to the accuracy, completeness, reliability, currency, or quality of any data provided herein. Any third-party links provided by this Tool should not be construed as an endorsement, affiliation, relationship, or sponsorship of such third-party content by or from ASCE.

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WIND COMPONENT & CLADDING - ASCE 7-16 CH. 30

Input:

Roof Shape:	Gable	
Building Width:	46.0	ft
Building Length:	18.0	ft
Building Height:	10.3	ft
Roof Slope:	23.0	degrees
Enclosure:	Enclosed	
V =	95	mph
Exposure Category =	C	
K_z =	**0.85**	
K_{zt} =	1.00	
K_d =	0.85	
K_e =	1.00	
q_z =	16.67	psf
q_{zASD} =	10.00	psf
a =	3	ft
GC_{pi} =	0.18	

Roof Pressures, 1.0W: $P_w = q_h(GC_P \pm GC_{pi})$

Zone	$A \le 2\ ft^2$	$A \le 10\ ft^2$	$A \le 100\ ft^2$	$A \le 500\ ft^2$
1	-28.0	-28.0	-21.3	-16.3
2e	-28.0	-28.0	-21.3	-16.3
2n	-44.7	-44.7	-26.3	-23.0
2r	-44.7	-44.7	-26.3	-23.0
3e	-44.7	-44.7	-26.3	-23.0
3r	-63.0	-48.8	-33.0	-33.0
All Zones	16.0	16.0	16.0	16.0

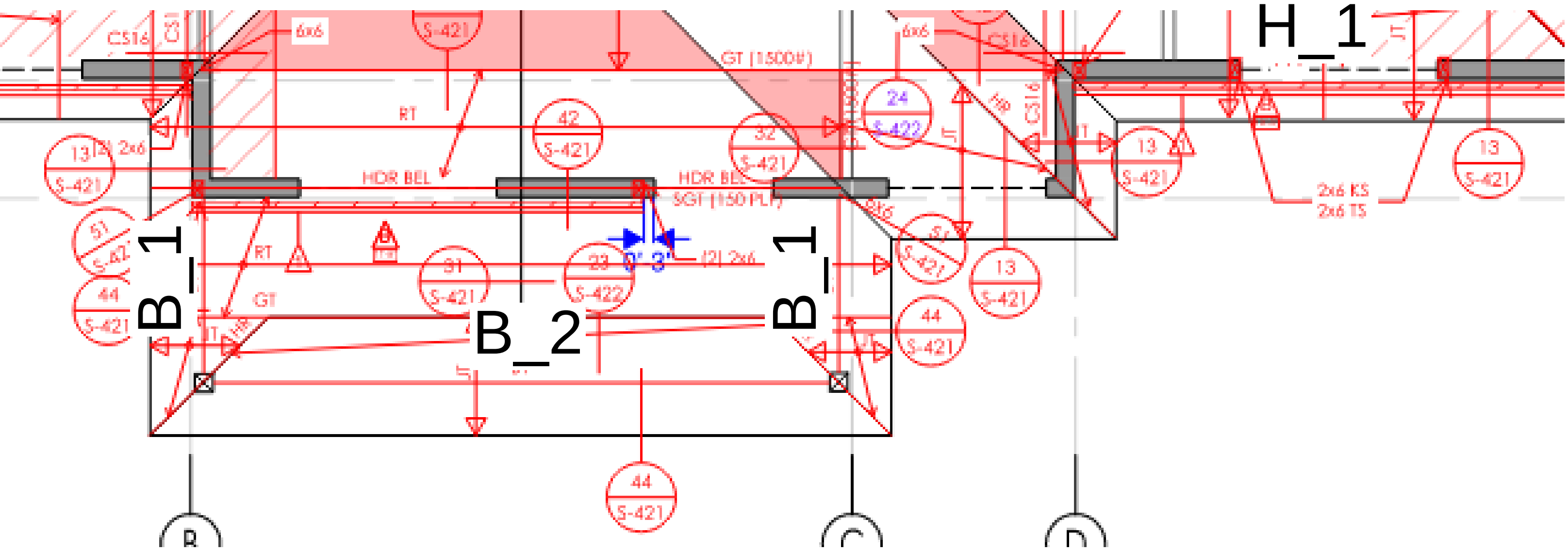
Overhang Pressures, 1.0W: $P_w = q_h(GC_P \pm GC_{pi})$

Zone	$A \le 2\ ft^2$	$A \le 10\ ft^2$	$A \le 100\ ft^2$	$A \le 500\ ft^2$
1	-36.3	-36.3	-34.7	-33.0
2e	-36.3	-36.3	-34.7	-33.0
2n	-53.0	-53.0	-42.2	-39.7
2r	-53.0	-53.0	-42.2	-39.7
3e	-63.0	-63.0	-43.0	-43.0
3r	-81.4	-63.0	-43.0	-43.0

Wall Pressures, 1.0W: $P_w = q_h(GC_P \pm GC_{pi})$

Zone	$A \le 10\ ft^2$	$A \le 100\ ft^2$	$A \le 500\ ft^2$
4	-21.3	-18.5	-16.3
5	-26.3	-20.5	-16.3
Positive	19.7	16.3	16.0

PLAN 3 - CAL RANCH GRAVITY MAP



RRM Design Group
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Suite 102
San Luis Obispo, CA 93405
805-597-5287

Project Title:
Engineer:
Project ID:
Project Descr:

Wood Beam

Project File: 2889-01 Laguna Niguel ADUs.ec6

LIC# : KW-06017541, Build:20.23.12.07 RRM Design Group (c) ENERCALC INC 1983-2023

DESCRIPTION: California Ranch H_1

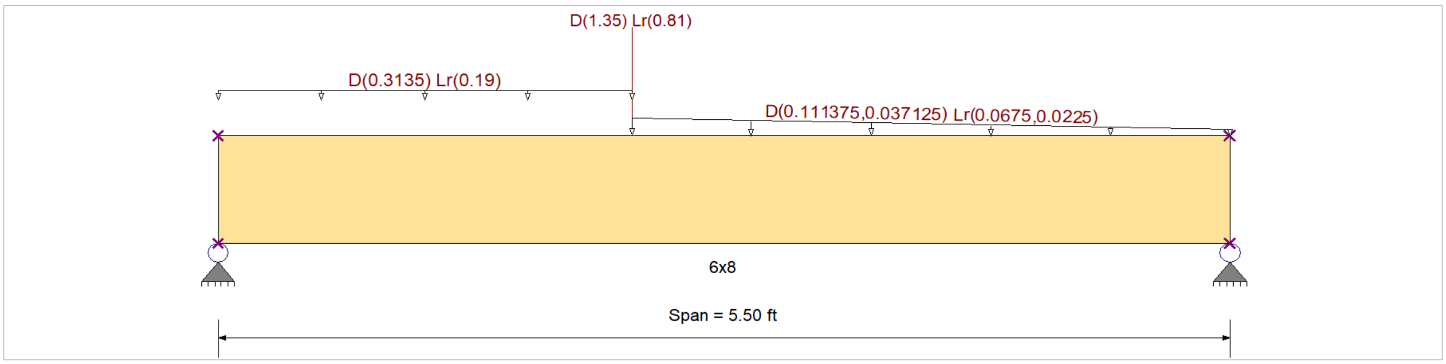
IBC 2021 is the model code for CBC 2022.
Enercalc does not have CBC 2022.

CODE REFERENCES

Calculations per NDS 2018, IBC 2021, ASCE 7-16
Load Combination Set : IBC 2021

Material Properties

Analysis Method :	Allowable Stress Design	Fb +	1,200.0 psi	*E : Modulus of Elasticity*	
Load Combination	IBC 2021	Fb -	1,200.0 psi	Ebend- xx	1,600.0ksi
		Fc - Prll	1,000.0 psi	Eminbend - xx	580.0ksi
Wood Species :	Douglas Fir-Larch	Fc - Perp	625.0 psi		
Wood Grade :	No.1	Fv	170.0 psi		
		Ft	825.0 psi	Density	31.210pcf
Beam Bracing :	Completely Unbraced				



Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Beam self weight calculated and added to loading

Point Load : D = 1.350, Lr = 0.810 k @ 2.250 ft, (Girder Truss)

Varying Uniform Load : D= 0.0330->0.0330, Lr= 0.020->0.020 ksf, Extent = 2.250 -->> 5.50 ft, Trib Width = 3.375->1.125 ft, (JT)

Uniform Load : D = 0.0330, Lr = 0.020 ksf, Extent = 0.0 -->> 2.250 ft, Tributary Width = 9.50 ft, (RT)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.616** : 1	Maximum Shear Stress Ratio	=	**0.345** : 1
Section used for this span		**6x8**	Section used for this span		**6x8**
fb: Actual	=	920.68 psi	fv: Actual	=	73.34 psi
F'b	=	1,494.44 psi	F'v	=	212.50 psi
Load Combination		+D+Lr	Load Combination		+D+Lr
Location of maximum on span	=	2.248ft	Location of maximum on span	=	0.000 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.022 in	Ratio =	3021	>=360	Span: 1 : Lr Only
Max Upward Transient Deflection	0 in	Ratio =	0	<360	n/a
Max Downward Total Deflection	0.059 in	Ratio =	1123	>=180	Span: 1 : +D+Lr
Max Upward Total Deflection	0 in	Ratio =	0	<180	n/a

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios										Moment Values			Shear Values		
Segment Length	Span #	M	V	C_D	C_M	C_t	C_{Lx}	C_F	C_{fu}	C_i	C_r	M	fb	F'b	V	fv	F'v
D Only														0.0	0.00	0.0	0.0
Length = 5.50 ft	1	0.536	0.301	0.90	1.00	1.00	1.00	1.000	1.00	1.00	1.00	2.48	577.7	1,077.2	1.27	46.0	153.0
+D+Lr					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 5.50 ft	1	0.616	0.345	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	3.96	920.7	1,494.4	2.02	73.3	212.5
+D+0.750Lr					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 5.50 ft	1	0.559	0.313	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	3.59	834.9	1,494.4	1.83	66.5	212.5
+0.60D					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 5.50 ft	1	0.181	0.102	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	1.49	346.6	1,910.7	0.76	27.6	272.0

RRM Design Group
3765 S. Higuera Street
Suite 102
San Luis Obispo, CA 93405
805-597-5287

Project Title:
Engineer:
Project ID:
Project Descr:

Wood Beam

Project File: 2889-01 Laguna Niguel ADUs.ec6

LIC# : KW-06017541, Build:20.23.12.07 RRM Design Group (c) ENERCALC INC 1983-2023

DESCRIPTION: California Ranch H_1

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+Lr	1	0.0587	2.609		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	2.336	1.394
Max Upward from Load Combinations	2.336	1.394
Max Upward from Load Cases	1.467	0.879
D Only	1.467	0.879
+D+Lr	2.336	1.394
+D+0.750Lr	2.118	1.265
+0.60D	0.880	0.528
Lr Only	0.869	0.515

Wood Beam

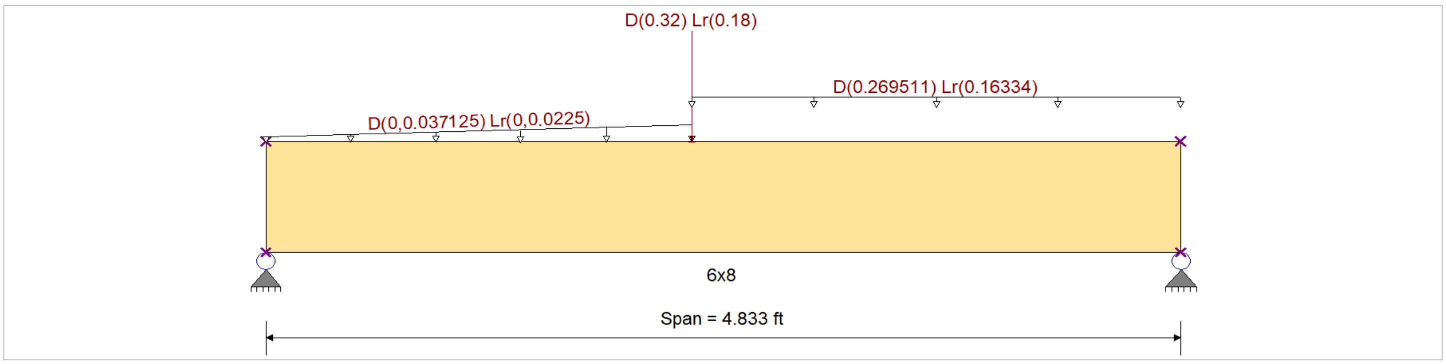
DESCRIPTION: California Ranch B_1

CODE REFERENCES

Calculations per NDS 2018, IBC 2021, ASCE 7-16
Load Combination Set : IBC 2021

Material Properties

Analysis Method :	Allowable Stress Design	Fb +	1,200.0 psi	*E : Modulus of Elasticity*	
Load Combination	IBC 2021	Fb -	1,200.0 psi	Ebend- xx	1,600.0ksi
		Fc - Prll	1,000.0 psi	Eminbend - xx	580.0ksi
Wood Species :	Douglas Fir-Larch	Fc - Perp	625.0 psi		
Wood Grade :	No.1	Fv	170.0 psi		
		Ft	825.0 psi	Density	31.210pcf
Beam Bracing :	Completely Unbraced				



Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Beam self weight calculated and added to loading
Load for Span Number 1
Varying Uniform Load : D= 0.0330->0.0330, Lr= 0.020->0.020 ksf, Extent = 0.0 -->> 2.250 ft, Trib Width = 0.0->1.125 ft, (Jack Truss)
Point Load : D = 0.320, Lr = 0.180 k @ 2.250 ft, (GT)
Uniform Load : D = 0.0330, Lr = 0.020 ksf, Extent = 2.250 -->> 4.833 ft, Tributary Width = 8.167 ft, (RT)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.211**: 1	Maximum Shear Stress Ratio	=	**0.141** : 1
Section used for this span		**6x8**	Section used for this span		**6x8**
fb: Actual	=	315.51 psi	fv: Actual	=	29.88 psi
F'b	=	1,495.03 psi	F'v	=	212.50 psi
Load Combination		+D+Lr	Load Combination		+D+Lr
Location of maximum on span	=	2.364ft	Location of maximum on span	=	4.216 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.006 in	Ratio =	9293 >=360	Span: 1 : Lr Only
Max Upward Transient Deflection	0 in	Ratio =	0 <360	n/a
Max Downward Total Deflection	0.017 in	Ratio =	3372 >=180	Span: 1 : +D+Lr
Max Upward Total Deflection	0 in	Ratio =	0 <180	n/a

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios										Moment Values			Shear Values		
Segment Length	Span #	M	V	CD	CM	C_t	CLx	C_F	Cfu	C_i	C_r	M	fb	F'b	V	fv	F'v
D Only														0.0	0.00	0.0	0.0
Length = 4.833 ft	1	0.187	0.124	0.90	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.86	201.1	1,077.5	0.52	19.0	153.0
+D+Lr					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 4.833 ft	1	0.211	0.141	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	1.36	315.5	1,495.0	0.82	29.9	212.5
+D+0.750Lr					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 4.833 ft	1	0.192	0.128	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	1.23	286.9	1,495.0	0.75	27.2	212.5
+0.60D					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0

RRM Design Group
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805-597-5287

Project Title:
Engineer:
Project ID:
Project Descr:

Wood Beam

Project File: 2889-01 Laguna Niguel ADUs.ec6

LIC# : KW-06017541, Build:20.23.12.07 RRM Design Group (c) ENERCALC INC 1983-2023

DESCRIPTION: California Ranch B_1

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios										Moment Values			Shear Values		
Segment Length	Span #	M	V	CD	CM	C_t	CLx	C_F	Cfu	C_i	C_r	M	fb	F'b	V	fv	F'v
Length = 4.833 ft	**1**	0.063	0.042	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.52	120.6	1,911.7	0.31	11.4	272.0

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+Lr	1	0.0172	2.487		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	0.634	1.094
Max Upward from Load Combinations	0.634	1.094
Max Upward from Load Cases	0.407	0.694
D Only	0.407	0.694
+D+Lr	0.634	1.094
+D+0.750Lr	0.577	0.994
+0.60D	0.244	0.416
Lr Only	0.226	0.401

Wood Beam

Project File: 2889-01 Laguna Niguel ADUs.ec6

LIC# : KW-06017541, Build:20.23.12.07 RRM Design Group (c) ENERCALC INC 1983-2023

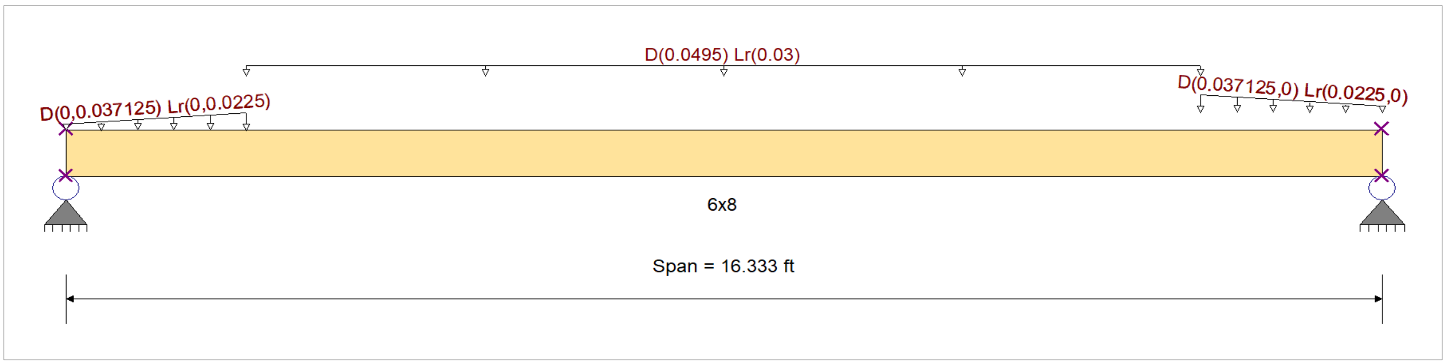
DESCRIPTION: California Ranch B_2

CODE REFERENCES

Calculations per NDS 2018, IBC 2021, ASCE 7-16
Load Combination Set : IBC 2021

Material Properties

Analysis Method :	Allowable Stress Design	Fb +	1,200.0 psi	*E : Modulus of Elasticity*	
Load Combination	IBC 2021	Fb -	1,200.0 psi	Ebend- xx	1,600.0 ksi
		Fc - Prll	1,000.0 psi	Eminbend - xx	580.0 ksi
Wood Species :	Douglas Fir-Larch	Fc - Perp	625.0 psi		
Wood Grade :	No.1	Fv	170.0 psi		
		Ft	825.0 psi	Density	31.210 pcf
Beam Bracing :	Completely Unbraced				



Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Beam self weight calculated and added to loading
Load for Span Number 1
Varying Uniform Load : D= 0.0330->0.0330, Lr= 0.020->0.020 ksf, Extent = 0.0 -->> 2.250 ft, Trib Width = 0.0->1.125 ft, (Jack Truss To Hip)
Varying Uniform Load : D= 0.0330->0.0330, Lr= 0.020->0.020 ksf, Extent = 14.083 -->> 16.333 ft, Trib Width = 1.125->0.0 ft, (Jack Truss To Hip)
Uniform Load : D = 0.0330, Lr = 0.020 ksf, Extent = 2.250 -->> 14.083 ft, Tributary Width = 1.50 ft, (JT)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.447**: 1	Maximum Shear Stress Ratio	=	**0.103** : 1
Section used for this span		**6x8**	Section used for this span		**6x8**
fb: Actual	=	662.92 psi	fv: Actual	=	21.83 psi
F'b	=	1,482.56 psi	F'v	=	212.50 psi
Load Combination		+D+Lr	Load Combination		+D+Lr
Location of maximum on span	=	8.167 ft	Location of maximum on span	=	15.737 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.149 in	Ratio =	1314 >=360	Span: 1 : Lr Only
Max Upward Transient Deflection	0 in	Ratio =	0 <360	n/a
Max Downward Total Deflection	0.442 in	Ratio =	443 >=180	Span: 1 : +D+Lr
Max Upward Total Deflection	0 in	Ratio =	0 <180	n/a

Maximum Forces & Stresses for Load Combinations

Load Combination / Segment Length	Span #	Max Stress Ratios M	V	CD	CM	C_t	CLx	C_F	Cfu	C_i	C_r	Moment Values M	fb	F'b	Shear Values V	fv	F'v
D Only														0.0	0.00	0.0	0.0
Length = 16.333 ft	**1**	0.410	0.095	0.90	1.00	1.00	0.99	1.000	1.00	1.00	1.00	1.89	438.9	1,071.4	0.40	14.5	153.0
+D+Lr					1.00	1.00	0.99	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 16.333 ft	**1**	0.447	0.103	1.25	1.00	1.00	0.99	1.000	1.00	1.00	1.00	2.85	662.9	1,482.6	0.60	21.8	212.5
+D+0.750Lr					1.00	1.00	0.99	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 16.333 ft	**1**	0.409	0.094	1.25	1.00	1.00	0.99	1.000	1.00	1.00	1.00	2.61	606.9	1,482.6	0.55	20.0	212.5

RRM Design Group
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805-597-5287

Project Title:
Engineer:
Project ID:
Project Descr:

Wood Beam

Project File: 2889-01 Laguna Niguel ADUs.ec6

LIC# : KW-06017541, Build:20.23.12.07 RRM Design Group (c) ENERCALC INC 1983-2023

DESCRIPTION: California Ranch B_2

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios											Moment Values			Shear Values		
Segment Length	Span #	M	V	CD	CM	C_t	CLx	C_F	Cfu	C_i	C_r	M	fb	F'b	V	fv	F'v
+0.60D					1.00	1.00	0.99	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 16.333 ft	**1**	0.139	0.032	1.60	1.00	1.00	0.98	1.000	1.00	1.00	1.00	1.13	263.4	1,889.7	0.24	8.7	272.0

Overall Maximum Deflections

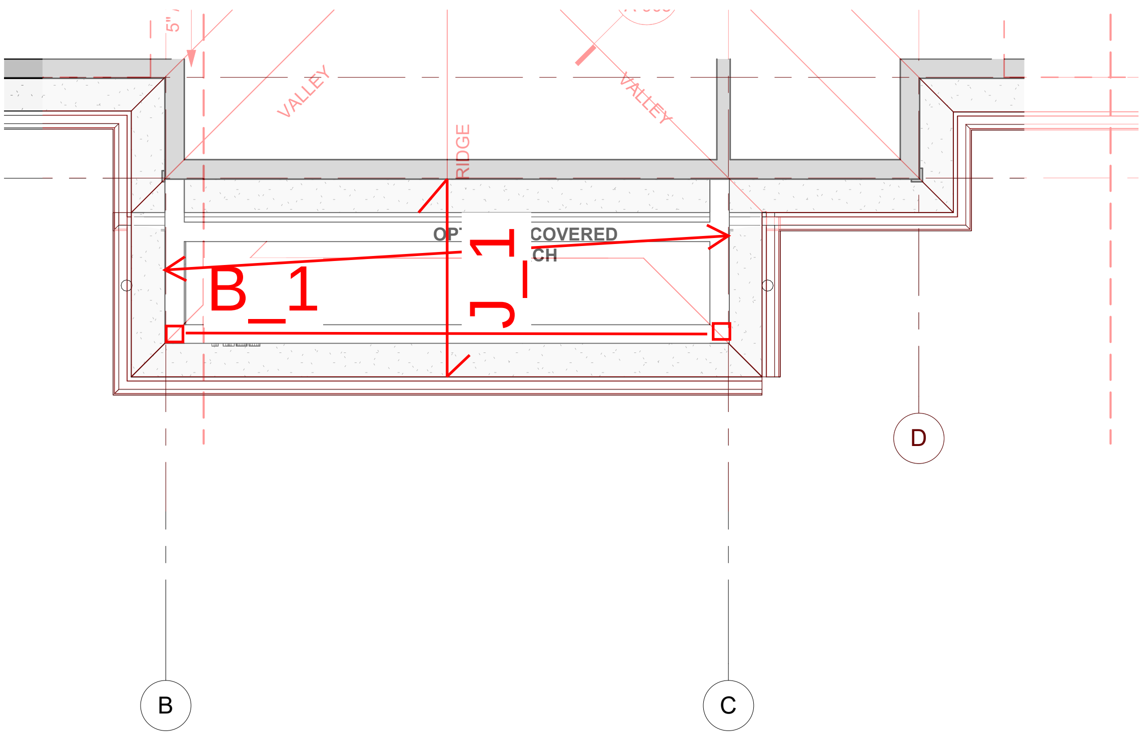
Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+Lr	1	0.4417	8.226		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	0.610	0.610
Max Upward from Load Combinations	0.610	0.610
Max Upward from Load Cases	0.408	0.408
D Only	0.408	0.408
+D+Lr	0.610	0.610
+D+0.750Lr	0.560	0.560
+0.60D	0.245	0.245
Lr Only	0.203	0.203

PLAN 3 - FARM HOUSE GRAVITY MAP



RRM Design Group
3765 S. Higuera Street
Suite 102
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805-597-5287

Wood Beam

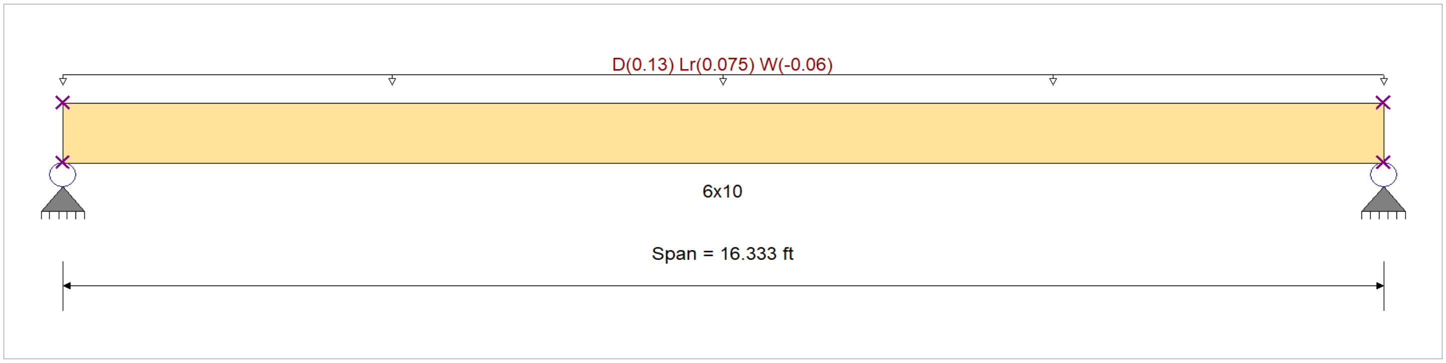
DESCRIPTION: Modern Farm B_1

CODE REFERENCES

Calculations per NDS 2018, IBC 2021, ASCE 7-16
Load Combination Set : IBC 2021

Material Properties

Analysis Method :	Allowable Stress Design	Fb +	1350 psi	*E : Modulus of Elasticity*		
Load Combination	IBC 2021	Fb -	1350 psi	Ebend- xx	1600 ksi	
		Fc - Prll	925 psi	Eminbend - xx	580 ksi	
Wood Species :	Douglas Fir-Larch	Fc - Perp	625 psi			
Wood Grade :	No.1	Fv	170 psi			
		Ft	675 psi	Density	31.21 pcf	
Beam Bracing :	Completely Unbraced					



Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Beam self weight calculated and added to loading
Uniform Load : D = 0.130, Lr = 0.0750, W = -0.060 , Tributary Width = 1.0 ft, (J_1)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.631**: 1	Maximum Shear Stress Ratio	=	**0.216** : 1
Section used for this span		**6x10**	Section used for this span		**6x10**
fb: Actual	=	1,046.33 psi	fv: Actual	=	45.90 psi
F'b	=	1,656.92 psi	F'v	=	212.50 psi
Load Combination		+D+Lr	Load Combination		+D+Lr
Location of maximum on span	=	8.167 ft	Location of maximum on span	=	15.558 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.192 in	Ratio =	1020	>=360	Span: 1 : Lr Only
Max Upward Transient Deflection	-0.154 in	Ratio =	1275	>=360	Span: 1 : W Only
Max Downward Total Deflection	0.554 in	Ratio =	353	>=180	Span: 1 : +D+Lr
Max Upward Total Deflection	0 in	Ratio =	0	<180	n/a

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios										Moment Values			Shear Values		
Segment Length	Span #	M	V	CD	CM	C_t	CLx	C_F	Cfu	C_i	C_r	M	fb	F'b	V	fv	F'v
D Only														0.0	0.00	0.0	0.0
Length = 16.333 ft	1	0.569	0.196	0.90	1.00	1.00	0.99	1.000	1.00	1.00	1.00	4.71	683.6	1,200.4	1.04	30.0	153.0
+D+Lr					1.00	1.00	0.99	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 16.333 ft	1	0.631	0.216	1.25	1.00	1.00	0.98	1.000	1.00	1.00	1.00	7.21	1,046.3	1,656.9	1.60	45.9	212.5
+D+0.750Lr					1.00	1.00	0.98	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 16.333 ft	1	0.577	0.197	1.25	1.00	1.00	0.98	1.000	1.00	1.00	1.00	6.59	955.6	1,656.9	1.46	41.9	212.5
+D+0.60W					1.00	1.00	0.98	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 16.333 ft	1	0.242	0.082	1.60	1.00	1.00	0.97	1.000	1.00	1.00	1.00	3.51	509.4	2,105.2	0.78	22.3	272.0
+D+0.750Lr+0.450W					1.00	1.00	0.97	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 16.333 ft	1	0.392	0.133	1.60	1.00	1.00	0.97	1.000	1.00	1.00	1.00	5.69	825.0	2,105.2	1.26	36.2	272.0

RRM Design Group
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805-597-5287

Project Title:
Engineer:
Project ID:
Project Descr:

Wood Beam

Project File: 2889-01 Laguna Niguel ADUs.ec6

LIC# : KW-06017541, Build:20.23.12.07 RRM Design Group (c) ENERCALC INC 1983-2023

DESCRIPTION: Modern Farm B_1

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios											Moment Values			Shear Values		
Segment Length	Span #	M	V	CD	CM	C_t	CLx	C_F	Cfu	C_i	C_r	M	fb	F'b	V	fv	F'v
+D+0.450W					1.00	1.00	0.97	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 16.333 ft	**1**	0.263	0.089	1.60	1.00	1.00	0.97	1.000	1.00	1.00	1.00	3.81	553.0	2,105.2	0.85	24.3	272.0
+0.60D+0.60W					1.00	1.00	0.97	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 16.333 ft	**1**	0.112	0.038	1.60	1.00	1.00	0.97	1.000	1.00	1.00	1.00	1.63	236.0	2,105.2	0.36	10.4	272.0
+0.60D					1.00	1.00	0.97	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 16.333 ft	**1**	0.195	0.066	1.60	1.00	1.00	0.97	1.000	1.00	1.00	1.00	2.83	410.1	2,105.2	0.63	18.0	272.0

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+Lr	1	0.5541	8.226		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	1.767	1.767
Max Upward from Load Combinations	1.767	1.767
Max Upward from Load Cases	1.154	1.154
Max Downward from all Load Conditio	-0.490	-0.490
Max Downward from Load Cases (Resis	-0.490	-0.490
D Only	1.154	1.154
+D+Lr	1.767	1.767
+D+0.750Lr	1.613	1.613
+D+0.60W	0.860	0.860
+D+0.750Lr+0.450W	1.393	1.393
+D+0.450W	0.934	0.934
+0.60D+0.60W	0.398	0.398
+0.60D	0.692	0.692
Lr Only	0.612	0.612
W Only	-0.490	-0.490

RRM Design Group
3765 S. Higuera Street
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805-597-5287

Project Title:
Engineer:
Project ID:
Project Descr:

Wood Beam

Project File: 2889-01 Laguna Niguel ADUs.ec6

LIC# : KW-06017541, Build:20.23.12.07 RRM Design Group (c) ENERCALC INC 1983-2023

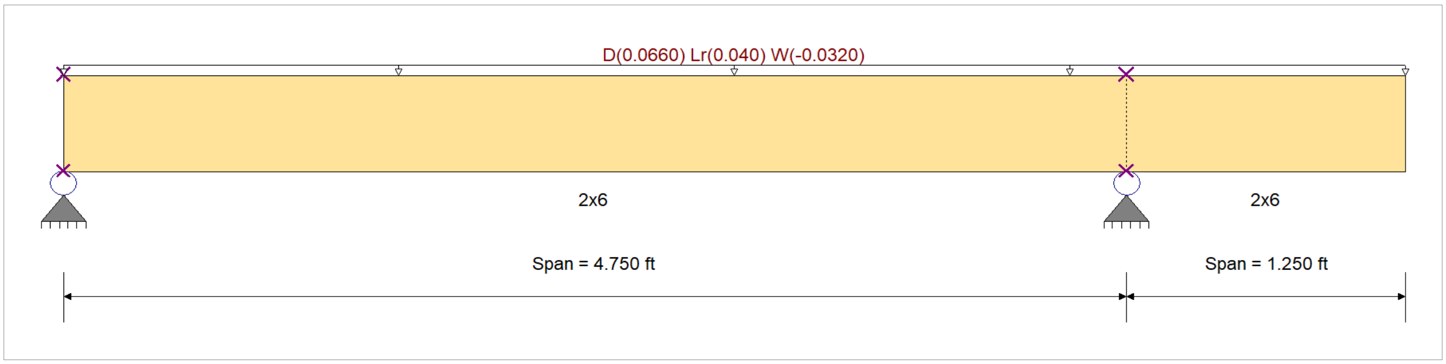
DESCRIPTION: Modern Farm J_1

CODE REFERENCES

Calculations per NDS 2018, IBC 2021, ASCE 7-16
Load Combination Set : IBC 2021

Material Properties

Analysis Method :	Allowable Stress Design	Fb +	900.0 psi	*E : Modulus of Elasticity*	
Load Combination	IBC 2021	Fb -	900.0 psi	Ebend- xx	1,600.0ksi
		Fc - Prll	1,350.0 psi	Eminbend - xx	580.0ksi
Wood Species :	Douglas Fir-Larch	Fc - Perp	625.0 psi		
Wood Grade :	No.2	Fv	180.0 psi		
		Ft	575.0 psi	Density	31.210pcf
Beam Bracing :	Completely Unbraced			Repetitive Member Stress Increase	



Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Beam self weight calculated and added to loading
Loads on all spans...
Uniform Load on ALL spans : D = 0.0330, Lr = 0.020, W = -0.0160 ksf, Tributary Width = 2.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.267**: 1	Maximum Shear Stress Ratio	=	**0.182** : 1
Section used for this span		**2x6**	Section used for this span		**2x6**
fb: Actual	=	417.87 psi	fv: Actual	=	40.93 psi
F'b	=	1,564.07 psi	F'v	=	225.00 psi
Load Combination		+D+Lr	Load Combination		+D+Lr
Location of maximum on span	=	2.203ft	Location of maximum on span	=	4.299 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.006 in	Ratio =	4836 >=360	Span: 2 : W Only
Max Upward Transient Deflection	-0.008 in	Ratio =	3870 >=360	Span: 2 : Lr Only
Max Downward Total Deflection	0.031 in	Ratio =	1824 >=180	Span: 1 : +D+Lr
Max Upward Total Deflection	-0.021 in	Ratio =	1436 >=180	Span: 2 : +D+Lr

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios										Moment Values			Shear Values		
Segment Length	Span #	M	V	CD	CM	C_t	CLx	C_F	Cfu	C_i	C_r	M	fb	F'b	V	fv	F'v
D Only														0.0	0.00	0.0	0.0
Length = 4.750 ft	1	0.226	0.159	0.90	1.00	1.00	0.96	1.300	1.00	1.00	1.15	0.17	262.8	1,163.9	0.14	25.7	162.0
Length = 1.250 ft	2	0.070	0.159	0.90	1.00	1.00	0.99	1.300	1.00	1.00	1.15	0.05	84.0	1,201.9	0.05	25.7	162.0
+D+Lr					1.00	1.00	0.99	1.300	1.00	1.00	1.15			0.0	0.00	0.0	0.0
Length = 4.750 ft	1	0.267	0.182	1.25	1.00	1.00	0.93	1.300	1.00	1.00	1.15	0.26	417.9	1,564.1	0.23	40.9	225.0
Length = 1.250 ft	2	0.080	0.182	1.25	1.00	1.00	0.99	1.300	1.00	1.00	1.15	0.08	133.6	1,663.6	0.09	40.9	225.0
+D+0.750Lr					1.00	1.00	0.99	1.300	1.00	1.00	1.15			0.0	0.00	0.0	0.0
Length = 4.750 ft	1	0.242	0.165	1.25	1.00	1.00	0.93	1.300	1.00	1.00	1.15	0.24	379.1	1,564.1	0.20	37.1	225.0
Length = 1.250 ft	2	0.073	0.165	1.25	1.00	1.00	0.99	1.300	1.00	1.00	1.15	0.08	121.2	1,663.6	0.08	37.1	225.0

RRM Design Group
3765 S. Higuera Street
Suite 102
San Luis Obispo, CA 93405
805-597-5287

Project Title:
Engineer:
Project ID:
Project Descr:

Wood Beam

Project File: 2889-01 Laguna Niguel ADUs.ec6

LIC# : KW-06017541, Build:20.23.12.07 RRM Design Group (c) ENERCALC INC 1983-2023

DESCRIPTION: Modern Farm J_1

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios										Moment Values			Shear Values		
Segment Length	Span #	M	V	CD	CM	C_t	CLx	C_F	Cfu	C_i	C_r	M	fb	F'b	V	fv	F'v
+D+0.60W					1.00	1.00	0.99	1.300	1.00	1.00	1.15			0.0	0.00	0.0	0.0
Length = 4.750 ft	**1**	0.099	0.064	1.60	1.00	1.00	0.88	1.300	1.00	1.00	1.15	0.12	188.4	1,897.4	0.10	18.4	288.0
Length = 1.250 ft	**2**	0.028	0.064	1.60	1.00	1.00	0.99	1.300	1.00	1.00	1.15	0.04	60.2	2,121.1	0.04	18.4	288.0
+D+0.750Lr+0.450W					1.00	1.00	0.99	1.300	1.00	1.00	1.15			0.0	0.00	0.0	0.0
Length = 4.750 ft	**1**	0.170	0.110	1.60	1.00	1.00	0.88	1.300	1.00	1.00	1.15	0.20	323.3	1,897.4	0.17	31.7	288.0
Length = 1.250 ft	**2**	0.049	0.110	1.60	1.00	1.00	0.99	1.300	1.00	1.00	1.15	0.07	103.4	2,121.1	0.07	31.7	288.0
+D+0.450W					1.00	1.00	0.99	1.300	1.00	1.00	1.15			0.0	0.00	0.0	0.0
Length = 4.750 ft	**1**	0.109	0.070	1.60	1.00	1.00	0.88	1.300	1.00	1.00	1.15	0.13	207.0	1,897.4	0.11	20.3	288.0
Length = 1.250 ft	**2**	0.031	0.070	1.60	1.00	1.00	0.99	1.300	1.00	1.00	1.15	0.04	66.2	2,121.1	0.04	20.3	288.0
+0.60D+0.60W					1.00	1.00	0.99	1.300	1.00	1.00	1.15			0.0	0.00	0.0	0.0
Length = 4.750 ft	**1**	0.044	0.028	1.60	1.00	1.00	0.88	1.300	1.00	1.00	1.15	0.05	83.2	1,897.4	0.04	8.2	288.0
Length = 1.250 ft	**2**	0.013	0.028	1.60	1.00	1.00	0.99	1.300	1.00	1.00	1.15	0.02	26.6	2,121.1	0.02	8.2	288.0
+0.60D					1.00	1.00	0.99	1.300	1.00	1.00	1.15			0.0	0.00	0.0	0.0
Length = 4.750 ft	**1**	0.083	0.054	1.60	1.00	1.00	0.88	1.300	1.00	1.00	1.15	0.10	157.7	1,897.4	0.08	15.4	288.0
Length = 1.250 ft	**2**	0.024	0.054	1.60	1.00	1.00	0.99	1.300	1.00	1.00	1.15	0.03	50.4	2,121.1	0.03	15.4	288.0

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+Lr	1	0.0312	2.335		0.0000	0.000
	2	0.0000	2.335	+D+Lr	-0.0209	1.250

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2	Support 3
Max Upward from all Load Conditions	0.238	0.408	
Max Upward from Load Combinations	0.238	0.408	
Max Upward from Load Cases	0.150	0.257	
Max Downward from all Load Conditio	-0.071	-0.121	
Max Downward from Load Cases (Resis	-0.071	-0.121	
D Only	0.150	0.257	
+D+Lr	0.238	0.408	
+D+0.750Lr	0.216	0.371	
+D+0.60W	0.107	0.184	
+D+0.750Lr+0.450W	0.184	0.316	
+D+0.450W	0.118	0.202	
+0.60D+0.60W	0.047	0.081	
+0.60D	0.090	0.154	
Lr Only	0.088	0.152	
W Only	-0.071	-0.121	

SHEET NO.:	
PROJECT NO.:	2889-01-C022
DATE:	12/2024
BY:	IPK
CHECKED BY:	JMM

PROJECT: LAGUNA NIGUEL ADUs

SUBJECT: EXTERIOR STUD WALL DESIGN

Stud Location: Gridline A

Input:

Lumber Grade:	DF No. 2	
Stud Spacing:	16	in
Stud Height:	8.083	ft
C_D for Lateral Pressure:	1.6	
C_D for Gravity Loads:	1.25	
Wind Pressure (W):	26.3	psf
Dead Load (D):	381.3	plf
Roof Live Load (L_r):	170	plf
Live Load (L):	0	plf
Total Stud Load:	551.3	plf

Assumptions:

$L_{eyy} = 0$

Lexx = Stud Height

F_c(Perp.) = 625 psi

$C_m = C_t = 1.0$

Basic Load Combinations

Wall Finish:	Stucco
Footnote m Fire-Rated, CBC T721.1(2):	No

Lumber Stresses:

F_c =	1,350	psi
F_b =	900	psi
E =	1,600,000	psi
E_{min} =	580,000	psi

Stud Design:

Stud Size: 2x6

A=	8.25	in^2
I=	20.80	in^4
S=	7.56	in^3
C_r=	1.15	

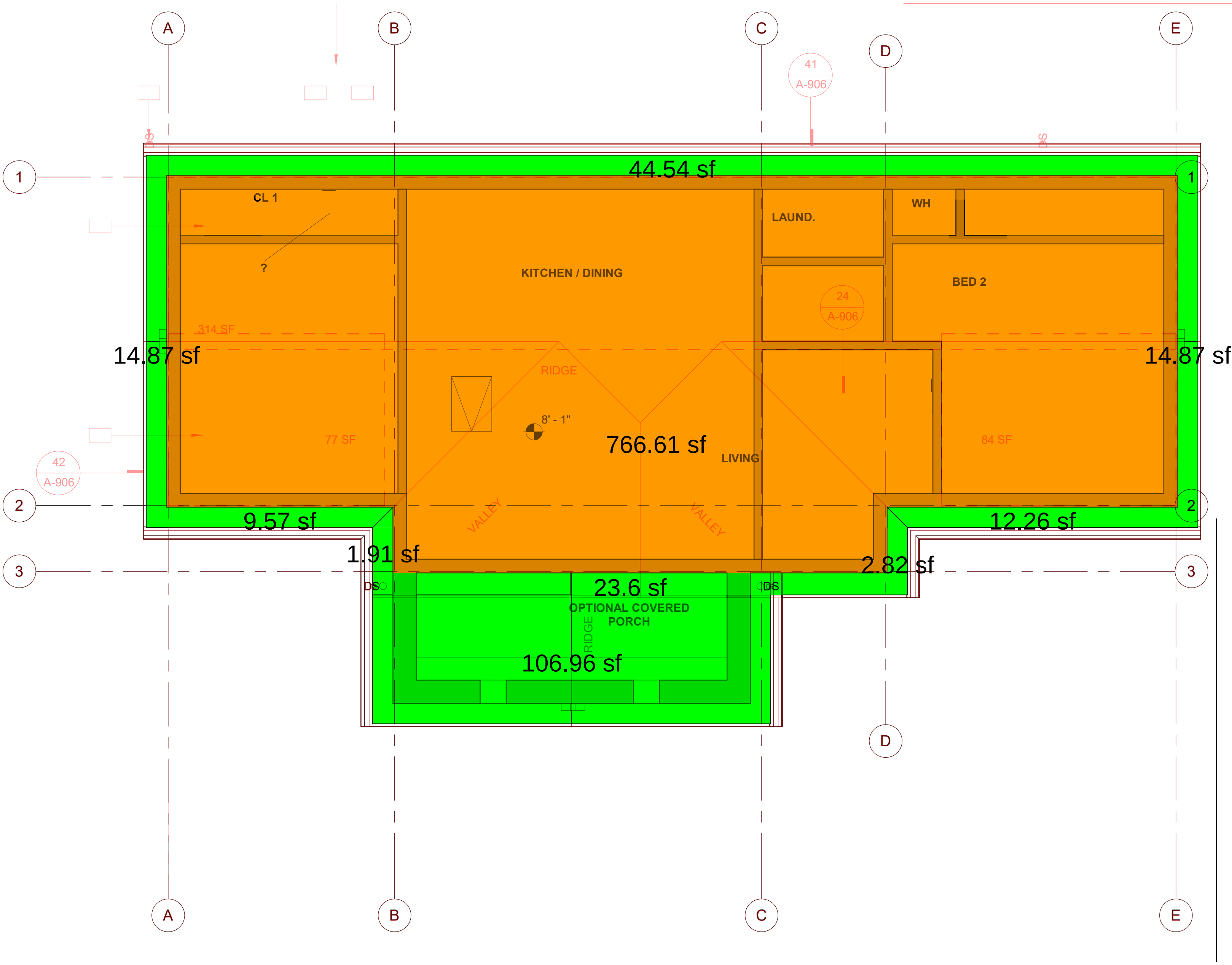
	D+L or D+L_r: (Eq. 16-9, 10)	**D+0.75L+0.75L_r:** (Eq. 16-11)	**D+0.75L+0.45W+0.75L_r:** (Eq. 16-13)	**D+0.6W:** (Eq.16-12)
P_{total} =	735.1 lbs.	678.4 lbs.	678.4 lbs.	508.4 lbs.
f_c =	89.1 psi	82.2 psi	82.2 psi	61.6 psi
F'_c =	1108.7 psi	1108.7 psi	1218.2 psi	1218.2 psi
Axial Stress Ratio:	0.08	0.07	0.07	0.05
Sill Plate Crushing:	**Adequate**	**Adequate**	**Adequate**	**Adequate**
M=	N/A	N/A	128.87 lb-ft	171.83 lb-ft
f_b =	N/A	N/A	204.5 psi	272.7 psi
F'_b=	N/A	N/A	1656 psi	1656 psi
Bending Stress Ratio:	N/A	N/A	0.12	0.16
Combined Stress Check:	N/A	N/A	0.14	0.17
Stud Design:	**Adequate**	**Adequate**	**Adequate**	**Adequate**

Stud Deflection[1]:	0.04 in.	
Deflection Ratio:	h/2425	**Adequate**

Notes:

1. For Stud Deflection, use 0.42 times the Wind Pressure (W) per footnote f of Table 1604.3

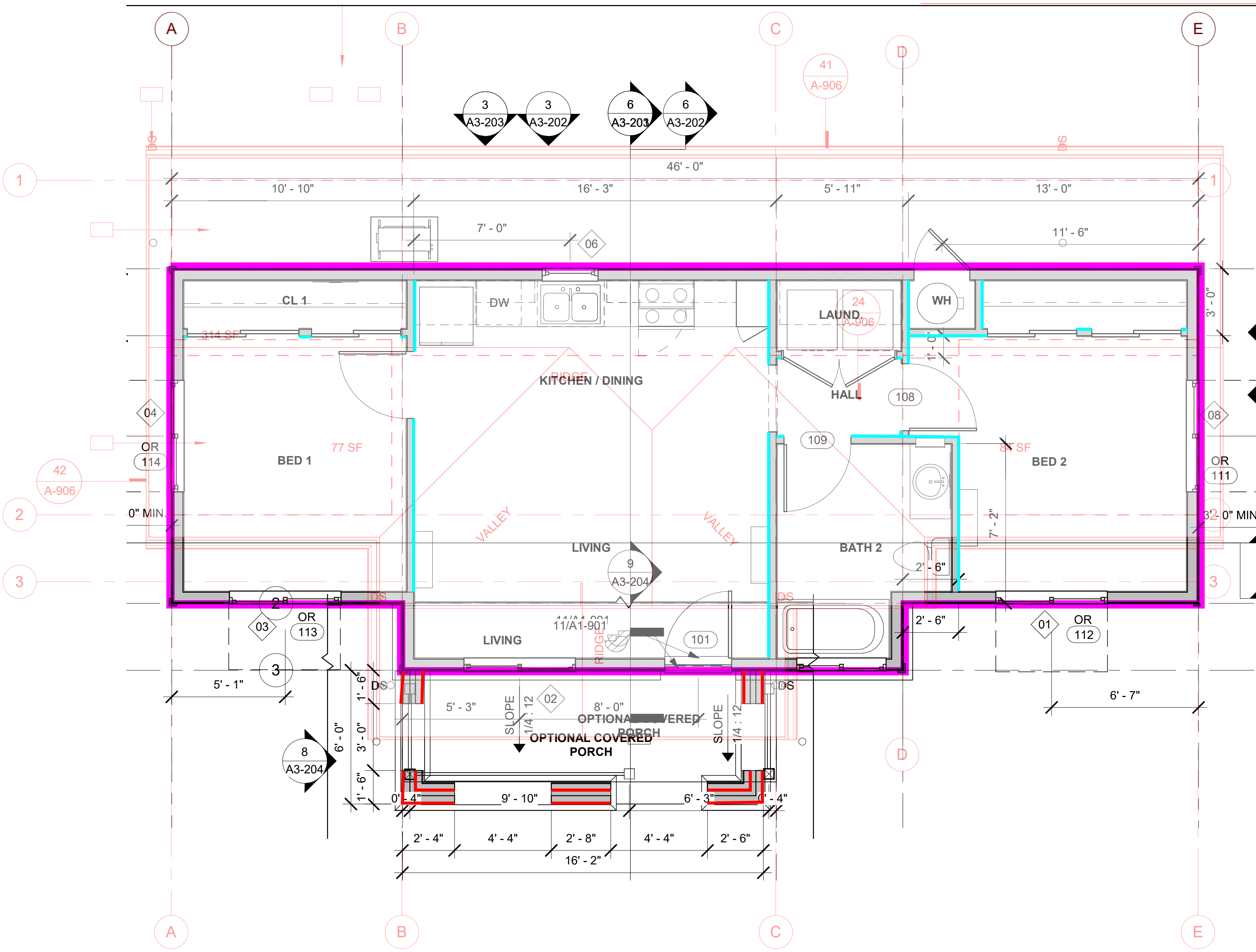
MASSING - PLAN 3



Legend

Description	Quantity	Unit
R1	766.61	sf
R2	231.39	sf

MASSING - PLAN 3



Legend

Description	Quantity	Unit
W1	128.5836	ft
W2	24.3756	ft
W3	49.0834	ft

BUILDING MASS
FLOOR AND WALL MASS SUMMARY
BUILDING: Plan 3

Floor ID	Floor	Plate Elev. (ft)	Floor Elev. (ft)	Height Below (ft)	Height Above (ft)
R	Roof (Mean)	8.1	10.3	10.3	0.0
1	Ground	0.0	0.0	0.0	10.3

Floor Type	Weight (psf)	Description
R1	27	Clay Tile Over Trusses w/ Gyp Ceiling & PV Panels
R2	33	Clay Tile Over Trusses w/ Stucco Ceiling

Wall Type	Weight (psf)	Description
W06ES	18	Plaster/Stucco Exterior Wall (2x6)
W04IP	8	Non-Bearing Partition Wall (2x4)
W04ES	17	Plaster/Stucco Exterior Wall (2x4)

FLOOR: ***Roof***

		Dimensions				Weight and Mass Info			Center of Mass			
Area Label	Floor Type	Area (sf)	Wall Length (ft)	Wall Trib. Below (ft)	Wall Trib. Above (ft)	Weight (psf)	Weight (lbs)	Mass (lbm/ft/s 2)	x_{cm} (ft)	y_{cm} (ft)	$W*x_{cm}$ (lb*ft)	$W*y_{cm}$ (lb*ft)
Floors												
RF1	R1	767				27	20,709	643.1			0	0
RF2	R2	232				33	7,656	237.8			0	0
Walls Above												
Walls Below												
WB1	W06ES		129.0	5.17	0	18	11,714	363.8			0	0
WB2	W04ES		24.0	5.17	0	17	2,108	65.5			0	0
WB3	W04IP		49.0	5.17	0	8	2,063	64.1			0	0
		999				Totals =	44,250	1,374	0.0	0.0	0	0

SEISMIC BASE SHEAR
EQUIVALENT LATERAL FORCE PROCEDURE
ASCE 7-16 SECTION 12.8 (As ammeded by 2022 CBC)
BUILDING: Plan 3

CODE SEISMIC GROUND MOTION VALUES

Parameter	Value	Description	Eqn	Code/Standard Reference
Site class =	D (DF)			C 1613.2.2
ELFP? =	YES	Structure is designed with ELFP per A 12.8.		
Regular Structure? =	NO	Structure DOES NOT meet the requirements of A 12.8.1.3.		A 12.8.1.3
Haz Anlys Provided?	NO	Site Specific Ground Motion Analysis Requirements		
Haz Anlys Reqd?	YES	Hazard Analysis required but not provided, adjust seismic parameters per		A 11.4.8
S_S =	1.375 g	Mapped spec acc for short periods (see below for max value cap)		C 1613.2.1
S_1 =	0.5 g	Mapped spec acc for T = 1 sec		C 1613.2.1
F_a =	1.200	Site Coefficient		C Table 1613.2.3(1)
F_v =	1.800	Site Coefficient		C Table 1613.2.3(2)
S_{MS} =	1.650 g	MCE, 5% damped, spec resp acc parameter at short periods adjusted for site effects	$S_{MS} = F_a S_S$	C 1613.2.3 (Eqn 16-36)
S_{M1} =	0.9 g	MCE, 5% damped, spec resp acc parameter at 1 sec period adjusted for site effects	$S_{M1} = F_v S_1$	C 1613.2.3 (Eqn 16-37)
$S_{DS} = 2/3\ S_{MS}$ =	1.100 g	Design, 5% damped, spectral response acceleration parameter at short periods	$S_{DS} = 2/3\ S_{MS}$	C 1613.3.4 (Eqn 16-38)
$S_{D1} = 2/3\ S_{M1}$ =	0.6 g	Design, 5% damped, spectral response acceleration parameter at 1 sec period	$S_{D1} = 2/3\ S_{M1}$	C 1613.3.4 (Eqn 16-39)

GOVERNING SEISMIC GROUND MOTION VALUES

S_{DS} = 1.100 g
S_{D1} = 0.6 g

BUILDING PARAMETERS

Risk Category =	I or II		C 1604.5
I =	1.00	Importance Factor	A Table 1.5-2
SDC =	D	Seismic design category	C 1613.2.5
LFRS =	15. Light Frame (wood) walls sheathed with wood structural panels rated for shear resistance		A Table 12.2-1
R =	6.5	Response Modification Coefficient	A Table 12.2-1
Ω_O =	3	System Overstrength Factor	A Table 12.2-1
C_d =	4	Deflection Amplification Factor	A Table 12.2-1

PERIOD LIMITS

T_a =	0.096 s		Approximate Fundamental Period	$T_a = C_t h_n^x$ =	A 12.8.2.1 (Eqn 12.8-7)
Structure Type =	All other structural systems				A Table 12.8-2
C_t =	0.02		Approximate Period Parameter		A Table 12.8-2
x =	0.75		Approximate Period Parameter		A Table 12.8-2
h_n =	8.1 ft		height above base		See plans
T_{MAX} =	0.134 s		Maximum period Limit	$T_{MAX} = C_u T_a$ =	A 12.8.2
C_u =	1.4		Coefficient for Upper Limit on Calculated Period		A Table 12.8-1
T_L =	8 s		Long period transition		A Fig. 22-14 (seismic map)
T_s =	0.545 s				
	X-Dir	Y-Dir			
T =	0.0959	0.0959	Analysis Derived Fundamental Period		A 12.8.2
T =	0.0959	0.0959	Governing Fundamental Period		A 12.8.2

SEISMIC BASE SHEAR
EQUIVALENT LATERAL FORCE PROCEDURE
ASCE 7-16 SECTION 12.8 (As ammeded by 2022 CBC)
BUILDING: Plan 3

BASE SHEAR

	X-Dir	Y-Dir			
C_s =	**0.169**	**0.169**	Base Shear Coefficient	$C_s = S_{DS}\ I / R$	A 12.8.1.1 (Eqn 12.8-2)
C_{sMAX} =	**0.963**	**0.963**	Max Base Shear Coefficient for $T \leq T_L$	$C_{sMAX} = S_{D1}\ I / (T\ R)$	A 12.8.1.1 (Eqn 12.8-3)
C_{sMAX} =	-	-	Max Base Shear Coefficient for $T \geq T_L$	$C_{sMAX} = S_{D1}\ T_L\ I / (T^2\ R)$	A 12.8.1.1 (Eqn 12.8-4)
C_{sMIN} =	**0.048**	**0.048**	Min Base Shear Coefficient	$C_{sMIN} = 0.044 S_{DS} I \geq 0.01$	A 12.8.1.1 (Eqn 12.8-5)
$C_{sMIN\ 2}$ =	-	-	Min Base Shear Coefficient for $S_1 \geq 0.6$	$C_{sMIN\ 2} = 0.5\ S_1\ I / R$	A 12.8.1.1 (Eqn 12.8-6)
C_s =	**0.169**	**0.169**	Governing Base Shear Coefficient		
V =	**7.5 k**	**7.5 k**	Equivalent lateral force procedure base shear	$V = C_s\ W$	A 12.8.1 (Eqn 12.8-1)
	0.12088	0.12088			

rrm design group

PROJECT:	LAGUNA NIGUEL ADUs	SHEET NO.:	
SUBJECT:	VERTICAL DISTRIBUTION OF FORCES	PROJECT NO.:	2889-01-CU22
		DATE:	12/2024
		BY:	IPK
		CHECKED BY:	JMM

VERTICAL DISTRIBUTION OF FORCES
EQUIVALENT LATERAL FORCE PROCEDURE
ASCE 7-16 SECTION 12.8.3
BUILDING: Plan 3

Parameter	Value	Description	Eqn	Code/Standard Reference
F_x =	See Table	Lateral Seismic Force at any level	$F_x = C_{vx}V$	A 12.8.3 (Eqn 12.8-11)
C_{vx} =	See Table	Vertical distribution factor	$C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k}$	A 12.8.3 (Eqn 12.8-12)
k =	1	Exponent related to structural period for structures having period of 0.5s or less		A 12.8.3
Fpx =	See Table		$Fpx = \frac{\sum_{i=x}^{n} F_i}{\sum_{i=x}^{n} w_i} w_{px}$	A 12.10.1.1 (Eqn 12.10-1)

EAST-WEST

C_s = 0.169 Base Shear Coefficient — See Previous Calcs
V = **7.5 k** Equivalent lateral force procedure base shear — See Previous Calcs

Vertical Distribution												Diaphragm Shear					
Level	Area (sf)	W_x (k)	W_{tot} (k)	h_x (ft)	h_x^K (ft)	$W_x h_x^k$ (k-ft)	$C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k}$	F_x (k)	f_x (psf)	F_x / W_x	F tot (k)	Min $0.2S_{DS}I_eW_{px}$ (k) A 12.10.1.1 (Eqn 12.10-1)	$Fpx = \frac{\sum_{i=x}^{n} F_i}{\sum_{i=x}^{n} w_i} w_{px}$ A 12.10.1.1 (Eqn 12.10-1)	Max $0.4S_{DS}I_eW_{px}$ (k) A 12.10.1.1 (Eqn 12.10-1)	F_{px} (k)	f_{px} (psf)	F_{px}/F_x Ratio
Roof	999	44.3	44.3	10.3	10.3	457.2	**1.00**	7.5	**7.5**	0.169	7.5	9.7	7.5	19.5	**9.7**	**9.74**	1.30
Total	**999**	44.3				**457.2**	1.00	7.5	**7.5**	0.169							

NORTH-SOUTH

C_s = 0.169 Base Shear Coefficient — See Previous Calcs
V = **7.5 k** Equivalent lateral force procedure base shear — See Previous Calcs

Vertical Distribution												Diaphragm Shear					
Level	Area (sf)	W_x (k)	W_{tot} (k)	h_x (ft)	h_x^K (ft)	$W_x h_x^k$ (k-ft)	$C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k}$	F_x (k)	f_x (psf)	F_x / W_x	F tot (k)	Min $0.2S_{DS}I_eW_{px}$ (k) A 12.10.1.1 (Eqn 12.10-1)	$Fpx = \frac{\sum_{i=x}^{n} F_i}{\sum_{i=x}^{n} w_i} w_{px}$ A 12.10.1.1 (Eqn 12.10-1)	Max $0.4S_{DS}I_eW_{px}$ (k) A 12.10.1.1 (Eqn 12.10-1)	F_{px} (k)	f_{px} (psf)	F_{px}/F_x Ratio
Roof	999	44.3	44.3	10.3	10.3	457.2	**1.00**	7.5	**7.5**	0.169	7.5	9.7	7.5	19.5	**9.7**	**9.74**	1.30
Total	**999**	44.3				**457.2**	1.00	7.5	**7.5**	0.169							

PROJECT:	LAGUNA NIGUEL ADUs
SUBJECT:	REDUNDANCY

SHEET NO.:	
PROJECT NO.:	2889-01-CU22
DATE:	12/2024
BY:	IPK
CHECKED BY:	JMM

REDUNDANCY
REDUNDANCY FACTOR, SEISMIC DESIGN CATEGORY D-F
ASCE 7-16 SECTION 12.3.4 (Including 12.3.4.2)
BUILDING: Plan 3

Floor	Height Below (ft)
Roof (Mean)	10.33
4th	0.00
3rd	0.00
2nd	0.00
Ground	0.00

ρ_{final} = 1

Light frame construction? YES

ROOF
Percent Base Shear = 1.00 Level Resists more than 35% of base shear, p Calc required

EAST-WEST

Wall Wall grid	Dimen. b (ft)	Ht/Width h/b ratio	Strength Loss? h/b >1, bwall/btotal ratio
Top Side			
R_1.1	13.583	0.76	NA
Total =	13.6	2 bays	
Bot Side			
R_3.1	11.4167	0.91	NA
Total =	11.4	2 bays	
Total Both =	25.0		

NORTH-SOUTH

Wall Wall grid	Dimen. b (ft)	Ht/Width h/b ratio	Strength Loss? h/b >1, bwall/btotal ratio
Left Side			
R_A.1	15	0.69	NA
Total =	15.0	2 bays	
Right Side			
R_E.1	15	0.69	NA
Total =	15.0	2 bays	
Total Both =	30.0		

		Code/Standard Reference
Condition a	YES	A 12.3.4.2 (a)
Condition b	YES	A 12.3.4.2 (b)

		Code/Standard Reference
Condition a	YES	A 12.3.4.2 (a)
Condition b	YES	A 12.3.4.2 (b)

ρ = 1

rrm design group

PROJECT:	LAGUNA NIGUEL ADUs	SHEET NO.:	
SUBJECT:	WIND BASE SHEAR CALCULATION	PROJECT NO.:	2889-01-CU22
		DATE:	12/2024
		BY:	IPK
		CHECKED BY:	JMM

WIND BASE SHEAR CALCULATION
ALTERNATE HEIGHTS WIND DESIGN METHOD
ASCE 7-16 CH. 26/27 AND 2022 CBC SECTION 1609
BUILDING: Plan 3

Parameter	Value		Description	Eqn	Code/Standard Reference
Building =	Plan 3				
Risk Category =	I or II				C 1604.5
Enclosure =	Enclosed				A 26.12
Exp. Category =	C		Exposure Category		C 1609.4.3
V_{ULT} =	95 mph		Ultimate Wind Speed		C Fig. 1609.3(1)
V_{ASD} =	74 mph		Nominal (Allowable) Desgin Wind Speed	$V_{asd} = V_{ult}\sqrt{0.6}$	C 1609.3.1 (Eqn 16-33)
z =	10.3 ft		Height above ground level		
h =	10.3 ft	< 75 ft	Mean roof height of building		
$K_{z,windward}$ =	0.85		MWFRS Velocity pressure exposure coefficient, windward or leeward	$K_z = 2.01(15/z_g)^{2/\alpha}$ z < 15 ft	A Tbl 26.10-1
$K_{z,leeward}$ =	0.85			$K_z = 2.01(z/z_g)^{2/\alpha}$ z > 15 ft	
$K_{zt,windward}$ =	1.00		Topographic factor, windward		A 26.8.2
$K_{zt,leeward}$ =	1.00				
K_d =	0.85		Directionality Factor		A Tbl 26.6-1
K_e =	1.00		Ground Elevation Factor		A Tbl 26.9-1
G =	0.85		Gust Effect Factor		A 26.11
GC_{pi} =	± 0.18		Internal Pressure Coefficient		A 26.13
$q_z = 0.00256K_zK_{zt}K_dK_eV^2$					*A Eqn 26.10-1*
$q_h = 0.00256K_hK_{zt}K_dK_eV^2$					*A Eqn 26.10-1*

MAIN WIND FORCE RESISTING SYSTEM

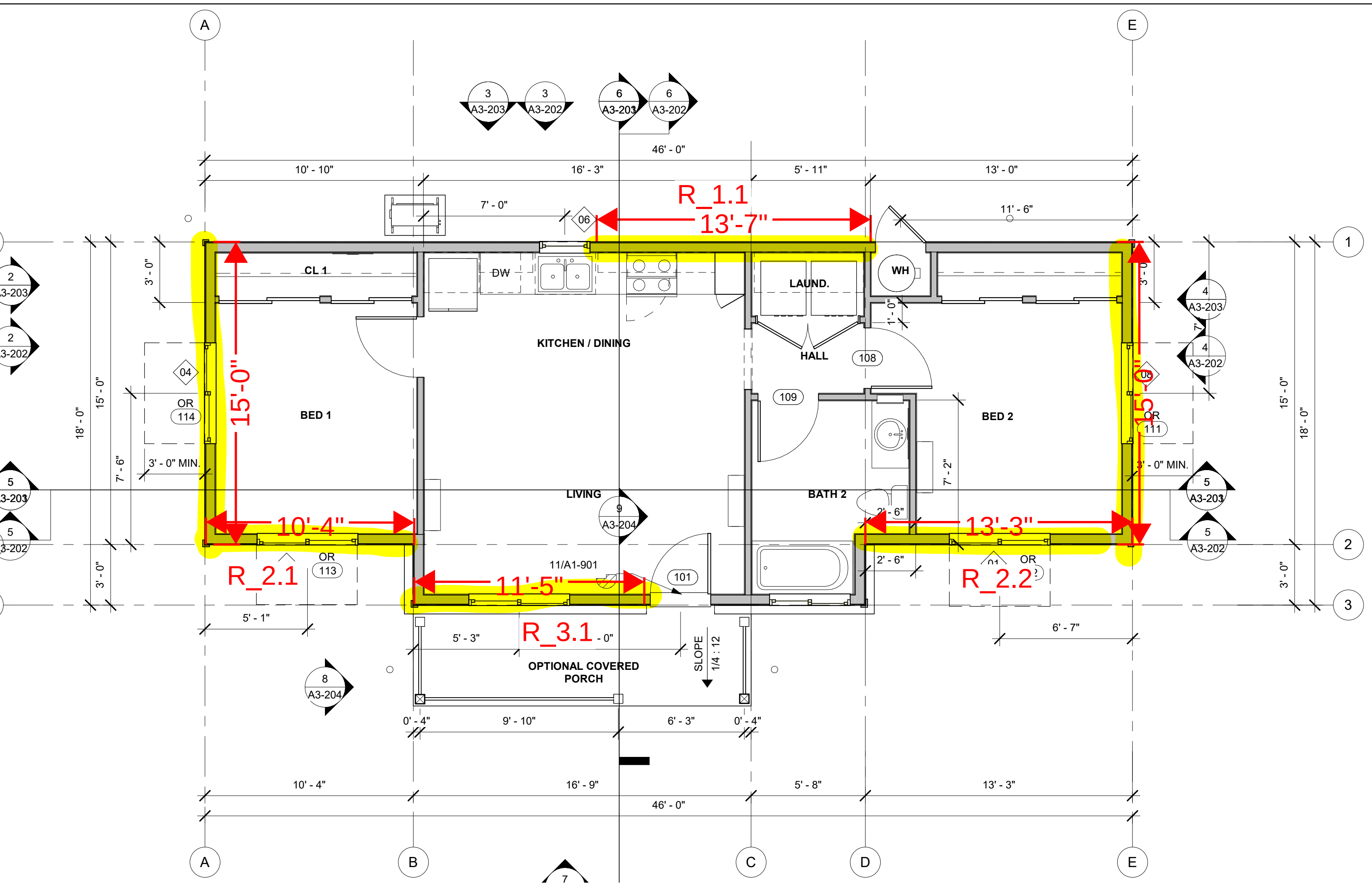
EAST-WEST

						+Internal													-Internal			
						Windward					Leeward					Total			Windward	Leeward	Total	Max
Level	h_x (ft)	Trib Ht. (ft)	Slope (deg)	Trib Width (ft)	Trib Area (ft)	K_z windward	q_z (psf)	C_p windward	P_{net} (psf)	Fx (kips)	K_h leeward	q_h (psf)	C_p leeward	P_{net} (psf)	Fx (kips)	Fx total (kips)	wx (plf)	Total P_{net} (psf)	C_p windward	C_p leeward	Total P_{net} (psf)	Max P_{net} (psf)
Roof	10.3 ft	2.21 ft	0	20.00 ft	44	0.85	16.67	-0.18	0.45	0.02	0.85	16.67	-0.9	-15.75	-0.7	0.7	35.8	14.1	-0.18	-0.9	10.2	14.1
Roof (Bot)	10.3 ft	4.04 ft	0	18.00 ft	73	0.85	16.67	0.8	8.34	0.6	0.85	16.67	-0.5	-10.09	-0.7	1.3	74.4	25.5	0.8	-0.5	18.4	25.5
Base	0.0 ft	0.00 ft	0	18.00 ft	0	0.85	16.67	0.8	8.34	0.0	0.85	16.67	-0.5	-10.09	0.0	0.0	0.0	25.5	0.8	-0.5	18.4	0.0
					117											**2.1 k**	**(Seismic Gov'n)**					

NORTH-SOUTH

						+Internal													-Internal			
						Windward					Leeward					Total			Windward	Leeward	Total	Max
Level	h_x (ft)	Trib Ht. (ft)	Slope (deg)	Trib Width (ft)	Trib Area (ft)	K_z windward	q_z (psf)	C_p windward	P_{net} (psf)	Fx (kips)	K_h leeward	q_h (psf)	C_p leeward	P_{net} (psf)	Fx (kips)	Fx total (kips)	wx (plf)	Total P_{net} (psf)	C_p windward	C_p leeward	Total P_{net} (psf)	Max P_{net} (psf)
Roof	10.3 ft	2.21 ft	0	48.00 ft	106	0.85	16.67	-0.18	0.45	0.05	0.85	16.67	-0.9	-15.75	-1.7	1.7	35.8	14.1	-0.18	-0.9	10.2	14.1
Roof (Bot)	10.3 ft	4.04 ft	0	46.00 ft	186	0.85	16.67	0.8	8.34	1.5	0.85	16.67	-0.5	-10.09	-1.9	3.4	74.4	25.5	0.8	-0.5	18.4	25.5
Base	0.0 ft	0.00 ft	0	46.00 ft	0	0.85	16.67	0.8	8.34	0.0	0.85	16.67	-0.5	-10.09	0.0	0.0	0.0	25.5	0.8	-0.5	18.4	0.0
					292											**5.1 k**	**(Seismic Gov'n)**					

SHEAR WALL LAYOUT - PLAN 3



SHEAR WALL DESIGN

SEGMENTED OR FORCE TRANSFER METHOD

Per SDPWS-2021 Table 4.3A

ROOF: EAST-WEST

$f_{p,\ Roof}$ = 7.50

$w_{p,\ Roof}$ = 110.3   ρ = 1

Diaph Fpx/Fx Ratio = 1.30 Ω_0 = 2.5   Reduce by 0.5 for structures with flexible diaphragms per A Tbl 12.2-1 footnote b.

Flexible Diaphragm? = YES

Grid	Wall	Dimen.				Seismic							Wind							Shear Wall (ASD Level)					
Wall grid line	Wall	b (ft)	Open (ft)	Final b (ft)	North or South Diaph	Trib Area (sf)	∑F_{Above} (lb)	∑F_{Above} Transf* North (lb)	∑$F_{x\ North}$ (lb)	F_{tot_e} (lb)	v_{seis} = ρF_{tot}/ (Final b) (plf)	$v_{seis,\ asd}$ = 0.7*ρF_{tot}/ (Final b) (plf)	Trib Width (ft)	∑F_{Above} (lb)	∑F_{Above} Transf* (lb)	∑F_x (lb)	F_{tot_w} (lb)	v_{wind} = F_{tot}/ (Final b) (plf)	$v_{wind,\ asd}$ = 0.6*Ftot/ (Final b) (plf)	Wall Height (ft)	Factor	Wall Type	Sheath Sides 1 or 2	Allow Shear Capcity S 4.3.3 (plf)	Edge Nail Spacing in
Wall?	R_1	13.6		13.6	North	45	0	0	337	3,051	225	157					1,378	101	61						
					South	362		0	2,714				13			1,378									
Wall	R_1.1	13.6		13.6						3,051	3,051						1,378	1,378		8.08	1.00	A	1	266	6
No Wall																									
No Wall																									
No Wall																									
Wall?	R_2	23.6		13.6	North	362	0	0	2,714	3,081	227	159	12.5			1,378	1,378	101	61						
					South	49		0	367																
Wall	R_2.1	10.3	5.0	5.3						1,210	1,210						541	541		8.08	1.00	A	1	266	6
Wall	R_2.2	13.3	5.0	8.3						1,871	1,871						837	837		8.08	1.00	A	1	266	6
No Wall																									
No Wall																									
No Wall																									
No Wall																									
No Wall																									
No Wall																									
No Wall																									
No Wall																									
Wall?	R_3	11.4		6.4	North	49	0	0	367	1,357	211	148													
					South	132		0	989																
Wall	R_3.1	11.4	5.0	6.4						1,357	1,357									8.08	1.00	A	1	266	6
No Wall																									
No Wall																									
No Wall																									
Wall?					North		0	0																	
					South			0																	
No Wall																									
No Wall																									
No Wall																									
No Wall																									
Wall?					North		0	0																	
					South			0																	
No Wall																									
No Wall																									
No Wall																									
No Wall																									
Wall?					North		0	0																	
					South			0																	
No Wall																									
No Wall																									
No Wall																									
No Wall																									
						999	0	0	7,489	14,977			25.0	0.0	0.0	2756.5	5513.0								

DIAPHRAGM DRAG DESIGN

ROOF: EAST-WEST

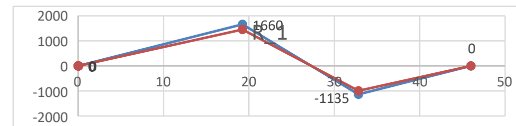
$f_{p,\,Roof}$ = 7.50 Irreg. Exists = YES f = 1.25 Per ASCE 7-16 Section 12.3.3.4 Increase forces by 25% if Type 1a, 1b, 2, 3 or 4 Horiz. Irregularity Exists

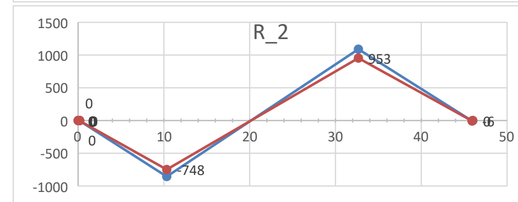
$w_{p,\,Roof}$ = 110.3 ρ = 1 ρ = 1.0 for diaphragm loads determined using Eq. 12.10-1 per A Ch 12.3.4.1 Note 7.

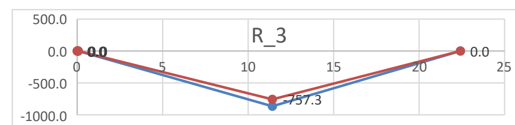
Diaph Fpx/Fx Ratio = 1.30 Ω_0 = 2.5 Reduce by 0.5 for structures with flexible diaphragms per A Tbl 12.2-1 footnote b.

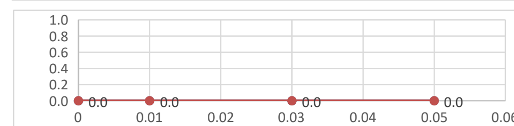
Per A Ch 12.10.2.1 In structures or portions thereof braced entirely by light frame shear walls overstrength need not apply

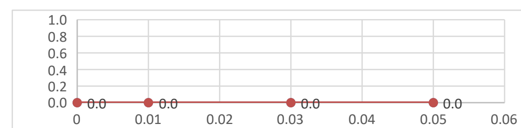
Wall	Dimen.	Seismic															
Line	b (ft)	Wall Shear (lb)	Diaph Shear (lb)	Dist. from West to Diaph Start (ft)	Dist. from West to Diaph End (ft)	Diaph L (ft)	Diaph W (ft)	v (plf)	Braced Entirely by Light Frame Walls	0.7*v*(f,Ω_0)	Dist. to Wall Start (ft)	Dist. to Wall End (ft)	v Wall (plf)	P_{Drag} Force @ Start of Diaph/ Wall (lb)	0.7*P_{Drag}*(ρ*f,Ω_0) @ Start (lb)	P_{Drag} Force @ End of Diaph/ Wall (lb)	0.7*P_{Drag}*(ρ*f,Ω_0) @ End (lb)
R_1	13.6	337	439	0	46.0	46	0.5	9.5	YES	8.3			32.3	0.0	0.0	0.0	0.0
		2,714	3528	0	46.0	46	0.5	76.7	YES	67.1			259.7	0.0	0.0	0.00	0.0
R_1.1	13.6										19.250	32.83		1659.7	1452.3	-1135.3	-993.4
												0.00		0.0	0.0	0.0	0.0
												0.00		0.0	0.0	0.0	0.0
												0.00		0.0	0.0	0.0	0.0
R_2	23.6	2,714	3528	0	46.0	46	0.5	76.7	YES	67.1			149.6	0.0	0.0	0.0	0.0
		367	477	0	46.0	46	0.5	10.4	YES	9.1			20.2	0.0	0.0	0.00	0.0
R_2.1	10.3										0.000	10.33		0.0	0.0	-855.2	-748.3
R_2.2	13.3										32.670	45.9		1089.6	953.4	-7.0	-6.1
												0.0		0.0	0.0	0.0	0.0
												0.0		0.0	0.0	0.0	0.0
												0.0		0.0	0.0	0.0	0.0
												0.0		0.0	0.0	0.0	0.0
												0.0		0.0	0.0	0.0	0.0
												0.0		0.0	0.0	0.0	0.0
												0.0		0.0	0.0	0.0	0.0
												0.0		0.0	0.0	0.0	0.0
R_3	11.4	367	477	0	22.4	22	0.5	21.3	YES	18.6			41.8	0.0	0.0	0.0	0.0
		989	1286	0	22.4	22	0.5	57.4	YES	50.2			112.7	0.0	0.0	0.000	0.0
R_3.1	11.4											11.42		0.0	0.0	-865.5	-757.3
												0.00		0.0	0.0	0.0	0.0
												0.0		0.0	0.0	0.0	0.0
												0.0		0.0	0.0	0.0	0.0
							0.5		YES								
							0.5		YES								

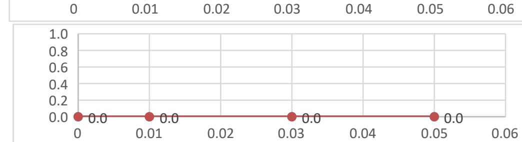












SHEAR WALL DESIGN
SEGMENTED OR FORCE TRANSFER METHOD
Per SDPWS-2021 Table 4.3A

ROOF: NORTH-SOUTH

$f_{p, Roof}$ = 7.50

$w_{p, Roof}$ = 110.3 ρ = 1

Diaph Fpx/Fx Ratio = 1.30 Ω_0 = 2.5 Reduce by 0.5 for structures with flexible diaphragms per A Tbl 12.2-1 footnote b.

Flexible Diaphragm? = YES

Grid	Wall	Dimen.				Seismic							Wind							Shear Wall (ASD Level)					
Wall grid line	Line	b (ft)	Open (ft)	Final b (ft)	West or East Diaph	Trib Area (sf)	$\sum F_{Above}$ (lb)	$\sum F_{Above}$ Transf* North (lb)	$\sum F_{x\ North}$ (lb)	F_{tot} (lb)	$v_{seis} = \rho F_{tot}/$(Final b) (plf)	$v_{seis,\ asd} = 0.7*\rho F_{tot}/$(Final b) (plf)	Trib Width (ft)	$\sum F_{Above}$ (lb)	$\sum F_{Above}$ Transf* (lb)	$\sum F_x$ (lb)	F_{tot} (lb)	$v_{wind} = F_{tot}/b$ (plf)	$v_{wind,\ asd} = 0.6*Ftot/b$ (plf)	Wall Height (ft)	Factor	Wall Type	Sheath Sides 1 or 2	Allow Shear Capcity S 4.3.3 (plf)	Edge Nail Spacing in
Wall?	R_A	14.9		9.9	West	16	0	0	120	4,303	434	304					772	78	47						
					East	558		0	4,183				7			772									
Wall	R_A.1	14.9	5.0	9.9						4,303	4,303						772	772		8.08	1.00	C	1	443	4
No Wall																									
No Wall																									
No Wall																									
Wall?	R_E	14.9		9.9	West	410	0	0	3,073	3,193	322	225	7.0			772	772	78	47						
					East	16		0	120																
Wall	R_E.1	14.9	5.0	9.9						3,193	3,193						772	772		8.08	1.00	A	1	266	6
No Wall																									
No Wall																									
No Wall																									
No Wall																									
No Wall																									
No Wall																									
No Wall																									
No Wall																									
No Wall																									
Wall?					West		0	0																	
					East			0																	
No Wall																									
No Wall																									
No Wall																									
No Wall																									
Wall?					West		0	0																	
					East			0																	
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Wall?					West		0	0																	
					East			0																	
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Wall?					West		0	0																	
					East			0																	
No Wall																									
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No Wall																									
						1,000	0	0	7,496	14,992			14.0	0.0	0.0	1543.6	3087.3								

DIAPHRAGM DRAG DESIGN

ROOF: NORTH-SOUTH

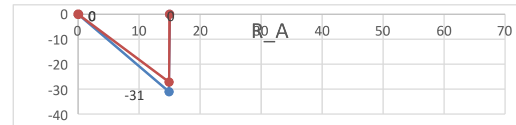
$f_{p,\,Roof}$ = 7.50 Irreg. Exists = YES f = 1.25 Per ASCE 7-16 Section 12.3.3.4 Increase forces by 25% if Type 1a, 1b, 2, 3 or 4 Horiz. Irregularity Exists

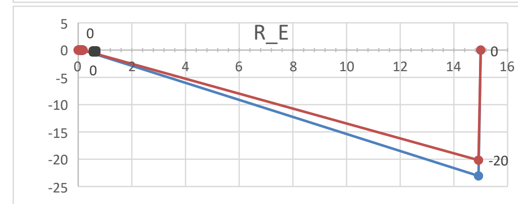
$w_{p,\,Roof}$ = 110.3 ρ = 1 ρ = 1.0 for diaphragm loads determined using Eq. 12.10-1 per A Ch 12.3.4.1 Note 7.

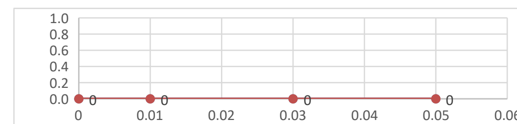
Diaph Fpx/Fx Ratio = 1.30 Ω_0 = 2.5 Reduce by 0.5 for structures with flexible diaphragms per A Tbl 12.2-1 footnote b.

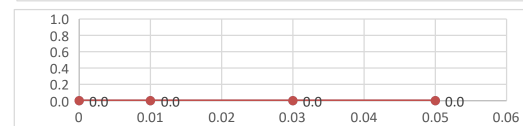
Per A Ch 12.10.2.1 In structures or portions thereof braced entirely by light frame shear walls overstrength need not apply

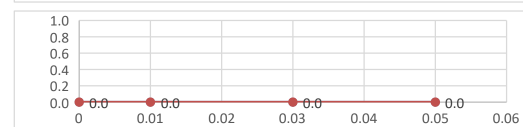
Wall	Dimen.	Seismic															
Line	**b (ft)**	**Wall Shear (lb)**	**Diaph Shear (lb)**	**Dist. from S to Diaph Start (ft)**	**Dist. from S to Diaph End**	**Diaph L (ft)**	**Diaph W (ft)**	**v (plf)**	**Braced Entirely by Light Frame Walls**	0.7*v*(f,Ω_0)	**Dist. to Wall Start (ft)**	**Dist. to Wall End (ft)**	v Wall (plf)	P_{Drag} Force @ Start of Diaph/ Wall (lb)	**0.7*P_{Drag}***(ρ*f,Ω_0) @ Start (lb)	P_{Drag} Force @ End of Diaph/ Wall (lb)	**0.7*P_{Drag}***(ρ*f,Ω_0) @ End (lb)
R_A	14.9	120	156	0	15.0	15	0.5	10.4	YES	9.1			10.5	0.0	0.0	0.0	0.0
		4,183	5438	0	15.0	15	0.5	362.5	YES	317.2			364.5	0.0	0.0	0.00	0.0
R_A.1	14.9										0.000	14.92		0.0	0.0	-31.1	-27.2
												0.00		0.0	0.0	0.0	0.0
												0.00		0.0	0.0	0.0	0.0
												0.00		0.0	0.0	0.0	0.0
R_E	14.9	3,073	3995	0	15.0	15	0.5	266.4	YES	233.1			267.8	0.0	0.0	0.0	0.0
		120	156	0	15.0	15	0.5	10.4	YES	9.1			10.5	0.0	0.0	0.00	0.0
R_E.1	14.9										0.000	14.92		0.0	0.0	-23.1	-20.2
												0.0		0.0	0.0	0.0	0.0
												0.0		0.0	0.0	0.0	0.0
												0.0		0.0	0.0	0.0	0.0
												0.0		0.0	0.0	0.0	0.0
												0.0		0.0	0.0	0.0	0.0
												0.0		0.0	0.0	0.0	0.0
												0.0		0.0	0.0	0.0	0.0
												0.0		0.0	0.0	0.0	0.0
												0.0		0.0	0.0	0.0	0.0
							0.5		YES								
							0.5		YES								
							0.5		YES								
							0.5		YES								
							0.5		YES								
							0.5		YES								
							0.5										
							0.5										

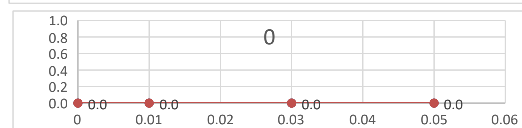












SHEAR WALL POST/HOLDOWN DESIGN

Building: Plan 3

S_{DS} 1.100 $M_{R,ASD} = M_R * (0.6\text{-}0.14S_{DS})$ **Compression members designed by Simpson. See ATS summary that follows

Stud Sp 16 $C_{ASD,\,Total} = M_{OT}/1.4d + w_{D+L}*2\text{ ft} + (P_D+P_L)$

Concentrated Dead Load Neglected in Uplift Calculation

e_{hold} = 5.5 Floor ID

e_{strap} = 2.75 ρ = 1 Use 80% ICC Capacity per DSA IR A-5, §4.2? No

Dimensions / LOADING

Level	Wall ID	Ht (ft)	Post Ht (ft)	Wall L: L_w (ft)	Ecc. Anch: d'_{hold} (ft)	Ecc. Anch: d'_{strap} (ft)	Distrib.: $w_{D,\,wall}$ (psf)	Distrib.: $w_{D,\,flr/rf}$ (psf)	Distrib.: w_L (psf)	Trib Width (ft)	Point Loads: P_D (lbs)	Point Loads: P_L (lbs)	Point Loads: P_E (lbs)	Point Loads: P_W (lbs)	P Loc. (ft)
R	R_A.1	10.3	8.1	14.9	14.23	14.5	18	27	20	2	0	0	0	0	0.0
R	R_E.1	10.3	8.1	14.9	14.23	14.5	18	27	20	2	0	0	0	0	0.0
R	R_1.1	10.3	8.1	13.6	12.9	13.1	18	27	20	7.5	0	0	0	0	0.0
R	R_2.1	10.3	8.1	10.3	9.646	9.9	18	27	20	7.5	0	0	0	0	0.0
R	R_2.2	10.3	8.1	13.3	12.56	12.8	18	27	20	7.5	0	0	0	0	0.0
R	R_3.1	10.3	8.1	11.4	10.73	11.0	18	27	20	2	0	0	0	0	0.0

SEISMIC

Wall ID	Shear: P_{seis} (lbs)	Moments: M_{ot} (ft-lbs)	Moments: M_R (ft-lbs)	Moments: $M_{R,\,ASD}$ (ft-lbs)	Overturn w/o Gravity	ASD Uplift Holdown: $[0.7r(M_{OT})-M_{R,ASD}]/d$ (lbs)	ASD Uplift Holdown: Diff. Load per floor (lbs)	ASD Uplift Strap: $[0.7r(M_{OT})-M_{R,ASD}]/d$ (lbs)	ASD Uplift Strap: Diff. Load per floor (lbs)	ASD COMP POST: $C_{ASD,\,Seis}$ (lbs)
R_A.1	4303	44,460	22,194	9,899	3,125	1,492	1,492	1,468	1,468	2,494
						Wall =	6"	Wall Cond =		Midwall
								Bolt Type =		SSTB
R_E.1	3193	32,996	22,194	9,899	2,319	928	928	913	913	1,930
						Wall =	6"	Wall Cond =		Midwall
								Bolt Type =		SSTB
R_1.1	3051	31,525	32,102	14,318	2,445	601	601	590	590	2,247
						Wall =	6"	Wall Cond =		Midwall
								Bolt Type =		SSTB
R_2.1	1210	12,499	18,578	8,286	1,296	48	48	47	47	1,443
						Wall =	6"	Wall Cond =		Midwall
								Bolt Type =		SSTB
R_2.2	1871	19,336	30,547	13,624	1,539	-7	-7	-7	-7	1,613
						Wall =	6"	Wall Cond =		Midwall
								Bolt Type =		SSTB
R_3.1	1357	14,020	13,001	5,799	1,307	374	374	366	366	1,222
						Wall =	6"	Wall Cond =		Midwall
								Bolt Type =		SSTB

WIND

Wall ID	Shear: v_{wind} (plf)	Moments: M_{ot} (ft-lbs)	Moments: $M_{R,\,ASD}$ (ft-lbs)	ASD Uplift Holdown: $[0.6(M_{OT})-M_{R,ASD}]/d$ (lbs)	ASD Uplift Holdown: Diff. Load per floor (lbs)	ASD Uplift Strap: $[0.6(M_{OT})-M_{R,ASD}]/d$ (lbs)	ASD Uplift Strap: Diff. Load per (lbs)	ASD COMP POST: $C_{ASD,\,Seis}$ (lbs)
R_A.1	771.8	7,975	13,317	-600	-600	-590	-590	602
R_E.1	771.8	7,975	13,317	-600	-600	-590	-590	602
R_1.1	1378.2	14,241	19,261	-831	-831	-817	-817	1,127
R_2.1	541.1	5,592	11,147	-808	-808	-789	-789	812
R_2.2	837.1	8,650	18,328	-1,046	-1,046	-1,027	-1,027	877
R_3.1	0.0	0	7,801	-727	-727	-712	-712	266

RESULTS – USE

Wall ID	Holddown: Model -	Holddown: Cap. (lbs)	Strap: Model -	Strap: Capacity (lbs)	Post: Memb -	Post: Tens. Cap. (lbs)	Post: Mem b -	Post: Comp. Cap. (lbs)	Post for Sill Crush: Memb -	Post for Sill Crush: Sill Comp. Cap. (lbs)
R_A.1	HDU2	3,075	CS16	1,705	2x6	9,867	2x6	8,680	2x6	4,383
	SSTB16	3780	SSTB16	2550	Bolt =	0				
	Foundation		Stem Wall							
R_E.1	HDU2	3,075	CS16	1,705	2x6	9,867	2x6	8,680	2x6	4,383
	SSTB16	3780	SSTB16	2550	Bolt =	0				
	Foundation		Stem Wall							
R_1.1	HDU2	3,075	CS16	1,705	2x6	9,867	2x6	8,680	2x6	4,383
	SSTB16	3780	SSTB16	2550	Bolt =	0				
	Foundation		Stem Wall							
R_2.1	HDU2	3,075	CS16	1,705	2x6	9,867	2x6	8,680	2x6	4,383
	SSTB16	3780	SSTB16	2550	Bolt =	0				
	Foundation		Stem Wall							
R_2.2	HDU2	3,075	CS16	1,705	2x6	9,867	2x6	8,680	2x6	4,383
	SSTB16	3780	SSTB16	2550	Bolt =	0				
	Foundation		Stem Wall							
R_3.1	HDU2	3,075	CS16	1,705	2x6	9,867	2x6	8,680	2x6	4,383
	SSTB16	3780	SSTB16	2550	Bolt =	0				
	Foundation		Stem Wall							

Force Transfer Around Openings Calculator

ONE OPENING

The force transfer around openings (FTAO) method of shear wall analysis is an approach that aims to reinforce the wall such that it performs as if there was no opening. This approach lends certain advantages over segmented shear walls: more versatility, because it allows for narrower wall segments while still meeting the height-to-width ratios and, often, fewer required hold-downs.

Project Information

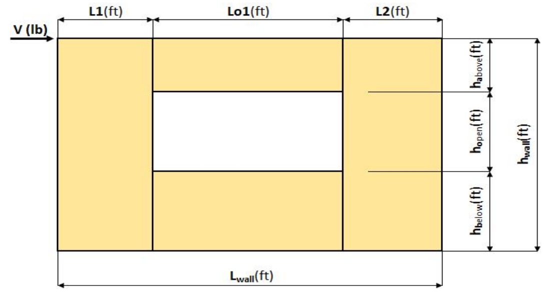
Code: **Date:**

Designer:

Client:

Project:

Wall Line: R_2.1



Shear Wall Calculation Variables

V	847 lbf	Opening 1		Adj. Factor Method =	1.25-0.125h/bs
L1	2.50 ft	h_a	1.42 ft	Wall Pier Aspect Ratio	Adj. Factor
L2	2.83 ft	h_o	4.00 ft	$P1=h_o/L1=$ 1.60	N/A
h_{wall}	8.09 ft	h_b	2.67 ft	$P2=h_o/L2=$ 1.41	N/A
L_{wall}	10.33 ft	Lo1	5.00 ft		

1. Hold-down forces: $H = Vh_{wall}/L_{wall}$ 663 lbf

2. Unit shear above + below opening

First opening: $va1 = vb1 = H/(h_a+h_b) =$ 162 plf

3. Total boundary force above + below openings

First opening: O1 = va1 x (Lo1) = 811 lbf

4. Corner forces

F1 = O1(L1)/(L1+L2) = 380 lbf

F2 = O1(L2)/(L1+L2) = 431 lbf

5. Tributary length of openings

T1 = (L1*Lo1)/(L1+L2) = 2.35 ft

T2 = (L2*Lo1)/(L1+L2) = 2.65 ft

6. Unit shear beside opening

v1 = (V/L)(L1+T1)/L1 = 159 plf

v2 = (V/L)(T2+L2)/L2 = 159 plf

Check v1*L1+v2*L2=V? 847 lbf **OK**

7. Resistance to corner forces

R1 = v1*L1 = 397 lbf

R2 = v2*L2 = 450 lbf

8. Difference corner force + resistance

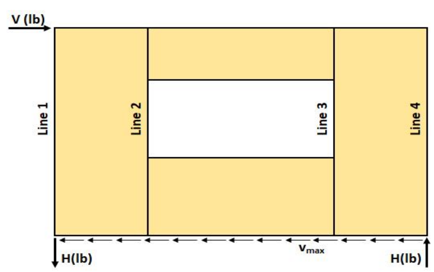
R1-F1 = 17 lbf

R2-F2 = 19 lbf

9. Unit shear in corner zones

vc1 = (R1-F1)/L1 = 7 plf

vc2 = (R2-F2)/L2 = 7 plf



Check Summary of Shear Values for One Opening

Line 1: $vc1(h_a+h_b)+v1(h_o)=H$?		27	635	663 lbf
Line 2: $va1(h_a+h_b)-vc1(h_a+h_b)-v1(h_o)=0$?	663	27	635	0
Line 3: $va1(h_a+h_b)-vc2(h_a+h_b)-v1(h_o)=0$?	663	27	635	0
Line 4: $vc2(h_a+h_b)+v2(h_o)=H$?		27	635	663 lbf

Design Summary*

Req. Sheathing Capacity	162 plf	**4-Term Deflection**		**3-Term Deflection**	
Req. Strap Force	431 lbf	**4-Term Story Drift %**		**3-Term Story Drift %**	
Req. HD Force (H)	663 lbf				
Req. Shear Wall Anchorage Force (v_{max})	82 plf				

***The Design Summary assumes that the shear wall is designed as blocked.**

APA

Force Transfer Around Openings Calculator

ONE OPENING

The force transfer around openings (FTAO) method of shear wall analysis is an approach that aims to reinforce the wall such that it performs as if there was no opening. This approach lends certain advantages over segmented shear walls: more versatility, because it allows for narrower wall segments while still meeting the height-to-width ratios and, often, fewer required hold-downs.

Project Information

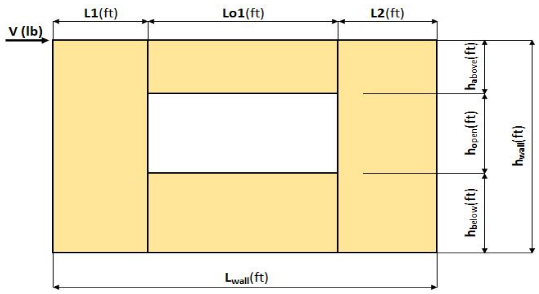
Code: **Date:**

Designer:

Client:

Project:

Wall Line: R_2.2



Shear Wall Calculation Variables

V	1310 lbf	Opening 1		Adj. Factor Method =	1.25-0.125h/bs
L1	4.17 ft	h_a	1.42 ft	Wall Pier Aspect Ratio	Adj. Factor
L2	4.08 ft	h_o	4.00 ft	$P1=h_o/L1=$ 0.96	N/A
h_{wall}	8.09 ft	h_b	2.67 ft	$P2=h_o/L2=$ 0.98	N/A
L_{wall}	13.25 ft	Lo1	5.00 ft		

1. Hold-down forces: $H = Vh_{wall}/L_{wall}$ 799 lbf

2. Unit shear above + below opening

First opening: $va1 = vb1 = H/(h_a+h_b) =$ 196 plf

3. Total boundary force above + below openings

First opening: O1 = va1 x (Lo1) = 978 lbf

4. Corner forces

F1 = O1(L1)/(L1+L2) = 494 lbf

F2 = O1(L2)/(L1+L2) = 484 lbf

5. Tributary length of openings

T1 = (L1*Lo1)/(L1+L2) = 2.53 ft

T2 = (L2*Lo1)/(L1+L2) = 2.47 ft

6. Unit shear beside opening

v1 = (V/L)(L1+T1)/L1 = 159 plf

v2 = (V/L)(T2+L2)/L2 = 159 plf

Check v1*L1+v2*L2=V? 1310 lbf **OK**

7. Resistance to corner forces

R1 = v1*L1 = 662 lbf

R2 = v2*L2 = 648 lbf

8. Difference corner force + resistance

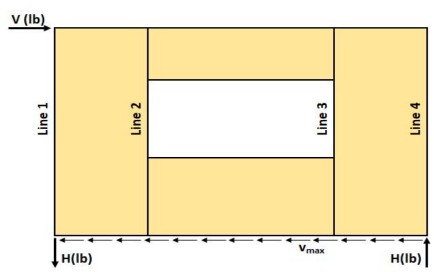
R1-F1 = 168 lbf

R2-F2 = 164 lbf

9. Unit shear in corner zones

vc1 = (R1-F1)/L1 = 40 plf

vc2 = (R2-F2)/L2 = 40 plf



Check Summary of Shear Values for One Opening

Line 1: $vc1(h_a+h_b)+v1(h_o)=H?$		164	635	799 lbf
Line 2: $va1(h_a+h_b)-vc1(h_a+h_b)-v1(h_o)=0?$	799	164	635	0
Line 3: $va1(h_a+h_b)-vc2(h_a+h_b)-v1(h_o)=0?$	799	164	635	0
Line 4: $vc2(h_a+h_b)+v2(h_o)=H?$		164	635	799 lbf

Design Summary*

Req. Sheathing Capacity	196 plf	**4-Term Deflection**		**3-Term Deflection**	
Req. Strap Force	494 lbf	**4-Term Story Drift %**		**3-Term Story Drift %**	
Req. HD Force (H)	799 lbf				
Req. Shear Wall Anchorage Force (v_{max})	99 plf				

***The Design Summary assumes that the shear wall is designed as blocked.**

APA

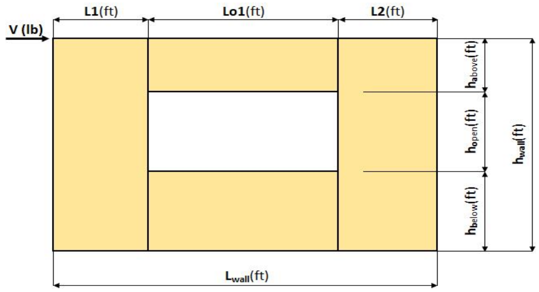
Force Transfer Around Openings Calculator

ONE OPENING

The force transfer around openings (FTAO) method of shear wall analysis is an approach that aims to reinforce the wall such that it performs as if there was no opening. This approach lends certain advantages over segmented shear walls: more versatility, because it allows for narrower wall segments while still meeting the height-to-width ratios and, often, fewer required hold-downs.

Project Information

Code:		**Date:**	
Designer:			
Client:			
Project:			
Wall Line:	R_3.1		



Shear Wall Calculation Variables

V	950 lbf		Opening 1	Adj. Factor Method =	1.25-0.125h/bs	
L1	2.67 ft	h_a	1.42 ft	Wall Pier Aspect Ratio		Adj. Factor
L2	3.75 ft	h_o	4.00 ft	$P1=h_o/L1=$	1.50	N/A
h_{wall}	8.09 ft	h_b	2.67 ft	$P2=h_o/L2=$	1.07	N/A
L_{wall}	11.42 ft	Lo1	5.00 ft			

1. Hold-down forces: $H = Vh_{wall}/L_{wall}$ 673 lbf

2. Unit shear above + below opening

First opening: $va1 = vb1 = H/(h_a+h_b) =$	165 plf

3. Total boundary force above + below openings

First opening: O1 = va1 x (Lo1) =	823 lbf

4. Corner forces

F1 = O1(L1)/(L1+L2) =	342 lbf
F2 = O1(L2)/(L1+L2) =	481 lbf

5. Tributary length of openings

T1 = (L1*Lo1)/(L1+L2) =	2.08 ft
T2 = (L2*Lo1)/(L1+L2) =	2.92 ft

6. Unit shear beside opening

v1 = (V/L)(L1+T1)/L1 =	148 plf	
v2 = (V/L)(T2+L2)/L2 =	148 plf	
Check v1*L1+v2*L2=V?	950 lbf	**OK**

7. Resistance to corner forces

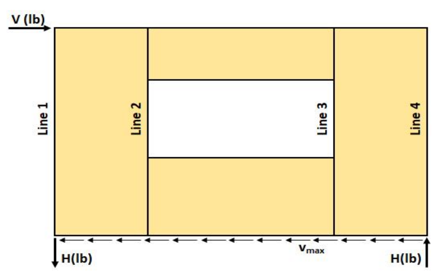
R1 = v1*L1 =	395 lbf
R2 = v2*L2 =	555 lbf

8. Difference corner force + resistance

R1-F1 =	53 lbf
R2-F2 =	74 lbf

9. Unit shear in corner zones

vc1 = (R1-F1)/L1 =	20 plf
vc2 = (R2-F2)/L2 =	20 plf



Check Summary of Shear Values for One Opening

Line 1: $vc1(h_a+h_b)+v1(h_o)=H?$		81	592	673 lbf
Line 2: $va1(h_a+h_b)-vc1(h_a+h_b)-v1(h_o)=0?$	673	81	592	0
Line 3: $va1(h_a+h_b)-vc2(h_a+h_b)-v1(h_o)=0?$	673	81	592	0
Line 4: $vc2(h_a+h_b)+v2(h_o)=H?$		81	592	673 lbf

Design Summary*

Req. Sheathing Capacity	165 plf	**4-Term Deflection**		**3-Term Deflection**	
Req. Strap Force	481 lbf	**4-Term Story Drift %**		**3-Term Story Drift %**	
Req. HD Force (H)	673 lbf				
Req. Shear Wall Anchorage Force (v_{max})	83 plf				

***The Design Summary assumes that the shear wall is designed as blocked.**

APA

Force Transfer Around Openings Calculator

ONE OPENING

The force transfer around openings (FTAO) method of shear wall analysis is an approach that aims to reinforce the wall such that it performs as if there was no opening. This approach lends certain advantages over segmented shear walls: more versatility, because it allows for narrower wall segments while still meeting the height-to-width ratios and, often, fewer required hold-downs.

Project Information

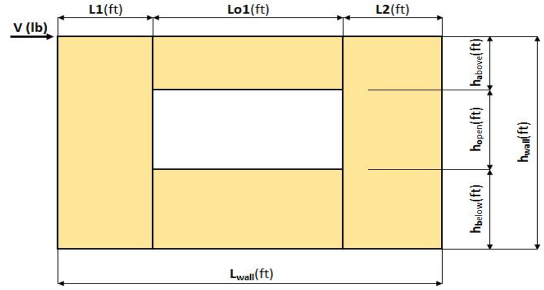
Code: **Date:**

Designer:

Client:

Project:

Wall Line: R_A.1



Shear Wall Calculation Variables

V	3012 lbf		Opening 1	Adj. Factor Method =	1.25-0.125h/bs
L1	4.92 ft	h_a	1.42 ft	Wall Pier Aspect Ratio	Adj. Factor
L2	5.00 ft	h_o	4.00 ft	$P1=h_o/L1=$ 0.81	N/A
h_{wall}	8.09 ft	h_b	2.67 ft	$P2=h_o/L2=$ 0.80	N/A
L_{wall}	14.92 ft	Lo1	5.00 ft		

1. Hold-down forces: $H = Vh_{wall}/L_{wall}$ 1633 lbf

2. Unit shear above + below opening

First opening: $va1 = vb1 = H/(h_a+h_b) =$	400 plf

3. Total boundary force above + below openings

First opening: O1 = va1 x (Lo1) =	1998 lbf

4. Corner forces

F1 = O1(L1)/(L1+L2) =	990 lbf
F2 = O1(L2)/(L1+L2) =	1007 lbf

5. Tributary length of openings

T1 = (L1*Lo1)/(L1+L2) =	2.48 ft
T2 = (L2*Lo1)/(L1+L2) =	2.52 ft

6. Unit shear beside opening

v1 = (V/L)(L1+T1)/L1 =	304 plf	
v2 = (V/L)(T2+L2)/L2 =	304 plf	
Check v1*L1+v2*L2=V?	3012 lbf	**OK**

7. Resistance to corner forces

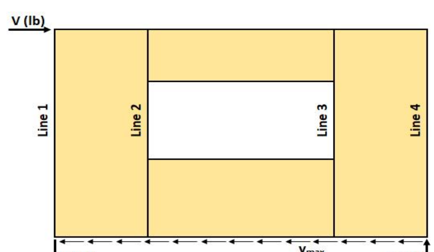
R1 = v1*L1 =	1493 lbf
R2 = v2*L2 =	1519 lbf

8. Difference corner force + resistance

R1-F1 =	503 lbf
R2-F2 =	511 lbf

9. Unit shear in corner zones

vc1 = (R1-F1)/L1 =	102 plf
vc2 = (R2-F2)/L2 =	102 plf



Check Summary of Shear Values for One Opening

Line 1: $vc1(h_a+h_b)+v1(h_o)=H$?		418	1215	1633 lbf
Line 2: $va1(h_a+h_b)-vc1(h_a+h_b)-v1(h_o)=0$?	1633	418	1215	0
Line 3: $va1(h_a+h_b)-vc2(h_a+h_b)-v1(h_o)=0$?	1633	418	1215	0
Line 4: $vc2(h_a+h_b)+v2(h_o)=H$?		418	1215	1633 lbf

Design Summary*

Req. Sheathing Capacity	400 plf	**4-Term Deflection**		**3-Term Deflection**	
Req. Strap Force	1007 lbf	**4-Term Story Drift %**		**3-Term Story Drift %**	
Req. HD Force (H)	1633 lbf				
Req. Shear Wall Anchorage Force (v_{max})	202 plf				

***The Design Summary assumes that the shear wall is designed as blocked.**

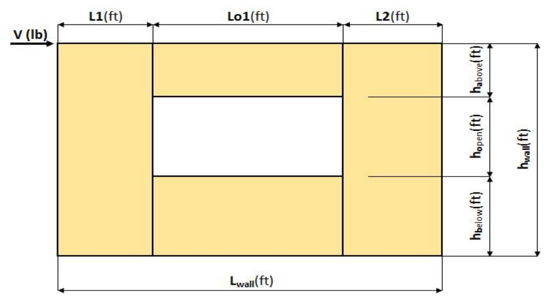
Force Transfer Around Openings Calculator

ONE OPENING

The force transfer around openings (FTAO) method of shear wall analysis is an approach that aims to reinforce the wall such that it performs as if there was no opening. This approach lends certain advantages over segmented shear walls: more versatility, because it allows for narrower wall segments while still meeting the height-to-width ratios and, often, fewer required hold-downs.

Project Information

Code: **Date:**
Designer:
Client:
Project:
Wall Line: R_E.1



Shear Wall Calculation Variables

V	2235 lbf	Opening 1		Adj. Factor Method =		1.25-0.125h/bs
L1	4.92 ft	h_a	1.42 ft	Wall Pier Aspect Ratio		Adj. Factor
L2	5.00 ft	h_o	4.00 ft	$P1=h_o/L1=$	0.81	N/A
h_{wall}	8.09 ft	h_b	2.67 ft	$P2=h_o/L2=$	0.80	N/A
L_{wall}	14.92 ft	Lo1	5.00 ft			

1. Hold-down forces: $H = Vh_{wall}/L_{wall}$ 1212 lbf

2. Unit shear above + below opening

First opening: $va1 = vb1 = H/(h_a+h_b) =$	297 plf

3. Total boundary force above + below openings

First opening: O1 = va1 x (Lo1) =	1483 lbf

4. Corner forces

F1 = O1(L1)/(L1+L2) =	735 lbf
F2 = O1(L2)/(L1+L2) =	748 lbf

5. Tributary length of openings

T1 = (L1*Lo1)/(L1+L2) =	2.48 ft
T2 = (L2*Lo1)/(L1+L2) =	2.52 ft

6. Unit shear beside opening

v1 = (V/L)(L1+T1)/L1 =	225 plf	
v2 = (V/L)(T2+L2)/L2 =	225 plf	
Check v1*L1+v2*L2=V?	2235 lbf	**OK**

7. Resistance to corner forces

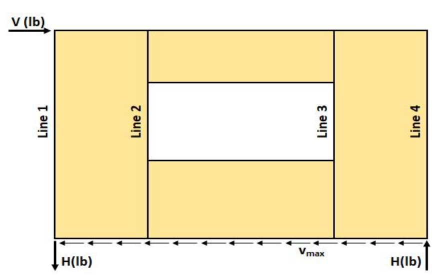
R1 = v1*L1 =	1108 lbf
R2 = v2*L2 =	1127 lbf

8. Difference corner force + resistance

R1-F1 =	373 lbf
R2-F2 =	380 lbf

9. Unit shear in corner zones

vc1 = (R1-F1)/L1 =	76 plf
vc2 = (R2-F2)/L2 =	76 plf



Check Summary of Shear Values for One Opening

Line 1: $vc1(h_a+h_b)+v1(h_o)=H?$		310	902	1212 lbf
Line 2: $va1(h_a+h_b)-vc1(h_a+h_b)-v1(h_o)=0?$	1212	310	902	0
Line 3: $va1(h_a+h_b)-vc2(h_a+h_b)-v1(h_o)=0?$	1212	310	902	0
Line 4: $vc2(h_a+h_b)+v2(h_o)=H?$		310	902	1212 lbf

Design Summary*

Req. Sheathing Capacity	297 plf	**4-Term Deflection**		**3-Term Deflection**	
Req. Strap Force	748 lbf	**4-Term Story Drift %**		**3-Term Story Drift %**	
Req. HD Force (H)	1212 lbf				
Req. Shear Wall Anchorage Force (v_{max})	150 plf				

***The Design Summary assumes that the shear wall is designed as blocked.**

FOUNDATION DESIGN
DESIGN OF FOOTING THICKENED EDGE
ADU

Max Bearing Pressure = 1,500 PSF Per IBC
Max Bearing Pressure = 2,000 PSF Per IBC

1-Story Condition 1 **Location:** 1-Story Sections

										Allowable Level		Strength Level		
Level	**Wall Height (ft)**	**Wall Weight (psf)**	**Floor Trib (ft)**	**DL (psf)**	**LL (psf)**	**RLL (psf)**	**w_{DL} (plf)**	**w_{LL} (plf)**	**w_{RLL} (plf)**	**w_{DL}+w_{LL} (plf)**	**w_{DL}+ 0.75*w_{LL} + 0.75*w_{RLL} (plf)**	**1.4w_{DL} (plf)**	**1.2w_{DL}+ 1.6w_{LL} + 0.5w_{RLL} (plf)**	**1.2w_{DL}+ 1.6w_{RLL} + w_{LL} (plf)**
Roof	10.3	18	7.5	27	0	20	388	0	150	388	501	544	541	706
Conc Wall	0.0	100	0.0	0	0	0	0	0	0	0	0	0	0	0
						Total =	388	0	150	388 plf	501 plf	544 plf	541 plf	706 plf
											^MAX^			

Footing Width Req'd =(w_{DL} +w_{LL})/$q_{a, max}$ = 4.0 in
Min Width per Geotech Report = 12.0 in
Standard min width = 15.0 in
Use = **15.0 in**

Maximum Point Load on 1 Story Foundation

Gravity

Edge/Corner Condition

$P_{max, grav}$=	(15.0 in) * / 12	(24.0 in + / 12	0.0 ft) *	1,500 psf =	3,750 lbs	- 501 plf	* (24.0 in + / 12	0.0 ft) =	2,748 lbs
	width	h_{ftg}	h_{wall}	$q_{a, grav}$					

Continuous Condition

$P_{max, grav}$=	(15.0 in) * / 12	2	(24.0 in + / 12	0.0 ft) *	1,500 psf =	7,500 lbs	- 501 plf	* 2 (24.0 in + / 12	0.0 ft) =	5,496 lbs
	width		h_{ftg}	h_{wall}	$q_{a, grav}$					

Seismic / Wind

Edge/Corner Condition

$P_{max, seis}$=	(15.0 in) * / 12	(24.0 in + / 12	0.0 ft) *	2,000 psf =	5,000 lbs	- 501 plf	* (24.0 in + / 12	0.0 ft) =	3,998 lbs
	width	h_{ftg}	h_{wall}	$q_{a, grav}$					

Continuous Condition

$P_{max, seis}$=	(15.0 in) * / 12	2	(24.0 in + / 12	0.0 ft) *	2,000 psf =	10,000 lbs	- 501 plf	* 2 (24.0 in + / 12	0.0 ft) =	7,996 lbs
	width		h_{ftg}	h_{wall}	$q_{a, grav}$					