Structural Calculations

SYGULLA RESIDENCE

34194 BRIDGEVIEW DRIVE NE
KINGSTON, WA 98436

Reviewed for code compliance
with IRC 2015
Kitsap County Building Department
GShapiro@co.kitsap.wa.us
04/29/2020

prepared for:

ARMSTRONG HOMES OF BREMERTON

2531 PERRY AVENUE
BREMERTON, WA 98310

prepared by:

Sitts & Hill Engineers, Inc.
4815 Center Street
Tacoma, WA 98409

February 5, 2020

S&H Job Number 18,720

Permit Number: 20-00976

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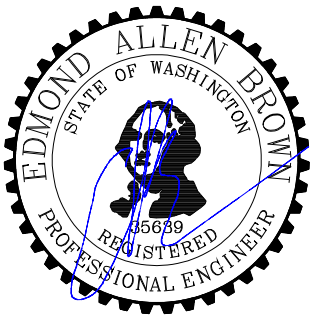

02/05/20

PREPARED BY

Sitts & Hill Engineers, Inc.
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Tacoma, WA 98409

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S & H Job Number: 18,720

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CALCULATION INDEX

BASIS	**2**
DESIGN LOADS	**3**
GRAVITY DESIGN	**4-57**
LATERAL DESIGN	**58-72**

Permit Number: 20-00976

BASIS FOR DESIGN

BUILDING CODE: 2015 Edition of the International Building Code with State of Washington amendments shall be used and supplemented with ASCE 7-10.

RISK CATEGORY: II.

GRAVITY LOADS:
Roof Snow Load = 25 psf.
Superimposed Dead Load on Roof Trusses = 13 psf.
Roof Dead Load = 16 psf.
Residential Floor Live Load = 40 psf (reducible).
Floor Dead Load = 12 psf.
Deck Live Load = 60 psf.

LATERAL LOAD CRITERIA:
Wind Speed = 110 mph, Exposure: C.
Seismic Site Class: D.
Seismic Design Category: D.

FOUNDATIONS: Allowable soil bearing pressure = 1,500 psf.

CONCRETE: Minimum compressive strength of concrete shall be 3,000 psi to comply with exposure requirement of IBC table 1904.2. Structural design is based on f'c of 2,500 psi. Per IBC 1705.3, no special inspection is required.

REINFORCING: All bars shall be ASTM A615, Grade 60, unless noted otherwise.

STRUCTURAL STEEL: Anchor Bolts: ASTM F1554, Grade 36.

WOOD:

Joists	2x4	HF #2
	2x6 and larger	HF #2
Beams	4" width or less	HF #2
	Greater than 4" width	DF #2
Ledger	4" width or less	HF #2
Top Plates	All Sizes	DF #2
Studs	2x4	HF #2
	2x6 and larger	HF #2
Posts	4x4	DF #2
	4x6 or larger	DF #2
	6x6 or larger	DF #2

Glue-laminated beams (glulams):
Fb = 2,400 PSI, Fv = 265 psi, Fc-perp = 650 psi, E = 1,800,000 psi.

Parallel Strand Lumber (PSL):
Fb = 2,900 psi, Fv = 290 psi, Fc-perp = 650 psi, E = 2,000,000 psi.

Laminated Strand Lumber (LSL):
Fb = 1,700 psi, Fv = 400 psi, Fc-perp = 680 psi, E = 1,300,000 psi.

Laminated Veneer Lumber (LVL):
Fb = 2,600 psi, Fv = 285 psi, Fc-perp = 750 psi, E = 1,900,000 psi.

Pre-fabricated wood trusses:
Pre-fabricated wood trusses shall be designed by a professional engineer registered in the State of Washington.

Permit Number: 20-00976

DESIGN LOADS

ROOF LIVE LOAD: **25.0 PSF** Snow Load

ROOF DEAD LOAD:

Roofing	3.0 PSF	
Plywood	2.0 PSF	
Insulation	1.0 PSF	
Ceiling	2.0 PSF	
Lighting, Mechanical, Electrical, Misc.	5.0 PSF	

	13.0 PSF	Superimposed on Roof Trusses
Trusses	2.0 PSF	
Beams	1.0 PSF	

	16.0 PSF	Total Dead Load

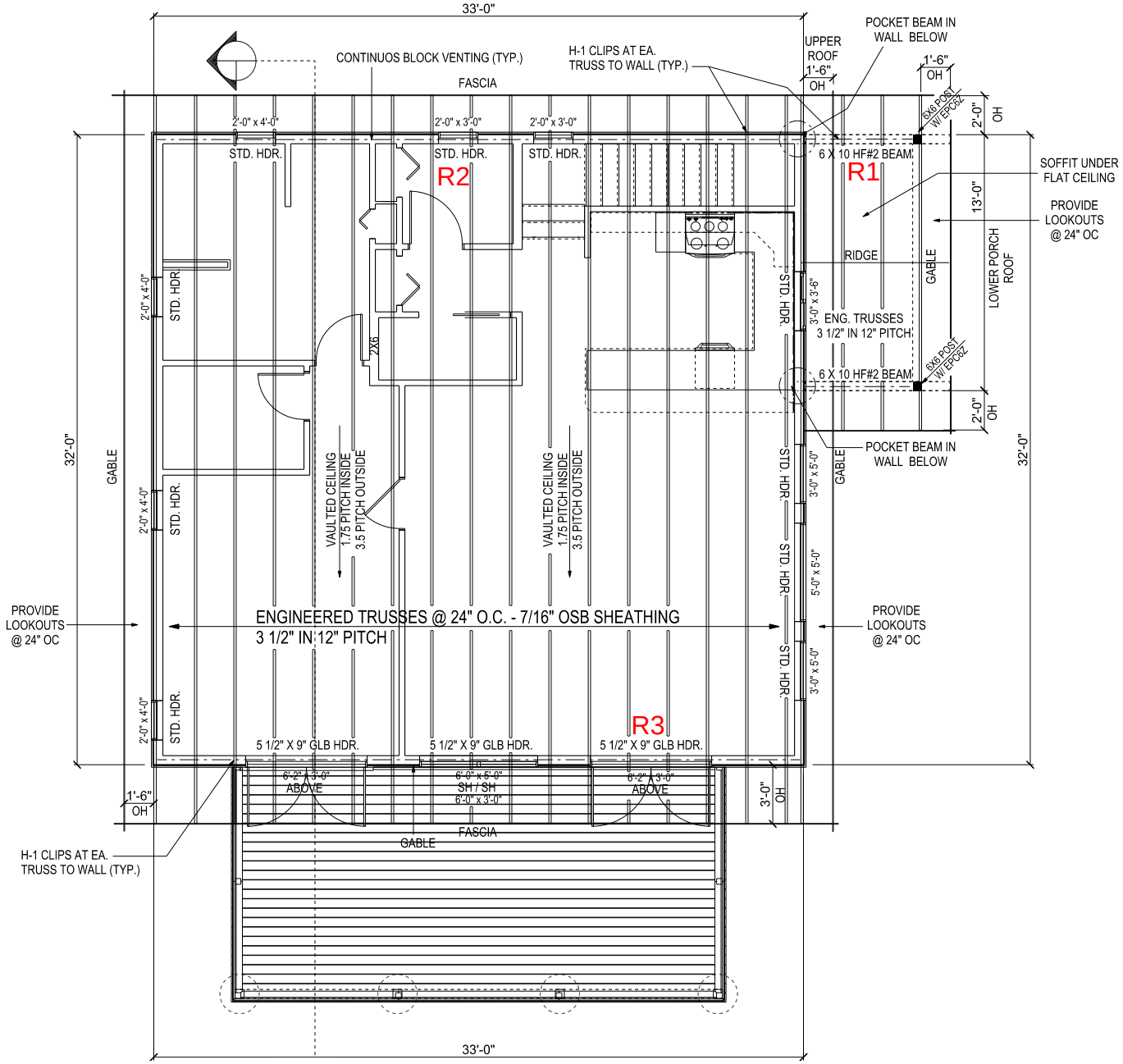
RESIDENTIAL FLOOR LIVE LOAD: **40.0 PSF** Reducible

RESIDENTIAL DECK LIVE LOAD: **60.0 PSF**

RESIDENTIAL FLOOR DEAD LOAD:

Flooring	1.0 PSF	
Insulation	1.0 PSF	
Floor Joists	2.0 PSF	
Ceiling	3.0 PSF	
Lighting, Mechanical, Electrical, Misc.	5.0 PSF	

	12.0 PSF	Total Dead Load



ROOF FRAMING PLAN
1/8" = 1'-0"

$DL_r := 16\ psf$
$SL_r := 25\ psf$

$DL_{fl} := 12\ psf$
$LL_{fl} := 40\ psf$

$LL_d := 60\ psf$

ROOF FRAMING

R_1
$l := 6\ ft$
$w := \left[DL_r\ \ SL_r\right] \cdot (13\ ft \cdot 0.5 + 2\ ft) = [136\ \ 213]\ plf$ 6x10 HF#2 P.T. OK

R_2
$l := 2\ ft$
$w := \left[DL_r\ \ SL_r\right] \cdot (32\ ft \cdot 0.5 + 2\ ft) = [288\ \ 450]\ plf$ 3 1/2x9 GLB OK

R_3
$l := 4\ ft$
$w_1 := \left[DL_r\ \ SL_r\right] \cdot (32\ ft \cdot 0.5 + 3\ ft) = [304\ \ 475]\ plf$ 5 1/2x9 GLB OK

Wood Beam

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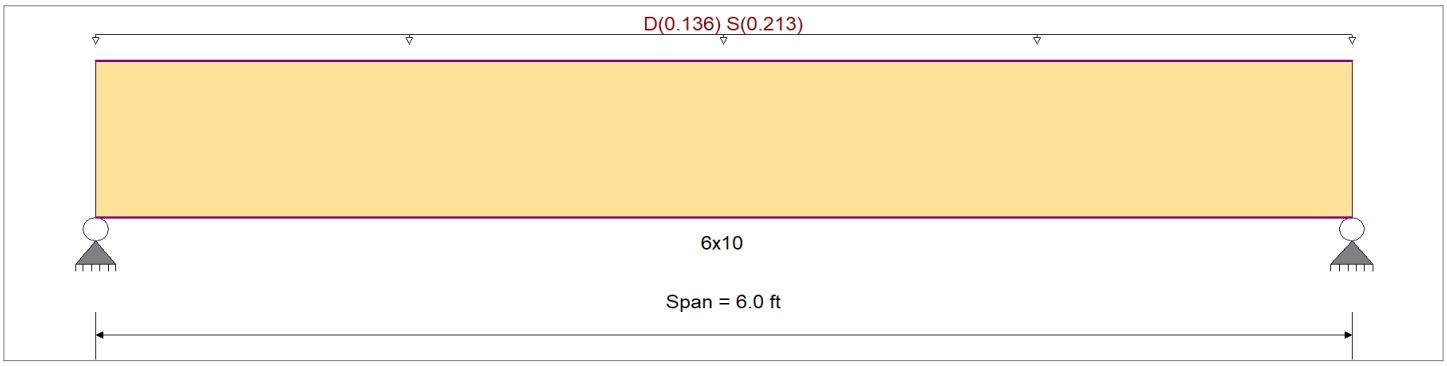
DESCRIPTION: R1

CODE REFERENCES

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

Material Properties

Analysis Method :	Allowable Stress Design	Fb +	675.0 psi	*E : Modulus of Elasticity*	
Load Combination	ASCE 7-10	Fb -	675.0 psi	Ebend- xx	1,100.0 ksi
		Fc - Prll	500.0 psi	Eminbend - xx	400.0 ksi
Wood Species	: Hem-Fir	Fc - Perp	405.0 psi		
Wood Grade	: No.2	Fv	140.0 psi		
		Ft	350.0 psi	Density	26.840 pcf
Beam Bracing	: Beam is Fully Braced against lateral-torsional buckling				



Applied Loads

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

Uniform Load : D = 0.1360, S = 0.2130 , Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.367 : 1	Maximum Shear Stress Ratio	=	0.172 : 1
Section used for this span		6x10	Section used for this span		6x10
	=	227.80 psi		=	22.16 psi
	=	621.00 psi		=	128.80 psi
Load Combination		+D+S	Load Combination		+D+S
Location of maximum on span	=	3.000 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	0.015 in	Ratio =	4732	>=360
Max Upward Transient Deflection	0.000 in	Ratio =	0	<360
Max Downward Total Deflection	0.025 in	Ratio =	2888	>=180
Max Upward Total Deflection	0.000 in	Ratio =	0	<180

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios									Moment Values			Shear Values		
Segment Length	Span #	M	V	C_d	$C_{F/V}$	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 = 6.0 ft	1	0.183	0.086	0.90	1.000	0.80	1.00	1.00	1.00	1.00	0.61	88.77	486.00	0.30	8.64	100.80
+D+S					1.000	0.80	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 6.0 ft	1	0.367	0.172	1.15	1.000	0.80	1.00	1.00	1.00	1.00	1.57	227.80	621.00	0.77	22.16	128.80
+D+0.750S					1.000	0.80	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 6.0 ft	1	0.311	0.146	1.15	1.000	0.80	1.00	1.00	1.00	1.00	1.33	193.05	621.00	0.65	18.78	128.80
+0.60D					1.000	0.80	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 6.0 ft	1	0.062	0.029	1.60	1.000	0.80	1.00	1.00	1.00	1.00	0.37	53.26	864.00	0.18	5.18	179.20

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+S	1	0.0249	3.022		0.0000	0.000

Wood Beam

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DESCRIPTION: R1

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	1.047	1.047
Overall MINimum	0.639	0.639
D Only	0.408	0.408
+D+S	1.047	1.047
+D+0.750S	0.887	0.887
+0.60D	0.245	0.245
S Only	0.639	0.639

Wood Beam

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DESCRIPTION: R2

CODE REFERENCES

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

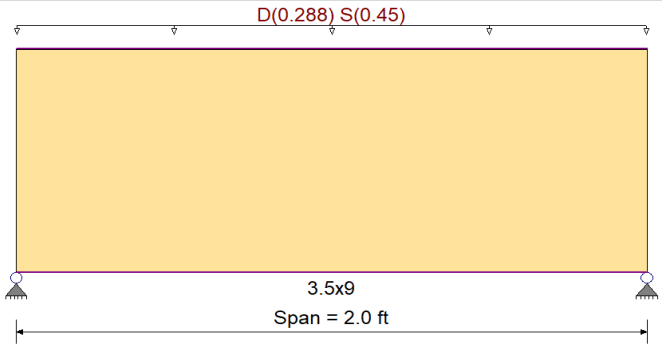
Material Properties

Analysis Method : Allowable Stress Design
Load Combination : ASCE 7-10

Wood Species : DF/DF
Wood Grade : 24F-V4

Beam Bracing : Beam is Fully Braced against lateral-torsional buckling

Fb +	2400 psi	*E : Modulus of Elasticity*	
Fb -	1850 psi	Ebend- xx	1800 ksi
Fc - Prll	1650 psi	Eminbend - xx	950 ksi
Fc - Perp	650 psi	Ebend- yy	1600 ksi
Fv	265 psi	Eminbend - yy	850 ksi
Ft	1100 psi	Density	31.21 pcf



Applied Loads

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

Uniform Load : D = 0.2880, S = 0.450 , Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.034 : 1	Maximum Shear Stress Ratio	=	0.029 : 1
Section used for this span		3.5x9	Section used for this span		3.5x9
	=	93.71 psi		=	8.98 psi
	=	2,760.00 psi		=	304.75 psi
Load Combination		+D+S	Load Combination		+D+S
Location of maximum on span	=	1.000 ft	Location of maximum on span	=	1.255 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.000 in	Ratio =	0 <360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.001 in	Ratio =	34372 >=180
Max Upward Total Deflection	0.000 in	Ratio =	0 <180

Maximum Forces & Stresses for Load Combinations

Load Combination / Segment Length	Span #	Max Stress Ratios M	Max Stress Ratios V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	Moment Values M	Moment Values fb	Moment Values F'b	Shear Values V	Shear Values fv	Shear Values F'v
D Only													0.00	0.00	0.00	0.00
Length = 2.0 ft	1	0.017	0.015	0.90	1.000	1.00	1.00	1.00	1.00	1.00	0.14	36.57	2160.00	0.07	3.50	238.50
+D+S					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 2.0 ft	1	0.034	0.029	1.15	1.000	1.00	1.00	1.00	1.00	1.00	0.37	93.71	2760.00	0.19	8.98	304.75
+D+0.750S					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 2.0 ft	1	0.029	0.025	1.15	1.000	1.00	1.00	1.00	1.00	1.00	0.31	79.43	2760.00	0.16	7.61	304.75
+0.60D					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 2.0 ft	1	0.006	0.005	1.60	1.000	1.00	1.00	1.00	1.00	1.00	0.09	21.94	3840.00	0.04	2.10	424.00

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+S	1	0.0007	1.007		0.0000	0.000

Wood Beam

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DESCRIPTION: R2

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	0.738	0.738
Overall MINimum	0.450	0.450
D Only	0.288	0.288
+D+S	0.738	0.738
+D+0.750S	0.626	0.626
+0.60D	0.173	0.173
S Only	0.450	0.450

Wood Beam

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DESCRIPTION: R3

CODE REFERENCES

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

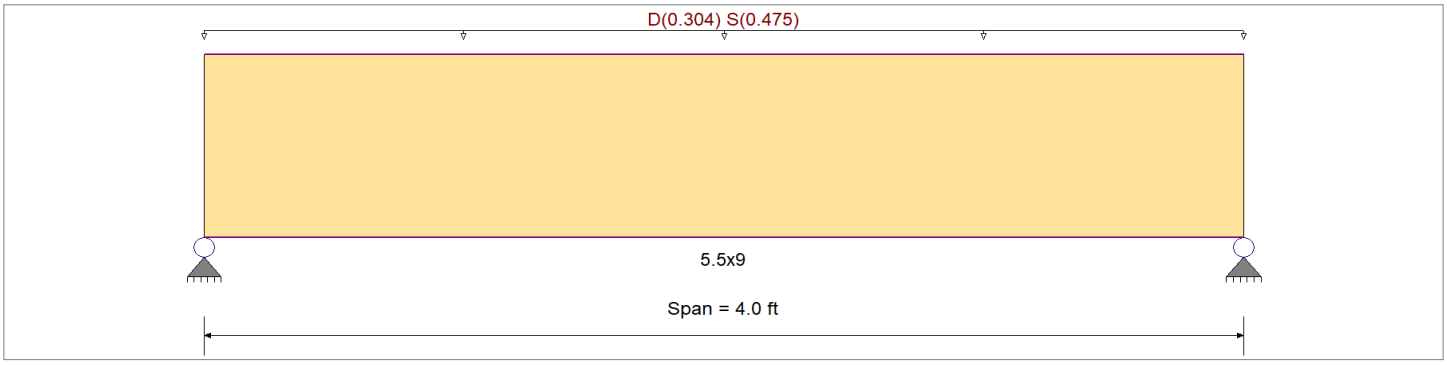
Material Properties

Analysis Method : Allowable Stress Design
Load Combination : ASCE 7-10

Wood Species : DF/DF
Wood Grade : 24F-V4

Beam Bracing : Beam is Fully Braced against lateral-torsional buckling

Fb +	2,400.0 psi	*E : Modulus of Elasticity*	
Fb -	1,850.0 psi	Ebend- xx	1,800.0 ksi
Fc - Prll	1,650.0 psi	Eminbend - xx	950.0 ksi
Fc - Perp	650.0 psi	Ebend- yy	1,600.0 ksi
Fv	265.0 psi	Eminbend - yy	850.0 ksi
Ft	1,100.0 psi	Density	31.210 pcf



Applied Loads

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

Uniform Load : D = 0.3040, S = 0.4750 , Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.091 : 1	Maximum Shear Stress Ratio	=	0.097 : 1
Section used for this span		5.5x9	Section used for this span		5.5x9
	=	251.80 psi		=	29.64 psi
	=	2,760.00 psi		=	304.75 psi
Load Combination		+D+S	Load Combination		+D+S
Location of maximum on span	=	2.000 ft	Location of maximum on span	=	3.255 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.005 in	Ratio =	10490	>=360
Max Upward Transient Deflection	0.000 in	Ratio =	0	<360
Max Downward Total Deflection	0.008 in	Ratio =	6396	>=180
Max Upward Total Deflection	0.000 in	Ratio =	0	<180

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios									Moment Values			Shear Values		
Segment Length	Span #	M	V	C_d	$C_{F/V}$	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 = 4.0 ft	1	0.045	0.048	0.90	1.000	1.00	1.00	1.00	1.00	1.00	0.61	98.26	2160.00	0.38	11.57	238.50
+D+S					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 4.0 ft	1	0.091	0.097	1.15	1.000	1.00	1.00	1.00	1.00	1.00	1.56	251.80	2760.00	0.98	29.64	304.75
+D+0.750S					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 4.0 ft	1	0.077	0.082	1.15	1.000	1.00	1.00	1.00	1.00	1.00	1.32	213.41	2760.00	0.83	25.12	304.75
+0.60D					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 4.0 ft	1	0.015	0.016	1.60	1.000	1.00	1.00	1.00	1.00	1.00	0.36	58.96	3840.00	0.23	6.94	424.00

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+S	1	0.0075	2.015		0.0000	0.000

Wood Beam

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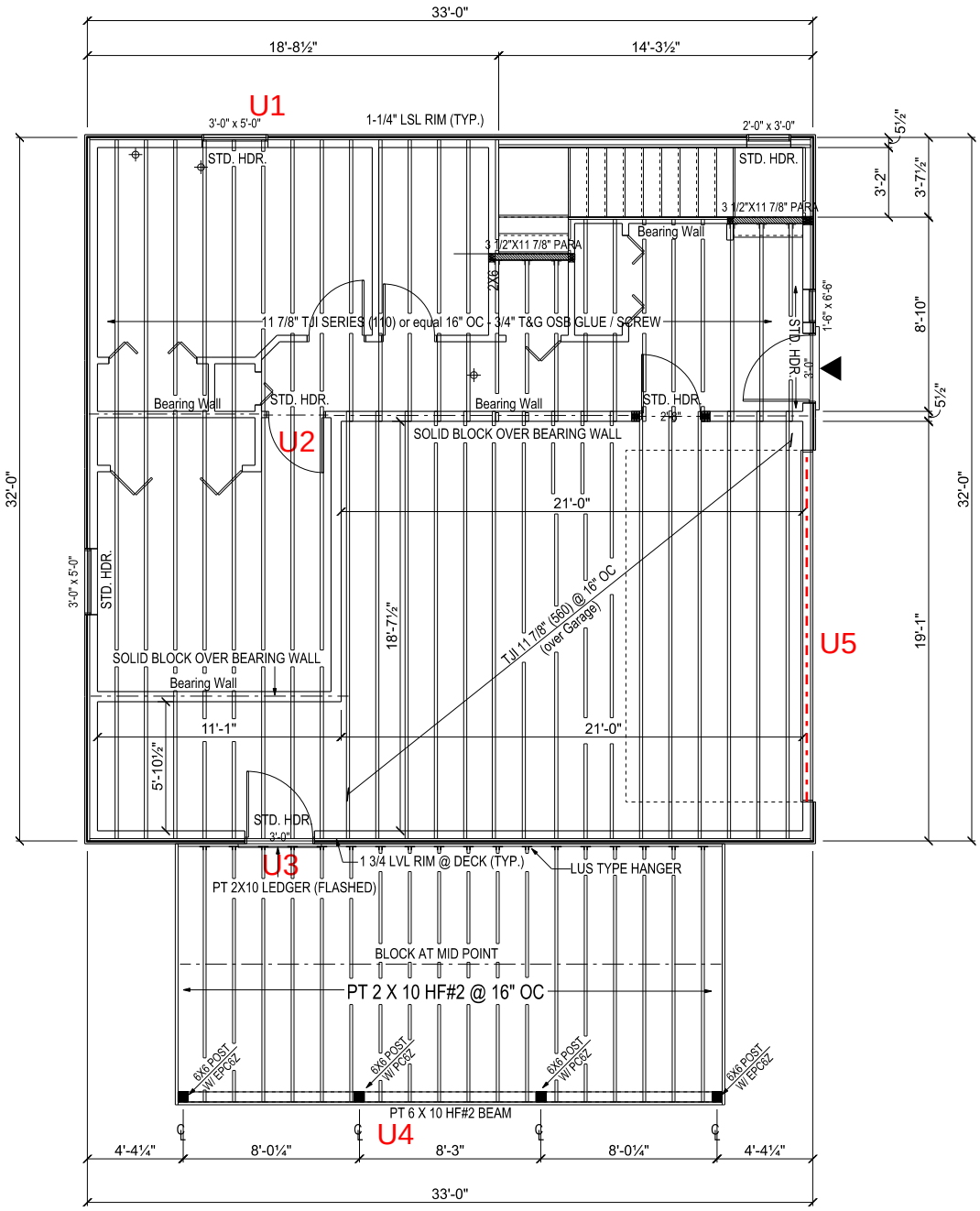
DESCRIPTION: R3

Vertical Reactions

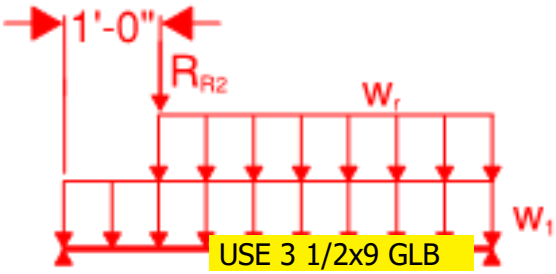
Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	1.558	1.558
Overall MINimum	0.950	0.950
D Only	0.608	0.608
+D+S	1.558	1.558
+D+0.750S	1.321	1.321
+0.60D	0.365	0.365
S Only	0.950	0.950



UPPER FLOOR PLAN
1/8" = 1'-0"

UPPER FLOOR FRAMING



U_1

$l := 3\ ft$

$w_r := [DL_r\ \ SL_r] \cdot (32\ ft \cdot 0.5 + 3\ ft) = [304\ \ 475]\ plf$

$w := [DL_{fl}\ \ LL_{fl}] \cdot (13\ ft \cdot 0.5) = [78\ \ 260]\ plf$

$R_{R2DL} := 288\ lbf$ $R_{R2SL} := 450\ lbf$ @ 1ft

$w_{wall} := 80\ plf$

USE 3 1/2x9 GLB

U_2

$l := 2.667\ ft$

$w := [DL_{fl}\ \ LL_{fl}] \cdot (25.5\ ft \cdot 0.5) = [153\ \ 510]\ plf$

USE 3 1/2x9 GLB

U_3

$l := 3\ ft$

$w := [DL_{fl}\ \ LL_d] \cdot (18.75\ ft \cdot 0.5) = [113\ \ 563]\ plf$

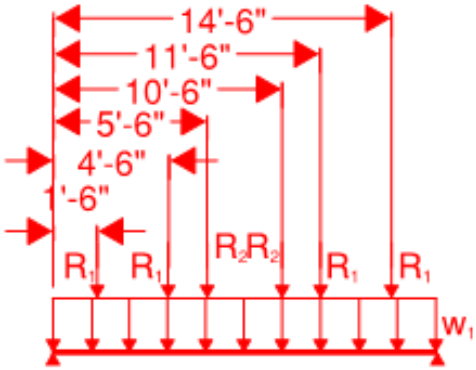
USE 3 1/2x9 GLB

U_4

$l := 8.25\ ft$

$w := [DL_{fl}\ \ LL_d] \cdot (12\ ft \cdot 0.5) = [72\ \ 360]\ plf$

USE 6x10 HF#2 P.T.



U_5

$l := 16\ ft$

$w := [DL_{fl}\ \ LL_{fl}] \cdot (2\ ft) = [24\ \ 80]\ plf$

$R_1 := [DL_r\ \ SL_r] \cdot (2\ ft) \cdot 3\ ft \cdot 0.5 = [48\ \ 75]\ lbf$ @ 1.5ft, 4.5ft, 11.5ft and 14.5ft

$R_2 := [DL_r\ \ SL_r] \cdot (2\ ft) \cdot 5\ ft \cdot 0.5 = [80\ \ 125]\ lbf$ @ 5.5ft, 10.5ft

USE 3 1/2x12 GLB

Deck Joist

$l := 12\ ft$ l = 11 ft. + 1 ft. Cantilever

$w_1 := [DL_{fl}\ \ LL_d] \cdot (16\ in) = [16\ \ 80]\ plf$

USE 2x10 DF#2 P.T. @ 16" O.C.

Joist Over Garage

$l := 19\ ft$

$w_1 := [DL_{fl}\ \ LL_{fl}] \cdot (16\ in) = [16\ \ 53]\ plf$

USE TJI 11 7/8" 560 @ 16" O.C.

Typical Joist

$l := 13\ ft$

$w_1 := [DL_{fl}\ \ LL_{fl}] \cdot (16\ in) = [16\ \ 53]\ plf$

USE TJI 11 7/8" 110 @ 16" O.C.

Wood Beam

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DESCRIPTION: U1

CODE REFERENCES

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

Material Properties

Analysis Method : Allowable Stress Design
Load Combination : ASCE 7-10

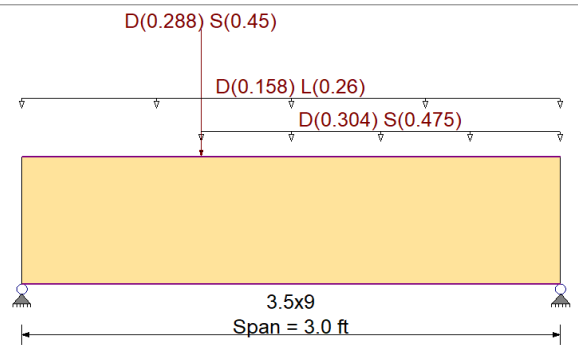
Wood Species : DF/DF
Wood Grade : 24F-V4

Beam Bracing : Beam is Fully Braced against lateral-torsional buckling

Fb +	2,400.0 psi
Fb -	1,850.0 psi
Fc - Prll	1,650.0 psi
Fc - Perp	650.0 psi
Fv	265.0 psi
Ft	1,100.0 psi

E : Modulus of Elasticity

Ebend- xx	1,800.0 ksi
Eminbend - xx	950.0 ksi
Ebend- yy	1,600.0 ksi
Eminbend - yy	850.0 ksi
Density	31.210 pcf



Applied Loads

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

Load for Span Number 1
Uniform Load : D = 0.3040, S = 0.4750 k/ft, Extent = 1.0 -->> 3.0 ft, Tributary Width = 1.0 ft
Uniform Load : D = 0.1580, L = 0.260 , Tributary Width = 1.0 ft
Point Load : D = 0.2880, S = 0.450 k @ 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.119 : 1	Maximum Shear Stress Ratio	=	0.177 : 1
Section used for this span		3.5x9	Section used for this span		3.5x9
	=	328.22 psi		=	53.84 psi
	=	2,760.00 psi		=	304.75 psi
Load Combination		+D+0.750L+0.750S	Load Combination		+D+S
Location of maximum on span	=	1.401 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	0.003 in	Ratio =	13370	>=360
Max Upward Transient Deflection	0.000 in	Ratio =	0	<360
Max Downward Total Deflection	0.005 in	Ratio =	6625	>=180
Max Upward Total Deflection	0.000 in	Ratio =	0	<180

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios									Moment Values			Shear Values		
Segment Length	Span #	M	V	C_d	$C_{F/V}$	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 = 3.0 ft	1	0.069	0.103	0.90	1.000	1.00	1.00	1.00	1.00	1.00	0.59	149.83	2160.00	0.51	24.48	238.50
+D+L					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 3.0 ft	1	0.093	0.128	1.00	1.000	1.00	1.00	1.00	1.00	1.00	0.88	223.92	2400.00	0.71	33.83	265.00
+D+S					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 3.0 ft	1	0.114	0.177	1.15	1.000	1.00	1.00	1.00	1.00	1.00	1.24	313.80	2760.00	1.13	53.84	304.75
+D+0.750L					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 3.0 ft	1	0.068	0.095	1.25	1.000	1.00	1.00	1.00	1.00	1.00	0.81	205.38	3000.00	0.66	31.49	331.25
+D+0.750L+0.750S					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 3.0 ft	1	0.119	0.176	1.15	1.000	1.00	1.00	1.00	1.00	1.00	1.29	328.22	2760.00	1.12	53.52	304.75
+0.60D					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00

Wood Beam

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DESCRIPTION: U1

Load Combination		Max Stress Ratios									Moment Values			Shear Values		
Segment Length	Span #	M	V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	M	fb	F'b	V	fv	F'v
Length = 3.0 ft	1	0.023	0.035	1.60	1.000	1.00	1.00	1.00	1.00	1.00	0.35	89.90	3840.00	0.31	14.69	424.00

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+0.750L+0.750S	1	0.0054	1.500		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	1.387	1.618
Overall MINimum	0.617	0.783
D Only	0.632	0.738
+D+L	1.022	1.128
+D+S	1.248	1.522
+D+0.750L	0.924	1.031
+D+0.750L+0.750S	1.387	1.618
+0.60D	0.379	0.443
L Only	0.390	0.390
S Only	0.617	0.783

Wood Beam

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DESCRIPTION: U2

CODE REFERENCES

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

Material Properties

Analysis Method : Allowable Stress Design
Load Combination : ASCE 7-10

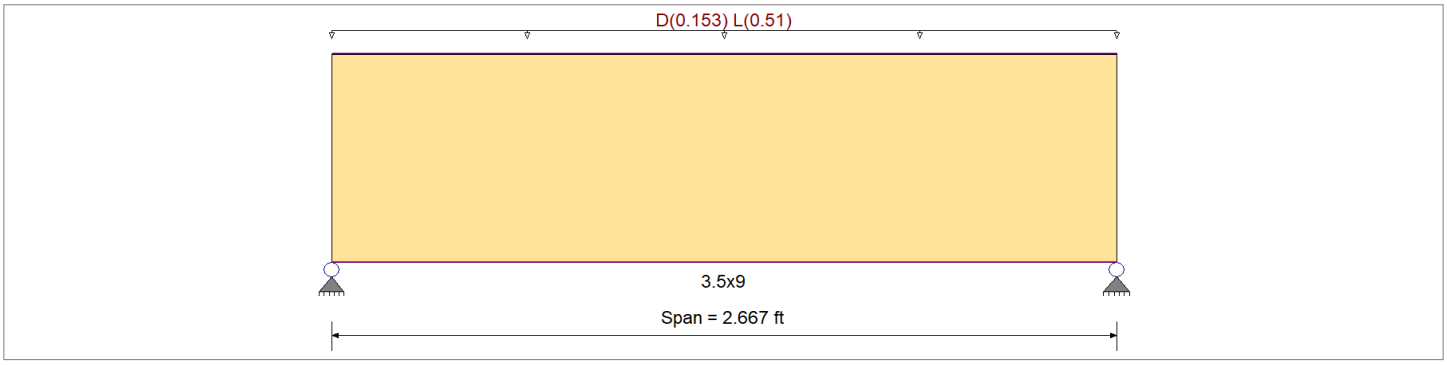
Wood Species : DF/DF
Wood Grade : 24F-V4

Beam Bracing : Beam is Fully Braced against lateral-torsional buckling

Fb +	2,400.0 psi
Fb -	1,850.0 psi
Fc - Prll	1,650.0 psi
Fc - Perp	650.0 psi
Fv	265.0 psi
Ft	1,100.0 psi

E : Modulus of Elasticity

Ebend- xx	1,800.0 ksi
Eminbend - xx	950.0 ksi
Ebend- yy	1,600.0 ksi
Eminbend - yy	850.0 ksi
Density	31.210 pcf



Applied Loads

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

Uniform Load : D = 0.1530, L = 0.510 , Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.062 : 1	Maximum Shear Stress Ratio	=	0.070 : 1
Section used for this span		3.5x9	Section used for this span		3.5x9
	=	149.71 psi		=	18.44 psi
	=	2,400.00 psi		=	265.00 psi
Load Combination		+D+L	Load Combination		+D+L
Location of maximum on span	=	1.334 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	0.002 in	Ratio =	20975	>=360
Max Upward Transient Deflection	0.000 in	Ratio =	0	<360
Max Downward Total Deflection	0.002 in	Ratio =	16135	>=180
Max Upward Total Deflection	0.000 in	Ratio =	0	<180

Maximum Forces & Stresses for Load Combinations

Load Combination / Segment Length	Span #	Max Stress Ratios M	Max Stress Ratios V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	Moment Values M	Moment Values fb	Moment Values F'b	Shear Values V	Shear Values fv	Shear Values F'v
D Only													0.00	0.00	0.00	0.00
Length = 2.667 ft	1	0.016	0.018	0.90	1.000	1.00	1.00	1.00	1.00	1.00	0.14	34.55	2160.00	0.09	4.25	238.50
+D+L					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 2.667 ft	1	0.062	0.070	1.00	1.000	1.00	1.00	1.00	1.00	1.00	0.59	149.71	2400.00	0.39	18.44	265.00
+D+0.750L					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 2.667 ft	1	0.040	0.045	1.25	1.000	1.00	1.00	1.00	1.00	1.00	0.48	120.92	3000.00	0.31	14.89	331.25
+0.60D					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 2.667 ft	1	0.005	0.006	1.60	1.000	1.00	1.00	1.00	1.00	1.00	0.08	20.73	3840.00	0.05	2.55	424.00

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+L	1	0.0020	1.343		0.0000	0.000

Wood Beam

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DESCRIPTION: U2

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	0.884	0.884
Overall MINimum	0.680	0.680
D Only	0.204	0.204
+D+L	0.884	0.884
+D+0.750L	0.714	0.714
+0.60D	0.122	0.122
L Only	0.680	0.680

Wood Beam

File = O:\18700\18720\Calculations\StructuralStructural .
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DESCRIPTION: U3

CODE REFERENCES

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

Material Properties

Analysis Method : Allowable Stress Design
Load Combination :ASCE 7-10

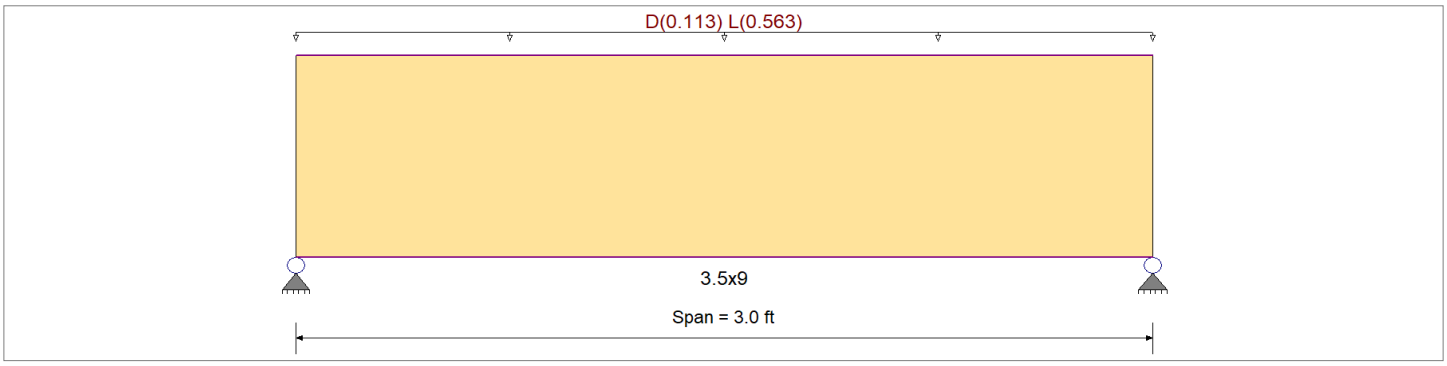
Wood Species : DF/DF
Wood Grade : 24F-V4

Beam Bracing : Beam is Fully Braced against lateral-torsional buckling

Fb +	2,400.0 psi
Fb -	1,850.0 psi
Fc - Prll	1,650.0 psi
Fc - Perp	650.0 psi
Fv	265.0 psi
Ft	1,100.0 psi

E : Modulus of Elasticity

Ebend- xx	1,800.0 ksi
Eminbend - xx	950.0 ksi
Ebend- yy	1,600.0 ksi
Eminbend - yy	850.0 ksi
Density	31.210 pcf



Applied Loads

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

Uniform Load : D = 0.1130, L = 0.5630 , Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.080 : 1	Maximum Shear Stress Ratio	=	0.092 : 1
Section used for this span		3.5x9	Section used for this span		3.5x9
	=	193.14 psi		=	24.32 psi
	=	2,400.00 psi		=	265.00 psi
Load Combination		+D+L	Load Combination		+D+L
Location of maximum on span	=	1.500 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	0.003 in	Ratio =	12682 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.003 in	Ratio =	10562 >=180
Max Upward Total Deflection	0.000 in	Ratio =	0 <180

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios									Moment Values			Shear Values		
Segment Length	Span #	M	V	C_d	$C_{F/V}$	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 = 3.0 ft	1	0.015	0.017	0.90	1.000	1.00	1.00	1.00	1.00	1.00	0.13	32.29	2160.00	0.09	4.07	238.50
+D+L					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 3.0 ft	1	0.080	0.092	1.00	1.000	1.00	1.00	1.00	1.00	1.00	0.76	193.14	2400.00	0.51	24.32	265.00
+D+0.750L					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 3.0 ft	1	0.051	0.058	1.25	1.000	1.00	1.00	1.00	1.00	1.00	0.60	152.93	3000.00	0.40	19.26	331.25
+0.60D					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 3.0 ft	1	0.005	0.006	1.60	1.000	1.00	1.00	1.00	1.00	1.00	0.08	19.37	3840.00	0.05	2.44	424.00

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+L	1	0.0034	1.511		0.0000	0.000

Wood Beam

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DESCRIPTION: U3

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	1.014	1.014
Overall MINimum	0.845	0.845
D Only	0.170	0.170
+D+L	1.014	1.014
+D+0.750L	0.803	0.803
+0.60D	0.102	0.102
L Only	0.845	0.845

Wood Beam

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DESCRIPTION: U4

CODE REFERENCES

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

Material Properties

Analysis Method : Allowable Stress Design
Load Combination : ASCE 7-10

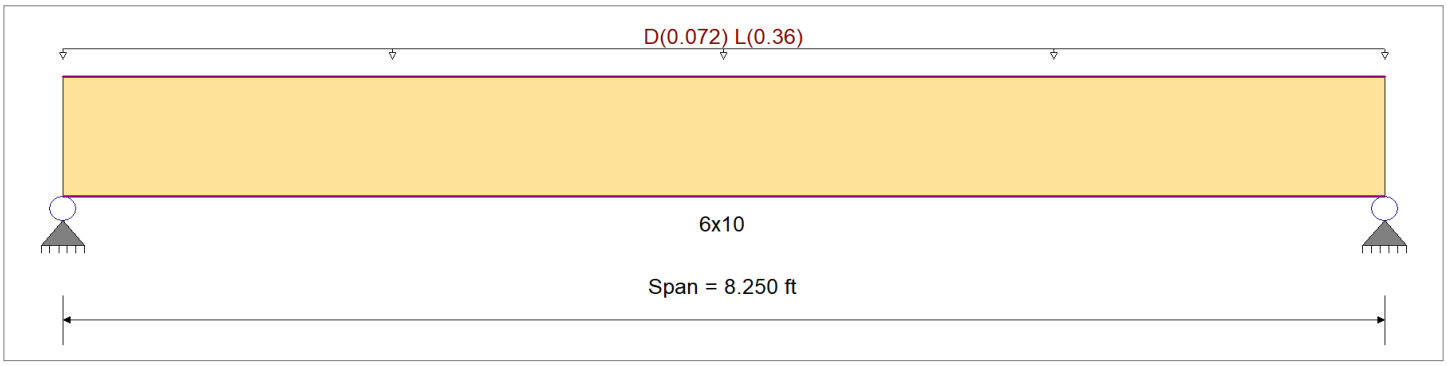
Wood Species : Hem-Fir
Wood Grade : No.2

Beam Bracing : Beam is Fully Braced against lateral-torsional buckling

Fb +	675.0 psi
Fb -	675.0 psi
Fc - Prll	500.0 psi
Fc - Perp	405.0 psi
Fv	140.0 psi
Ft	350.0 psi

E : Modulus of Elasticity

Ebend- xx	1,100.0 ksi
Eminbend - xx	400.0 ksi
Density	26.840 pcf



Applied Loads

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

Uniform Load : D = 0.0720, L = 0.360 , Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.987 : 1	Maximum Shear Stress Ratio	=	0.370 : 1
Section used for this span		6x10	Section used for this span		6x10
	=	533.12 psi		=	41.45 psi
	=	540.00 psi		=	112.00 psi
Load Combination		+D+L	Load Combination		+D+L
Location of maximum on span	=	4.125 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	0.092 in	Ratio =	1077	>=360
Max Upward Transient Deflection	0.000 in	Ratio =	0	<360
Max Downward Total Deflection	0.110 in	Ratio =	897	>=180
Max Upward Total Deflection	0.000 in	Ratio =	0	<180

Maximum Forces & Stresses for Load Combinations

Load Combination / Segment Length	Span #	Max Stress Ratios M	Max Stress Ratios V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	Moment Values M	Moment Values fb	Moment Values F'b	Shear Values V	Shear Values fv	Shear Values F'v
D Only													0.00	0.00	0.00	0.00
Length = 8.250 ft	1	0.183	0.069	0.90	1.000	0.80	1.00	1.00	1.00	1.00	0.61	88.85	486.00	0.24	6.91	100.80
+D+L					1.000	0.80	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 8.250 ft	1	0.987	0.370	1.00	1.000	0.80	1.00	1.00	1.00	1.00	3.68	533.12	540.00	1.44	41.45	112.00
+D+0.750L					1.000	0.80	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 8.250 ft	1	0.625	0.234	1.25	1.000	0.80	1.00	1.00	1.00	1.00	2.91	422.05	675.00	1.14	32.81	140.00
+0.60D					1.000	0.80	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 8.250 ft	1	0.062	0.023	1.60	1.000	0.80	1.00	1.00	1.00	1.00	0.37	53.31	864.00	0.14	4.14	179.20

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+L	1	0.1103	4.155		0.0000	0.000

Wood Beam

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DESCRIPTION: U4

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	1.782	1.782
Overall MINimum	1.485	1.485
D Only	0.297	0.297
+D+L	1.782	1.782
+D+0.750L	1.411	1.411
+0.60D	0.178	0.178
L Only	1.485	1.485

Wood Beam

File = O:\18700\18720\Calculations\Structural\18720.ec6 .
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DESCRIPTION: U5

CODE REFERENCES

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

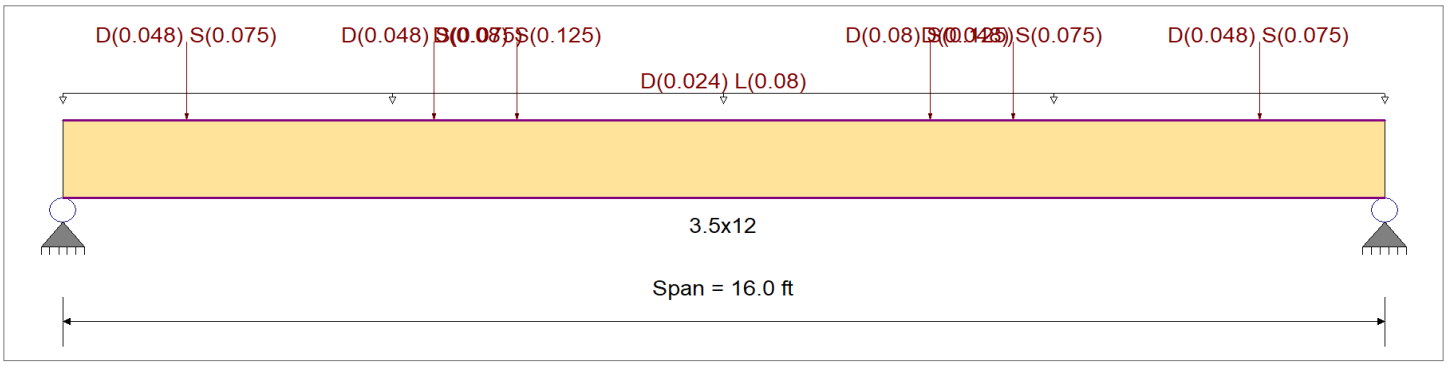
Material Properties

Analysis Method : Allowable Stress Design
Load Combination : ASCE 7-10

Wood Species : DF/DF
Wood Grade : 24F-V4

Beam Bracing : Beam is Fully Braced against lateral-torsional buckling

Fb +	2400 psi	*E : Modulus of Elasticity*	
Fb -	1850 psi	Ebend- xx	1800 ksi
Fc - Prll	1650 psi	Eminbend - xx	950 ksi
Fc - Perp	650 psi	Ebend- yy	1600 ksi
Fv	265 psi	Eminbend - yy	850 ksi
Ft	1100 psi	Density	31.21 pcf



Applied Loads

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

Uniform Load : D = 0.0240, L = 0.080 , Tributary Width = 1.0 ft
Point Load : D = 0.0480, S = 0.0750 k @ 1.50 ft
Point Load : D = 0.0480, S = 0.0750 k @ 4.50 ft
Point Load : D = 0.0480, S = 0.0750 k @ 11.50 ft
Point Load : D = 0.0480, S = 0.0750 k @ 14.50 ft
Point Load : D = 0.080, S = 0.1250 k @ 5.50 ft
Point Load : D = 0.080, S = 0.1250 k @ 10.50 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.241 : 1	Maximum Shear Stress Ratio	=	0.122 : 1
Section used for this span		3.5x12	Section used for this span		3.5x12
	=	579.43 psi		=	32.31 psi
	=	2,400.00 psi		=	265.00 psi
Load Combination		+D+L	Load Combination		+D+L
Location of maximum on span	=	8.000 ft	Location of maximum on span	=	15.007 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.131 in	Ratio =	1467	>=360
Max Upward Transient Deflection	0.000 in	Ratio =	0	<360
Max Downward Total Deflection	0.222 in	Ratio =	865	>=180
Max Upward Total Deflection	0.000 in	Ratio =	0	<180

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios									Moment Values			Shear Values		
Segment Length	Span #	M	V	C_d	$C_{F/V}$	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 = 16.0 ft	1	0.099	0.052	0.90	1.000	1.00	1.00	1.00	1.00	1.00	1.50	213.71	2160.00	0.34	12.29	238.50
+D+L					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 16.0 ft	1	0.241	0.122	1.00	1.000	1.00	1.00	1.00	1.00	1.00	4.06	579.43	2400.00	0.90	32.31	265.00
+D+S					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 16.0 ft	1	0.136	0.073	1.15	1.000	1.00	1.00	1.00	1.00	1.00	2.63	376.21	2760.00	0.62	22.11	304.75
+D+0.750L					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 16.0 ft	1	0.163	0.082	1.25	1.000	1.00	1.00	1.00	1.00	1.00	3.42	488.00	3000.00	0.76	27.31	331.25

Wood Beam

File = O:\18700\18720\Calculations\Structural\18720.ec6 .
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DESCRIPTION: U5

Load Combination		Max Stress Ratios									Moment Values			Shear Values		
Segment Length	Span #	M	V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	M	fb	F'b	V	fv	F'v
+D+0.750L+0.750S					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 16.0 ft	1	0.221	0.114	1.15	1.000	1.00	1.00	1.00	1.00	1.00	4.27	609.88	2760.00	0.97	34.67	304.75
+0.60D					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 16.0 ft	1	0.033	0.017	1.60	1.000	1.00	1.00	1.00	1.00	1.00	0.90	128.23	3840.00	0.21	7.38	424.00

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+0.750L+0.750S	1	0.2219	8.058		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	1.054	1.054
Overall MINimum	0.275	0.275
D Only	0.368	0.368
+D+L	1.008	1.008
+D+S	0.643	0.643
+D+0.750L	0.848	0.848
+D+0.750L+0.750S	1.054	1.054
+0.60D	0.221	0.221
L Only	0.640	0.640
S Only	0.275	0.275

Sitts & Hill Engineers, Inc.
4815 Center Street
Tacoma, WA 98409

Project Title:
Engineer:
Project ID:
Project Descr:

Title Block Line 6

Printed: 6 FEB 2020, 2:47PM

Wood Beam

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DESCRIPTION: Deck Joist

CODE REFERENCES

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

Material Properties

Analysis Method : Allowable Stress Design
Load Combination :ASCE 7-10

Wood Species : Hem-Fir
Wood Grade : No.2

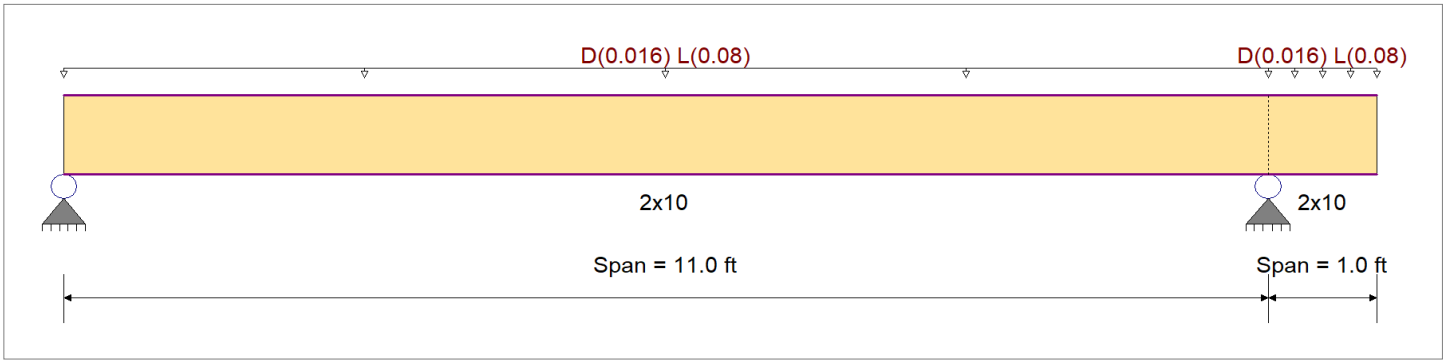
Beam Bracing : Beam is Fully Braced against lateral-torsional buckling

Fb +	850.0 psi
Fb -	850.0 psi
Fc - Prll	1,300.0 psi
Fc - Perp	405.0 psi
Fv	150.0 psi
Ft	525.0 psi

E : Modulus of Elasticity

Ebend- xx	1,300.0 ksi
Eminbend - xx	470.0 ksi
Density	26.840 pcf

Repetitive Member Stress Increase



Applied Loads

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

Load for Span Number 1
Uniform Load : D = 0.0160, L = 0.080 , Tributary Width = 1.0 ft
Load for Span Number 2
Uniform Load : D = 0.0160, L = 0.080 , Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.810 : 1	Maximum Shear Stress Ratio	=	0.416 : 1
Section used for this span		2x10	Section used for this span		2x10
	=	801.15 psi		=	49.90 psi
	=	989.23 psi		=	120.00 psi
Load Combination		+D+L	Load Combination		+D+L
Location of maximum on span	=	5.469 ft	Location of maximum on span	=	10.263 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.213 in	Ratio =	618 >=360
Max Upward Transient Deflection	-0.061 in	Ratio =	396 >=360
Max Downward Total Deflection	0.256 in	Ratio =	515 >=180
Max Upward Total Deflection	-0.073 in	Ratio =	330 >=180

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios									Moment Values			Shear Values		
Segment Length	Span #	M	V	C_d	$C_{F/V}$	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 = 11.0 ft	1	0.150	0.077	0.90	1.100	0.80	1.15	1.00	1.00	1.00	0.24	133.52	890.31	0.08	8.32	108.00
Length = 1.0 ft	2	0.005	0.077	0.90	1.100	0.80	1.15	1.00	1.00	1.00	0.01	4.49	890.31	0.00	8.32	108.00
+D+L					1.100	0.80	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 11.0 ft	1	0.810	0.416	1.00	1.100	0.80	1.15	1.00	1.00	1.00	1.43	801.15	989.23	0.46	49.90	120.00
Length = 1.0 ft	2	0.027	0.416	1.00	1.100	0.80	1.15	1.00	1.00	1.00	0.05	26.93	989.23	0.02	49.90	120.00
+D+0.750L					1.100	0.80	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 11.0 ft	1	0.513	0.263	1.25	1.100	0.80	1.15	1.00	1.00	1.00	1.13	634.24	1236.54	0.37	39.50	150.00
Length = 1.0 ft	2	0.017	0.263	1.25	1.100	0.80	1.15	1.00	1.00	1.00	0.04	21.32	1236.54	0.02	39.50	150.00
+0.60D					1.100	0.80	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 11.0 ft	1	0.051	0.026	1.60	1.100	0.80	1.15	1.00	1.00	1.00	0.14	80.11	1582.77	0.05	4.99	192.00

Permit Number: 20-00976

Sitts & Hill Engineers, Inc.
4815 Center Street
Tacoma, WA 98409

Project Title:
Engineer:
Project ID:
Project Descr:

Title Block Line 6

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Wood Beam

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DESCRIPTION: Deck Joist

Load Combination		Max Stress Ratios									Moment Values			Shear Values		
Segment Length	Span #	M	V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	M	fb	F'b	V	fv	F'v
Length = 1.0 ft	2	0.002	0.026	1.60	1.100	0.80	1.15	1.00	1.00	1.00	0.00	2.69	1582.77	0.00	4.99	192.00

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+L	1	0.2560	5.531		0.0000	0.000
	2	0.0000	5.531	+D+L	-0.0726	1.000

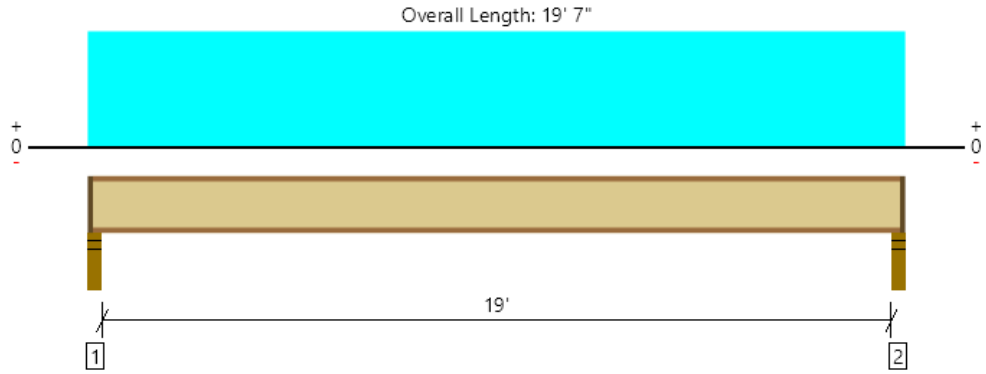
Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2	Support 3
Overall MAXimum	0.524	0.628	
Overall MINimum	0.436	0.524	
D Only	0.087	0.105	
+D+L	0.524	0.628	
+D+0.750L	0.415	0.497	
+0.60D	0.052	0.063	
L Only	0.436	0.524	

Permit Number: 20-00976

Level, Joist Over Garage
1 piece(s) 11 7/8" TJI® 560 @ 16" OC



All locations are measured from the outside face of left support (or left cantilever end). All dimensions are horizontal.

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	672 @ 2 1/2"	1396 (2.25")	Passed (48%)	1.00	1.0 D + 1.0 L (All Spans)
Shear (lbs)	659 @ 3 1/2"	2050	Passed (32%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	3184 @ 9' 9 1/2"	9500	Passed (34%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.262 @ 9' 9 1/2"	0.479	Passed (L/877)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.341 @ 9' 9 1/2"	0.958	Passed (L/675)	--	1.0 D + 1.0 L (All Spans)
TJ-Pro™ Rating	51	40	Passed	--	--

System : Floor
Member Type : Joist
Building Use : Residential
Building Code : IBC 2015
Design Methodology : ASD

- Deflection criteria: LL (L/480) and TL (L/240).
- Top Edge Bracing (Lu): Top compression edge must be braced at 10' o/c based on loads applied, unless detailed otherwise.
- Bottom Edge Bracing (Lu): Bottom compression edge must be braced at 19' 5" o/c based on loads applied, unless detailed otherwise.
- A structural analysis of the deck has not been performed.
- Deflection analysis is based on composite action with a single layer of decking_2332Panels that is gluedAndNailedDown.
- Additional considerations for the TJ-Pro™ Rating include: 5/8" Gypsum ceiling.

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Floor Live	Total	
1 - Stud wall - SPF	3.50"	2.25"	1.75"	157	522	679	1 1/4" Rim Board
2 - Stud wall - SPF	3.50"	2.25"	1.75"	157	522	679	1 1/4" Rim Board

- Rim Board is assumed to carry all loads applied directly above it, bypassing the member being designed.

Vertical Load	Location (Side)	Spacing	Dead (0.90)	Floor Live (1.00)	Comments
1 - Uniform (PSF)	0 to 19' 7"	16"	12.0	40.0	Default Load

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The product application, input design loads, dimensions and support information have been provided by ForteWEB Software Operator

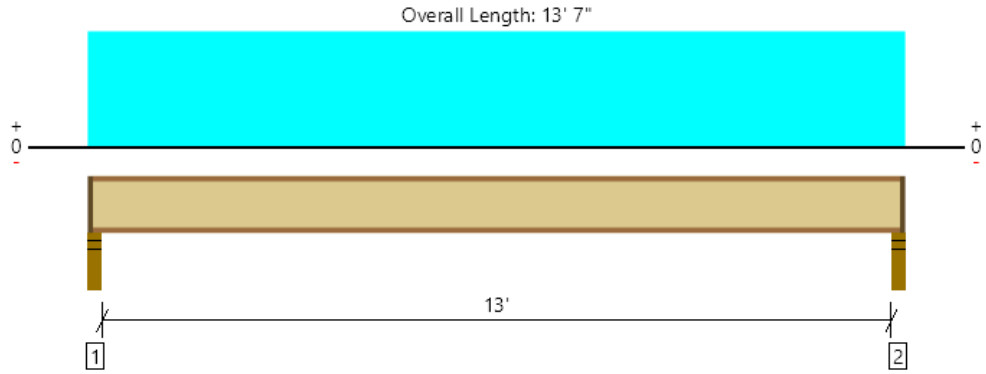
ForteWEB Software Operator	Job Notes
Solomon Schlenker Sitts & Hill Engineers (253) 474-9449 solomons@sittshill.com	

Permit Number: 20-00976

Weyerhaeuser

2/3/2020 11:46:27 PM UTC
ForteWEB v2.3, Engine: V8.0.0.21, Data: V7.3.1.1
File Name: 18720

Level, Typical Upper Floor Joist
Current Solution: 1 piece(s) 11 7/8" TJI® 110 @ 16" OC



All locations are measured from the outside face of left support (or left cantilever end). All dimensions are horizontal.

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	464 @ 2 1/2"	1041 (2.25")	Passed (45%)	1.00	1.0 D + 1.0 L (All Spans)
Shear (lbs)	451 @ 3 1/2"	1560	Passed (29%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	1502 @ 6' 9 1/2"	3160	Passed (48%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.128 @ 6' 9 1/2"	0.329	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.167 @ 6' 9 1/2"	0.658	Passed (L/948)	--	1.0 D + 1.0 L (All Spans)
TJ-Pro™ Rating	52	40	Passed	--	--

System : Floor
Member Type : Joist
Building Use : Residential
Building Code : IBC 2015
Design Methodology : ASD

All Product Solutions					
Depth	Series	Plies	Spacing	**TJ-Pro™ Rating**	Wood Volume
11 7/8"	TJI® 110	1	16"	52	0.69

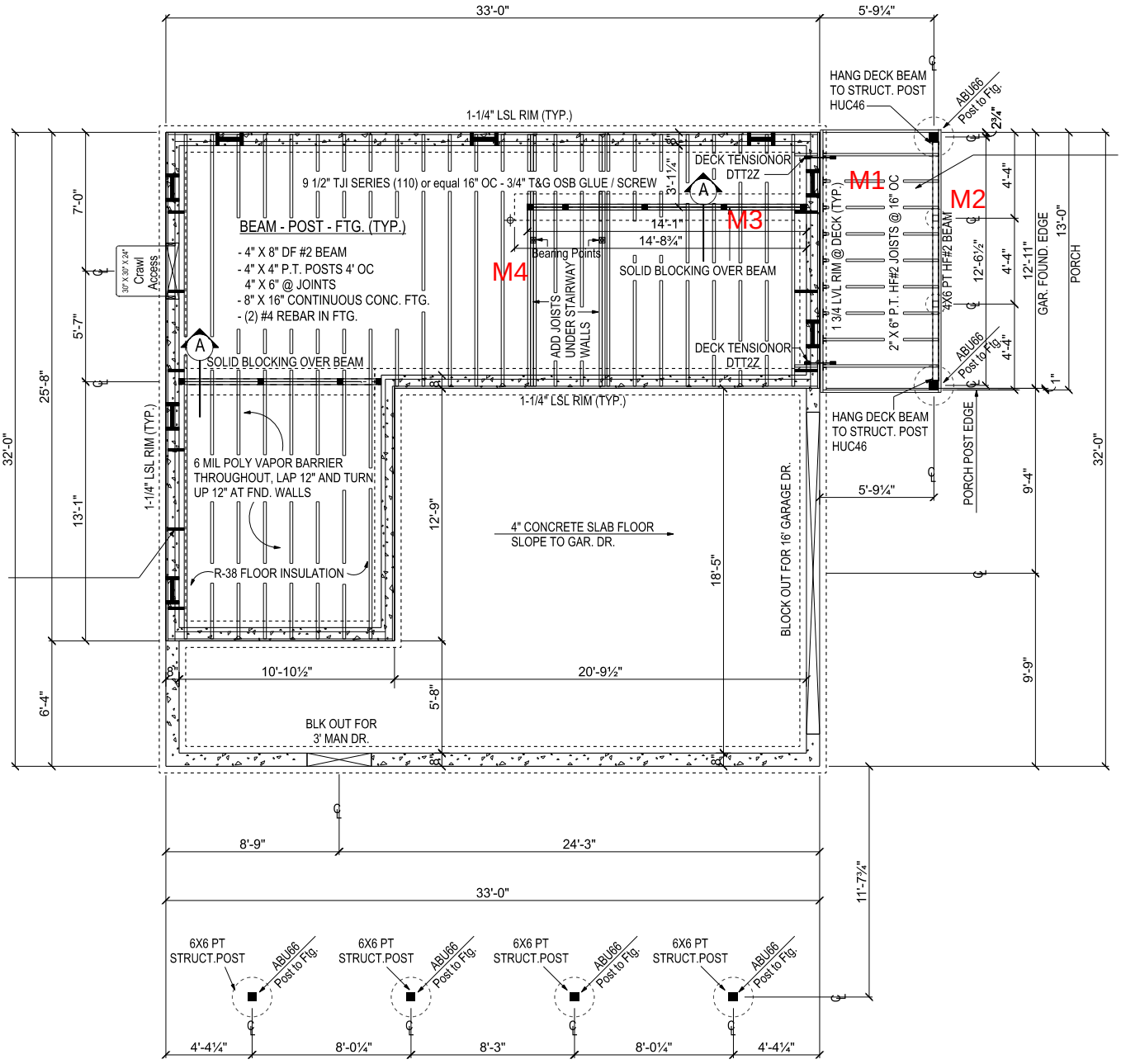
The purpose of this report is for product comparison only. Load and support information necessary for professional design review is not displayed here. Please print an individual Member Report for submittal purposes.

ForteWEB Software Operator	Job Notes
Solomon Schlenker Sitts & Hill Engineers (253) 474-9449 solomons@sittshill.com	

Weyerhaeuser

Permit Number: 20-00976

2/3/2020 11:59:39 PM UTC
ForteWEB v2.3, Engine: V8.0.0.21, Data: V7.3.1.1
File Name: 18720



MAIN FLOOR PLAN
1/8" = 1'-0"

MAIN FLOOR FRAMING

M_1

$l := 5.75\ ft$

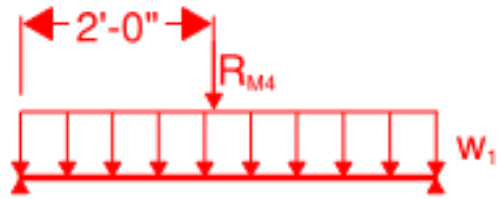
$w := [DL_{fl}\ \ LL_d] \cdot 16\ in = [16\ \ 80]\ plf$ USE 2x6 HF#2 P.T. @ 16" O.C.

M_2

$l := 4.33\ ft$

$w := [DL_{fl}\ \ LL_d] \cdot (5.75\ ft \cdot 0.5) = [35\ \ 173]\ plf$ USE 4x6 HF#2 P.T.

M_3



$l := 4\ ft$

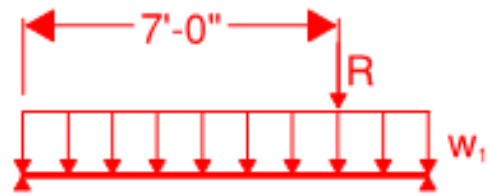
$w := [DL_{fl}\ \ LL_{fl}] \cdot (16\ in) = [16\ \ 53]\ plf$

$w := [DL_{fl}\ \ LL_{fl}] \cdot (9\ ft \cdot 0.5) = [54\ \ 180]\ plf$ Bearing Wall Above USE (2) TJI 9 1/2" 110

$R_{M4DL} := 481\ lbf$ $R_{M4LL} := 1602\ lbf$ @ 2 ft

$w_{wall} := 80\ plf$

M_4



$l := 9\ ft$

$w := [DL_{fl}\ \ LL_{fl}] \cdot (16\ in) = [16\ \ 53]\ plf$

$R := [DL_{fl}\ \ LL_{fl}] \cdot (7\ ft \cdot 0.5) \cdot 3.5\ ft \cdot 0.5 = [74\ \ 245]\ lbf$ @ 7 ft USE (2) TJI 9 1/2" 110

Typical Joist

$l := 14\ ft$

$w_1 := [DL_{fl}\ \ LL_{fl}] \cdot (16\ in) = [16\ \ 53]\ plf$ USE TJI 9 1/2 110 @ 16" O.C.

Wood Beam

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DESCRIPTION: M1

CODE REFERENCES

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

Material Properties

Analysis Method : Allowable Stress Design
Load Combination : ASCE 7-10

Wood Species : Hem-Fir
Wood Grade : No.2

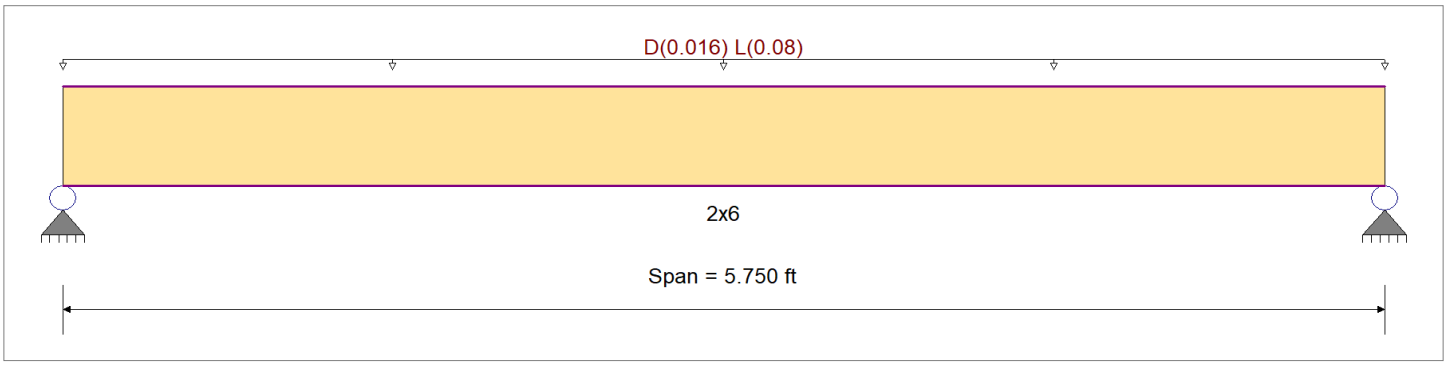
Beam Bracing : Beam is Fully Braced against lateral-torsional buckling

Fb +	675.0 psi
Fb -	675.0 psi
Fc - Prll	500.0 psi
Fc - Perp	405.0 psi
Fv	140.0 psi
Ft	350.0 psi

E : Modulus of Elasticity

Ebend- xx	1,100.0 ksi
Eminbend - xx	400.0 ksi
Density	26.840 pcf

Repetitive Member Stress Increase



Applied Loads

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

Uniform Load : D = 0.0160, L = 0.080 , Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.678 : 1	Maximum Shear Stress Ratio	=	0.379 : 1
Section used for this span		2x6	Section used for this span		2x6
	=	629.55 psi		=	42.49 psi
	=	928.40 psi		=	112.00 psi
Load Combination		+D+L	Load Combination		+D+L
Location of maximum on span	=	2.875 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	0.091 in	Ratio =	757 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.109 in	Ratio =	631 >=180
Max Upward Total Deflection	0.000 in	Ratio =	0 <180

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios									Moment Values			Shear Values		
Segment Length	Span #	M	V	C_d	$C_{F/V}$	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 = 5.750 ft	1	0.126	0.070	0.90	1.300	0.80	1.15	1.00	1.00	1.00	0.07	104.93	835.56	0.04	7.08	100.80
+D+L					1.300	0.80	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 5.750 ft	1	0.678	0.379	1.00	1.300	0.80	1.15	1.00	1.00	1.00	0.40	629.55	928.40	0.23	42.49	112.00
+D+0.750L					1.300	0.80	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 5.750 ft	1	0.429	0.240	1.25	1.300	0.80	1.15	1.00	1.00	1.00	0.31	498.40	1160.49	0.19	33.64	140.00
+0.60D					1.300	0.80	1.15	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 5.750 ft	1	0.042	0.024	1.60	1.300	0.80	1.15	1.00	1.00	1.00	0.04	62.96	1485.43	0.02	4.25	179.20

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+L	1	0.1093	2.896		0.0000	0.000

Wood Beam

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DESCRIPTION: M1

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	0.276	0.276
Overall MINimum	0.230	0.230
D Only	0.046	0.046
+D+L	0.276	0.276
+D+0.750L	0.219	0.219
+0.60D	0.028	0.028
L Only	0.230	0.230

Wood Beam

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DESCRIPTION: M2

CODE REFERENCES

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

Material Properties

Analysis Method : Allowable Stress Design
Load Combination : ASCE 7-10

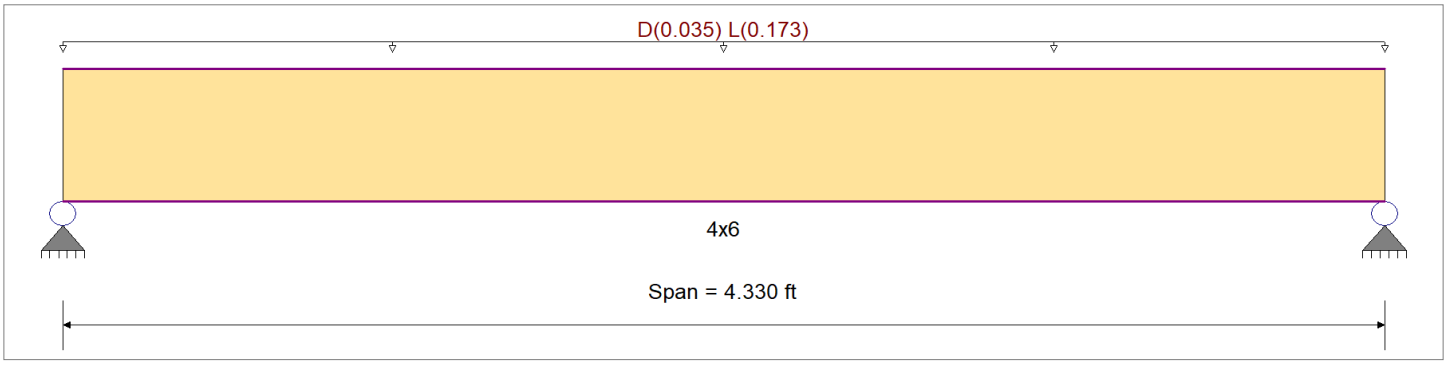
Wood Species : Hem-Fir
Wood Grade : No.2

Beam Bracing : Beam is Fully Braced against lateral-torsional buckling

Fb +	675.0 psi
Fb -	675.0 psi
Fc - Prll	500.0 psi
Fc - Perp	405.0 psi
Fv	140.0 psi
Ft	350.0 psi

E : Modulus of Elasticity

Ebend- xx	1,100.0 ksi
Eminbend - xx	400.0 ksi
Density	26.840 pcf



Applied Loads

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

Uniform Load : D = 0.0350, L = 0.1730 , Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.472 : 1	Maximum Shear Stress Ratio	=	0.247 : 1
Section used for this span		4x6	Section used for this span		4x6
	=	331.50 psi		=	27.66 psi
	=	702.00 psi		=	112.00 psi
Load Combination		+D+L	Load Combination		+D+L
Location of maximum on span	=	2.165 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	0.027 in	Ratio =	1914 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.033 in	Ratio =	1592 >=180
Max Upward Total Deflection	0.000 in	Ratio =	0 <180

Maximum Forces & Stresses for Load Combinations

Load Combination / Segment Length	Span #	Max Stress Ratios M	Max Stress Ratios V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	Moment Values M	Moment Values fb	Moment Values F'b	Shear Values V	Shear Values fv	Shear Values F'v
D Only													0.00	0.00	0.00	0.00
Length = 4.330 ft	1	0.088	0.046	0.90	1.300	0.80	1.00	1.00	1.00	1.00	0.08	55.78	631.80	0.06	4.65	100.80
+D+L					1.300	0.80	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 4.330 ft	1	0.472	0.247	1.00	1.300	0.80	1.00	1.00	1.00	1.00	0.49	331.50	702.00	0.35	27.66	112.00
+D+0.750L					1.300	0.80	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 4.330 ft	1	0.299	0.157	1.25	1.300	0.80	1.00	1.00	1.00	1.00	0.39	262.57	877.50	0.28	21.91	140.00
+0.60D					1.300	0.80	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 4.330 ft	1	0.030	0.016	1.60	1.300	0.80	1.00	1.00	1.00	1.00	0.05	33.47	1123.20	0.04	2.79	179.20

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+L	1	0.0326	2.181		0.0000	0.000

Wood Beam

File = O:\18700\18720\Calculations\Structural\18720.ec6 .
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DESCRIPTION: M2

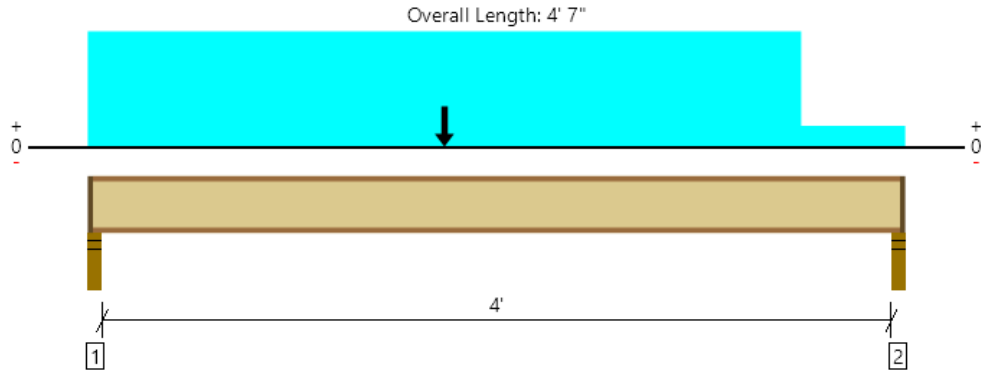
Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	0.450	0.450
Overall MINimum	0.375	0.375
D Only	0.076	0.076
+D+L	0.450	0.450
+D+0.750L	0.357	0.357
+0.60D	0.045	0.045
L Only	0.375	0.375

Level, M3

Current Solution: 2 piece(s) 9 1/2" TJI® 110 @ 16" OC



All locations are measured from the outside face of left support (or left cantilever end). All dimensions are horizontal.

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	1847 @ 2 1/2"	2083 (2.25")	Passed (89%)	1.00	1.0 D + 1.0 L (All Spans)
Shear (lbs)	1790 @ 3 1/2"	2440	Passed (73%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	2766 @ 2'	5000	Passed (55%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.039 @ 2'	0.104	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.051 @ 2'	0.208	Passed (L/989)	--	1.0 D + 1.0 L (All Spans)
TJ-Pro™ Rating	73	40	Passed	--	--

System : Floor
Member Type : Joist
Building Use : Residential
Building Code : IBC 2015
Design Methodology : ASD

All Product Solutions					
Depth	Series	Plies	Spacing	**TJ-Pro™ Rating**	Wood Volume
9 1/2"	TJI® 110	2	16"	73	1.30

The purpose of this report is for product comparison only. Load and support information necessary for professional design review is not displayed here. Please print an individual Member Report for submittal purposes.

ForteWEB Software Operator	Job Notes
Solomon Schlenker Sitts & Hill Engineers (253) 474-9449 solomons@sittshill.com	

Weyerhaeuser

Permit Number: 20-00976

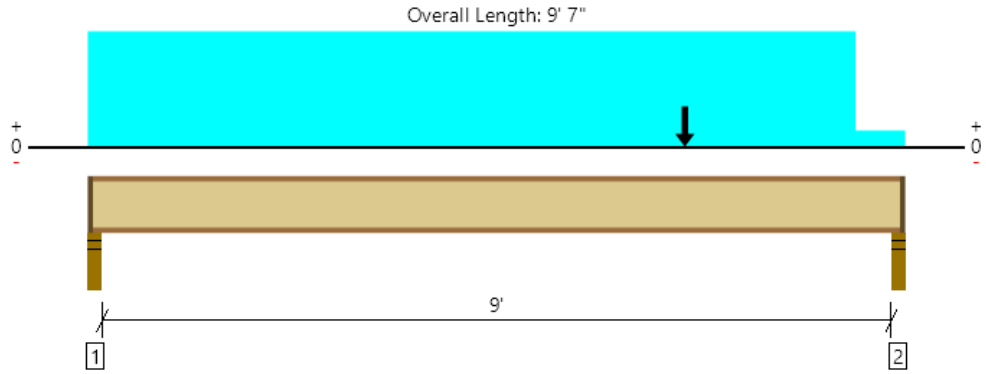
2/4/2020 12:06:21 AM UTC

ForteWEB v2.3, Engine: V8.0.0.21, Data: V7.3.1.1

File Name: 18720

Level, M4

Current Solution: 2 piece(s) 9 1/2" TJI® 110 @ 16" OC



All locations are measured from the outside face of left support (or left cantilever end). All dimensions are horizontal.

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	1906 @ 9' 4 1/2"	2083 (2.25")	Passed (92%)	1.00	1.0 D + 1.0 L (All Spans)
Shear (lbs)	1893 @ 9' 3 1/2"	2440	Passed (78%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	4455 @ 5'	5000	Passed (89%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.180 @ 4' 9 15/16"	0.229	Passed (L/612)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.234 @ 4' 9 15/16"	0.458	Passed (L/470)	--	1.0 D + 1.0 L (All Spans)
TJ-Pro™ Rating	66	40	Passed	--	--

System : Floor
Member Type : Joist
Building Use : Residential
Building Code : IBC 2015
Design Methodology : ASD

All Product Solutions					
Depth	Series	Plies	Spacing	**TJ-Pro™ Rating**	Wood Volume
9 1/2"	TJI® 110	2	16"	66	1.30

The purpose of this report is for product comparison only. Load and support information necessary for professional design review is not displayed here. Please print an individual Member Report for submittal purposes.

ForteWEB Software Operator	Job Notes
Solomon Schlenker Sitts & Hill Engineers (253) 474-9449 solomons@sittshill.com	

Weyerhaeuser

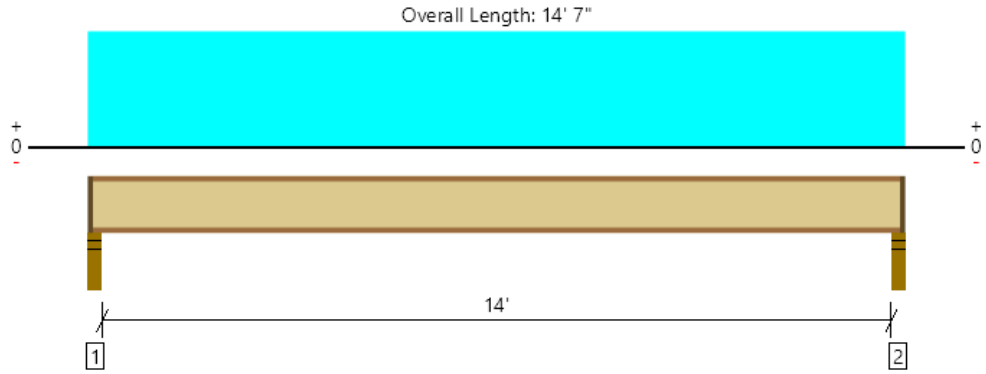
Permit Number: 20-00976

2/4/2020 12:06:50 AM UTC

ForteWEB v2.3, Engine: V8.0.0.21, Data: V7.3.1.1

File Name: 18720

Level, Typical Main Floor Joist
Current Solution: 1 piece(s) 9 1/2" TJI® 110 @ 16" OC



All locations are measured from the outside face of left support (or left cantilever end). All dimensions are horizontal.

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	498 @ 2 1/2"	1041 (2.25")	Passed (48%)	1.00	1.0 D + 1.0 L (All Spans)
Shear (lbs)	485 @ 3 1/2"	1220	Passed (40%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	1739 @ 7' 3 1/2"	2500	Passed (70%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.270 @ 7' 3 1/2"	0.354	Passed (L/629)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.351 @ 7' 3 1/2"	0.708	Passed (L/484)	--	1.0 D + 1.0 L (All Spans)
TJ-Pro™ Rating	39	39	Passed	--	--

System : Floor
Member Type : Joist
Building Use : Residential
Building Code : IBC 2015
Design Methodology : ASD

All Product Solutions					
Depth	Series	Plies	Spacing	**TJ-Pro™ Rating**	Wood Volume
9 1/2"	TJI® 110	1	16"	39	0.65

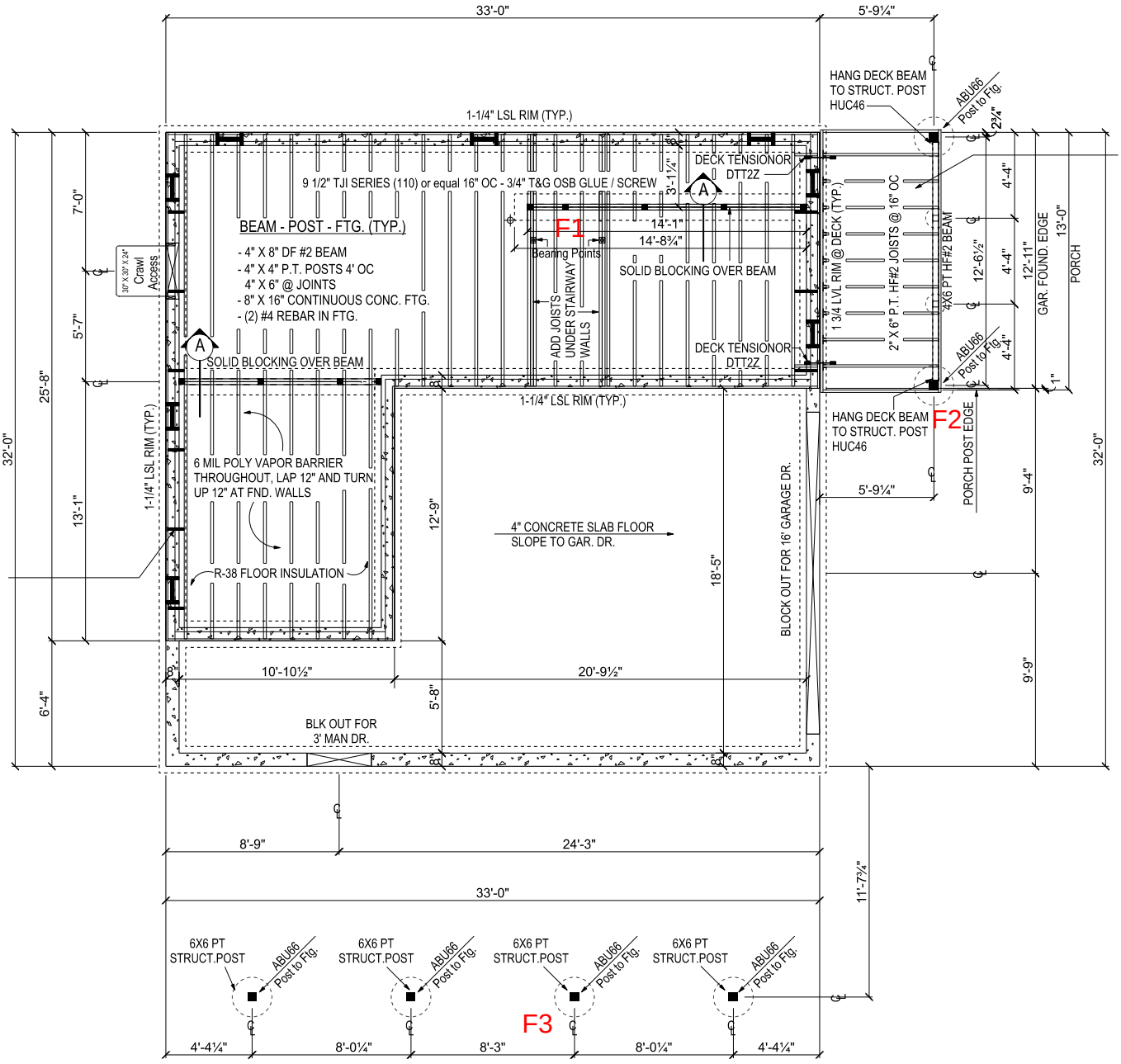
The purpose of this report is for product comparison only. Load and support information necessary for professional design review is not displayed here. Please print an individual Member Report for submittal purposes.

ForteWEB Software Operator	Job Notes
Solomon Schlenker Sitts & Hill Engineers (253) 474-9449 solomons@sittshill.com	

Weyerhaeuser

Permit Number: 20-00976

2/4/2020 12:00:50 AM UTC
ForteWEB v2.3, Engine: V8.0.0.21, Data: V7.3.1.1
File Name: 18720



MAIN FLOOR PLAN
1/8" = 1'-0"

FOUNDATION

F_1 Typical Footing

$R_{M4} := 1906\ lbf$ CONTINUOUS FOOTING OK

F_2

$R_{M2} := 450\ lbf$

$P := 2 \cdot R_{M2} = 900\ lbf$ 18" DIAMETER FOOTING OK

F_3

$R_{U4} := 1316\ lbf$

$P := 2 \cdot R_{U4} = 2632\ lbf$ 18" DIAMETER FOOTING OK

SITTS & HILL ENGINEERS, INC.
TACOMA. WASHINGTON
TEL: (253)474-9449

DESIGNED	SDS	DATE	2/3/20	JOB	18,720
CHECKED	EAB	DATE	2/3/20	SHEET	
PROJECT Armstrong Homes, Sygulla Residence				OF	

SPREAD FOOTING ANALYSIS
SOIL BEARING CAPACITY = 1,500 PSF

FOOTING TYPE	DIMENSION (FT) W	B	BEARING CAPACITY (LBS)	REINFORCEMENT
36" DIA. X 10" FTG		3.00	10603	4#4 E.W.
30" DIA. X 10" FTG		2.50	7363	4#4 E.W.
24" DIA. X 10" FTG		2.00	4712	3#4 E.W.
18" DIA. X 10" FTG		1.5	2651	2#4 E.W.
48"X48"X 12" FTG	4	4	24000	6#4 E.W.
42"X42"X 12" FTG	3.5	3.5	18375	5#4 E.W.
36"X36"X 12" FTG	3	3	13500	4#4 E.W.
30"X30"X 10" FTG	2.5	2.5	9375	4#4 E.W.
24"X24"X 10" FTG	2	2	6000	3#4 E.W.
18"X18"X 10" FTG	1.5	1.5	3375	2#4 E.W.
12"X8" CONT. FTG	1.00	0.67	6000	2#4 Cont.
16"X8" CONT. FTG	1.33	0.67	7980	2#4 Cont.

Note:

1. Minimum reinforcement ratio per ACI 318-14 7.12.2.1.

(1)-2x6 HF#2 Built-Up Post $H := 9\ ft$

$F_c := 1300\ psi$ $C_D := 1.0$ $C_M := 1.0$ $C_t := 1.0$ $C_{Fc} := 1.15$

$F_{cp} := 405\ psi$

$E' := 1300000$ $E'min := 470000\ psi$

$F''_c := F_c \cdot C_D \cdot C_{Fc} = 1495\ psi$

Axial Load Capacity

Slenderness Ratio (SL)

$SL := \frac{H}{h}$ $c := 0.8$

$F_{CE} := \frac{0.822 \cdot E'min}{SL^2} = (1 \cdot 10^3)\ psi$

(1)-2x6 HF#2 Built-Up Post Properties

$K_f := 1$ Kf= 0.6 for unbraced nailed built up posts- 0.75 for bolted

$t := 1.5\ in$

$h := 5.5\ in$

$A := t \cdot h = 8.3\ in^2$

$I := \frac{t \cdot h^3}{12} = 20.8\ in^4$

$S := \frac{I \cdot 2}{h} = 7.6\ in^3$

$$C_p := \left(\frac{1 + \left(\frac{F_{CE}}{F''_c}\right)}{2 \cdot c} - \sqrt{\left(\frac{1 + \frac{F_{CE}}{F''_c}}{2 \cdot c}\right)^2 - \frac{\frac{F_{CE}}{F''_c}}{c}} \right) \cdot K_f \qquad C_p = 0.54$$

$F'_c := C_p \cdot F''_c = 810\ psi$ $P_{max1} := F'_c \cdot A = 6684\ lbf$

(1)-2x6 HF#2 Built-Up Post

$P_{max1} = 6684\ lbf$

$P_{max2} := F_{cp} \cdot A = 3341\ lbf$

$P_{maximum} := min(P_{max1}, P_{max2})$ $P_{maximum} = 3341\ lbf$ At discontinuous plate

Base Plate Bearing Stress

$C_b := \frac{t + 0.375\ in}{t} = 1.25$

$P_{max3} := C_b \cdot F_{cp} \cdot A$ $P_{max3} = 4177\ lbf$ At continuous plate

(2)-2x6 HF#2 Built-Up Post $H := 9\ ft$

$F_c := 1300\ psi$ $C_D := 1.0$ $C_M := 1.0$ $C_t := 1.0$ $C_{Fc} := 1.15$

$F_{cp} := 405\ psi$

$E' := 1300000$ $E'min := 470000\ psi$

$F''_c := F_c \cdot C_D \cdot C_{Fc} = 1495\ psi$

Axial Load Capacity

Slenderness Ratio (SL)

$SL := \frac{H}{h}$ $c := 0.8$

$F_{CE} := \frac{0.822 \cdot E'min}{SL^2} = (1 \cdot 10^3)\ psi$

$$C_p := \left(\frac{1 + \left(\frac{F_{CE}}{F''_c}\right)}{2 \cdot c} - \sqrt{\left(\frac{1 + \frac{F_{CE}}{F''_c}}{2 \cdot c}\right)^2 - \frac{\frac{F_{CE}}{F''_c}}{c}} \right) \cdot K_f \qquad C_p = 0.54$$

$F'_c := C_p \cdot F''_c = 810\ psi$ $P_{max1} := F'_c \cdot A = 13369\ lbf$

(2)-2x6 HF#2 Built-Up Post Properties

$K_f := 1$ Kf= 0.6 for unbraced nailed built up posts- 0.75 for bolted

$t := 2 \cdot 1.5\ in$

$h := 5.5\ in$

$A := t \cdot h = 16.5\ in^2$

$I := \frac{t \cdot h^3}{12} = 41.6\ in^4$

$S := \frac{I \cdot 2}{h} = 15.1\ in^3$

(2)-2x6 HF#2 Built-Up Post

$P_{max1} = 13369\ lbf$

$P_{max2} := F_{cp} \cdot A = 6683\ lbf$

$P_{maximum} := min(P_{max1}, P_{max2})$ $P_{maximum} = 6683\ lbf$ At discontinuous plate

Base Plate Bearing Stress

$C_b := \frac{t + 0.375\ in}{t} = 1.13$

$P_{max3} := C_b \cdot F_{cp} \cdot A$ $P_{max3} = 7518\ lbf$ At continuous plate

(3)-2x6 HF#2 Built-Up Post $H := 9\ ft$

$F_c := 1300\ psi$ $C_D := 1.0$ $C_M := 1.0$ $C_t := 1.0$ $C_{Fc} := 1.15$

$F_{cp} := 405\ psi$

$E' := 1300000$ $E'min := 470000\ psi$

$F''_c := F_c \cdot C_D \cdot C_{Fc} = 1495\ psi$

Axial Load Capacity

Slenderness Ratio (SL)

$SL := \frac{H}{h}$ $c := 0.8$

$F_{CE} := \frac{0.822 \cdot E'min}{SL^2} = \left(1 \cdot 10^3\right)\ psi$

(3)-2x6 HF#2 Built-Up Post Properties

$K_f := 1$ Kf= 0.6 for unbraced nailed built up posts- 0.75 for bolted

$t := 3 \cdot 1.5\ in$

$h := 5.5\ in$

$A := t \cdot h = 24.8\ in^2$

$I := \frac{t \cdot h^3}{12} = 62.4\ in^4$

$S := \frac{I \cdot 2}{h} = 22.7\ in^3$

$$C_p := \left(\frac{1 + \left(\frac{F_{CE}}{F''_c}\right)}{2 \cdot c} - \sqrt{\left(\frac{1 + \frac{F_{CE}}{F''_c}}{2 \cdot c}\right)^2 - \frac{\frac{F_{CE}}{F''_c}}{c}}\right) \cdot K_f \qquad C_p = 0.54$$

$F'_c := C_p \cdot F''_c = 810\ psi$ $P_{max1} := F'_c \cdot A = 20053\ lbf$

(3)-2x6 HF#2 Built-Up Post

$P_{max1} = 20053\ lbf$

$P_{max2} := F_{cp} \cdot A = 10024\ lbf$

$P_{maximum} := min\left(P_{max1}, P_{max2}\right)$ $P_{maximum} = 10024\ lbf$ At discontinuous plate

Base Plate Bearing Stress

$C_b := \frac{t + 0.375\ in}{t} = 1.08$

$P_{max3} := C_b \cdot F_{cp} \cdot A$ $P_{max3} = 10859\ lbf$ At continuous plate

(4)-2x6 HF#2 Built-Up Post $H := 9\ ft$

$F_c := 1300\ psi$ $C_D := 1.0$ $C_M := 1.0$ $C_t := 1.0$ $C_{Fc} := 1.15$

$F_{cp} := 405\ psi$

$E' := 1300000$ $E'min := 470000\ psi$

$F''_c := F_c \cdot C_D \cdot C_{Fc} = 1495\ psi$

Axial Load Capacity

Slenderness Ratio (SL)

$SL := \frac{H}{h}$ $c := 0.8$

$F_{CE} := \frac{0.822 \cdot E'min}{SL^2} = (1 \cdot 10^3)\ psi$

$$C_p := \left(\frac{1 + \left(\frac{F_{CE}}{F''_c}\right)}{2 \cdot c} - \sqrt{\left(\frac{1 + \frac{F_{CE}}{F''_c}}{2 \cdot c}\right)^2 - \frac{\frac{F_{CE}}{F''_c}}{c}} \right) \cdot K_f \qquad C_p = 0.54$$

$F'_c := C_p \cdot F''_c = 810\ psi$ $P_{max1} := F'_c \cdot A = 26737\ lbf$

(4)-2x6 HF#2 Built-Up Post Properties

$K_f := 1$ Kf= 0.6 for unbraced nailed built up posts- 0.75 for bolted

$t := 4 \cdot 1.5\ in$

$h := 5.5\ in$

$A := t \cdot h = 33\ in^2$

$I := \frac{t \cdot h^3}{12} = 83.2\ in^4$

$S := \frac{I \cdot 2}{h} = 30.3\ in^3$

(4)-2x6 HF#2 Built-Up Post

$P_{max1} = 26737\ lbf$

$P_{max2} := F_{cp} \cdot A = 13365\ lbf$

$P_{maximum} := min(P_{max1}, P_{max2})$ $P_{maximum} = 13365\ lbf$ At discontinuous plate

Base Plate Bearing Stress

$C_b := \frac{t + 0.375\ in}{t} = 1.06$

$P_{max3} := C_b \cdot F_{cp} \cdot A$ $P_{max3} = 14200\ lbf$ At continuous plate

4x4 HF#2 Post $H := 9\ ft$

$F_c := 1300\ psi$ $C_D := 1.0$ $C_M := 1.0$ $C_t := 1.0$ $C_{Fc} := 1.15$

$F_{cp} := 405\ psi$

$E' := 1300000$ $E'min := 470000\ psi$

$F''_c := F_c \cdot C_D \cdot C_{Fc} = 1495\ psi$

Axial Load Capacity

Slenderness Ratio (SL)

$SL := \frac{H}{h}$ $c := 0.8$

$F_{CE} := \frac{0.822 \cdot E'min}{SL^2} = 406\ psi$

4x4 HF#2 Post Properties

$K_f := 1$ Kf= 0.6 for unbraced nailed built up posts- 0.75 for bolted

$t := 3.5\ in$

$h := 3.5\ in$

$A := t \cdot h = 12.3\ in^2$

$I := \frac{t \cdot h^3}{12} = 12.5\ in^4$

$S := \frac{I \cdot 2}{h} = 7.1\ in^3$

$$C_p := \left(\frac{1 + \left(\frac{F_{CE}}{F''_c}\right)}{2 \cdot c} - \sqrt{\left(\frac{1 + \frac{F_{CE}}{F''_c}}{2 \cdot c}\right)^2 - \frac{\frac{F_{CE}}{F''_c}}{c}} \right) \cdot K_f \qquad C_p = 0.25$$

$F'_c := C_p \cdot F''_c = 380\ psi$ $P_{max1} := F'_c \cdot A = 4653\ lbf$

4x4 HF#2 Post

$P_{max1} = 4653\ lbf$

$P_{max2} := F_{cp} \cdot A = 4961\ lbf$

$P_{maximum} := min(P_{max1}, P_{max2})$ $P_{maximum} = 4653\ lbf$ At discontinuous plate

Base Plate Bearing Stress

$C_b := \frac{t + 0.375\ in}{t} = 1.11$

$P_{max3} := C_b \cdot F_{cp} \cdot A$ $P_{max3} = 5493\ lbf$ At continuous plate

4x6 DF#2 Post $H := 9\ ft$

$F_c := 1350\ psi$ $C_D := 1.0$ $C_M := 1.0$ $C_t := 1.0$ $C_{Fc} := 1.1$

$F_{cp} := 625\ psi$

$E' := 1600000$ $E'min := 580000\ psi$

$F''_c := F_c \cdot C_D \cdot C_{Fc} = 1485\ psi$

4x6 DF#2 Post Properties

$K_f := 1$ Kf= 0.6 for unbraced nailed built up posts- 0.75 for bolted

$t := 5.5\ in$

$h := 3.5\ in$

$A := t \cdot h = 19.3\ in^2$

$I := \frac{t \cdot h^3}{12} = 19.7\ in^4$

$S := \frac{I \cdot 2}{h} = 11.2\ in^3$

Axial Load Capacity

Slenderness Ratio (SL)

$SL := \frac{H}{h}$ $c := 0.8$

$F_{CE} := \frac{0.822 \cdot E'min}{SL^2} = 501\ psi$

$$C_p := \left(\frac{1 + \left(\frac{F_{CE}}{F''_c}\right)}{2 \cdot c} - \sqrt{\left(\frac{1 + \frac{F_{CE}}{F''_c}}{2 \cdot c}\right)^2 - \frac{\frac{F_{CE}}{F''_c}}{c}} \right) \cdot K_f \qquad C_p = 0.31$$

$F'_c := C_p \cdot F''_c = 460\ psi$ $P_{max1} := F'_c \cdot A = 8846\ lbf$

4x6 DF#2 Post

$P_{max1} = 8846\ lbf$

$P_{max2} := F_{cp} \cdot A = 12031\ lbf$

$P_{maximum} := min(P_{max1}, P_{max2})$ $P_{maximum} = 8846\ lbf$ At discontinuous plate

Base Plate Bearing Stress

$C_b := \frac{t + 0.375\ in}{t} = 1.07$

$P_{max3} := C_b \cdot F_{cp} \cdot A$ $P_{max3} = 12852\ lbf$ At continuous plate

4x8 DF#2 Post $H := 9\ ft$

$F_c := 1350\ psi$ $C_D := 1.0$ $C_M := 1.0$ $C_t := 1.0$ $C_{Fc} := 1.05$

$F_{cp} := 625\ psi$

$E' := 1600000$ $E'min := 580000\ psi$

$F''_c := F_c \cdot C_D \cdot C_{Fc} = 1418\ psi$

Axial Load Capacity

Slenderness Ratio (SL)

$SL := \frac{H}{h}$ $c := 0.8$

$F_{CE} := \frac{0.822 \cdot E'min}{SL^2} = 501\ psi$

4x8 DF#2 Post Properties

$K_f := 1$ Kf= 0.6 for unbraced nailed built up posts- 0.75 for bolted

$t := 7.5\ in$

$h := 3.5\ in$

$A := t \cdot h = 26.3\ in^2$

$I := \frac{t \cdot h^3}{12} = 26.8\ in^4$

$S := \frac{I \cdot 2}{h} = 15.3\ in^3$

$$C_p := \left(\frac{1 + \left(\frac{F_{CE}}{F''_c}\right)}{2 \cdot c} - \sqrt{\left(\frac{1 + \frac{F_{CE}}{F''_c}}{2 \cdot c}\right)^2 - \frac{\frac{F_{CE}}{F''_c}}{c}} \right) \cdot K_f \qquad C_p = 0.32$$

$F'_c := C_p \cdot F''_c = 457\ psi$ $P_{max1} := F'_c \cdot A = 12001\ lbf$

4x8 DF#2 Post

$P_{max1} = 12001\ lbf$

$P_{max2} := F_{cp} \cdot A = 16406\ lbf$

$P_{maximum} := min(P_{max1}, P_{max2})$ $P_{maximum} = 12001\ lbf$ At discontinuous plate

Base Plate Bearing Stress

$C_b := \frac{t + 0.375\ in}{t} = 1.05$

$P_{max3} := C_b \cdot F_{cp} \cdot A$ $P_{max3} = 17227\ lbf$ At continuous plate

6x6 DF#2 Post $H := 9\ ft$

$F_c := 700\ psi$ $C_D := 1.0$ $C_M := 1.0$ $C_t := 1.0$ $C_{Fc} := 1.0$ $C_i := 0.8$

$F_{cp} := 625\ psi$

$E' := 1300000$ $E'min := 470000\ psi$

$F''_c := F_c \cdot C_D \cdot C_{Fc} \cdot C_i = 560\ psi$

Axial Load Capacity

Slenderness Ratio (SL)

$SL := \frac{H}{h}$ $c := 0.8$

$F_{CE} := \frac{0.822 \cdot E'min}{SL^2} = 1192\ psi$

6x6 DF#2 Post Properties

$K_f := 1$ Kf= 0.6 for unbraced nailed built up posts- 0.75 for bolted

$t := 6\ in$

$h := 6\ in$

$A := t \cdot h = 36\ in^2$

$I := \frac{t \cdot h^3}{12} = 108\ in^4$

$S := \frac{I \cdot 2}{h} = 36\ in^3$

$$C_p := \left(\frac{1 + \left(\frac{F_{CE}}{F''_c}\right)}{2 \cdot c} - \sqrt{\left(\frac{1 + \frac{F_{CE}}{F''_c}}{2 \cdot c}\right)^2 - \frac{\frac{F_{CE}}{F''_c}}{c}} \right) \cdot K_f \qquad C_p = 0.88$$

$F'_c := C_p \cdot F''_c = 491\ psi$ $P_{max1} := F'_c \cdot A = 17683\ lbf$

6x6 DF#2 Post

$P_{max1} = 17683\ lbf$

$P_{max2} := F_{cp} \cdot A = 22500\ lbf$

$P_{maximum} := min(P_{max1}, P_{max2})$ $P_{maximum} = 17683\ lbf$ At discontinuous plate

Base Plate Bearing Stress

$C_b := \frac{t + 0.375\ in}{t} = 1.06$

$P_{max3} := C_b \cdot F_{cp} \cdot A$ $P_{max3} = 23906\ lbf$ At continuous plate

WOOD STUD WALL DESIGN

STUD WALL MARK: Typical Wall Occupancy = II

DEFLECTION LIMIT: Δ_{all} = L / 180

STUD PROPERTIES:

Stud Size = 2x6 Stud Species = Hem-Fir Stud Spacing = 16"
Qty. of Studs = Single Stud Grade = No.2 Wall Height = 9 ft
b = 1.50 in Plate Species = Hem-Fir Wall Weight = 10 psf
d = 5.50 in c = 0.8

ADJUSTMENT FACTORS:

C_T = 1.00
Incising ? ☐ C_i = 1.00 C_M = 1.00 C_t = 1.00 C_L = 1.00
C_{fu} = 1.00 C_F (F_b) = 1.30 C_F (F_c) = 1.10 C_b = 1.00

Repetitive Member ? ☑
Meet requirements of SDPWS-08, Sec. 3.1.1.1 ? ☑ C_r = 1.35

LUMBER DESIGN VALUES:

F_b = 850 psi F_v = 150 psi F_c = 1,300 psi $F_{c\perp}$ = 405 psi E = 1,300,000 psi
E_{min} = 470,000 psi

GRAVITY LOADS:

Load	Type	Trib Length (ft)	Load (psf)	Qty.	Load (plf)
Roof	Dead	16.00	16	(1)	256
Roof	Snow	16.00	25	(1)	400
Floor	Dead	19.00	12	(0)	0
Floor	Live	19.00	40	(0)	0

WIND LOAD: Application = Exterior

Wind Speed, V = 110 mph Wind Exposure = C Hurricane Prone Region ? ☐
I_W = 1.00 Speed-up Factor, k_{zt} = 1.00 h = 9 ft
α = 9.5 z_g = 900 ft k_h = 0.76
k_d = 0.85 Enclosure = Enclosed GC_{pi} = 0.18
q_h = 20.1 psf Effective Wind Area = 27 ft^2 Zone = 4
GC_p = 1.02

p = 24.2 psf

SEISMIC LOAD:

I_p = 1.00 S_{DS} = 0.846 F_p = 2.7 psf w_D = 256 plf w_E = 43 plf

WOOD STUD WALL DESIGN

LOAD COMBINATIONS:

	C_D	w (plf)	F_p or p (psf)	Modified Axial (plf)	
D	0.90	256	0.0	284	
D + L	1.00	256	0.0	256	
D + S	1.15	656	0.0	570	*(Axial)(Plate Crushing)(Combined)*
D +0.6 W	1.60	256	14.5	160	*(Shear)(Bending)(Deflection)(Combined)*
D + 0.7E	1.60	286	1.9	179	
D + 0.45W + 0.75L + 0.75S	1.60	556	10.9	348	*(Combined)*
0.6D + 0.6W	1.60	154	14.5	96	*(Shear)(Bending)(Deflection)*
0.6D - 0.7E	1.60	123	1.9	77	

SHEAR CHECK: Governing Load Combination: D +0.6 W

$F'_v = F_v \times C_D \times C_M \times C_t \times C_i$
= 150 psi (1.6)(1)(1)(1)
= 240 psi

V = w x s x L / 2
= 14.5 psf x 16 in / (12 in / ft) x 9.0 ft / 2
= 87 #

A = b x d
= 1.50 in x 5.50 in
= 8.3 in^2

f_v = 3 x V / (2 x A)
= 3 x 87 # / (2 x 8.25 in^2)
= 16 psi

f_v	≤	F'_v	
16 psi	≤	240 psi	*SHEAR IS OKAY*

DEFLECTION: Governing Load Combination: D +0.6 W

I = b x d^3 / 12
= 1.50 in x (5.50 in)^3 / 12
= 20.8 in^4

δ = 5 x w x s x L^4 / (384 E I)
= 5 x 14.5 psf x 16 in / (12 in / ft) x (9.0 ft)^4 x (12 in / ft)^3 / (384 x 1,300,000 psi x 20.8 in
= 0.11 in

Δ_{all} = 9.0 ft x 12 in / ft / 180
= 0.60 in

δ	≤	Δ_{all}	
0.11 in	≤	0.60 in	*DEFLECTION IS OKAY*

WOOD STUD WALL DESIGN

PLATE CRUSHING: Governing Load Combination: D + S

$F'_{c\perp} = F_{c\perp} \times C_M \times C_t \times C_i \times C_b$
$= 405 \text{ psi} (1)(1)(1)(1)$
$= 405 \text{ psi}$

$f_{c\perp} = P \times s / A$
$= 570 \text{ plf} \times 16 \text{ in} / (12 \text{ in / ft}) / 8.25 \text{ in}^2$
$= 92 \text{ psi}$

$f_{c\perp}$	≤	$F'_{c\perp}$	
92 psi	≤	405 psi	*PLATE CRUSHING IS OKAY*

COMPRESSION: Governing Load Combination: D + S

$F_c^* = F_c \times C_D \times C_M \times C_t \times C_F \times C_i$
$= 1{,}300 \text{ psi} (1.15)(1)(1)(1.1)(1)$
$= 1{,}645 \text{ psi}$

$\ell_e = 9.0 \text{ ft} \times 12 \text{ in / ft}$
$= 108.0 \text{ in}$

$E_{min}' = E_{min} \times C_M \times C_t \times C_i \times C_T$
$= 470{,}000 \text{ psi} (1)(1)(1)(1)$
$= 470{,}000 \text{ psi}$

$\ell_e / d = 108.0 \text{ in} / 5.50 \text{ in}$
$= 19.6 \quad \leq \quad 50$ *SLENDERNESS RATIO IS OKAY*

$F_{cE} = 0.822 \times E_{min}' / (\ell_e / d)^2$
$= 0.822 \times 470{,}000 \text{ psi} / (19.6 \text{ in})^2$
$= 1{,}002 \text{ psi}$

$F_{cE} / F_c^* = 1{,}002 \text{ psi} / 1{,}645 \text{ psi}$
$= 0.61$

$C_p = [1 + (F_{cE} / F_c^*) / (2 \times c)] - \text{sqrt} \{ [1 + (F_{cE} / F_c^*) / (2 \times c)]^2 - (F_{cE} / F_c^*) / c \}$
$= [1 + 0.61 / (2 \times 0.80)] - \text{sqrt} \{ [(1 + 0.61) / (2 \times 0.80)]^2 - 0.61 / 0.80 \}$
$= 0.51$

$F_c' = F_c^* \times C_p$
$= 1{,}645 \text{ psi} \times 0.51$
$= 832 \text{ psi}$

$f_c = w \times s / A$
$= 656 \text{ plf} \times 16 \text{ in} / (12 \text{ in / ft}) / 8.25 \text{ in}^2$
$= 106 \text{ psi}$

f_c	≤	F'_c	
106 psi	≤	832 psi	*COMPRESSION IS OKAY*

Permit Number: 20-00976

WOOD STUD WALL DESIGN

BENDING: Governing Load Combination: D +0.6 W

$F_b^* = F'_b = F_b \times C_D \times C_M \times C_t \times C_L \times C_F \times C_{fu} \times C_i \times C_r$
= 850 psi (1.6)(1)(1)(1)(1.3)(1)(1)(1.35)
= 2,387 psi

M = w x s x L^2 / 8
= 14.5 psf x 16 in / (12 in / ft) x (9.0 ft)^2 x (12 in / ft) / 8
= 2,349 # in

S = 2 x I / d
= 2 x 20.8 in^4 / 5.50 in
= 7.56 in^3

f_b = M / S
= 2,349 # in / 7.56 in^3
= 311 psi

f_b	≤	F'_b	
311 psi	≤	2,387 psi	*BENDING IS OKAY*

BENDING AND COMPRESSION:

LOAD COMBINATION	f_b	F_b'	f_c	C_p	F_c'	CSI
D	0 psi	1,343 psi	41 psi	0.60	771 psi	0.00
D + L	0 psi	1,492 psi	41 psi	0.56	799 psi	0.00
D + S	0 psi	1,716 psi	106 psi	0.51	832 psi	0.02
D +0.6 W	311 psi	2,387 psi	41 psi	0.39	889 psi	0.14
D + 0.7E	41 psi	2,387 psi	46 psi	0.39	889 psi	0.02
D + 0.45W + 0.75L + 0.75S	233 psi	2,387 psi	90 psi	0.39	889 psi	0.12
0.6D + 0.6W	311 psi	2,387 psi	25 psi	0.39	889 psi	0.13
0.6D - 0.7E	41 psi	2,387 psi	20 psi	0.39	889 psi	0.02

Governing Load Combination: D +0.6 W

CSI = $(f_c / F_c')^2 + f_b / \{ F_b' \times [1 - (f_c / F_{cE})] \}$
= (41 psi / 889 psi)^2 + 311 psi / { 2,387 x [1 - (41 psi / 1,002 psi)] }

= 0.14 ≤ 1.00 *COMBINED STRESS RATIO IS OKAY*

WOOD STUD WALL DESIGN

STUD WALL MARK: 12' Tall Wall Occupancy = II

DEFLECTION LIMIT: Δ_{all} = L / 180

STUD PROPERTIES:

Stud Size = 2x6 Stud Species = Hem-Fir Stud Spacing = 16"
Qty. of Studs = Single Stud Grade = No.2 Wall Height = 12 ft
b = 1.50 in Plate Species = Hem-Fir Wall Weight = 10 psf
d = 5.50 in c = 0.8

ADJUSTMENT FACTORS:

C_T = 1.00
Incising ? ☐ C_i = 1.00 C_M = 1.00 C_t = 1.00 C_L = 1.00
C_{fu} = 1.00 C_F (F_b) = 1.30 C_F (F_c) = 1.10 C_b = 1.00

Repetitive Member ? ☑
Meet requirements of SDPWS-08, Sec. 3.1.1.1 ? ☑ C_r = 1.35

LUMBER DESIGN VALUES:

F_b = 850 psi F_v = 150 psi F_c = 1,300 psi $F_{c\perp}$ = 405 psi E = 1,300,000 psi
E_{min} = 470,000 psi

GRAVITY LOADS:

Load	Type	Trib Length (ft)	Load (psf)	Qty.	Load (plf)
Roof	Dead	16.00	16	(1)	256
Roof	Snow	16.00	25	(1)	400
Floor	Dead	0.00	12	(0)	0
Floor	Live	0.00	40	(0)	0

WIND LOAD: Application = Exterior

Wind Speed, V = 110 mph Wind Exposure = C Hurricane Prone Region ? ☐
I_W = 1.00 Speed-up Factor, k_{zt} = 1.00 h = 12 ft
α = 9.5 z_g = 900 ft k_h = 0.81
k_d = 0.85 Enclosure = Enclosed GC_{pi} = 0.18
q_h = 21.3 psf Effective Wind Area = 48 ft^2 Zone = 4
GC_p = 0.98

p = 24.7 psf

SEISMIC LOAD:

I_p = 1.00 S_{DS} = 0.846 F_p = 2.7 psf w_D = 256 plf w_E = 43 plf

WOOD STUD WALL DESIGN

LOAD COMBINATIONS:

	C_D	w (plf)	F_p or p (psf)	Modified Axial (plf)	
D	0.90	256	0.0	284	
D + L	1.00	256	0.0	256	
D + S	1.15	656	0.0	570	*(Axial)(Plate Crushing)(Combined)*
D +0.6 W	1.60	256	14.8	160	*(Shear)(Bending)(Deflection)(Combined)*
D + 0.7E	1.60	286	1.9	179	
D + 0.45W + 0.75L + 0.75S	1.60	556	11.1	348	*(Combined)*
0.6D + 0.6W	1.60	154	14.8	96	*(Shear)(Bending)(Deflection)*
0.6D - 0.7E	1.60	123	1.9	77	

SHEAR CHECK: Governing Load Combination: D +0.6 W

$F'_v = F_v \times C_D \times C_M \times C_t \times C_i$
= 150 psi (1.6)(1)(1)(1)
= 240 psi

V = w x s x L / 2
= 14.8 psf x 16 in / (12 in / ft) x 12.0 ft / 2
= 119 #

A = b x d
= 1.50 in x 5.50 in
= 8.3 in^2

f_v = 3 x V / (2 x A)
= 3 x 119 # / (2 x 8.25 in^2)
= 22 psi

f_v	≤	F'_v	
22 psi	≤	240 psi	*SHEAR IS OKAY*

DEFLECTION: Governing Load Combination: D +0.6 W

I = b x d^3 / 12
= 1.50 in x (5.50 in)^3 / 12
= 20.8 in^4

δ = 5 x w x s x L^4 / (384 E I)
= 5 x 14.8 psf x 16 in / (12 in / ft) x (12.0 ft)^4 x (12 in / ft)^3 / (384 x 1,300,000 psi x 20.8 i
= 0.34 in

Δ_{all} = 12.0 ft x 12 in / ft / 180
= 0.80 in

δ	≤	Δ_{all}	
0.34 in	≤	0.80 in	*DEFLECTION IS OKAY*

WOOD STUD WALL DESIGN

PLATE CRUSHING: Governing Load Combination: D + S

$F'_{c\perp} = F_{c\perp} \times C_M \times C_t \times C_i \times C_b$
$= 405 \text{ psi} (1)(1)(1)(1)$
$= 405 \text{ psi}$

$f_{c\perp} = P \times s / A$
$= 570 \text{ plf} \times 16 \text{ in} / (12 \text{ in / ft}) / 8.25 \text{ in}^2$
$= 92 \text{ psi}$

$f_{c\perp}$	≤	$F'_{c\perp}$	
92 psi	≤	405 psi	*PLATE CRUSHING IS OKAY*

COMPRESSION: Governing Load Combination: D + S

$F_c^* = F_c \times C_D \times C_M \times C_t \times C_F \times C_i$
$= 1{,}300 \text{ psi} (1.15)(1)(1)(1.1)(1)$
$= 1{,}645 \text{ psi}$

$\ell_e = 12.0 \text{ ft} \times 12 \text{ in / ft}$
$= 144.0 \text{ in}$

$E_{min}' = E_{min} \times C_M \times C_t \times C_i \times C_T$
$= 470{,}000 \text{ psi} (1)(1)(1)(1)$
$= 470{,}000 \text{ psi}$

$\ell_e / d = 144.0 \text{ in} / 5.50 \text{ in}$
$= 26.2 \quad \leq \quad 50$ *SLENDERNESS RATIO IS OKAY*

$F_{cE} = 0.822 \times E_{min}' / (\ell_e / d)^2$
$= 0.822 \times 470{,}000 \text{ psi} / (26.2 \text{ in})^2$
$= 564 \text{ psi}$

$F_{cE} / F_c^* = 564 \text{ psi} / 1{,}645 \text{ psi}$
$= 0.34$

$C_p = [1 + (F_{cE} / F_c^*) / (2 \times c)] - \text{sqrt} \{ [1 + (F_{cE} / F_c^*) / (2 \times c)]^2 - (F_{cE} / F_c^*) / c \}$
$= [1 + 0.34 / (2 \times 0.80)] - \text{sqrt} \{ [(1 + 0.34) / (2 \times 0.80)]^2 - 0.34 / 0.80 \}$
$= 0.31$

$F_c' = F_c^* \times C_p$
$= 1{,}645 \text{ psi} \times 0.31$
$= 516 \text{ psi}$

$f_c = w \times s / A$
$= 656 \text{ plf} \times 16 \text{ in} / (12 \text{ in / ft}) / 8.25 \text{ in}^2$
$= 106 \text{ psi}$

f_c	≤	F'_c	
106 psi	≤	516 psi	*COMPRESSION IS OKAY*

Permit Number: 20-00976

WOOD STUD WALL DESIGN

BENDING: Governing Load Combination: D +0.6 W

$F_b^* = F'_b = F_b \times C_D \times C_M \times C_t \times C_L \times C_F \times C_{fu} \times C_i \times C_r$
= 850 psi (1.6)(1)(1)(1)(1.3)(1)(1)(1.35)
= 2,387 psi

M = w x s x L^2 / 8
= 14.8 psf x 16 in / (12 in / ft) x (12.0 ft)^2 x (12 in / ft) / 8
= 4,273 # in

S = 2 x I / d
= 2 x 20.8 in^4 / 5.50 in
= 7.56 in^3

f_b = M / S
= 4,273 # in / 7.56 in^3
= 565 psi

f_b	≤	F'_b	
565 psi	≤	2,387 psi	*BENDING IS OKAY*

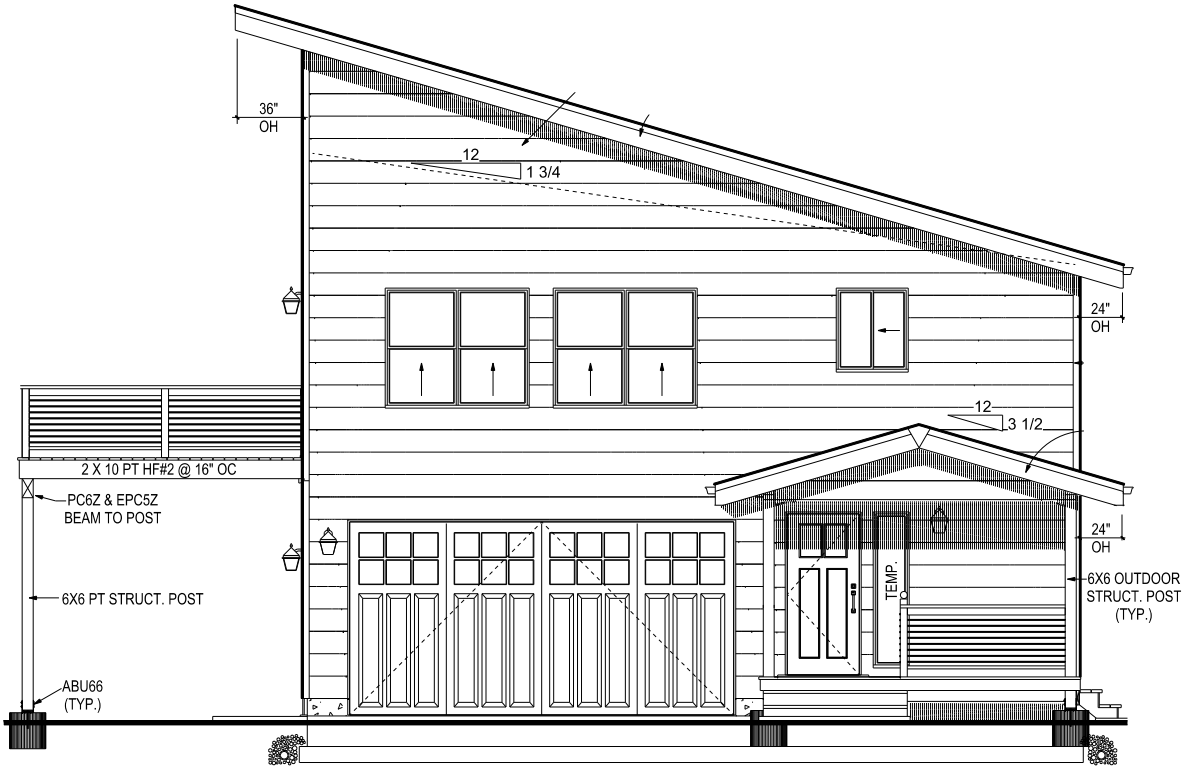
BENDING AND COMPRESSION:

LOAD COMBINATION	f_b	F_b'	f_c	C_p	F_c'	CSI
D	0 psi	1,343 psi	41 psi	0.39	500 psi	0.01
D + L	0 psi	1,492 psi	41 psi	0.36	508 psi	0.01
D + S	0 psi	1,716 psi	106 psi	0.31	516 psi	0.04
D +0.6 W	565 psi	2,387 psi	41 psi	0.23	531 psi	0.26
D + 0.7E	72 psi	2,387 psi	46 psi	0.23	531 psi	0.04
D + 0.45W + 0.75L + 0.75S	424 psi	2,387 psi	90 psi	0.23	531 psi	0.24
0.6D + 0.6W	565 psi	2,387 psi	25 psi	0.23	531 psi	0.25
0.6D - 0.7E	72 psi	2,387 psi	20 psi	0.23	531 psi	0.03

Governing Load Combination: D +0.6 W

CSI = $(f_c / F_c')^2 + f_b / \{ F_b' \times [1 - (f_c / F_{cE})] \}$
= (41 psi / 531 psi)^2 + 565 psi / { 2,387 x [1 - (41 psi / 564 psi)] }

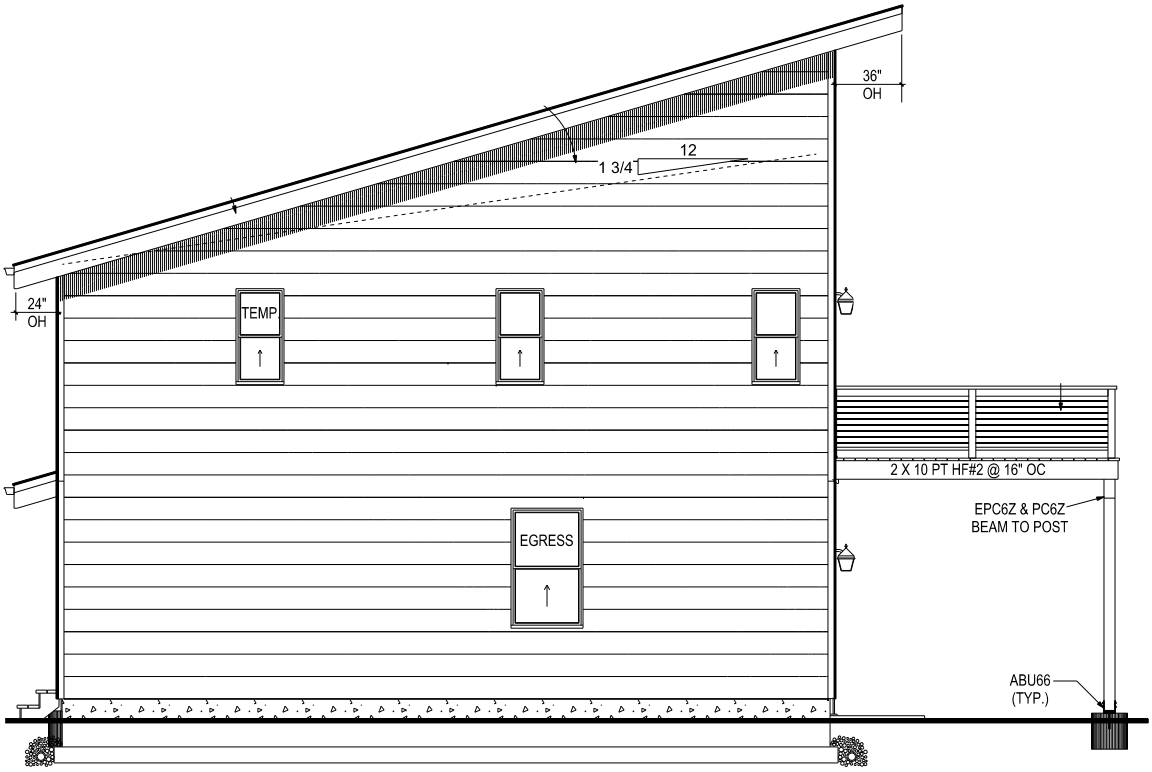
= 0.26 ≤ 1.00 *COMBINED STRESS RATIO IS OKAY*



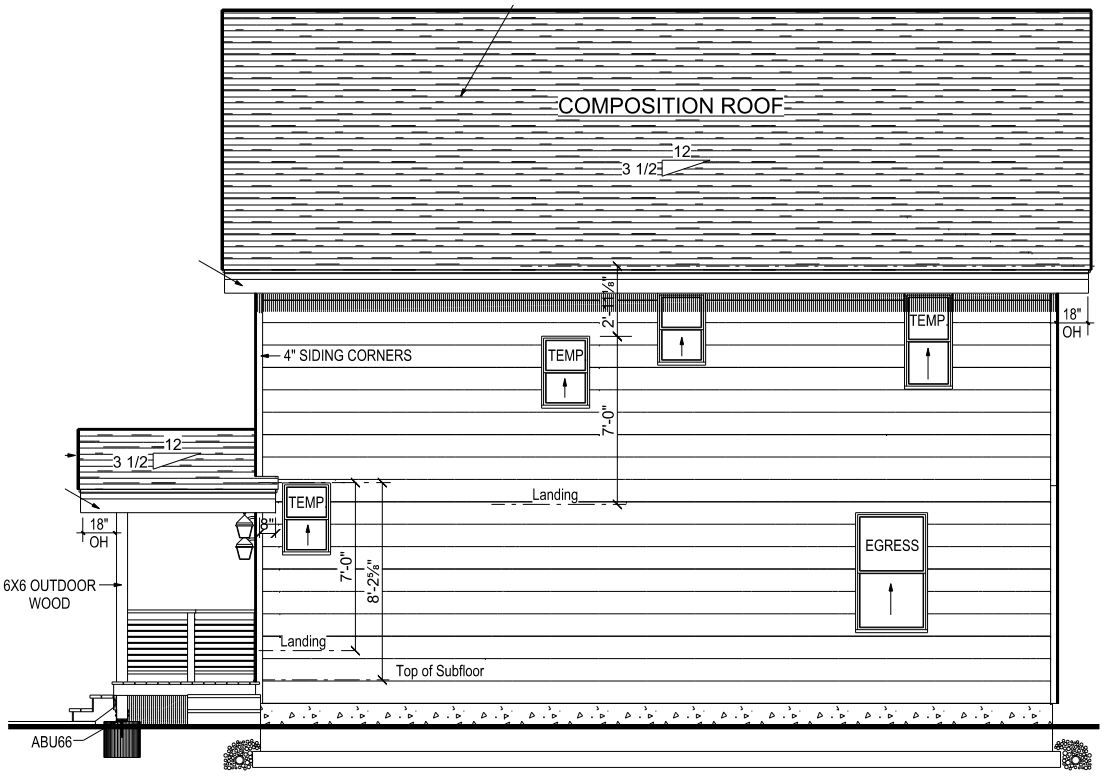
SOUTH ELEVATION
1/8" = 1'-0"

WEST ELEVATION
1/8" = 1'-0"

Permit Number: 20-00976



NORTH ELEVATION
1/8" = 1'-0"

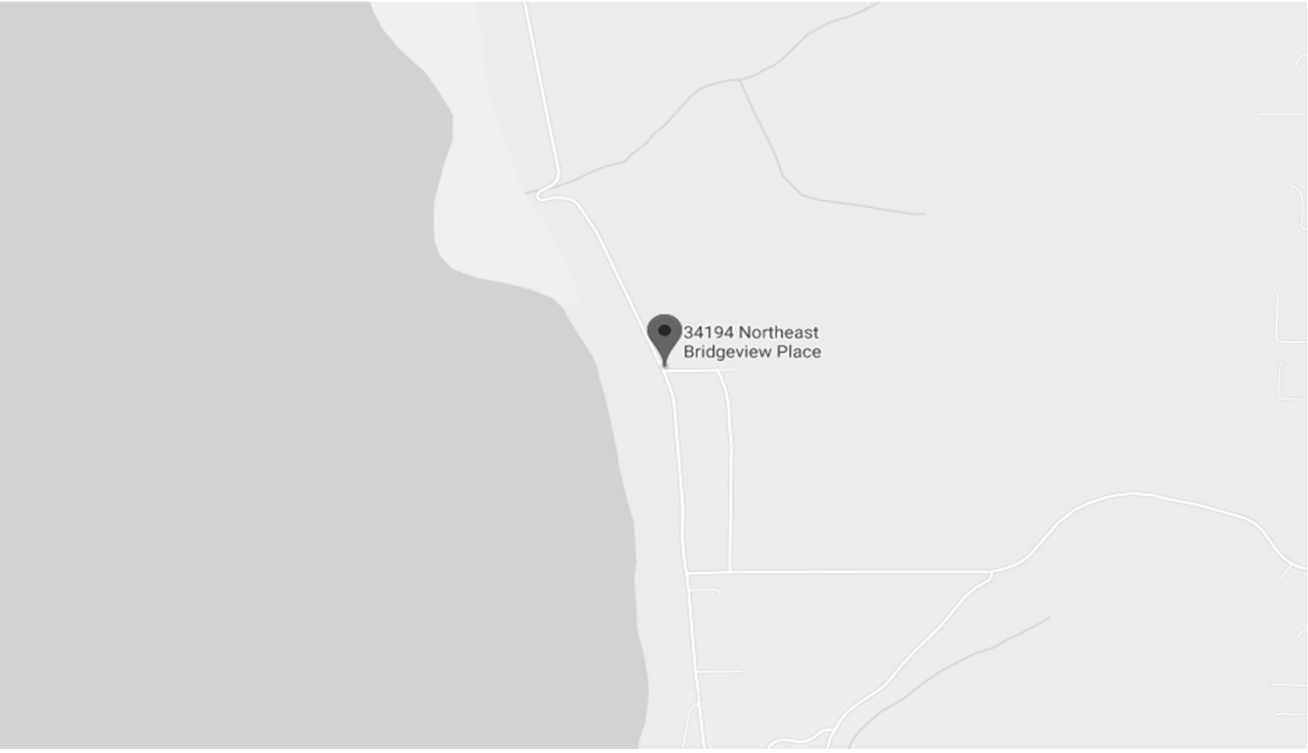


EAST ELEVATION
1/8" = 1'-0"

SEISMIC DESIGN CRITERIA - SITE

Address: 34194 BRIDGEVIEW DRIVE NE
KINGSTON, WA 98436

Latitude: 47.8777 Longitude: -122.5720
S_S = 1.268 S_1 = 0.506 (USGS)



Site Class: D (Assumed)

ASCE 7-10, SECTION 11.4:

F_a = 1.00 (Table 11.4-1)
$S_{MS} = S_S F_a$ = 1.268 x 1.00 (Equation 11.4-1)
S_{MS} = 1.268
$S_{DS} = {}^2/_3\, S_{MS}$ = 2/3 x 1.268 (Equation 11.4-3)
S_{DS} = 0.846

F_v = 1.50 (Table 11.4-2)
$S_{M1} = S_1 F_v$ = 0.506 x 1.50 (Equation 11.4-2)
S_{M1} = 0.759
$S_{D1} = {}^2/_3\, S_{M1}$ = 2/3 x 0.759 (Equation 11.4-4)
S_{D1} = 0.506

ASCE 7-10, SECTION 11.5:

Risk Category: II I_e = 1.00 (Table 1.5-2)

ASCE 7-10, SECTION 11.6:

Seismic Design Category: D (Based on Tables 11.6-1 and 11.6-2)

LATERAL ANALYSIS

PER SECTION 1613.1 OF THE 2015 EDITION OF THE INTERNATIONAL BUILDING CODE, THE HORIZONTAL SEISMIC FORCE "Qe" SHALL BE DETERMINED BY ONE OF THE TYPES OF ANALYSES PERMITTED BY SECTION 12.6 OF ASCE 7-10. AS PERMITTED IN SECTION 12.6 OF ASCE 7-10, THE EQUIVALENT LATERAL FORCE PROCEDURE DESCRIBED IN SECTION 12.8 OF ASCE 7-10 SHALL BE USED TO DETERMINE THE SEISMIC FORCE "Qe".

THE SEISMIC LOAD EFFECT "E" FOR USE IN THE BASIC LOAD COMBINATIONS OF SECTION 2.4.1 OF ASCE 7-10 (OR SECTION 1605.3 OF THE 2015 EDITION OF THE IBC) SHALL BE DETERMINED IN ACCORDANCE WITH SECTION 12.4.2 OF ASCE 7-10.

$R := 6.5$	RESPONSE MODIFICATION COEFFICIENT, TABLE 12.2-1
$S_S := 1.2683$	MAPPED SPECTRAL RESPONSE ACCELERATION AT SHORT PERIODS
$S_1 := 0.5060$	MAPPED SPECTRAL RESPONSE ACCELERATION AT ONE SECOND PERIODS
S_{DS}	DIGITAL SPECTRAL RESPONSE ACCELERATION AT SHORT PERIODS
S_{D1}	DIGITAL SPECTRAL RESPONSE ACCELERATION AT ONE SECOND PERIODS
$F_a := 1.0$	ACCELERATION BASED SITE COEFFICIENT, TABLE 11.4-1
$F_v := 1.5$	VELOCITY BASED SITE COEFFICIENT AT ONE SECOND PERIOD, TABLE 11.4-2
$I := 1.0$	OCCUPANCY IMPORTANCE FACTOR, TABLE 1.5-2
T	FUNDAMENTAL PERIOD OF THE STRUCTURE IN SECONDS, EQUATION 12.8-7
$C_t := 0.02$	FUNDAMENTAL PERIOD COEFFICIENT, TABLE 12.8-2
$x := 0.75$	FUNDAMENTAL PERIOD COEFFICIENT, TABLE 12.8-2
k	EXPONENT RELATING TO THE PERIOD OF THE STRUCTURE, SECTION 12.8.3
C_{vx}	VERTICAL SEISMIC DISTRIBUTION FACTOR, EQUATION 12.8-12
$h_n := 18 \cdot ft$	HEIGHT IN FEET ABOVE THE BASE TO THE HIGHEST LEVEL OF THE STRUCTURE
h_i	HEIGHT IN FEET ABOVE THE BASE OF THE STUCTURE TO LEVEL "i"
w_i	WEIGHT OF THE STUCTURE AT LEVEL "i"
D	SEISMIC DESIGN CATEGORY
W	SEISMIC WEIGHT OF STRUCTURE

$$T := C_t \cdot \left(\frac{h_n}{ft}\right)^x = 0.175 \qquad k := \text{if}\left(T \le 0.5, 1, \left(\text{if}\left(T \ge 2.5, 2, 1 + \frac{2.5 - T}{2}\right)\right)\right) = 1$$

$$S_{DS} := \frac{2}{3} \cdot F_a \cdot S_S \qquad S_{DS} = 0.846 \qquad S_{D1} := \frac{2}{3} \cdot F_v \cdot S_1 \qquad S_{D1} = 0.506$$

$$C_{s1} := \frac{S_{DS}}{\frac{R}{I}} \qquad C_{s2} := \frac{S_{D1}}{T \cdot \frac{R}{I}} \qquad C_{s3} := 0.01$$

$$C_s := \max\left(min\left(C_{s1}, C_{s2}\right), C_{s3}\right) \qquad C_s = 0.13$$

Permit Number: 20-00976

SEISMIC FORCES SHALL BE DISTRIBUTED VERTICALLY IN ACCORDANCE WITH SECTION 12.8.3

$w_2 := (37\ ft \cdot 36\ ft) \cdot 16\ psf$
$w_1 := (33\ ft \cdot 32\ ft + 12\ ft \cdot 24\ ft) \cdot (12\ psf + 10\ psf) + (7.5\ ft \cdot 17\ ft) \cdot 16\ psf$

$w_2 = 21312\ lbf$ $\quad h_2 := 18 \cdot ft$
$w_1 = 31608\ lbf$ $\quad h_1 := 10 \cdot ft$

THE SEISMIC FORCE AT EACH LEVEL OF THE STRUCTURE SHALL BE DETERMINED IN ACCORDANCE WITH EQUATIONS 12.8-11 AND 12.8-12

$$C_{v2} := \frac{w_2 \cdot h_2^{\ k}}{w_2 \cdot h_2^{\ k} + w_1 \cdot h_1^{\ k}} = 0.548 \qquad V_2 := C_{v2} \cdot C_s \cdot (w_2 + w_1) = 3774\ lbf$$

$$C_{v1} := \frac{w_1 \cdot h_1^{\ k}}{w_2 \cdot h_2^{\ k} + w_1 \cdot h_1^{\ k}} = 0.452 \qquad V_1 := C_{v1} \cdot C_s \cdot (w_2 + w_1) = 3110\ lbf$$

CONSERVATIVELY, ASSUME A REDUNDANCY FACTOR OF 1.3 PER ASCE 7-10 SECTION 12.3.4.2

$\rho := 1.3$

THE SEISMIC FORCE "V" IS A STRENTH LEVEL FORCE- REDUCE THE SEISMIC FORCE TO THE ALLOWABLE STRESS LEVEL "E"

$EE_2 := 0.7 \cdot V_2 \cdot \rho = 3435\ lbf$
$EE_1 := 0.7 \cdot V_1 \cdot \rho = 2830\ lbf$

THE CONTROLLING LOAD COMBINATION FOR THE SEISMIC ANALYSIS OF THE WOOD SHEARWALLS IS LOAD COMBINATION 16-16 OF THE 2015 IBC, SHOWN BELOW:

0.6(D+F)+0.7E+H

AS THE SHEARWALLS WILL NOT BE LOADED BY EITHER LATERAL EARTH PRESSURE, GROUND WATER PRESSURE OR PRESSURE DUE TO THE PRESENCE OF BULK MATERIALS, "H' MAY BE TAKEN AS EQUAL TO ZERO. THE COEFFICIENT "0.7" BEFORE THE SEISMIC LOAD EFFECT FACTOR "E" IS THE REDUCTION TERM TO CONVERT THE STRENGTH LEVEL FORCES TO THE ALLOWABLE STRESS LEVEL, THE EFFECTIVE LOAD COMBINATION IS AS FOLLOWS:

0.6D+E

Permit Number: 20-00976

WIND
USE THE DIRECTIONAL PROCEDURE PER ASCE 7-10 CHAPTER 27

WIND EXPOSURE = C 110mph 3-SECOND GUST WIND SPEED
RISK CATEGORY = II

$K_z := 0.87 \quad K_d := 0.85 \quad K_{zt} := 1.0 \quad V := 110$ MPH $\quad G := 0.85$

$q_z := 0.00256 \cdot K_z \cdot K_{zt} \cdot K_d \cdot V^2 \cdot psf = 22.9\ psf$ EQ 27.3-1

$C_{pww} := 0.8$ EXTERNAL PRESSURE COEFFICIENTS FOR WINDWARD/LEEWARD WALLS AND WINDWARD/LEEWARD ROOF SURFACE PER FIGURE 27.4-1

$C_{plw} := -0.5$

$C_{pwr} := -0.5$

$C_{plr} := -0.5$

$P_{ww} := C_{pww} \cdot G \cdot q_z = 15.6\ psf$ WIND PRESSURES ON WINDWARD/LEEWARD WALL AND ROOF SURFACES PER 27.4-1

$P_{lw} := C_{plw} \cdot G \cdot q_z = -9.7\ psf$

$P_{wr} := C_{pwr} \cdot G \cdot q_z = -9.7\ psf$

$P_{lr} := C_{plr} \cdot G \cdot q_z = -9.7\ psf$

THE CONTROLLING LOAD COMBINATION FOR THE WIND ANALYSIS OF THE WOOD SHEARWALLS IS LOAD COMBINATION 16-15, SHOWN BELOW:

0.6D+0.6W

WIND PERPENDICULAR TO ROOF RIDGE:

UPPER FLOOR:

$A_{wallU} := 0.5 \cdot 17\ ft \cdot 33\ ft = 280.5\ ft^2 \quad F_{wperpU} := 0.6\ (P_{ww} - P_{lw}) \cdot A_{wallU} = 4260\ lbf$

MAIN FLOOR:

$A_{wallM} := 0.5 \cdot 20\ ft \cdot 33\ ft = 330\ ft^2 \quad F_{wperpM} := 0.6\ (P_{ww} - P_{lw}) \cdot A_{wallM} = 5012\ lbf$

Permit Number: 20-00976

WIND PARALLEL TO ROOF RIDGE:

UPPER FLOOR:

$$A_{wallU} := (32\ ft \cdot 8\ ft \cdot 0.5) + (0.5 \cdot 9\ ft \cdot 32\ ft) = 272\ ft^2$$

$$F_{wparaU} := 0.6\ (P_{ww} - P_{lw}) \cdot A_{wallU} = 4131\ lbf$$

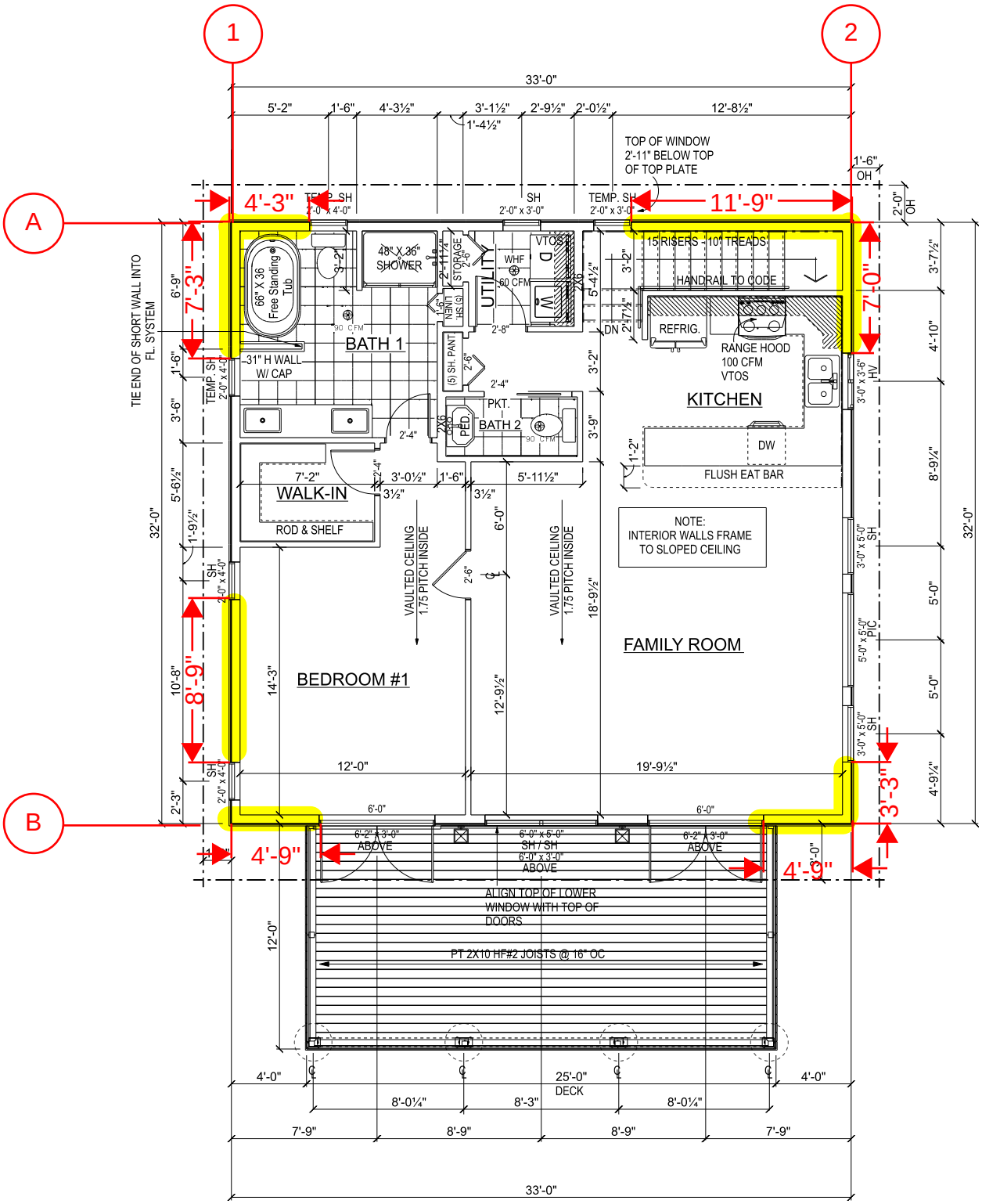
MAIN FLOOR:

$$A_{wallM} := (32\ ft \cdot 16\ ft \cdot 0.5) = 256\ ft^2$$

$$F_{wparaM} := 0.6\ (P_{ww} - P_{lw}) \cdot A_{wallM} = 3888\ lbf$$

LATERAL FORCES WILL BE DISTRIBUTED ALONG LINES OF FORCE/RESISTANCE. LINES OF FORCE/RESISTANCE WILL BE INVESTIGATED FOR BOTH WIND AND SEISMIC LATERAL LOADS. LATERAL FORCES WILL BE RESISTED BY PLYWOOD DIAPHRAGMS AND WOOD SHEARWALLS SUPPORTED ON CONCRETE FOUNDATIONS.

Permit Number: 20-00976



UPPER FLOOR PLAN
1/8" = 1'-0"

Upper Floor Walls

Upper Floor Wall Line A:

Total Wall Length: $L_{1a} := (4.25 + 11.75)\ ft$

Plate Height: $H_{1a} := 8 \cdot ft$

Total Wind Force: $V_{W1a} := (0.5 \cdot F_{wparaU}) = 2065\ lbf$

Wind Shear: $v_{W1a} := \frac{V_{W1a}}{L_{1a}} = 129\ plf$

Shear Wall Type P1-6
8d Nails @ 6" O.C.
Capacity= 339 PLF

Seismic Base Shear: $V_{E1a} := EE_2 \cdot 0.5 = 1717\ lbf$

Seismic Force: $v_{E1a} := \frac{V_{E1a}}{L_{1a}} = 107\ plf$

Shear Wall Type P1-6
8d Nails @ 6" O.C.
Capacity = 242 PLF

Check wall section: $l_{1a} := 4.25 \cdot ft$

Dead load resisting:

$Roof := 16 \cdot psf \cdot 16\ ft \cdot l_{1a} = 1088\ lbf$
$Wall := 10 \cdot psf \cdot H_{1a} \cdot l_{1a} = 340\ lbf$
$Total := Roof + Wall = 1428\ lbf$

Wind Overturning: $OTM_W := v_{W1a} \cdot l_{1a} \cdot H_{1a} = 4389\ lbf \cdot ft$

Seismic Overturning: $OTM_S := v_{E1a} \cdot l_{1a} \cdot H_{1a} = 3649\ lbf \cdot ft$

Wind Resisting Moment: $DLR_W := 0.6 \cdot Total \cdot \frac{l_{1a}}{2} = 1821\ lbf \cdot ft$

Seismic Resisting Moment: $DLR_S := (0.6 - 0.14 \cdot S_{DS}) \cdot Total \cdot \frac{l_{1a}}{2} = 1461\ lbf \cdot ft$

Net Uplift: $U_W := \frac{OTM_W - DLR_W}{l_{1a}} = 604\ lbf$

$U_S := \frac{OTM_S - DLR_S}{l_{1a}} = 515\ lbf$

$U := \max(U_W, U_S) = 604\ lbf$

No Hold Down Required

Permit Number: 20-00976

Upper Floor Wall Line B:

Total Wall Length: $L_{1b} := (4.75 + 4.75)\ ft$

Plate Height: $H_{1b} := 12 \cdot ft$

Total Wind Force: $V_{W1b} := (0.5 \cdot F_{wparaU}) = 2065\ lbf$

Wind Shear: $v_{W1b} := \frac{V_{W1b}}{L_{1b}} = 217\ plf$

Shear Wall Type P1-6
8d Nails @ 6" O.C.
Capacity= 339 PLF

Seismic Base Shear: $V_{E1b} := EE_2 \cdot 0.5 = 1717\ lbf$

Seismic Force: $v_{E1b} := \frac{V_{E1b}}{L_{1b}} = 181\ plf$

Check Aspect Ratio: $A_{1b} := \frac{v_{E1b}}{2\left(\frac{4.75 \cdot ft}{H_{1b}}\right)} = 228\ plf$

Shear Wall Type P1-6
8d Nails @ 6" O.C.
Capacity = 242 PLF

Check wall section: $l_{1b} := 4.75 \cdot ft$

Dead load resisting:

$Roof := 16 \cdot psf \cdot 16\ ft \cdot l_{1b} = 1216\ lbf$
$Wall := 10 \cdot psf \cdot H_{1b} \cdot l_{1b} = 570\ lbf$
$Total := Roof + Wall = 1786\ lbf$

Wind Overturning: $OTM_W := v_{W1b} \cdot l_{1b} \cdot H_{1b} = 12393\ lbf \cdot ft$

Seismic Overturning: $OTM_S := v_{E1b} \cdot l_{1b} \cdot H_{1b} = 10304\ lbf \cdot ft$

Wind Resisting Moment: $DLR_W := 0.6 \cdot Total \cdot \frac{l_{1b}}{2} = 2545\ lbf \cdot ft$

Seismic Resisting Moment: $DLR_S := (0.6 - 0.14 \cdot S_{DS}) \cdot Total \cdot \frac{l_{1b}}{2} = 2043\ lbf \cdot ft$

Net Uplift: $U_W := \frac{OTM_W - DLR_W}{l_{1b}} = 2073\ lbf$

$U_S := \frac{OTM_S - DLR_S}{l_{1b}} = 1739\ lbf$

$U := \max(U_W, U_S) = 2073\ lbf$

Use Simpson MST48

Permit Number: 20-00976

Upper Floor Wall Line 1:

Total Wall Length: $L_{11} := (7.25 + 8.75)\ ft$

Plate Height: $H_{11} := 8 \cdot ft$

Total Wind Force: $V_{W11} := (0.5 \cdot F_{wperpU}) = 2130\ lbf$

Wind Shear: $v_{W11} := \frac{V_{W11}}{L_{11}} = 133\ plf$

Shear Wall Type P1-6
8d Nails @ 6" O.C.
Capacity= 339 PLF

Seismic Base Shear: $V_{E11} := EE_2 \cdot 0.5 = 1717\ lbf$

Seismic Force: $v_{E11} := \frac{V_{E11}}{L_{11}} = 107\ plf$

Shear Wall Type P1-6
8d Nails @ 6" O.C.
Capacity = 242 PLF

Check wall section: $l_{11} := 7.25 \cdot ft$

Dead load resisting:

$Roof := 16 \cdot psf \cdot 2\ ft \cdot l_{11} = 232\ lbf$

$Wall := 10 \cdot psf \cdot H_{11} \cdot l_{11} = 580\ lbf$

$Total := Roof + Wall = 812\ lbf$

Wind Overturning: $OTM_W := v_{W11} \cdot l_{11} \cdot H_{11} = 7721\ lbf \cdot ft$

Seismic Overturning: $OTM_S := v_{E11} \cdot l_{11} \cdot H_{11} = 6225\ lbf \cdot ft$

Wind Resisting Moment: $DLR_W := 0.6 \cdot Total \cdot \frac{l_{11}}{2} = 1766\ lbf \cdot ft$

Seismic Resisting Moment: $DLR_S := (0.6 - 0.14 \cdot S_{DS}) \cdot Total \cdot \frac{l_{11}}{2} = 1418\ lbf \cdot ft$

Net Uplift: $U_W := \frac{OTM_W - DLR_W}{l_{11}} = 821\ lbf$

$U_S := \frac{OTM_S - DLR_S}{l_{11}} = 663\ lbf$

$U := \max(U_W, U_S) = 821\ lbf$

Use Simpson MST37

Permit Number: 20-00976

Upper Floor Wall Line 2:

Total Wall Length: $L_{12} := (7 + 3.25)\ ft$

Plate Height: $H_{12} := 8 \cdot ft$

Total Wind Force: $V_{W12} := (0.5 \cdot F_{wperpU}) = 2130\ lbf$

Wind Shear: $v_{W12} := \frac{V_{W12}}{L_{12}} = 208\ plf$

Shear Wall Type P1-4
8d Nails @ 4" O.C.
Capacity = 495 PLF

Seismic Base Shear: $V_{E12} := EE_2 \cdot 0.5 = 1717\ lbf$

Seismic Force: $v_{E12} := \frac{V_{E12}}{L_{12}} = 168\ plf$

Check Aspect Ratio: $A_{12} := \frac{v_{E12}}{2\left(\frac{3.25 \cdot ft}{H_{12}}\right)} = 206\ plf$

Shear Wall Type P1-4
8d Nails @ 4" O.C.
Capacity = 353 PLF

Check wall section: $l_{12} := 3.25 \cdot ft$

Dead load resisting:

$Roof := 16 \cdot psf \cdot 2\ ft \cdot l_{12} = 104\ lbf$
$Wall := 10 \cdot psf \cdot H_{12} \cdot l_{12} = 260\ lbf$
$Total := Roof + Wall = 364\ lbf$

Wind Overturning: $OTM_W := v_{W12} \cdot l_{12} \cdot H_{12} = 5403\ lbf \cdot ft$

Seismic Overturning: $OTM_S := v_{E12} \cdot l_{12} \cdot H_{12} = 4356\ lbf \cdot ft$

Wind Resisting Moment: $DLR_W := 0.6 \cdot Total \cdot \frac{l_{12}}{2} = 355\ lbf \cdot ft$

Seismic Resisting Moment: $DLR_S := (0.6 - 0.14 \cdot S_{DS}) \cdot Total \cdot \frac{l_{12}}{2} = 285\ lbf \cdot ft$

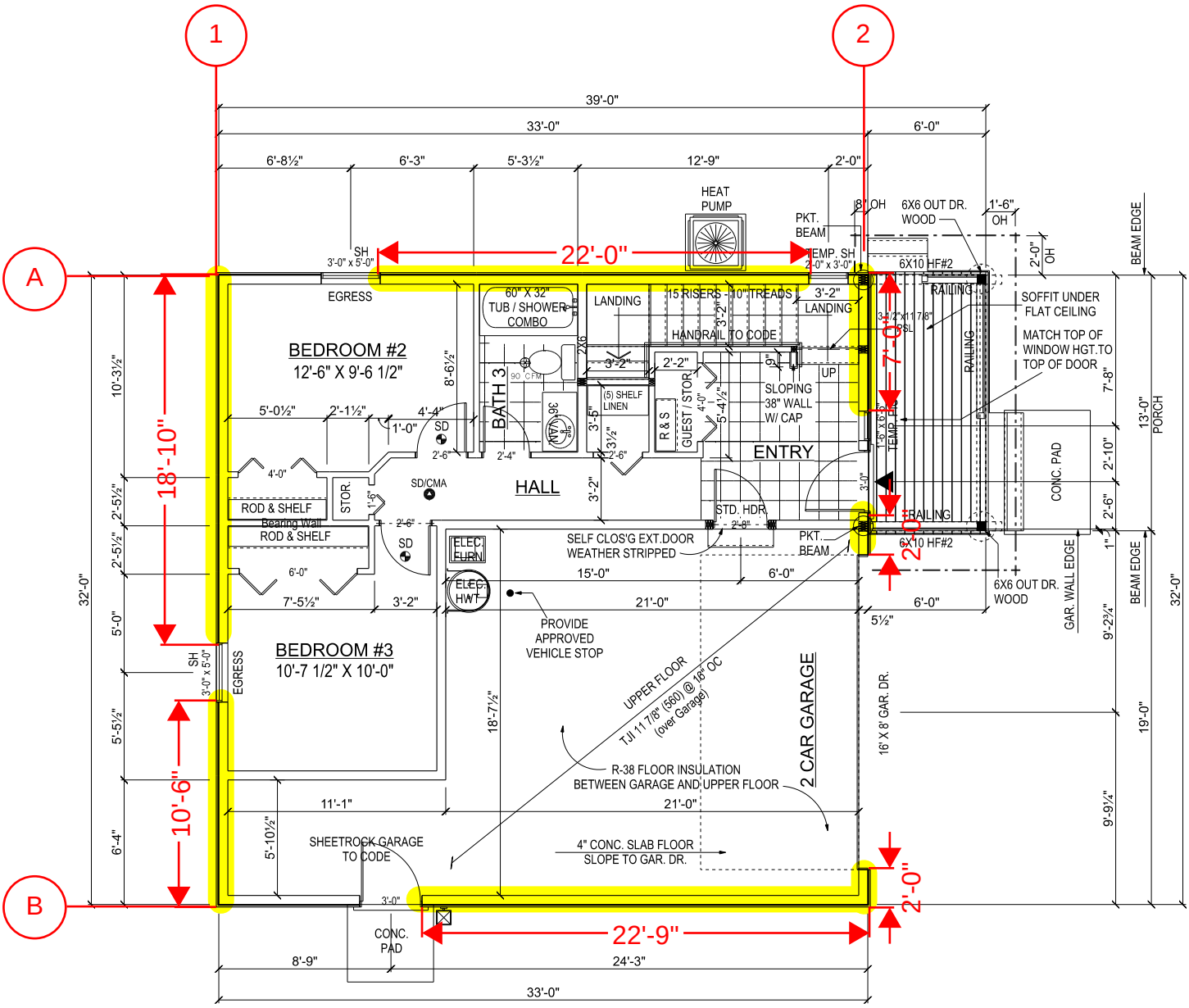
Net Uplift: $U_W := \frac{OTM_W - DLR_W}{l_{12}} = 1553\ lbf$

$U_S := \frac{OTM_S - DLR_S}{l_{12}} = 1253\ lbf$

$U := \max(U_W, U_S) = 1553\ lbf$

Use Simpson MST48

Permit Number: 20-00976



MAIN FLOOR PLAN
1/8" = 1'-0"

Main Floor Walls

Main Floor Wall Line A:

Total Wall Length: $L_{2a} := (22)\ ft$

Plate Height: $H_{2a} := 8 \cdot ft$

Total Wind Force: $V_{W2a} := (F_{wparaM}) \cdot 0.5 + V_{W1a} = 4009\ lbf$

Wind Shear: $v_{W2a} := \frac{V_{W2a}}{L_{2a}} = 182\ plf$

Shear Wall Type P1-6
8d Nails @ 6" O.C.
Capacity= 339 PLF

Seismic Base Shear: $V_{E2a} := EE_1 \cdot 0.5 + V_{E1a} = 3132\ lbf$

Seismic Force: $v_{E2a} := \frac{V_{E2a}}{L_{2a}} = 142\ plf$

Shear Wall Type P1-6
8d Nails @ 6" O.C.
Capacity = 242 PLF

Check wall section: $l_{2a} := 22\ ft$

Dead load resisting:

$Roof := 16 \cdot psf \cdot 13\ ft \cdot l_{2a} = 4576\ lbf$
$Wall := 10 \cdot psf \cdot H_{2a} \cdot l_{2a} = (1.76 \cdot 10^3)\ lbf$
$Floor := 12 \cdot psf \cdot 5\ ft \cdot l_{2a} = (1.32 \cdot 10^3)\ lbf$
$Total := Roof + Wall + Floor = 7656\ lbf$

Wind Overturning: $OTM_W := v_{W2a} \cdot l_{2a} \cdot H_{2a} = 32075\ lbf \cdot ft$

Seismic Overturning: $OTM_S := v_{E2a} \cdot l_{2a} \cdot H_{2a} = 25058\ lbf \cdot ft$

Wind Resisting Moment: $DLR_W := 0.6 \cdot Total \cdot \frac{l_{2a}}{2} = 50530\ lbf \cdot ft$

Seismic Resisting Moment: $DLR_S := (0.6 - 0.14 \cdot S_{DS}) \cdot Total \cdot \frac{l_{2a}}{2} = 40561\ lbf \cdot ft$

Net Uplift: $U_W := \frac{OTM_W - DLR_W}{l_{2a}} = -839\ lbf$

$U_S := \frac{OTM_S - DLR_S}{l_{2a}} = -705\ lbf$

$U := \max(U_W, U_S) = -705\ lbf$ No Hold Down Required

Permit Number: 20-00976

Main Floor Wall Line B:

Total Wall Length: $L_{2b} := (22.75)\ ft$

Plate Height: $H_{2b} := 8 \cdot ft$

Total Wind Force: $V_{W2b} := (F_{wparaM}) \cdot 0.5 + V_{W1b} = 4009\ lbf$

Wind Shear: $v_{W2b} := \frac{V_{W2b}}{L_{2b}} = 176\ plf$

Shear Wall Type P1-6
8d Nails @ 6" O.C.
Capacity= 339 PLF

Seismic Base Shear: $V_{E2b} := EE_1 \cdot (0.5) + V_{E1b} = 3132\ lbf$

Seismic Force: $v_{E2b} := \frac{V_{E2b}}{L_{2b}} = 138\ plf$

Shear Wall Type P1-6
8d Nails @ 6" O.C.
Capacity = 242 PLF

Check wall section: $l_{2b} := 22.75 \cdot ft$

Dead load resisting:

$Roof := 16 \cdot psf \cdot 13\ ft \cdot l_{2b} = 4732\ lbf$
$Wall := 10 \cdot psf \cdot H_{2b} \cdot l_{2b} = (1.82 \cdot 10^3)\ lbf$
$Floor := 12 \cdot psf \cdot 5\ ft \cdot l_{2b} = (1.365 \cdot 10^3)\ lbf$
$Total := Roof + Wall + Floor = 7917\ lbf$

Wind Overturning: $OTM_W := v_{W2b} \cdot l_{2b} \cdot H_{2b} = 32075\ lbf \cdot ft$

Seismic Overturning: $OTM_S := v_{E2b} \cdot l_{2b} \cdot H_{2b} = 25058\ lbf \cdot ft$

Wind Resisting Moment: $DLR_W := 0.6 \cdot Total \cdot \frac{l_{2b}}{2} = 54034\ lbf \cdot ft$

Seismic Resisting Moment: $DLR_S := (0.6 - 0.14 \cdot S_{DS}) \cdot Total \cdot \frac{l_{2b}}{2} = 43373\ lbf \cdot ft$

Net Uplift: $U_W := \frac{OTM_W - DLR_W}{l_{2b}} = -965\ lbf$

$U_S := \frac{OTM_S - DLR_S}{l_{2b}} = -805\ lbf$

$U := \max(U_W, U_S) = -805\ lbf$ No Hold Down Required

Permit Number: 20-00976

Main Floor Wall Line 1:

Total Wall Length: $L_{21} := (10.5 + 18.833)\ ft$

Plate Height: $H_{21} := 8 \cdot ft$

Total Wind Force: $V_{W21} := (F_{wperpM}) \cdot 0.5 + V_{W11} = 4636\ lbf$

Wind Shear: $v_{W21} := \frac{V_{W21}}{L_{21}} = 158\ plf$

Shear Wall Type P1-6
8d Nails @ 6" O.C.
Capacity= 339 PLF

Seismic Base Shear: $V_{E21} := EE_1 \cdot (0.5) + V_{E11} = 3132\ lbf$

Seismic Force: $v_{E21} := \frac{V_{E21}}{L_{21}} = 107\ plf$

Shear Wall Type P1-6
8d Nails @ 6" O.C.
Capacity = 242 PLF

Check wall section: $l_{21} := 10.5 \cdot ft$

Dead load resisting:

$Roof := 16 \cdot psf \cdot 2\ ft \cdot l_{21} = 336\ lbf$
$Wall := 10 \cdot psf \cdot H_{21} \cdot l_{21} = 840\ lbf$
$Floor := 12 \cdot psf \cdot 2\ ft \cdot l_{21} = 252\ lbf$
$Total := Roof + Wall + Floor = 1428\ lbf$

Wind Overturning: $OTM_W := v_{W21} \cdot l_{21} \cdot H_{21} = 13276\ lbf \cdot ft$

Seismic Overturning: $OTM_S := v_{E21} \cdot l_{21} \cdot H_{21} = 8970\ lbf \cdot ft$

Wind Resisting Moment: $DLR_W := 0.6 \cdot Total \cdot \frac{l_{21}}{2} = 4498\ lbf \cdot ft$

Seismic Resisting Moment: $DLR_S := (0.6 - 0.14 \cdot S_{DS}) \cdot Total \cdot \frac{l_{21}}{2} = 3611\ lbf \cdot ft$

Net Uplift: $U_W := \frac{OTM_W - DLR_W}{l_{21}} = 836\ lbf$

$U_S := \frac{OTM_S - DLR_S}{l_{21}} = 510\ lbf$

$U := \max(U_W, U_S) = 836\ lbf$

Use Simpson STHD10

Permit Number: 20-00976

Main Floor Wall Line 2:

Total Wall Length: $L_{22} := (7+2+2)\ ft$

Plate Height: $H_{22} := 6 \cdot ft$

Total Wind Force: $V_{W22} := (F_{wperpM}) \cdot 0.5 + V_{W21} = 7142\ lbf$

Wind Shear: $v_{W22} := \frac{V_{W22}}{L_{22}} = 649\ plf$

Shear Wall Type P1-2
8d Nails @ 2" O.C.
Capacity = 832 PLF

Seismic Base Shear: $V_{E22} := EE_1 \cdot 0.5 + V_{E12} = 3132\ lbf$

Seismic Force: $v_{E22} := \frac{V_{E22}}{L_{22}} = 285\ plf$

Check Aspect Ratio: $A_{22} := \frac{v_{E22}}{2\left(\frac{2 \cdot ft}{H_{22}}\right)} = 427\ plf$

Shear Wall Type P1-2
8d Nails @ 2" O.C.
Capacity = 595 PLF

Check wall section: $l_{22} := 2 \cdot ft$

Dead load resisting:

$Roof := 16 \cdot psf \cdot 3\ ft \cdot l_{22} = 96\ lbf$
$Wall := 10 \cdot psf \cdot H_{22} \cdot l_{22} = 120\ lbf$
$Floor := 12 \cdot psf \cdot 2\ ft \cdot l_{22} = 48\ lbf$
$Total := Roof + Wall + Floor = 264\ lbf$

Wind Overturning: $OTM_W := v_{W22} \cdot l_{22} \cdot H_{22} = 7791\ lbf \cdot ft$

Seismic Overturning: $OTM_S := v_{E22} \cdot l_{22} \cdot H_{22} = 3417\ lbf \cdot ft$

Wind Resisting Moment: $DLR_W := 0.6 \cdot Total \cdot \frac{l_{22}}{2} = 158\ lbf \cdot ft$

Seismic Resisting Moment: $DLR_S := (0.6 - 0.14 \cdot S_{DS}) \cdot Total \cdot \frac{l_{22}}{2} = 127\ lbf \cdot ft$

Net Uplift: $U_W := \frac{OTM_W - DLR_W}{l_{22}} = 3816\ lbf$

$U_S := \frac{OTM_S - DLR_S}{l_{22}} = 1645\ lbf$

$U := \max(U_W, U_S) = 3816\ lbf$

Use Simpson STHD14

Permit Number: 20-00976