LOBANA ENGINEERING INC
Structural Engineering
paullobana@aol.com

REVIEWED
FOR
CODE COMPLIANCE
Jun 03, 2024
INTERWEST CONSULTING GROUP

PROJECT: **Two Story Addition For:**
Bird Residence @
510 Santa Paula Street
Santa Paula, Ca 93060

Governing Codes: 2022 California Building Code

Engineering Design: Two Story Building

REGISTERED PROFESSIONAL ENGINEER
MANINDER P. S. LOBANA
C050989
CIVIL
STATE OF CALIFORNIA

05/29/2023

885 Patriot Dr, Unit #G, Moorpark, Ca. 93021 805-529-5435 Office, 818-203-2527 Cell

Client Name:	Bird Residence	Date:	05/29/23
Job Address:	510 Santa Paula Street		
City:	Santa Paula, Ca 93060	JOB No.:	Bird

ROOF DEAD LOAD :

ROOFING MATERIAL	=	5.00	psf
ROOF SHTG.		1.80	psf
ROOF RAFTERS or Trusses	=	2.50	psf
Clg Jst	=	2.00	psf
Solar Panels	=	2.00	psf
Insulation-loose	=	1.00	psf
Mech, Elec ETC	=	0.70	psf
SUB-TOTAL	=	15.00	psf

FLAT ROOF DL	=	12.00	psf
ROOF DEAD LOAD	=	16.00	psf
ROOF SNOW LOAD	=	0.00	psf
ROOF LIVE LOAD	=	20.00	psf

EXTERIOR WALL LOADS:

Type 1:

7/8" Stucco.	=	9.00	psf
1/2 in Plywood Sheathing	=	1.50	psf
Wood Studs (2x4 @ 16" o/c)	=	0.90	psf
R19 Insulation	=	0.50	psf
Gypsum Bd. (One Sides)	=	2.50	psf
Miscellaneous	=	0.60	psf
TOTAL EXTERIOR WALL	=	15.00	psf

SOILS DATA:

SOILS REPORT	=	
DATE OF REPORT	=	

Basic Bearing Pressure:

Pads & Cont. Fndt. = Qa	=	1,500	psf
Increase for Width	=	0.0	%
Increase for Depth	=	0.00	%
Max. Soil Pressure - Qa	=	1,500.0	psf
Isolated Foundations	=	1,500	psf
Equiv. Fluid Pressure	=	45.0	pcf

(TYP) FLOOR DEAD LOAD

¾-in Plywood (3 psf per inch.)	=	4.00	psf
FLOOR COVERING	=	4.00	psf
Framing (Estimate 2x12 @ 16" o/c	=	4.50	psf
Ceiling (1/2-in Drywall, 5psf x 1/2 i	=	2.50	psf
Light Wt Conc.	=	0.00	psf
R19 Insulation (3 inch x 0.5psf)	=	1.00	psf
Miscellaneous (Lt Wt Conc)	=	2.0	psf
FLOOR DEAD LOAD	=	18.00	psf
Deck LIVE LOAD	=	60.00	psf

CEILING DEAD LOAD

CEILING JST	=	3.00	psf
5/8" Gypsum Ceiling	=	2.50	psf
Insulation-loose	=	0.50	psf
Total Ceiling Load		6.00	psf

Roof Dead Load w/o clg load	12.00	psf
Ceiling LL	10.00	psf

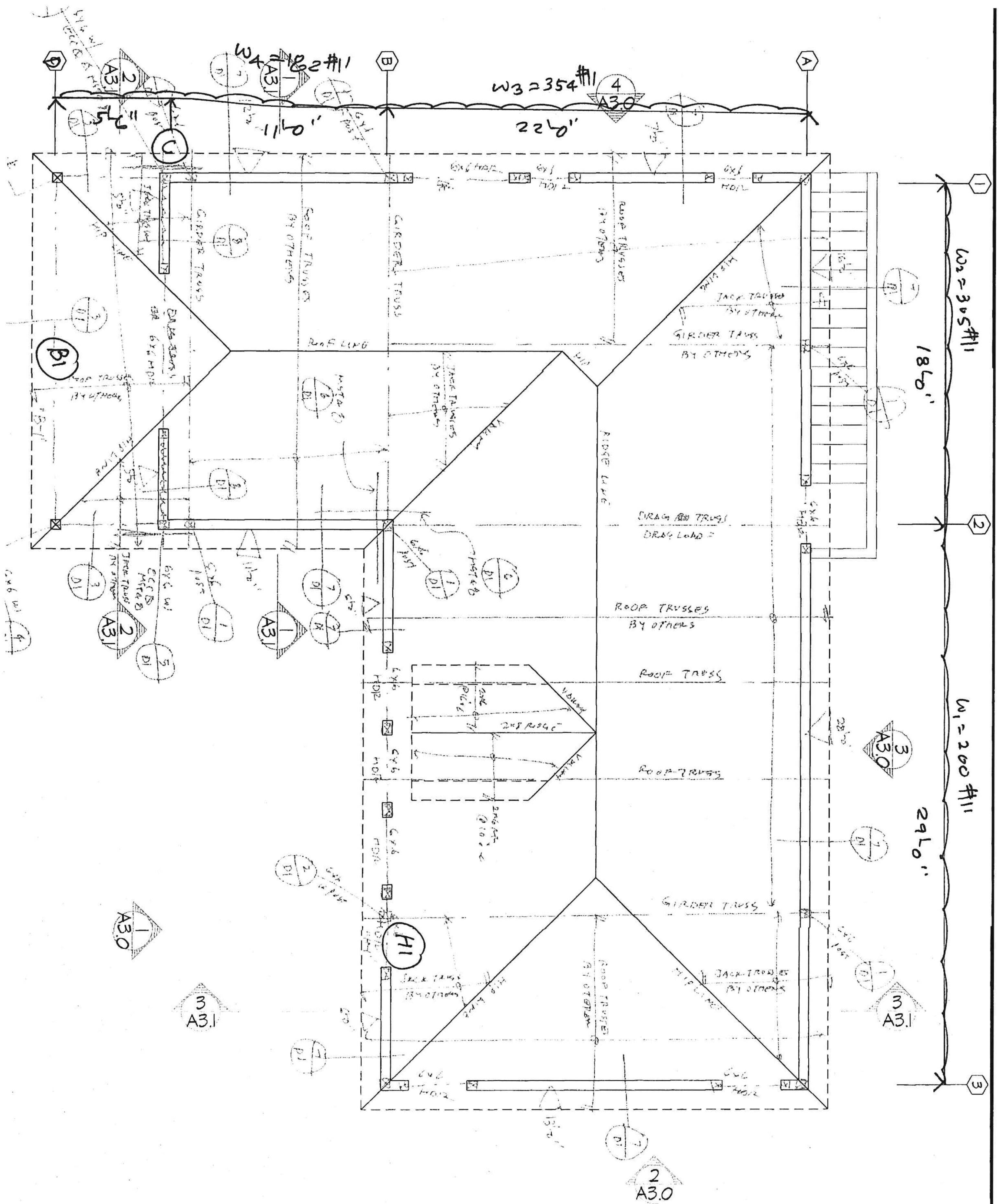
INTERIOR WALL LOADS:

Gypsum Bd. (Both Sides)	=	5.00	psf
1/2-in Plywood sheathing	=	1.50	psf
Wood Studs (2x4 @16" o/c)	=	1.00	psf
Miscellaneous	=	2.50	psf
TOTAL INTERIOR WALL	=	10.00	psf

OTHER LIVE LOADS:

Balcony (Exterior)	=	60.00	psf
	=		
Stairs & Exits	=	60.00	psf

Allow Passive Pressure	=	250	pcf
Allow Soil Friction	=	0.25	


W4 = 182#11
W3 = 354#11
W2 = 305#11
W1 = 200#11
5'-6"
11'-0"
22'-0"
18'-0"
29'-0"
ROOF LINE
RIDGE LINE
HIP LINE
GIRDER TRUSS
ROOF TRUSSES BY OTHERS
ROOF TRUSS
JACK TRUSSES BY OTHERS
GIRDER TRUSS BY OTHERS
DRAG TRUSS
2x8 RIDGE
B1
C
H1
A3.0
A3.1
D1

2ND FLOOR FRAMING

H-1 $L = 4'-0''$

$w = (15+20) \times 22/2 = (165+220)$

$P = (15+20) \times 9/2 \times 22/2 = (743+990)$ @ $1'-6''$

See Comp. output Use 6x6 HDR

B-1 $L = 18'-0''$

$w = (15+20) \times 8/2 = (60+80)$

see comp. output Use 6x12 #1

Project Title:
Engineer:
Project ID:
Project Descr:

(4)

Wood Beam

Project File: 510 Santa Paula-Bird.ec6

LIC# : KW-06014816, Build:20.23.05.01 LOBANA ENGINEERING

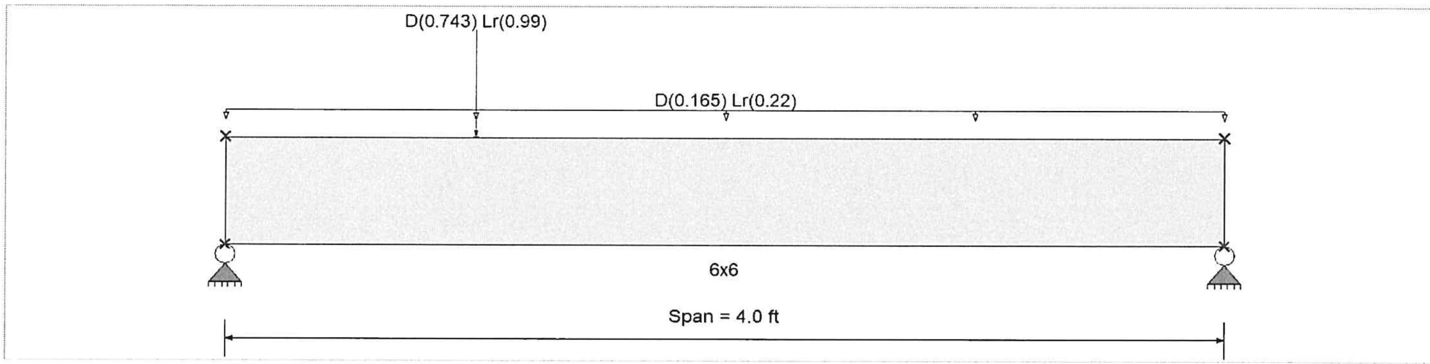
DESCRIPTION: H-1

CODE REFERENCES

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

Material Properties

Analysis Method :	Allowable Stress Design	Fb +	1,350.0 psi	*E : Modulus of Elasticity*	
Load Combination :	IBC 2021	Fb -	1,350.0 psi	Ebend- xx	1,600.0 ksi
		Fc - Prll	925.0 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	675.0 psi	Density	31.210 pcf
Beam Bracing :	Completely Unbraced			Repetitive Member Stress Increase	



Applied Loads

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

Beam self weight NOT internally calculated and added
Uniform Load : D = 0.1650, Lr = 0.220 , Tributary Width = 1.0 ft
Point Load : D = 0.7430, Lr = 0.990 k @ 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.419** : 1	Maximum Shear Stress Ratio	=	**0.442** : 1
Section used for this span		**6x6**	Section used for this span		**6x6**
fb: Actual	=	812.24 psi	fv: Actual	=	93.99 psi
F'b	=	1,940.63 psi	F'v	=	212.50 psi
Load Combination		+D+Lr	Load Combination		+D+Lr
Location of maximum on span	=	1.007 ft	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	0.024 in	Ratio =	2042 >=360	Span: 1 : Lr Only
Max Upward Transient Deflection	0 in	Ratio =	0 <360	n/a
Max Downward Total Deflection	0.041 in	Ratio =	1166 >=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 = 4.0 ft	1	0.249	0.263	0.90	1.00	1.00	1.00	1.000	1.00	1.00	1.15	0.80	348.2	1,397.3	0.81	40.3	153.0
+D+Lr					1.00	1.00	1.00	1.000	1.00	1.00	1.15			0.0	0.00	0.0	0.0
Length = 4.0 ft	1	0.419	0.442	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.15	1.88	812.2	1,940.6	1.90	94.0	212.5
+D+0.750Lr					1.00	1.00	1.00	1.000	1.00	1.00	1.15			0.0	0.00	0.0	0.0
Length = 4.0 ft	1	0.359	0.379	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.15	1.61	696.2	1,940.6	1.62	80.6	212.5
+0.60D					1.00	1.00	1.00	1.000	1.00	1.00	1.15			0.0	0.00	0.0	0.0
Length = 4.0 ft	1	0.084	0.089	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.15	0.48	208.9	2,484.0	0.49	24.2	272.0

Wood Beam

Project File: 510 Santa Paula-Bird.ec6

LIC# : KW-06014816, Build:20.23.05.01 LOBANA ENGINEERING

DESCRIPTION: H-1

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+Lr	1	0.0411	1.883		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	2.070	1.203
Max Upward from Load Combinations	2.070	1.203
Max Upward from Load Cases	1.183	0.688
D Only	0.887	0.516
+D+Lr	2.070	1.203
+D+0.750Lr	1.774	1.031
+0.60D	0.532	0.309
Lr Only	1.183	0.688

6

Project Title:
Engineer:
Project ID:
Project Descr:

Wood Beam — Project File: 510 Santa Paula-Bird.ec6

LIC# : KW-06014816, Build:20.23.05.01 — LOBANA ENGINEERING — (c) ENERCALC INC 1983-2023

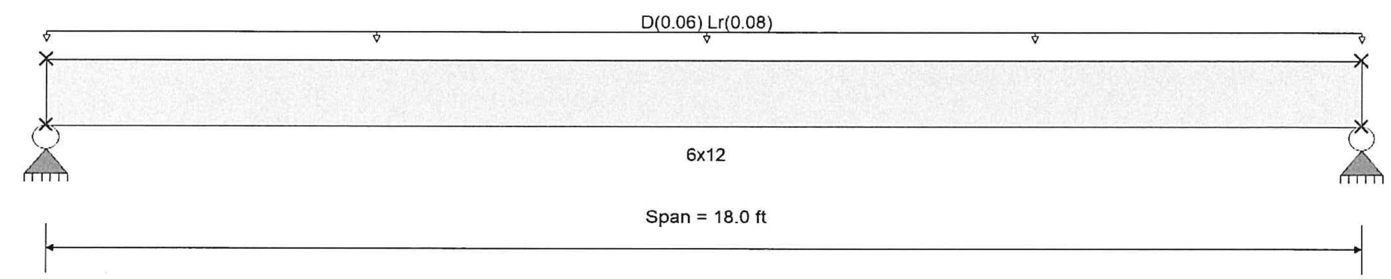
DESCRIPTION: B-1

CODE REFERENCES

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

Material Properties

Analysis Method :	Allowable Stress Design	Fb +	1,350.0 psi	*E : Modulus of Elasticity*	
Load Combination :	IBC 2021	Fb -	1,350.0 psi	Ebend- xx	1,600.0ksi
		Fc - Prll	925.0 psi	Eminbend - xx	580.0ksi
Wood Species :	Douglas Fir-Larch	Fc - Perp	625.0 psi		
Wood Grade :	No.1	Fv	170.0 psi		
		Ft	675.0 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 NOT internally calculated and added
Uniform Load : D = 0.060, Lr = 0.080 , Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.299: 1**	Maximum Shear Stress Ratio	=	**0.126 : 1**
Section used for this span		**6x12**	Section used for this span		**6x12**
fb: Actual	=	561.25psi	fv: Actual	=	26.83 psi
F'b	=	1,876.16psi	F'v	=	212.50 psi
Load Combination		+D+Lr	Load Combination		+D+Lr
Location of maximum on span	=	9.000ft	Location of maximum on span	=	17.080 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.170 in	Ratio =	1267 >=360	Span: 1 : Lr Only
Max Upward Transient Deflection	0 in	Ratio =	0 <360	n/a
Max Downward Total Deflection	0.298 in	Ratio =	724 >=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 = 18.0 ft	1	0.176	0.075	0.90	1.00	1.00	0.98	1.000	1.00	1.00	1.15	2.43	240.5	1,368.3	0.48	11.5	153.0
+D+Lr					1.00	1.00	0.98	1.000	1.00	1.00	1.15			0.0	0.00	0.0	0.0
Length = 18.0 ft	1	0.299	0.126	1.25	1.00	1.00	0.97	1.000	1.00	1.00	1.15	5.67	561.3	1,876.2	1.13	26.8	212.5
+D+0.750Lr					1.00	1.00	0.97	1.000	1.00	1.00	1.15			0.0	0.00	0.0	0.0
Length = 18.0 ft	1	0.256	0.108	1.25	1.00	1.00	0.97	1.000	1.00	1.00	1.15	4.86	481.1	1,876.2	0.97	23.0	212.5
+0.60D					1.00	1.00	0.97	1.000	1.00	1.00	1.15			0.0	0.00	0.0	0.0
Length = 18.0 ft	1	0.061	0.025	1.60	1.00	1.00	0.95	1.000	1.00	1.00	1.15	1.46	144.3	2,360.0	0.29	6.9	272.0

Wood Beam

Project File: 510 Santa Paula-Bird.ec6

DESCRIPTION: B-1

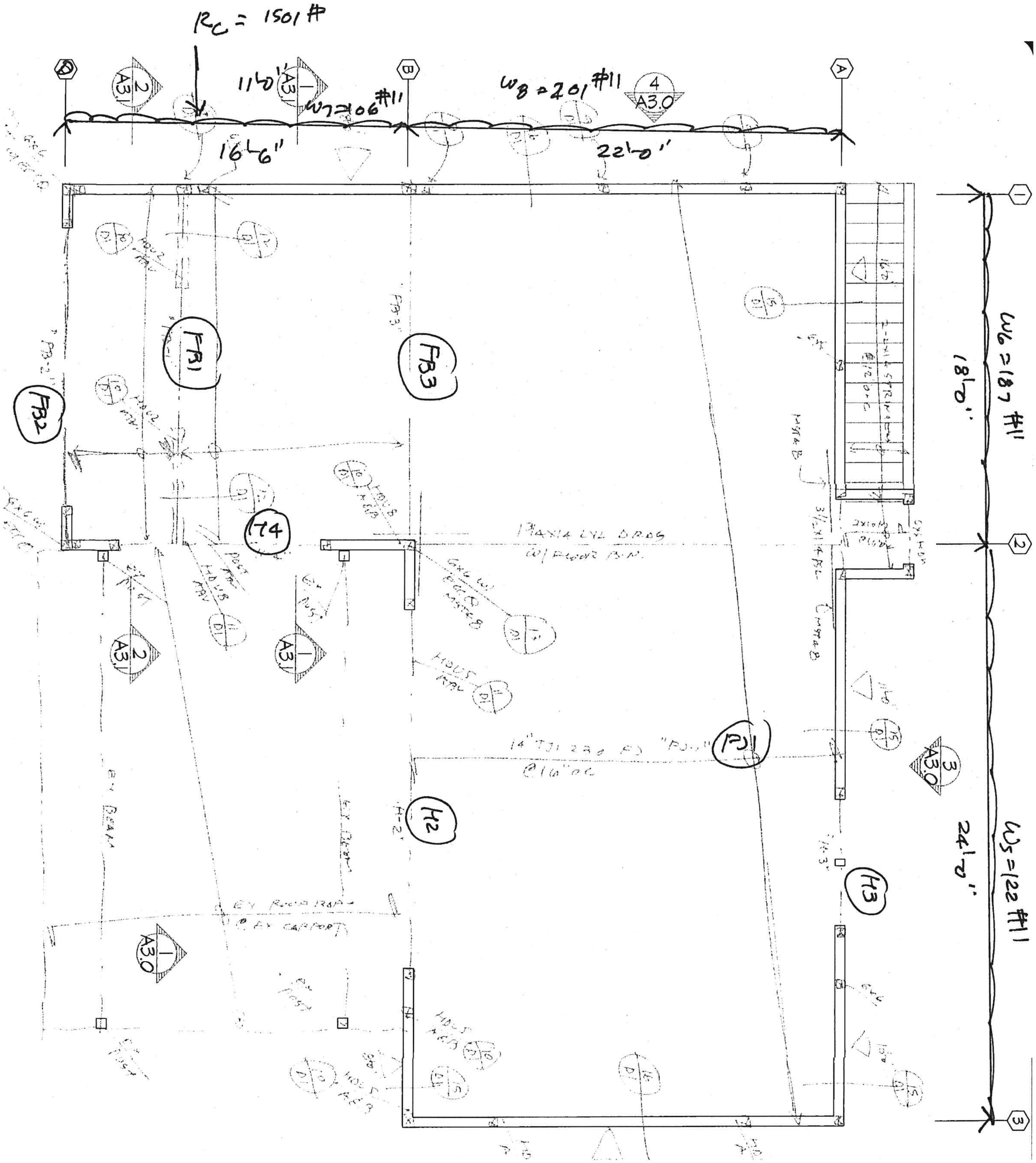
Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+Lr	1	0.2982	9.066		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	1.260	1.260
Max Upward from Load Combinations	1.260	1.260
Max Upward from Load Cases	0.720	0.720
D Only	0.540	0.540
+D+Lr	1.260	1.260
+D+0.750Lr	1.080	1.080
+0.60D	0.324	0.324
Lr Only	0.720	0.720


Rc = 1501 #
W8 = 201 #/l
W7 = 106 #/l
11'-0"
16'-6"
22'-0"
W6 = 187 #/l
18'-0"
W5 = 122 #/l
24'-0"
FB1
FB2
FB3
H4
FJ1
H2
H3

FLOOR FRAMING

FJ-1 $L = 22'0''$

$w = (18+40) \times 16/12 = (24+53.2) = 77.20^{\#} < 88^{\#/1}$

Use 14" TJI 360 @ 16" o.c

FB-1 Seismic

$(1.2+.25DS)DL = 1.51DL$

$L = 18'0''$

$w = (24+60) \times 6/2 + 15\times9 = (207+180)$

$1.5W_{DL} + W_{LL} = (313+180)$

$P_{seismic}$ @ grid (C) = $2.5 \times 1022 = 2555^{\#}$ @ $5'0''$

See comp output Use 7X14 PSL BM

FB-2

$L = 14'0''$

$w = (24+60) \times 6/2 = (72+180)$ Use 5¼X14 PSL

FB-3

$L = 18'-0''$

$w = (18+40) \times 22/2 = (198+440)$ Use 5¼X14 PSL

H-2 Seismic

$L = 18'0''$

$w = (15+20) \times 22/2 + (18+40) \times 22/2 + 15\times9 = (496+660)$

$1.51 W_{DL} + W_{LL} = (750+660)$

$P_{seismic}$ @ grid (B) = $2.5 \times 3156 = 7890^{\#}$ @ $3'0''$

See comp. output Use 7x16 PSL HDR

H-3

$L = 7'0''$

$w = (496+660)$ Use

H-4 Seismic

$L = 10'0''$

$w = (15+20) \times 18/2 + (18+40) \times 18/2 + 15\times9 = (432+540)$

$P = (15+20) \times 7/2 \times 18/2 = (473+630)$ @ $6'0''$

$1.51 W_{DL} + W_{LL} = (652+540)$

$1.51 P_{DL} + P_{LL} = (714+630)$ @ $6'0''$

$P_2 = (2807 + 1620 + \overset{FB-1}{1845})$ @ $7'0''$

$P_{seismic}$ @ grid (2) = $2.5 \times 4091 = 10227^{\#}$ @ $7'0''$

See Comp. output Use 5¼x11⅞ PSL

Project Title:
Engineer:
Project ID:
Project Descr:

Wood Beam

Project File: 510 Santa Paula-Bird.ec6

LIC# : KW-06014816, Build:20.23.05.01 LOBANA ENGINEERING (c) ENERCALC INC 1983-2023

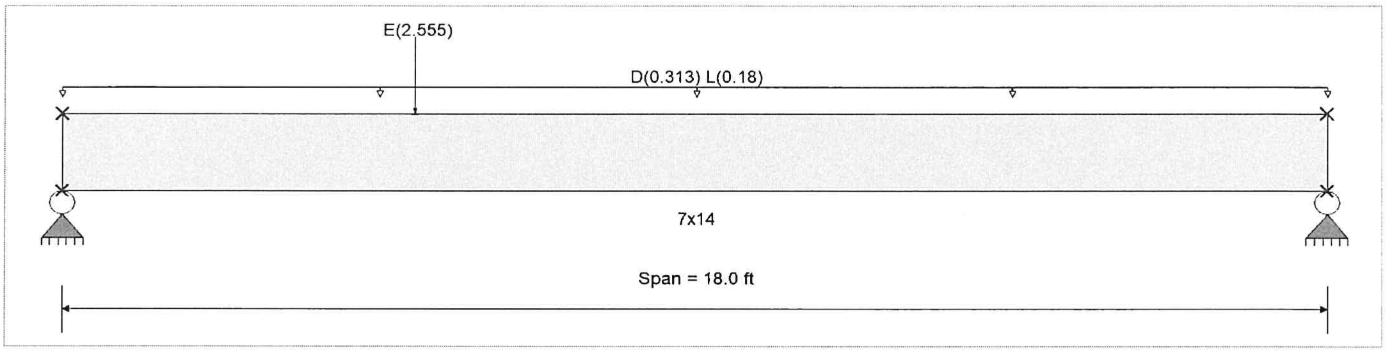
DESCRIPTION: FB-1

CODE REFERENCES

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

Material Properties

Analysis Method :	Allowable Stress Design	Fb +	2900 psi	*E : Modulus of Elasticity*	
Load Combination :	IBC 2021	Fb -	2900 psi	Ebend- xx	2000 ksi
		Fc - Prll	2900 psi	Eminbend - xx	1016.535 ksi
Wood Species :	iLevel Truss Joist	Fc - Perp	750 psi		
Wood Grade :	Parallam PSL 2.0E	Fv	290 psi		
		Ft	2025 psi	Density	45.07 pcf
Beam Bracing :	Completely Unbraced			Repetitive Member Stress Increase	



Applied Loads

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

Beam self weight NOT internally calculated and added
Uniform Load : D = 0.3130, L = 0.180 , Tributary Width = 1.0 ft
Point Load : E = 2.555 k @ 5.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.360**: 1	Maximum Shear Stress Ratio	=	**0.205** : 1
Section used for this span		**7x14**	Section used for this span		**7x14**
fb: Actual	=	1,047.80 psi	fv: Actual	=	59.49 psi
F'b	=	2,910.74 psi	F'v	=	290.00 psi
Load Combination		+D+L	Load Combination		+D+L
Location of maximum on span	=	9.000 ft	Location of maximum on span	=	16.883 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.134 in	Ratio =	1617 >=360	Span: 1 : L Only
Max Upward Transient Deflection	0 in	Ratio =	0 <360	n/a
Max Downward Total Deflection	0.399 in	Ratio =	541 >=180	Span: 1 : +D+0.750L+0.5250E
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 = 18.0 ft	1	0.253	0.145	0.90	1.00	1.00	0.98	0.983	1.00	1.00	1.04	12.68	665.2	2,625.9	2.47	37.8	261.0
+D+L					1.00	1.00	0.98	0.983	1.00	1.00	1.04			0.0	0.00	0.0	0.0
Length = 18.0 ft	1	0.360	0.205	1.00	1.00	1.00	0.98	0.983	1.00	1.00	1.04	19.97	1,047.8	2,910.7	3.89	59.5	290.0
+D+0.750L					1.00	1.00	0.98	0.983	1.00	1.00	1.04			0.0	0.00	0.0	0.0
Length = 18.0 ft	1	0.263	0.149	1.25	1.00	1.00	0.98	0.983	1.00	1.00	1.04	18.14	952.2	3,614.4	3.53	54.1	362.5
+D+0.70E					1.00	1.00	0.98	0.983	1.00	1.00	1.04			0.0	0.00	0.0	0.0
Length = 18.0 ft	1	0.201	0.124	1.60	1.00	1.00	0.96	0.983	1.00	1.00	1.04	17.54	920.6	4,574.1	3.76	57.5	464.0
+D+0.750L+0.5250E					1.00	1.00	0.96	0.983	1.00	1.00	1.04			0.0	0.00	0.0	0.0

(11)

Project Title:
Engineer:
Project ID:
Project Descr:

Wood Beam

Project File: 510 Santa Paula-Bird.ec6

LIC# : KW-06014816, Build:20.23.05.01 LOBANA ENGINEERING

DESCRIPTION: FB-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 = 18.0 ft	1	0.248	0.148	1.60	1.00	1.00	0.96	0.983	1.00	1.00	1.04	21.65	1,136.3	4,574.1	4.50	68.9	464.0
+0.60D					1.00	1.00	0.96	0.983	1.00	1.00	1.04			0.0	0.00	0.0	0.0
Length = 18.0 ft	1	0.087	0.049	1.60	1.00	1.00	0.96	0.983	1.00	1.00	1.04	7.61	399.1	4,574.1	1.48	22.7	464.0
+0.60D+0.70E					1.00	1.00	0.96	0.983	1.00	1.00	1.04			0.0	0.00	0.0	0.0
Length = 18.0 ft	1	0.146	0.091	1.60	1.00	1.00	0.96	0.983	1.00	1.00	1.04	12.73	668.3	4,574.1	2.77	42.4	464.0

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+0.750L+0.5250E	1	0.3988	8.869		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	5.001	4.437
Max Upward from Load Combinations	5.001	4.437
Max Upward from Load Cases	2.817	2.817
D Only	2.817	2.817
+D+L	4.437	4.437
+D+0.750L	4.032	4.032
+D+0.70E	4.109	3.314
+D+0.750L+0.5250E	5.001	4.405
+0.60D	1.690	1.690
+0.60D+0.70E	2.982	2.187
L Only	1.620	1.620
E Only	1.845	0.710

Project Title:
Engineer:
Project ID:
Project Descr:

Wood Beam

Project File: 510 Santa Paula-Bird.ec6

LIC# : KW-06014816, Build:20.23.05.01 LOBANA ENGINEERING (c) ENERCALC INC 1983-2023

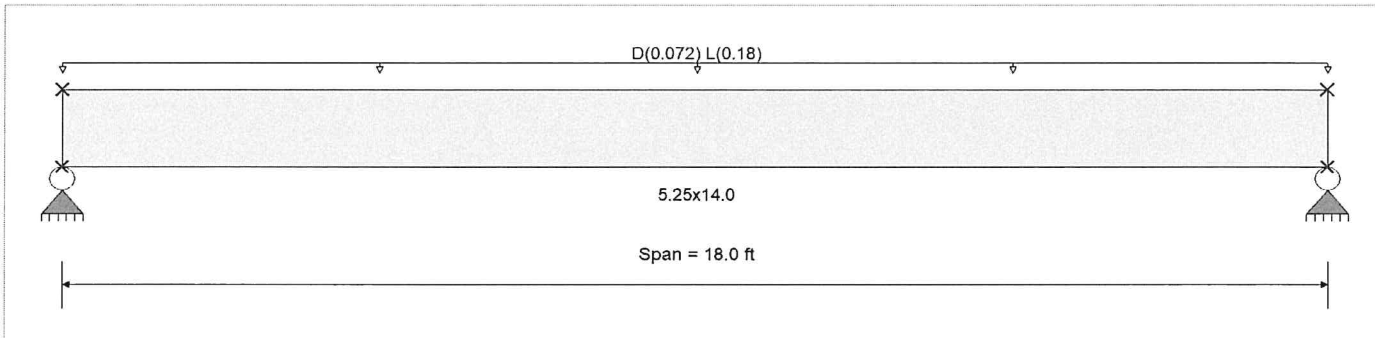
DESCRIPTION: FB-2

CODE REFERENCES

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

Material Properties

Analysis Method :	Allowable Stress Design	Fb +	2,900.0 psi	*E : Modulus of Elasticity*	
Load Combination :	IBC 2021	Fb -	2,900.0 psi	Ebend- xx	2,000.0 ksi
		Fc - Prll	2,900.0 psi	Eminbend - xx	1,016.54 ksi
Wood Species :	iLevel Truss Joist	Fc - Perp	750.0 psi		
Wood Grade :	Parallam PSL 2.0E	Fv	290.0 psi		
		Ft	2,025.0 psi	Density	45.070 pcf
Beam Bracing :	Completely Unbraced			Repetitive Member Stress Increase	



Applied Loads

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

Beam self weight NOT internally calculated and added
Uniform Load : D = 0.0720, L = 0.180 , Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.252** 1	Maximum Shear Stress Ratio	=	**0.140** : 1
Section used for this span		**5.25x14.0**	Section used for this span		**5.25x14.0**
fb: Actual	=	714.12 psi	fv: Actual	=	40.54 psi
F'b	=	2,839.03 psi	F'v	=	290.00 psi
Load Combination		+D+L	Load Combination		+D+L
Location of maximum on span	=	9.000 ft	Location of maximum on span	=	16.883 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.178 in	Ratio =	1212 >=360	Span: 1 : L Only
Max Upward Transient Deflection	0 in	Ratio =	0 <360	n/a
Max Downward Total Deflection	0.249 in	Ratio =	866 >=180	Span: 1 : +D+L
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 = 18.0 ft	1	0.079	0.044	0.90	1.00	1.00	0.96	0.983	1.00	1.00	1.04	2.92	204.0	2,573.0	0.57	11.6	261.0
+D+L					1.00	1.00	0.96	0.983	1.00	1.00	1.04			0.0	0.00	0.0	0.0
Length = 18.0 ft	1	0.252	0.140	1.00	1.00	1.00	0.96	0.983	1.00	1.00	1.04	10.21	714.1	2,839.0	1.99	40.5	290.0
+D+0.750L					1.00	1.00	0.96	0.983	1.00	1.00	1.04			0.0	0.00	0.0	0.0
Length = 18.0 ft	1	0.169	0.092	1.25	1.00	1.00	0.94	0.983	1.00	1.00	1.04	8.38	586.6	3,471.6	1.63	33.3	362.5
+0.60D					1.00	1.00	0.94	0.983	1.00	1.00	1.04			0.0	0.00	0.0	0.0
Length = 18.0 ft	1	0.029	0.015	1.60	1.00	1.00	0.89	0.983	1.00	1.00	1.04	1.75	122.4	4,243.4	0.34	7.0	464.0

Project Title:
Engineer:
Project ID:
Project Descr:

Wood Beam

Project File: 510 Santa Paula-Bird.ec6

LIC# : KW-06014816, Build:20.23.05.01 LOBANA ENGINEERING

DESCRIPTION: FB-2

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+L	1	0.2494	9.066		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	2.268	2.268
Max Upward from Load Combinations	2.268	2.268
Max Upward from Load Cases	1.620	1.620
D Only	0.648	0.648
+D+L	2.268	2.268
+D+0.750L	1.863	1.863
+0.60D	0.389	0.389
L Only	1.620	1.620

14

Project Title:
Engineer:
Project ID:
Project Descr:

Wood Beam

Project File: 510 Santa Paula-Bird.ec6

LIC# : KW-06014816, Build:20.23.05.01 LOBANA ENGINEERING (c) ENERCALC INC 1983-2023

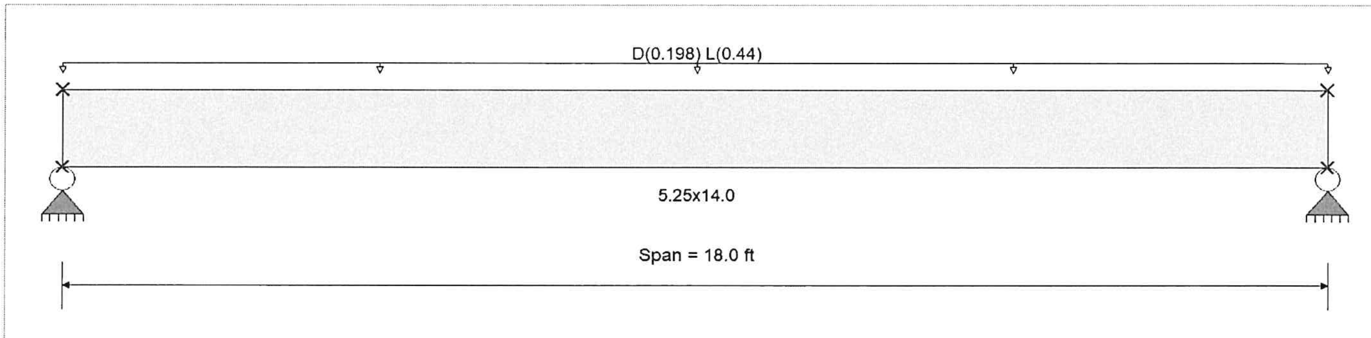
DESCRIPTION: FB-3

CODE REFERENCES

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

Material Properties

Analysis Method :	Allowable Stress Design	Fb +	2,900.0 psi	*E : Modulus of Elasticity*	
Load Combination :	IBC 2021	Fb -	2,900.0 psi	Ebend- xx	2,000.0 ksi
		Fc - Prll	2,900.0 psi	Eminbend - xx	1,016.54 ksi
Wood Species :	iLevel Truss Joist	Fc - Perp	750.0 psi		
Wood Grade :	Parallam PSL 2.0E	Fv	290.0 psi		
		Ft	2,025.0 psi	Density	45.070 pcf
Beam Bracing :	Completely Unbraced			Repetitive Member Stress Increase	



Applied Loads

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

Beam self weight NOT internally calculated and added
Uniform Load : D = 0.1980, L = 0.440 , Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.637**: 1	Maximum Shear Stress Ratio	=	**0.354** : 1
Section used for this span		**5.25x14.0**	Section used for this span		**5.25x14.0**
fb: Actual	=	1,807.98 psi	fv: Actual	=	102.64 psi
F'b	=	2,839.03 psi	F'v	=	290.00 psi
Load Combination		+D+L	Load Combination		+D+L
Location of maximum on span	=	9.000 ft	Location of maximum on span	=	16.883 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.435 in Ratio =	496 >=360	Span: 1 : L Only
Max Upward Transient Deflection	0 in Ratio =	0 <360	n/a
Max Downward Total Deflection	0.631 in Ratio =	342 >=180	Span: 1 : +D+L
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 = 18.0 ft	1	0.218	0.122	0.90	1.00	1.00	0.96	0.983	1.00	1.00	1.04	8.02	561.1	2,573.0	1.56	31.9	261.0
+D+L					1.00	1.00	0.96	0.983	1.00	1.00	1.04			0.0	0.00	0.0	0.0
Length = 18.0 ft	1	0.637	0.354	1.00	1.00	1.00	0.96	0.983	1.00	1.00	1.04	25.84	1,808.0	2,839.0	5.03	102.6	290.0
+D+0.750L					1.00	1.00	0.96	0.983	1.00	1.00	1.04			0.0	0.00	0.0	0.0
Length = 18.0 ft	1	0.431	0.234	1.25	1.00	1.00	0.94	0.983	1.00	1.00	1.04	21.38	1,496.3	3,471.6	4.16	84.9	362.5
+0.60D					1.00	1.00	0.94	0.983	1.00	1.00	1.04			0.0	0.00	0.0	0.0
Length = 18.0 ft	1	0.079	0.041	1.60	1.00	1.00	0.89	0.983	1.00	1.00	1.04	4.81	336.7	4,243.4	0.94	19.1	464.0

Wood Beam

DESCRIPTION: FB-3

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+L	1	0.6313	9.066		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	5.742	5.742
Max Upward from Load Combinations	5.742	5.742
Max Upward from Load Cases	3.960	3.960
D Only	1.782	1.782
+D+L	5.742	5.742
+D+0.750L	4.752	4.752
+0.60D	1.069	1.069
L Only	3.960	3.960

Project Title:
Engineer:
Project ID:
Project Descr:

Wood Beam

Project File: 510 Santa Paula-Bird.ec6

LIC# : KW-06014816, Build:20.23.05.01 LOBANA ENGINEERING (c) ENERCALC INC 1983-2023

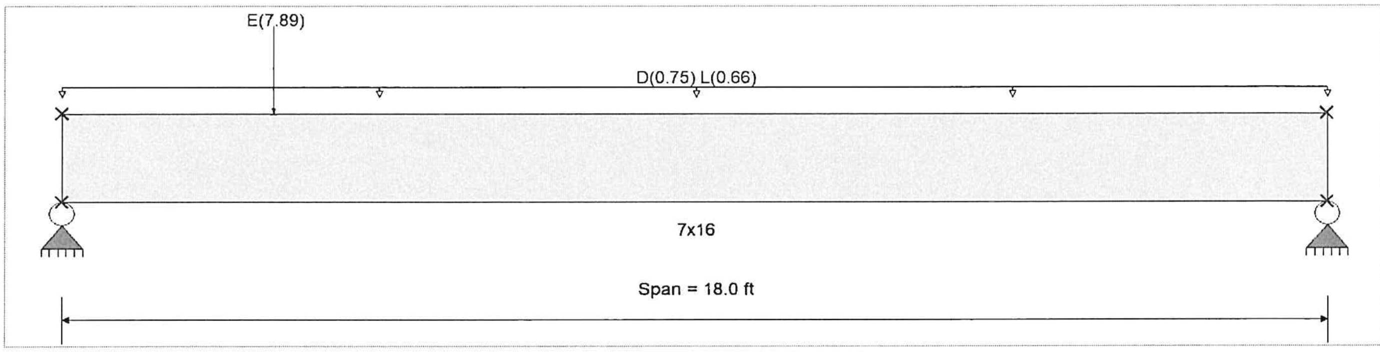
DESCRIPTION: H-2

CODE REFERENCES

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

Material Properties

Analysis Method :	Allowable Stress Design	Fb +	2,900.0 psi	*E : Modulus of Elasticity*	
Load Combination :	IBC 2021	Fb -	2,900.0 psi	Ebend- xx	2,000.0 ksi
		Fc - Prll	2,900.0 psi	Eminbend - xx	1,016.54 ksi
Wood Species :	iLevel Truss Joist	Fc - Perp	750.0 psi		
Wood Grade :	Parallam PSL 2.0E	Fv	290.0 psi		
		Ft	2,025.0 psi	Density	45.070 pcf
Beam Bracing :	Completely Unbraced			Repetitive Member Stress Increase	



Applied Loads

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

Beam self weight NOT internally calculated and added
Uniform Load : D = 0.750, L = 0.660 , Tributary Width = 1.0 ft
Point Load : E = 7.890 k @ 3.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.803** : 1	Maximum Shear Stress Ratio	=	**0.500** : 1
Section used for this span		**7x16**	Section used for this span		**7x16**
fb: Actual	=	2,294.40 psi	fv: Actual	=	145.14 psi
F'b	=	2,858.17 psi	F'v	=	290.00 psi
Load Combination		+D+L	Load Combination		+D+L
Location of maximum on span	=	9.000 ft	Location of maximum on span	=	16.686 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.328 in	Ratio =	658 >=360	Span: 1 : L Only
Max Upward Transient Deflection	0 in	Ratio =	0 <360	n/a
Max Downward Total Deflection	0.707 in	Ratio =	305 >=180	Span: 1 : +D+0.750L+0.5250E
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 = 18.0 ft	1	0.473	0.296	0.90	1.00	1.00	0.98	0.969	1.00	1.00	1.04	30.38	1,220.4	2,579.8	5.76	77.2	261.0
+D+L					1.00	1.00	0.98	0.969	1.00	1.00	1.04			0.0	0.00	0.0	0.0
Length = 18.0 ft	1	0.803	0.500	1.00	1.00	1.00	0.98	0.969	1.00	1.00	1.04	57.11	2,294.4	2,858.2	10.84	145.1	290.0
+D+0.750L					1.00	1.00	0.98	0.969	1.00	1.00	1.04			0.0	0.00	0.0	0.0
Length = 18.0 ft	1	0.572	0.354	1.25	1.00	1.00	0.97	0.969	1.00	1.00	1.04	50.42	2,025.9	3,543.2	9.57	128.2	362.5
+D+0.70E					1.00	1.00	0.97	0.969	1.00	1.00	1.04			0.0	0.00	0.0	0.0
Length = 18.0 ft	1	0.353	0.299	1.60	1.00	1.00	0.96	0.969	1.00	1.00	1.04	39.22	1,576.0	4,468.8	10.37	138.8	464.0
+D+0.750L+0.5250E					1.00	1.00	0.96	0.969	1.00	1.00	1.04			0.0	0.00	0.0	0.0

(17)

Project Title:
Engineer:
Project ID:
Project Descr:

Wood Beam

Project File: 510 Santa Paula-Bird.ec6

LIC# : KW-06014816, Build:20.23.05.01 LOBANA ENGINEERING (c) ENERCALC INC 1983-2023

DESCRIPTION: H-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
Length = 18.0 ft	1	0.511	0.376	1.60	1.00	1.00	0.96	0.969	1.00	1.00	1.04	56.83	2,283.2	4,468.8	13.02	174.4	464.0
+0.60D					1.00	1.00	0.96	0.969	1.00	1.00	1.04			0.0	0.00	0.0	0.0
Length = 18.0 ft	1	0.164	0.100	1.60	1.00	1.00	0.96	0.969	1.00	1.00	1.04	18.23	732.3	4,468.8	3.46	46.3	464.0
+0.60D+0.70E					1.00	1.00	0.96	0.969	1.00	1.00	1.04			0.0	0.00	0.0	0.0
Length = 18.0 ft	1	0.247	0.233	1.60	1.00	1.00	0.96	0.969	1.00	1.00	1.04	27.45	1,102.9	4,468.8	8.06	108.0	464.0

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+0.750L+0.5250E	1	0.7075	8.869		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	14.657	12.690
Max Upward from Load Combinations	14.657	12.690
Max Upward from Load Cases	6.750	6.750
D Only	6.750	6.750
+D+L	12.690	12.690
+D+0.750L	11.205	11.205
+D+0.70E	11.353	7.671
+D+0.750L+0.5250E	14.657	11.895
+0.60D	4.050	4.050
+0.60D+0.70E	8.653	4.971
L Only	5.940	5.940
E Only	6.575	1.315

18

Project Title:
Engineer:
Project ID:
Project Descr:

Wood Beam

Project File: 510 Santa Paula-Bird.ec6

LIC# : KW-06014816, Build:20.23.05.01 LOBANA ENGINEERING (c) ENERCALC INC 1983-2023

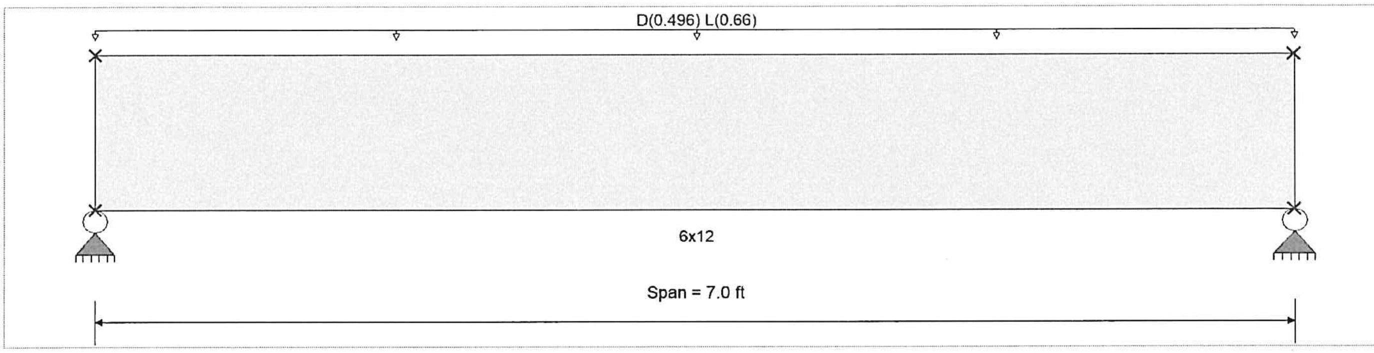
DESCRIPTION: H-3

CODE REFERENCES

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

Material Properties

Analysis Method :	Allowable Stress Design	Fb +	875 psi	*E : Modulus of Elasticity*	
Load Combination :	IBC 2021	Fb -	875 psi	Ebend- xx	1300 ksi
		Fc - Prll	600 psi	Eminbend - xx	470 ksi
Wood Species :	Douglas Fir-Larch	Fc - Perp	625 psi		
Wood Grade :	No.2	Fv	170 psi		
		Ft	425 psi	Density	31.21 pcf
Beam Bracing :	Completely Unbraced			Repetitive Member Stress Increase	



Applied Loads

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

Beam self weight NOT internally calculated and added
Uniform Load : D = 0.4960, L = 0.660 , Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.701**: 1	Maximum Shear Stress Ratio	=	**0.412** : 1
Section used for this span		**6x12**	Section used for this span		**6x12**
fb: Actual	=	700.87 psi	fv: Actual	=	70.04 psi
F'b	=	999.72 psi	F'v	=	170.00 psi
Load Combination		+D+L	Load Combination		+D+L
Location of maximum on span	=	3.500 ft	Location of maximum on span	=	6.055 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.040 in	Ratio =	2122 >=360	Span: 1 : L Only
Max Upward Transient Deflection	0 in	Ratio =	0 <360	n/a
Max Downward Total Deflection	0.069 in	Ratio =	1211 >=180	Span: 1 : +D+L
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 = 7.0 ft	1	0.334	0.196	0.90	1.00	1.00	0.99	1.000	1.00	1.00	1.15	3.04	300.7	900.4	1.27	30.1	153.0
+D+L					1.00	1.00	0.99	1.000	1.00	1.00	1.15			0.0	0.00	0.0	0.0
Length = 7.0 ft	1	0.701	0.412	1.00	1.00	1.00	0.99	1.000	1.00	1.00	1.15	7.08	700.9	999.7	2.95	70.0	170.0
+D+0.750L					1.00	1.00	0.99	1.000	1.00	1.00	1.15			0.0	0.00	0.0	0.0
Length = 7.0 ft	1	0.482	0.283	1.25	1.00	1.00	0.99	1.000	1.00	1.00	1.15	6.07	600.8	1,247.3	2.53	60.0	212.5
+0.60D					1.00	1.00	0.99	1.000	1.00	1.00	1.15			0.0	0.00	0.0	0.0
Length = 7.0 ft	1	0.113	0.066	1.60	1.00	1.00	0.99	1.000	1.00	1.00	1.15	1.82	180.4	1,592.1	0.76	18.0	272.0

Wood Beam

Project File: 510 Santa Paula-Bird.ec6

LIC# : KW-06014816, Build:20.23.05.01 LOBANA ENGINEERING

DESCRIPTION: H-3

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+L	1	0.0693	3.526		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	4.046	4.046
Max Upward from Load Combinations	4.046	4.046
Max Upward from Load Cases	2.310	2.310
D Only	1.736	1.736
+D+L	4.046	4.046
+D+0.750L	3.469	3.469
+0.60D	1.042	1.042
L Only	2.310	2.310

20

Project Title:
Engineer:
Project ID:
Project Descr:

Wood Beam

Project File: 510 Santa Paula-Bird.ec6

LIC# : KW-06014816, Build:20.23.05.01 LOBANA ENGINEERING (c) ENERCALC INC 1983-2023

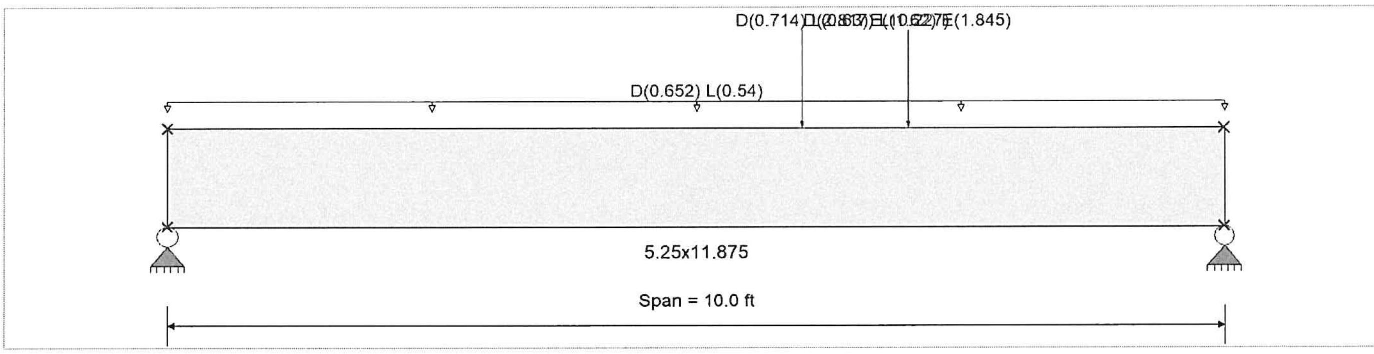
DESCRIPTION: H-4

CODE REFERENCES

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

Material Properties

Analysis Method :	Allowable Stress Design	Fb +	2,900.0 psi	*E : Modulus of Elasticity*	
Load Combination :	IBC 2021	Fb -	2,900.0 psi	Ebend- xx	2,000.0 ksi
		Fc - Prll	2,900.0 psi	Eminbend - xx	1,016.54 ksi
Wood Species :	iLevel Truss Joist	Fc - Perp	750.0 psi		
Wood Grade :	Parallam PSL 2.0E	Fv	290.0 psi		
		Ft	2,025.0 psi	Density	45.070 pcf
Beam Bracing :	Completely Unbraced			Repetitive Member Stress Increase	



Applied Loads

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

Beam self weight NOT internally calculated and added
Uniform Load : D = 0.6520, L = 0.540 , Tributary Width = 1.0 ft
Point Load : D = 0.7140, L = 0.630 k @ 6.0 ft
Point Load : D = 2.817, L = 1.620, E = 1.845 k @ 7.0 ft
Point Load : E = 10.227 k @ 7.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.835**: 1	Maximum Shear Stress Ratio	=	**0.722** : 1
Section used for this span		**5.25x11.875**	Section used for this span		**5.25x11.875**
fb: Actual	=	2,480.57 psi	fv: Actual	=	209.27 psi
F'b	=	2,971.89 psi	F'v	=	290.00 psi
Load Combination		+D+L	Load Combination		+D+L
Location of maximum on span	=	5.985 ft	Location of maximum on span	=	9.015 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.239 in Ratio =	502 >=360	Span: 1 : E Only
Max Upward Transient Deflection	0 in Ratio =	0 <360	n/a
Max Downward Total Deflection	0.395 in Ratio =	303 >=180	Span: 1 : +D+0.750L+0.5250E
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 = 10.0 ft	1	0.530	0.463	0.90	1.00	1.00	0.99	1.001	1.00	1.00	1.04	14.60	1,420.2	2,680.0	5.02	120.7	261.0
+D+L					1.00	1.00	0.99	1.001	1.00	1.00	1.04			0.0	0.00	0.0	0.0
Length = 10.0 ft	1	0.835	0.722	1.00	1.00	1.00	0.98	1.001	1.00	1.00	1.04	25.51	2,480.6	2,971.9	8.70	209.3	290.0
+D+0.750L					1.00	1.00	0.98	1.001	1.00	1.00	1.04			0.0	0.00	0.0	0.0
Length = 10.0 ft	1	0.600	0.516	1.25	1.00	1.00	0.98	1.001	1.00	1.00	1.04	22.78	2,215.4	3,694.6	7.78	187.1	362.5
+D+0.70E					1.00	1.00	0.98	1.001	1.00	1.00	1.04			0.0	0.00	0.0	0.0
Length = 10.0 ft	1	0.659	0.567	1.60	1.00	1.00	0.97	1.001	1.00	1.00	1.04	31.74	3,087.3	4,686.0	10.93	263.1	464.0

21

Wood Beam

Project File: 510 Santa Paula-Bird.ec6

DESCRIPTION: H-4

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+0.750L+0.5250E					1.00	1.00	0.97	1.001	1.00	1.00	1.04			0.0	0.00	0.0	0.0
Length = 10.0 ft	1	0.726	0.633	1.60	1.00	1.00	0.97	1.001	1.00	1.00	1.04	35.00	3,404.1	4,686.0	12.21	293.9	464.0
+0.60D					1.00	1.00	0.97	1.001	1.00	1.00	1.04			0.0	0.00	0.0	0.0
Length = 10.0 ft	1	0.182	0.156	1.60	1.00	1.00	0.97	1.001	1.00	1.00	1.04	8.76	852.1	4,686.0	3.01	72.4	464.0
+0.60D+0.70E					1.00	1.00	0.97	1.001	1.00	1.00	1.04			0.0	0.00	0.0	0.0
Length = 10.0 ft	1	0.542	0.463	1.60	1.00	1.00	0.97	1.001	1.00	1.00	1.04	26.12	2,539.8	4,686.0	8.93	214.8	464.0

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+0.750L+0.5250E	1	0.3952	5.292		0.0000	0.000

Vertical Reactions

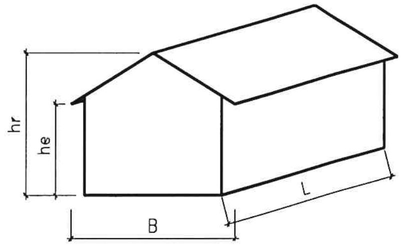
Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	8.871	13.256
Max Upward from Load Combinations	8.871	13.256
Max Upward from Load Cases	4.391	8.450
D Only	4.391	5.660
+D+L	7.829	9.872
+D+0.750L	6.969	8.819
+D+0.70E	6.926	11.576
+D+0.750L+0.5250E	8.871	13.256
+0.60D	2.634	3.396
+0.60D+0.70E	5.170	9.311
L Only	3.438	4.212
E Only	3.622	8.450

Wind Analysis for Low-rise Building, Based on ASCE 7-22

INPUT DATA

Exposure category (B, C or D, ASCE 7-22 26.7.3)		C	
Importance factor (ASCE 7-22 Table 1.5-2)	I_w =	1.00	for all Category
Basic wind speed (ASCE 7-22 26.5.1)	V =	110	*mph, (177.03 kph)*
Topographic factor (ASCE 7-22 26.8 & Figure 26.8-1)	K_{zt} =	1	Flat
Building height to eave	h_e =	19.5	*ft, (5.94 m)*
Building height to ridge	h_r =	28	*ft, (8.53 m)*
Building length	L =	47.5	*ft, (14.48 m)*
Building width, including overhangs	B =	42.5	*ft, (12.95 m)*
Overhang sloped width	O_h =	1.5	*ft, (0.46 m)*
Effective area of components (or Solar Panel area)	A =	12	ft^2, <== Overhang? (Yes or No) No
	(	1.12	m^2)



DESIGN SUMMARY

Max horizontal force normal to building length, L, face	=	22.21 *kips, (99 kN), SD level (LRFD level), Typ.*
Max horizontal force normal to building length, B, face	=	18.54 *kips, (82 kN)*
Max total horizontal torsional load	=	138.991 *ft-kips, (188 kN-m)*
Max total upward force	=	36.89 *kips, (164 kN)*

ANALYSIS

$\omega_{WIND} = 23.27\left(8/2 + \frac{8.5}{2}\right)$

$= 192\#/l$

Velocity pressure

$q_h K_d = (0.00256\, K_z K_{zt} K_e V^2) K_d$ = **28.81** x **0.85** = **24.49** psf

where: q_h = velocity pressure at mean roof height, h. (Eq. 26.10-1 page 277)

K_z = velocity pressure exposure coefficient evaluated at height, h, (Tab. 26.10-1, pg 277) = **0.93**

K_d = wind directionality factor. (Tab. 26.6-1, for building, page 274) = **0.85**

h = mean roof height = **23.75** ft

K_e = ground elevation factor. (*1.0* per Sec. 26.9, page 275)

< 60 ft, [Satisfactory] (ASCE 7-22 26.2.1)

< Min (L, B), [Satisfactory] (ASCE 7-22 26.2.2)

Design pressures for MWFRS

$p = q_h K_d [(G C_{pf})-(G C_{pi})]$ $= (.78 + .17) \times 24.49 = 23.27 psf$

where: p = pressure in appropriate zone. (Eq. 28.3-1, page 294). p_{min} = **16** psf (ASCE 7-22 28.3.6)

$G C_{pf}$ = product of gust effect factor and external pressure coefficient, see table below. (Fig. 28.3-1, page 295)

$G C_{pi}$ = product of gust effect factor and internal pressure coefficient.(Tab. 26.13-1, Enclosed Building, page 280)

= **0.18** or **-0.18**

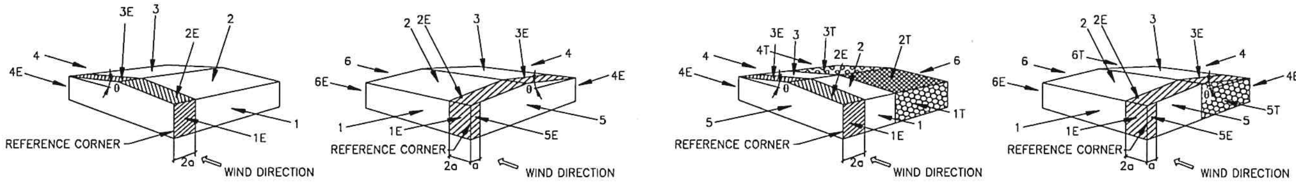
a = width of edge strips, Fig 28.3-1, page 295, MAX[MIN(0.1B, 0.1L, 0.4h), MIN(0.04B, 0.04L), 3] = **4.25** ft

Net Pressures (psf), Basic Load Cases

Surface	Roof angle θ = 21.80			Roof angle θ = 0.00		
	$G C_{pf}$	Net Pressure with (+GC_{pi})	Net Pressure with (-GC_{pi})	$G C_{pf}$	Net Pressure with (+GC_{pi})	Net Pressure with (-GC_{pi})
1	0.54	8.70	17.52	-0.45	-15.43	-6.61
2	-0.53	-17.33	-8.52	-0.69	-21.30	-12.49
3	-0.47	-15.94	-7.13	-0.37	-13.47	-4.65
4	-0.42	-14.67	-5.86	-0.45	-15.43	-6.61
5				0.40	5.39	14.20
6				-0.29	-11.51	-2.69
1E	0.78	14.70	23.51	-0.48	-16.16	-7.35
2E	-0.83	-24.70	-15.88	-1.07	-30.61	-21.79
3E	-0.66	-20.60	-11.78	-0.53	-17.39	-8.57
4E	-0.61	-19.37	-10.56	-0.48	-16.16	-7.35
5E				0.61	10.53	19.34
6E				-0.43	-14.94	-6.12

Net Pressures (psf), Torsional Load Cases

Surface	$G C_{pf}$	Net Pressure with (+GC_{pi})	Net Pressure with (-GC_{pi})
Roof angle θ = 21.80			
1T	0.13	-1.18	7.63
2T	-0.13	-7.60	1.22
3T	-0.12	-7.30	1.51
4T	-0.11	-7.01	1.80
Roof angle θ = 0.00			
5T	0.10	-1.96	6.86
6T	-0.07	-6.12	2.69



Load Case 1 (Transverse) Load Case 2 (Longitudinal) Load Case 3 (Transverse) Load Case 4 (Longitudinal)

Basic Load Cases Torsional Load Cases

Basic Load Case 1 (Transverse Direction)

Surface	Area (ft^2)	Pressure (k) with (+GC_{pi})	Pressure (k) with (-GC_{pi})
1	761	6.62	13.32
2	893	-15.47	-7.60
3	893	-14.23	-6.36
4	761	-11.16	-4.45
1E	166	2.44	3.90
2E	195	-4.80	-3.09
3E	195	-4.01	-2.29
4E	166	-3.21	-1.75
Σ	Horiz.	22.67	22.67
	Vert.	-35.76	-17.96
Min. wind 28.4.4	Horiz.	18.05	18.05
	Vert.	-32.30	-32.30

$p_{overhang}$ = **-17.14** psf
(ASCE 7-22 28.3.5)

Basic Load Case 2 (Longitudinal Direction)

Surface	Area (ft^2)	Pressure (k) with (+GC_{pi})	Pressure (k) with (-GC_{pi})
2	893	-19.02	-11.15
3	893	-12.02	-4.15
5	836	4.51	11.88
6	836	-9.63	-2.25
2E	195	-5.95	-4.24
3E	195	-3.38	-1.67
5E	173	1.82	3.35
6E	173	-2.58	-1.06
Σ	Horiz.	18.54	18.54
	Vert.	-29.52	-12.08
Min. wind 28.4.4	Horiz.	16.15	16.15
	Vert.	-32.30	-32.30

Torsional Load Case 3 (Transverse Direction)

Surface	Area (ft^2)	Pressure (k) with (+GC_{pi})	Pressure (k) with (-GC_{pi})	Torsion (ft-k) (+GC_{pi})	Torsion (ft-k) (-GC_{pi})
1	297	2.59	5.21	25	51
2	349	-6.05	-2.97	-22	-11
3	349	-5.56	-2.49	20	9
4	297	-4.36	-1.74	43	17
1E	166	2.44	3.90	48	76
2E	195	-4.80	-3.09	-35	-22
3E	195	-4.01	-2.29	29	17
4E	166	-3.21	-1.75	63	34
1T	463	-0.55	3.54	6	-42
2T	544	-4.13	0.66	18	-3
3T	544	-3.97	0.82	-18	4
4T	463	-3.25	0.83	-39	10
Total Horiz. Torsional Load, M_T				139	139

Torsional Load Case 4 (Longitudinal Direction)

Surface	Area (ft^2)	Pressure (k) with (+GC_{pi})	Pressure (k) with (-GC_{pi})	Torsion (ft-k) (+GC_{pi})	Torsion (ft-k) (-GC_{pi})
2	893	-19.02	-11.15	-15	-9
3	893	-12.02	-4.15	9	3
5	332	1.79	4.71	14	38
6	332	-3.82	-0.89	31	7
2E	195	-5.95	-4.24	48	34
3E	195	-3.38	-1.67	-27	-13
5E	173	1.82	3.35	35	64
6E	173	-2.58	-1.06	49	20
5T	505	-0.99	3.46	10	-35
6T	505	-3.09	1.36	-31	14
Total Horiz. Torsional Load, M_T				123.7	123.7

Design pressures for components and cladding

$p = q_h K_d [(G C_p) - (G C_{pi})]$

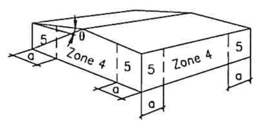
where: p = pressure on component. (Eq. 30.3-1, pg 316)

p_{min} = 16.00 psf (ASCE 7-22 30.2.2)

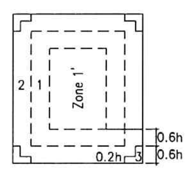
$G C_p$ = external pressure coefficient.

see table below. (ASCE 7-22 30.3.2)

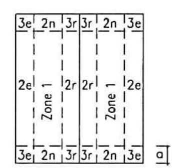
θ = 21.80 °



Walls



Roof θ≤7°



Roof θ>7°

Comp.	Effective Area (ft^4)	Zone 1 GC_P	Zone 1 - GC_P	Zone 1' GC_P	Zone 1' - GC_P	Zone 2 GC_P	Zone 2 - GC_P	Zone 2e GC_P	Zone 2e - GC_P	Zone 2n GC_P	Zone 2n - GC_P	Zone 2r GC_P	Zone 2r - GC_P
	12	0.52	-1.50	-	-	-	-	0.52	-1.50	0.52	-2.43	0.52	-2.43

Comp.	Effective Area (ft^4)	Zone 3 GC_P	Zone 3 - GC_P	Zone 3e GC_P	Zone 3e - GC_P	Zone 3r GC_P	Zone 3r - GC_P	Zone 4 GC_P	Zone 4 - GC_P	Zone 5 GC_P	Zone 5 - GC_P
	12	-	-	0.52	-2.43	0.52	-2.82	0.99	-1.09	0.99	-1.37

Comp. & Cladding Pressure (psf)	Zone 1 Positive	Zone 1 Negative	Zone 1' Positive	Zone 1' Negative	Zone 2 Positive	Zone 2 Negative	Zone 2e Positive	Zone 2e Negative	Zone 2n Positive	Zone 2n Negative	Zone 2r Positive	Zone 2r Negative
	17.06	-41.14					17.06	-41.14	17.06	-63.82	17.06	-63.82

Comp. & Cladding Pressure (psf)	Zone 3 Positive	Zone 3 Negative	Zone 3e Positive	Zone 3e Negative	Zone 3r Positive	Zone 3r Negative	Zone 4 Positive	Zone 4 Negative	Zone 5 Positive	Zone 5 Negative
			17.06	-63.82	17.06	-73.39	28.55	-31.00	28.55	-38.00

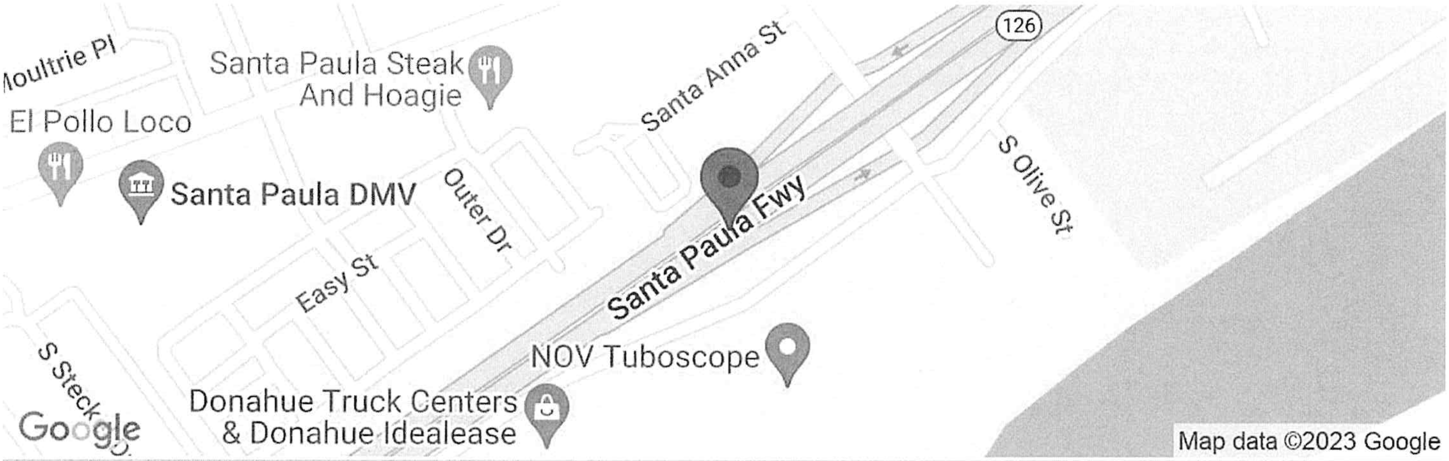
*(The Max Pressure **73.39** psf)*

24

OSHPD

Santa Paula Fwy, Santa Paula, CA, USA

Latitude, Longitude: 34.344569, -119.0699637



Date	5/19/2023, 4:21:57 PM
Design Code Reference Document	ASCE7-16
Risk Category	III
Site Class	D - Default (See Section 11.4.3)

Type	Value	Description
S_S	1.96	MCE_R ground motion. (for 0.2 second period)
S_1	0.747	MCE_R ground motion. (for 1.0s period)
S_{MS}	2.351	Site-modified spectral acceleration value
S_{M1}	null -See Section 11.4.8	Site-modified spectral acceleration value
S_{DS}	1.568	Numeric seismic design value at 0.2 second SA
S_{D1}	null -See Section 11.4.8	Numeric seismic design value at 1.0 second SA

Type	Value	Description
SDC	null -See Section 11.4.8	Seismic design category
F_a	1.2	Site amplification factor at 0.2 second
F_v	null -See Section 11.4.8	Site amplification factor at 1.0 second
PGA	0.868	MCE_G peak ground acceleration
F_{PGA}	1.2	Site amplification factor at PGA
PGA_M	1.041	Site modified peak ground acceleration
T_L	8	Long-period transition period in seconds
SsRT	1.96	Probabilistic risk-targeted ground motion. (0.2 second)
SsUH	2.221	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration
SsD	2.697	Factored deterministic acceleration value. (0.2 second)
S1RT	0.747	Probabilistic risk-targeted ground motion. (1.0 second)
S1UH	0.845	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration.
S1D	0.854	Factored deterministic acceleration value. (1.0 second)
PGAd	1.087	Factored deterministic acceleration value. (Peak Ground Acceleration)
PGA_{UH}	0.868	Uniform-hazard (2% probability of exceedance in 50 years) Peak Ground Acceleration
C_{RS}	0.882	Mapped value of the risk coefficient at short periods
C_{R1}	0.884	Mapped value of the risk coefficient at a period of 1 s

Lobana Engineering Inc.
885 Patriot Dr. Unit G
Moorpark, CA 93021

PROJECT :
CLIENT :
JOB NO. :
DATE :

DESIGN BY :
REVIEW BY :

Seismic Analysis Based on ASCE 7-22 - Equivalent Lateral Force (ELF) Procedure

INPUT DATA

Typical floor height	h =	8.5	ft, (2.6 m)
Typical floor weight	w_x =	45	kips, (200.2 kN)
Number of floors	n =	2	
Importance factor (ASCE 11.5.1)	I_e =	1	
Design spectral response	S_{DS} =	1.568	g
	S_{D1} =	0.747	g
Mapped spectral response	S_1 =	0.747	g
The coefficient (ASCE Tab 12.8-2)	C_t =	0.02	
The coefficient(ASCE Tab. 12.2.1)	R =	6.5	

DESIGN SUMMARY

Total base shear (1 kips = 4.448 kN)

V = 0.24 W, (SD) = 22 k, (SD)

= 0.17 W, (ASD) = 16 k, (ASD)

Seismic design category = D

h_n = 18.5 ft, (5.6 m)

W = 93 kips, (413.7 kN)

k = 1.00 , (ASCE 12.8.3, page 125)

$\Sigma w_x h^k$ = 1,313

x = 0.75 , (ASCE Tab 12.8-2)

$T_a = C_t (h_n)^x$ = 0.18 Sec, (ASCE 12.8.2.1)

VERTICAL DISTRIBUTION OF LATERAL FORCES

Level No.	Level Name	Floor to floor Height ft	Height h_x ft	Weight w_x k	$w_x h_x^k$	C_{vx}	Lateral force @ each level F_x k	V_x k	O. M. k-ft	Diaphragm force ΣF_i k	ΣW_i k	F_{px} k
2	Roof		18.5	45	833	0.634	14.2			14.2	45	14
		8.50						14.2				
1	2nd		10.0	48	480	0.366	8.2		121	22.4	93	15
		10.00						22.4				
	Ground		0.0						345			

$w_1 = \left(\frac{14.2}{45}\right)\left(24\times15 + 2\times15\times8.5/2\right)\times1.3 = 200$ #/'

$w_2 = \left(\frac{14.2}{45}\right)\left(41\times15 + 2\times15\times8.5/2\right)\times1.3 = 305$ #/'

$w_3 = \left(\frac{14.2}{45}\right)\left(49\times15 + 2\times15\times8.5/2\right)\times1.3 = 354$ #/'

$w_4 = \left(\frac{14.2}{45}\right)\left(21\times15 + 2\times15\times8.5/2\right)\times1.3 = 182$ #/'

$R_1 = R_2 = 305\times18/2 = 2745$ #

$R_2 = R_3 = 200\times29/2 = 2900$ #

$R_A = R_B = 354\times22/2 = 3894$ #

$R_B = R_C = 182\times11/2 = 1001$ #

$R_C = R_D = 182\times5.5/2 = 500$ #

ROOF Diaph

$R_1 = 2745/39 = 70$ #/'

$R_2 = \underbrace{(2745+2900)}_{=5645\#}/39 = 145$ #/'

$R_3 = 2900/22 = 132$ #/'

$R_A = 3894/47.5 = 82$ #/'

$R_B = \underbrace{(3894+1001)}_{=4895\#}/47.5 = 103$ #/'

$R_C = \underbrace{(1001+500)}_{=1501\#}/18 = 83$ #/'

$R_D = 500/18 = 28$ #/'

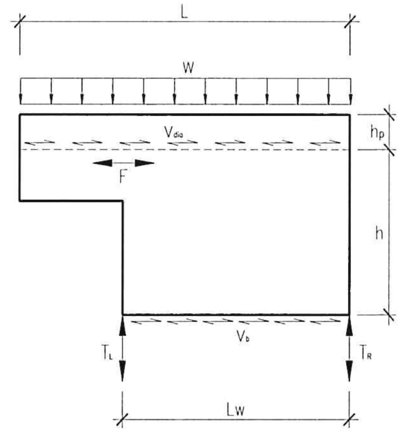
$\boxed{1}$ Use 1/2" CDX w/ 8d @ 6" o.c.

Lobana Engineering Inc.
885 Patriot Dr. Unit G
Moorpark, CA 93021

PROJECT : GRID 1
CLIENT :
JOB NO. :
DATE :

Shear Wall Design Based on 2021 IBC / 2022 CBC / SDPWS-21

INPUT DATA



LATERAL FORCE ON DIAPHRAGM: $v_{dia,\,WIND}$ = 0 plf,for wind, ASD

$v_{dia,\,SEISMIC}$ = 70 plf,for seismic, ASD

GRAVITY LOADS ON THE ROOF: w_{DL} = (7+12) plf,for dead load

w_{LL} = 0 plf,for live load

DIMENSIONS: L_w = 19 ft , h = 8.5 ft

L = 39 ft , h_p = 0 ft

PANEL GRADE (0 or 1) = 1 <= Sheathing and Single-Floor

MINIMUM NOMINAL PANEL THICKNESS = 15/32 in

COMMON NAIL SIZE (0=6d, 1=8d, 2=10d) 2 10d

SPECIFIC GRAVITY OF FRAMING MEMBERS 0.5

EDGE STUD SECTION 2 pcs, b = 2 in , h = 4 in

SPECIES (1 = DFL, 2 = SP) 1 DOUGLAS FIR-LARCH

GRADE (1, 2, 3, 4, 5, or 6) 4 No. 2

STORY OPTION (1=ground level, 2=upper level) 2 upper level shear wall

THE SHEAR WALL DESIGN IS ADEQUATE.

DESIGN SUMMARY

BLOCKED 15/32 SHEATHING WITH 10d COMMON NAILS
@ 6 in O.C. BOUNDARY & ALL EDGES / 12 in O.C. FIELD,
SILL PLATE ATTACHMENT 16d AT 6" O.C.

HOLD-DOWN FORCES: T_L = 0.70 k , T_R = 0.70 k (USE CS16 SIMPSON HOLD-DOWN)

DRAG STRUT FORCES: F = 3.50 k

EDGE STUD: 2 - 2" x 4" DOUGLAS FIR-LARCH No. 2, CONTINUOUS FULL HEIGHT.

SHEAR WALL DEFLECTION: Δ = 0.11 in

ANALYSIS

CHECK MAX SHEAR WALL DIMENSION RATIO L / B = 0.4 < 3.5 **[Satisfactory]**

DETERMINE REQUIRED CAPACITY v_b = 144 plf, (A Side Diaphragm Required, the Max. Nail Spacing = 6 in)

THE SHEAR CAPACITIES PER IBC Table 2306.3 / SDPWS-21 Table 4.3A with ASD reduction factor 2.0)

Use MST48

Panel Grade	Common Nail	Min. Penetration (in)	Min. Thickness (in)	Blocked Nail Spacing Boundary & All Edges 6	4	3	2
Sheathing and Single-Floor	10d	1 5/8	15/32	310	460	600	770

Note: 1. The indicated shear numbers have reduced by specific gravity factor per IBC note a.
2. Since the wall is blocked, SDPW-15 Table 4.3.3.2 does not apply.

DETERMINE DRAG STRUT FORCE: $F = (L-L_w)\ \mathrm{MAX}(v_{dia,\,WIND},\ \Omega_0 v_{dia,\,SEISMIC})$ = 3.50 k (Ω_0 = 2.5) (Sec. 1633.2.6)

DETERMINE FLOOR SILL PLATE ATTACHMENT (NDS 2015, Table 11Q & Table 11L)
SILL PLATE ATTACHMENT 16d AT 6" O.C.
THE HOLD-DOWN FORCES:

	v_{dia} (plf)	Wall Seismic at mid-story (lbs)	Overturning Moments (ft-lbs)		Resisting Moments (ft-lbs)	Safety Factors	Net Uplift (lbs)		Holddown SIMPSON
SEISMIC	70	258	24303	Left	12274	0.9	T_L =	698	CS16
				Right	12274	0.9	T_R =	698	
WIND	0		0	Left	12274	2/3	T_L =	0	
				Right	12274	2/3	T_R =	0	

(T_L & T_R values should include upper level UPLIFT forces if applicable)

CHECK MAXIMUM SHEAR WALL DEFLECTION: (IBC Section 2305.3 / SDPWS-21 4.3.4)

$$\Delta = \Delta_{Bending} + \Delta_{Shear} + \Delta_{Nail\ slip} + \Delta_{Chord\ splice\ slip} = \frac{8 v_b h^3}{E A L_w} + \frac{v_b h}{G t} + 0.75 h e_n + \frac{h d_a}{L_w}$$

= 0.112 in, ASD < $\delta_{xe,allowable,\,ASD}$ = 0.364 in **[Satisfactory]** (ASCE 7-22 12.8.6)

Where: v_b = 144 plf , ASD L_w = 19 ft E = 1.7E+06 psi

A = 16.50 in^2 h = 9 ft G = 9.0E+04 psi C_d = 4 I = 1

t = 0.298 in e_n = 0.004 in, SD d_a = 0.15 in, SD ,(ASCE 7-22 Tab 12.2-1 & Tab 11.5-1)

$$\frac{v_b h}{1000 G_a} = \frac{v_b h}{G t} + 0.75 h e_n$$

C_M = 1.0 (NDS 4.1.4) Δ_a = 0.02 h_{sx} , (ASCE 7-22 Tab 12.12-1)

CHECK EDGE STUD CAPACITY

P_{max} = 1.28 kips, (this value should include upper level DOWNWARD loads if applicable)

F_c = 1350 psi C_D = 1.60 C_P = 0.21 A = 10.5 in^2

E = 1600 ksi C_F = 1.15 F_c' = 532 psi > f_c = 122 psi

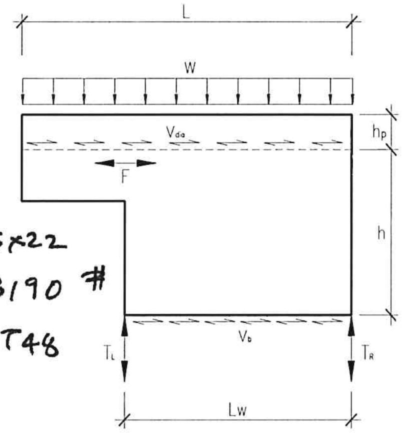
[Satisfactory]

Lobana Engineering Inc.
885 Patriot Dr. Unit G
Moorpark, CA 93021

PROJECT : GRID 2
CLIENT :
JOB NO. :
DATE :

Shear Wall Design Based on 2021 IBC / 2022 CBC / SDPWS-21

INPUT DATA



LATERAL FORCE ON DIAPHRAGM: $v_{dia,\ WIND}$ = 0 plf,for wind, ASD

$v_{dia,\ SEISMIC}$ = 145 plf,for seismic, ASD

GRAVITY LOADS ON THE ROOF: w_{DL} = plf,for dead load

w_{LL} = 0 plf,for live load

DIMENSIONS: L_w = 11 ft , h = 8.5 ft

L = 39 ft , h_p = 0 ft

PANEL GRADE (0 or 1) = 1 <= Sheathing and Single-Floor

MINIMUM NOMINAL PANEL THICKNESS = 15/32 in

COMMON NAIL SIZE (0=6d, 1=8d, 2=10d) 2 10d

SPECIFIC GRAVITY OF FRAMING MEMBERS 0.5

EDGE STUD SECTION 2 pcs, b = 2 in , h = 4 in

SPECIES (1 = DFL, 2 = SP) 1 DOUGLAS FIR-LARCH

GRADE (1, 2, 3, 4, 5, or 6) 4 No. 2

STORY OPTION (1=ground level, 2=upper level) 2 upper level shear wall

Drag = 145x22
= 3190 #
MST48

THE SHEAR WALL DESIGN IS ADEQUATE.

DESIGN SUMMARY

BLOCKED 15/32 SHEATHING WITH 10d COMMON NAILS
@ 3 in O.C. BOUNDARY & ALL EDGES / 12 in O.C. FIELD,
SILL PLATE ATTACHMENT #14 WOOD SCREWS x 5" LONG AT 6" O.C.

HOLD-DOWN FORCES: T_L = 4.09 k , T_R = 4.09 k (USE CMSTC16 SIMPSON HOLD-DOWN)

DRAG STRUT FORCES: F = 10.15 k

EDGE STUD: 2 - 2" x 4" DOUGLAS FIR-LARCH No. 2, CONTINUOUS FULL HEIGHT.

SHEAR WALL DEFLECTION: Δ = 0.27 in

ANALYSIS

CHECK MAX SHEAR WALL DIMENSION RATIO L / B = 0.8 < 3.5 **[Satisfactory]**

DETERMINE REQUIRED CAPACITY v_b = 514 plf, (C Side Diaphragm Required, the Max. Nail Spacing = 3 in)

THE SHEAR CAPACITIES PER IBC Table 2306.3 / SDPWS-21 Table 4.3A with ASD reduction factor 2.0)

Panel Grade	Common Nail	Min. Penetration (in)	Min. Thickness (in)	Blocked Nail Spacing Boundary & All Edges			
				6	4	3	2
Sheathing and Single-Floor	10d	1 5/8	15/32	310	460	600	770

Use HDU8
w/6x6

Note: 1. The indicated shear numbers have reduced by specific gravity factor per IBC note a.
2. Since the wall is blocked, SDPW-15 Table 4.3.3.2 does not apply.

DETERMINE DRAG STRUT FORCE: $F = (L-L_w)\ \mathrm{MAX}(v_{dia,\ WIND},\ \Omega_0 v_{dia,\ SEISMIC})$ = 10.15 k (Ω_0 = 2.5) (Sec. 1633.2.6)

DETERMINE FLOOR SILL PLATE ATTACHMENT (NDS 2015, Table 11Q & Table 11L)
SILL PLATE ATTACHMENT #14 WOOD SCREWS x 5" LONG AT 6" O.C.
THE HOLD-DOWN FORCES:

	v_{dia} (plf)	Wall Seismic at mid-story (lbs)	Overturning Moments (ft-lbs)		Resisting Moments (ft-lbs)	Safety Factors	Net Uplift (lbs)		Holddown SIMPSON
SEISMIC	145	150	48703	Left	4114	0.9	T_L =	4091	CMSTC16
				Right	4114	0.9	T_R =	4091	
WIND	0		0	Left	4114	2/3	T_L =	0	
				Right	4114	2/3	T_R =	0	

(T_L & T_R values should include upper level UPLIFT forces if applicable)

CHECK MAXIMUM SHEAR WALL DEFLECTION: (IBC Section 2305.3 / SDPWS-21 4.3.4)

$$\Delta = \Delta_{Bending} + \Delta_{Shear} + \Delta_{Nail\ slip} + \Delta_{Chord\ splice\ slip} = \frac{8 v_b h^3}{E A L_w} + \frac{v_b h}{G t} + 0.75 h e_n + \frac{h d_a}{L_w}$$

= 0.272 in, ASD < $\delta_{xe,allowable,\ ASD}$ = 0.364 in

Where: v_b = 514 plf , ASD L_w = 11 ft E = 1.7E+06 psi **[Satisfactory]** (ASCE 7-22 12.8.6)

A = 16.50 in^2 h = 9 ft G = 9.0E+04 psi C_d = 4 I = 1

t = 0.298 in e_n = 0.004 in, SD d_a = 0.15 in, SD ,(ASCE 7-22 Tab 12.2-1 & Tab 11.5-1)

$$\frac{v_b h}{1000 G_a} = \frac{v_b h}{G t} + 0.75 h e_n$$

C_M = 1.0 (NDS 4.1.4) Δ_a = 0.02 h_{sx} , (ASCE 7-22 Tab 12.12-1)

CHECK EDGE STUD CAPACITY

P_{max} = 3.20 kips, (this value should include upper level DOWNWARD loads if applicable)

F_c = 1350 psi C_D = 1.60 C_P = 0.21 A = 10.5 in^2

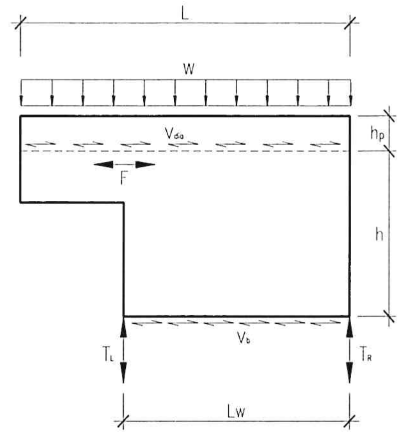
E = 1600 ksi C_F = 1.15 F_c' = 532 psi > f_c = 305 psi

[Satisfactory]

Lobana Engineering Inc.
885 Patriot Dr. Unit G
Moorpark, CA 93021

PROJECT : GRID 3
CLIENT :
JOB NO. :
DATE :
PAGE :
DESIGN BY :
REVIEW BY :
DS

Shear Wall Design Based on 2021 IBC / 2022 CBC / SDPWS-21



INPUT DATA

LATERAL FORCE ON DIAPHRAGM: $v_{dia,\ WIND}$ = 0 plf,for wind, ASD

$v_{dia,\ SEISMIC}$ = 132 plf,for seismic, ASD

GRAVITY LOADS ON THE ROOF: w_{DL} = plf,for dead load

w_{LL} = 0 plf,for live load

DIMENSIONS: L_w = 13 ft , h = 8.5 ft

L = 22 ft , h_p = 0 ft

PANEL GRADE (0 or 1) = 1 <= Sheathing and Single-Floor
MINIMUM NOMINAL PANEL THICKNESS = 15/32 in
COMMON NAIL SIZE (0=6d, 1=8d, 2=10d) 2 10d
SPECIFIC GRAVITY OF FRAMING MEMBERS 0.5
EDGE STUD SECTION 2 pcs, b = 2 in , h = 4 in
SPECIES (1 = DFL, 2 = SP) 1 DOUGLAS FIR-LARCH
GRADE (1, 2, 3, 4, 5, or 6) 4 No. 2
STORY OPTION (1=ground level, 2=upper level) 2 upper level shear wall

THE SHEAR WALL DESIGN IS ADEQUATE.

DESIGN SUMMARY

BLOCKED 15/32 SHEATHING WITH 10d COMMON NAILS
@ 6 in O.C. BOUNDARY & ALL EDGES / 12 in O.C. FIELD,
SILL PLATE ATTACHMENT 16d AT 6" O.C.

HOLD-DOWN FORCES: T_L = 1.56 k , T_R = 1.56 k (USE CS16 SIMPSON HOLD-DOWN)

DRAG STRUT FORCES: F = 2.97 k

EDGE STUD: 2 - 2" x 4" DOUGLAS FIR-LARCH No. 2, CONTINUOUS FULL HEIGHT.

SHEAR WALL DEFLECTION: Δ = 0.18 in

ANALYSIS

CHECK MAX SHEAR WALL DIMENSION RATIO L / B = 0.7 < 3.5 **[Satisfactory]**

DETERMINE REQUIRED CAPACITY v_b = 223 plf, (A) 1 Side Diaphragm Required, the Max. Nail Spacing = 6 in)

THE SHEAR CAPACITIES PER IBC Table 2306.3 / SDPWS-21 Table 4.3A with ASD reduction factor 2.0)

Panel Grade	Common Nail	Min. Penetration (in)	Min. Thickness (in)	Blocked Nail Spacing Boundary & All Edges			
				6	4	3	2
Sheathing and Single-Floor	10d	1 5/8	15/32	310	460	600	770

Use HDU2

Note: 1. The indicated shear numbers have reduced by specific gravity factor per IBC note a.
2. Since the wall is blocked, SDPW-15 Table 4.3.3.2 does not apply.

DETERMINE DRAG STRUT FORCE: $F = (L-L_w)\ MAX(\ v_{dia,\ WIND},\ \Omega_0 v_{dia,\ SEISMIC}\ )$ = 2.97 k (Ω_0 = 2.5) (Sec. 1633.2.6)

DETERMINE FLOOR SILL PLATE ATTACHMENT (NDS 2015, Table 11Q & Table 11L)
SILL PLATE ATTACHMENT 16d AT 6" O.C.
THE HOLD-DOWN FORCES:

	v_{dia} (plf)	Wall Seismic at mid-story (lbs)	Overturning Moments (ft-lbs)		Resisting Moments (ft-lbs)	Safety Factors	Net Uplift (lbs)		Holddown SIMPSON
SEISMIC	132	177	25435	Left	5746	0.9	T_L =	1559	CS16
				Right	5746	0.9	T_R =	1559	
WIND	0		0	Left	5746	2/3	T_L =	0	
				Right	5746	2/3	T_R =	0	

(T_L & T_R values should include upper level UPLIFT forces if applicable)

CHECK MAXIMUM SHEAR WALL DEFLECTION: (IBC Section 2305.3 / SDPWS-21 4.3.4)

$$\Delta = \Delta_{Bending} + \Delta_{Shear} + \Delta_{Nail\ \ slip} + \Delta_{Chord\ \ splice\ \ slip} = \frac{8 v_b h^3}{E A L_w} + \frac{v_b h}{G t} + 0.75 h e_n + \frac{h d_a}{L_w}$$

= 0.180 in, ASD <

$\delta_{xe,allowable,\ ASD}$ = 0.364 in

Where: v_b = 223 plf , ASD L_w = 13 ft E = 1.7E+06 psi **[Satisfactory]** (ASCE 7-22 12.8.6)

A = 16.50 in^2 h = 9 ft G = 9.0E+04 psi C_d = 4 I = 1

t = 0.298 in e_n = 0.008 in, SD d_a = 0.15 in, SD ,(ASCE 7-22 Tab 12.2-1 & Tab 11.5-1)

$$\frac{v_b h}{1000 G_a} = \frac{v_b h}{G t} + 0.75 h e_n$$

C_M = 1.0 (NDS 4.1.4) Δ_a = 0.02 h_{sx} , (ASCE 7-22 Tab 12.12-1)

CHECK EDGE STUD CAPACITY

P_{max} = 1.60 kips, (this value should include upper level DOWNWARD loads if applicable)

F_c = 1350 psi C_D = 1.60 C_P = 0.21 A = 10.5 in^2

E = 1600 ksi C_F = 1.15 F_c' = 532 psi > f_c = 152 psi

[Satisfactory]

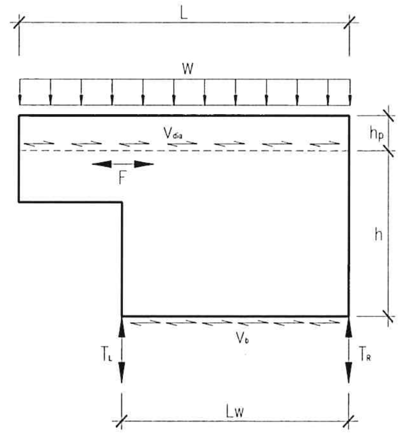
Lobana Engineering Inc.
885 Patriot Dr. Unit G
Moorpark, CA 93021

PROJECT : GRID A
CLIENT :
JOB NO. :
DATE :

DESIGN BY :
REVIEW BY :

Shear Wall Design Based on 2021 IBC / 2022 CBC / SDPWS-21

INPUT DATA



THE SHEAR WALL DESIGN IS ADEQUATE.

LATERAL FORCE ON DIAPHRAGM: $v_{dia,\ WIND}$ = 0 plf,for wind, ASD

$v_{dia,\ SEISMIC}$ = 82 plf,for seismic, ASD

GRAVITY LOADS ON THE ROOF: w_{DL} = plf,for dead load

(6+28) w_{LL} = 0 plf,for live load

DIMENSIONS: L_w = 44 ft , h = 8.5 ft

L = 47.5 ft , h_p = 0 ft

PANEL GRADE (0 or 1) = 1 <= Sheathing and Single-Floor
MINIMUM NOMINAL PANEL THICKNESS = 15/32 in
COMMON NAIL SIZE (0=6d, 1=8d, 2=10d) 2 10d
SPECIFIC GRAVITY OF FRAMING MEMBERS 0.5
EDGE STUD SECTION 2 pcs, b = 2 in , h = 4 in
SPECIES (1 = DFL, 2 = SP) 1 DOUGLAS FIR-LARCH
GRADE (1, 2, 3, 4, 5, or 6) 4 No. 2
STORY OPTION (1=ground level, 2=upper level) 2 upper level shear wall

DESIGN SUMMARY

BLOCKED 15/32 SHEATHING WITH 10d COMMON NAILS
@ 6 in O.C. BOUNDARY & ALL EDGES / 12 in O.C. FIELD,
SILL PLATE ATTACHMENT 16d AT 6" O.C.

HOLD-DOWN FORCES: T_L = 0.00 k , T_R = 0.00 k (HOLD-DOWN NOT REQUIRED)

DRAG STRUT FORCES: F = 0.72 k
EDGE STUD: 2 - 2" x 4" DOUGLAS FIR-LARCH No. 2, CONTINUOUS FULL HEIGHT.
SHEAR WALL DEFLECTION: Δ = 0.07 in

ANALYSIS

CHECK MAX SHEAR WALL DIMENSION RATIO L / B = 0.2 < 3.5 **[Satisfactory]**
DETERMINE REQUIRED CAPACITY v_b = 89 plf, (A 1 Side Diaphragm Required, the Max. Nail Spacing = 6 in)

THE SHEAR CAPACITIES PER IBC Table 2306.3 / SDPWS-21 Table 4.3A with ASD reduction factor 2.0)

Panel Grade	Common Nail	Min. Penetration (in)	Min. Thickness (in)	Blocked Nail Spacing Boundary & All Edges 6	4	3	2
Sheathing and Single-Floor	10d	1 5/8	15/32	310	460	600	770

Note: 1. The indicated shear numbers have reduced by specific gravity factor per IBC note a.
2. Since the wall is blocked, SDPW-15 Table 4.3.3.2 does not apply.

DETERMINE DRAG STRUT FORCE: $F = (L-L_w)\ MAX(\ v_{dia,\ WIND},\ \Omega_0 v_{dia,\ SEISMIC})$ = 0.72 k (Ω_0 = 2.5) (Sec. 1633.2.6)

No uplift

DETERMINE FLOOR SILL PLATE ATTACHMENT (NDS 2015, Table 11Q & Table 11L)
SILL PLATE ATTACHMENT 16d AT 6" O.C.
THE HOLD-DOWN FORCES:

NO HDU needed

	v_{dia} (plf)	Wall Seismic at mid-story (lbs)	Overturning Moments (ft-lbs)		Resisting Moments (ft-lbs)	Safety Factors	Net Uplift (lbs)		Holddown SIMPSON
SEISMIC	82	598	35651	Left	65824	0.9	T_L =	0	CS16
				Right	65824	0.9	T_R =	0	
WIND	0		0	Left	65824	2/3	T_L =	0	
				Right	65824	2/3	T_R =	0	

(T_L & T_R values should include upper level UPLIFT forces if applicable)

CHECK MAXIMUM SHEAR WALL DEFLECTION: (IBC Section 2305.3 / SDPWS-21 4.3.4)

$$\Delta = \Delta_{Bending} + \Delta_{Shear} + \Delta_{Nail\ slip} + \Delta_{Chord\ splice\ slip} = \frac{8 v_b h^3}{EAL_w} + \frac{v_b h}{Gt} + 0.75 h e_n + \frac{h d_a}{L_w}$$

= 0.070 in, ASD < $\delta_{xe,allowable,\ ASD}$ = 0.364 in

[Satisfactory] (ASCE 7-22 12.8.6)

Where: v_b = 89 plf , ASD L_w = 44 ft E = 1.7E+06 psi
A = 16.50 in^2 h = 9 ft G = 9.0E+04 psi C_d = 4 I = 1
t = 0.298 in e_n = 0.005 in, SD d_a = 0.15 in, SD ,(ASCE 7-22 Tab 12.2-1 & Tab 11.5-1)

$$\frac{v_b h}{1000 G_a} = \frac{v_b h}{Gt} + 0.75 h e_n$$

C_M = 1.0 (NDS 4.1.4) Δ_a = 0.02 h_{sx} , (ASCE 7-22 Tab 12.12-1)

CHECK EDGE STUD CAPACITY

P_{max} = 1.54 kips, (this value should include upper level DOWNWARD loads if applicable)

F_c = 1350 psi C_D = 1.60 C_P = 0.21 A = 10.5 in^2

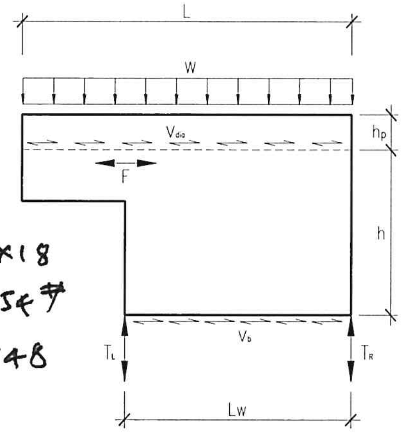
E = 1600 ksi C_F = 1.15 F_c' = 532 psi > f_c = 146 psi

[Satisfactory]

Lobana Engineering Inc. ***885 Patriot Dr. Unit G*** ***Moorpark, CA 93021***	PROJECT : GRID B CLIENT : JOB NO. : DATE :	PAGE : 30 DESIGN BY : REVIEW BY :

Shear Wall Design Based on 2021 IBC / 2022 CBC / SDPWS-21

INPUT DATA



LATERAL FORCE ON DIAPHRAGM: $V_{dia,\ WIND}$ = 0 plf,for wind, ASD

$V_{dia,\ SEISMIC}$ = 103 plf,for seismic, ASD

GRAVITY LOADS ON THE ROOF: W_{DL} = plf,for dead load

(6+6) W_{LL} = 0 plf,for live load

DIMENSIONS: L_w = 12 ft , h = 8.5 ft

L = 47.5 ft , h_p = 0 ft

PANEL GRADE (0 or 1) = 1 <= Sheathing and Single-Floor Drag = 103×18

MINIMUM NOMINAL PANEL THICKNESS = 15/32 in = 1854#

COMMON NAIL SIZE (0=6d, 1=8d, 2=10d) 2 10d

SPECIFIC GRAVITY OF FRAMING MEMBERS 0.5

EDGE STUD SECTION 2 pcs, b = 2 in , h = 4 in MST48

SPECIES (1 = DFL, 2 = SP) 1 DOUGLAS FIR-LARCH

GRADE (1, 2, 3, 4, 5, or 6) 4 No. 2

STORY OPTION (1=ground level, 2=upper level) 2 upper level shear wall

THE SHEAR WALL DESIGN IS ADEQUATE.

DESIGN SUMMARY

BLOCKED 15/32 SHEATHING WITH 10d COMMON NAILS
@ 4 in O.C. BOUNDARY & ALL EDGES / 12 in O.C. FIELD,
SILL PLATE ATTACHMENT 16d AT 4" O.C.

HOLD-DOWN FORCES: T_L = 3.16 k , T_R = 3.16 k (USE 2-CS16 SIMPSON HOLD-DOWN)

DRAG STRUT FORCES: F = 9.14 k

EDGE STUD: 2 - 2" x 4" DOUGLAS FIR-LARCH No. 2, CONTINUOUS FULL HEIGHT.

SHEAR WALL DEFLECTION: Δ = 0.23 in

ANALYSIS

CHECK MAX SHEAR WALL DIMENSION RATIO L / B = 0.7 < 3.5 **[Satisfactory]**

DETERMINE REQUIRED CAPACITY v_b = 408 plf, (B 1 Side Diaphragm Required, the Max. Nail Spacing = 4 in)

THE SHEAR CAPACITIES PER IBC Table 2306.3 / SDPWS-21 Table 4.3A with ASD reduction factor 2.0)

Panel Grade	Common Nail	Min. Penetration (in)	Min. Thickness (in)	Blocked Nail Spacing Boundary & All Edges			
				6	4	3	2
Sheathing and Single-Floor	10d	1 5/8	15/32	310	460	600	770

Use HDU5 w/
6×6

Note: 1. The indicated shear numbers have reduced by specific gravity factor per IBC note a.
2. Since the wall is blocked, SDPW-15 Table 4.3.3.2 does not apply.

DETERMINE DRAG STRUT FORCE: F = $(L-L_w)\ \mathrm{MAX}(\ v_{dia,\ WIND},\ \Omega_0 v_{dia,\ SEISMIC})$ = 9.14 k (Ω_0 = 2.5) (Sec. 1633.2.6)

DETERMINE FLOOR SILL PLATE ATTACHMENT (NDS 2015, Table 11Q & Table 11L)
SILL PLATE ATTACHMENT 16d AT 4" O.C.
THE HOLD-DOWN FORCES:

	v_{dia} (plf)	Wall Seismic at mid-story (lbs)	Overturning Moments (ft-lbs)		Resisting Moments (ft-lbs)	Safety Factors	Net Uplift (lbs)		Holddown SIMPSON
SEISMIC	103	163	42280	Left	4896	0.9	T_L =	3156	2-CS16
				Right	4896	0.9	T_R =	3156	
WIND	0		0	Left	4896	2/3	T_L =	0	
				Right	4896	2/3	T_R =	0	

(T_L & T_R values should include upper level UPLIFT forces if applicable)

CHECK MAXIMUM SHEAR WALL DEFLECTION: (IBC Section 2305.3 / SDPWS-21 4.3.4)

$$\Delta = \Delta_{Bending} + \Delta_{Shear} + \Delta_{Nail\ slip} + \Delta_{Chord\ splice\ slip} = \frac{8 v_b h^3}{E A L_w} + \frac{v_b h}{G t} + 0.75 h e_n + \frac{h d_a}{L_w}$$

= 0.228 in, ASD < $\delta_{xe,allowable,\ ASD}$ = 0.364 in

[Satisfactory] (ASCE 7-22 12.8.6)

Where: v_b = 408 plf , ASD; L_w = 12 ft; E = 1.7E+06 psi

A = 16.50 in^2; h = 9 ft; G = 9.0E+04 psi; C_d = 4; I = 1

t = 0.298 in; e_n = 0.004 in, SD; d_a = 0.15 in, SD ,(ASCE 7-22 Tab 12.2-1 & Tab 11.5-1)

$$\frac{v_b h}{1000 G_a} = \frac{v_b h}{G t} + 0.75 h e_n$$

C_M = 1.0 (NDS 4.1.4); Δ_a = 0.02 h_{sx} , (ASCE 7-22 Tab 12.12-1)

CHECK EDGE STUD CAPACITY

P_{max} = 2.62 kips, (this value should include upper level DOWNWARD loads if applicable)

F_c = 1350 psi; C_D = 1.60; C_P = 0.21; A = 10.5 in^2

E = 1600 ksi; C_F = 1.15; F_c' = 532 psi > f_c = 250 psi

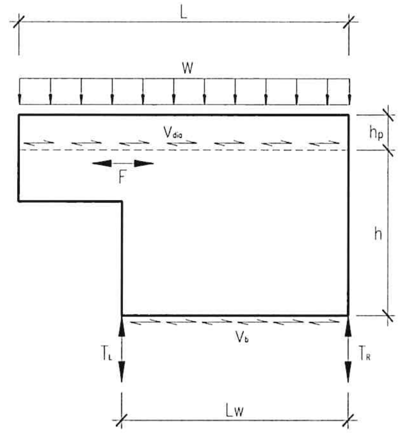
[Satisfactory]

Lobana Engineering Inc.
885 Patriot Dr. Unit G
Moorpark, CA 93021

PROJECT : GRID C
CLIENT :
JOB NO. :
DATE :
PAGE : 31
DESIGN BY :
REVIEW BY :

Shear Wall Design Based on 2021 IBC / 2022 CBC / SDPWS-21

INPUT DATA



LATERAL FORCE ON DIAPHRAGM: $v_{dia,\ WIND}$ = 0 plf,for wind, ASD

$v_{dia,\ SEISMIC}$ = 83 plf,for seismic, ASD

GRAVITY LOADS ON THE ROOF: w_{DL} = plf,for dead load

w_{LL} = 0 plf,for live load

(5+5)

DIMENSIONS: L_w = 10 ft , h = 8.5 ft

L = 18 ft , h_p = 0 ft

PANEL GRADE (0 or 1) = 1 <= Sheathing and Single-Floor

MINIMUM NOMINAL PANEL THICKNESS = 15/32 in

COMMON NAIL SIZE (0=6d, 1=8d, 2=10d) 2 10d

SPECIFIC GRAVITY OF FRAMING MEMBERS 0.5

EDGE STUD SECTION 2 pcs, b = 2 in , h = 4 in

SPECIES (1 = DFL, 2 = SP) 1 DOUGLAS FIR-LARCH

GRADE (1, 2, 3, 4, 5, or 6) 4 No. 2

STORY OPTION (1=ground level, 2=upper level) 2 upper level shear wall

THE SHEAR WALL DESIGN IS ADEQUATE.

DESIGN SUMMARY

BLOCKED 15/32 SHEATHING WITH 10d COMMON NAILS
@ 6 in O.C. BOUNDARY & ALL EDGES / 12 in O.C. FIELD,
SILL PLATE ATTACHMENT 16d AT 6" O.C.

HOLD-DOWN FORCES: T_L = 1.02 k , T_R = 1.02 k (USE CS16 SIMPSON HOLD-DOWN)

DRAG STRUT FORCES: F = 1.66 k

EDGE STUD: 2 - 2" x 4" DOUGLAS FIR-LARCH No. 2, CONTINUOUS FULL HEIGHT.

SHEAR WALL DEFLECTION: Δ = 0.16 in

ANALYSIS

CHECK MAX SHEAR WALL DIMENSION RATIO L / B = 0.9 < 3.5 **[Satisfactory]**

DETERMINE REQUIRED CAPACITY v_b = 149 plf, (A Side Diaphragm Required, the Max. Nail Spacing = 6 in)

THE SHEAR CAPACITIES PER IBC Table 2306.3 / SDPWS-21 Table 4.3A with ASD reduction factor 2.0)

Panel Grade	Common Nail	Min. Penetration (in)	Min. Thickness (in)	Blocked Nail Spacing Boundary & All Edges 6	4	3	2
Sheathing and Single-Floor	10d	1 5/8	15/32	310	460	600	770

Use HDU2

Note: 1. The indicated shear numbers have reduced by specific gravity factor per IBC note a.
2. Since the wall is blocked, SDPW-15 Table 4.3.3.2 does not apply.

DETERMINE DRAG STRUT FORCE: $F = (L-L_w)\ MAX(v_{dia,\ WIND},\ \Omega_0 v_{dia,\ SEISMIC})$ = 1.66 k (Ω_0 = 2.5) (Sec. 1633.2.6)

DETERMINE FLOOR SILL PLATE ATTACHMENT (NDS 2015, Table 11Q & Table 11L)
SILL PLATE ATTACHMENT 16d AT 6" O.C.
THE HOLD-DOWN FORCES:

	v_{dia} (plf)	Wall Seismic at mid-story (lbs)	Overturning Moments (ft-lbs)		Resisting Moments (ft-lbs)	Safety Factors	Net Uplift (lbs)		Holddown SIMPSON
SEISMIC	83	136	13277	Left	3400	0.9	T_L =	1022	CS16
				Right	3400	0.9	T_R =	1022	
WIND	0		0	Left	3400	2/3	T_L =	0	
				Right	3400	2/3	T_R =	0	

(T_L & T_R values should include upper level UPLIFT forces if applicable)

CHECK MAXIMUM SHEAR WALL DEFLECTION: (IBC Section 2305.3 / SDPWS-21 4.3.4)

$$\Delta = \Delta_{Bending} + \Delta_{Shear} + \Delta_{Nail\ slip} + \Delta_{Chord\ splice\ slip} = \frac{8 v_b h^3}{E A L_w} + \frac{v_b h}{G t} + 0.75 h e_n + \frac{h d_a}{L_w} = 0.162 \text{ in, ASD} < \delta_{xe,allowable,\ ASD} = 0.364 \text{ in}$$

Where: v_b = 149 plf , ASD L_w = 10 ft E = 1.7E+06 psi **[Satisfactory]** (ASCE 7-22 12.8.6)

A = 16.50 in^2 h = 9 ft G = 9.0E+04 psi C_d = 4 I = 1

t = 0.298 in e_n = 0.005 in, SD d_a = 0.15 in, SD ,(ASCE 7-22 Tab 12.2-1 & Tab 11.5-1)

$$\frac{v_b h}{1000 G_a} = \frac{v_b h}{G t} + 0.75 h e_n$$

C_M = 1.0 (NDS 4.1.4) Δ_a = 0.02 h_{sx} , (ASCE 7-22 Tab 12.12-1)

CHECK EDGE STUD CAPACITY

P_{max} = 1.11 kips, (this value should include upper level DOWNWARD loads if applicable)

F_c = 1350 psi C_D = 1.60 C_P = 0.21 A = 10.5 in^2

E = 1600 ksi C_F = 1.15 F_c' = 532 psi > f_c = 106 psi

[Satisfactory]

Knee brace @ grid (D)

R_D | Post = $\frac{500 \times 6.5}{1.5} \times 1/2 = 1084$#

[R = 1.5 wooden Post]

Try 2-3/4" M. bolts

$V_B = \frac{1084}{2} \times 8/2 = 2168$#

$P_B = \sqrt{2} \times 2168 = 3066$#

Allowable shear = $2 \times 1.33 \times 1570 = 4176$#

Allowable Tension = $2 \times 1.33 \times 513 \times 5 = 6823$#

$\Big\} > V_B$

Combine shear + Tension

$= \frac{4176 \times 6823}{.5(4176 + 6823)} = 5182$# > 3066#

Use 6x6 knee brace
w/ 2-3/4" M. bolts

$w_5 = \left(\frac{8.2}{48}\right)\left(22\times18+15\times10\right)\times1.3 = 122$ #/'

$w_6 = \left(\frac{8.2}{48}\right)\left(38.5\times18+15\times10\right)\times1.3 = 187$ #/'

$w_7 = \left(\frac{8.2}{48}\right)\left(18\times18+150\right)\times1.3 = 106$ #/'

$w_8 = \left(\frac{8.2}{48}\right)\left(42\times18+150\right)\times1.3 = 201$ #/'

$R_1 = R_2 = 187\times18/2 = 1683$ #

$R_2 = R_3 = 122\times24/2 = 1464$ #

$R_A = R_B = 201\times22/2 = 2211$ #

$R_B = \left(106\times16.5\times16.5/2 + 1501\times5.5\right)/16.5 = 1375$ #

$R_D = 1875$ #

Floor Drapm

$R_1 = (1683+2745)/38.5 = 115$ #/'

$R_2 = (1683+1464+5645)/38.5 = 228$ #/'
$= 8792$ #

$R_3 = (1464+2900)/22 = 198$ #/'

$R_A = (2211+3894)/42 = 145$ #/'

$R_B = (2211+1375+4895)/42 = 202$ #/'
$= 8481$ #

$R_D = (1875+500)/18 = 137.5$ #/'
$= 2475$ #

$\boxed{2}$ Use 3/4" T & G 10d @ 6" o.c

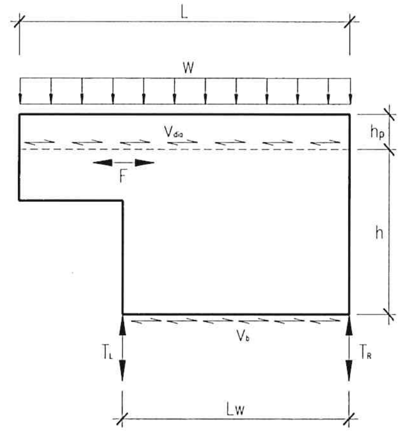
Lobana Engineering Inc.
885 Patriot Dr. Unit G
Moorpark, CA 93021

PROJECT : GRID 1
CLIENT :
JOB NO. :
DATE :

PAGE : 34
DESIGN BY :
REVIEW BY :

Shear Wall Design Based on 2021 IBC / 2022 CBC / SDPWS-21

INPUT DATA



LATERAL FORCE ON DIAPHRAGM: $v_{dia,\ WIND}$ = 0 plf,for wind, ASD

$v_{dia,\ SEISMIC}$ = 115 plf,for seismic, ASD

GRAVITY LOADS ON THE ROOF: w_{DL} = plf,for dead load

w_{LL} = 0 plf,for live load

DIMENSIONS: L_w = 38 ft , h = 10 ft

L = 38.5 ft , h_p = 0 ft

PANEL GRADE (0 or 1) = 1 <= Sheathing and Single-Floor

MINIMUM NOMINAL PANEL THICKNESS = 15/32 in

COMMON NAIL SIZE (0=6d, 1=8d, 2=10d) 2 10d

SPECIFIC GRAVITY OF FRAMING MEMBERS 0.5

EDGE STUD SECTION 2 pcs, b = 2 in , h = 4 in

SPECIES (1 = DFL, 2 = SP) 1 DOUGLAS FIR-LARCH

GRADE (1, 2, 3, 4, 5, or 6) 4 No. 2

STORY OPTION (1=ground level, 2=upper level) 1 ground level shear wall

THE SHEAR WALL DESIGN IS ADEQUATE.

DESIGN SUMMARY

BLOCKED 15/32 SHEATHING WITH 10d COMMON NAILS
@ 6 in O.C. BOUNDARY & ALL EDGES / 12 in O.C. FIELD,
5/8 in DIA. x 10 in LONG ANCHOR BOLTS @ 48 in O.C. (or 1/2 in DIA. x 10 in LONG ANCHOR BOLTS @ 48 in O.C.)

HOLD-DOWN FORCES: T_L = 0.00 k , T_R = 0.00 k (HOLD-DOWN NOT REQUIRED)

DRAG STRUT FORCES: F = 0.14 k

EDGE STUD: 2 - 2" x 4" DOUGLAS FIR-LARCH No. 2, CONTINUOUS FULL HEIGHT.

SHEAR WALL DEFLECTION: Δ = 0.11 in

ANALYSIS

CHECK MAX SHEAR WALL DIMENSION RATIO L / B = 0.3 < 3.5 **[Satisfactory]**

DETERMINE REQUIRED CAPACITY v_b = 117 plf, (A 1 Side Diaphragm Required, the Max. Nail Spacing = 6 in)

THE SHEAR CAPACITIES PER IBC Table 2306.3 / SDPWS-21 Table 4.3A with ASD reduction factor 2.0)

Panel Grade	Common Nail	Min. Penetration (in)	Min. Thickness (in)	Blocked Nail Spacing Boundary & All Edges 6	4	3	2
Sheathing and Single-Floor	10d	1 5/8	15/32	310	460	600	770

No uplift
No HDU
needed

Note: 1. The indicated shear numbers have reduced by specific gravity factor per IBC note a.
2. Since the wall is blocked, SDPW-15 Table 4.3.3.2 does not apply.

DETERMINE DRAG STRUT FORCE: $F = (L-L_w)\ \mathrm{MAX}(\ v_{dia,\ WIND},\ \Omega_0 v_{dia,\ SEISMIC})$ = 0.14 k (Ω_0 = 2.5) (Sec. 1633.2.6)

DETERMINE MAX SPACING OF 5/8" DIA (or 1/2" DIA) ANCHOR BOLT (NDS 2015, Tab.11E)
5/8 in DIA. x 10 in LONG ANCHOR BOLTS @ 48 in O.C. (or 1/2 in DIA. x 10 in LONG ANCHOR BOLTS @ 48 in O.C.)
THE HOLD-DOWN FORCES:

	v_{dia} (plf)	Wall Seismic at mid-story (lbs)	Overturning Moments (ft-lbs)		Resisting Moments (ft-lbs)	Safety Factors	Net Uplift (lbs)		Holddown SIMPSON
SEISMIC	115	608	47315	Left	57760	0.9	T_L =	0	HDU2-1/4x2.5
				Right	57760	0.9	T_R =	0	
WIND	0		0	Left	57760	2/3	T_L =	0	
				Right	57760	2/3	T_R =	0	

(T_L & T_R values should include upper level UPLIFT forces if applicable)

CHECK MAXIMUM SHEAR WALL DEFLECTION: (IBC Section 2305.3 / SDPWS-21 4.3.4)

$$\Delta = \Delta_{Bending} + \Delta_{Shear} + \Delta_{Nail\ slip} + \Delta_{Chord\ splice\ slip} = \frac{8v_b h^3}{EAL_w} + \frac{v_b h}{Gt} + 0.75he_n + \frac{hd_a}{L_w}$$

= 0.106 in, ASD < $\delta_{xe,allowable,\ ASD}$ = 0.429 in

Where: v_b = 117 plf , ASD L_w = 38 ft E = 1.7E+06 psi **[Satisfactory]** (ASCE 7-22 12.8.6)

A = 16.50 in^2 h = 10 ft G = 9.0E+04 psi C_d = 4 I = 1

t = 0.298 in e_n = 0.006 in, SD d_a = 0.15 in, SD ,(ASCE 7-22 Tab 12.2-1 & Tab 11.5-1)

$$\frac{v_b h}{1000G_a} = \frac{v_b h}{Gt} + 0.75he_n$$

C_M = 1.0 (NDS 4.1.4) Δ_a = 0.02 h_{sx} , (ASCE 7-22 Tab 12.12-1)

CHECK EDGE STUD CAPACITY

P_{max} = 1.84 kips, (this value should include upper level DOWNWARD loads if applicable)

F_c = 1350 psi C_D = 1.60 C_P = 0.16 A = 10.5 in^2

E = 1600 ksi C_F = 1.15 F_c' = 391 psi > f_c = 176 psi

[Satisfactory]

R2 = 8792# < 2x 5230 = 10460#   Drag = 228x22 = 5016#

Use 2-HFx 24x9 △E High St-bolt   Use MST60   35

MiTek
HARDY FRAME
Shear Wall Systems

Table 1.1A MiTek® Hardy Frame® Installation - on Concrete[1,2]

Model Number	Net Height H (in)	Concrete Compressive Strength f'c (psi)	HD Bolt Dia (in) and Grade [3]	Applied Axial Load [4]	Seismic R=6.5, C_d =4.0: Allowable In-Plane Shear V [5] (lbs)	Seismic: Drift at V [5] (in)	Seismic: Uplift at V [5,6] (lbs)	Wind: Allowable In-Plane Shear V [5] (lbs)	Wind: Drift at V [5] (in)	Wind: Uplift at V [5,6] (lbs)
HFX-24x9	104 1/4	2,500	1 1/8" STD	1,000	3,140	0.175	17,810	3,525	0.197	20,490
				3,500						
				6,500						
			1 1/8" HS	1,000	5,230	0.294	35,310	6,015	0.338	45,935
				3,500				5,910	0.332	44,165
				6,500				5,755	0.324	41,850
		3,000	1 1/8" STD	1,000	3,140	0.175	17,270	3,620	0.202	20,380
				3,500						
				6,500						
			1 1/8" HS	1,000	5,230	0.294	32,375	6,350	0.357	43,195
				3,500						
				6,500						
		4,000	1 1/8" STD	1,000	3,140	0.175	16,680	3,685	0.206	19,925
				3,500						
				6,500						
			1 1/8" HS	1,000	5,230	0.294	29,900	6,350	0.357	38,105
				3,500						
				6,500						
HFX-32x9	104 1/4	2,500	7/8" STD	1,000	2,190	0.181	9,320	2,500	0.207	10,630
				3,500	1,910	0.158	8,130	1,910	0.158	8,130
				6,500	1,205	0.100	5,130	1,205	0.100	5,130
			7/8" HS	1,000	2,655	0.220	11,295	2,655	0.220	11,295
				3,500	2,065	0.171	8,795	2,065	0.171	8,795
				6,500	1,360	0.113	5,795	1,360	0.113	5,795
		3,000	7/8" STD	1,000	2,190	0.181	9,320	2,665	0.221	11,350
				3,500				2,195	0.182	9,335
				6,500	1,490	0.123	6,335	1,490	0.123	6,335
			7/8" HS	1,000	3,230	0.268	13,755	3,230	0.268	13,755
				3,500	2,645	0.219	11,255	2,645	0.219	11,255
				6,500	1,940	0.161	8,255	1,940	0.161	8,255
		4,000	7/8" STD	1,000	2,190	0.181	9,320	2,665	0.221	11,350
				3,500				2,550	0.211	10,845
				6,500	1,845	0.152	7,845	1,845	0.153	7,845
			7/8" HS	1,000	3,885	0.322	16,530	4,310	0.357	18,330
				3,500	3,720	0.308	15,830	3,720	0.308	15,830
				6,500	3,015	0.250	12,830	3,015	0.250	12,830
HFX-44x9	104 1/4	2,500	7/8" STD	1,000	2,745	0.121	8,005	3,405	0.151	9,930
				3,500				2,870	0.127	8,365
				6,500	1,840	0.081	5,365	1,840	0.081	5,365
			7/8"HS	1,000	3,995	0.177	11,645	3,995	0.177	11,645
				3,500	3,135	0.139	9,145	3,135	0.139	9,145
				6,500	2,105	0.093	6,145	2,105	0.093	6,145
		3,000	7/8" STD	1,000	2,745	0.121	8,005	3,405	0.151	9,930
				3,500				3,220	0.142	9,385
				6,500	2,190	0.096	6,385	2,190	0.097	6,385
			7/8" HS	1,000	4,860	0.215	14,175	4,860	0.215	14,175
				3,500	4,005	0.177	11,675	4,005	0.177	11,675
				6,500	2,975	0.132	8,670	2,975	0.132	8,670
		4,000	7/8" STD	1,000	2,745	0.121	8,005	3,405	0.151	9,930
				3,500						
				6,500	2,625	0.116	7,655	2,625	0.116	7,655
			7/8" HS	1,000	5,260	0.233	15,340	6,525	0.289	19,030
				3,500				5,670	0.251	16,530
				6,500	4,640	0.206	13,530	4,640	0.205	13,530
HFX-12x10	116 1/4	2,500	1 1/8" STD	1,000	1,175	0.273	19,595	1,175	0.273	19,595
				3,500	1,080	0.252	17,005	1,080	0.252	17,005
				6,500	965	0.225	14,325	965	0.225	14,325
			1 1/8" HS	1,000	1,175	0.274	19,595	1,175	0.275	19,595
				3,500	1,080	0.253	17,005	1,080	0.253	17,005
				6,500	965	0.226	14,325	965	0.226	14,325
		3,000	1 1/8" STD	1,000	1,185	0.276	17,740	1,340	0.313	21,575
				3,500				1,325	0.308	21,075
				6,500				1,215	0.283	18,375
			1 1/8" HS	1,000	1,350	0.316	21,810	1,415	0.331	23,750
				3,500	1,325	0.310	21,075	1,325	0.310	21,075
				6,500	1,215	0.284	18,375	1,215	0.284	18,375
		4,000	1 1/8" STD	1,000	1,185	0.276	16,095	1,485	0.346	21,615
				3,500						
				6,500						
			1 1/8" HS	1,000	1,350	0.316	19,015	1,900	0.444	32,065
				3,500				1,805	0.423	29,275
				6,500				1,695	0.397	26,380

Lobana Engineering Inc.
885 Patriot Dr. Unit G
Moorpark, CA 93021

PROJECT : GRID 3
CLIENT :
JOB NO. :
DATE :

PAGE : 36
DESIGN BY :
REVIEW BY :

Shear Wall Design Based on 2021 IBC / 2022 CBC / SDPWS-21

INPUT DATA

LATERAL FORCE ON DIAPHRAGM: $v_{dia,\ WIND}$ = 0 plf,for wind, ASD

$v_{dia,\ SEISMIC}$ = 198 plf,for seismic, ASD

GRAVITY LOADS ON THE ROOF: w_{DL} = plf,for dead load

w_{LL} = 0 plf,for live load

DIMENSIONS: L_w = 21 ft , h = 10 ft

L = 22 ft , h_p = 0 ft

PANEL GRADE (0 or 1) = 1 <= Sheathing and Single-Floor

MINIMUM NOMINAL PANEL THICKNESS = 15/32 in

COMMON NAIL SIZE (0=6d, 1=8d, 2=10d) 2 10d

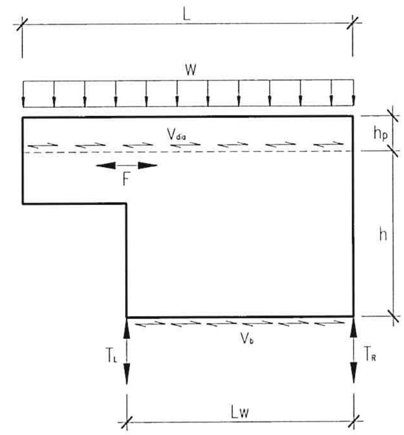
SPECIFIC GRAVITY OF FRAMING MEMBERS 0.5

EDGE STUD SECTION 2 pcs, b = 2 in , h = 4 in

SPECIES (1 = DFL, 2 = SP) 1 DOUGLAS FIR-LARCH

GRADE (1, 2, 3, 4, 5, or 6) 4 No. 2

STORY OPTION (1=ground level, 2=upper level) 1 ground level shear wall



THE SHEAR WALL DESIGN IS ADEQUATE.

DESIGN SUMMARY

BLOCKED 15/32 SHEATHING WITH 10d COMMON NAILS
@ 6 in O.C. BOUNDARY & ALL EDGES / 12 in O.C. FIELD,
5/8 in DIA. x 10 in LONG ANCHOR BOLTS @ 48 in O.C. (or 1/2 in DIA. x 10 in LONG ANCHOR BOLTS @ 36 in O.C.)

HOLD-DOWN FORCES: T_L = 1.40 k , T_R = 1.40 k (USE HDU2-1/4x2.5 SIMPSON HOLD-DOWN)

DRAG STRUT FORCES: F = 0.50 k

EDGE STUD: 2 - 2" x 4" DOUGLAS FIR-LARCH No. 2, CONTINUOUS FULL HEIGHT.

SHEAR WALL DEFLECTION: Δ = 0.22 in

ANALYSIS

CHECK MAX SHEAR WALL DIMENSION RATIO L / B = 0.5 < 3.5 **[Satisfactory]**

DETERMINE REQUIRED CAPACITY v_b = 207 plf, (A 1 Side Diaphragm Required, the Max. Nail Spacing = 6 in)

THE SHEAR CAPACITIES PER IBC Table 2306.3 / SDPWS-21 Table 4.3A with ASD reduction factor 2.0)

Panel Grade	Common Nail	Min. Penetration (in)	Min. Thickness (in)	Blocked Nail Spacing Boundary & All Edges			
				6	4	3	2
Sheathing and Single-Floor	10d	1 5/8	15/32	310	460	600	770

Use HDU2

Note: 1. The indicated shear numbers have reduced by specific gravity factor per IBC note a.
2. Since the wall is blocked, SDPW-15 Table 4.3.3.2 does not apply.

DETERMINE DRAG STRUT FORCE: $F = (L-L_w)\ MAX(v_{dia,\ WIND},\ \Omega_0 v_{dia,\ SEISMIC})$ = 0.50 k (Ω_0 = 2.5) (Sec. 1633.2.6)

DETERMINE MAX SPACING OF 5/8" DIA (or 1/2" DIA) ANCHOR BOLT (NDS 2015, Tab.11E)
5/8 in DIA. x 10 in LONG ANCHOR BOLTS @ 48 in O.C. (or 1/2 in DIA. x 10 in LONG ANCHOR BOLTS @ 36 in O.C.)
THE HOLD-DOWN FORCES:

	v_{dia} (plf)	Wall Seismic at mid-story (lbs)	Overturning Moments (ft-lbs)		Resisting Moments (ft-lbs)	Safety Factors	Net Uplift (lbs)		Holddown SIMPSON
SEISMIC	198	336	45240	Left	17640	0.9	T_L =	1398	HDU2-1/4x2.5
				Right	17640	0.9	T_R =	1398	
WIND	0		0	Left	17640	2/3	T_L =	0	
				Right	17640	2/3	T_R =	0	

(T_L & T_R values should include upper level UPLIFT forces if applicable)

CHECK MAXIMUM SHEAR WALL DEFLECTION: (IBC Section 2305.3 / SDPWS-21 4.3.4)

$$\Delta = \Delta_{Bending} + \Delta_{Shear} + \Delta_{Nail\ slip} + \Delta_{Chord\ splice\ slip} = \frac{8 v_b h^3}{EAL_w} + \frac{v_b h}{Gt} + 0.75 h e_n + \frac{h d_a}{L_w}$$

= 0.220 in, ASD <

$\delta_{xe,allowable,\ ASD}$ = 0.429 in

Where: v_b = 207 plf , ASD; L_w = 21 ft; E = 1.7E+06 psi; **[Satisfactory]** (ASCE 7-22 12.8.6)

A = 16.50 in^2; h = 10 ft; G = 9.0E+04 psi; C_d = 4; I = 1

t = 0.298 in; e_n = 0.017 in, SD; d_a = 0.15 in, SD ,(ASCE 7-22 Tab 12.2-1 & Tab 11.5-1)

$$\frac{v_b h}{1000 G_a} = \frac{v_b h}{Gt} + 0.75 h e_n$$

C_M = 1.0 (NDS 4.1.4)

Δ_a = 0.02 h_{sx} , (ASCE 7-22 Tab 12.12-1)

CHECK EDGE STUD CAPACITY

P_{max} = 2.00 kips, (this value should include upper level DOWNWARD loads if applicable)

F_c = 1350 psi; C_D = 1.60; C_P = 0.16; A = 10.5 in^2

E = 1600 ksi; C_F = 1.15; F_c' = 391 psi > f_c = 190 psi

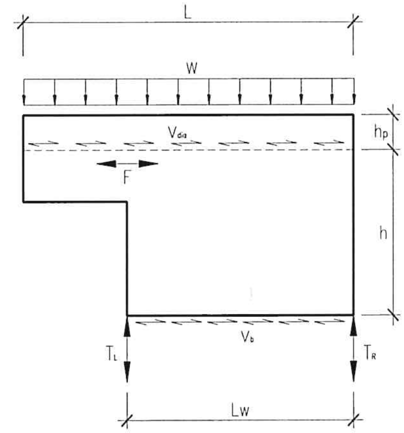
[Satisfactory]

Lobana Engineering Inc.
885 Patriot Dr. Unit G
Moorpark, CA 93021

PROJECT : GRID A
CLIENT :
JOB NO. :
DATE :
PAGE : 37
DESIGN BY :
REVIEW BY :

Shear Wall Design Based on 2021 IBC / 2022 CBC / SDPWS-21

INPUT DATA



LATERAL FORCE ON DIAPHRAGM: $v_{dia,\ WIND}$ = 0 plf,for wind, ASD

$v_{dia,\ SEISMIC}$ = 145 plf,for seismic, ASD

GRAVITY LOADS ON THE ROOF: w_{DL} = plf,for dead load

(16 + 11 + 10) w_{LL} = 0 plf,for live load

DIMENSIONS: L_w = 37 ft , h = 10 ft

L = 42 ft , h_p = 0 ft

PANEL GRADE (0 or 1) = 1 <= Sheathing and Single-Floor
MINIMUM NOMINAL PANEL THICKNESS = 15/32 in
COMMON NAIL SIZE (0=6d, 1=8d, 2=10d) 2 10d
SPECIFIC GRAVITY OF FRAMING MEMBERS 0.5
EDGE STUD SECTION 2 pcs, b = 2 in , h = 4 in
SPECIES (1 = DFL, 2 = SP) 1 DOUGLAS FIR-LARCH
GRADE (1, 2, 3, 4, 5, or 6) 4 No. 2
STORY OPTION (1=ground level, 2=upper level) 1 ground level shear wall

THE SHEAR WALL DESIGN IS ADEQUATE.

DESIGN SUMMARY

BLOCKED 15/32 SHEATHING WITH 10d COMMON NAILS
@ 6 in O.C. BOUNDARY & ALL EDGES / 12 in O.C. FIELD,
5/8 in DIA. x 10 in LONG ANCHOR BOLTS @ 48 in O.C. (or 1/2 in DIA. x 10 in LONG ANCHOR BOLTS @ 46 in O.C.)

HOLD-DOWN FORCES: T_L = 0.39 k , T_R = 0.39 k (USE HDU2-1/4x2.5 SIMPSON HOLD-DOWN)

DRAG STRUT FORCES: F = 1.81 k
EDGE STUD: 2 - 2" x 4" DOUGLAS FIR-LARCH No. 2, CONTINUOUS FULL HEIGHT.
SHEAR WALL DEFLECTION: Δ = 0.14 in

ANALYSIS

CHECK MAX SHEAR WALL DIMENSION RATIO L / B = 0.3 < 3.5 **[Satisfactory]**
DETERMINE REQUIRED CAPACITY v_b = 165 plf, (A Side Diaphragm Required, the Max. Nail Spacing = 6 in)

THE SHEAR CAPACITIES PER IBC Table 2306.3 / SDPWS-21 Table 4.3A with ASD reduction factor 2.0)

Panel Grade	Common Nail	Min. Penetration (in)	Min. Thickness (in)	Blocked Nail Spacing Boundary & All Edges			
				6	4	3	2
Sheathing and Single-Floor	10d	1 5/8	15/32	310	460	600	770

Use HDU2

Note: 1. The indicated shear numbers have reduced by specific gravity factor per IBC note a.
2. Since the wall is blocked, SDPW-15 Table 4.3.3.2 does not apply.

DETERMINE DRAG STRUT FORCE: F = $(L-L_w)$ MAX($v_{dia,\ WIND}$, $\Omega_0 v_{dia,\ SEISMIC}$) = 1.81 k (Ω_0 = 2.5) (Sec. 1633.2.6)

DETERMINE MAX SPACING OF 5/8" DIA (or 1/2" DIA) ANCHOR BOLT (NDS 2015, Tab.11E)
5/8 in DIA. x 10 in LONG ANCHOR BOLTS @ 48 in O.C. (or 1/2 in DIA. x 10 in LONG ANCHOR BOLTS @ 46 in O.C.)
THE HOLD-DOWN FORCES:

	v_{dia} (plf)	Wall Seismic at mid-story (lbs)	Overturning Moments (ft-lbs)		Resisting Moments (ft-lbs)	Safety Factors	Net Uplift (lbs)		Holddown SIMPSON
SEISMIC	145	592	63860	Left	54760	0.9	T_L =	394	HDU2-1/4x2.5
				Right	54760	0.9	T_R =	394	
WIND	0		0	Left	54760	2/3	T_L =	0	
				Right	54760	2/3	T_R =	0	

(T_L & T_R values should include upper level UPLIFT forces if applicable)

CHECK MAXIMUM SHEAR WALL DEFLECTION: (IBC Section 2305.3 / SDPWS-21 4.3.4)

$$\Delta = \Delta_{Bending} + \Delta_{Shear} + \Delta_{Nail\ slip} + \Delta_{Chord\ splice\ slip} = \frac{8 v_b h^3}{E A L_w} + \frac{v_b h}{G t} + 0.75 h e_n + \frac{h d_a}{L_w} = 0.141 \text{ in, ASD} < \delta_{xe,allowable,\ ASD} = 0.429 \text{ in}$$

Where: v_b = 165 plf , ASD; L_w = 37 ft; E = 1.7E+06 psi **[Satisfactory]** (ASCE 7-22 12.8.6)
A = 16.50 in^2; h = 10 ft; G = 9.0E+04 psi; C_d = 4; I = 1
t = 0.298 in; e_n = 0.009 in, SD; d_a = 0.15 in, SD ,(ASCE 7-22 Tab 12.2-1 & Tab 11.5-1)

$$\frac{v_b h}{1000 G_a} = \frac{v_b h}{G t} + 0.75 h e_n$$

C_M = 1.0 (NDS 4.1.4)
Δ_a = 0.02 h_{sx} , (ASCE 7-22 Tab 12.12-1)

CHECK EDGE STUD CAPACITY

P_{max} = 2.14 kips, (this value should include upper level DOWNWARD loads if applicable)

F_c = 1350 psi; C_D = 1.60; C_P = 0.16; A = 10.5 in^2
E = 1600 ksi; C_F = 1.15; F_c' = 391 psi > f_c = 204 psi

[Satisfactory]

Drag = 202×18 = 3636 # MST 48

Lobana Engineering Inc.
885 Patriot Dr. Unit G
Moorpark, CA 93021

PROJECT : GRID B
CLIENT :
JOB NO. :
DATE :
PAGE :
DESIGN BY :
REVIEW BY :

38

Shear Wall Design Based on 2021 IBC / 2022 CBC / SDPWS-21

INPUT DATA

LATERAL FORCE ON DIAPHRAGM: $v_{dia,\ WIND}$ = 0 plf,for wind, ASD

$v_{dia,\ SEISMIC}$ = 202 plf,for seismic, ASD

GRAVITY LOADS ON THE ROOF: w_{DL} = plf,for dead load

w_{LL} = 0 plf,for live load

DIMENSIONS: L_w = 8 ft , h = 10 ft

L = 42 ft , h_p = 0 ft

PANEL GRADE (0 or 1) = 1 <= Sheathing and Single-Floor

MINIMUM NOMINAL PANEL THICKNESS = 15/32 in

COMMON NAIL SIZE (0=6d, 1=8d, 2=10d) 2 10d

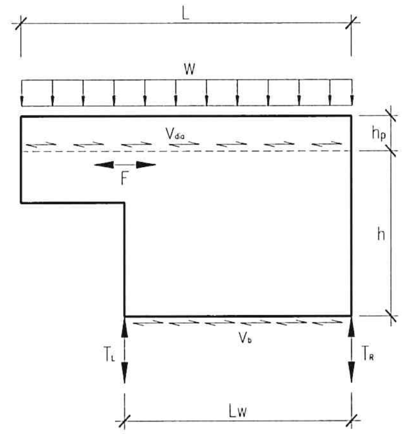
SPECIFIC GRAVITY OF FRAMING MEMBERS 0.5

EDGE STUD SECTION 1 pcs, b = 6 in , h = 6 in

SPECIES (1 = DFL, 2 = SP) 1 DOUGLAS FIR-LARCH

GRADE (1, 2, 3, 4, 5, or 6) 4 No. 1

STORY OPTION (1=ground level, 2=upper level) 1 ground level shear wall



THE SHEAR WALL DESIGN IS ADEQUATE.

DESIGN SUMMARY

BLOCKED 15/32 SHEATHING, EACH SIDE, WITH 10d COMMON NAILS
@ 3 in O.C. BOUNDARY & ALL EDGES / 12 in O.C. FIELD,
5/8 in DIA. x 10 in LONG ANCHOR BOLTS @ 10 in O.C. (or 1/2 in DIA. x 10 in LONG ANCHOR BOLTS @ 6 in O.C.)

HOLD-DOWN FORCES: T_L = 10.40 k , T_R = 10.40 k (USE SPECIAL SIMPSON HOLD-DOWN)

DRAG STRUT FORCES: F = 17.17 k

EDGE STUD: 1 - 6" x 6" DOUGLAS FIR-LARCH No. 1, CONTINUOUS FULL HEIGHT.

SHEAR WALL DEFLECTION: Δ = 0.54 in

CC

ANALYSIS

CHECK MAX SHEAR WALL DIMENSION RATIO L / B = 1.3 < 3.5 **[Satisfactory]**

DETERMINE REQUIRED CAPACITY v_b = 1061 plf, 2 Sides Diaphragm Required, the Max. Nail Spacing = 3 in)

THE SHEAR CAPACITIES PER IBC Table 2306.3 / SDPWS-21 Table 4.3A with ASD reduction factor 2.0)

Panel Grade	Common Nail	Min. Penetration (in)	Min. Thickness (in)	Blocked Nail Spacing Boundary & All Edges 6	4	3	2
Sheathing and Single-Floor	10d	1 5/8	15/32	310	460	600	770

Use HDU14 w/6×6

Note: 1. The indicated shear numbers have reduced by specific gravity factor per IBC note a.
2. Since the wall is blocked, SDPW-15 Table 4.3.3.2 does not apply.

DETERMINE DRAG STRUT FORCE: $F = (L-L_w)\ MAX(v_{dia,\ WIND},\ \Omega_0 v_{dia,\ SEISMIC})$ = 17.17 k (Ω_0 = 2.5) (Sec. 1633.2.6)

DETERMINE MAX SPACING OF 5/8" DIA (or 1/2" DIA) ANCHOR BOLT (NDS 2015, Tab.11E)
5/8 in DIA. x 10 in LONG ANCHOR BOLTS @ 10 in O.C. (or 1/2 in DIA. x 10 in LONG ANCHOR BOLTS @ 6 in O.C.)
THE HOLD-DOWN FORCES:

	v_{dia} (plf)	Wall Seismic at mid-story (lbs)	Overturning Moments (ft-lbs)		Resisting Moments (ft-lbs)	Safety Factors	Net Uplift (lbs)		Holddown SIMPSON
SEISMIC	202	128	85480	Left	2560	0.9	T_L =	10397	SPECIAL
				Right	2560	0.9	T_R =	10397	
WIND	0		0	Left	2560	2/3	T_L =	0	
				Right	2560	2/3	T_R =	0	

(T_L & T_R values should include upper level UPLIFT forces if applicable)

CHECK MAXIMUM SHEAR WALL DEFLECTION: (IBC Section 2305.3 / SDPWS-21 4.3.4)

$$\Delta = \Delta_{Bending} + \Delta_{Shear} + \Delta_{Nail\ slip} + \Delta_{Chord\ splice\ slip} = \frac{8 v_b h^3}{E A L_w} + \frac{v_b h}{G t} + 0.75 h e_n + \frac{h d_a}{L_w} = 0.537 \text{ in, ASD} <$$

$\delta_{xe,allowable,\ ASD}$ = 0.571 in

Where: v_b = 1061 plf , ASD L_w = 8 ft E = 1.7E+06 psi **[Satisfactory]** (ASCE 7-22 12.8.6)

A = 16.50 in^2 h = 10 ft G = 9.0E+04 psi C_d = 3 I = 1

t = 0.298 in e_n = 0.003 in, SD d_a = 0.10 in, SD ,(ASCE 7-22 Tab 12.2-1 & Tab 11.5-1)

C_M = 1.0 Δ_a = 0.02 h_{sx}

$$\frac{v_b h}{1000 G_a} = \frac{v_b h}{G t} + 0.75 h e_n$$

(NDS 4.1.4) , (ASCE 7-22 Tab 12.12-1)

CHECK EDGE STUD CAPACITY

P_{max} = 7.34 kips, (this value should include upper level DOWNWARD loads if applicable)

F_c = 1000 psi C_D = 1.60 C_P = 0.52 A = 30.25 in^2

E = 1600 ksi C_F = 1.00 F_c' = 826 psi > f_c = 243 psi

[Satisfactory]

GRID D P_D = 2475# < 2×1310 = 2620#

39

Use 2-HFX 12x9 D

MiTek
HARDY FRAME
Shear Wall Systems

Table 1.1A MiTek® Hardy Frame® Installation - on Concrete[1,2]

Model Number	Net Height H (in)	Concrete Compressive Strength f'_c (psi)	HD Bolt Dia (in) and Grade[3]	Applied Axial Load[4]	Seismic R=6.5, C_d=4.0: Allowable In-Plane Shear V^5 (lbs)	Seismic: Drift at V^5 (in)	Seismic: Uplift at $V^{5,6}$ (lbs)	Wind: Allowable In-Plane Shear V^5 (lbs)	Wind: Drift at V^5 (in)	Wind: Uplift at $V^{5,6}$ (lbs)
HFX-12x9	104 1/4	2,500	1 1/8" STD	1,000	1,310	0.248	19,595	1,310	0.248	19,595
				3,500	1,205	0.229	17,005	1,205	0.229	17,005
				6,500	1,080	0.205	14,325	1,080	0.205	14,325
			1 1/8" HS	1,000	1,310	0.250	19,595	1,310	0.250	19,595
				3,500	1,205	0.230	17,005	1,205	0.230	17,005
				6,500	1,080	0.206	14,325	1,080	0.206	14,325
		3,000	1 1/8" STD	1,000	1,475	0.280	21,065	1,495	0.284	21,575
				3,500				1,475	0.280	21,075
				6,500	1,355	0.257	18,375	1,355	0.257	18,375
			1 1/8" HS	1,000	1,575	0.301	23,750	1,575	0.301	23,750
				3,500	1,475	0.282	21,075	1,475	0.282	21,075
				6,500	1,355	0.259	18,375	1,355	0.258	18,375
		4,000	1 1/8" STD	1,000	1,475	0.280	18,515	1,655	0.314	21,615
				3,500						
				6,500						
			1 1/8" HS	1,000	1,680	0.321	22,085	2,115	0.404	32,065
				3,500				2,015	0.385	29,275
				6,500				1,890	0.361	26,380
HFX-15x9	104 1/4	2,500	1 1/8" STD	1,000	1,815	0.361	21,615	1,815	0.360	21,615
				3,500	1,800	0.359	21,380	1,800	0.357	21,380
				6,500	1,760	0.351	20,560	1,760	0.349	20,560
			1 1/8" HS	1,000	2,135	0.426	31,340	2,135	0.426	31,340
				3,500	2,000	0.399	26,150	2,000	0.399	26,150
				6,500	1,815	0.362	21,625	1,815	0.362	21,625
		3,000	1 1/8" STD	1,000	1,940	0.387	21,620	1,940	0.385	21,620
				3,500						
				6,500						
			1 1/8" HS	1,000	2,285	0.456	28,390	2,575	0.513	38,195
				3,500				2,440	0.487	32,600
				6,500	2,260	0.451	27,795	2,260	0.451	27,795
		4,000	1 1/8" STD	1,000	2,095	0.418	21,615	2,095	0.416	21,615
				3,500						
				6,500						
			1 1/8" HS	1,000	2,285	0.456	24,265	2,905	0.579	34,670
				3,500						
				6,500						
HFX-18x9	104 1/4	2,500	1 1/8" STD	1,000	2,435	0.256	21,615	2,435	0.256	21,615
				3,500						
				6,500						
			1 1/8" HS	1,000	3,310	0.350	39,500	3,310	0.350	39,500
				3,500	3,140	0.331	33,700	3,140	0.332	33,700
				6,500	2,905	0.307	28,745	2,905	0.307	28,745
		3,000	1 1/8" STD	1,000	2,450	0.258	20,405	2,560	0.269	21,620
				3,500						
				6,500						
			1 1/8" HS	1,000	3,760	0.397	40,260	3,915	0.414	44,955
				3,500				3,805	0.402	41,385
				6,500	3,595	0.379	36,500	3,595	0.380	36,500
		4,000	1 1/8" STD	1,000	2,450	0.258	19,105	2,715	0.286	21,620
				3,500						
				6,500						
			1 1/8" HS	1,000	3,760	0.397	32,880	4,210	0.445	38,865
				3,500						
				6,500						
HFX-21x9	104 1/4	2,500	1 1/8" STD	1,000	3,050	0.304	21,565	3,050	0.304	21,565
				3,500	3,020	0.300	21,255	3,020	0.300	21,255
				6,500	3,010	0.299	21,175	3,010	0.299	21,175
			1 1/8" HS	1,000	4,495	0.451	40,495	4,660	0.468	44,825
				3,500				4,520	0.454	41,070
				6,500	4,260	0.428	36,045	4,260	0.428	36,045
		3,000	1 1/8" STD	1,000	3,155	0.314	21,400	3,155	0.314	21,400
				3,500	3,115	0.310	21,070	3,115	0.310	21,070
				6,500	3,105	0.309	20,965	3,105	0.309	20,965
			1 1/8" HS	1,000	4,495	0.451	34,645	5,270	0.529	46,095
				3,500				5,195	0.522	44,690
				6,500				5,080	0.510	42,755
		4,000	1 1/8" STD	1,000	3,285	0.327	21,220	3,285	0.327	21,220
				3,500	3,240	0.322	20,865	3,240	0.322	20,865
				6,500	3,225	0.321	20,770	3,225	0.321	20,770
			1 1/8" HS	1,000	4,495	0.451	30,985	5,460	0.548	40,220
				3,500						
				6,500						

Capacity @ 15"W x 24"D Ftg = $\frac{15}{12} \times 1500 = 1875$ #/' (40)

CF-1 $w_1 = (15+20) \times 22/2 + (18+40) \times 22/2 + 15 \times 20 = 1323$ #/' < 1875 #/'

15"W x 24"D Ftg OK