Structural Calculations For:

Masuda Remodel

6829 SE 32nd ST

Mercer Island, WA

Prepared for:	Mercer Builders, LLC
Job #:	11641-2020-05-01
Date:	May 2, 2021
Project Engineer:	Scott Wible

Criteria Sheet

Codes:

Structural:	IBC 2015
Loading:	ASCE 7-10
Wood:	NDS 2015
Steel:	AISC 360-10
Concrete:	ACI 318-14
Masonry:	TMS 402/602-13

Project Location:

Street & Number:	6829 se 32ND st		
City:	Mercer Islanc	State:	WA
ZIP:	98040		
Latitude:	47.5813 N		
Longitude:	-122.2465 W		

Occupancy Category

Risk Category: II ASCE 7 Table 1.5-1

Seismic Load Summary:

Analysis Procedure: Equivalent Lateral Force Procedure

Lateral System: Light-frame (wood) Walls Sheathed with Wood Structural Panels Rated for Shear Resistance

R:	6.50	C_d=	4
Base Shear V =	18 kips	Ω_o=	2.5
S_S=	1.393	S_1=	0.536
S_{DS}=	0.93	S_{D1}=	0.54
C_s=	0.143	I_E=	1.0

Wind Load Summary:

V=	110	K_{ZT}=	1.24
Exposure =	B		

Dead Loads:

Roof		
Roofing	2.5	psf
1/2" Sheathing	1.8	psf
Trusses @ 24" oc	2.5	psf
Misc./Mech.	1.5	psf
Ceiling Finish	2.8	psf
Solar Panels	4	
	15	psf
Use	15	psf
Seismic	**22.5**	psf
Floor		
Finish Floor	1	psf
3/4" Sheathing	2.7	psf
Joists @ 16" oc	2.2	psf
Misc./Mech.	2	psf
Ceiling Finish	2.8	
	10.7	psf
Use	12	psf
Seismic	**27**	psf

Level	Trib Area	
Roof	2108	sf
Upper Level	1818	sf
Main Level	1146	sf

Wall Height 9.8

Live Loads:

Snow	25	psf
Floor	40	psf

Soils:

Allowable Bearing 1500 psf

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Project		DATE	10/29/2020
Criteria		PROJ. #	
		DESIGN	ENG
		SHEET	1

Seismic Design

ASCE 7-10 Seismic Analysis — Equivalent Lateral Force Procedure

Seismic Force Resisting System: Per Table 12.2-1	System:	Bearing Wall Systems
	Type:	Light-frame (wood) Walls Sheathed with Wood Structural Panels Rated for Shear Resistance

Seismic Design Cat.	D	
Risk Category	II	I, II, or III, or IV per Table 1.5-1
Site Class	D	per soils report (D assumed, without soils report)
Diaphragm Flexibility	Flexible	

Ω_o	2.5	
S_S	1.393 g	2% in 50 yr, Latitude & Longitude lookup
S_1	0.536 g	2% in 50 yr, Latitude & Longitude lookup
h_n	29.5 ft	
R	6.50	
I_e	1.0	Table 1.5-2
C_d	4	
Ct	0.02	Table 12.8-2
x	0.75	Table 12.8-2
T	0.25 sec	Eq. 12.8-7
T_0	0.12 sec	
T_S	0.58 sec	
k	1.000	
Fa	1.00	Table 11.4-1
Fv	1.50	Table 11.4-2
S_{MS}	1.39 g	Eq. 11.4-1
S_{M1}	0.80 g	Eq. 11.4-2
S_{DS}	0.929 g	Eq. 11.4-3
S_{D1}	0.536 g	Eq. 11.4-4
Cs	***0.143 Controls***	Eq. 12.8-2
	0.326	Eq. 12.8-3 need not exceed, $T < T_L$
	0.010	Eq. 12.8-5 or 12.8-6 minimum
Cs, design	0.143	
Bldg. Weight	127.5 k	
$V = C_S W$	18.2 k	Eq. 12.8-1, Strength Level Base Shear
$V = C_{Sasd} W$	12.7 k	Eq. 12.8-1 ASD Base Shear

$T_a = C_t h_n^x$ Eq. 12.8.7

$S_{MS} = F_a S_S$ Eq. 11.4-1

$S_{M1} = F_v S_1$ Eq. 11.4-2

$S_{DS} = {}^2/_3 S_{MS}$ Eq. 11.4-3

$S_{D1} = {}^2/_3 S_{M1}$ Eq. 11.4-4

$C_S = \frac{S_{DS}}{(R/I_e)}$ Eq. 12.8-2

$C_S = \frac{S_{D1}}{T(R/I_e)}$ Eq. 12.8-3

$C_S = \frac{S_{D1} T_L}{T^2(R/I_e)}$ Eq. 12.8-4

$C_S \geq 0.044 S_{DS} I_e$ Eq. 12.8-5

$C_S \geq 0.01$ Eq. 12.8-5

$C_{VX} = w_x h_x^k / \sum_{i=1}^{n} w_x h_i^k$ Eq. 12.8-12

$F_{px} = \sum_{i=x}^{n} F_i \Big/ \sum_{i=x}^{n} w_i \, w_{px}$ Eq. 12.10-1

$F_{px} \geq 0.2 S_{DS} I_e w_{px}$ Eq. 12.10-2

$F_{px} \leq 0.4 S_{DS} I_e w_{px}$ Eq. 12.10-3

Vertical Distribution ASD ρ= 1.3

Level	h_x (ft)	W_x (k)	h_x^k (ft)	$W_x h_x^k$	Story Shear ASD			Diaphragm Force (ρ not included)				
					C_{vx} (%)	F_x (k)	SV (k)	$F_{px,calc}$	$F_{px,min}$	$F_{px,max}$	$F_{px,design}$	$y=F_{px}/F_x$
Roof	29.5	47.43	29.5	1398	0.524	8.7	**8.7**	6.7	6.2	12.3	**6.7**	0.77
Upper Level	19.6	49.086	19.6	964	0.362	6.0	**14.7**	5.7	6.4	12.8	**6.4**	1.06
Lower Level	9.8	31	9.8	304	0.114	1.9	**16.6**	3.1	4.0	8.0	**4.0**	2.13
Σ		127.5		2666		**16.6**						

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LATERAL CALCS

ROOF DIAPH / UPPER WALLS

F_x ROOF = 8.7K ASD

TRIB AREA BY % =>

	2	4
	27%	46.9%
V=	2.35K	4.08 kips
l_{wall}=	(2)×2.67 ft	22 ft
v =	0.44 plf	186 plf

$\frac{h}{b} = \frac{8.83'}{2.67} = 3.31$ N.A.

RED. = 1.25 − 0.125×3.31 = 0.836

W2 CAP = 595 plf × 0.836 = 497 plf > 391 plf

USE W2 SHEARWALLS ✓ USE WG ✓ ✓

O.T. 440 plf × 8.83' =

= 3885#

USE HDU4

186 × 8.83 = 1642#

DL RESIST:

(4'×9'×10psf

+9'×2'×15psf)×0.6

= 378 #

→ NET UP 1264#

OK

UPPER DIAPH / MAIN WALLS

F_x UPPER = 6 kips

	2	4
V_{above}	2.35K	4.08K
%	26%	49%
V =	2.35K + 0.26×6 = 3.91 kips	4.08K + 0.49×6 = 7.02 kips
l_{wall} =	8.25 ft	= 16.5 ft
v =	474 plf	425 plf

W2 CAPACITY:

$\frac{h}{b} = \frac{8.83'}{2.67} = 3.31$

RED = 1.25 − 0.125 × 3.31

= 0.84.

$W2_c$ = 0.84×595

= 498 plf ✓ OK

USE W2

W3 CAPACITY

= 456 plf ✓ ✓ OK

OT = 425 plf × 8.83'

= 3.75 kips

(E) HDU8

WAY GOOD

O.T. REGULAR

= 498 plf × 8.83' = 4.4 kips

CHECK OT @

STACKED WALLS

= 4.4 kips + 3.9 kips

= 8.3 kips

HDU 11 CAPACITY

= 9.335 K

DCR = 0.9 ✓ OK

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PROJECT MASUDA CALCS

DATE 10/29/20

PROJ. # SLW

DESIGN

SHEET

CHECK W12x53

$w_{DL} = 6' \times 12psf + 53plf = 125plf$

$w_{LL} = 6' \times 40psf = 240plf$ LL

$P_1 = 0.26k$ DL + $0.44k$ SL

$P_2 = 0.86k$ DL + $1.44k$ SL

$P_3 = 2.5 \times 4.4k = 11k$ $\Omega_o Q_E$

$P_4 = 2.5 \times 8.3k = 21$ K $\Omega_o Q_E$

LOAD COMBO:

$(1.0 + 0.14 S_{DS})D + 0.7E_{mh}$

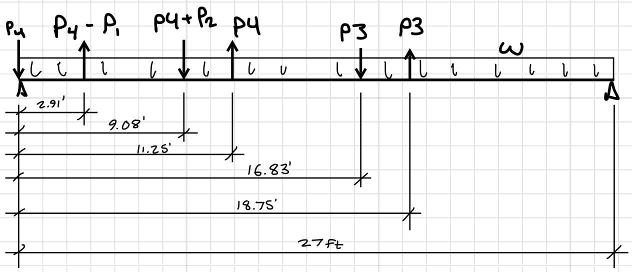
$w = 125plf$

$P_1 = 0.26K$

$P_2 = 0.86K$

$P_3 = 11K$

$P_4 = 21$ kips



⇒ REF NEXT PAGE
W12x53 OK FOR
O.T. LOADS.

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Your Company Name	Project:	Page # ___
	Engineer:	4/19/2021
	Descrip: Masuda (E) Beam	

ASDIP Steel 4.6.6 **STEEL BEAM DESIGN** *www.asdipsoft.com*

GEOMETRY

Beam Designation	W12X53		
Steel Yield Strength Fy ...	50.0	ksi	OK
Modulus of Elasticity Es ..	29000	ksi	
Member Length L	27.00	ft	
Left Cantilever	0.00	ft	
Right Cantilever	0.00	ft	
Unbraced Length Lb top ..	0.00	ft	
Unbraced Length Lb bot ..	27.00	ft	

PROPERTIES

Area ..	15.6	in^2	Sx ...	70.6	in^3
Depth	12.1	in	Zx ...	77.9	in^3
bf	10.0	in	rx	5.23	in
tw	0.35	in	Iy	95.8	in^4
tf	0.58	in	Sy ...	19.2	in^3
k des .	1.18	in	Zy ...	29.1	in^3
Ix	425.0	in^4	ry	2.48	in
Cw	3160.0	in^6	J	1.58	in^4

ASD - SERVICE LOADS (Selfweight not calculated internally)

	Uniform (k/ft)		Concentrated (kip)						Moments (k-ft)	
SPAN 1	w1	w2	P1	P2	P3	P4	P5	P6	ML	MR
During Construction	0.13	0.00	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Structure In Service	0.00	0.00	-21.0	21.0	-21.0	11.0	-11.0	0.0	0.0	0.0
Start Distance (ft) ...	0.00	0.00	2.91	9.08	11.25	16.80	18.75	0.00		
End Distance (ft)	27.00	0.00								

STEEL BEAM DESIGN

SLAB AND DECK

Overall Slab Thickness	0.0	in
Interior Beam. Beam Spacing = 5.0 ft		
Effective Slab Width	5.00	ft
Concrete Strength f'c	3000	psi
Concrete Density	150	pcf
Metal Deck Type	VULCRAFT 2 VLI	
Deck Ribs Height hr	2.0	in
Deck Ribs Avg. Width wr ..	6.0	in
Deck Ribs Run Perpendicular to the Beam		

FLEXURE DESIGN (STEEL)

L. T. Buckling Cb-factor	1.21		
Max. Bending Moment M ..	-45.1	k-ft	
Limit States	Nominal Mn		
Yielding	324.6	k-ft	⟵
Lateral Torsional Buckling	257.7	k-ft	
Flange Local Buckling	N.A.	k-ft	
Web Local Buckling	N.A.	k-ft	
Nominal Strength Mn	154.3	k-ft	
Safety Factor Ω	1.67		
Allowable Strength Mn/Ω ...	154.3	k-ft	
M / Mn/Ω Design Ratio	0.29		OK

FLEXURE DESIGN (COMPOSITE)

Max. Bending Moment M	N.A.
Limit States	Nominal Mn
Plastic Yielding	N.A.
Elastic Yielding	N.A.
Nominal Strength Mn	N.A.
Safety Factor Ω	1.67
Allowable Strength Mn/Ω	N.A.
M / Mn/Ω Design Ratio	N.A.

DESIGN CODES

Steel Design	AISC 360-10 (14th Ed.)
Load Combinations ...	

DESIGN FOR SHEAR

Shear Coefficient Cv	1.00		
Maximum Shear Force V ...	16.3	kip	
Limit States	Nominal Vn		
Shear Yielding	125.2	kip	⟵
Shear Buckling	125.2	kip	
Nominal Strength Vn	125.2	kip	
Safety Factor Ω	1.50		
Allowable Strength Vn/Ω ..	83.5	kip	
V / Vn/Ω Design Ratio	0.19		OK

LOCAL BUCKLING

Flanges in Flexure	Compact
Flanges in Compression	Non-compact
Web in Flexure	Compact
Web in Compression	Non-compact

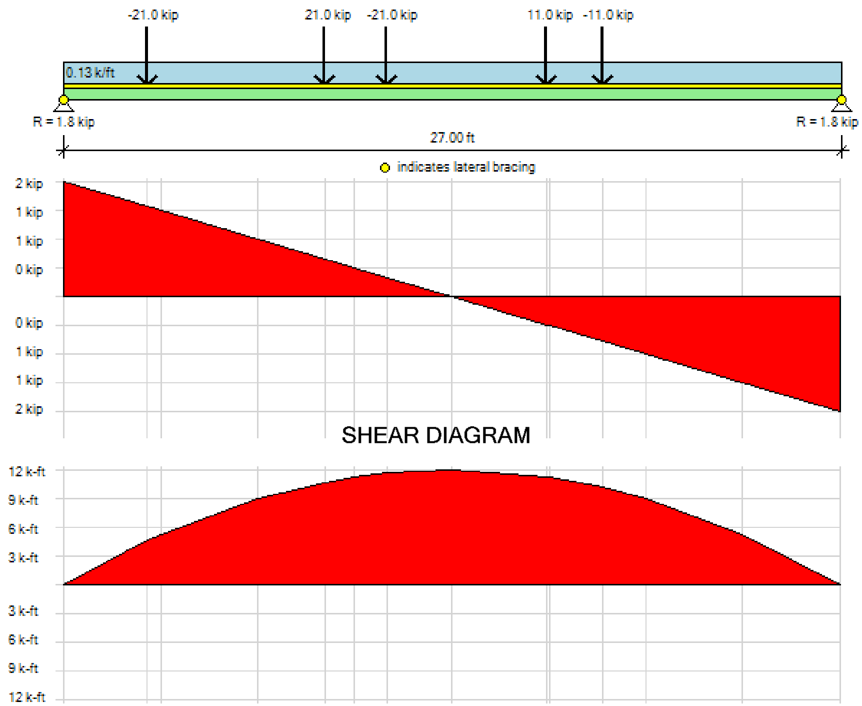
SHEAR CONNECTORS

Shear Stud Diameter	N.A
Shear Stud Length	N.A
Tensile Strength Fu	N.A
Nominal Strength Qn	N.A
Horizontal Shear Force	N.A
# of Studs for Full Composite	N.A
# of Studs for Partial Composite ..	N.A
Partial Composite Action %	N.A
Minimum Spacing Allowed	N.A
# of Studs at Any Section	N.A
Max. Spacing Required	N.A

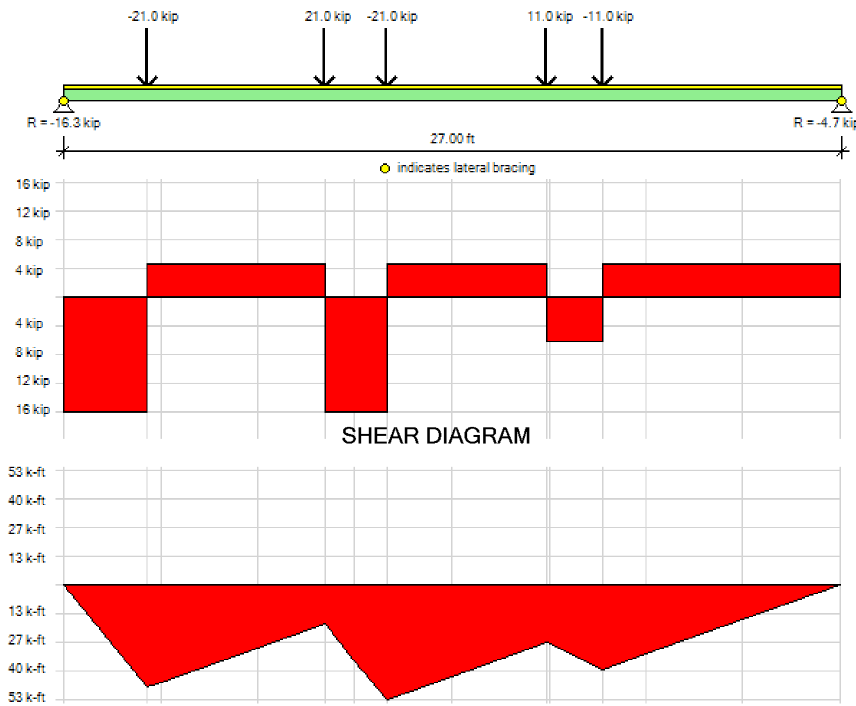
DEFLECTIONS

Stiffness factor	1.0	
Required Camber	0.00	in
Long-term Deflection	N.A.	

Loading	δ (in)	L/δ	L/δ Min	Ratio	
Construct.	0.13	2569	240	0.09	OK
In Service	-0.36	889	240	0.27	OK

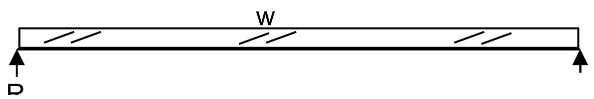


MOMENT DIAGRAM
(Stage: During Construction)

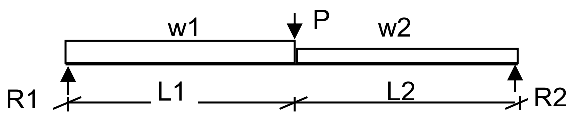


MOMENT DIAGRAM
(Stage: Structure In Service)

Beam		B2 (N) Roof		LSL	3 1/2 x 11 7/8	
w=	540	plf		R=	3,240	lbs
L=	12	ft		M=	9,720	ft-lbs
b=	3.50	in		Fb=	1,418	psi
d=	11.88	in		Fv=	98	psi
E=	1550	ksi		Δ=	0.33	in
Cv=	1.00	≤1.0		l /	433	

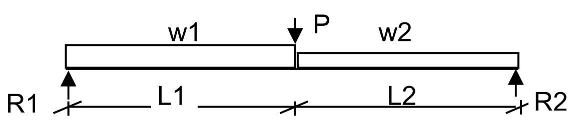


Beam		B3 (N) Roof		LSL	5 1/4 x 11 7/8	
w1=	-	plf		R1 =	1,291	lbs
w2=	-	plf		R2 =	4,679	lbs
L1=	7.25	ft		M =	9,358	lb-ft
L2=	2	ft		Fb =	910	psi
X=	5.5	ft		Fv =	113	psi
P=	5,970	lbs		Δ=	0.09	in
b=	5.25	in		l /	1,201	
d=	11.88	in		Cv=	1.00	
E=	1,550	ksi				



TJI Size	9.50	in		RE 1.75	9.5 TJI 110	
EI =	157	k-in^2		Ma=	2500	lb-ft
Δ =	0.821	in		Va=	1220	lbs
l /	135			Ra=	910	lbs

Beam		W14x82 loads		PSL	1 x 1	
w1=	685	plf		R1 =	13,181	lbs
w2=	685	plf		R2 =	11,214	lbs
L1=	9	ft		M =	90,885	lb-ft
L2=	18	ft		Fb =	6,543,720	psi
X=	13.0	ft		Fv =	19,686	psi
P=	5,900	lbs		Δ=	#########	in
b=	1.00	in		l /	0	
d=	1.00	in		Cv=	1.00	
E=	1	ksi				



Steel Size Beam calc				W14X82	
I =	881	in^4	Fy=	50	ksi
Δ =	0.460	in	Mn/Ω =	346.8	k-ft
l /	704		Vn/Ω =	203.1	kips

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Project: Masuda

Date: 02/19/21

Project #:

Design: SLW

Sheet:

DIAPHRAGM CALCS:

F_x = 8.7 kips (ROOF)

V_{BTWN} GRIDS 2 + 4 @ ROOF

$= \frac{1028\ SF}{2069\ SF} \times 8.7k = 4.322\ kips$

SPAN = 33 Ft

INCREASE DIAPHRAGM FORCE BY 25% @ CHORDS

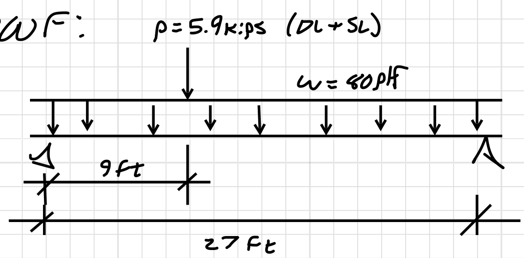
$w = \frac{4.322k \times 1.25}{33\ Ft} = 164\ plf$

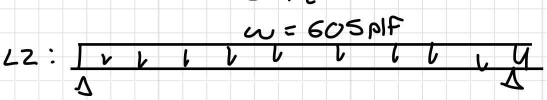
$M = \frac{wl^2}{8} = \frac{164\ plf \times 33\ Ft^2}{8} = 22324$ lb-Ft

$T = \frac{M}{D} = \frac{22324\ lb\text{-}ft}{27\ Ft} = 826\ \#$

USE CS16 STRAP ✓✓

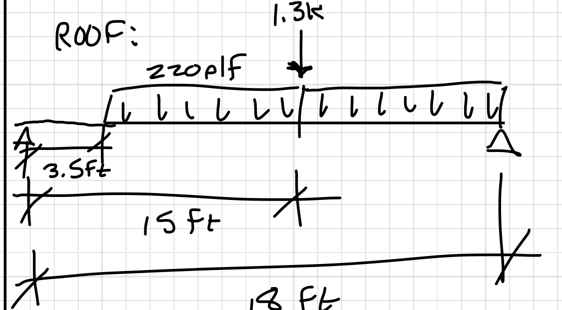
CHECK W14 x 82

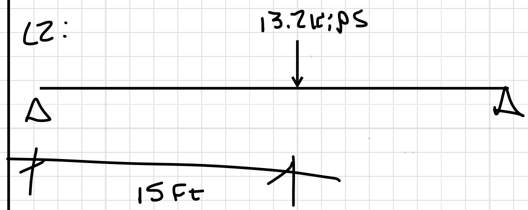


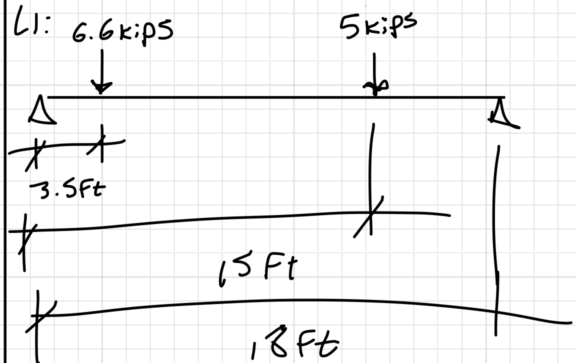


COMBINED w = 685 plf, P = 5.9k @ 9ft (CONT)

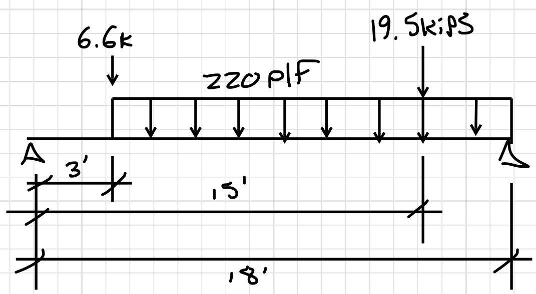
CHECK GL 5½ x 22½ HDR







COMBINED:



Δ = 0.343 in ✓OK

M = 57.33 k-Ft

$F_b = \frac{57.33 \times 12 \times 6 \times 1000}{5.5 \times 22.5^2} = 1482$ psi ✓OK

$F_v = \frac{19{,}440\ \#}{5.5\ in \times 22.5\ in} = 157$ psi ✓OK

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SINGLE-SPAN BEAM ANALYSIS

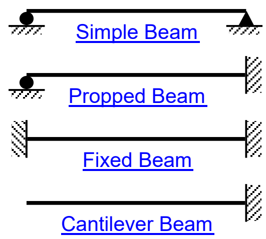
For Simple, Propped, Fixed, or Cantilever Beams

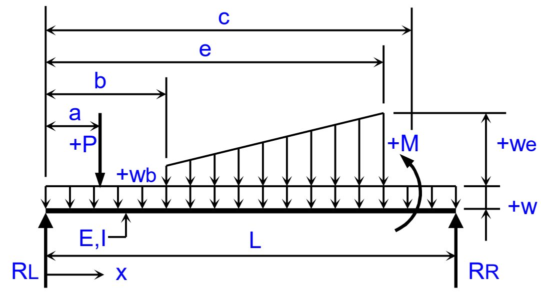
Job Name:	Masuda Remodel	Subject:	GL 5½ x 22½		
Job Number:		Originator:		Checker:	

Input Data:

Beam Data:

Span Type?	Simple	
Span, L =	18.0000	ft.
Modulus, E =	1800	ksi
Inertia, I =	5220.70	in.^4





Nomenclature

Beam Loadings:

Full Uniform:

w = ______ kips/ft.

Distributed:

	Start		End	
	b (ft.)	Wb (kips/ft.)	e (ft.)	We (kips/ft.)
#1:	3.5000	0.2200	18.0000	0.2200
#2:				
#3:				
#4:				
#5:				
#6:				
#7:				
#8:				

Point Loads:

	a (ft.)	P (kips)
#1:	3.5000	6.60
#2:	15.0000	19.50
#3:		
#4:		
#5:		
#6:		
#7:		
#8:		
#9:		
#10:		
#11:		
#12:		
#13:		
#14:		
#15:		

Moments:

	c (ft.)	M (ft-kips)
#1:		
#2:		
#3:		
#4:		

Results:

Reactions:

RL =	9.85 k	RR =	19.44 k
ML =	N.A.	MR =	N.A.

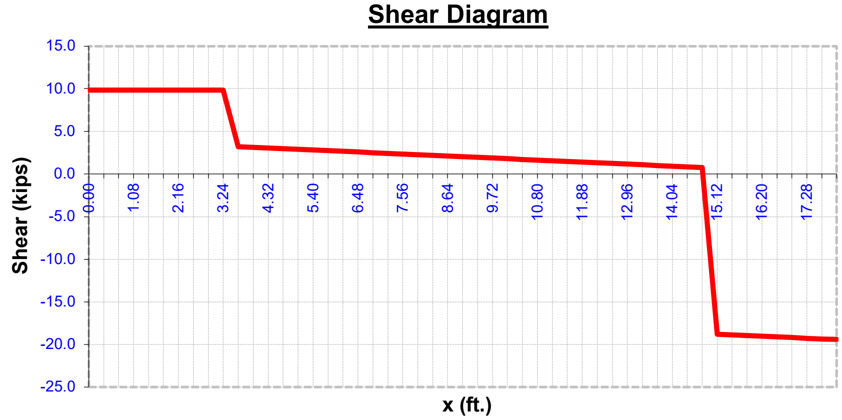
Maximum Moments:

+M(max) =	57.33 ft-k	@ x =	15.00 ft.
-M(max) =	0.00 ft-k	@ x =	0.00 ft.

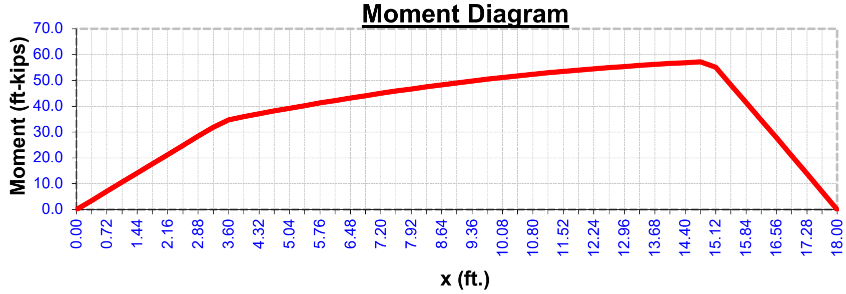
Maximum Deflections:

-Δ(max) =	-0.343 in.	@ x =	9.51 ft.
+Δ(max) =	0.000 in.	@ x =	0.00 ft.
Δ(ratio) =	L/630		

Shear Diagram



Moment Diagram



Tabulation of Single-Span Beam Shear, Moment, Slope, and Deflection for 50 Equal Segments					
Point #	x (ft.)	Shear (k)	Moment (ft-k)	Slope or Rotation (deg.)	Deflection (in.)
1	0.0000	9.85	0.00	-0.2795	0.0000
2	0.3600	9.85	3.55	-0.2789	-0.0211
3	0.7200	9.85	7.09	-0.2772	-0.0420
4	1.0800	9.85	10.64	-0.2744	-0.0628
5	1.4400	9.85	14.19	-0.2705	-0.0834
6	1.8000	9.85	17.73	-0.2654	-0.1036
7	2.1600	9.85	21.28	-0.2593	-0.1234
8	2.5200	9.85	24.83	-0.2520	-0.1427
9	2.8800	9.85	28.37	-0.2436	-0.1613
10	3.2400	9.85	31.92	-0.2341	-0.1794
11	3.6000	3.23	34.80	-0.2234	-0.1966
12	3.9600	3.15	35.95	-0.2123	-0.2130
13	4.3200	3.07	37.07	-0.2007	-0.2286
14	4.6800	2.99	38.16	-0.1888	-0.2433
15	5.0400	2.91	39.23	-0.1766	-0.2571
16	5.4000	2.83	40.26	-0.1640	-0.2699
17	5.7600	2.75	41.27	-0.1511	-0.2818
18	6.1200	2.68	42.24	-0.1379	-0.2927
19	6.4800	2.60	43.19	-0.1244	-0.3026
20	6.8400	2.52	44.11	-0.1106	-0.3115
21	7.2000	2.44	45.01	-0.0966	-0.3193
22	7.5600	2.36	45.87	-0.0822	-0.3260
23	7.9200	2.28	46.70	-0.0676	-0.3317
24	8.2800	2.20	47.51	-0.0527	-0.3362
25	8.6400	2.12	48.29	-0.0375	-0.3396
26	9.0000	2.04	49.04	-0.0222	-0.3418
27	9.3600	1.96	49.76	-0.0065	-0.3429
28	9.7200	1.88	50.45	0.0093	-0.3428
29	10.0800	1.80	51.11	0.0254	-0.3415
30	10.4400	1.72	51.75	0.0416	-0.3390
31	10.8000	1.65	52.35	0.0581	-0.3352
32	11.1600	1.57	52.93	0.0747	-0.3302
33	11.5200	1.49	53.48	0.0915	-0.3240
34	11.8800	1.41	54.00	0.1085	-0.3164
35	12.2400	1.33	54.50	0.1257	-0.3076
36	12.6000	1.25	54.96	0.1430	-0.2975
37	12.9600	1.17	55.40	0.1604	-0.2861
38	13.3200	1.09	55.80	0.1780	-0.2733
39	13.6800	1.01	56.18	0.1957	-0.2592
40	14.0400	0.93	56.53	0.2135	-0.2438
41	14.4000	0.85	56.85	0.2314	-0.2270
42	14.7600	0.77	57.15	0.2494	-0.2089
43	15.1200	-18.80	55.07	0.2674	-0.1894
44	15.4800	-18.88	48.29	0.2837	-0.1686
45	15.8400	-18.96	41.47	0.2979	-0.1467
46	16.2000	-19.04	34.63	0.3099	-0.1237
47	16.5600	-19.12	27.76	0.3198	-0.1000
48	16.9200	-19.20	20.87	0.3275	-0.0756
49	17.2800	-19.28	13.94	0.3330	-0.0507
50	17.6400	-19.36	6.98	0.3363	-0.0254
51	18.0000	-19.44	0.00	0.3374	0.0000

Column Buckling Calculations

NDS 2015

Apply Fire Rating reduction per Footnote m from IBC table 721.1(2)?
(Applies to 2x4 stud posts in bearing walls) No

Column Geometry Data

Hem-Fir #2 Studs		
Doug Fir Plates		
b	4.5	in
d	5.5	in
Le_1	8.70	ft
Le_2	8.70	ft
$le_{bending}$	8.70	ft

Bracing

Braced
No Brace

Column Stability Factor Calculation

Strong Axis

Fce_1	1072	psi
Fc^*_1	1300	psi
Fce_1/Fc^*_1	0.825	
Cp_1	0.621	

Weak Axis

Fce_2	217316	psi
Fc^*_2	1300	psi
Fce_2/Fc^*_2	167.166	
Cp_2	1.000	

Beam Stability Factor Calculation

Strong Axis

Fbe_1	19890	psi
Fb'_1	850	psi
Fbe_1/Fb'_1	23.4	
le	8.7	ft
CL_1	1.00	

Weak Axis

Fbe_2	351,049	psi
Fb'_2	850	psi
Fbe_2/Fb'_2	413	

Column Design Values

F_b	850	psi
F_c	1300	psi
E'min	470	ksi
Fcperp	625	psi
cb	1.00	

Bearing Area Increase
No

Adjusted Allowable Stresses

Strong Axis

Fc'_1	807	psi
Fb'_1	850	psi

Weak Axis

Fc'_2	1300	psi
Fb'_2	850	psi

Column Loading

P	15469	lbs
W_1	5	plf
M1	47	ft-lbs
W_2	0	plf
M2 (Braced)	0	ft-lbs

Imposed Column Stresses

Strong Axis

fc_1	625	psi
fb_1	25	psi

Weak Axis

fc_2	625	psi
fb_2	0	psi

Flexural Stress Adjustment Factors

Roof/EQ / Wind - C_D	1.00
Size Factor - C_F	1.00
Repetitive - C_r	1.00

Perpendicular to Grain Stress Check fcp/Fcp =	***625 / 625***	**OK**
Slenderness Check le/d	**19**	**OK**
Slenderness Check le/b	**23**	**OK**

Compressive Parallel Adjustment Factors

Roof/EQ / Wind - C_D	1.00
Size Factor - C_F	1.00

(1) $$\left(\frac{f_c}{F_c'}\right)^2 + \frac{f_{b1}}{F_{b1}'\left[1 - f_c/F_{cE1}\right]} + \frac{f_{b2}}{F_{b2}'\left[1 - f_c/F_{cE2} - \left(f_{b1}/F_{bE}\right)\right]} \leq 1.0$$

(2) $$\frac{f_c}{F_{cE2}} + \left(\frac{f_{b1}}{F_{bE}}\right)^2 < 1.0$$

(3) $$\frac{f_c}{F_{c'}}, \frac{f_{b1}}{F_{b1}'}, \frac{f_{b2}}{F_{b2}'} < 1.0$$

Allowable Stress Interaction Formula	**1.00**	**OK**

Other Factors

Visually Graded Lumber	
c	0.8
Solid Column	
Kf	1
Column: Pinned Pinned	
Ke	1

Project:	Masuda	*Date:*	*2/19/2021*
	Multi-Stud Posts	Project #:	
	(3) 2x6	Design:	SLW
		Sheet:	

Column Buckling Calculations

NDS 2015

Column Geometry Data

Hem-Fir #2 Studs		
Doug Fir Plates		
b	3.5	in
d	4.5	in
Le_1	8.70	ft
Le_2	8.70	ft
$le_{bending}$	8.70	ft

Bracing

No Brace
Braced

Column Design Values

F_b	850	psi
F_c	1300	psi
E'min	470	ksi
Fcperp	625	psi
cb	1.00	

Bearing
Area
Increase
No

Column Loading

P	5225.7	lbs
W_1	0	plf
M1 (Braced)	0	ft-lbs
W_2	5	plf
M2	47	ft-lbs

Flexural Stress Adjustment Factors

Roof/EQ / Wind - C_D	1.00
Size Factor - C_F	1.00
Repetitive - C_r	1.00

Compressive Parallel Adjustment Factors

Roof/EQ / Wind - C_D	1.00
Size Factor - C_F	1.00

Other Factors

Visually Graded Lumber	
c	0.8
Solid Column	
Kf	1
Column: Pinned Pinned	
Ke	1

Apply Fire Rating reduction per Footnote m from IBC table 721.1(2)?
(Applies to 2x4 stud posts in bearing walls) No

Column Stability Factor Calculation

Strong Axis

Fce_1	217316	psi
Fc^*_1	1300	psi
Fce_1/Fc^*_1	167.166	
Cp_1	1.000	

Weak Axis

Fce_2	434	psi
Fc^*_2	1300	psi
Fce_2/Fc^*_2	0.334	
Cp_2	0.307	

Beam Stability Factor Calculation

Strong Axis

Fbe_1	14706	psi
Fb'_1	850	psi
Fbe_1/Fb'_1	17.3	
le	8.7	ft
CL_1	1.00	

Weak Axis

Fbe_2	17,365	psi
Fb'_2	850	psi
Fbe_2/Fb'_2	20	

Adjusted Allowable Stresses

Strong Axis

Fc'_1	1300	psi
Fb'_1	847	psi

Weak Axis

Fc'_2	399	psi
Fb'_2	850	psi

Imposed Column Stresses

Strong Axis

fc_1	332	psi
fb_1	0	psi

Weak Axis

fc_2	332	psi
fb_2	62	psi

Perpendicular to Grain Stress Check fcp/Fcp =	***332 / 625***	**OK**
Slenderness Check le/d	**23**	**OK**
Slenderness Check le/b	**30**	**OK**

(1) $$\left(\frac{f_c}{F_c'}\right)^2+\frac{f_{b1}}{F_{b1}'\left[1-f_c/F_{cE1}\right]}+\frac{f_{b2}}{F_{b2}'\left[1-f_c/F_{cE2}-\left(f_{b1}/F_{bE}\right)\right]}\leq 1.0$$

(2) $$\frac{f_c}{F_{cE2}}+\left(\frac{f_{b1}}{F_{bE}}\right)^2<1.0$$

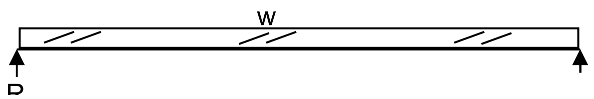
(3) $$\frac{f_c}{F_{c'}},\frac{f_{b1}}{F_{b1}'},\frac{f_{b2}}{F_{b2}'}<1.0$$

Allowable Stress Interaction Formula	**1.00**	**OK**

SSF

Project:	Masuda	*Date:*	*2/19/2021*
	Multi-Stud Posts	Project #:	
	(3) 2x4	Design:	SLW
		Sheet:	

Beam		B1		HF	2	x 8
w=	30	plf		R=	248	lbs
L=	16.5	ft		M=	1,021	ft-lbs
b=	1.50	in		Fb=	932	psi
d=	7.25	in		Fv=	32	psi
E=	1300	ksi		Δ=	0.81	in
Cv=	1.00	≤1.0		l /	245	



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Project: MASUDA STRUCTURAL FASCIA

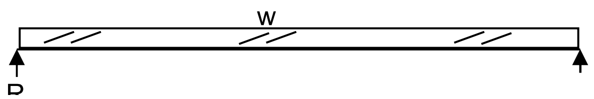
Date: 04/20/21

Project #:

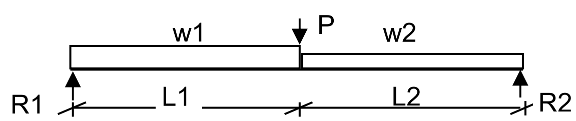
Design: SLW

Sheet:

Beam	B1		DF-L	6 x 8	
w=	160	plf	R=	1,160	lbs
L=	14.5	ft	M=	4,205	ft-lbs
b=	5.50	in	Fb=	979	psi
d=	7.50	in	Fv=	39	psi
E=	1700	ksi	Δ=	0.48	in
Cv=	1.00	≤1.0	l /	359	



Beam	B2		LSL	3 1/2 x 11 7/8	
w1=	240	plf	R1 =	2,263	lbs
w2=	535	plf	R2 =	3,206	lbs
L1=	6.25	ft	M =	9,457	lb-ft
L2=	5.25	ft	Fb =	1,380	psi
X=	6.3	ft	Fv =	97	psi
P=	1,160	lbs	Δ=	0.27	in
b=	3.50	in	l /	505	
d=	11.88	in	Cv=	1.00	
E=	1,550	ksi			



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Date: 05/01/21

Project #: ____

Design: ENG

Sheet: ____