BKG STRUCTURAL ENGINEERS

STRUCTURAL CALCULATIONS

The Sea Ranch Lodge

Loading Dock Extension

Sea Ranch, CA

Design Criteria

Building Information			
Floor Information			
Number of floor levels above grade:			1
Floor Level	Floor Height (ft)	Plan Dimension N-S (ft)	Plan Dimension E-W (ft)
Average Ridge Height	0.1	-	-
dock	3	20	20

Seismic			
Design Criteria			
Analysis Procedure	ELF Procedure		
Importance Factor	1		
Response Modification, R	6.5		
Redundany Factor, r	1.3		
Site Latitude	38.67000		
Site Longitude	123.42800		
Site Class	E		
Long-Period Transition, T_L (s)	12		
Spectral Accelerations			
S_S	2.285	S_1	0.954
S_{MS}	2.742	S_{M1}	N/A
S_{DS}	1.828	S_{D1}	N/A
Approximate Fundamental Period			
Ct	0.02		
x	0.75		
Ta (s)	0.047		
Seismic Response Coefficient			
Cs	0.281		
Cs, min	0.080		
Cs, max	N/A		
Governing C_s	0.281		

Wind			
Design Criteria			
Design Procedure Used	Directional Procedure		
Exposure Category	D		
Enclosure Classification	Enclosed		
Basic Wind Speed (mph)	110		
Directionality Factor, K_d	0.85		
Ground Elevation Factor, K_e	1.00		
Topographic Factor, K_{zt}	1.00		
Gust Effect Factor, G	0.85		
Velocity Pressure			
Floor	Avg Height (ft)	K_z	q_z (psf)
dock	2.3	1.03	27.13
Wind Pressure Coefficients			
Floor	Windward Cp	Leeward Cp N-S	Leeward Cp E-W
dock	0.8	-0.50	-0.50

USGS web services were down for some period of time and as a result this tool wasn't operational, resulting in *timeout* error.
USGS web services are now operational so this tool should work as expected.

OSHPD

60 Sea Walk Dr, Sea Ranch, CA 95497, USA

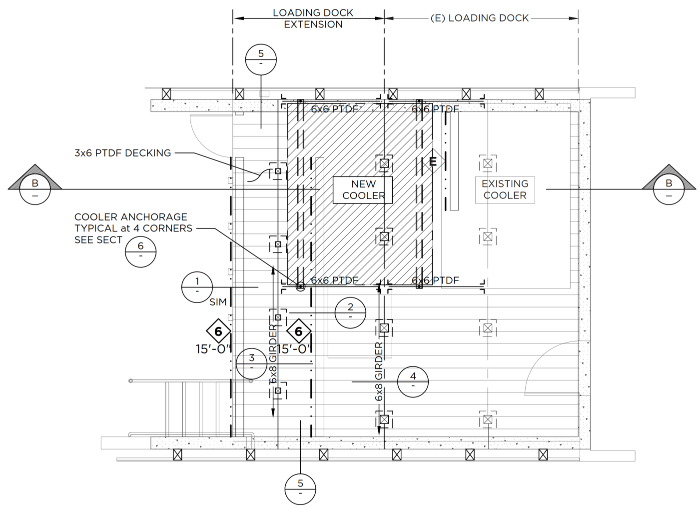
Latitude, Longitude: 38.6793597, -123.4275454

Date	4/11/2024, 7:32:33 AM
Design Code Reference Document	ASCE7-16
Risk Category	II
Site Class	D - Default (See Section 11.4.3)

Type	Value	Description
S_S	2.285	MCE_R ground motion. (for 0.2 second period)
S_1	0.954	MCE_R ground motion. (for 1.0s period)
S_{MS}	2.742	Site-modified spectral acceleration value
S_{M1}	null -See Section 11.4.8	Site-modified spectral acceleration value
S_{DS}	1.828	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	0.977	MCE_G peak ground acceleration
F_{PGA}	1.2	Site amplification factor at PGA
PGA_M	1.173	Site modified peak ground acceleration
T_L	12	Long-period transition period in seconds
SsRT	2.532	Probabilistic risk-targeted ground motion. (0.2 second)
SsUH	2.906	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration
SsD	2.285	Factored deterministic acceleration value. (0.2 second)
S1RT	1.072	Probabilistic risk-targeted ground motion. (1.0 second)
S1UH	1.237	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration.
S1D	0.954	Factored deterministic acceleration value. (1.0 second)
PGAd	0.977	Factored deterministic acceleration value. (Peak Ground Acceleration)
PGA_{UH}	1.162	Uniform-hazard (2% probability of exceedance in 50 years) Peak Ground Acceleration
C_{RS}	0.871	Mapped value of the risk coefficient at short periods

Loading Dock Lateral:



Length = 18.5 ft
Width = 8 ft
Weight = 5 psf

Area = 148 sf

Mass = 740 lbs

Cooler = 3500 lbs

Shear = 4240 lbs x .281 x .7
= 840 lbs

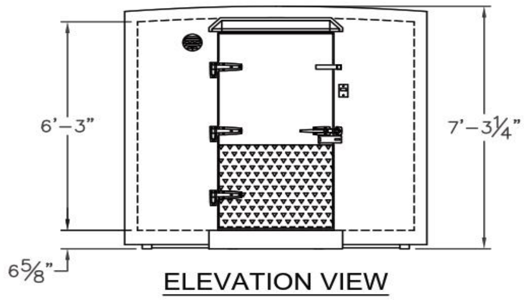
Shear walls = 420# / 15ft = 28 plf (10d @ 6"oc okay by inspection)
Holdows = 420 x 3ft / 14.5 ft = 86 lbs (no holdowns required)

Mechanical Anchorage

unit
cooler

operating weight (Wp):	
3800	lbs

physical dimensions		
L	96	in
W	84	in
H	87	in



center of gravity		
w'	42.0	in
h'	29.0	in

seismic design criterial			
ap	2.5	SDS	1.88
Rp	6	Ip	1

unit location		
z	3	ft
h	3	ft

Check Seismic:

check shear:

Fp = $\dfrac{.4 \times Ap \times SDS \times Wp\ (1 + 2\ z/h)}{Rp\ /\ Ip}$ = $\dfrac{.4 \times 2.5 \times 1.88 \times 3800 \times (1 + 2 \times 1)}{6\ /\ 1}$

Fp = 3572 lbs (lrfd) = 2500 lbs (asd)

point of connection	4

shear demand per connection = 625 lbs

check uplift:

vertial shear:

Ev = .2 x SDS x Wp = .2 x 1.88 x 3800

Ev = 1429 lbs (lrfd) = 1000 lbs (asd)

uplift demand per connection = 250 lbs

horizontal shear:

Mot = Fp x h' = 2500 x 29.0

Mot = 72512 lb-in (asd)

M res = (Wp x .6) x w' = 2280 x 42.0

M res = 95760 lb-in (asd)

Uplift = Mot - Mres / W = $\dfrac{72512 - 95760}{84}$

Uplift = -277 lb (asd)

uplift demand per connection = -138 lbs — no uplift due to vertial shear

max uplift demand per connection = 250 lbs

use (4) - 5/8" diamter all-thread
per manufacturer

Wood Beam

Project File: The Sea Ranch Lodge.ec6

LIC# : KW-06015204, Build:20.23.08.30 BKG Structural Engineers (c) ENERCALC INC 1983-2023

DESCRIPTION: 3x decking

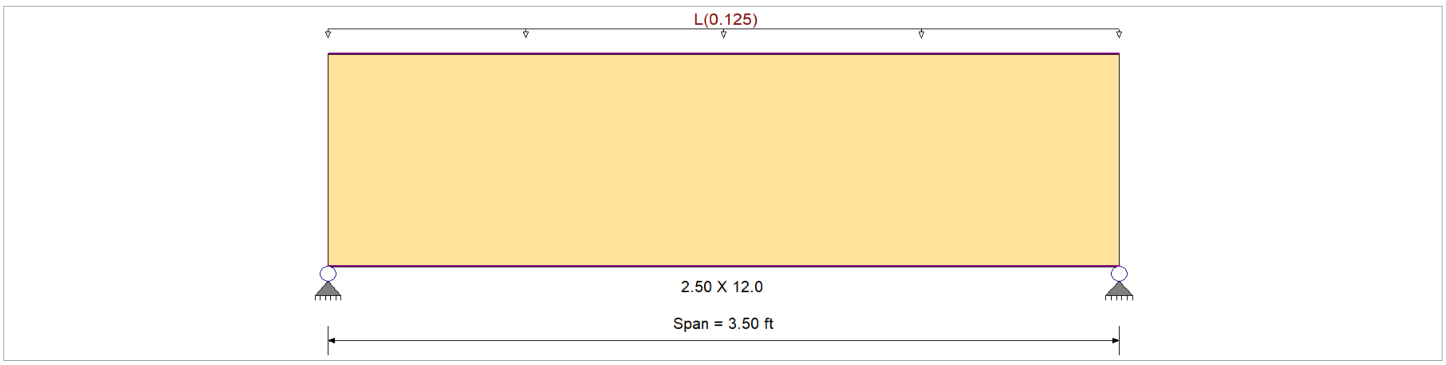
CODE REFERENCES

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

Load Combination Set : IBC 2021

Material Properties

Analysis Method :	Allowable Stress Design	Fb +	900 psi	*E : Modulus of Elasticity*		
Load Combination :	IBC 2021	Fb -	900 psi	Ebend- xx	1600 ksi	
		Fc - Prll	1350 psi	Eminbend - xx	580 ksi	
Wood Species :	Douglas Fir-Larch	Fc - Perp	625 psi			
Wood Grade :	No.2	Fv	180 psi			
		Ft	575 psi	Density	31.21 pcf	
Beam Bracing :	Beam is Fully Braced against lateral-torsional buckling			Repetitive Member Stress Increase		



Applied Loads

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

Beam self weight calculated and added to loading

Uniform Load : L = 0.1250 ksf, Tributary Width = 1.0 ft, (Deck Joists)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.233** : 1	Maximum Shear Stress Ratio	=	**0.034** : 1
Section used for this span		**2.50 X 12.0**	Section used for this span		**2.50 X 12.0**
fb: Actual	=	193.31 psi	fv: Actual	=	4.96 psi
F'b	=	828.00 psi	F'v	=	144.00 psi
Load Combination		+D+L	Load Combination		+D+L
Location of maximum on span	=	1.750 ft	Location of maximum on span	=	2.504 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.018 in	Ratio =	2349	>=360	Span: 1 : L Only
Max Upward Transient Deflection	0 in	Ratio =	0	<360	n/a
Max Downward Total Deflection	0.019 in	Ratio =	2233	>=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 = 3.50 ft	1	0.013	0.002	0.90	1.00	1.00	1.00	1.000	1.00	0.80	1.15	0.01	9.6	745.2	0.00	0.2	129.6
+D+L					1.00	1.00	1.00	1.000	1.00	0.80	1.15			0.0	0.00	0.0	0.0
Length = 3.50 ft	1	0.233	0.034	1.00	1.00	1.00	1.00	1.000	1.00	0.80	1.15	0.20	193.3	828.0	0.10	5.0	144.0
+D+0.750L					1.00	1.00	1.00	1.000	1.00	0.80	1.15			0.0	0.00	0.0	0.0
Length = 3.50 ft	1	0.142	0.021	1.25	1.00	1.00	1.00	1.000	1.00	0.80	1.15	0.15	147.4	1,035.0	0.08	3.8	180.0
+0.60D					1.00	1.00	1.00	1.000	1.00	0.80	1.15			0.0	0.00	0.0	0.0
Length = 3.50 ft	1	0.004	0.001	1.60	1.00	1.00	1.00	1.000	1.00	0.80	1.15	0.01	5.7	1,324.8	0.00	0.1	230.4

Wood Beam

Project File: The Sea Ranch Lodge.ec6

LIC# : KW-06015204, Build:20.23.08.30 BKG Structural Engineers (c) ENERCALC INC 1983-2023

DESCRIPTION: 6x girder

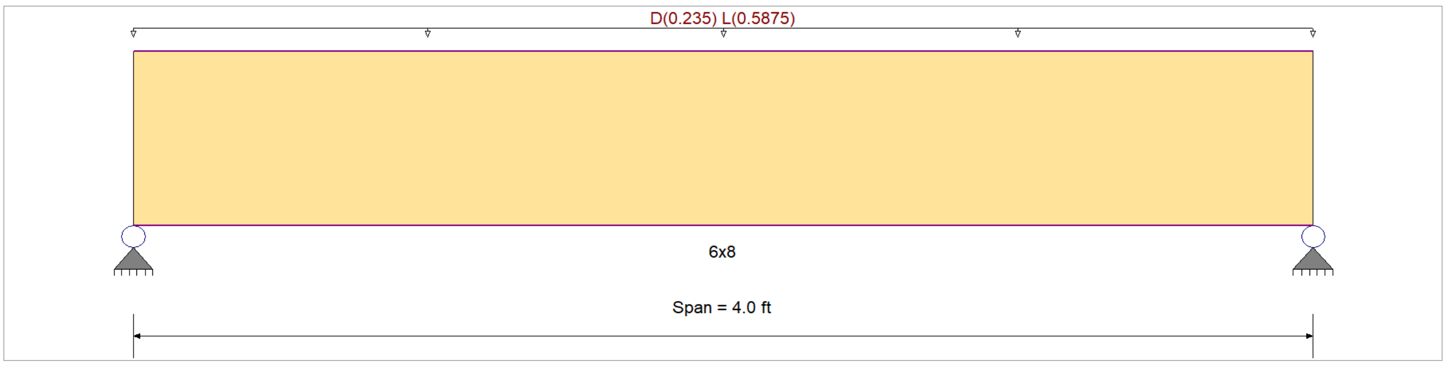
CODE REFERENCES

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

Load Combination Set : IBC 2021

Material Properties

Analysis Method :	Allowable Stress Design	Fb +	900 psi	*E : Modulus of Elasticity*	
Load Combination :	IBC 2021	Fb -	900 psi	Ebend- xx	1600 ksi
		Fc - Prll	1350 psi	Eminbend - xx	580 ksi
Wood Species :	Douglas Fir-Larch	Fc - Perp	625 psi		
Wood Grade :	No.2	Fv	180 psi		
		Ft	575 psi	Density	31.21 pcf
Beam Bracing :	Beam is Fully Braced against lateral-torsional buckling				



Applied Loads

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

Beam self weight calculated and added to loading

Uniform Load : D = 0.050, L = 0.1250 ksf, Tributary Width = 4.70 ft, (loading dock)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.537**: 1	Maximum Shear Stress Ratio	=	**0.291** : 1
Section used for this span		**6x8**	Section used for this span		**6x8**
fb: Actual	=	387.00 psi	fv: Actual	=	41.93 psi
F'b	=	720.00 psi	F'v	=	144.00 psi
Load Combination		+D+L	Load Combination		+D+L
Location of maximum on span	=	2.000 ft	Location of maximum on span	=	3.387 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.012 in	Ratio =	4144 >=360	Span: 1 : L Only
Max Upward Transient Deflection	0 in	Ratio =	0 <360	n/a
Max Downward Total Deflection	0.016 in	Ratio =	2928 >=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 = 4.0 ft	1	0.175	0.095	0.90	1.00	1.00	1.00	1.000	1.00	0.80	1.00	0.49	113.5	648.0	0.34	12.3	129.6
+D+L					1.00	1.00	1.00	1.000	1.00	0.80	1.00			0.0	0.00	0.0	0.0
Length = 4.0 ft	1	0.537	0.291	1.00	1.00	1.00	1.00	1.000	1.00	0.80	1.00	1.66	387.0	720.0	1.15	41.9	144.0
+D+0.750L					1.00	1.00	1.00	1.000	1.00	0.80	1.00			0.0	0.00	0.0	0.0
Length = 4.0 ft	1	0.354	0.192	1.25	1.00	1.00	1.00	1.000	1.00	0.80	1.00	1.37	318.6	900.0	0.95	34.5	180.0
+0.60D					1.00	1.00	1.00	1.000	1.00	0.80	1.00			0.0	0.00	0.0	0.0
Length = 4.0 ft	1	0.059	0.032	1.60	1.00	1.00	1.00	1.000	1.00	0.80	1.00	0.29	68.1	1,152.0	0.20	7.4	230.4

DESCRIPTION: 6x girder

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+L	1	0.0164	2.015		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	1.663	1.663
Max Upward from Load Combinations	1.663	1.663
Max Upward from Load Cases	1.175	1.175
D Only	0.488	0.488
+D+L	1.663	1.663
+D+0.750L	1.369	1.369
+0.60D	0.293	0.293
L Only	1.175	1.175

Check Footing:

Rmax = 1.663 x2 = 3.326k

Sp allow = 1.5 ksf

Min Ftg Size = (3.326 / 1.5)^.5 = 18" square