



LAGUNA NIGUEL ADUs

PLAN 1

LAGUNA NIGUEL, CA

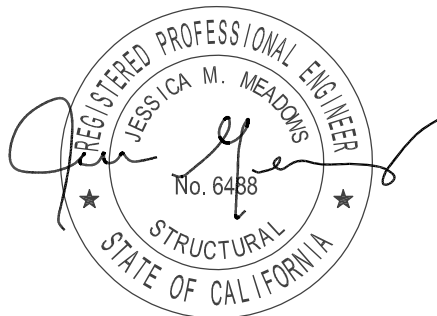
RRM Project No. 2889-01-CU22

STRUCTURAL CALCULATIONS

December 27, 2024

Prepared by: Ian Kelly EIT

Under Direction of: Jessica Meadows, SE



3765 S. Higuera St., Ste. 102 • San Luis Obispo, CA 93401
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LAGUNA NIGUEL ADUs
PLAN 1
LAGUNA NIGUEL, CA
RRM Project No. 2889-01-CU22

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SHEET NO.: Page 1 of 37
PROJECT NO.: 2889-01-CU22
DATE: 12/2024
BY: IPK
CHECKED BY: JMM

PROJECT: LAGUNA NIGUEL ADUs
SUBJECT: PROJECT DESIGN CRITERIA

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
W	2018 NDS	2018 National Design Specification for Wood Construction
SDP	2021 SDPWS	2021 Seismic Design 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 lateral 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.



SHEET NO.: Page 2 of 37
PROJECT NO.: 2889-01-CU22
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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: N/A
Friction + Passive: 100% Greater + 100% Lesser

Retaining Walls:

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

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: 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	
	24 "	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	
	24 "	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
15/32 "	16 "	Stucco	10.4	1	10.4	B Appendix B
		Sheathing	3	1	1.4	B Appendix B
		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
15/32 "	16 "	Stucco	10.4	1	10.4	B Appendix B
		Sheathing	3	1	1.4	B Appendix B
		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



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PROJECT NO.: 2889-01-CU22
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PROJECT: LAGUNA NIGUEL ADUs
SUBJECT: TYPICAL CHORD SPLICE CALCULATIONS

TYPICAL CALCULATIONS

TYPICAL CHORD SPLICE @ 2x6 STUDS

NAILED SPLICES

ALLOWABLE TENSION

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

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

NOTE: NAIL VALUES GOVERN ALL CASES

16d COMMON NAIL CAPACITY

PER 2018 NDS TBL 12N

$$\begin{aligned} t_s &= 1.5 \text{ IN} \\ t_m &= 1.5 \text{ IN} \\ G &= 0.5 \text{ (DF-L)} \\ Z &= 141 \text{ LBS} \\ 16\text{d NAIL DIA} &= 0.162 \text{ IN} \\ \text{PENETRATION, } p &= 1.5" / 1.62" = 0.926 \\ p * Z &= 130 \text{ LBS} \end{aligned}$$

PER 2018 NDS TBL 11.3.1

$$Z' = Z \times C_D \times C_M \times C_t \times C_g \times C_{\Delta} \times C_{eg} \times C_{di} \times C_{tn} \times p$$

$$C_D = 1.6 \text{ [2018 NDS TBL 2.3.2]}$$

$$C_M = C_t = C_g = C_{\Delta} = C_{eg} = C_{di} = C_{tn} = 1.0$$

PROJECT: LAGUNA NIGUEL ADUs
 SUBJECT: SHEAR WALL SCHEDULE

TYPICAL CALCULATIONS

SHEAR WALL SCHEDULE

CONVENTIONAL HOLDOWNS¹: **Yes** 8% Reduction SDPWS Table 4.3A, Footnote 10
 SHORT NAILS²: **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) 1/4"x1 1/2" SDS	1825
DTT2Z-2.5	(2) 2x MIN.	(8) 1/4"x2 1/2" SDS	2145
HDU2	(2) 2x MIN.	(6) 1/4"x2 1/2" SDS	3075
HDU4	(2) 2x MIN.	(10) 1/4"x2 1/2" SDS	4565
HDU5	(2) 2x MIN.	(14) 1/4"x2 1/2" SDS	5645
HDU8	4x MIN.	(20) 1/4"x2 1/2" SDS	6970
HDU11	6x MIN.	(30) 1/4"x2 1/2" SDS	9535
HDU14	6x MIN.	(36) 1/4"x2 1/2" SDS	14445
Hdq8	4x MIN.	(20) 1/4"x3" SDS	7630
HHDQ11	6x MIN.	(24) 1/4"x2 1/2" SDS	9535
HHDQ14	6x MIN.	(30) 1/4"x2 1/2" SDS	13710
HD19	6x MIN.	(5) 1" BOLTS	19070

PROJECT: LAGUNA NIGUEL ADUs
 SUBJECT: TYPICAL PAD FOOTING CALCULATIONS

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

$$\begin{aligned} \text{MAX DL+LL} &= q_{\text{ALLOW}}(b)^2 / 1000 = 6.00 \text{ kips} \\ \text{MAX UPLIFT} &= (0.150 \text{ kcf})b^2t = 0.90 \text{ kips} \end{aligned}$$

BENDING CAPACITY

$$\begin{aligned} \text{FOR } f_y = 60 \text{ ksi, } \rho &\geq (0.0018)bt; & A_{s_req'd} &= 0.78 \text{ in}^2 \\ \text{FOR \#5 EA. WAY; } A_s &= 0.31 \text{ in}^2 & \#5 &= 6 \text{ bars; 3 Bottom Bars} \\ \text{FOR (3) \#5 BARS @ 24" FTG. w/ 3" CLR.;} & & b_w &= 8.69 \text{ in} \\ \text{FOR \#5 BARS @ 18" THICK FTG.;} & & d &= 14.06 \text{ in} \end{aligned}$$

$$\begin{aligned} \phi M_N &= \phi A_s f_y \left(d - \frac{A_s f_y}{1.7 f_c' b_w} \right) = 226.97 \text{ kip-in} \\ \text{AT ASD: } M_{N_ASD} &= 11822 \text{ ft-lbs} \end{aligned}$$

$$q_{U_ftg} = \frac{2M_{N_ASD}}{\left(\frac{1}{2}(b-1')\right)^2} = 94572.1669 \text{ psf/ft} > q_{\text{allow}}, \text{ FLEXURE OK}$$

TWO WAY (PUNCHING) SHEAR

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

$$V_U = 1.6 \left[q_{\text{ALLOW}} \left[b^2 - \left(\frac{6" + d}{12"/ft} \right)^2 \right] \right] = 2.9 \text{ kips} < \phi V_c, \text{ PUNCHING SHEAR OK}$$

ONE WAY (BEAM ACTION) SHEAR

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

$$V_U = 1.6 \left[q_{\text{ALLOW}} \left[\frac{b}{2} - \frac{6" + d}{12"/ft} \right] b \right] = -3.225 \text{ kips} < \phi V_c, \text{ BEAM ACTION SHEAR OK}$$

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

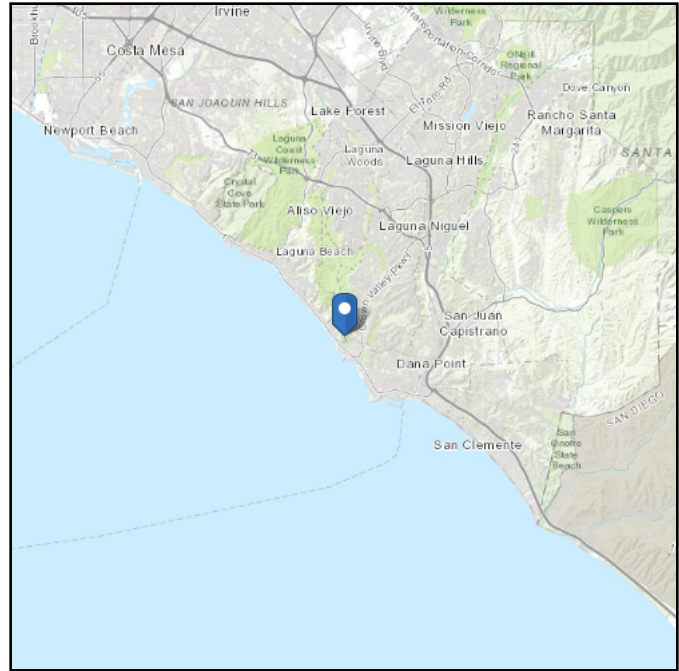
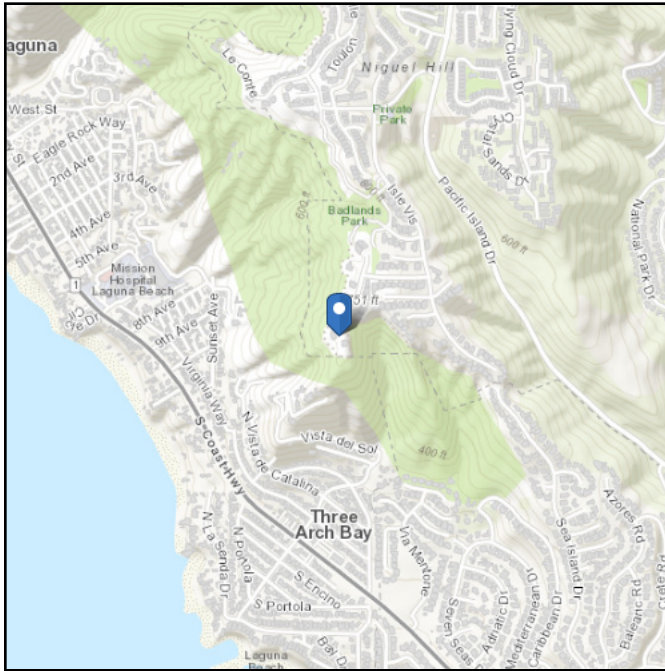


ASCE Hazards Report

Address:
No Address at This Location

Standard: ASCE/SEI 7-16
Risk Category: II
Soil Class: D - Default (see Section 11.4.3)

Latitude: 33.498759
Longitude: -117.733465
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.

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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PROJECT: LAGUNA NIGUEL ADUS
 SUBJECT: WIND COMPONENT & CLADDING - ASCE 7-16

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

Input:

Roof Shape: Gable
 Building Width: 14.0 ft
 Building Length: 25.0 ft
 Building Height: 12.0 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	$A \leq 2 \text{ ft}^2$	$A \leq 10 \text{ ft}^2$	$A \leq 100 \text{ ft}^2$	$A \leq 500 \text{ ft}^2$
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

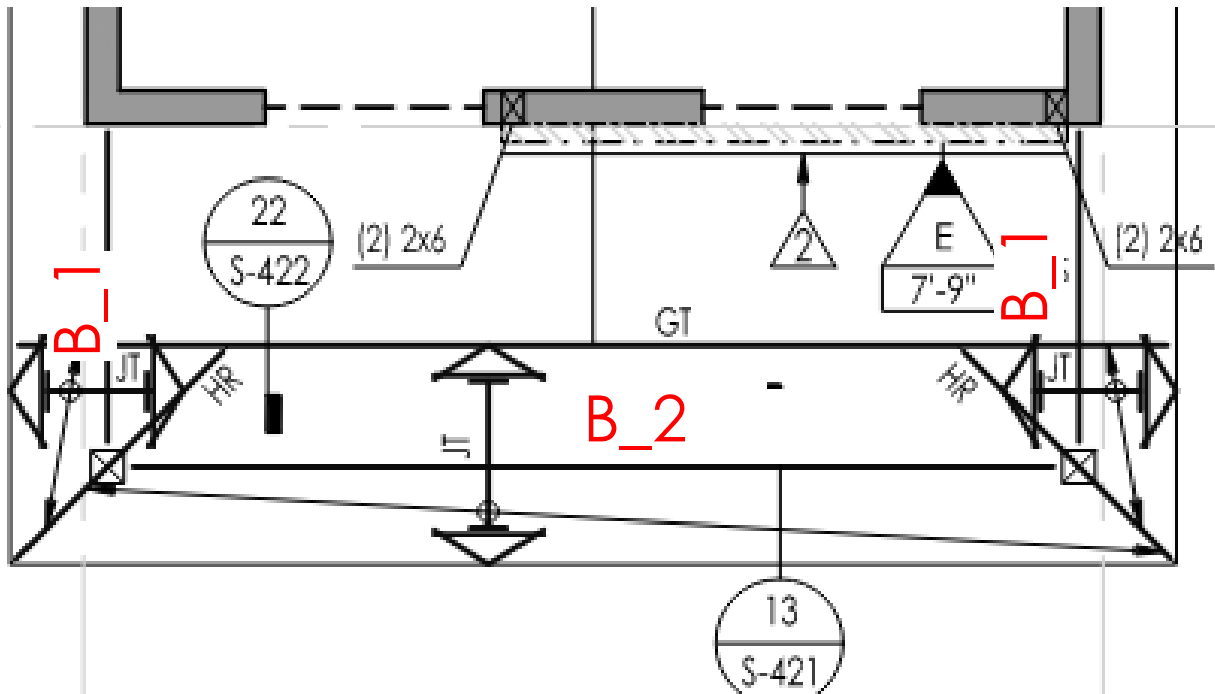
Overhang Pressures, 1.0W: $P_w = q_h(GC_P \pm GC_{pi})$

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

CALIFORNIA RANCH GRAVITY MAP





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

Project Title:
Engineer:
Project ID:
Project Descr:

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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

CODE REFERENCES

Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16
Load Combination Set : IBC 2021

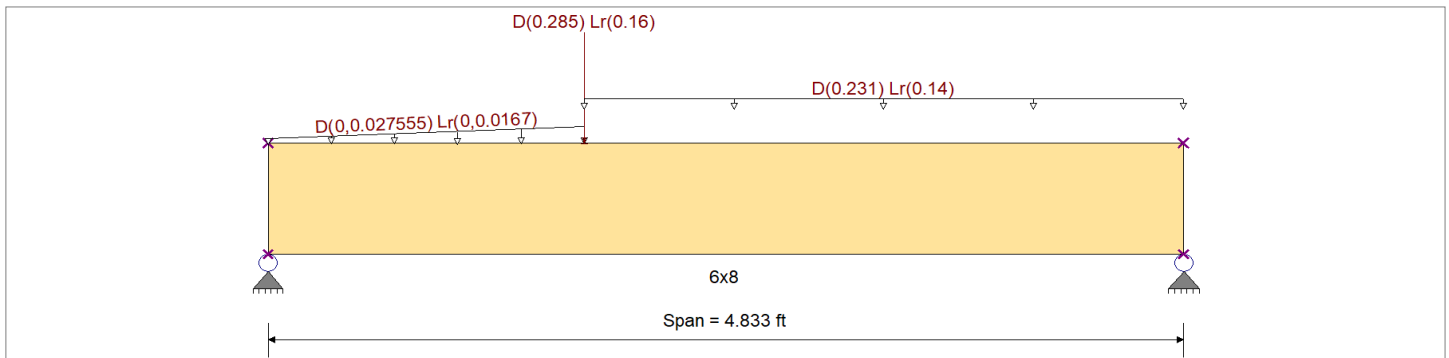
Material Properties

Analysis Method : Allowable Stress Design
Load Combination IBC 2021

Wood Species : Douglas Fir-Larch
Wood Grade : No.1

Beam Bracing : Completely Unbraced

Fb +	1,200.0 psi	E : Modulus of Elasticity	
Fb -	1,200.0 psi	Ebend- xx	1,600.0ksi
Fc - Prll	1,000.0 psi	Eminbend - xx	580.0ksi
Fc - Perp	625.0 psi		
Fv	170.0 psi		
Ft	825.0 psi	Density	31.210pcf



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 -->> 1.670 ft, Trib Width = 0.0->0.8350 ft, (Jack Truss)

Point Load : D = 0.2850, Lr = 0.160 k @ 1.670 ft, (GT)

Uniform Load : D = 0.0330, Lr = 0.020 ksf, Extent = 1.670 -->> 4.833 ft, Tributary Width = 7.0 ft, (RT)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio		=	0.194 : 1	Maximum Shear Stress Ratio		=	0.126 : 1
Section used for this span			6x8	Section used for this span			6x8
fb: Actual	=		290.17 psi	fv: Actual	=		26.87 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	=		2.275ft	Location of maximum on span	=		4.216ft
Span # where maximum occurs	=		Span # 1	Span # where maximum occurs	=		Span # 1
Maximum Deflection							
Max Downward Transient Deflection	0.006 in	Ratio =	9553 >=360	Span: 1 : Lr Only			
Max Upward Transient Deflection	0 in	Ratio =	0 <360	n/a			
Max Downward Total Deflection	0.017 in	Ratio =	3475 >=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	C _{fu}	C _i	C _r	M	fb	F'b	V	fv	F'v
D Only																	
Length = 4.833 ft	1	0.171	0.111	0.90	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.79	184.6	1,077.5	0.47	17.0	153.0
+D+Lr																	
Length = 4.833 ft	1	0.194	0.126	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	1.25	290.2	1,495.0	0.74	26.9	212.5
+D+0.750Lr																	
Length = 4.833 ft	1	0.176	0.115	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	1.13	263.8	1,495.0	0.67	24.4	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



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Wood Beam

Project File: 2889-01 Laguna Niguel ADUs.ec6

LIC# : KW-06017541, Build:20.23.12.07

RRM Design Group

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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	C _{fu}	C _i	C _r	M	fb	F'b	V	fv	F'v
Length = 4.833 ft	1	0.058	0.038	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.48	110.8	1,911.7	0.28	10.2	272.0

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+Lr	1	0.0167	2.417		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	0.725	0.973
Max Upward from Load Combinations	0.725	0.973
Max Upward from Load Cases	0.465	0.617
D Only	0.465	0.617
+D+Lr	0.725	0.973
+D+0.750Lr	0.660	0.884
+0.60D	0.279	0.370
Lr Only	0.260	0.356



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Wood Beam

Project File: 2889-01 Laguna Niguel ADUs.ec6

LIC# : KW-06017541, Build:20.23.12.07

RRM Design Group

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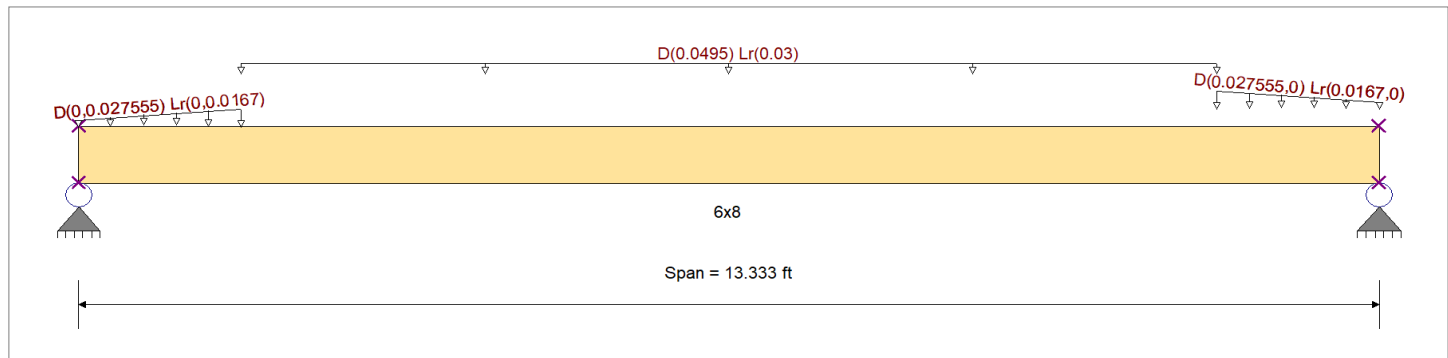
DESCRIPTION: California Ranch B_2

CODE REFERENCES

Calculations per NDS 2018, IBC 2018, CBC 2019, 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 -->> 1.670 ft, Trib Width = 0.0->0.8350 ft, (Jack Truss To Hip)

Varying Uniform Load : D= 0.0330->0.0330, Lr= 0.020->0.020 ksf, Extent = 11.670 -->> 13.333 ft, Trib Width = 0.8350->0.0 ft, (Jack Truss To Hip)

Uniform Load : D = 0.0330, Lr = 0.020 ksf, Extent = 1.670 -->> 11.670 ft, Tributary Width = 1.50 ft, (JT)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.297	1	Maximum Shear Stress Ratio	=	0.083	1
Section used for this span		6x8		Section used for this span		6x8	
fb: Actual	=	441.21 psi		fv: Actual	=	17.61 psi	
F'b	=	1,486.34 psi		F'v	=	212.50 psi	
Load Combination		+D+Lr		Load Combination		+D+Lr	
Location of maximum on span	=	6.667ft		Location of maximum on span	=	12.749ft	
Span # where maximum occurs	=	Span # 1		Span # where maximum occurs	=	Span # 1	
Maximum Deflection							
Max Downward Transient Deflection	0.066 in	Ratio =	2420 >=360	Span: 1 : Lr Only			
Max Upward Transient Deflection	0 in	Ratio =	0 <360	n/a			
Max Downward Total Deflection	0.196 in	Ratio =	816 >=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	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 = 13.284 ft	1	0.272	0.077	0.90	1.00	1.00	0.99	1.000	1.00	1.00	1.00	1.26	292.2	1,073.2	0.32	11.7	153.0
Length = 0.04866 ft	1	0.003	0.077	0.90	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.02	3.7	1,080.0	0.32	11.7	153.0
+D+Lr														0.0	0.00	0.0	0.0
Length = 13.284 ft	1	0.297	0.083	1.25	1.00	1.00	0.99	1.000	1.00	1.00	1.00	1.90	441.2	1,486.3	0.48	17.6	212.5
Length = 0.04866 ft	1	0.004	0.083	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.02	5.6	1,500.0	0.48	17.6	212.5



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Wood Beam

Project File: 2889-01 Laguna Niguel ADUs.ec6

LIC# : KW-06017541, Build:20.23.12.07

RRM Design Group

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DESCRIPTION: California Ranch B_2

Maximum Forces & Stresses for Load Combinations

Load Combination	Segment Length	Span #	Max Stress Ratios										Moment Values			Shear Values		
			M	V	CD	CM	C _t	CLx	C _F	C _{fu}	C _i	C _r	M	fb	F'b	V	fv	F'v
+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 = 13.284 ft	1		0.272	0.076	1.25	1.00	1.00	0.99	1.000	1.00	1.00	1.00	1.74	403.9	1,486.3	0.44	16.1	212.5
Length = 0.04866 ft	1		0.003	0.076	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.02	5.1	1,500.0	0.44	16.1	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 = 13.284 ft	1		0.092	0.026	1.60	1.00	1.00	0.99	1.000	1.00	1.00	1.00	0.75	175.3	1,896.6	0.19	7.0	272.0
Length = 0.04866 ft	1		0.001	0.026	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.01	2.2	1,919.9	0.19	7.0	272.0

Overall Maximum Deflections

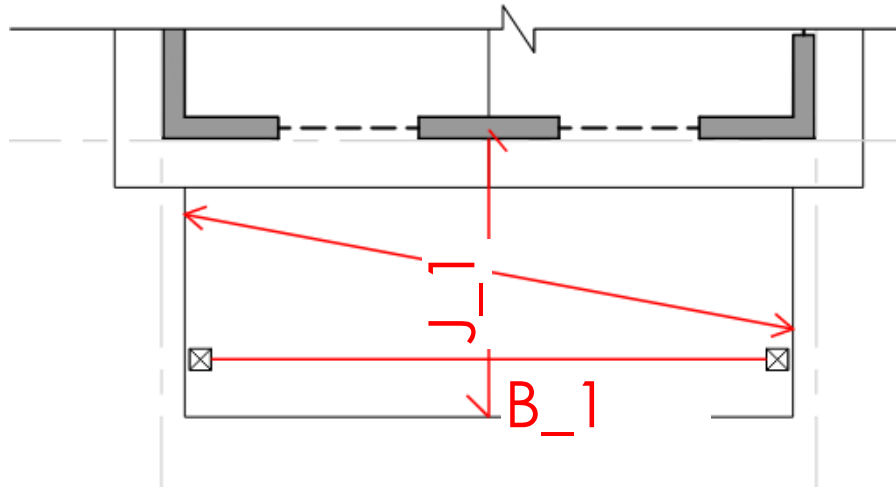
Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+Lr	1	0.1959	6.715		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	0.494	0.494
Max Upward from Load Combinations	0.494	0.494
Max Upward from Load Cases	0.330	0.330
D Only	0.330	0.330
+D+Lr	0.494	0.494
+D+0.750Lr	0.453	0.453
+0.60D	0.198	0.198
Lr Only	0.164	0.164





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Wood Beam

Project File: 2889-01 Laguna Niguel ADUs.ec6

LIC# : KW-06017541, Build:20.23.12.07

RRM Design Group

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DESCRIPTION: Modern Farm J_1

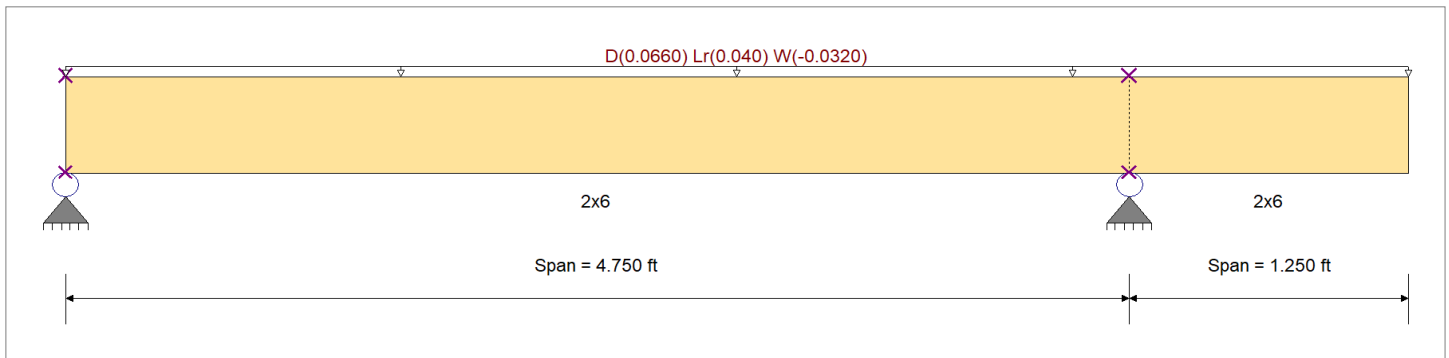
CODE REFERENCES

Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16

Load Combination Set : IBC 2021

Material Properties

Analysis Method :	Allowable Stress Design	Fb +	900 psi	E : Modulus of Elasticity	
Load Combination	IBC 2021	Fb -	900 psi	Ebend- xx	1600ksi
		Fc - Prll	1350 psi	Eminbend - xx	580ksi
Wood Species :	Douglas Fir-Larch	Fc - Perp	625 psi		
Wood Grade :	No.2	Fv	180 psi		
		Ft	575 psi	Density	31.21pcf
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	C _{fu}	C _i	C _r	M	fb	F'b	V	fv	F'v
D Only																	
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																	
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																	
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



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Wood Beam

Project File: 2889-01 Laguna Niguel ADUs.ec6

LIC# : KW-06017541, Build:20.23.12.07

RRM Design Group

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DESCRIPTION: Modern Farm J_1

Maximum Forces & Stresses for Load Combinations

Load Combination	Segment Length	Span #	Max Stress Ratios										Moment Values			Shear Values		
			M	V	CD	CM	C _t	CLx	C _F	C _{fu}	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 Conditions	-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	



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Wood Beam

Project File: 2889-01 Laguna Niguel ADUs.ec6

LIC# : KW-06017541, Build:20.23.12.07

RRM Design Group

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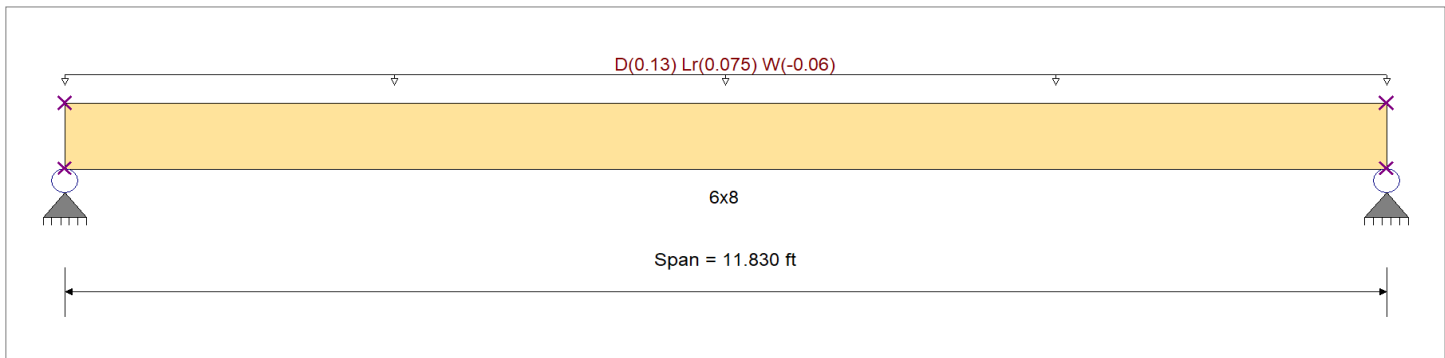
DESCRIPTION: Modern Farm B_1

CODE REFERENCES

Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16
Load Combination Set : IBC 2021

Material Properties

Analysis Method : Allowable Stress Design	Fb +	1200 psi	E : Modulus of Elasticity	
Load Combination IBC 2021	Fb -	1200 psi	Ebend- xx	1600ksi
	Fc - Prll	1000 psi	Eminbend - xx	580ksi
Wood Species : Douglas Fir-Larch	Fc - Perp	625 psi		
Wood Grade : No.1	Fv	170 psi		
	Ft	825 psi	Density	31.21pcf
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	=	0.585 : 1	Maximum Shear Stress Ratio	=	0.194 : 1
Section used for this span		6x8	Section used for this span		6x8
fb: Actual	=	871.00 psi	fv: Actual	=	41.31 psi
F'b	=	1,488.04 psi	F'v	=	212.50 psi
Load Combination		+D+Lr	Load Combination		+D+Lr
Location of maximum on span	=	5.915ft	Location of maximum on span	=	0.000ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1
Maximum Deflection					
Max Downward Transient Deflection	0.107 in	Ratio = 1321 >=360	Span: 1 : Lr Only		
Max Upward Transient Deflection	-0.086 in	Ratio = 1651 >=360	Span: 1 : W Only		
Max Downward Total Deflection	0.307 in	Ratio = 463 >=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	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 = 11.830 ft	1	0.527	0.175	0.90	1.00	1.00	0.99	1.000	1.00	1.00	1.00	2.43	565.7	1,074.0	0.74	26.8	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 = 11.830 ft	1	0.585	0.194	1.25	1.00	1.00	0.99	1.000	1.00	1.00	1.00	3.74	871.0	1,488.0	1.14	41.3	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 = 11.830 ft	1	0.534	0.177	1.25	1.00	1.00	0.99	1.000	1.00	1.00	1.00	3.41	794.7	1,488.0	1.04	37.7	212.5	
+D+0.60W					1.00	1.00	0.99	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0	
Length = 11.830 ft	1	0.221	0.073	1.60	1.00	1.00	0.99	1.000	1.00	1.00	1.00	1.80	419.1	1,899.6	0.55	19.9	272.0	
+D+0.750Lr+0.450W					1.00	1.00	0.99	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0	
Length = 11.830 ft	1	0.360	0.119	1.60	1.00	1.00	0.99	1.000	1.00	1.00	1.00	2.94	684.7	1,899.6	0.89	32.5	272.0	



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

Project Title:
Engineer:
Project ID:
Project Descr:

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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	Segment Length	Span #	Max Stress Ratios										Moment Values			Shear Values		
			M	V	CD	CM	C _t	CLx	C _F	C _{fu}	C _i	C _r	M	fb	F'b	V	fv	F'v
+D+0.450W						1.00	1.00	0.99	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 11.830 ft	1		0.240	0.079	1.60	1.00	1.00	0.99	1.000	1.00	1.00	1.00	1.96	455.7	1,899.6	0.59	21.6	272.0
+0.60D+0.60W						1.00	1.00	0.99	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 11.830 ft	1		0.102	0.034	1.60	1.00	1.00	0.99	1.000	1.00	1.00	1.00	0.83	192.8	1,899.6	0.25	9.1	272.0
+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 = 11.830 ft	1		0.179	0.059	1.60	1.00	1.00	0.99	1.000	1.00	1.00	1.00	1.46	339.4	1,899.6	0.44	16.1	272.0

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+Lr	1	0.3065	5.958		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	1.265	1.265
Max Upward from Load Combinations	1.265	1.265
Max Upward from Load Cases	0.822	0.822
Max Downward from all Load Conditio	-0.355	-0.355
Max Downward from Load Cases (Resis	-0.355	-0.355
D Only	0.822	0.822
+D+Lr	1.265	1.265
+D+0.750Lr	1.155	1.155
+D+0.60W	0.609	0.609
+D+0.750Lr+0.450W	0.995	0.995
+D+0.450W	0.662	0.662
+0.60D+0.60W	0.280	0.280
+0.60D	0.493	0.493
Lr Only	0.444	0.444
W Only	-0.355	-0.355

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):	367.8	plf
Roof Live Load (L _r):	160	plf
Live Load (L):	0	plf
Total Stud Load:	527.8	plf

Assumptions:

$L_{eyy} = 0$
 $L_{exx} = \text{Stud Height}$
 $F_c(\text{Perp.}) = 625 \text{ psi}$
 $C_m = C_t = 1.0$
 Basic Load Combinations

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

Lumber Stresses:

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

Stud Design:

Stud Size: 2x6

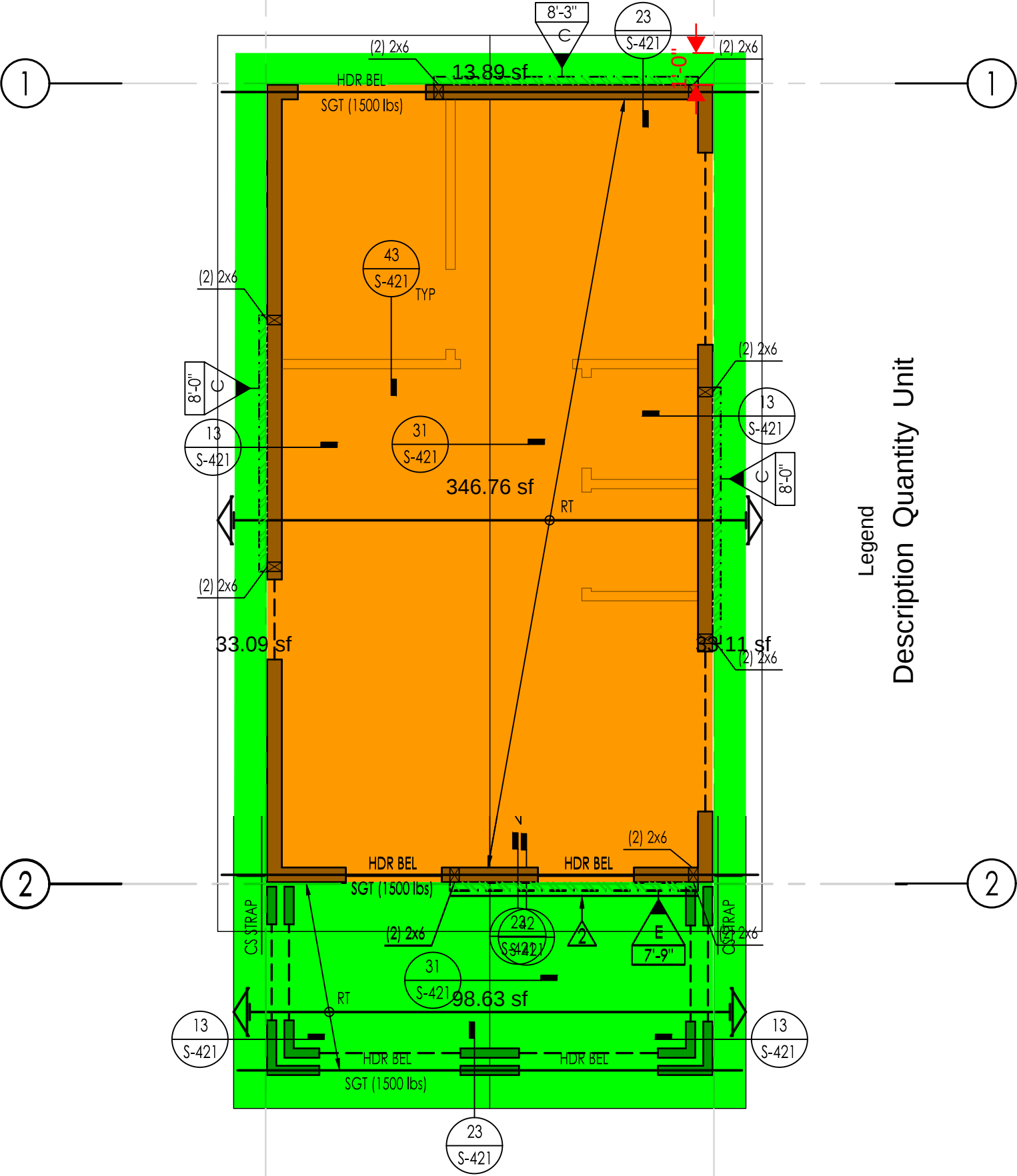
$A = 8.25 \text{ in}^2$
 $I = 20.80 \text{ in}^4$
 $S = 7.56 \text{ in}^3$
 $C_r = 1.15$

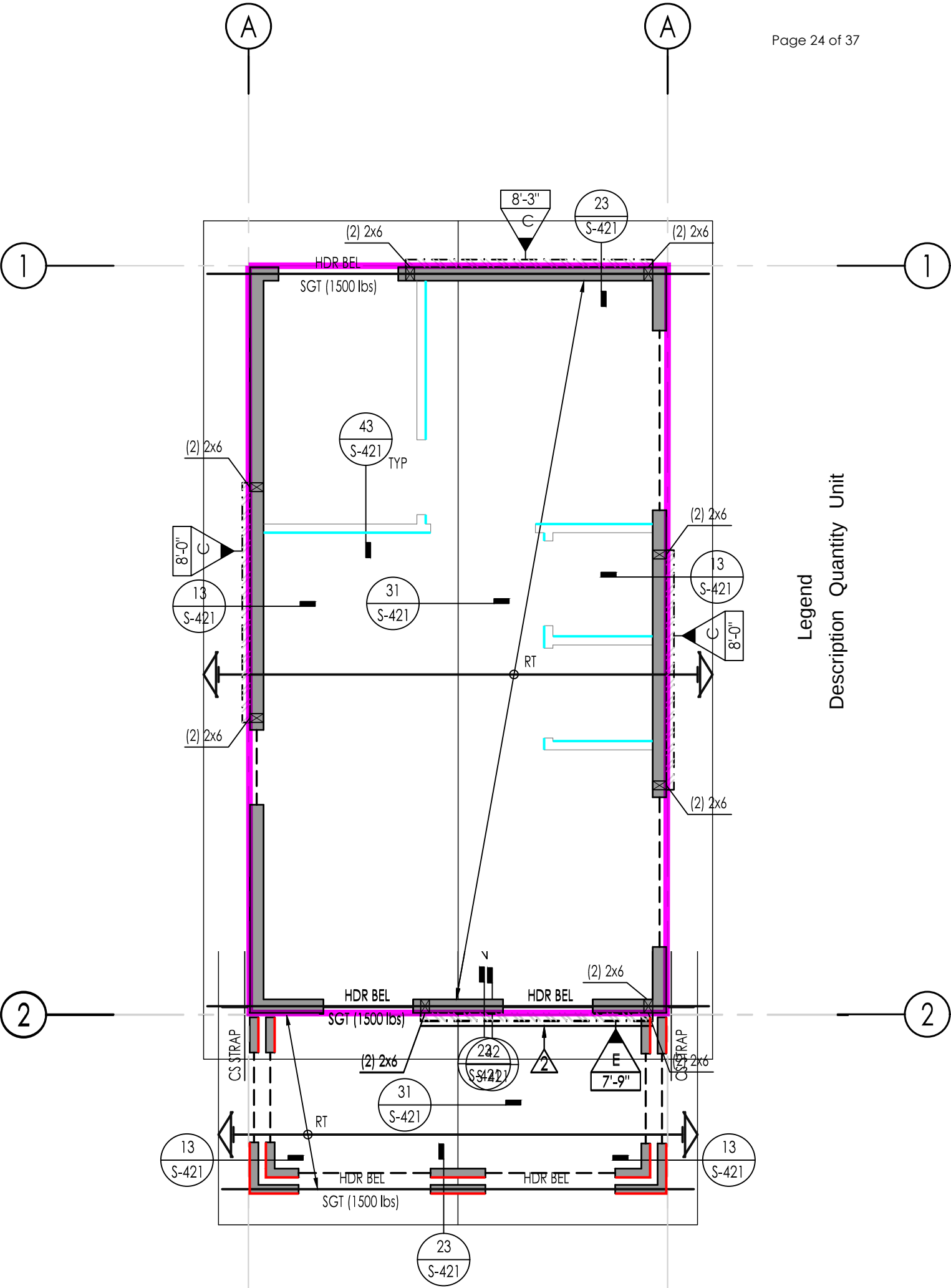
	D+L or D+L_r: (Eq. 16-9, 10)	D+0.75L+0.75L_r: (Eq. 16-11)	D+0.75L+0.45W+0.75L_r: (Eq. 16-13)	D+0.6W: (Eq. 16-12)
P _{total} =	703.7 lbs.	650.4 lbs.	650.4 lbs.	490.4 lbs.
f _c =	85.3 psi	78.8 psi	78.8 psi	59.4 psi
F' _c =	1108.7 psi	1108.7 psi	1218.2 psi	1218.2 psi
Axial Stress Ratio:	0.08	0.07	0.06	0.05
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	0.13	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 1 Mediterranean





SEISMIC BASE SHEAR

EQUIVALENT LATERAL FORCE PROCEDURE
ASCE 7-16 SECTION 12.8 (As amended by 2022 CBC)
BUILDING: **Plan 1**

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 Req'd?	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		
$R =$	6.5	Response Modification Coefficient	A Table 12.2-1
$\Omega_o =$	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 amended by 2022 CBC)

BUILDING: Plan 1

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		
$V =$	4.3 k	4.3 k	Equivalent lateral force procedure base shear	$V = C_s W$	A 12.8.1 (Eqn 12.8-1)
	0.12088	0.12088			



PROJECT: LAGUNA NIGUEL ADUS
SUBJECT: VERTICAL DISTRIBUTION OF FORCES

SHEET NO.:
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 1

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
$F_{px} =$	See Table		$F_{px} = \frac{\sum_{i=x}^n F_i}{\sum_{i=x}^n w_i} w_{px}$	A 12.10.1.1 (Eqn 12.10-1)

EAST-WEST

$C_s =$ 0.169 Base Shear Coefficient See Previous Calcs
 $V =$ 4.3 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 0.2SDS1eW _{px} (k) A 12.10.1.1 (Eqn 12.10-1)	$F_{px} = \frac{\sum_{i=x}^n F_i}{\sum_{i=x}^n w_i} w_{px}$ A 12.10.1.1 (Eqn 12.10-1)	Max 0.4SDS1eW _{px} (k) A 12.10.1.1 (Eqn 12.10-1)	F _{px} (k)	f _{px} (psf)	F _{px} /F _x Ratio
Roof	526	25.1	25.1	10.3	10.3	259.6	1.00	4.3	8.1	0.169	4.3	5.5	4.3	11.1	5.5	10.51	1.30
Total	526	25.1				259.6	1.00	4.3	8.1	0.169							

NORTH-SOUTH

$C_s =$ 0.169 Base Shear Coefficient See Previous Calcs
 $V =$ 4.3 k Equivalent lateral force procedure base shear See Previous Calcs

Vertical Distribution												Diaphragm Shear					
							$C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^n w_i h_i^k}$					Min 0.2SDS1eW_{px} (k) A 12.10.1.1 (Eqn 12.10-1)	$\sum_{i=x}^n F_i$ $F_{px} = \frac{\sum_{i=x}^n F_i}{\sum_{i=x}^n w_i} w_{px}$ A 12.10.1.1 (Eqn 12.10-1)	Max 0.4SDS1eW_{px} (k) A 12.10.1.1 (Eqn 12.10-1)			
Level	Area (sf)	W _x (k)	W _{tot} (k)	h _x (ft)	h _x ^k (ft)	W _x h _x ^k (k-ft)		F _x (k)	f _x (psf)	F _x / W _x	F _{tot} (k)				F _{px} (k)	f _{px} (psf)	F _{px} /F _x Ratio
Roof	526	25.1	25.1	10.3	10.3	259.6	1.00	4.3	8.1	0.169	4.3	5.5	4.3	11.1	5.5	10.51	1.30
Total	526	25.1				259.6	1.00	4.3	8.1	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 1

Floor Height
Below (ft)

Roof (Mean) 10.33
4th 0.00
3rd 0.00
2nd 0.00
Ground 0.00

$\rho_{final} = 1.3$

Light frame construction? YES

ROOF

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

EAST-WEST

Wall	Dimen.	Ht/Width	Strength Loss?
Wall grid	b (ft)	h/b ratio	h/b > 1, bwall/btotal ratio

Top Side

R_1.1	8.3	1.25	0.516

Total = 8.3 < 2 bays

Bot Side

R_2.1	7.8	1.33	0.484

Total = 7.8 < 2 bays

Total Both = 16.0

Code/Standard Reference

Condition a NO
Condition b NO

A 12.3.4.2 (a)
A 12.3.4.2 (b)

$\rho = 1.3$

NORTH-SOUTH

Wall	Dimen.	Ht/Width	Strength Loss?
Wall grid	b (ft)	h/b ratio	h/b > 1, bwall/btotal ratio

Left Side

R_A.1	8.0	1.29	0.500

Total = 8.0 < 2 bays

Right Side

R_B.1	8.0	1.29	0.500

Total = 8.0 < 2 bays

Total Both = 16.0

Code/Standard Reference

Condition a NO
Condition b NO

A 12.3.4.2 (a)
A 12.3.4.2 (b)



PROJECT: LAGUNA NIGUEL ADUS

SUBJECT: WIND BASE SHEAR CALCULATION

SHEET NOPage 30 of 37

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 1

Parameter	Value	Description	Eqn	Code/Standard Reference
Building =	Plan 1			
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	Mean roof height of building		
K _{z,windward} =	0.85	MWFRS Velocity pressure exposure coefficient, windward	$K_z = 2.01 (15/z_g)^{2/\alpha}$ z < 15 ft	A Tbl 26.10-1
K _{z,leeward} =	0.85	or leeward	$K_z = 2.01 (z/z_g)^{2/\alpha}$ z > 15 ft	
K _{zt,windward} =	1.00			
K _{zt,leeward} =	1.00	Topographic factor, windward		A 26.8.2
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
$q_z = 0.00256 K_z K_{zt} K_d K_e V^2$				A Eqn 26.10-1
$q_h = 0.00256 K_h K_{zt} K_d K_e V^2$				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)	F _x (kips)	K _h leeward	q _h (psf)	C _p leeward	P _{net} (psf)	F _x (kips)	F _x total (kips)	w _x (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	27.00 ft	60	0.85	16.67	-0.18	0.45	0.03	0.85	16.67	-0.9	-15.75	-0.9	1.0	35.8	14.1	-0.18	-0.9	10.2	14.1
Roof (Bot)	10.3 ft	4.04 ft	0	25.00 ft	101	0.85	16.67	0.8	8.34	0.8	0.85	16.67	-0.5	-10.09	-1.0	1.9	74.4	25.5	0.8	-0.5	18.4	25.5
Base	0.0 ft	0.00 ft	0	25.00 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
161						2.8 k (Seismic Gov'n)																

NORTH-SOUTH						+Internal										-Internal						
						Windward					Leeward					Total		Windward		Leeward		Total
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)	F _x (kips)	K _h leeward	q _h (psf)	C _p leeward	P _{net} (psf)	F _x (kips)	F _x total (kips)	w _x (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	16.00 ft	35	0.85	16.67	-0.18	0.45	0.02	0.85	16.67	-0.9	-15.75	-0.6	0.6	35.8	14.1	-0.18	-0.9	10.2	14.1
Roof (Bot)	10.3 ft	4.04 ft	0	14.00 ft	57	0.85	16.67	0.8	8.34	0.5	0.85	16.67	-0.5	-10.09	-0.6	1.0	74.4	25.5	0.8	-0.5	18.4	25.5
Base	0.0 ft	0.00 ft	0	14.00 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
92						1.6 k (Seismic Gov'n)																

$f_{p, \text{Roof}}$ =	8.08
$w_{p, \text{Roof}}$ =	110.3
Diaph F_{px}/F_x Ratio =	1.30

 $\Omega_0 = 2.5$

Reduce by 0.5 for structures with flexible diaphragms per A Tbl 12.2-1 footnote b.

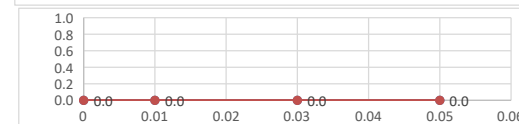
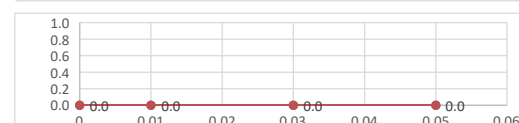
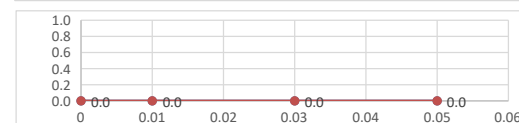
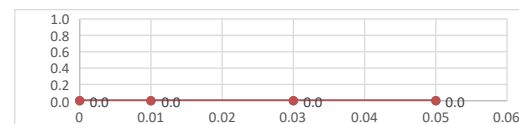
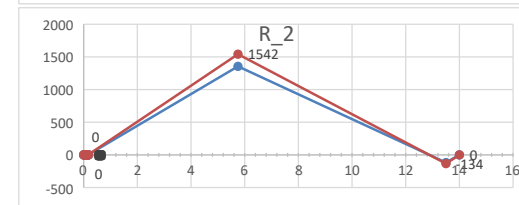
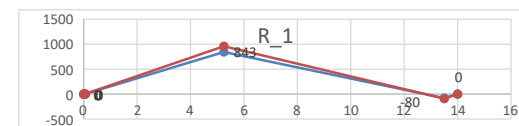
Flexible Diaphragm? = YES

B1_Wall Shear

$f_{p, \text{Roof}}$	=	8.08
$w_{p, \text{Roof}}$	=	110.3
Diaph Fpx/Fx Ratio	=	1.30

Irreg. Exists =	YES
$\rho =$	1.3
$\Omega_0 =$	2.5

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
 $\rho = 1.0$ for diaphragm loads determined using Eq. 12.10-1 per A Ch 12.3.4.1 Note 7.
 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

[illegible]

SHEAR WALL DESIGN SEGMENTED OR FORCE TRANSFER METHOD

Per SDPWS-2021 Table 4.3A

ROOF: NORTH-SOUTH

f_p , Roof = 8.08
 W_p , Roof = 110.3
 $\rho = 1.3$
 $\Omega_0 = 2.5$ Reduce by 0.5 for structures with flexible diaphragms per A Tbl 12.2-1 footnote b.
 Diaph Fpx/Fx Ratio = 1.30 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)	ΣF ^{Above} (lb)	ΣF ^{Above} Transf* North (lb)	ΣF _x North (lb)	F _{tot} (lb)	V _{seis} = pF _{tot} / (Final b) (plf)	V _{seis, asd} = 0.7*pF _{tot} / (Final b) (plf)	Trib Width (ft)	ΣF ^{Above} (lb)	ΣF ^{Above} Transf* (lb)	ΣF _x (lb)	F _{tot} (lb)	V _{wind} = F _{tot} /b (plf)	V _{wind, asd} = 0.6*F _{tot} /b (plf)	Wall Height (ft)	Factor	Wall Type	Sheath Sides 1 or 2	Allow Shear Capacity S 4.3.3 (plf)	Edge Nail Spacing in		
Wall?	R_A	8.0		8.0	West	33	0	0	267	2,126	345	242					772	96	58								
Wall	R_A.1	8.0		8.0	East	230		0	1,859	2,126	2,126		7				772	772		8.08	1.00	A	1	266	6		
No Wall																											
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Wall?	R_B	8.0		8.0	West	230	0	0	1,859	2,126	345	242	7.0				772	772	96	58							
					East	33		0	267									772	772								
Wall	R_B.1	8.0		8.0						2,126	2,126							772	772		8.08	1.00	A	1	266	6	
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Per A Ch 12.10.2.1 In structures or portions thereof braced entirely by light frame shear walls overstrength need not apply

SHEAR WALL POST/HOLDOWN DESIGN

Building: Plan 1

S_{DS}1.100

M_{R,ASD} = M_R * (0.6-0.14S_{DS})

**Compression members designed by Simpson. See ATS summary that follows

Stud Spc16

C_{ASD, Total} = M_{OI}/1.4d + w_{D+L}*2 ft + (P_D+P_L)

Concentrated Dead Load Neglected in Uplift Calculation

E_{hold} = 5.5

Floor ID

E_{strap} = 2.75

ρ = 1.3

Use 80% ICC Capacity per DSA IR A-5, §4.2?

No

Dimensions				LOADING												SEISMIC										WIND										RESULTS									
LEVEL & ID				LENGTHS			LOADS									FORCES				ASD UPLIFT				ASD COMP	FORCES				ASD UPLIFT				ASD COMP	USE											
				Wall L		Ecc. Anch	Distrib.				Point Loads					Shear	Moments				Holdown		Strap		POST	Shear	Moments		Holdown		Strap		POST	Holdown		Strap		Post						Post for Sill Crush	
Level	Wall ID	Ht (ft)	Post Ht (ft)	Lw (ft)	d'hold (ft)	d'strap (ft)	W _D , wall (psf)	W _D , ft/ft (psf)	W _L (psf)	Trib Width (ft)	P _D (lbs)	P _L (lbs)	P _E (lbs)	P _W (lbs)	P Loc. (ft)	P _{seis} (lbs)	M _{OI} (ft-lbs)	M _R (ft-lbs)	M _{R, ASD} (ft-lbs)	Overturn w/o Gravity	[0.7r(M _{OI})-M _{R,ASD}]/d (lbs)	Diff. Load per floor (lbs)	[0.7r(M _{OI})-M _{R,ASD}]/d (lbs)	Diff. Load per floor (lbs)	C _{ASD, Seis} (lbs)	V _{wind} (plf)	M _{OI} (ft-lbs)	M _{R, ASD} (ft-lbs)	[0.6(M _{OI})-M _{R,ASD}]/d (lbs)	Diff. Load per floor (lbs)	[0.6(M _{OI})-M _{R,ASD}]/d (lbs)	Diff. Load per (lbs)	C _{ASD, Seis} (lbs)	Model -	Cap. (lbs)	Model -	Capacity (lbs)	Memb -	Tens. Cap. (lbs)	Mem b -	Comp. Cap. (lbs)	Memb -	Sill Comp. Cap. (lbs)		
R	R_1.1	10.3	8.1	8.3	7.563	7.8	18	27	20	2.5	0	0	0	0	0.0	1730	17,876	7,248	3,233	2,364	1,724	1,724	1,673	1,673	2,479	1378.2	14,241	4,349	555	555	538	538	1,414	HDU2	3,075	CS16	1,705	2x6	9,867	2x6	8,680	2x6	4,383		
Wall = 6"																					Wall Cond = Midwall												SSTB16		3780	SSTB16	2550	Bolt = 0		Foundation		Stem Wall			
R	R_2.1	10.3	8.1	7.8	7.063	7.3	18	27	20	2.5	0	0	0	0	0.0	2538	26,229	6,396	2,853	3,714	2,976	2,976	2,882	2,882	3,707	1378.2	14,241	3,838	666	666	646	646	1,494	HDU2	3,075	CMSTC16	4,610	2x6	9,867	2x6	8,680	2x6	4,383		
Wall = 6"																					Wall Cond = Midwall												SSTB16		3780	SSTB20	3145	Bolt = 0		Foundation		Stem Wall			
R	R_A.1	10.3	8.1	8.0	7.313	7.5	18	27	20	8	0	0	0	0	0.0	2126	21,969	11,568	5,159	3,004	2,028	2,028	1,967	1,967	3,290	771.8	7,975	6,941	-295	-295	-286	-286	1,136	HDU2	3,075	CS14	2,490	2x6	9,867	2x6	8,680	2x6	4,383		
Wall = 6"																					Wall Cond = Midwall												SSTB16		3780	SSTB16	2550	Bolt = 0		Foundation		Stem Wall			
R	R_B.1	10.3	8.1	8.0	7.313	7.5	18	27	20	8	0	0	0	0	0.0	2126	21,969	11,568	5,159	3,004	2,028	2,028	1,967	1,967	3,290	771.8	7,975	6,941	-295	-295	-286	-286	1,136	HDU2	3,075	CS14	2,490	2x6	9,867	2x6	8,680	2x6	4,383		
Wall = 6"																					Wall Cond = Midwall												SSTB16		3780	SSTB16	2550	Bolt = 0		Foundation		Stem Wall			



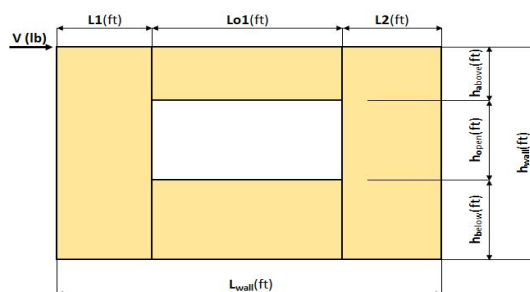
Force Transfer Around Openings Calculator

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:	Gridline 2A		



Shear Wall Calculation Variables

Shear Wall Calculation Variables					
V	1777 lbf	Opening 1		Adj. Factor Method = 1.25-0.125h/bs	
L1	2.75 ft	h _a	1.42 ft	Wall Pier Aspect Ratio	Adj. Factor
L2	2.00 ft	h _o	4.00 ft	P1=h _o /L1= 1.45	N/A
h _{wall}	8.09 ft	h _b	2.67 ft	P2=h _o /L2= 2.00	N/A
L _{wall}	7.75 ft	L _{o1}	3.00 ft		

1. Hold-down forces: $H = V_{hwall}/L_{wall}$ 1854 lbf

2. Unit shear above + below opening
First opening: $va1 = vb1 = H/(h_a + h_b) = 454 \text{ plf}$

3. Total boundary force above + below openings
First opening: $O1 = va1 \times (Lo1) = 1361 \text{ lbf}$

4. Corner forces
 $F1 = O1(L1)/(L1+L2) = 788 \text{ lbf}$
 $F2 = O1(L2)/(L1+L2) = 573 \text{ lbf}$

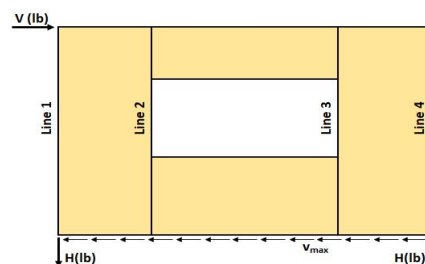
5. Tributary length of openings
 $T1 = (L1 \times Lo1)/(L1+L2) = 1.74 \text{ ft}$
 $T2 = (L2 \times Lo1)/(L1+L2) = 1.26 \text{ ft}$

6. Unit shear beside opening
 $v1 = (V/L)(L1+T1)/L1 = 374 \text{ plf}$
 $v2 = (V/L)(T2+L2)/L2 = 374 \text{ plf}$
Check $v1 \times L1 + v2 \times L2 = V?$ 1777 lbf **OK**

7. Resistance to corner forces
 $R1 = v1 \times L1 = 1029 \text{ lbf}$
 $R2 = v2 \times L2 = 748 \text{ lbf}$

8. Difference corner force + resistance
 $R1 - F1 = 241 \text{ lbf}$
 $R2 - F2 = 175 \text{ lbf}$

9. Unit shear in corner zones
 $vc1 = (R1 - F1)/L1 = 88 \text{ plf}$
 $vc2 = (R2 - F2)/L2 = 88 \text{ plf}$



Check Summary of Shear Values for One Opening

Line 1: $vc1(h_a + h_b) + v1(h_o) = H?$	358	1496	1854 lbf
Line 2: $va1(h_a + h_b) - vc1(h_a + h_b) - v1(h_o) = 0?$	1854	358	0
Line 3: $va1(h_a + h_b) - vc2(h_a + h_b) - v1(h_o) = 0?$	1854	358	0
Line 4: $vc2(h_a + h_b) + v2(h_o) = H?$	358	1496	1854 lbf

Design Summary*

Req. Sheathing Capacity	454 plf	4-Term Deflection		3-Term Deflection	
Req. Strap Force	788 lbf	4-Term Story Drift %		3-Term Story Drift %	
Req. HD Force (H)	1854 lbf				
Req. Shear Wall Anchorage Force (V_{max})	229 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

Level	Wall Height (ft)	Wall Weight (psf)	Floor Trib (ft)	DL (psf)	LL (psf)	RLL (psf)	WDL (plf)	WLL (plf)	WRL (plf)	Allowable Level		Strength Level		
										WDL+WLL (plf)	WDL+0.75*WLL + 0.75*WRL (plf)	1.4WDL (plf)	1.2WDL+1.6WLL + 0.5WRL (plf)	1.2WDL+1.6WRL + WLL (plf)
Roof	10.3	18	8.0	27	0	20	402	0	160	402	522	563	562	738
Conc Wall	0.0	100	0.0	0	0	0	0	0	0	0	0	0	0	0
Total =							402	0	160	402 plf	522 plf	563 plf	562 plf	738 plf

Footing Width Req'd = (WDL + WLL) / q_{a, max} = 4.2 in
 Min Width per Geotech Report = 12.0 in
 Standard min width = 15.0 in
 Use = 15.0 in

^MAX^

Maximum Point Load on 1 Story Foundation

Gravity

Edge/Corner Condition

$$P_{\max, \text{grav}} = \frac{(15.0 \text{ in}) *}{12} \frac{(24.0 \text{ in} +)}{12} 0.0 \text{ ft} * 1,500 \text{ psf} = 3,750 \text{ lbs} - 522 \text{ plf} \frac{* (24.0 \text{ in} +)}{12} 0.0 \text{ ft} = 2,706 \text{ lbs}$$

width h_{ftg} h_{wall} q_{a, grav}

Continuous Condition

$$P_{\max, \text{grav}} = \frac{(15.0 \text{ in}) *}{12} 2 \frac{(24.0 \text{ in} +)}{12} 0.0 \text{ ft} * 1,500 \text{ psf} = 7,500 \text{ lbs} - 522 \text{ plf} \frac{* 2 (24.0 \text{ in} +)}{12} 0.0 \text{ ft} = 5,412 \text{ lbs}$$

width h_{ftg} h_{wall} q_{a, grav}

Seismic / Wind

Edge/Corner Condition

$$P_{\max, \text{seis}} = \frac{(15.0 \text{ in}) *}{12} \frac{(24.0 \text{ in} +)}{12} 0.0 \text{ ft} * 2,000 \text{ psf} = 5,000 \text{ lbs} - 522 \text{ plf} \frac{* (24.0 \text{ in} +)}{12} 0.0 \text{ ft} = 3,956 \text{ lbs}$$

width h_{ftg} h_{wall} q_{a, grav}

Continuous Condition

$$P_{\max, \text{seis}} = \frac{(15.0 \text{ in}) *}{12} 2 \frac{(24.0 \text{ in} +)}{12} 0.0 \text{ ft} * 2,000 \text{ psf} = 10,000 \text{ lbs} - 522 \text{ plf} \frac{* 2 (24.0 \text{ in} +)}{12} 0.0 \text{ ft} = 7,912 \text{ lbs}$$

width h_{ftg} h_{wall} q_{a, grav}