



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 dock	0.1 - 3	- 20	- 20



Seismic			
Design Criteria			
Analysis Procedure	ELF		
Importance Factor	Procedure		
Response Modification, R	1		
Redundancy Factor, r	6.5		
Site Latitude	1.3		
Site Longitude	38.67000		
Site Class	123.42800		
Long-Period Transition, T_L (s)	E		
	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			
C_t	0.02		
x	0.75		
T_a (s)	0.047		
Seismic Response Coefficient			
C_s	0.281		
$C_{s, \min}$	0.080		
$C_{s, \max}$	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			
	Avg Height		
Floor	(ft)	K_z	q_z (psf)
dock	2.3	1.03	27.13
Wind Pressure Coefficients			
	Windward C_p	Leeward C_p N-S	Leeward C_p 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.



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

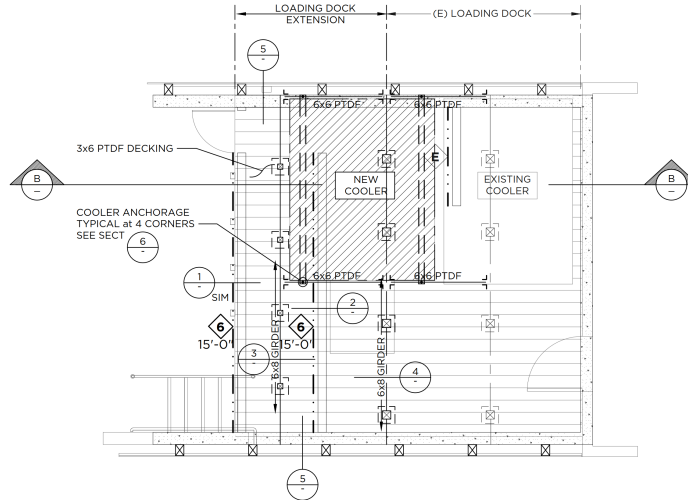
Weight = 5 psf

Area = 148 sf

Mass = 740 lbs

Cooler = 3500 lbs

Shear = $4240 \text{ lbs} \times .281 \times .7$
= 840 lbs



Shear walls = $420\# / 15\text{ft} = 28 \text{ plf}$ (10d @ 6"oc okay by inspection)

Holdowns = $420 \times 3\text{ft} / 14.5 \text{ ft} = 86 \text{ lbs}$ (no holdowns required)

Mechanical Anchorage

unit
cooler

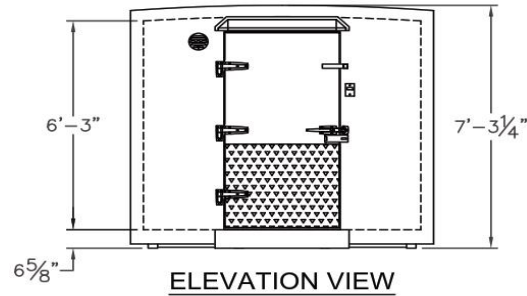
operating weight (Wp):
3800 lbs

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:

$$F_p = \frac{.4 \times A_p \times SDS \times W_p (1 + 2 z/h)}{R_p / I_p} = \frac{.4 \times 2.5 \times 1.88 \times 3800 \times (1 + 2 \times \frac{1}{1})}{6 / 1}$$

$$F_p = 3572 \text{ lbs (lrfd)} = 2500 \text{ lbs (asd)}$$

point of connection	4
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shear demand per connection =	625 lbs
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check uplift:

vertial shear:

$$E_v = .2 \times SDS \times W_p = .2 \times 1.88 \times 3800$$

$$E_v = 1429 \text{ lbs (lrfd)} = 1000 \text{ lbs (asd)}$$

$$\text{uplift demand per connection} = 250 \text{ lbs}$$

horizontal shear:

$$M_{ot} = F_p \times h' = 2500 \times 29.0$$

$$M_{ot} = 72512 \text{ lb-in (asd)}$$

$$M_{res} = (W_p \times .6) \times w' = 2280 \times 42.0$$

$$M_{res} = 95760 \text{ lb-in (asd)}$$

$$\text{Uplift} = \frac{M_{ot} - M_{res}}{W} = \frac{72512 - 95760}{84}$$

$$\text{Uplift} = -277 \text{ lb (asd)}$$

$$\text{uplift demand per connection} = -138 \text{ lbs} \quad \text{no uplift due to vertial shear}$$

max uplift demand per connection =	250 lbs
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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

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DESCRIPTION: 3x decking

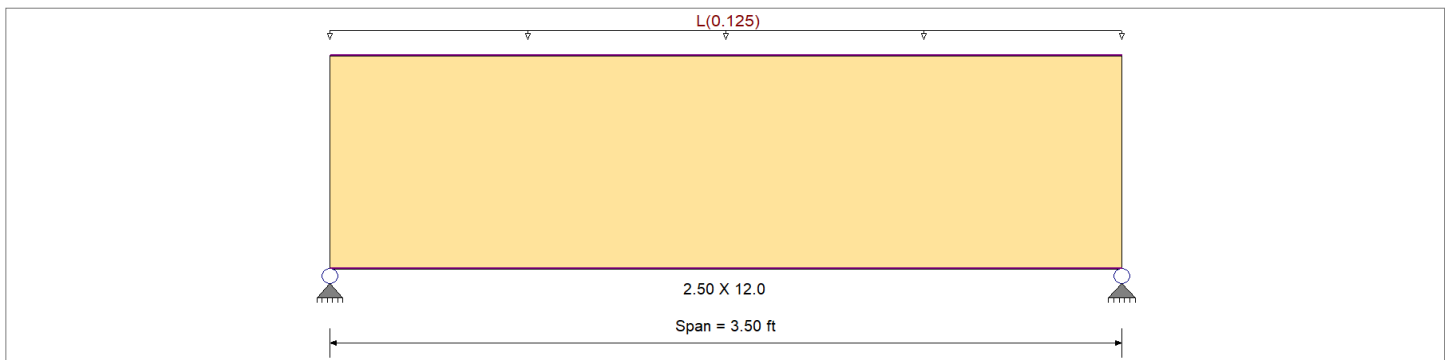
CODE REFERENCES

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				Maximum Shear Stress Ratio			
Section used for this span	=	0.233	1	Section used for this span	=	0.034	1
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	C _{fu}	C _i	C _r	M	fb	F'b	V	fv	F'v	
D Only															0.0	0.00	0.0	0.0
Length = 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															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															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															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

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LIC# : KW-06015204, Build:20.23.08.30

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DESCRIPTION: 6x girder

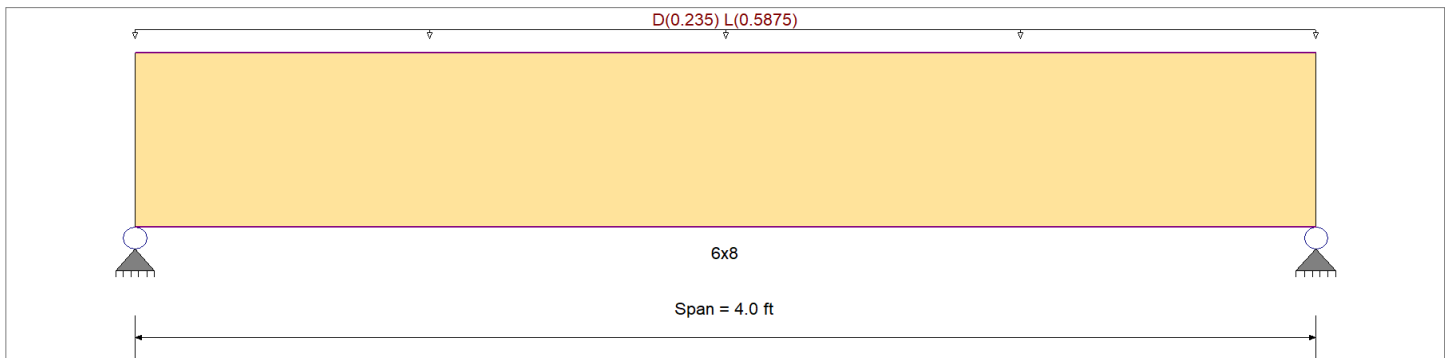
CODE REFERENCES

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			=	387.00psi		fv: Actual			=	41.93 psi	
F'b			=	720.00psi		F'v			=	144.00 psi	
Load Combination				+D+L		Load Combination				+D+L	
Location of maximum on span			=	2.000ft		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	C _{fu}	C _i	C _r	M	fb	F'b	V	fv	F'v	
D Only															0.0	0.00	0.0	0.0
Length = 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															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															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															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	

Wood Beam

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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:

$$R_{max} = 1.663 \times 2 = 3.326k$$

$$Sp \text{ allow} = 1.5 \text{ ksf}$$

$$\text{Min Ftg Size} = (3.326 / 1.5)^{.5} = 18" \text{ square}$$