Structural Engineering Calculations

THESE ATTACHMENTS ARE PART
OF THE APPROVED PLANS.
*DO NOT REMOVE THEM *

FEB 05 2020

PERMIT AND RESOURCE
MANAGEMENT DEPARTMENT
BUILDING PLAN CHECK
PERMIT # BLD19-3421

SEA RANCH LODGE RESTAURANT

60 SEA WALK DRIVE
SEA RANCH, CALIFORNIA 95497

SE PROJECT NUMBER 1977

3RD SUBMITTAL
1/15/20

REGISTERED PROFESSIONAL ENGINEER
DAVID M. STRANDBERG
No. 62993
Exp. 6-30-20
CIVIL
STATE OF CALIFORNIA

PERMIT SUBMITTAL **OCTOBER 30, 2019**
PERMIT RESUBMITTAL **JANUARY 13, 2020**

415 | 7788726

1511 15TH STREET SAN FRANCISCO CALIFORNIA 94103
STRANDBERG ENGINEERING

Strandberg Engineering
1511 15th Street
San Francisco, CA 94103

Project Title: The Sea Ranch Lodge Restaurant
Engineer: CVF
Project ID: 1977
Project Descr: Gravity Design

Printed: 9 JAN 2020, 3:43PM

Wood Beam

File = P:\2019\1977 - The Sea Ranch Lodge Restaurant\CALCS\GRAVITY\The Sea Ranch Lodge Restaurant.ec6

Lic. # : KW-06009955 STRANDBERG ENGINEERING

DESCRIPTION: 1B-5: CASE 2 - With Seismic Point Load from SW at GL 2

Load Combination / Segment Length	Span #	Max Stress Ratios M	V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	Moment Values M	fb	F'b	Shear Values V	fv	F'v
Length = 9.50 ft	1	0.024	0.016	1.60	1.000	1.00	1.00	1.00	1.00	0.98	0.97	104.87	4379.11	0.29	7.47	456.00
+0.4026D+1.750E					1.000	1.00	1.00	1.00	1.00	0.98			0.00	0.00	0.00	0.00
Length = 9.50 ft	1	0.520	0.336	1.60	1.000	1.00	1.00	1.00	1.00	0.98	21.01	2,276.34	4379.11	6.03	153.11	456.00

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	-3.537	-2.063
Overall MINimum	1.263	0.842
D Only	0.523	0.376
+D+L	1.787	1.218
+D+0.750L	1.471	1.008
+D+0.70E	-1.952	-1.068
+D+0.750L+0.5250E	-0.386	-0.075
+0.60D	0.314	0.226
+0.60D+0.70E	-2.162	$\boxed{-1.219}$
L Only	1.263	0.842
E Only	-3.537	-2.063

THE HIGHEST ASD UPLIFT LOAD IS USED FOR HOLDOWN DESIGN

1B-27.3 3

Strandberg Engineering
1511 15th Street
San Francisco, CA 94103

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Wood Beam

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Lic. # : KW-06009955 STRANDBERG ENGINEERING

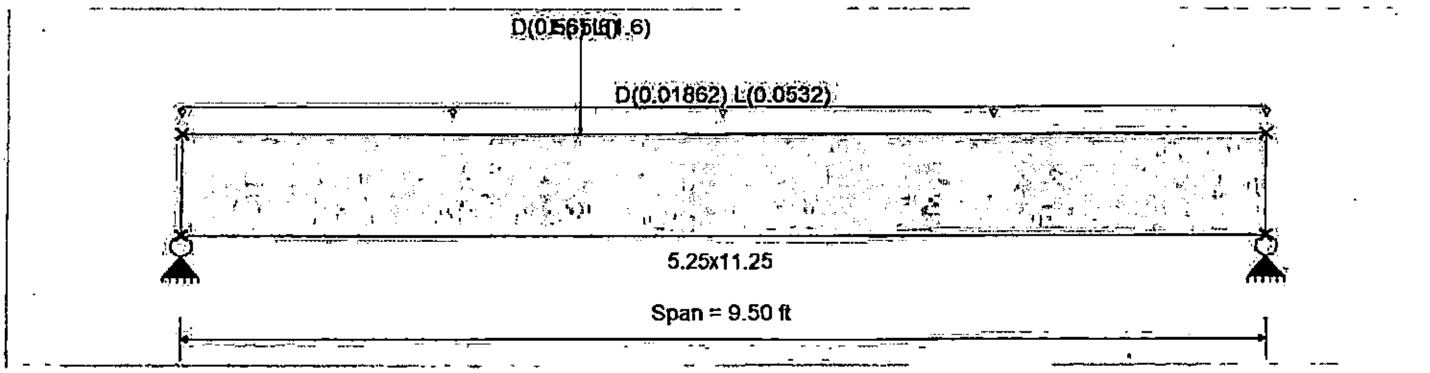
DESCRIPTION: 1B-5: CASE 2 - With Seismic Point Load from SW at GL 2

CODE REFERENCES

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10
Load Combination Set : IBC 2015

Material Properties

Analysis Method :	Allowable Stress Design	Fb +	2,800.0 psi	*E : Modulus of Elasticity*	
Load Combination	IBC 2015	Fb -	2,800.0 psi	Ebend- xx	2,000.0 ksi
		Fc - Prll	3,000.0 psi	Eminbend - xx	1,036.83 ksi
Wood Species	: Boise Cascade	Fc - Perp	750.0 psi		
Wood Grade	: Versa Lam 2.0 2800 West	Fv	285.0 psi		
		Ft	1,950.0 psi	Density	41.760 pcf
Beam Bracing	: Completely Unbraced				



Applied Loads

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

Beam self weight calculated and added to loads
Uniform Load : D = 0.0140, L = 0.040 ksf, Tributary Width = 1.330 ft, (w: Floor Loading)
Point Load : E = -5.60 k @ 3.50 ft, (P2, Seismic)
Point Load : D = 0.560, L = 1.60 k @ 3.50 ft, (P1)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.520 1**	Maximum Shear Stress Ratio	=	**0.336 : 1**
Section used for this span		**5.25x11.25**	Section used for this span		**5.25x11.25**
	=	2,276.34 psi		=	153.11 psi
	=	4,379.11 psi		=	456.00 psi
Load Combination		+0.4026D+1.750E	Load Combination		+0.4026D+1.750E
Location of maximum on span	=	3.502ft	Location of maximum on span	=	3.467 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.044 in	Ratio =	2580 >=360
Max Upward Transient Deflection	-0.127 in	Ratio =	896 >=360
Max Downward Total Deflection	0.062 in	Ratio =	1833 >=240
Max Upward Total Deflection	-0.078 in	Ratio =	1456 >=240

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios									Moment Values			Shear Values		
Segment Length	Span #	M	V	C_d	C_{FN}	C_i	C_r	C_m	C_t	C_L	M	fb	F'b	V	fv	F'v
D Only													0.00	0.00	0.00	0.00
Length = 9.50 ft	1	0.070	0.049	0.90	1.000	1.00	1.00	1.00	1.00	0.99	1.61	174.78	2492.72	0.49	12.45	256.50
+D+L					1.000	1.00	1.00	1.00	1.00	0.99			0.00	0.00	0.00	0.00
Length = 9.50 ft	1	0.224	0.152	1.00	1.000	1.00	1.00	1.00	1.00	0.99	5.71	618.46	2765.60	1.70	43.26	285.00
+D+0.750L					1.000	1.00	1.00	1.00	1.00	0.99			0.00	0.00	0.00	0.00
Length = 9.50 ft	1	0.147	0.100	1.25	1.000	1.00	1.00	1.00	1.00	0.98	4.68	507.54	3443.22	1.40	35.56	356.25
+1.197D+1.750E					1.000	1.00	1.00	1.00	1.00	0.98			0.00	0.00	0.00	0.00
Length = 9.50 ft	1	0.488	0.318	1.60	1.000	1.00	1.00	1.00	1.00	0.98	19.73	2,137.42	4379.11	5.71	145.04	456.00
+1.148D+0.750L+1.313E					1.000	1.00	1.00	1.00	1.00	0.98			0.00	0.00	0.00	0.00
Length = 9.50 ft	1	0.280	0.188	1.60	1.000	1.00	1.00	1.00	1.00	0.98	11.32	1,226.61	4379.11	3.37	85.70	456.00
+0.60D					1.000	1.00	1.00	1.00	1.00	0.98			0.00	0.00	0.00	0.00

Note: Enercalc applies overstrength factor by multiplying it to all "E" load combo values, typ.

(5-27.2) △

Wood Beam File = P:\2019\1977 - The Sea Ranch Lodge Restaurant\CALCS\GRAVITY\The Sea Ranch Lodge Restaurant.ec6
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Lic. # : KW-06009955 STRANDBERG ENGINEERING

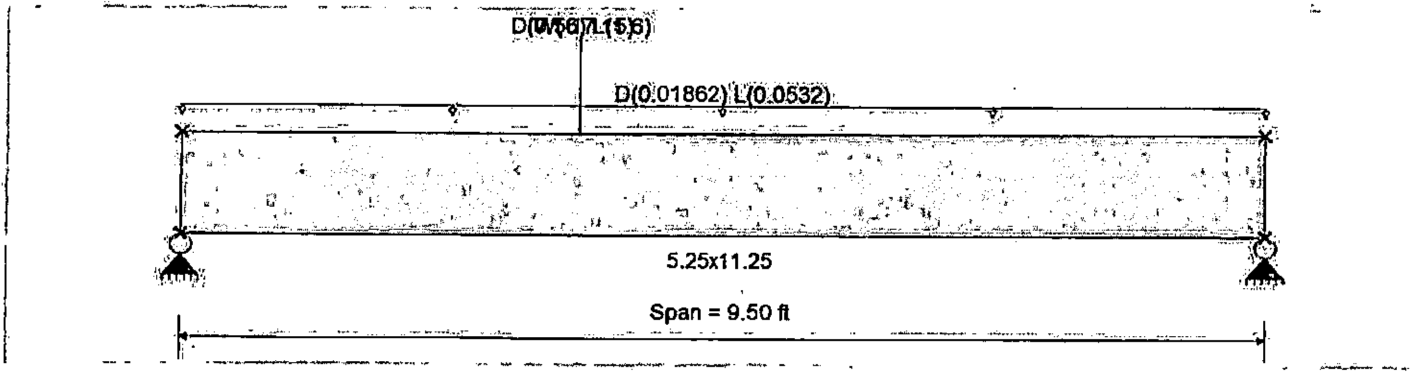
DESCRIPTION: 1B-5: CASE 1 - With Wind Point Load from SW at GL 2

CODE REFERENCES

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10
Load Combination Set : IBC 2015

Material Properties

Analysis Method :	Allowable Stress Design	Fb +	2,800.0 psi	*E : Modulus of Elasticity*	
Load Combination	IBC 2015	Fb -	2,800.0 psi	Ebend- xx	2,000.0 ksi
		Fc - Prll	3,000.0 psi	Eminbend - xx	1,036.83 ksi
Wood Species	: Boise Cascade	Fc - Perp	750.0 psi		
Wood Grade	: Versa Lam 2.0 2800 West	Fv	285.0 psi		
		Ft	1,950.0 psi	Density	41.760 pcf
Beam Bracing	: Completely Unbraced				



Applied Loads

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

Beam self weight calculated and added to loads
Uniform Load : D = 0.0140, L = 0.040 ksf, Tributary Width = 1.330 ft, (w: Floor Loading)
Point Load : W = -17.150 k @ 3.50 ft, (P2: Wind Load)
Point Load : D = 0.560, L = 1.60 k @ 3.50 ft, (P1)

DESIGN SUMMARY

Design N.G.

Maximum Bending Stress Ratio	=	**0.539 : 1**	Maximum Shear Stress Ratio	=	**0.349 : 1**
Section used for this span		**5.25x11.25**	Section used for this span		**5.25x11.25**
	=	2,359.17 psi		=	158.96 psi
	=	4,379.11 psi		=	456.00 psi
Load Combination		+0.60D+0.60W	Load Combination		+0.60D+0.60W
Location of maximum on span	=	3.502 ft	Location of maximum on span	=	3.467 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.044 in	Ratio =	2580 >=360
Max Upward Transient Deflection	-0.390 in	Ratio =	292 <360
Max Downward Total Deflection	0.062 in	Ratio =	1833 >=240
Max Upward Total Deflection	-0.223 in	Ratio =	511 >=240

PER "CBC 2015" TABLE 1604.3 "DEFLECTION LIMITS", THERE IS NO WIND. DEFLECTION LIMIT FOR FLOOR MEMBERS. THIS UPWARD TRANSIENT DEFLECTION IS A WIND DEFLECTION, SO DEFLECTION DESIGN IS **OKAY.**

Vertical Reactions

Support notation : Far left is #1

Load Combination	Support 1	Support 2
Overall MAXimum	-10.832	-6.318
Overall MINimum	1.263	0.842
D Only	0.523	0.376
+D+L	1.787	1.218
+D+0.750L	1.471	1.008
+D+0.60W	-5.975	-3.415
+D+0.750L+0.450W	-3.403	-1.836
+0.60D+0.60W	-6.185	$\boxed{-3.565}$
+0.60D	0.314	0.226
L Only	1.263	0.842
W Only	-10.832	-6.318

THE HIGHEST ASD UPLIFT LOAD IS USED FOR HOLDOWN DESIGN

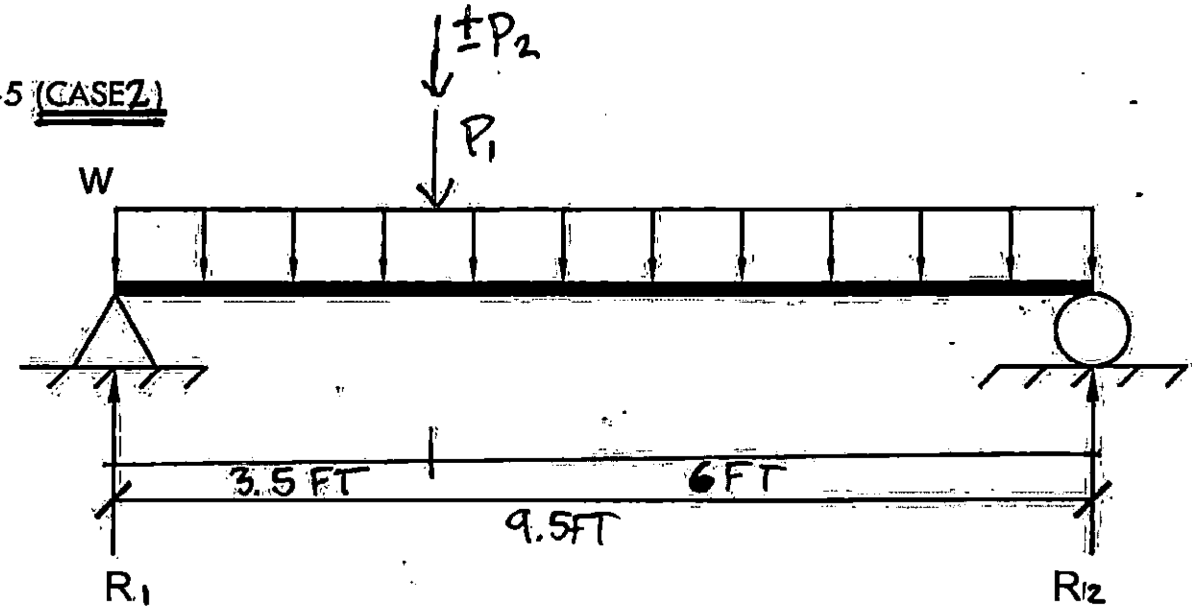
G-27.1 3

PROJECT: The Sea Ranch Lodge Restaurant DATE: 01/2020

JOB NUMBER: 1977 ENGINEER: CVF

GRAVITY DESIGN

1B-5 (CASE 2)



Loading:

i) w: Lower Floor
DL = 14 psf
RL = 40 psf
tw = 1.33 ft

ii) P1: Pont Load from perpendicular beam
A,trib = 40 ft^2
P,DL = 14 psf X A,trib = 560 lbs.
P,LL = 40 psf X A,trib = 1,600 lbs.

iii). P_2: SEISMIC LOADING, PER CALC SHEET L-12.1 thru L12.6

$$P_{2,SEISMIC} = \frac{4,960 \#}{0.7 \times 1.3} \simeq 5,600 \#$$

SEE ENERCALC ATTACHED.

USE 5 1/4"x 11 1/4" VERSA-LAM 2800

STRANDBERG ENGINEERING

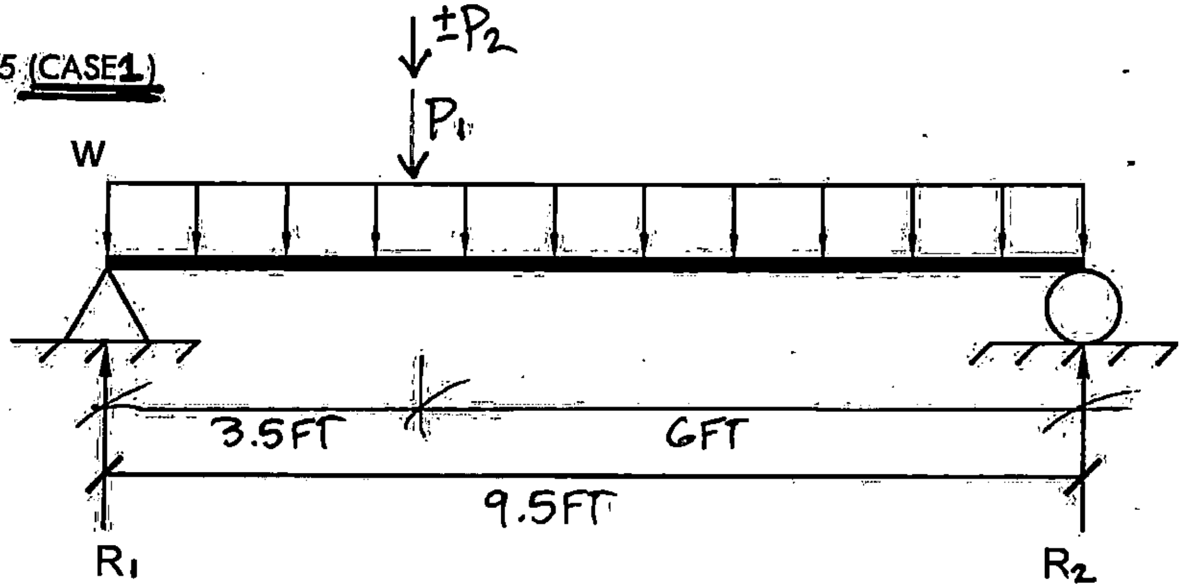
G-26.2 3

PROJECT: The Sea Ranch Lodge Restaurant DATE: 01/2020

JOB NUMBER: 1977 ENGINEER: CVF

GRAVITY DESIGN

1B-5 (CASE 1)



Loading:

i) w: Lower Floor
DL = 14 psf
RL = 40 psf
tw = 1.33 ft

ii) P1: Pont Load from perpendicular beam
A,trib = 40 ft^2
P,DL = 14 psf X A,trib = 560 lbs.
P,LL = 40 psf X A,trib = 1,600 lbs.

iii) P_2: WIND LOADING PER CALC SHEET L-16.1

$P_{2,WIND} = \frac{10,285\#}{0.6} = 17,150\#$

SEE ENERCALC ATTACHED.

USE 5 1/4" x 11 1/4" VERSA-LAM 2800

STRANDBERG ENGINEERING

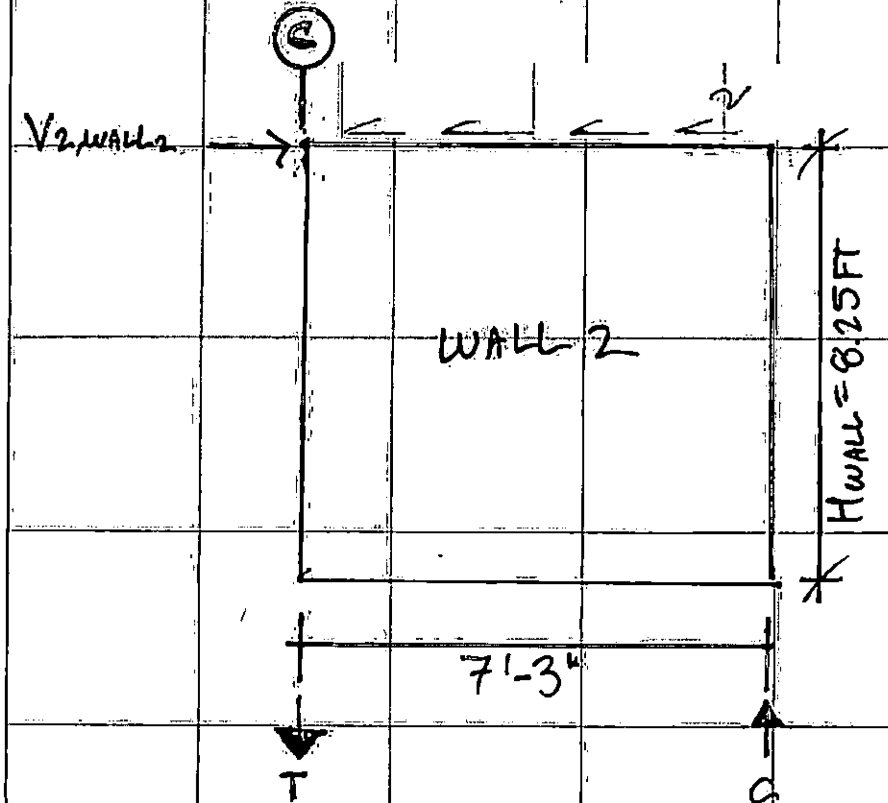
PROJECT: SEA RANCH DATE: 1/20

JOB NUMBER: 1977 ENGINEER: CVF

SHEARWALL DESIGN CONT'D. (WIND)

↳ LONGER SHEARWALL (7.25 FT SHEARWALL)

HOLDOWN DESIGN DIAGRAM



DEMAND

$$v = \frac{V_2}{\Sigma(L_i)} = \frac{14{,}239\#}{6' + 7.25'} = 1{,}074 \text{ plf}$$

$$V_{2,WALL2} = v \times 7.25\text{FT}$$
$$= 1{,}074 \text{ plf} \times 7.25\text{FT}$$
$$= 7{,}790\#$$

$$T = \frac{V_{2,WALL2} \times H_{WALL}}{7.25\text{FT}}$$
$$= \frac{7{,}790\# \times 8.25\text{FT}}{7.25\text{FT} - 1\text{FT}} = \underline{\underline{10{,}285\#}}$$

GRID: 1/8"

STRANDBERG ENGINEERING

PROJECT: SEA RANCH

DATE: 1/20

JOB NUMBER: 1977

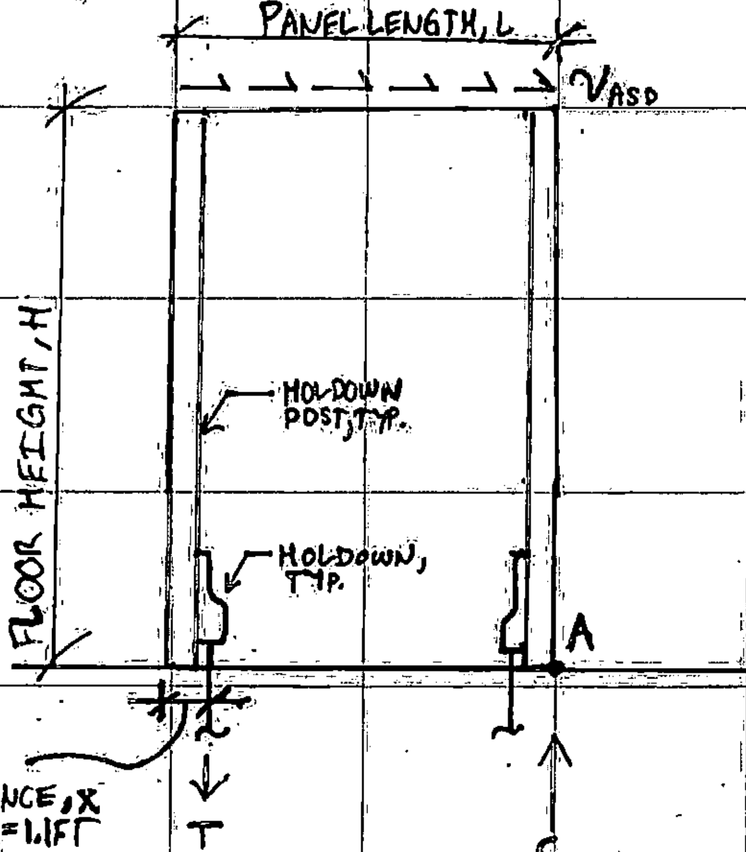
ENGINEER: CVF

SEISMIC

UPLIFT FOR S.WALLS @ G.L. 2

↳ WALL LENGTH = 7.25 FT. ONLY

LOADING DIAGRAM



LOADS

↳ SEE CALCULATION SHEETS L12.1 thru L12.3 FOR VALUES BELOW

V_{ASD} = 510 PLF

H = 8.25 FT ; 6FT WAS NOT RECALCULATED (SEE PREVIOUS SHEET FOR THIS)

PANEL LENGTH, L = 7.25 FT

HOLDOWN EDGE DIST. = 1.1 FT

SOLVE FOR T BY $\Sigma M_A = 0$

$\circlearrowleft + \Sigma M_A = (V_{ASD} \times \text{PANEL LENGTH}) \times H - T(L - \text{HOLDOWN EDGE DIST}) = 0$

$$T = \frac{(V_{ASD} \times L) \times H}{(L - \text{HOLDOWN EDGE DIST})} = \frac{(510\ \text{PLF} \times 7.25\ \text{FT}) \times 8.25\ \text{FT}}{(7.25\ \text{FT} - 1.1\ \text{FT})}$$

$= \underline{4{,}960\ \#\ \text{(TENSION)}}$

STRANDBERG ENGINEERING

GRID: 1/8"

PROJECT: The Sea Ranch Lodge Restaurant

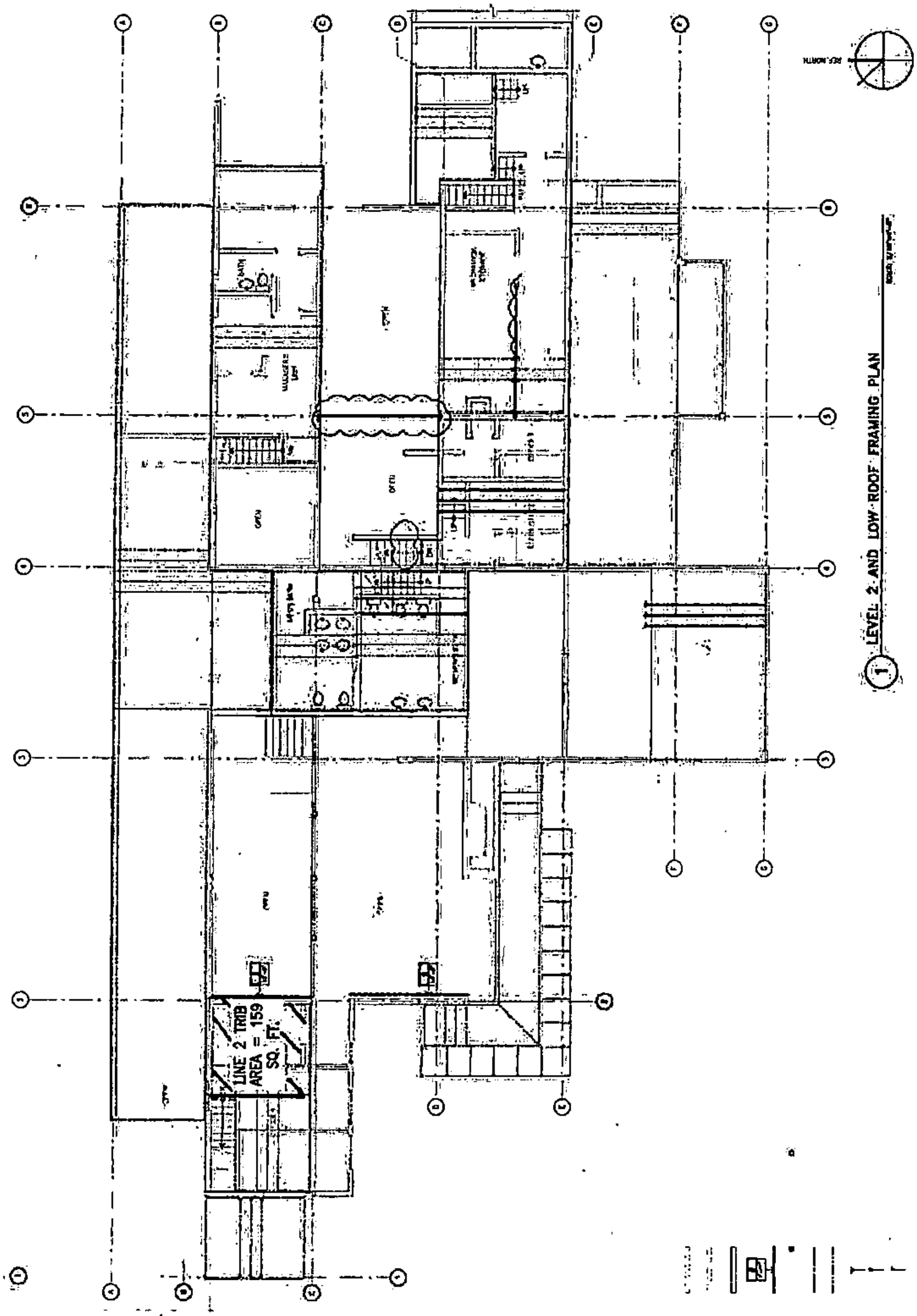
DATE: 10.18.2019

JOB NO: 1977

ENGINEER: CVF

LATERAL KEY (N/S DIRECTION) - S2.1

Level 2



STRANDBERG ENGINEERING

L12.5 3

PROJECT: Sea Ranch Lodge Restaurant

DATE: 01/08/20

JOB NO: 1977

ENGINEER: CVF

SEISMIC DESIGN @ LEVEL 2 FLOOR

Shear Wall Design:

Design Method: ASD
Sheathing Type: Struct. 1

This SEISMIC spreadsheet is used compare the holdown load to WIND holdown load at LEVEL 2 Line 2@ BEAM 1B-5

Line of Resistance	Unit Shear, v_{ASD} $(Q_E \times 0.7\rho)/\Sigma L$	Capacity Reduction	Type	Wood SW DCR	Uplift (lbs)	Holdown	Net Uplift (lbs)	Holdown	Shared with lines...	Combined Uplifts (lbs)	Holdown	Notes
A												
B												

Line of Resistance	Unit Shear, v_{ASD} $(Q_E \times 0.7\rho)/\Sigma L$	Capacity Reduction	Type	Wood SW DCR	Uplift (lbs)	Holdown	Net Uplift (lbs)	Holdown	Shared with lines...	Combined Uplifts (lbs)	Holdown	Notes
2	510 plf	1.00	TYPE 4	1.00	5,088	HDU5						

Seismic Uplift

SPREADSHEET SOLVES FOR THE WORST CASE UPLIFT (PANEL LENGTH = 6FT) USING DIAGRAM ON NEXT SHEET.

↳ $UPLIFT_{6FT, SWALL}$ = 5,088#

↳ $UPLIFT_{7.25FT, SWALL}$ = 4,960# (SEE NEXT SHEET)

L12.4 3

PROJECT: Sea Ranch Lodge Restaurant

JOB NO: 1977

DATE: 01/08/20

ENGINEER: CVF

SEISMIC DESIGN @ LEVEL 2 FLOOR

Horizontal Distribution of Seismic Force:

This SEISMIC spreadsheet is used compare the holdown load to WIND holdown load at LEVEL 2 Line 2 @ BEAM 1B-5

ROOF **LEVEL 2** LEVEL 1

L-12.3

3

Line of Resistance	Lateral System	C_s	Seismic Tributary Areas, TA (ft^2)					Lat Force, Q_E $F_x \times TA \times C_S/C_{S(SW)}$	Panel Lengths, L (ft)				Floor Height (ft)	Floor Depth (in)	Notes
A															
B															

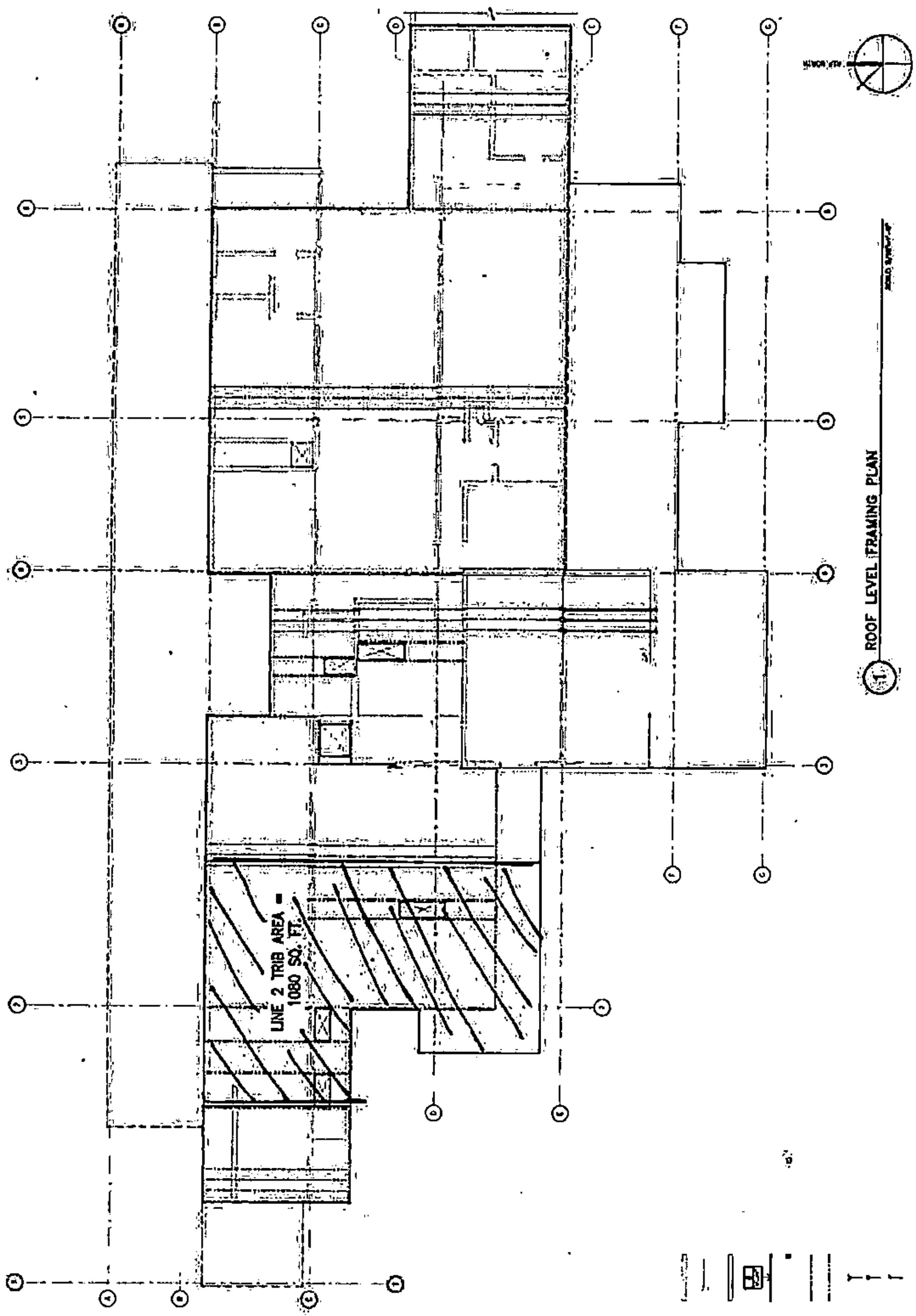
Line of Resistance	Lateral System	C_s	Seismic Tributary Areas, TA (ft^2)					Lat Force, Q_E $F_x \times TA \times C_S/C_{S(SW)}$	Panel Lengths, L (ft)				Floor Height (ft)	Floor Depth (in)	Notes
2	Wood SW	0.217	1,080	159				7,421 lbs	7.25	6.00			8.25	12	

PROJECT: The Sea Ranch Lodge Restaurant DATE: 10.18.2019

JOB NO: 1977 ENGINEER: CVF

LATERAL KEY (N/S DIRECTION) - S2.2

Roof Level



STRANDBERG ENGINEERING

L-12.2 3

PROJECT: Sea Ranch Lodge Restaurant

DATE: 01/08/20

JOB NO: 1977

ENGINEER: CVF

SEISMIC DESIGN @ ROOF

Horizontal Distribution of Seismic Force:

This SEISMIC spreadsheet is used compare the holdown load to WIND holdown load at LEVEL 2 Line 2 @ BEAM 1B-5

$\boxed{\text{ROOF}}$ LEVEL 2 LEVEL 1

Line of Resistance	Lateral System	C_s	Seismic Tributary Areas, TA (ft^2)					Lat Force, Q_E $F_x \times TA \times C_S/C_{S(SW)}$	Panel Lengths, L (ft)				Floor Height (ft)	Floor Depth (in)	Notes
A	Wood SW										10.00				
B	Wood SW														

L-12.1

3

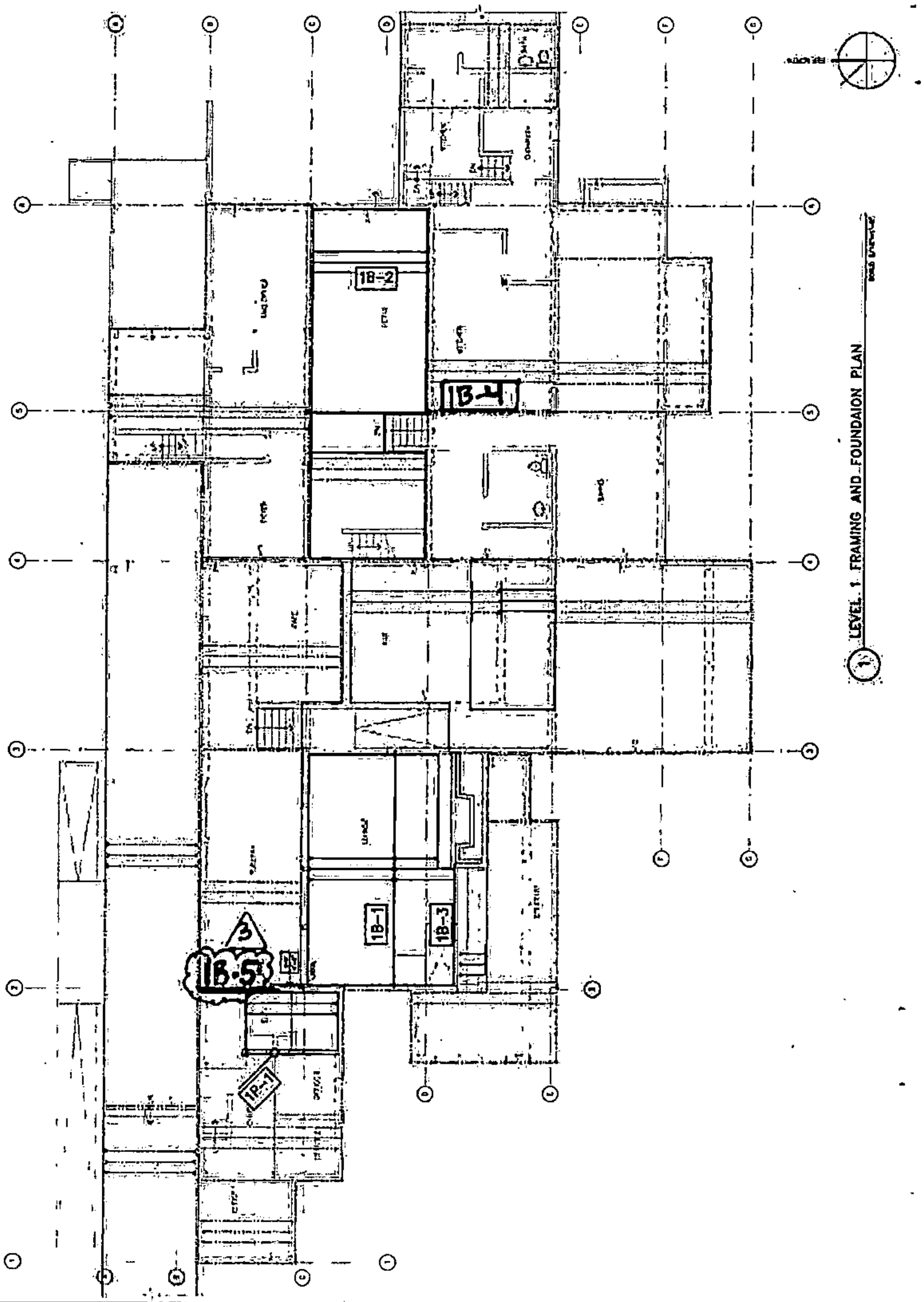
Line of Resistance	Lateral System	C_s	Seismic Tributary Areas, TA (ft^2)					Lat Force, Q_E $F_x \times TA \times C_S/C_{S(SW)}$	Panel Lengths, L (ft)				Floor Height (ft)	Floor Depth (in)	ROOF RAFTERS
2	(E) Wood SW	0.217	1,080					6,535 lbs							

PROJECT: The Sea Ranch Lodge Restaurant
DATE: 10.11.2019

JOB NO: 1977
ENGINEER: CVF

GRAVITY KEY - S2.0

Level 1



G-17 3

Holdowns (cont.)

These products are available with additional corrosion protection. For more information, see p. 15.

SS For stainless-steel fasteners, see p. 21.

SD Many of these products are approved for installation with Strong-Drive® SD Connector screws. See pp. 335-337 for more information.

Model No.	Ga.	Dimensions (in.) W	H	B	CL	SO	Fasteners (in.) Anchor Bolt Dia. (in.)	Wood Fasteners	Minimum Wood Member Size (in.)	Allowable Tension Loads (160) DF/SP	SPF/HF	Deflection at Allowable Load (in.)	Code Ref.
DTT1Z	14	1½	7⅛	1 7/16	¾	3/16	⅜	(6) SD #9 x 1½	1½ x 5½	840	840	0.17	IBC, FL, LA
								(6) 0.148 x 1½		910	640	0.167	
								(8) 0.148 x 1½		910	850	0.167	
SS DTT2Z	14	3¼	6 15/16	1⅝	13/16	3/16	½	(8) ¼ x 1½ SDS	1½ x 3½	1,825	1,800	0.105	
								(8) ¼ x 1½ SDS	3 x 3½	2,145	1,835	0.128	
SS DTT2Z-SDS2.5								(8) ¼ x 2½ SDS	3 x 3½	2,145	2,105	0.128	
HDU2-SDS2.5	14	3	8 11/16	3¼	1 5/16	1⅜	⅝	(6) ¼ x 2½ SDS	3 x 3½	3,075	2,215	0.088	
HDU4-SDS2.5	14	3	10 15/16	3¼	1 5/16	1⅜	⅝	(10) ¼ x 2½ SDS	3 x 3½	4,565	3,285	0.114	
HDU5-SDS2.5	14	3	13 3/16	3¼	1 5/16	1⅜	⅝	(14) ¼ x 2½ SDS	3 x 3½	5,645	4,340	0.115	
HDU8-SDS2.5	10	3	16⅝	3½	1⅜	1½	⅞	(20) ¼ x 2½ SDS	3 x 3½	6,765	5,820	0.11	
									3½ x 3½	6,970	5,995	0.116	
									3½ x 4½	7,870	6,580	0.113	
HDU11-SDS2.5	10	3	22¼	3½	1⅜	1½	1	(30) ¼ x 2½ SDS	3½ x 5½	9,335	8,030	0.137	
									3½ x 7¼	11,175	9,610	0.137	
HDU14-SDS2.5	7	3	25 11/16	3½	1 9/16	1 9/16	1	(36) ¼ x 2½ SDS	3½ x 5½	10,770	9,260	0.122	—
									3½ x 7¼	14,390	12,375	0.177	IBC, FL, LA
									5½ x 5½	14,445	12,425	0.172	

1. HDU14 requires heavy-hex anchor nut to achieve tabulated loads (supplied with holdown).
2. HDU14 loads on 4x6 post are applicable to installation on either the narrow or the wide face of the post.

LOADS

↳ SEE G-27.1 & G-27.2 FOR HIGHEST ASD UPLIFT LOAD TO USE FOR HOLDOWN DESIGN

1B-5: CASE 1 (WIND)

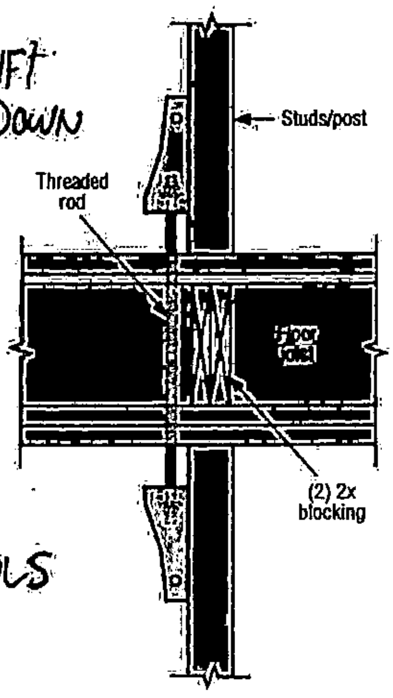
$\therefore T_{CASE\ 1} = 3{,}565\#$

1B-5: CASE 2 (SEISMIC)

$T_{CASE\ 2} = 1{,}219\#$

CASE 1 CONTROLS
USE T_{CASE1} FOR DESIGN

TRY "HDU4-SDS2.5"
CAPACITY = 4,565#



Typical HDU Tie Between Floors

M-6 3

UPDATED 06/01/19

DCR CHECK

$DCR = \frac{DEMAND}{CAPACITY}$

$= \frac{T_{CASE\ 1}}{CAPACITY}$

$= \frac{3{,}565\#}{4{,}565\#}$

$= 0.8 < 1.0$, OKAY

$\boxed{\text{USE "HDU4-SDS2.5"}}$