LONGITUDE
ONE TWENTY°
ENGINEERING & DESIGN

Calculation Package for

QUI RESIDENCE REMODEL

8028 SE 36TH ST
MERCER ISLAND, WA 98040

PROJECT #: S200831-6

DATE: 09/02/20

MANS R. THURFJELL
STATE OF WASHINGTON
44745
REGISTERED
PROFESSIONAL ENGINEER

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Project Number:	Plan Name:	Sheet Number:
S200831-6	Qui Residence Remodel	DC
Engineer: xxx	Specifics: Design Criteria	Date: 9/2/2020

Gravity Criteria: BLUE = Review and update as required - Typical Input Code: IBC 2015

ROOF SYSTEM		
Live Load:		
Snow	25.0	psf
Dead Load:		
Composite Roofing	2.0	psf
19/32" Plywood Sheathing	2.5	psf
Trusses at 24" o.c.	3.0	psf
Insulation	1.8	psf
(2) Layers 5/8" GWB	4.4	psf
Misc or Tile Roof	1.3	psf
Total	**15.0**	**psf**

FLOOR SYSTEM		
Live Load:		
Residential	40.0	psf
Dead Load:		
Flooring	3.0	psf
3/4" T & G Plywood	2.5	psf
Floor Joists at 16" o.c.	2.5	psf
Insulation	0.5	psf
(1) Layers 5/8" GWB	2.2	psf
Misc or Tile Flooring	1.3	psf
Total	**12.0**	**psf**

EXTERIOR WALL SYSTEM		
2x6 at 16" o.c.	1.7	psf
Insulation	1.0	psf
1/2" Plywood Sheathing	1.5	psf
(2) layers 5/8" GWB	4.4	psf
Misc or Brick Covered Wall	3.4	psf
Total	**12.0**	**psf**

INTERIOR WALL SYSTEM		
2x4 at 16" o.c.	1.1	psf
Insulation	0.5	psf
(2) Layers 5/8" GWB	4.4	psf
Misc	2.0	psf
Total	**8.0**	**psf**

SEISMIC PARAMETERS:

Code Reference: ASCE 7-10
R = **6.5** Bearing Wall System, Wood Structural Panel Walls
Mapped Spectral Acceleration, Ss = **1.406**
Mapped Spectral Acceleration, S1 = **0.535**
Soil Site Class = **D**

WIND PARAMETERS:

Code Reference: ASCE 7-10
Basic Wind Speed (3 second Gust) = **110** mph
Exposure : **B**
Kzt = **1.40**

SOIL PARAMETERS:

Soil Bearing Pressure = **1,500** psf competent native soil or structural fill
1/3 increase for short-term wind or seismic loading is acceptable
Frost Depth = **18** in

Lateral Wall Pressures:
Unrestrained Active Pressure = **35** pcf Cantilevered walls
Restrained Active Pressure = **50** pcf Plate Wall Design/Tank Walls
Passive Pressure = **350** pcf
Soil Friction Coeff. = **0.35**

LONGITUDE
ONE TWENTY°
ENGINEERING & DESIGN

FRAMING CALCULATIONS

BEAM REFERENCE PER PLAN

FAILED

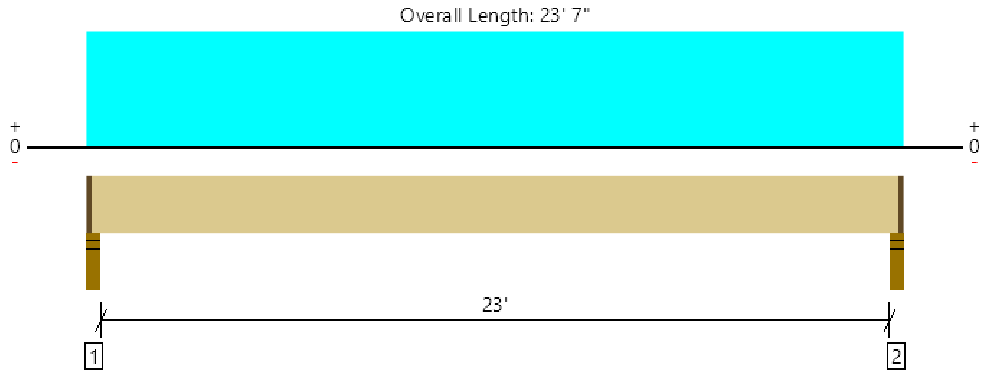
MEMBER REPORT

Roof, GT-1 (RXN ONLY)

3 piece(s) 1 3/4" x 11 7/8" 1.55E TimberStrand® LSL

Support 1 failed reaction check due to insufficient bearing capacity.

Support 2 failed reaction check due to insufficient bearing capacity.



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)	6305 @ 2"	4784 (2.25")	Failed (132%)	--	1.0 D + 1.0 S (All Spans)
Shear (lbs)	5670 @ 1' 3 3/8"	14817	Passed (38%)	1.15	1.0 D + 1.0 S (All Spans)
Moment (Ft-lbs)	36452 @ 11' 9 1/2"	27519	Failed (132%)	1.15	1.0 D + 1.0 S (All Spans)
Live Load Defl. (in)	1.934 @ 11' 9 1/2"	0.581	Failed (L/144)	--	1.0 D + 1.0 S (All Spans)
Total Load Defl. (in)	3.210 @ 11' 9 1/2"	1.163	Failed (L/87)	--	1.0 D + 1.0 S (All Spans)

- Deflection criteria: LL (L/480) and TL (L/240).
- Allowed moment does not reflect the adjustment for the beam stability factor.

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

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Snow	Total	
1 - Stud wall - HF	3.50"	2.25"	2.97"	2527	3832	6359	1 1/4" Rim Board
2 - Stud wall - HF	3.50"	2.25"	2.97"	2527	3832	6359	1 1/4" Rim Board

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

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	6" o/c	
Bottom Edge (Lu)	23' 5" o/c	

- Maximum allowable bracing intervals based on applied load.

Vertical Loads	Location (Side)	Tributary Width	Dead (0.90)	Snow (1.15)	Comments
0 - Self Weight (PLF)	1 1/4" to 23' 5 3/4"	N/A	19.5	--	
1 - Uniform (PSF)	0 to 23' 7" (Front)	13'	15.0	25.0	Default Load

Weyerhaeuser Notes

ForteWEB Software Operator	Job Notes
Kenny Jones L120 Engineering (817) 727-2136 kjones@l120engineering.com	

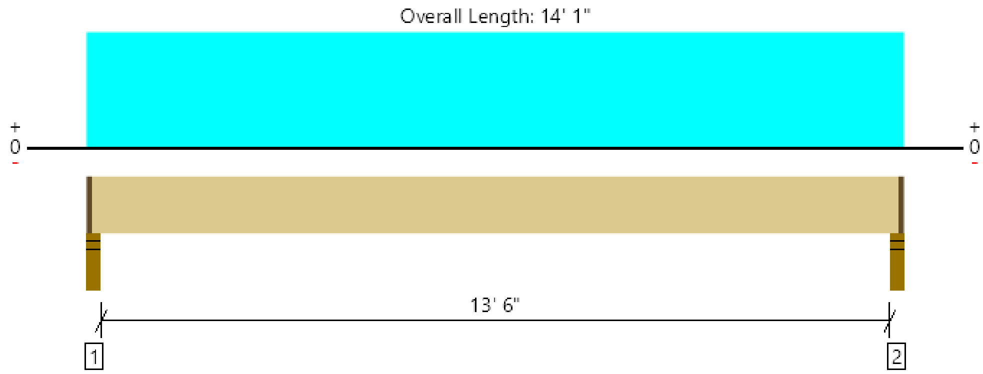
Weyerhaeuser

MEMBER REPORT

PASSED

Roof, RJ-1

1 piece(s) 2 x 10 Hem-Fir No. 2 @ 24" 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)	555 @ 2 1/2"	1367 (2.25")	Passed (41%)	--	1.0 D + 1.0 S (All Spans)
Shear (lbs)	478 @ 1' 3/4"	1596	Passed (30%)	1.15	1.0 D + 1.0 S (All Spans)
Moment (Ft-lbs)	1868 @ 7' 1/2"	2204	Passed (85%)	1.15	1.0 D + 1.0 S (All Spans)
Live Load Defl. (in)	0.305 @ 7' 1/2"	0.342	Passed (L/537)	--	1.0 D + 1.0 S (All Spans)
Total Load Defl. (in)	0.488 @ 7' 1/2"	0.683	Passed (L/336)	--	1.0 D + 1.0 S (All Spans)
TJ-Pro™ Rating	N/A	N/A	N/A	--	N/A

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

- Deflection criteria: LL (L/480) and TL (L/240).
- Allowed moment does not reflect the adjustment for the beam stability factor.
- A 15% increase in the moment capacity has been added to account for repetitive member usage.
- Applicable calculations are based on NDS.
- No composite action between deck and joist was considered in analysis.

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Snow	Total	
1 - Stud wall - HF	3.50"	2.25"	1.50"	211	352	563	1 1/4" Rim Board
2 - Stud wall - HF	3.50"	2.25"	1.50"	211	352	563	1 1/4" Rim Board

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

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	4' 2" o/c	
Bottom Edge (Lu)	13' 11" o/c	

•Maximum allowable bracing intervals based on applied load.

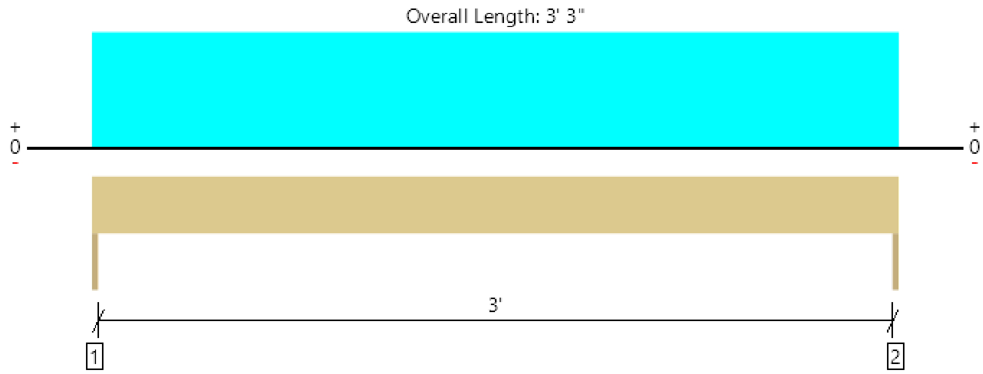
Vertical Load	Location (Side)	Spacing	Dead (0.90)	Snow (1.15)	Comments
1 - Uniform (PSF)	0 to 14' 1"	24"	15.0	25.0	roof

Weyerhaeuser Notes

ForteWEB Software Operator	Job Notes
Kenny Jones L120 Engineering (817) 727-2136 kjones@l120engineering.com	

Roof, RH-1

1 piece(s) 4 x 6 Douglas Fir-Larch No. 2



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)	853 @ 0	3281 (1.50")	Passed (26%)	--	1.0 D + 1.0 S (All Spans)
Shear (lbs)	547 @ 7"	2657	Passed (21%)	1.15	1.0 D + 1.0 S (All Spans)
Moment (Ft-lbs)	693 @ 1' 7 1/2"	1979	Passed (35%)	1.15	1.0 D + 1.0 S (All Spans)
Live Load Defl. (in)	0.011 @ 1' 7 1/2"	0.108	Passed (L/999+)	--	1.0 D + 1.0 S (All Spans)
Total Load Defl. (in)	0.017 @ 1' 7 1/2"	0.162	Passed (L/999+)	--	1.0 D + 1.0 S (All Spans)

System : Wall
Member Type : Header
Building Use : Residential
Building Code : IBC 2015
Design Methodology : ASD

- Deflection criteria: LL (L/360) and TL (L/240).
- Allowed moment does not reflect the adjustment for the beam stability factor.
- Applicable calculations are based on NDS.

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Snow	Total	
1 - Trimmer - HF	1.50"	1.50"	1.50"	325	528	853	None
2 - Trimmer - HF	1.50"	1.50"	1.50"	325	528	853	None

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	3' 3" o/c	
Bottom Edge (Lu)	3' 3" o/c	

•Maximum allowable bracing intervals based on applied load.

Vertical Loads	Location (Side)	Tributary Width	Dead (0.90)	Snow (1.15)	Comments
0 - Self Weight (PLF)	0 to 3' 3"	N/A	4.9	--	
1 - Uniform (PSF)	0 to 3' 3"	13'	15.0	25.0	ROOF

Weyerhaeuser Notes

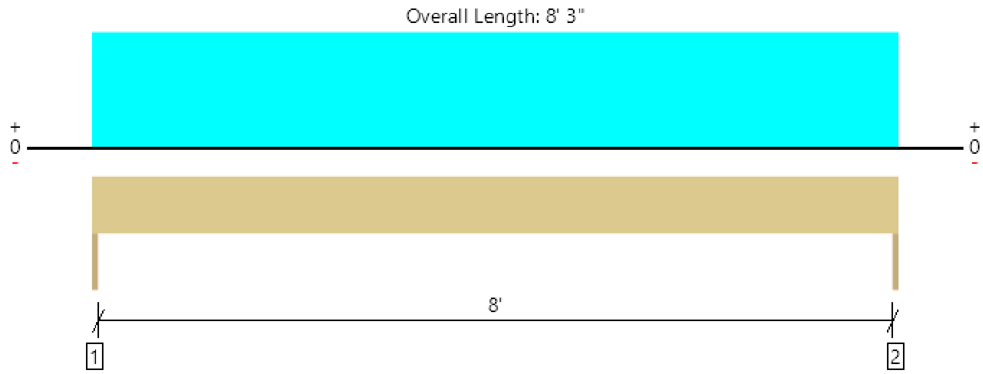
ForteWEB Software Operator	Job Notes
Kenny Jones L120 Engineering (817) 727-2136 kjones@l120engineering.com	

MEMBER REPORT

PASSED

Roof, RH-2

1 piece(s) 4 x 6 Douglas Fir-Larch No. 2



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)	515 @ 0	3281 (1.50")	Passed (16%)	--	1.0 D + 1.0 S (All Spans)
Shear (lbs)	442 @ 7"	2657	Passed (17%)	1.15	1.0 D + 1.0 S (All Spans)
Moment (Ft-lbs)	1062 @ 4' 1 1/2"	1979	Passed (54%)	1.15	1.0 D + 1.0 S (All Spans)
Live Load Defl. (in)	0.101 @ 4' 1 1/2"	0.275	Passed (L/983)	--	1.0 D + 1.0 S (All Spans)
Total Load Defl. (in)	0.168 @ 4' 1 1/2"	0.313	Passed (L/591)	--	1.0 D + 1.0 S (All Spans)

System : Wall
Member Type : Header
Building Use : Residential
Building Code : IBC 2015
Design Methodology : ASD

- Deflection criteria: LL (L/360) and TL (L/5/16").
- Allowed moment does not reflect the adjustment for the beam stability factor.
- Applicable calculations are based on NDS.

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Snow	Total	
1 - Trimmer - HF	1.50"	1.50"	1.50"	206	309	515	None
2 - Trimmer - HF	1.50"	1.50"	1.50"	206	309	515	None

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	8' 3" o/c	
Bottom Edge (Lu)	8' 3" o/c	

•Maximum allowable bracing intervals based on applied load.

Vertical Loads	Location (Side)	Tributary Width	Dead (0.90)	Snow (1.15)	Comments
0 - Self Weight (PLF)	0 to 8' 3"	N/A	4.9	--	
1 - Uniform (PSF)	0 to 8' 3"	3'	15.0	25.0	ROOF

Weyerhaeuser Notes

ForteWEB Software Operator	Job Notes
Kenny Jones L120 Engineering (817) 727-2136 kjones@l120engineering.com	

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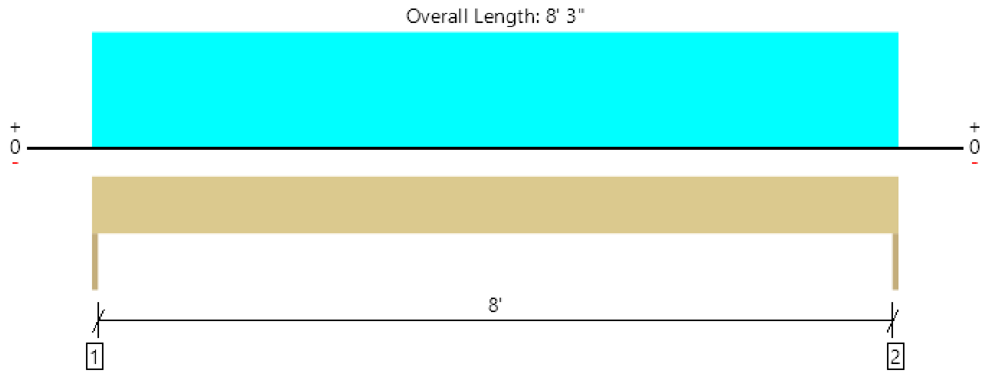
MEMBER REPORT

PASSED

Roof, RH-3

1 piece(s) 4 x 6 Douglas Fir-Larch No. 2



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)	515 @ 0	3281 (1.50")	Passed (16%)	--	1.0 D + 1.0 S (All Spans)
Shear (lbs)	442 @ 7"	2657	Passed (17%)	1.15	1.0 D + 1.0 S (All Spans)
Moment (Ft-lbs)	1062 @ 4' 1 1/2"	1979	Passed (54%)	1.15	1.0 D + 1.0 S (All Spans)
Live Load Defl. (in)	0.101 @ 4' 1 1/2"	0.275	Passed (L/983)	--	1.0 D + 1.0 S (All Spans)
Total Load Defl. (in)	0.168 @ 4' 1 1/2"	0.313	Passed (L/591)	--	1.0 D + 1.0 S (All Spans)

System : Wall
Member Type : Header
Building Use : Residential
Building Code : IBC 2015
Design Methodology : ASD

- Deflection criteria: LL (L/360) and TL (L/5/16").
- Allowed moment does not reflect the adjustment for the beam stability factor.
- Applicable calculations are based on NDS.

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Snow	Total	
1 - Trimmer - HF	1.50"	1.50"	1.50"	206	309	515	None
2 - Trimmer - HF	1.50"	1.50"	1.50"	206	309	515	None

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	8' 3" o/c	
Bottom Edge (Lu)	8' 3" o/c	

•Maximum allowable bracing intervals based on applied load.

Vertical Loads	Location (Side)	Tributary Width	Dead (0.90)	Snow (1.15)	Comments
0 - Self Weight (PLF)	0 to 8' 3"	N/A	4.9	--	
1 - Uniform (PSF)	0 to 8' 3"	3'	15.0	25.0	ROOF

Weyerhaeuser Notes

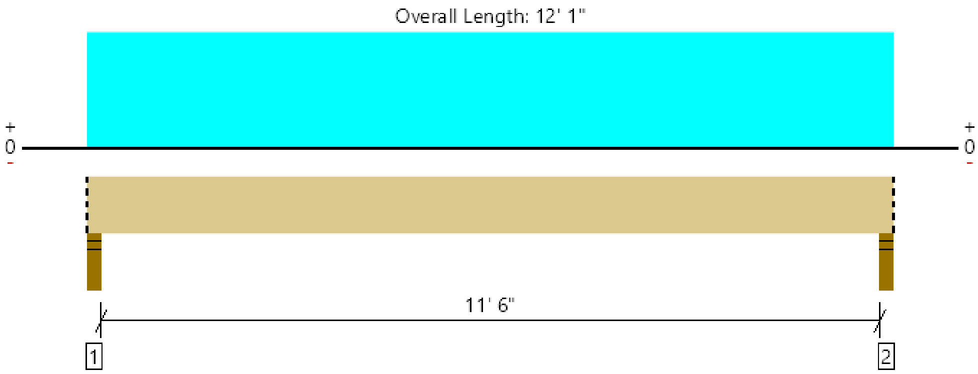
ForteWEB Software Operator	Job Notes
Kenny Jones L120 Engineering (817) 727-2136 kjones@l120engineering.com	

MEMBER REPORT

PASSED

Roof, RH-4

1 piece(s) 5 1/2" x 7 1/2" 24F-V4 DF Glulam



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)	2236 @ 2"	8181 (3.50")	Passed (27%)	--	1.0 D + 1.0 S (All Spans)
Shear (lbs)	1896 @ 11"	8381	Passed (23%)	1.15	1.0 D + 1.0 S (All Spans)
Pos Moment (Ft-lbs)	6386 @ 6' 1/2"	11859	Passed (54%)	1.15	1.0 D + 1.0 S (All Spans)
Live Load Defl. (in)	0.277 @ 6' 1/2"	0.392	Passed (L/509)	--	1.0 D + 1.0 S (All Spans)
Total Load Defl. (in)	0.456 @ 6' 1/2"	0.587	Passed (L/309)	--	1.0 D + 1.0 S (All Spans)

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

- Deflection criteria: LL (L/360) and TL (L/240).
- Allowed moment does not reflect the adjustment for the beam stability factor.
- Critical positive moment adjusted by a volume factor of 1.00 that was calculated using length L = 11' 9".
- The effects of positive or negative camber have not been accounted for when calculating deflection.
- The specified glulam is assumed to have its strong laminations at the bottom of the beam. Install with proper side up as indicated by the manufacturer.
- Applicable calculations are based on NDS.

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Snow	Total	
1 - Stud wall - SPF	3.50"	3.50"	1.50"	876	1359	2235	Blocking
2 - Stud wall - SPF	3.50"	3.50"	1.50"	876	1359	2235	Blocking

- Blocking Panels are assumed to carry no loads applied directly above them and the full load is applied to the member being designed.

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	12' 1" o/c	
Bottom Edge (Lu)	12' 1" o/c	

•Maximum allowable bracing intervals based on applied load.

Vertical Loads	Location (Side)	Tributary Width	Dead (0.90)	Snow (1.15)	Comments
0 - Self Weight (PLF)	0 to 12' 1"	N/A	10.0	--	
1 - Uniform (PSF)	0 to 12' 1" (Front)	9'	15.0	25.0	Roof

Weyerhaeuser Notes

ForteWEB Software Operator	Job Notes
Kenny Jones L120 Engineering (817) 727-2136 kjones@l120engineering.com	

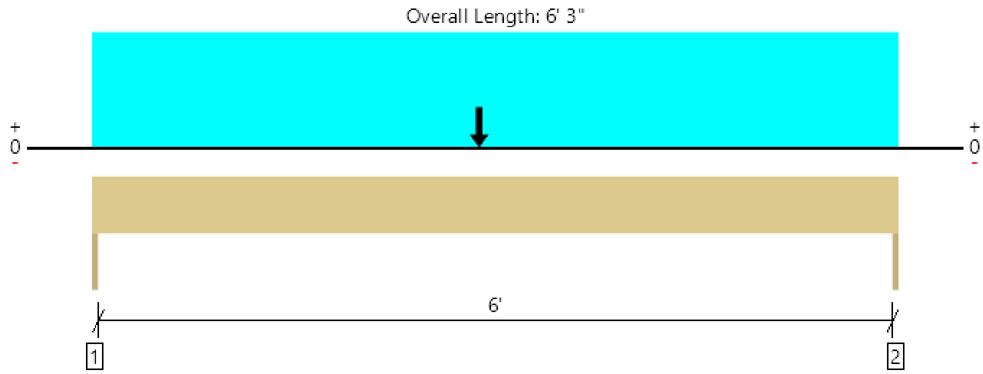
FORTE WEB

MEMBER REPORT

PASSED

Roof, RH-4.1

1 piece(s) 3 1/2" x 9" 24F-V4 DF Glulam



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)	2311 @ 0	3413 (1.50")	Passed (68%)	--	1.0 D + 1.0 S (All Spans)
Shear (lbs)	1989 @ 10 1/2"	6400	Passed (31%)	1.15	1.0 D + 1.0 S (All Spans)
Pos Moment (Ft-lbs)	5279 @ 3'	10868	Passed (49%)	1.15	1.0 D + 1.0 S (All Spans)
Live Load Defl. (in)	0.051 @ 3' 1 5/16"	0.208	Passed (L/999+)	--	1.0 D + 1.0 S (All Spans)
Total Load Defl. (in)	0.084 @ 3' 1 5/16"	0.313	Passed (L/891)	--	1.0 D + 1.0 S (All Spans)

System : Wall
Member Type : Header
Building Use : Residential
Building Code : IBC 2015
Design Methodology : ASD

- Deflection criteria: LL (L/360) and TL (L/240).
- Allowed moment does not reflect the adjustment for the beam stability factor.
- Critical positive moment adjusted by a volume factor of 1.00 that was calculated using length L = 6' 3".
- The effects of positive or negative camber have not been accounted for when calculating deflection.
- The specified glulam is assumed to have its strong laminations at the bottom of the beam. Install with proper side up as indicated by the manufacturer.
- Applicable calculations are based on NDS.

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Snow	Total	
1 - Trimmer - HF	1.50"	1.50"	1.50"	901	1410	2311	None
2 - Trimmer - HF	1.50"	1.50"	1.50"	866	1355	2221	None

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	6' 3" o/c	
Bottom Edge (Lu)	6' 3" o/c	

•Maximum allowable bracing intervals based on applied load.

Vertical Loads	Location (Side)	Tributary Width	Dead (0.90)	Snow (1.15)	Comments
0 - Self Weight (PLF)	0 to 6' 3"	N/A	7.7	--	
1 - Uniform (PSF)	0 to 6' 3"	9'	15.0	25.0	Default Load
2 - Point (lb)	3'	N/A	876	1359	Linked from: RH-4, Support 1

Weyerhaeuser Notes

ForteWEB Software Operator	Job Notes
Kenny Jones L120 Engineering (817) 727-2136 kjones@l120engineering.com	

Weyerhaeuser

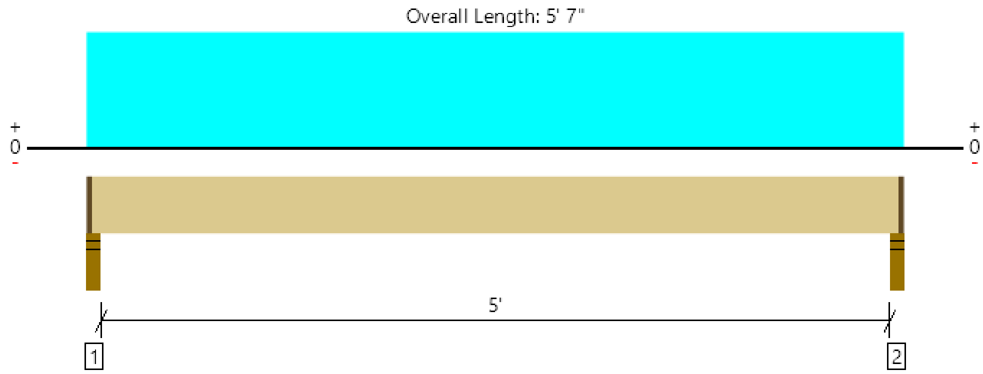
FORTE WEB

MEMBER REPORT

PASSED

Second Floor, SB-1

1 piece(s) 4 x 10 Douglas Fir-Larch No. 2



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)	1280 @ 2"	3189 (2.25")	Passed (40%)	--	1.0 D + 1.0 L (All Spans)
Shear (lbs)	823 @ 1' 3/4"	3885	Passed (21%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	1641 @ 2' 9 1/2"	4492	Passed (37%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.017 @ 2' 9 1/2"	0.131	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.022 @ 2' 9 1/2"	0.262	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)

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

- Deflection criteria: LL (L/480) and TL (L/240).
- Allowed moment does not reflect the adjustment for the beam stability factor.
- Applicable calculations are based on NDS.

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Floor Live	Total	
1 - Stud wall - HF	3.50"	2.25"	1.50"	324	1005	1329	1 1/4" Rim Board
2 - Stud wall - HF	3.50"	2.25"	1.50"	324	1005	1329	1 1/4" Rim Board

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

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	5' 5" o/c	
Bottom Edge (Lu)	5' 5" o/c	

•Maximum allowable bracing intervals based on applied load.

Vertical Loads	Location (Side)	Tributary Width	Dead (0.90)	Floor Live (1.00)	Comments
0 - Self Weight (PLF)	1 1/4" to 5' 5 3/4"	N/A	8.2	--	
1 - Uniform (PSF)	0 to 5' 7" (Front)	9'	12.0	40.0	Default Load

Weyerhaeuser Notes

ForteWEB Software Operator	Job Notes
Kenny Jones L120 Engineering (817) 727-2136 kjones@l120engineering.com	

Weyerhaeuser

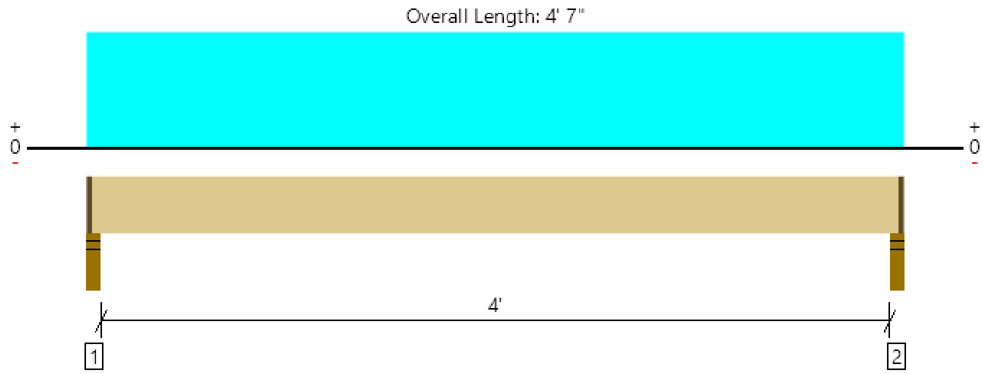
FORTE WEB

MEMBER REPORT

PASSED

Second Floor, SB-2

1 piece(s) 4 x 8 Douglas Fir-Larch No. 2



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)	1152 @ 2"	3189 (2.25")	Passed (36%)	--	1.0 D + 1.0 L (All Spans)
Shear (lbs)	735 @ 10 3/4"	3045	Passed (24%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	1189 @ 2' 3 1/2"	2989	Passed (40%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.017 @ 2' 3 1/2"	0.106	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.022 @ 2' 3 1/2"	0.213	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)

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

- Deflection criteria: LL (L/480) and TL (L/240).
- Allowed moment does not reflect the adjustment for the beam stability factor.
- Applicable calculations are based on NDS.

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Floor Live	Total	
1 - Stud wall - HF	3.50"	2.25"	1.50"	289	917	1206	1 1/4" Rim Board
2 - Stud wall - HF	3.50"	2.25"	1.50"	289	917	1206	1 1/4" Rim Board

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

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	4' 5" o/c	
Bottom Edge (Lu)	4' 5" o/c	

•Maximum allowable bracing intervals based on applied load.

Vertical Loads	Location (Side)	Tributary Width	Dead (0.90)	Floor Live (1.00)	Comments
0 - Self Weight (PLF)	1 1/4" to 4' 5 3/4"	N/A	6.4	--	
1 - Uniform (PSF)	0 to 4' 7" (Front)	10'	12.0	40.0	Default Load

Weyerhaeuser Notes

ForteWEB Software Operator	Job Notes
Kenny Jones L120 Engineering (817) 727-2136 kjones@l120engineering.com	

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FORTE WEB

MEMBER REPORT

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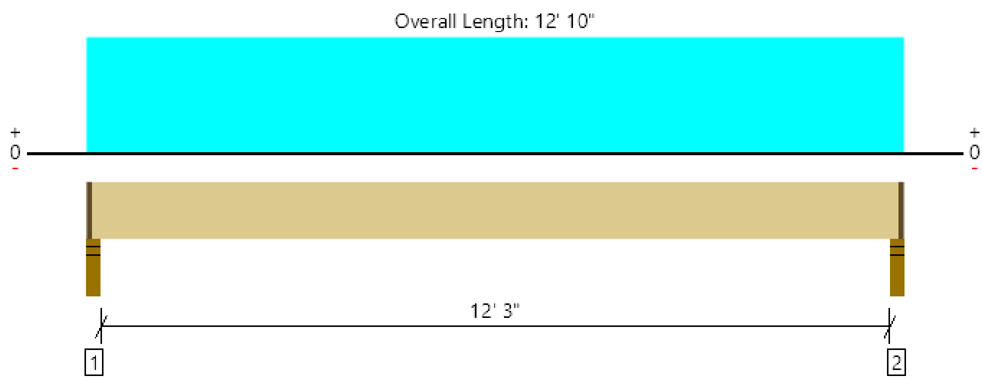
Second Floor, SJ-1

1 piece(s) 2 x 12 Hem-Fir No. 2 @ 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)	438 @ 2 1/2"	1367 (2.25")	Passed (32%)	--	1.0 D + 1.0 L (All Spans)
Shear (lbs)	360 @ 1' 2 3/4"	1688	Passed (21%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	1336 @ 6' 5"	2577	Passed (52%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.123 @ 6' 5"	0.310	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.160 @ 6' 5"	0.621	Passed (L/930)	--	1.0 D + 1.0 L (All Spans)
TJ-Pro™ Rating	N/A	N/A	N/A	--	N/A

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

- Deflection criteria: LL (L/480) and TL (L/240).
- Allowed moment does not reflect the adjustment for the beam stability factor.
- A 15% increase in the moment capacity has been added to account for repetitive member usage.
- Applicable calculations are based on NDS.
- No composite action between deck and joist was considered in analysis.

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Floor Live	Total	
1 - Stud wall - HF	3.50"	2.25"	1.50"	103	342	445	1 1/4" Rim Board
2 - Stud wall - HF	3.50"	2.25"	1.50"	103	342	445	1 1/4" Rim Board

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

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	9' 1" o/c	
Bottom Edge (Lu)	12' 8" o/c	

•Maximum allowable bracing intervals based on applied load.

Vertical Load	Location (Side)	Spacing	Dead (0.90)	Floor Live (1.00)	Comments
1 - Uniform (PSF)	0 to 12' 10"	16"	12.0	40.0	Default Load

Weyerhaeuser Notes

ForteWEB Software Operator	Job Notes
Kenny Jones L120 Engineering (817) 727-2136 kjones@l120engineering.com	

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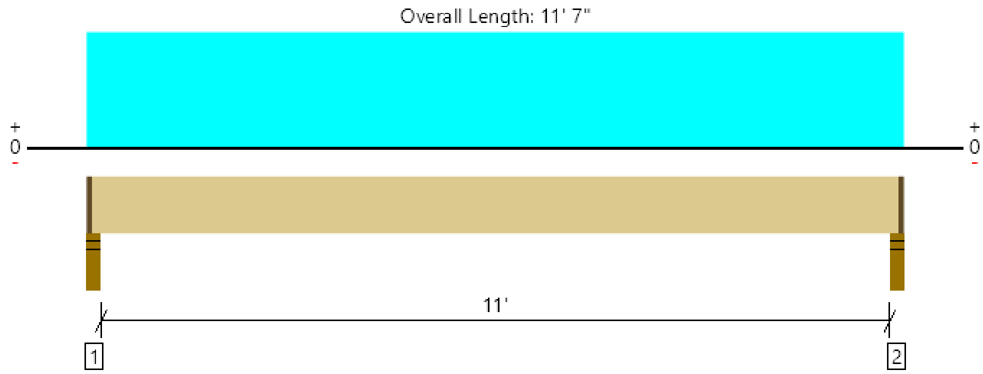
FORTE WEB

MEMBER REPORT

PASSED

Second Floor, DJ-1

1 piece(s) 2 x 10 Hem-Fir No. 2 @ 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)	546 @ 2 1/2"	1367 (2.25")	Passed (40%)	--	1.0 D + 1.0 L (All Spans)
Shear (lbs)	454 @ 1' 3/4"	1388	Passed (33%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	1496 @ 5' 9 1/2"	1917	Passed (78%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.218 @ 5' 9 1/2"	0.279	Passed (L/616)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.261 @ 5' 9 1/2"	0.558	Passed (L/513)	--	1.0 D + 1.0 L (All Spans)
TJ-Pro™ Rating	N/A	N/A	N/A	--	N/A

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

- Deflection criteria: LL (L/480) and TL (L/240).
- Allowed moment does not reflect the adjustment for the beam stability factor.
- A 15% increase in the moment capacity has been added to account for repetitive member usage.
- Applicable calculations are based on NDS.
- No composite action between deck and joist was considered in analysis.

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Floor Live	Total	
1 - Stud wall - HF	3.50"	2.25"	1.50"	93	463	556	1 1/4" Rim Board
2 - Stud wall - HF	3.50"	2.25"	1.50"	93	463	556	1 1/4" Rim Board

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

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	5' 8" o/c	
Bottom Edge (Lu)	11' 5" o/c	

•Maximum allowable bracing intervals based on applied load.

Vertical Load	Location (Side)	Spacing	Dead (0.90)	Floor Live (1.00)	Comments
1 - Uniform (PSF)	0 to 11' 7"	16"	12.0	60.0	deck

Weyerhaeuser Notes

ForteWEB Software Operator	Job Notes
Kenny Jones L120 Engineering (817) 727-2136 kjones@l120engineering.com	

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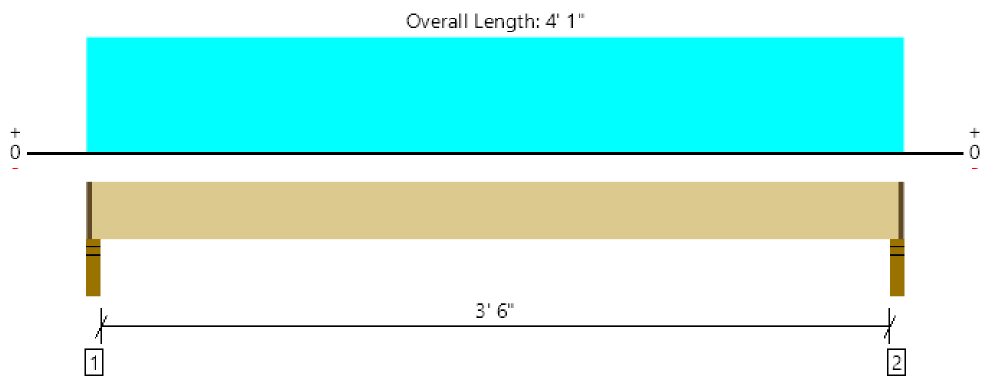
MEMBER REPORT

PASSED

Second Floor, DB-1

2 piece(s) 2 x 10 Hem-Fir No. 2



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)	618 @ 2"	2734 (2.25")	Passed (23%)	--	1.0 D + 1.0 L (All Spans)
Shear (lbs)	312 @ 1' 3/4"	2775	Passed (11%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	561 @ 2' 1/2"	3333	Passed (17%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.004 @ 2' 1/2"	0.094	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.006 @ 2' 1/2"	0.188	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)

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

- Deflection criteria: LL (L/480) and TL (L/240).
- Allowed moment does not reflect the adjustment for the beam stability factor.
- Applicable calculations are based on NDS.

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Floor Live	Total	
1 - Stud wall - HF	3.50"	2.25"	1.50"	161	490	651	1 1/4" Rim Board
2 - Stud wall - HF	3.50"	2.25"	1.50"	161	490	651	1 1/4" Rim Board

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

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	3' 11" o/c	
Bottom Edge (Lu)	3' 11" o/c	

•Maximum allowable bracing intervals based on applied load.

Vertical Loads	Location (Side)	Tributary Width	Dead (0.90)	Floor Live (1.00)	Comments
0 - Self Weight (PLF)	1 1/4" to 3' 11 3/4"	N/A	7.0	--	
1 - Uniform (PSF)	0 to 4' 1" (Front)	6'	12.0	40.0	Default Load

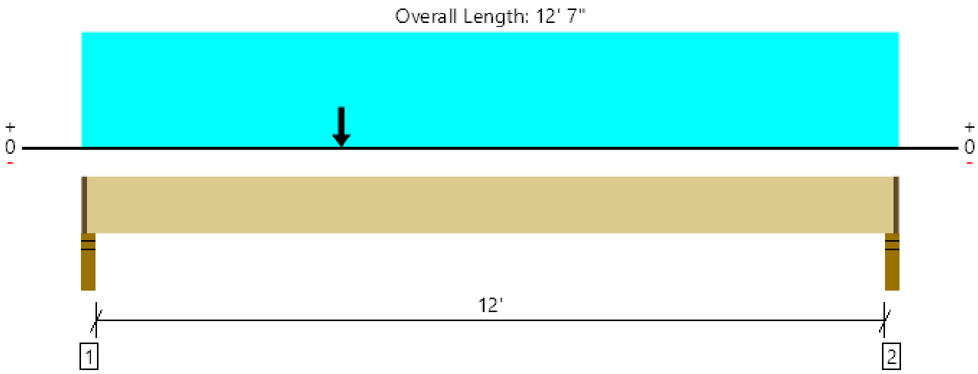
Weyerhaeuser Notes

ForteWEB Software Operator	Job Notes
Kenny Jones L120 Engineering (817) 727-2136 kjones@l120engineering.com	

MEMBER REPORT

PASSED

Second Floor, DB-2
3 piece(s) 2 x 10 Hem-Fir No. 2



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)	1478 @ 2"	4101 (2.25")	Passed (36%)	--	1.0 D + 1.0 L (All Spans)
Shear (lbs)	1318 @ 1' 3/4"	4163	Passed (32%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	4496 @ 5' 13/16"	5000	Passed (90%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.226 @ 6' 1 1/2"	0.306	Passed (L/649)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.310 @ 6' 1 9/16"	0.613	Passed (L/474)	--	1.0 D + 1.0 L (All Spans)

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

- Deflection criteria: LL (L/480) and TL (L/240).
- Allowed moment does not reflect the adjustment for the beam stability factor.
- Applicable calculations are based on NDS.

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Floor Live	Total	
1 - Stud wall - HF	3.50"	2.25"	1.50"	402	1092	1494	1 1/4" Rim Board
2 - Stud wall - HF	3.50"	2.25"	1.50"	342	908	1250	1 1/4" Rim Board

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

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	10' 7" o/c	
Bottom Edge (Lu)	12' 5" o/c	

•Maximum allowable bracing intervals based on applied load.

Vertical Loads	Location (Side)	Tributary Width	Dead (0.90)	Floor Live (1.00)	Comments
0 - Self Weight (PLF)	1 1/4" to 12' 5 3/4"	N/A	10.6	--	
1 - Uniform (PSF)	0 to 12' 7" (Front)	3'	12.0	40.0	Default Load
2 - Point (lb)	4' (Front)	N/A	161	490	Linked from: DB-1, Support 1

Weyerhaeuser Notes

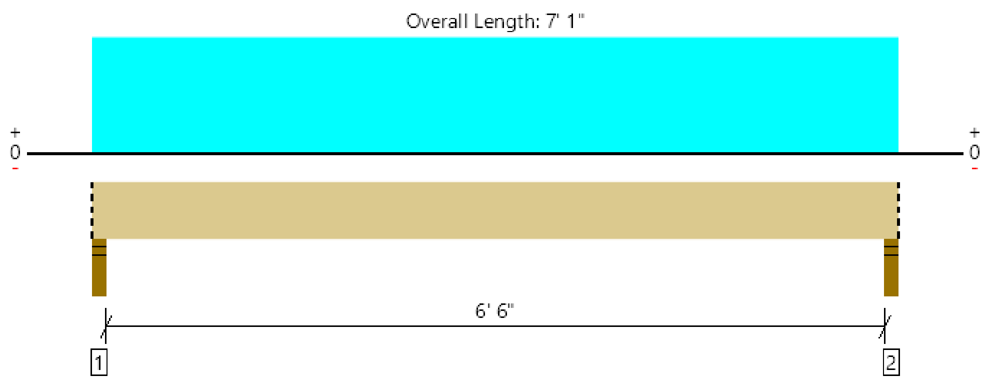
ForteWEB Software Operator	Job Notes
Kenny Jones L120 Engineering (817) 727-2136 kjones@l120engineering.com	

MEMBER REPORT

PASSED

Second Floor, DH-2

1 piece(s) 6 x 10 Douglas Fir-Larch No. 2



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)	1832 @ 2"	8181 (3.50")	Passed (22%)	--	1.0 D + 1.0 L (All Spans)
Shear (lbs)	1272 @ 1' 1"	5922	Passed (21%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	2946 @ 3' 6 1/2"	6032	Passed (49%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.038 @ 3' 6 1/2"	0.225	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.047 @ 3' 6 1/2"	0.338	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)

- Deflection criteria: LL (L/360) and TL (L/240).
- Allowed moment does not reflect the adjustment for the beam stability factor.
- Applicable calculations are based on NDS.

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

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Floor Live	Total	
1 - Stud wall - SPF	3.50"	3.50"	1.50"	344	1488	1832	Blocking
2 - Stud wall - SPF	3.50"	3.50"	1.50"	344	1488	1832	Blocking

• Blocking Panels are assumed to carry no loads applied directly above them and the full load is applied to the member being designed.

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	7' 1" o/c	
Bottom Edge (Lu)	7' 1" o/c	

•Maximum allowable bracing intervals based on applied load.

Vertical Loads	Location (Side)	Tributary Width	Dead (0.90)	Floor Live (1.00)	Comments
0 - Self Weight (PLF)	0 to 7' 1"	N/A	13.2	--	
1 - Uniform (PSF)	0 to 7' 1" (Front)	7'	12.0	60.0	DECK

Weyerhaeuser Notes

ForteWEB Software Operator	Job Notes
Kenny Jones L120 Engineering (817) 727-2136 kjones@l120engineering.com	

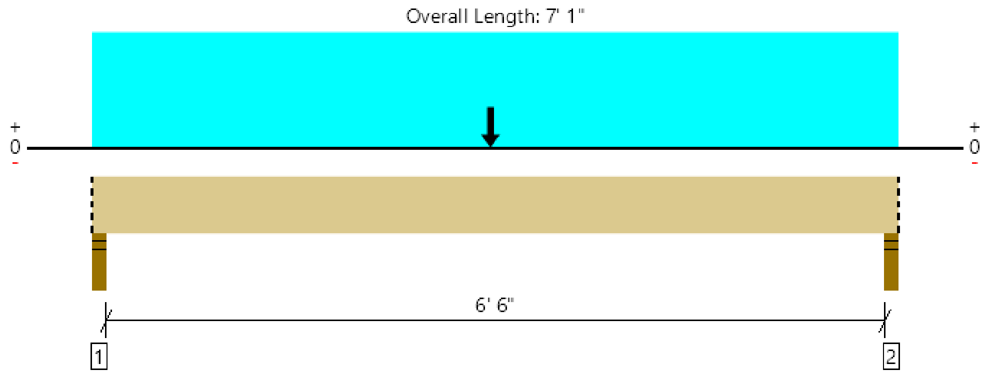
Weyerhaeuser

MEMBER REPORT

PASSED

Second Floor, DH-3

1 piece(s) 6 x 10 Douglas Fir-Larch No. 2



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)	2419 @ 2"	8181 (3.50")	Passed (30%)	--	1.0 D + 0.75 L + 0.75 S (All Spans)
Shear (lbs)	1973 @ 1' 1"	6810	Passed (29%)	1.15	1.0 D + 0.75 L + 0.75 S (All Spans)
Moment (Ft-lbs)	5545 @ 3' 6"	6937	Passed (80%)	1.15	1.0 D + 0.75 L + 0.75 S (All Spans)
Live Load Defl. (in)	0.051 @ 3' 6 7/16"	0.225	Passed (L/999+)	--	1.0 D + 0.75 L + 0.75 S (All Spans)
Total Load Defl. (in)	0.079 @ 3' 6 7/16"	0.338	Passed (L/999+)	--	1.0 D + 0.75 L + 0.75 S (All Spans)

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

- Deflection criteria: LL (L/360) and TL (L/240).
- Allowed moment does not reflect the adjustment for the beam stability factor.
- Applicable calculations are based on NDS.

Supports	Bearing Length			Loads to Supports (lbs)				Accessories
	Total	Available	Required	Dead	Floor Live	Snow	Total	
1 - Stud wall - SPF	3.50"	3.50"	1.50"	788	1488	688	2964	Blocking
2 - Stud wall - SPF	3.50"	3.50"	1.50"	777	1488	671	2936	Blocking

• Blocking Panels are assumed to carry no loads applied directly above them and the full load is applied to the member being designed.

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	7' 1" o/c	
Bottom Edge (Lu)	7' 1" o/c	

•Maximum allowable bracing intervals based on applied load.

Vertical Loads	Location (Side)	Tributary Width	Dead (0.90)	Floor Live (1.00)	Snow (1.15)	Comments
0 - Self Weight (PLF)	0 to 7' 1"	N/A	13.2	--	--	
1 - Uniform (PSF)	0 to 7' 1" (Front)	7'	12.0	60.0	-	DECK
2 - Point (lb)	3' 6" (Front)	N/A	876	-	1359	Linked from: RH-4, Support 1

Weyerhaeuser Notes

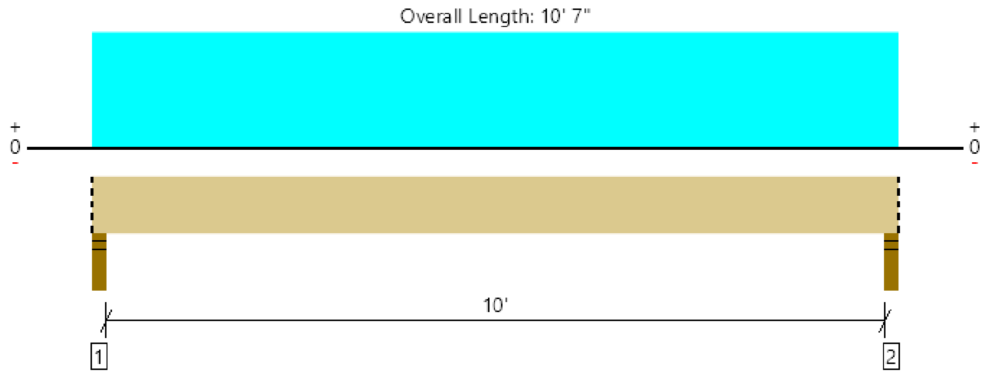
ForteWEB Software Operator	Job Notes
Kenny Jones L120 Engineering (817) 727-2136 kjones@l120engineering.com	

MEMBER REPORT

PASSED

Second Floor, DH-4

1 piece(s) 5 1/2" x 9 1/2" 24F-V4 DF Glulam



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)	2734 @ 2"	8181 (3.50")	Passed (33%)	--	1.0 D + 1.0 L (All Spans)
Shear (lbs)	2174 @ 1' 1"	9231	Passed (24%)	1.00	1.0 D + 1.0 L (All Spans)
Pos Moment (Ft-lbs)	6786 @ 5' 3 1/2"	16546	Passed (41%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.147 @ 5' 3 1/2"	0.342	Passed (L/834)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.181 @ 5' 3 1/2"	0.512	Passed (L/678)	--	1.0 D + 1.0 L (All Spans)

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

- Deflection criteria: LL (L/360) and TL (L/240).
- Allowed moment does not reflect the adjustment for the beam stability factor.
- Critical positive moment adjusted by a volume factor of 1.00 that was calculated using length L = 10' 3".
- The effects of positive or negative camber have not been accounted for when calculating deflection.
- The specified glulam is assumed to have its strong laminations at the bottom of the beam. Install with proper side up as indicated by the manufacturer.
- Applicable calculations are based on NDS.

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Floor Live	Total	
1 - Stud wall - SPF	3.50"	3.50"	1.50"	512	2223	2735	Blocking
2 - Stud wall - SPF	3.50"	3.50"	1.50"	512	2223	2735	Blocking

- Blocking Panels are assumed to carry no loads applied directly above them and the full load is applied to the member being designed.

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	10' 7" o/c	
Bottom Edge (Lu)	10' 7" o/c	

•Maximum allowable bracing intervals based on applied load.

Vertical Loads	Location (Side)	Tributary Width	Dead (0.90)	Floor Live (1.00)	Comments
0 - Self Weight (PLF)	0 to 10' 7"	N/A	12.7	--	
1 - Uniform (PSF)	0 to 10' 7" (Front)	7'	12.0	60.0	DECK

Weyerhaeuser Notes

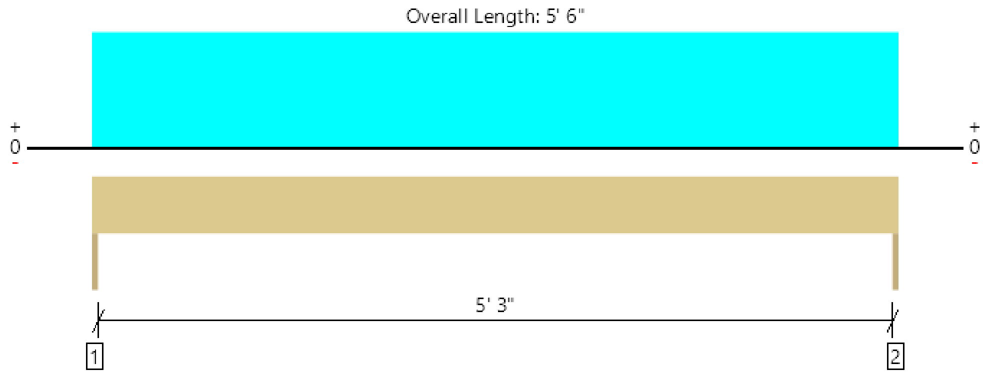
ForteWEB Software Operator	Job Notes
Kenny Jones L120 Engineering (817) 727-2136 kjones@l120engineering.com	

MEMBER REPORT

PASSED

Second Floor, SH-1

1 piece(s) 4 x 8 Douglas Fir-Larch No. 2



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)	1288 @ 0	3281 (1.50")	Passed (39%)	--	1.0 D + 1.0 L (All Spans)
Shear (lbs)	947 @ 8 3/4"	3045	Passed (31%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	1771 @ 2' 9"	2989	Passed (59%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.028 @ 2' 9"	0.183	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.054 @ 2' 9"	0.275	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)

System : Wall
Member Type : Header
Building Use : Residential
Building Code : IBC 2015
Design Methodology : ASD

- Deflection criteria: LL (L/360) and TL (L/240).
- Allowed moment does not reflect the adjustment for the beam stability factor.
- Applicable calculations are based on NDS.

Supports	Bearing Length			Loads to Supports (lbs)				Accessories
	Total	Available	Required	Dead	Floor Live	Snow	Total	
1 - Trimmer - HF	1.50"	1.50"	1.50"	628	660	138	1426	None
2 - Trimmer - HF	1.50"	1.50"	1.50"	628	660	138	1426	None

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	5' 6" o/c	
Bottom Edge (Lu)	5' 6" o/c	

•Maximum allowable bracing intervals based on applied load.

Vertical Loads	Location (Side)	Tributary Width	Dead (0.90)	Floor Live (1.00)	Snow (1.15)	Comments
0 - Self Weight (PLF)	0 to 5' 6"	N/A	6.4	--	--	
1 - Uniform (PSF)	0 to 5' 6"	6'	12.0	40.0	-	Default Load
2 - Uniform (PSF)	0 to 5' 6"	2'	15.0	-	25.0	Default Load
3 - Uniform (PLF)	0 to 5' 6"	N/A	120.0	-	-	

Weyerhaeuser Notes

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

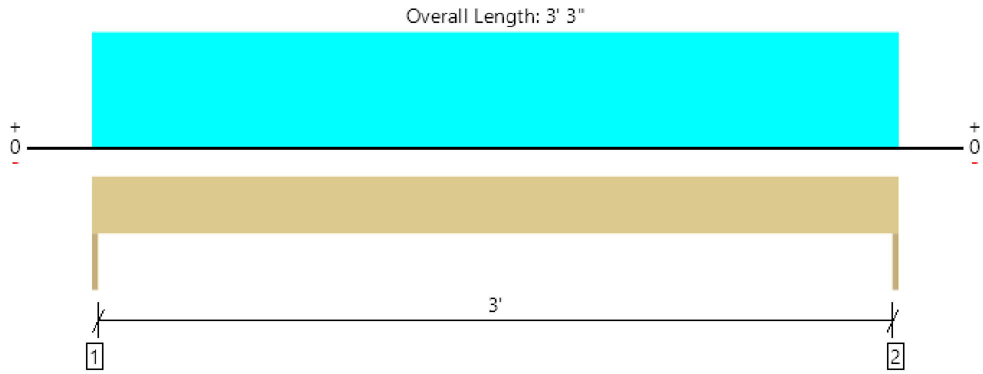
ForteWEB Software Operator	Job Notes
Kenny Jones L120 Engineering (817) 727-2136 kjones@l120engineering.com	

Weyerhaeuser

PASSED

Second Floor, SH-2

1 piece(s) 4 x 6 Douglas Fir-Larch No. 2



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)	759 @ 0	3281 (1.50")	Passed (23%)	--	1.0 D + 1.0 L (All Spans)
Shear (lbs)	486 @ 7"	2310	Passed (21%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	616 @ 1' 7 1/2"	1720	Passed (36%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.008 @ 1' 7 1/2"	0.108	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.015 @ 1' 7 1/2"	0.162	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)

System : Wall
Member Type : Header
Building Use : Residential
Building Code : IBC 2015
Design Methodology : ASD

- Deflection criteria: LL (L/360) and TL (L/240).
- Allowed moment does not reflect the adjustment for the beam stability factor.
- Applicable calculations are based on NDS.

Supports	Bearing Length			Loads to Supports (lbs)				Accessories
	Total	Available	Required	Dead	Floor Live	Snow	Total	
1 - Trimmer - HF	1.50"	1.50"	1.50"	369	390	81	840	None
2 - Trimmer - HF	1.50"	1.50"	1.50"	369	390	81	840	None

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	3' 3" o/c	
Bottom Edge (Lu)	3' 3" o/c	

•Maximum allowable bracing intervals based on applied load.

Vertical Loads	Location (Side)	Tributary Width	Dead (0.90)	Floor Live (1.00)	Snow (1.15)	Comments
0 - Self Weight (PLF)	0 to 3' 3"	N/A	4.9	--	--	
1 - Uniform (PSF)	0 to 3' 3"	6'	12.0	40.0	-	Default Load
2 - Uniform (PSF)	0 to 3' 3"	2'	15.0	-	25.0	Default Load
3 - Uniform (PLF)	0 to 3' 3"	N/A	120.0	-	-	

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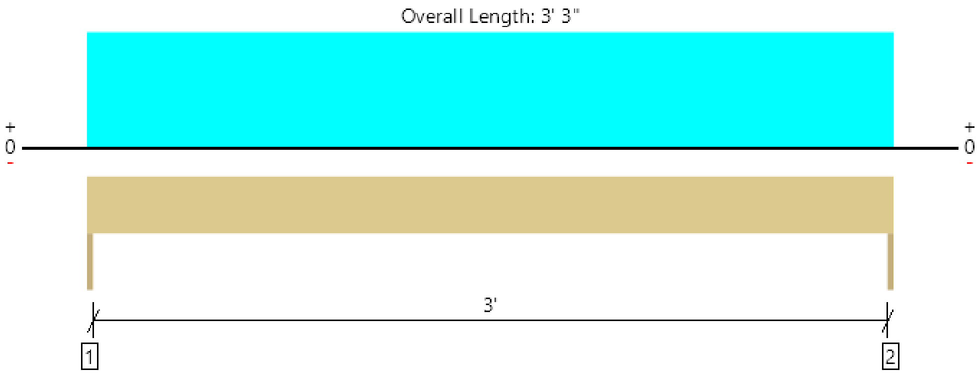
FORTE WEB

MEMBER REPORT

PASSED

Second Floor, SH-3

1 piece(s) 4 x 6 Douglas Fir-Larch No. 2



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)	1189 @ 0	3281 (1.50")	Passed (36%)	--	1.0 D + 0.75 L + 0.75 S (All Spans)
Shear (lbs)	762 @ 7"	2657	Passed (29%)	1.15	1.0 D + 0.75 L + 0.75 S (All Spans)
Moment (Ft-lbs)	966 @ 1' 7 1/2"	1979	Passed (49%)	1.15	1.0 D + 0.75 L + 0.75 S (All Spans)
Live Load Defl. (in)	0.012 @ 1' 7 1/2"	0.108	Passed (L/999+)	--	1.0 D + 0.75 L + 0.75 S (All Spans)
Total Load Defl. (in)	0.024 @ 1' 7 1/2"	0.162	Passed (L/999+)	--	1.0 D + 0.75 L + 0.75 S (All Spans)

System : Wall
Member Type : Header
Building Use : Residential
Building Code : IBC 2015
Design Methodology : ASD

- Deflection criteria: LL (L/360) and TL (L/240).
- Allowed moment does not reflect the adjustment for the beam stability factor.
- Applicable calculations are based on NDS.

Supports	Bearing Length			Loads to Supports (lbs)				Accessories
	Total	Available	Required	Dead	Floor Live	Snow	Total	
1 - Trimmer - HF	1.50"	1.50"	1.50"	598	260	528	1386	None
2 - Trimmer - HF	1.50"	1.50"	1.50"	598	260	528	1386	None

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	3' 3" o/c	
Bottom Edge (Lu)	3' 3" o/c	

•Maximum allowable bracing intervals based on applied load.

Vertical Loads	Location (Side)	Tributary Width	Dead (0.90)	Floor Live (1.00)	Snow (1.15)	Comments
0 - Self Weight (PLF)	0 to 3' 3"	N/A	4.9	--	--	
1 - Uniform (PSF)	0 to 3' 3"	4'	12.0	40.0	-	Default Load
2 - Uniform (PSF)	0 to 3' 3"	13'	15.0	-	25.0	Default Load
3 - Uniform (PLF)	0 to 3' 3"	N/A	120.0	-	-	

Weyerhaeuser Notes

ForteWEB Software Operator	Job Notes
Kenny Jones L120 Engineering (817) 727-2136 kjones@l120engineering.com	

Weyerhaeuser

9/2/2020 7:13:45 PM UTC

ForteWEB v3.0, Engine: V8.1.3.1, Data: V8.0.0.0

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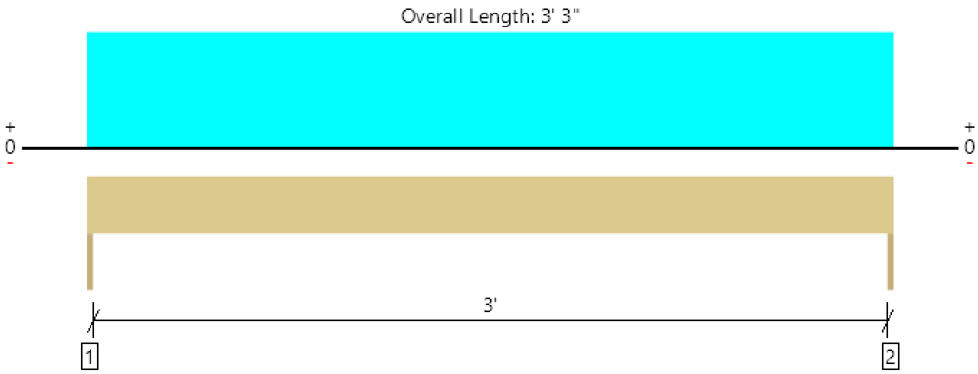
FORTE WEB

MEMBER REPORT

PASSED

Second Floor, SH-4

1 piece(s) 4 x 6 Douglas Fir-Larch No. 2



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)	1106 @ 0	3281 (1.50")	Passed (34%)	--	1.0 D + 1.0 L (All Spans)
Shear (lbs)	709 @ 7"	2310	Passed (31%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	899 @ 1' 7 1/2"	1720	Passed (52%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.017 @ 1' 7 1/2"	0.108	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.022 @ 1' 7 1/2"	0.162	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)

System : Wall
Member Type : Header
Building Use : Residential
Building Code : IBC 2015
Design Methodology : ASD

- Deflection criteria: LL (L/360) and TL (L/240).
- Allowed moment does not reflect the adjustment for the beam stability factor.
- Applicable calculations are based on NDS.

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Floor Live	Total	
1 - Trimmer - HF	1.50"	1.50"	1.50"	261	845	1106	None
2 - Trimmer - HF	1.50"	1.50"	1.50"	261	845	1106	None

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	3' 3" o/c	
Bottom Edge (Lu)	3' 3" o/c	

•Maximum allowable bracing intervals based on applied load.

Vertical Loads	Location (Side)	Tributary Width	Dead (0.90)	Floor Live (1.00)	Comments
0 - Self Weight (PLF)	0 to 3' 3"	N/A	4.9	--	
1 - Uniform (PSF)	0 to 3' 3"	13'	12.0	40.0	Default Load

Weyerhaeuser Notes

ForteWEB Software Operator	Job Notes
Kenny Jones L120 Engineering (817) 727-2136 kjones@l120engineering.com	

Weyerhaeuser

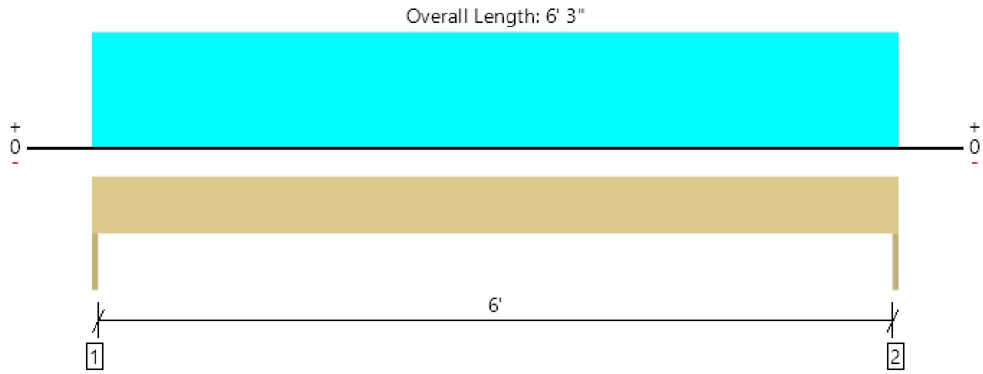
FORTE WEB

MEMBER REPORT

PASSED

Second Floor, SH-5

1 piece(s) 4 x 10 Douglas Fir-Larch No. 2



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)	2513 @ 0	3281 (1.50")	Passed (77%)	--	1.0 D + 1.0 L (All Spans)
Shear (lbs)	1793 @ 10 3/4"	3885	Passed (46%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	3927 @ 3' 1 1/2"	4492	Passed (87%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.059 @ 3' 1 1/2"	0.208	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.075 @ 3' 1 1/2"	0.313	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)

System : Wall
Member Type : Header
Building Use : Residential
Building Code : IBC 2015
Design Methodology : ASD

- Deflection criteria: LL (L/360) and TL (L/240).
- Allowed moment does not reflect the adjustment for the beam stability factor.
- Applicable calculations are based on NDS.

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Floor Live	Total	
1 - Trimmer - HF	1.50"	1.50"	1.50"	513	2000	2513	None
2 - Trimmer - HF	1.50"	1.50"	1.50"	513	2000	2513	None

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	6' 3" o/c	
Bottom Edge (Lu)	6' 3" o/c	

•Maximum allowable bracing intervals based on applied load.

Vertical Loads	Location (Side)	Tributary Width	Dead (0.90)	Floor Live (1.00)	Comments
0 - Self Weight (PLF)	0 to 6' 3"	N/A	8.2	--	
1 - Uniform (PSF)	0 to 6' 3"	7'	12.0	40.0	FLOOR
2 - Uniform (PSF)	0 to 6' 3"	6'	12.0	60.0	DECK

Weyerhaeuser Notes

ForteWEB Software Operator	Job Notes
Kenny Jones L120 Engineering (817) 727-2136 kjones@l120engineering.com	

Weyerhaeuser

LONGITUDE
ONE TWENTY°
ENGINEERING & DESIGN

FOUNDATION CALCULATIONS

FOOTING REFERENCE PER PLAN

Project: Metrostructure - Gravity

Location: 16" Cont FTG - Max
Footing
[2015 International Building Code(2015 NDS)]
Footing Size: 16.0 IN Wide x 8.0 IN Deep Continuous Footing With 8.0 IN Thick x 18.0 IN Tall Stemwall
LongitudinalReinforcement: (2) Continuous #4 Bars
TransverseReinforcement: #4 Bars @ 12.00 IN. O.C. (unnecessary)
Section Footing Design Adequate

StruCalc 9.0

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StruCalc Version 10.0.1.6 4/20/2020 7:37:31 PM

FOOTING PROPERTIES

Allowable Soil Bearing Pressure:	Qs =	1500 psf
Concrete Compressive Strength:	F'c =	2500 psi
Reinforcing Steel Yield Strength:	Fy =	40000 psi
Concrete Reinforcement Cover:	c =	3 in

FOOTING SIZE

Width:	W =	16 in
Depth:	Depth =	8 in
Effective Depth to Top Layer of Steel:	d =	4.25 in

STEMWALL SIZE

Stemwall Width:	8 in
Stemwall Height:	18 in
Stemwall Weight:	150 pcf

FOOTING CALCULATIONS

Bearing Calculations:

Ultimate Bearing Pressure:	Qu =	1388 psf
Effective Allowable Soil Bearing Pressure:	Qe =	1400 psf
Width Required:	Wreq =	1.32 ft

Beam Shear Calculations (One Way Shear):

Beam Shear:	Vu1 =	0 lb
Allowable Beam Shear:	Vc1 =	3825 lb

Transverse Direction:

Bending Calculations:

Factored Moment:	Mu =	1310 in-lb
Nominal Moment Strength:	Mn =	0 in-lb

Reinforcement Calculations:

Concrete Compressive Block Depth:	a =	0.30 in
Steel Required Based on Moment:	As(1) =	0.01 in2
Min. Code Req'd Reinf. Shrink./Temp. (ACI-10.5.4):	As(2) =	0.19 in2
Controlling Reinforcing Steel:	As-reqd =	0.19 in2
Selected Reinforcement:	Trans:	#4's @ 12.0 in. o.c.
Reinforcement Area Provided:	As =	0.19 in2

Development Length Calculations:

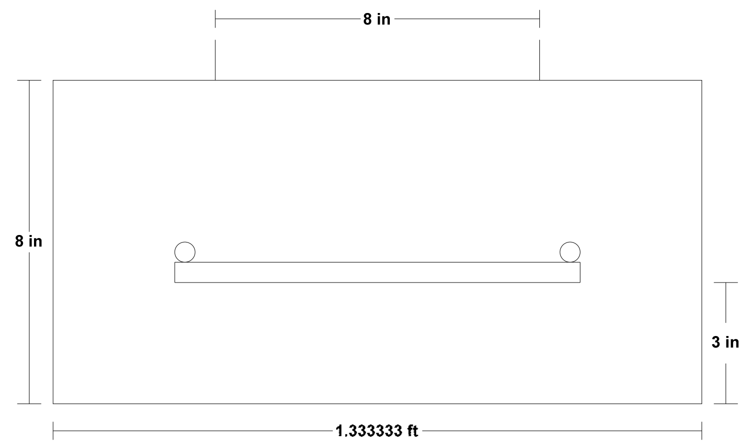
Development Length Required:	Ld =	15 in
Development Length Supplied:	Ld-sup =	1 in

Longitudinal Direction:

Reinforcement Calculations:

Min. Code Req'd Reinf. Shrink./Temp. (ACI-10.5.4):	As(2) =	0.26 in2
Controlling Reinforcing Steel:	As-reqd =	0.26 in2
Selected Reinforcement:	Longitudinal:	(2) Cont. #4 Bars
Reinforcement Area Provided:	As =	0.39 in2

LOADING DIAGRAM



FOOTING LOADING

Live Load:	PL =	1000 plf
Dead Load:	PD =	700 plf
Total Load:	PT =	1850 plf
Ultimate Factored Load:	Pu =	2620 plf

General Footing

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DESCRIPTIO 30x30x10

Code References

Calculations per ACI 318-14, IBC 2015, CBC 2016, ASCE 7-10

Load Combinations Used : ASCE 7-10

General Information

Material Properties

f'c : Concrete 28 day strength	=	3.0	ksi
fy : Rebar Yield	=	60.0	ksi
Ec : Concrete Elastic Modulus	=	3,155.92	ksi
Concrete Density	=	145.0	pcf
φ Values Flexure	=	0.90	
Shear	=	0.750	

Analysis Settings

Min Steel % Bending Reinf.	=		
Min Allow % Temp Reinf.	=	0.00180	
Min. Overturning Safety Factor	=	1.0	: 1
Min. Sliding Safety Factor	=	1.0	: 1
Add Ftg Wt for Soil Pressure	:	No	
Use ftg wt for stability, moments & shears	:	Yes	
Add Pedestal Wt for Soil Pressure	:	No	
Use Pedestal wt for stability, mom & shear	:	No	

Soil Design Values

Allowable Soil Beari	=	1.50	ksf
Increase Bearing By Footing Weight	=	No	
Soil Passive Resistance (for Sliding)	=	250.0	pcf
Soil/Concrete Friction Coeff.	=	0.30	

Increases based on footing Depth

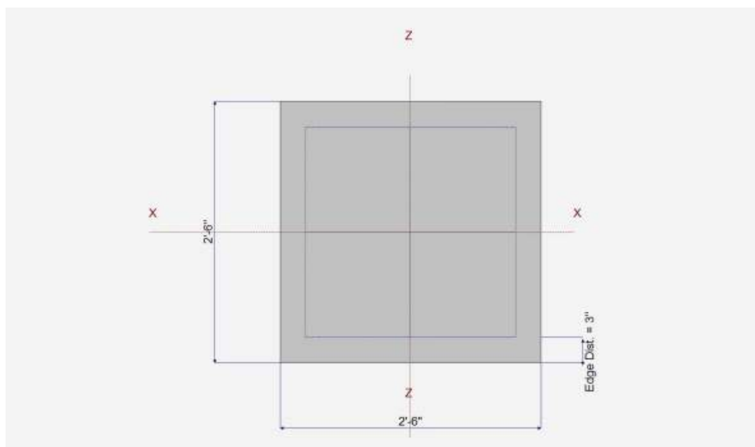
Footing base depth below soil surface	=	1.0	ft
Allow press. increase per foot of depth	=		ksf
when footing base is below	=		ft

Increases based on footing plan dimension

Allowable pressure increase per foot of depth	=		ksf
when max. length or width is greater than	=		ft

Dimensions

Width parallel to X-X Axis	=	2.50	ft
Length parallel to Z-Z Axis	=	2.50	ft
Footing Thickness	=	10.0	in



Pedestal dimensions...

px : parallel to X-X Axis	=		in
pz : parallel to Z-Z Axis	=		in
Height			in

Rebar Centerline to Edge of Concrete...

at Bottom of footing	=	3.0	in

Reinforcing

Bars parallel to X-X Axis

Number of Bars	=		3.0
Reinforcing Bar Size	=	#	4

Bars parallel to Z-Z Axis

Number of Bars	=		3.0
Reinforcing Bar Size	=	#	4

Bandwidth Distribution Check (ACI 15.4.4.2)

Direction Requiring Closer Separatio n/a

\# Bars required within zone n/a

\# Bars required on each side of zone n/a

Applied Loads

		D	Lr	L	S	W	E	H	
P : Column Load	=	5.0		4.20					k
OB : Overburden	=								ksf
M-xx	=								k-ft
M-zz	=								k-ft
V-x	=								k
V-z	=								k

General Footing

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DESCRIPTIO 30x30x10

DESIGN SUMMARY

Design OK

	Min. Ratio	Item	Applied	Capacity	Governing Load Combination
PASS	0.9953	Soil Bearing	1.493 ksf	1.50 ksf	+D+L+H about Z-Z axis
PASS	n/a	Overturning - X-X	0.0 k-ft	0.0 k-ft	No Overturning
PASS	n/a	Overturning - Z-Z	0.0 k-ft	0.0 k-ft	No Overturning
PASS	n/a	Sliding - X-X	0.0 k	0.0 k	No Sliding
PASS	n/a	Sliding - Z-Z	0.0 k	0.0 k	No Sliding
PASS	n/a	Uplift	0.0 k	0.0 k	No Uplift
PASS	0.2176	Z Flexure (+X)	1.590 k-ft/ft	7.306 k-ft/ft	+1.20D+0.50Lr+1.60L+1.60H
PASS	0.2176	Z Flexure (-X)	1.590 k-ft/ft	7.306 k-ft/ft	+1.20D+0.50Lr+1.60L+1.60H
PASS	0.2176	X Flexure (+Z)	1.590 k-ft/ft	7.306 k-ft/ft	+1.20D+0.50Lr+1.60L+1.60H
PASS	0.2176	X Flexure (-Z)	1.590 k-ft/ft	7.306 k-ft/ft	+1.20D+0.50Lr+1.60L+1.60H
PASS	0.1991	1-way Shear (+X)	16.354 psi	82.158 psi	+1.20D+0.50Lr+1.60L+1.60H
PASS	0.1991	1-way Shear (-X)	16.354 psi	82.158 psi	+1.20D+0.50Lr+1.60L+1.60H
PASS	0.1991	1-way Shear (+Z)	16.354 psi	82.158 psi	+1.20D+0.50Lr+1.60L+1.60H
PASS	0.1991	1-way Shear (-Z)	16.354 psi	82.158 psi	+1.20D+0.50Lr+1.60L+1.60H
PASS	0.3722	2-way Punching	61.160 psi	164.317 psi	+1.20D+0.50Lr+1.60L+1.60H

Detailed Results

Soil Bearing

Rotation Axis & Load Combination...	Gross Allowable	Xecc (in)	Zecc (in)	Actual Soil Bearing Stress @ Location Bottom, -Z	Top, +Z	Left, -X	Right, +X	Actual / Allow Ratio
X-X, +D+H	1.50	n/a	0.0	0.8208	0.8208	n/a	n/a	0.547
X-X, +D+L+H	1.50	n/a	0.0	1.493	1.493	n/a	n/a	0.995
X-X, +D+Lr+H	1.50	n/a	0.0	0.8208	0.8208	n/a	n/a	0.547
X-X, +D+S+H	1.50	n/a	0.0	0.8208	0.8208	n/a	n/a	0.547
X-X, +D+0.750Lr+0.750L+H	1.50	n/a	0.0	1.325	1.325	n/a	n/a	0.883
X-X, +D+0.750L+0.750S+H	1.50	n/a	0.0	1.325	1.325	n/a	n/a	0.883
X-X, +D+0.60W+H	1.50	n/a	0.0	0.8208	0.8208	n/a	n/a	0.547
X-X, +D+0.70E+H	1.50	n/a	0.0	0.8208	0.8208	n/a	n/a	0.547
X-X, +D+0.750Lr+0.750L+0.450W	1.50	n/a	0.0	1.325	1.325	n/a	n/a	0.883
X-X, +D+0.750L+0.750S+0.450W	1.50	n/a	0.0	1.325	1.325	n/a	n/a	0.883
X-X, +D+0.750L+0.750S+0.5250E	1.50	n/a	0.0	1.325	1.325	n/a	n/a	0.883
X-X, +0.60D+0.60W+0.60H	1.50	n/a	0.0	0.4925	0.4925	n/a	n/a	0.328
X-X, +0.60D+0.70E+0.60H	1.50	n/a	0.0	0.4925	0.4925	n/a	n/a	0.328
Z-Z, +D+H	1.50	0.0	n/a	n/a	n/a	0.8208	0.8208	0.547
Z-Z, +D+L+H	1.50	0.0	n/a	n/a	n/a	1.493	1.493	0.995
Z-Z, +D+Lr+H	1.50	0.0	n/a	n/a	n/a	0.8208	0.8208	0.547
Z-Z, +D+S+H	1.50	0.0	n/a	n/a	n/a	0.8208	0.8208	0.547
Z-Z, +D+0.750Lr+0.750L+H	1.50	0.0	n/a	n/a	n/a	1.325	1.325	0.883
Z-Z, +D+0.750L+0.750S+H	1.50	0.0	n/a	n/a	n/a	1.325	1.325	0.883
Z-Z, +D+0.60W+H	1.50	0.0	n/a	n/a	n/a	0.8208	0.8208	0.547
Z-Z, +D+0.70E+H	1.50	0.0	n/a	n/a	n/a	0.8208	0.8208	0.547
Z-Z, +D+0.750Lr+0.750L+0.450W	1.50	0.0	n/a	n/a	n/a	1.325	1.325	0.883
Z-Z, +D+0.750L+0.750S+0.450W	1.50	0.0	n/a	n/a	n/a	1.325	1.325	0.883
Z-Z, +D+0.750L+0.750S+0.5250E	1.50	0.0	n/a	n/a	n/a	1.325	1.325	0.883
Z-Z, +0.60D+0.60W+0.60H	1.50	0.0	n/a	n/a	n/a	0.4925	0.4925	0.328
Z-Z, +0.60D+0.70E+0.60H	1.50	0.0	n/a	n/a	n/a	0.4925	0.4925	0.328

Overturning Stability

Rotation Axis & Load Combination...	Overturning Moment	Resisting Moment	Stability Ratio	Status

Footing Has NO Overturning

Sliding Stability

All units k

Force Application Axis Load Combination...	Sliding Force	Resisting Force	Stability Ratio	Status

Footing Has NO Sliding

General Footing

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Licensee : L120 Engineering and Design, KW-06011993

DESCRIPTIO 30x30x10

Footing Flexure

Flexure Axis & Load Combination	Mu k-ft	Side	Tension Surface	As Req'd in^2	Gvrn. As in^2	Actual As in^2	Phi*Mn k-ft	Status
X-X, +1.40D+1.60H	0.8750	+Z	Bottom	0.2160	Min Temp %	0.240	7.306	OK
X-X, +1.40D+1.60H	0.8750	-Z	Bottom	0.2160	Min Temp %	0.240	7.306	OK
X-X, +1.20D+0.50Lr+1.60L+1.60H	1.590	+Z	Bottom	0.2160	Min Temp %	0.240	7.306	OK
X-X, +1.20D+0.50Lr+1.60L+1.60H	1.590	-Z	Bottom	0.2160	Min Temp %	0.240	7.306	OK
X-X, +1.20D+1.60L+0.50S+1.60H	1.590	+Z	Bottom	0.2160	Min Temp %	0.240	7.306	OK
X-X, +1.20D+1.60L+0.50S+1.60H	1.590	-Z	Bottom	0.2160	Min Temp %	0.240	7.306	OK
X-X, +1.20D+1.60Lr+0.50L+1.60H	1.013	+Z	Bottom	0.2160	Min Temp %	0.240	7.306	OK
X-X, +1.20D+1.60Lr+0.50L+1.60H	1.013	-Z	Bottom	0.2160	Min Temp %	0.240	7.306	OK
X-X, +1.20D+1.60Lr+0.50W+1.60	0.750	+Z	Bottom	0.2160	Min Temp %	0.240	7.306	OK
X-X, +1.20D+1.60Lr+0.50W+1.60	0.750	-Z	Bottom	0.2160	Min Temp %	0.240	7.306	OK
X-X, +1.20D+0.50L+1.60S+1.60H	1.013	+Z	Bottom	0.2160	Min Temp %	0.240	7.306	OK
X-X, +1.20D+0.50L+1.60S+1.60H	1.013	-Z	Bottom	0.2160	Min Temp %	0.240	7.306	OK
X-X, +1.20D+1.60S+0.50W+1.60I	0.750	+Z	Bottom	0.2160	Min Temp %	0.240	7.306	OK
X-X, +1.20D+1.60S+0.50W+1.60I	0.750	-Z	Bottom	0.2160	Min Temp %	0.240	7.306	OK
X-X, +1.20D+0.50Lr+0.50L+W+1.	1.013	+Z	Bottom	0.2160	Min Temp %	0.240	7.306	OK
X-X, +1.20D+0.50Lr+0.50L+W+1.	1.013	-Z	Bottom	0.2160	Min Temp %	0.240	7.306	OK
X-X, +1.20D+0.50L+0.50S+W+1.6	1.013	+Z	Bottom	0.2160	Min Temp %	0.240	7.306	OK
X-X, +1.20D+0.50L+0.50S+W+1.6	1.013	-Z	Bottom	0.2160	Min Temp %	0.240	7.306	OK
X-X, +1.20D+0.50L+0.20S+E+1.6	1.013	+Z	Bottom	0.2160	Min Temp %	0.240	7.306	OK
X-X, +1.20D+0.50L+0.20S+E+1.6	1.013	-Z	Bottom	0.2160	Min Temp %	0.240	7.306	OK
X-X, +0.90D+W+0.90H	0.5625	+Z	Bottom	0.2160	Min Temp %	0.240	7.306	OK
X-X, +0.90D+W+0.90H	0.5625	-Z	Bottom	0.2160	Min Temp %	0.240	7.306	OK
X-X, +0.90D+E+0.90H	0.5625	+Z	Bottom	0.2160	Min Temp %	0.240	7.306	OK
X-X, +0.90D+E+0.90H	0.5625	-Z	Bottom	0.2160	Min Temp %	0.240	7.306	OK
Z-Z, +1.40D+1.60H	0.8750	-X	Bottom	0.2160	Min Temp %	0.240	7.306	OK
Z-Z, +1.40D+1.60H	0.8750	+X	Bottom	0.2160	Min Temp %	0.240	7.306	OK
Z-Z, +1.20D+0.50Lr+1.60L+1.60H	1.590	-X	Bottom	0.2160	Min Temp %	0.240	7.306	OK
Z-Z, +1.20D+0.50Lr+1.60L+1.60H	1.590	+X	Bottom	0.2160	Min Temp %	0.240	7.306	OK
Z-Z, +1.20D+1.60L+0.50S+1.60H	1.590	-X	Bottom	0.2160	Min Temp %	0.240	7.306	OK
Z-Z, +1.20D+1.60L+0.50S+1.60H	1.590	+X	Bottom	0.2160	Min Temp %	0.240	7.306	OK
Z-Z, +1.20D+1.60Lr+0.50L+1.60H	1.013	-X	Bottom	0.2160	Min Temp %	0.240	7.306	OK
Z-Z, +1.20D+1.60Lr+0.50L+1.60H	1.013	+X	Bottom	0.2160	Min Temp %	0.240	7.306	OK
Z-Z, +1.20D+1.60Lr+0.50W+1.60	0.750	-X	Bottom	0.2160	Min Temp %	0.240	7.306	OK
Z-Z, +1.20D+1.60Lr+0.50W+1.60	0.750	+X	Bottom	0.2160	Min Temp %	0.240	7.306	OK
Z-Z, +1.20D+0.50L+1.60S+1.60H	1.013	-X	Bottom	0.2160	Min Temp %	0.240	7.306	OK
Z-Z, +1.20D+0.50L+1.60S+1.60H	1.013	+X	Bottom	0.2160	Min Temp %	0.240	7.306	OK
Z-Z, +1.20D+1.60S+0.50W+1.60I	0.750	-X	Bottom	0.2160	Min Temp %	0.240	7.306	OK
Z-Z, +1.20D+1.60S+0.50W+1.60I	0.750	+X	Bottom	0.2160	Min Temp %	0.240	7.306	OK
Z-Z, +1.20D+0.50Lr+0.50L+W+1.	1.013	-X	Bottom	0.2160	Min Temp %	0.240	7.306	OK
Z-Z, +1.20D+0.50Lr+0.50L+W+1.	1.013	+X	Bottom	0.2160	Min Temp %	0.240	7.306	OK
Z-Z, +1.20D+0.50L+0.50S+W+1.6	1.013	-X	Bottom	0.2160	Min Temp %	0.240	7.306	OK
Z-Z, +1.20D+0.50L+0.50S+W+1.6	1.013	+X	Bottom	0.2160	Min Temp %	0.240	7.306	OK
Z-Z, +1.20D+0.50L+0.20S+E+1.6	1.013	-X	Bottom	0.2160	Min Temp %	0.240	7.306	OK
Z-Z, +1.20D+0.50L+0.20S+E+1.6	1.013	+X	Bottom	0.2160	Min Temp %	0.240	7.306	OK
Z-Z, +0.90D+W+0.90H	0.5625	-X	Bottom	0.2160	Min Temp %	0.240	7.306	OK
Z-Z, +0.90D+W+0.90H	0.5625	+X	Bottom	0.2160	Min Temp %	0.240	7.306	OK
Z-Z, +0.90D+E+0.90H	0.5625	-X	Bottom	0.2160	Min Temp %	0.240	7.306	OK
Z-Z, +0.90D+E+0.90H	0.5625	+X	Bottom	0.2160	Min Temp %	0.240	7.306	OK

One Way Shear

Load Combination...	Vu @ -X	Vu @ +X	Vu @ -Z	Vu @ +Z	Vu:Max	Phi Vn	Vu / Phi*Vn	Status
+1.40D+1.60H	9.00 psi	9.00 psi	9.00 psi	9.00 psi	9.00 psi	82.16 psi	0.11	OK
+1.20D+0.50Lr+1.60L+1.60H	16.35 psi	16.35 psi	16.35 psi	16.35 psi	16.35 psi	82.16 psi	0.20	OK
+1.20D+1.60L+0.50S+1.60H	16.35 psi	16.35 psi	16.35 psi	16.35 psi	16.35 psi	82.16 psi	0.20	OK
+1.20D+1.60Lr+0.50L+1.60H	10.41 psi	10.41 psi	10.41 psi	10.41 psi	10.41 psi	82.16 psi	0.13	OK
+1.20D+1.60Lr+0.50W+1.60H	7.71 psi	7.71 psi	7.71 psi	7.71 psi	7.71 psi	82.16 psi	0.09	OK
+1.20D+0.50L+1.60S+1.60H	10.41 psi	10.41 psi	10.41 psi	10.41 psi	10.41 psi	82.16 psi	0.13	OK
+1.20D+1.60S+0.50W+1.60H	7.71 psi	7.71 psi	7.71 psi	7.71 psi	7.71 psi	82.16 psi	0.09	OK
+1.20D+0.50Lr+0.50L+W+1.60H	10.41 psi	10.41 psi	10.41 psi	10.41 psi	10.41 psi	82.16 psi	0.13	OK
+1.20D+0.50L+0.50S+W+1.60H	10.41 psi	10.41 psi	10.41 psi	10.41 psi	10.41 psi	82.16 psi	0.13	OK
+1.20D+0.50L+0.20S+E+1.60H	10.41 psi	10.41 psi	10.41 psi	10.41 psi	10.41 psi	82.16 psi	0.13	OK
+0.90D+W+0.90H	5.79 psi	5.79 psi	5.79 psi	5.79 psi	5.79 psi	82.16 psi	0.07	OK

General Footing

Lic. # : KW-06011993

Licensee : L120 Engineering and Design, KW-06011993

DESCRIPTIO 30x30x10

One Way Shear

Load Combination...	Vu @ -X	Vu @ +X	Vu @ -Z	Vu @ +Z	Vu:Max	Phi Vn	Vu / Phi*Vn	Status
+0.90D+E+0.90H	5.79 psi	5.79 psi	5.79 psi	5.79 psi	5.79 psi	82.16 psi	0.07	OK

Two-Way "Punching" Shear

All units k

Load Combination...	Vu	Phi*Vn	Vu / Phi*Vn	Status
+1.40D+1.60H	33.66 psi	164.32 psi	0.2048	OK
+1.20D+0.50Lr+1.60L+1.60H	61.16 psi	164.32 psi	0.3722	OK
+1.20D+1.60L+0.50S+1.60H	61.16 psi	164.32 psi	0.3722	OK
+1.20D+1.60Lr+0.50L+1.60H	38.95 psi	164.32 psi	0.237	OK
+1.20D+1.60Lr+0.50W+1.60H	28.85 psi	164.32 psi	0.1756	OK
+1.20D+0.50L+1.60S+1.60H	38.95 psi	164.32 psi	0.237	OK
+1.20D+1.60S+0.50W+1.60H	28.85 psi	164.32 psi	0.1756	OK
+1.20D+0.50Lr+0.50L+W+1.60H	38.95 psi	164.32 psi	0.237	OK
+1.20D+0.50L+0.50S+W+1.60H	38.95 psi	164.32 psi	0.237	OK
+1.20D+0.50L+0.20S+E+1.60H	38.95 psi	164.32 psi	0.237	OK
+0.90D+W+0.90H	21.64 psi	164.32 psi	0.1317	OK
+0.90D+E+0.90H	21.64 psi	164.32 psi	0.1317	OK

LONGITUDE
ONE TWENTY°
ENGINEERING & DESIGN

LATERAL CALCULATIONS

SHEAR-WALL REFERENCE PER PLAN

Hazards by Location

Search Information

Address: 8028 SE 36th St, Mercer Island, WA 98040, USA

Coordinates: 47.579157, -122.2310302

Elevation: 203 ft

Timestamp: 2020-09-01T23:18:04.765Z

Hazard Type: Wind

ASCE 7-16

MRI 10-Year	67 mph
MRI 25-Year	73 mph
MRI 50-Year	78 mph
MRI 100-Year	83 mph
Risk Category I	92 mph
Risk Category II	97 mph
Risk Category III	104 mph
Risk Category IV	108 mph

ASCE 7-10

MRI 10-Year	72 mph
MRI 25-Year	79 mph
MRI 50-Year	85 mph
MRI 100-Year	91 mph
Risk Category I	100 mph
Risk Category II	110 mph
Risk Category III-IV	115 mph

ASCE 7-05

ASCE 7-05 Wind Speed	85 mph

The results indicated here DO NOT reflect any state or local amendments to the values or any delineation lines made during the building code adoption process. Users should confirm any output obtained from this tool with the local Authority Having Jurisdiction before proceeding with design.

Disclaimer

Hazard loads are interpolated from data provided in ASCE 7 and rounded up to the nearest whole integer. Per ASCE 7, islands and coastal areas outside the last contour should use the last wind speed contour of the coastal area – in some cases, this website will extrapolate past the last wind speed contour and therefore, provide a wind speed that is slightly higher. NOTE: For queries near wind-borne debris region boundaries, the resulting determination is sensitive to rounding which may affect whether or not it is considered to be within a wind-borne debris region.

Mountainous terrain, gorges, ocean promontories, and special wind regions shall be examined for unusual wind conditions.

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ATC Hazards by Location

Search Information

Address:	8028 SE 36th St, Mercer Island, WA 98040, USA
Coordinates:	47.579157, -122.2310302
Elevation:	203 ft
Timestamp:	2020-09-01T23:18:28.127Z
Hazard Type:	Seismic
Reference Document:	ASCE7-16
Risk Category:	II
Site Class:	D

Basic Parameters

Name	Value	Description
S_S	1.406	MCE_R ground motion (period=0.2s)
S_1	0.489	MCE_R ground motion (period=1.0s)
S_{MS}	1.406	Site-modified spectral acceleration value
S_{M1}	* null	Site-modified spectral acceleration value
S_{DS}	0.937	Numeric seismic design value at 0.2s SA
S_{D1}	* null	Numeric seismic design value at 1.0s SA

* See Section 11.4.8

Additional Information

Name	Value	Description
SDC	* null	Seismic design category
F_a	1	Site amplification factor at 0.2s
F_v	* null	Site amplification factor at 1.0s
CR_S	0.902	Coefficient of risk (0.2s)
CR_1	0.897	Coefficient of risk (1.0s)
PGA	0.602	MCE_G peak ground acceleration
F_{PGA}	1.1	Site amplification factor at PGA
PGA_M	0.662	Site modified peak ground acceleration

T_L	6	Long-period transition period (s)
SsRT	1.406	Probabilistic risk-targeted ground motion (0.2s)
SsUH	1.558	Factored uniform-hazard spectral acceleration (2% probability of exceedance in 50 years)
SsD	3.454	Factored deterministic acceleration value (0.2s)
S1RT	0.489	Probabilistic risk-targeted ground motion (1.0s)
S1UH	0.546	Factored uniform-hazard spectral acceleration (2% probability of exceedance in 50 years)
S1D	1.393	Factored deterministic acceleration value (1.0s)
PGAd	1.184	Factored deterministic acceleration value (PGA)

* See Section 11.4.8

The results indicated here DO NOT reflect any state or local amendments to the values or any delineation lines made during the building code adoption process. Users should confirm any output obtained from this tool with the local Authority Having Jurisdiction before proceeding with design.

Disclaimer

Hazard loads are provided by the U.S. Geological Survey Seismic Design Web Services.

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ATC Hazards by Location

Search Information

Address: 8028 SE 36th St, Mercer Island, WA 98040, USA

Coordinates: 47.579157, -122.2310302

Elevation: 203 ft

Timestamp: 2020-09-01T23:18:50.888Z

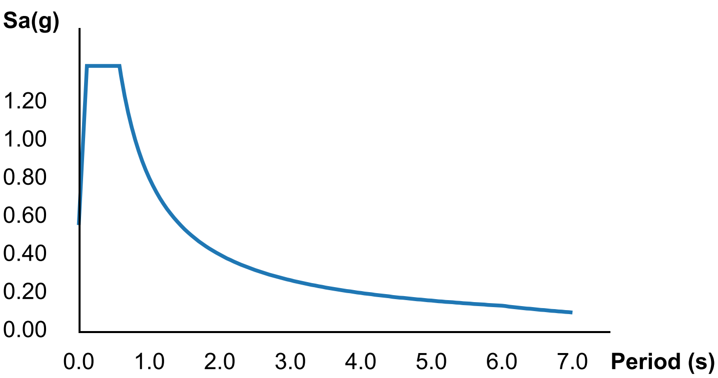
Hazard Type: Seismic

Reference Document: ASCE7-10

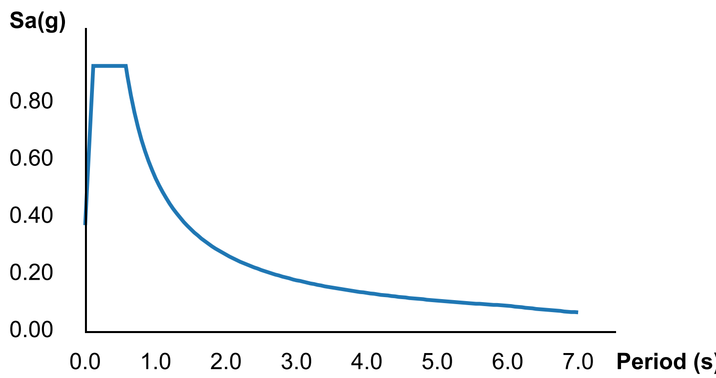
Risk Category: II

Site Class: D

MCER Horizontal Response Spectrum



Design Horizontal Response Spectrum



Basic Parameters

Name	Value	Description
S_S	1.392	MCE_R ground motion (period=0.2s)
S_1	0.535	MCE_R ground motion (period=1.0s)
S_{MS}	1.392	Site-modified spectral acceleration value
S_{M1}	0.803	Site-modified spectral acceleration value
S_{DS}	0.928	Numeric seismic design value at 0.2s SA
S_{D1}	0.535	Numeric seismic design value at 1.0s SA

Additional Information

Name	Value	Description
SDC	D	Seismic design category
F_a	1	Site amplification factor at 0.2s
F_v	1.5	Site amplification factor at 1.0s

CR_S	0.959	Coefficient of risk (0.2s)
CR_1	0.934	Coefficient of risk (1.0s)
PGA	0.574	MCE_G peak ground acceleration
F_{PGA}	1	Site amplification factor at PGA
PGA_M	0.574	Site modified peak ground acceleration
T_L	6	Long-period transition period (s)
SsRT	1.392	Probabilistic risk-targeted ground motion (0.2s)
SsUH	1.451	Factored uniform-hazard spectral acceleration (2% probability of exceedance in 50 years)
SsD	2.894	Factored deterministic acceleration value (0.2s)
S1RT	0.535	Probabilistic risk-targeted ground motion (1.0s)
S1UH	0.573	Factored uniform-hazard spectral acceleration (2% probability of exceedance in 50 years)
S1D	1.202	Factored deterministic acceleration value (1.0s)
PGAd	1.113	Factored deterministic acceleration value (PGA)

The results indicated here DO NOT reflect any state or local amendments to the values or any delineation lines made during the building code adoption process. Users should confirm any output obtained from this tool with the local Authority Having Jurisdiction before proceeding with design.

Disclaimer

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Project Number:	Plan:	Sheet Number:
S200831-6	Qui Residence Remodel	L1
Engineer:	Specifics:	Date
xxx	WIND FORCES	9/2/2020

IBC 2015 Section 1609 → ASCE 7-10 Section 28.6 - Simplified Procedure → Main Wind-Force Resisting System

WIND DESIGN CRITERIA:

Basic Wind Speed, V_s =	110 mph	(ASCE 7-10, Section 26.5 page 246)
Exposure =	B	(ASCE 7-10, Section 26.7 page 246)

BUILDING DIMENSIONS:

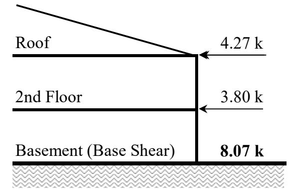
Roof Slope =	6.00 :12	= 26.57 degrees	
Loads From Front/Back - Width (ft)=	23.00 ft	Roof: Gable	
Loads From Side - Width (ft) =	24.00 ft	Roof: Hip	
Average Eave Height =	16.00 ft		
Mean Roof Ht. , h =	19.00 ft		(ASCE 7-10, Figure 27.6-2 page 275)
Edge Strip Width, a =	3 ft		(ASCE 7-10, Figure 28.6-1 page 303)
End Zone Width, 2a =	6.00 ft		(ASCE 7-10, Figure 28.6-1 page 303)

TOPOGRAPHIC DESIGN CONSIDERATIONS:

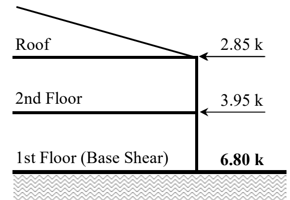
Topographic Factor , Kzt =	**1.40**	(ASCE 7-10, Section 26.8, page 251)
Adjustment Factor, λ =	1.00	(ASCE 7-10, Figure 28.6-1, page 305)

WIND LOAD SUMMARY:

Front / Back Direction



Side / Side Direction

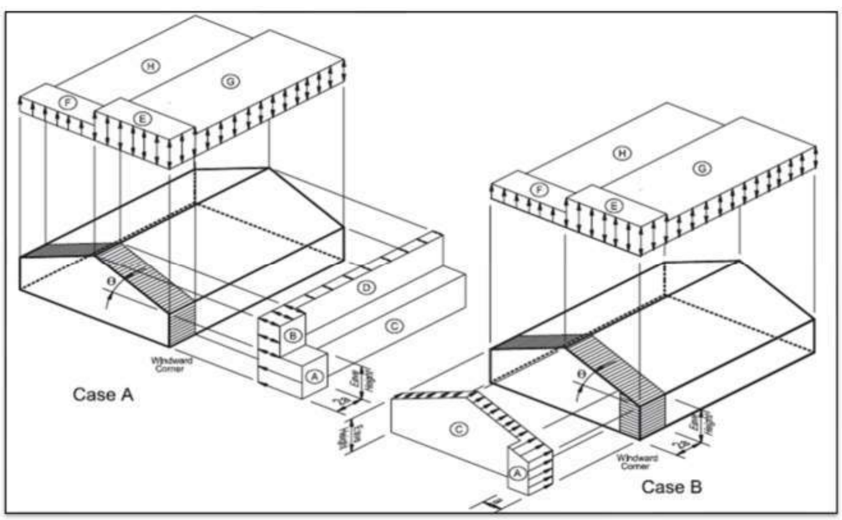


SIMPLIFIED DESIGN WIND PRESSURE, P_{S30} (psf)

(Exposure B at h = 30ft.)

Basic Wind Speed, Vs (mph)	Roof Angle (Degrees)	Load Case	ZONES*									
			Horizontal Pressure				Vertical Presssure				Overhang	
			A	B	C	D	E	F	G	H	E_{OH}	G_{OH}
110	26.57	A	23.32	7.31	17.34	6.44	-6.82	-14.13	-5.10	-11.57	-16.05	-14.40

* *Values Interpolated from Figure 28.6-1 ASCE 7 - 10 p. 303 to 305*



Project Number:	Plan:	Sheet Number:
S200831-6	**Qui Residence Remodel**	**L1**
Engineer: **xxx**	Specifics: **WIND FORCES**	Date **9/2/2020**

IBC 2015 Section 1609 → ASCE 7-10 Section 28.6 - Simplified Procedure → Main Wind-Force Resisting System

HORIZONTAL LOADS (psf) $p_s = \lambda * Kzt*Ps30$				MIN. LOADS (psf) Per ASCE 7-10, 28.6.3	
End zone		Interior zone		Roof	Wall
A (Wall)	B (Roof)	C (Wall)	D (Roof)		
32.64	10.24	24.27	9.02	8.0	16.0

ASD WIND FORCES: FRONT / BACK LOADING DIRECTION

	Location	Width (ft)	Height (ft)	Plane	End Zone Length (ft)	End Zone Pressure (W) (psf)	Interior zone Length (ft)	Interior zone Pressure (W) (psf)	Force 0.6 ω*W (kips)	Min Force 0.6 ω*W (kips)
ROOF	Height" of Roof to Plate (see note)	23.0	5.00	(roof)	6.0	32.64	17.0	24.27	2.37	0.72
	Plate to Mid 2nd LVL	23.0	4.00	(wall)	6.0	32.64	17.0	24.27	1.90	1.15
								Σ =	4.27	1.87
2nd FLOOR	Mid 2nd LVL to Floor	23.0	4.00	(wall)	6.0	32.64	17.0	24.27	1.90	1.15
	ight" Low-Roof to Plate (see note)	0.0	0.00	(roof)	6.0	32.64	-6.0	24.27	0.00	0.00
	Floor to Mid 1st LVL	23.0	4.00	(wall)	6.0	32.64	17.0	24.27	1.90	1.15
								Σ =	3.80	2.30
								Total Wind Base Shear (kips)	8.07	4.16

ASD WIND FORCES: SIDE / SIDE LOADING DIRECTION

	Location	Width (ft)	Height (ft)	Plane	End Zone Length (ft)	End Zone Pressure (W) (psf)	Interior zone Length (ft)	Interior zone Pressure (W) (psf)	Force 0.6 ω*W kips	Min Force 0.6 ω*W kips
ROOF	Height" of Roof to Plate (see note)	24.0	5.00	(roof)	6.0	10.24	18.0	9.02	0.87	0.75
	Plate to Mid 2nd LVL	24.0	4.00	(wall)	6.0	32.64	18.0	24.27	1.97	1.20
								Σ =	2.85	1.95
2nd FLOOR	Mid 2nd LVL to Floor	24.0	4.00	(wall)	6.0	32.64	18.0	24.27	1.97	1.20
	ight" Low-Roof to Plate (see note)	0.0	0.00	(roof)	6.0	10.24	-6.0	9.02	0.00	0.00
	Floor to Mid 1st LVL	24.0	4.00	(wall)	6.0	32.64	18.0	24.27	1.97	1.20
								Σ =	3.95	2.40
								Total Wind Base Shear (kips)	6.80	4.34

Project Number: S200831-6	Plan Name: Qui Residence Remodel	Sheet Number: L2
Engineer: xxx	Specifics: SEISMIC WEIGHTS	Date: 9/2/2020

Unit Weights (psf)

Roof:	15	psf
Floor:	12	psf
Exterior Wall:	12	psf
Interior Wall:	8	psf
Concrete Deck:	0	psf

Seismic Weights include: (REF §12.7)
25% of storage Live loads
Actual partition weight or 10 psf min if applicable
Operating weight of permenant equipment
20% of uniform design snow loads for areas where Pf > 30 psf

LEVEL	ITEM	AREA / LENGTH	HEIGHT (ft)	WEIGHT (psf)		Item Total Weight. (lbs)	Sub-Total (kips)	Average Pressure (psf)
ROOF								
	Roof	400	1.10	15	=	6,629		
	Ext. Wall Below	75	4.00	12	=	3,600		
	Corridor Wall Below	50	4.00	8	=	1,600		
							12	**30**
2nd FLOOR								
	Floor	350	1.00	12	=	4,200		
	Deck	0	1.00	0	=	0		
	Low Roof	0	1.10	15	=	0		
	Ext. Wall Above	75	4.00	12	=	3,600		
	Corridor Wall Above	50	4.00	8	=	1,600		
	Ext. Wall Below	75	4.00	12	=	3,600		
	Corridor Wall Below	50	4.00	8	=	1,600		
							15	**42**
1st FLOOR								
	Ext. Wall Above	75	4.00	12	=	3,600		
	Corridor Wall Above	50	4.00	8	=	1,600		
							5	

STRUCTURE WEIGHT FOR SEISMIC BASE SHEAR: 26 kips

TOTAL WEIGHT OF STRUCTURE: 32 kips
(Includes Basement Dead Load)

Project Number:	Plan Name:	Sheet Number:
S200831-6	Qui Residence Remodel	L3
Engineer: xxx	Specifics: SEISMIC FORCES	Date: 9/2/2020

Equivelant Lateral Force Analysis per IBC 2015 1613.1 →ASCE 7-10 Table 12.6-1 →Sec 12.8

Data generated by: Seismic Design Values for Buildin **"Java Ground Motion Parameter Calculation"**

S_1 =	0.489		Maps
S_{DS} =	0.937		(ASCE 7 EQ 11.4.-3)
S_{D1} =	0.535		(ASCE 7 EQ 11.4.-4)
Seismic Importance Factor =	1.00		(ASCE 7 Table 11.5-1)
Seismic Design Category =	D		(ASCE 7 Table 11.6-1 & 11.6.2)
Response Modification Factor, R =	6.5		(ASCE 7 Table 12.2-1)
Seismic Force-Resisting System Description =	A.13 - light framed walls		

Building Height, h_n =	21.0	ft	
Building Period Coefficient, C_T =	0.020		(ASCE 7 Table 12.8.-2)
Approx. Fundamental Period, T_a =	0.196	$(C_T*(h_n^{0.75})$	(ASCE 7 EQ 12.8.-7)
Approx. Fundamental Period, T_L =	6.0	sec	(ASCE 7 11.4.5)

Seismic Response Coefficient

$C_s = S_{DS}/(R/I)$	C_s =	0.144		(ASCE 7 EQ 12.8.-2)

Seismic Response Coefficient, Maximum

$C_{s,\,MAX} = S_{D1}/(T*R/I)$	$C_{s,\,MAX}$ =	0.420	$T \leq T_L$	(ASCE 7 EQ 12.8.-3)
$C_{s,\,MAX} = S_{D1}\ T_L/(T^2*R/$	$C_{s,\,MAX}$ =	NA	$T > T_L$	(ASCE 7 EQ 12.8.-4)

Seismic Response Coefficient, Minimum

$C_{s,\,MIN} = 0.01$	$C_{s,\,MIN}$ =	0.010		(ASCE 7 EQ 12.8.-5)
$C_{s,\,MIN} = 0.5\ S_1 / (R/I)$	$C_{s,\,MIN}$ =	NA	if S1 > 0.6	(ASCE 7 EQ 12.8.-6)

C_s =	**0.144**		
Dead Load W =	26	kips	
V = Cs W =	3.8	kips	(ASCE 7 EQ 12.8.-1)
Q_E = V =	3.8	kips	(ASCE 7 EQ 12.4-3)
ρ =	1.0		(ASCE 7 12.3.4.2)
$E_H = \rho\ Q_E$	3.8	kips	(ASCE 7 EQ 12.4-3)
Ev = .2 S_{DS} D =	0.19	x D kips	

Factor for Alternate Basic Load conbinations - 2015 IBC 1605.3.2

E_H/1.4 =	**2.7**	**kips**	IBC 2015 1605.3.2
k =	1		(ASCE 7 12.8.3)

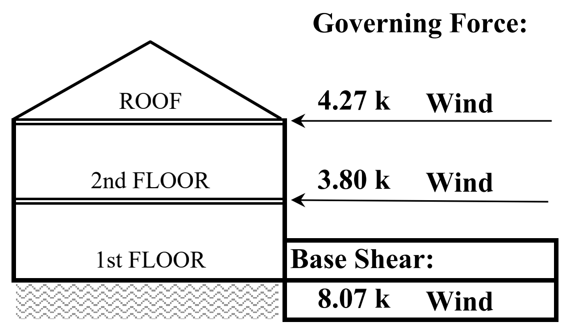
VERTICAL DISTRIBUTION (Per ASCE 7 - 12.8.3)

Floor	Area (ft^2)	Story Height H (ft)	Total Height h_x (ft)	Story Weight w_x (kips)	$w_x h_x^k$ (k-ft)	Vert Dist Factor Cvx	Story Force Fx (kips)	Factored Story Force (ASD) Fx ρ/1.4 = E_H/1.4 (kips)
Roof	400	8.08	16.16	12	191	0.62	2.4	**1.7**
2nd	350	8.08	8.08	15	118	0.38	1.5	**1.0**
				Sum =	309	1.000	3.8	**2.7**

Project Number:	Plan Name:	Sheet Number:
S200831-6	**Qui Residence Remodel**	**L4**
Engineer:	**Specifics:**	**Date:**
xxx	**DESIGN LOADS**	**9/2/2020**

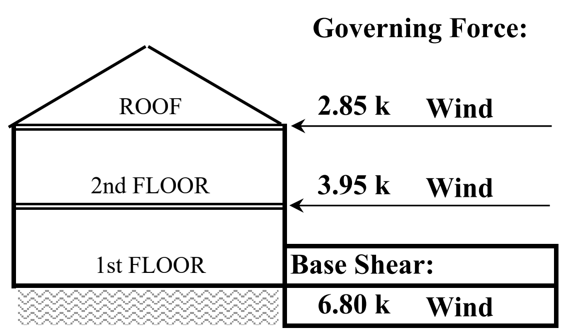
FRONT / BACK APPLIED FORCES

Wind Force *0.6 ω * W_S (kips)*		Seismic Force *E/1.4 (kips)*	
Per Level	Sum	Per Level	Sum
4.27		1.68	
3.80	4.27	1.04	1.68
	8.07		2.72



SIDE / SIDE APPLIED FORCES

Wind Force *0.6 ω * W_S (kips)*		Seismic Force *E/1.4 (kips)*	
Per Level	Sum	Per Level	Sum
2.85		1.68	
3.95	2.85	1.04	1.68
	6.80		2.72



Project Number: S200831-6	Plan Name: Qui Residence Remodel	Sheet Number: L5
Engineer: xxx	Specifics: Shear walls	Date: 9/2/2020

Notes:

* All walls designed with Force-Transfer should meet a minimum height to width ratio of 2:1 at Pier (SDPWS 2015, Table 4.3.4 p.25)
* Maximum allowed height to width ratio 3.5:1 for walls w/o openings (increased shear design values per SDPWS 2015, Table 4.3.4 p.25)
* Shear panel height is height to underside or roof or floor framing.

RED = Update Formula as required - Important
BLUE = Review and update as required - Typical Input

2nd Story Walls (Front - Back Direction)

"Adjusted" Story shear(kips) = **4.27**
Story height (ft) = 8.08
Shear Panel height (ft) = 8.08
Total Diaphragm width (ft) = **23.00**

Temporary Shoring shear (kips) **60%**

100% story shear **YES**

Stud Species HF
Governing Force (F/B Direction) = **Wind**
Dead load factor (F/B Direction) = **0.67** IBC 2015 Equation 16-18
Shear panel capacity (Wind or Seismic) = **Wind**
load balance check = OK

Gyp capacity = (PLF) 60.00

2nd Story Walls (Front - Back Direction)
Hold downs and window straps

Story	Wall Mark	Wall L(ft)	Opening Width (ft)	Opening Height (ft)	Opening (max) to Edge (ft)	Plate to Opening (ft)	Effective Length (ft)	Trib. Width (ft)	Percent Sharing (%)	Effective Trib. Width	Story V(kips)	Sum V(kips)	Panel Shear (plf)	Height/Width Reduction (%) R = 2*L/H	Design Panel Shear (plf)	Wall Type	Roof DL Trib(ft)	Story DL(klf)	Sum DL(klf)	OTM (k-ft)	RM (k-ft)	Resultant HD(kips)	HD TYPE	HD/Strap to DF or HF?	HD location Edge/Interior?	Resultant HD	Force at Window (Kips)	Window Strap
2	1.0	14.75	0.00	0.00	0.00	0.00	14.75	11.50	1.00	11.50	2.14	2.14	145	1.00	145	SW6	10.00	0.25	0.25	17.3	18.0	-0.05	flr-flr	HF	Edge	No HD	0.00	No strap
2	2.0	12.00	0.00	0.00	0.00	0.00	12.00	11.50	1.00	11.50	2.14	2.14	178	1.00	178	SW6	10.00	0.25	0.25	17.3	11.9	0.46	flr-flr	HF	Edge	No HD	0.00	No strap

New shearwalls to resist new remodel addition. All existing shearwall resistance to remain for existing residence elements.

Total Length GYP required in F/B direction to resist 100% lateral forces (ft) **Not required**
(including discounted capacity accounted for by OSB)

S = 26.75 | Total OSB wall length = (feet) 26.75 | S = 23.00 | 4.27 | **4.27** | **OK** | Total OSB Capacity (kips) 4.27

1st Story Walls (Front - Back Direction)

Shear panel capacity (Wind or Seismic) = **Wind**

1st Story Walls (Front - Back Direction)
Hold downs and window straps

"Adjusted" Story shear(kips) = **3.80**
Story height (ft) = 8.08
Shear Panel height (ft) = 8.08
Total Diaphragm width (ft) = **23.00**

Accumulated Shear = **8.07**
load balance check = Warning-Wall loads do not match story shear

Story	Wall Mark	Wall L(ft)	Opening Width (ft)	Opening Height (ft)	Opening (max) to Edge (ft)	Plate to Opening (ft)	Effective Length (ft)	Trib. Width (ft)	Percent Sharing (%)	Effective Trib. Width	Story V(kips)	Sum V(kips)	Panel Shear (plf)	Height/Width Reduction (%) R = 2*L/H	Design Panel Shear (plf)	Wall Type	Floor DL Trib(ft)	Story DL(klf)	Walls/DL Stacks?	Sum DL(klf)	OTM (k-ft)	RM (k-ft)	Resultant HD(kips)	HD TYPE	HD/Strap to DF or HF?	HD location Edge/Interior?	Resultant HD	Force at Window (Kips)	Window Strap
1	1.0	11.00	0.00	0.00	0.00	0.00	11.00	22.00	1.00	22.00	3.63	5.77	524	1.00	524	SW2	2.00	0.12	YES	0.37	63.9	14.9	4.66	flr-conc	HF	Edge	HDU8	0.00	No strap
1	2.0	11.00	0.00	0.00	0.00	0.00	11.00	11.50	1.00	11.50	1.90	4.03	367	1.00	367	SW3	2.00	0.12	YES	0.37	49.9	14.9	3.33	flr-conc	HF	Edge	HDU5	0.00	No strap

New shearwalls to resist new remodel addition. All existing shearwall resistance to remain for existing residence elements. Additional shear has been added to shearwall 2.0 from the remodeled deck.

Total Length GYP required in F/B direction to resist 100% lateral forces (ft) **Not required**
(including discounted capacity accounted for by OSB)

S = 22.00 | Total OSB wall length = (feet) 22.00 | S = 33.50 | 5.53 | **9.80** | **Warning-** | Total OSB Capacity (kips) 3.80

Project Number: S200831-6	Plan Name: Qui Residence Remodel	Sheet Number: L6
Engineer: xxx	Specifics: Shear walls	Date: 9/2/2020

Notes:
* All walls designed with Force-Transfer should meet a minimum height to width ratio of 2:1 at Pier (SDPWS 2015, Table 4.3.4 p.25)
* Maximum allowed height to width ratio 3.5:1 for walls w/o openings (increased shear design values per SDPWS 2015, Table 4.3.4 p.25)
* Shear panel height is height to underside or roof or floor framing.

RED = Update Formula as required - Important
BLUE = Review and update as required - Typical Input

2nd Story Walls (Side / Side Direction)

2nd Story Walls (Side / Side Direction)
Hold downs and window straps

"Adjusted" Story shear(kips) = 2.85
Story height (ft) = 8.08
Shear Panel height (ft) = 8.08
Total Diaphragm width (ft) = 24.00

Temporary Shoring shear (kips) **60%**

100% story shear **YES**

Stud Species HF
Governing Force (F/B Direction) = Wind
Dead load factor (F/B Direction) = 0.67
Shear panel capacity (Wind or Seismic) = Wind
load balance check = OK

IBC 2015 Equation 16-18

Gyp capacity = (PLF) 60.00

Story	Wall Mark	Wall L(ft)	Opening Width (ft)	Opening Height (ft)	Opening (max) to Edge (ft)	Plate to Opening (ft)	Effective Length (ft)	Trib. Width (ft)	Percent Sharing (%)	Effective Trib. Width	Story V(kips)	Sum V(kips)	Panel Shear (plf)	Height/Width Reduction (%) R = 2*L/H	Design Panel Shear (plf)	Wall Type	Roof DL Trib(ft)	Story DL(klf)	Sum DL(klf)	OTM (k-ft)	RM (k-ft)	Resultant HD(kips)	HD TYPE	HD/Strap to DF or HF?	HD location Edge/Interior?	Resultant HD	Force at Window (Kips)	Window Strap
2	A1	14.00	8.00	6.00	2.00	1.08	6.00	12.00	1.00	12.00	1.42	1.42	237	1.00	237	SW6	4.00	0.16	0.16	11.5	10.3	0.09	flr-flr	HF	Edge	No HD	1.79	CS14
2	B1	5.25	0.00	0.00	0.00	0.00	5.25	12.00	0.33	3.96	0.47	0.47	89	1.00	89	SW6	4.00	0.16	0.16	3.8	1.4	0.49	flr-flr	HF	Edge	No HD	0.00	No strap
2	B2	5.25	0.00	0.00	0.00	0.00	5.25	12.00	0.33	3.96	0.47	0.47	89	1.00	89	SW6	4.00	0.16	0.16	3.8	1.4	0.49	flr-flr	HF	Edge	No HD	0.00	No strap
2	B3	5.25	0.00	0.00	0.00	0.00	5.25	12.00	0.33	3.96	0.47	0.47	89	1.00	89	SW6	4.00	0.16	0.16	3.8	1.4	0.49	flr-flr	HF	Edge	No HD	0.00	No strap

New shearwalls to resist new remodel addition. All existing shearwall resistance to remain for existing residence elements. Additional shear has been included in shearwalls B1 and B2 to resist a portion of the existing residence

Total Length GYP required in F/B direction to resist 100% lateral forces (ft) **Not Required**
(including discounted capacity accounted for by OSB)

S = 29.75 | Total OSB wall length = (feet) 21.75 | S = 23.88 | 2.83 | **2.83** | **OK** | Total OSB Capacity (kips) | **2.85**

1st Story Walls (Side / Side Direction)

1st Story Walls (Side / Side Direction)
Hold downs and window straps

Shear panel capacity (Wind or Seismic) = Wind

"Adjusted" Story shear(kips) = 3.95
Story height (ft) = 8.08
Shear Panel height (ft) = 8.08
Total Diaphragm width (ft) = 24.00

Accumulated Shear = 6.80
load balance check = Warning-Wall loads do not match story shear

Story	Wall Mark	Wall L(ft)	Opening Width (ft)	Opening Height (ft)	Opening (max) to Edge (ft)	Plate to Opening (ft)	Effective Length (ft)	Trib. Width (ft)	Percent Sharing (%)	Effective Trib. Width	Story V(kips)	Sum V(kips)	Panel Shear (plf)	Height/Width Reduction (%) R = 2*L/H	Design Panel Shear (plf)	Wall Type	Floor DL Trib(ft)	Story DL(klf)	Walls/DL Stacks?	Sum DL(klf)	OTM (k-ft)	RM (k-ft)	Resultant HD(kips)	HD TYPE	HD/Strap to DF or HF?	HD location Edge/Interior?	Resultant HD	Force at Window (Kips)	Window Strap
1	A	18.50	0.00	0.00	0.00	0.00	18.50	12.00	0.66	7.93	1.30	2.73	147	1.00	147	SW6	6.00	0.17	YES	0.33	33.5	37.4	-0.21	flr-conc	HF	Edge	No HD	0.00	No strap
1	B	9.50	0.00	0.00	0.00	0.00	9.50	12.00	0.34	4.07	0.67	2.08	219	1.00	219	SW6	10.00	0.22	NO	0.22	20.6	6.6	1.56	flr-conc	HF	Edge	HDU5	0.00	No strap

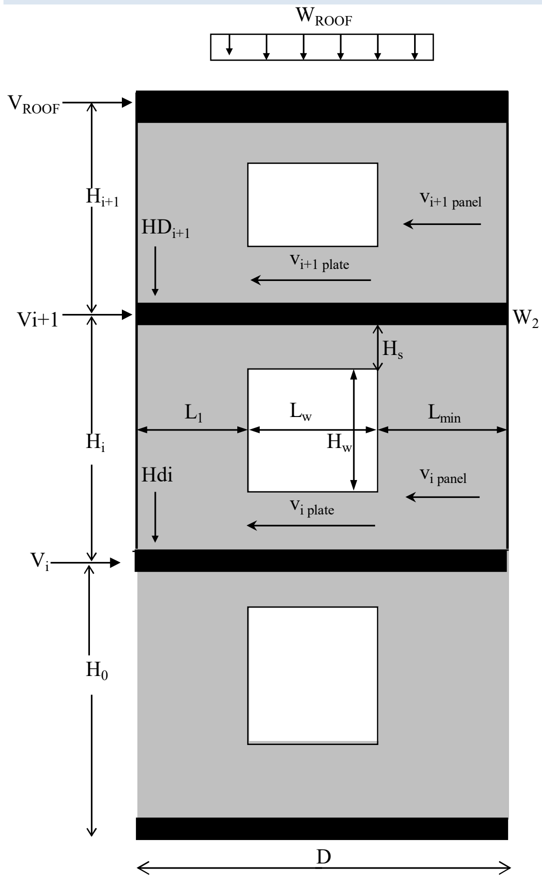
New shearwalls to resist new remodel addition. All existing shearwall resistance to remain for existing residence elements. Additional shear has been included in shearwalls B to resist a portion of the existing residence

Total Length GYP required in F/B direction to resist 100% lateral forces (ft) **Not Required**
(including discounted capacity accounted for by OSB)

S = 28.00 | Total OSB wall length = (feet) 28.00 | S = 12.00 | 1.97 | **4.81** | **Warning-** | Total OSB Capacity (kips) | **3.95**

Project **Qui Residence Remodel**	sheet number: **L7**
Subject **SHEAR WALL EQUATION DIAGRAM**	Date **9/2/2020**

SHEAR WALL WITH WINDOW BASED ON SHEAR TRANSFER:



Where:

V_i = Story Shear
W_i = Story Dead Load
HD_i = Story Holdown
M_{OTi} = Story Over Turning Moment
M_{Ri} = Story Resisting Moment

$M_{OT\ ROOF} = V_{ROOF} \text{ x } H_{1+1}$

$M_{OTi} = [(V_{i+1} + V_{ROOF}) \text{ x } H_i] + M_{OT\ ROOF}$

$M_{R\ ROOF} = 0.6 \text{ x } W_{ROOF} \text{ x } D^2/2$

$M_{Ri} = 0.6 \text{ x } (W_{i+1} + W_{ROOF}) \text{ x } D^2/2$

$HD_{i+1} = (M_{OT\ ROOF} - M_{R\ ROOF})/(D - 6")$

$HD_i = (M_{OTi} - M_{R1i})/(D - 6")$

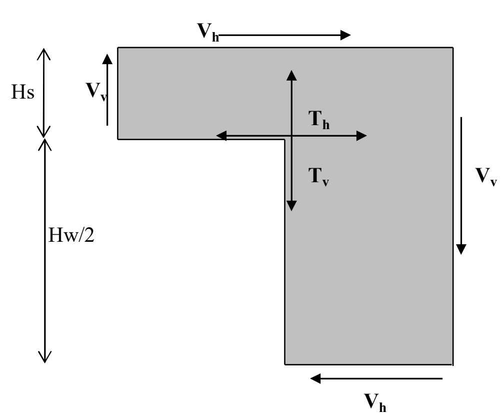
$V_{i+1\ panel} = V_{ROOF} / (L_1 + L_{max})$

$V_{i\ panel} = (V_{ROOF} + V_{i+1}) / (L_1 + L_{max})$

$V_{i+1\ plate} = V_{ROOF} / D$

$V_{i\ plate} = (V_{ROOF} + V_{i+1}) / D$

FORCE TRANSFER AROUND WINDOW CALCULATION (CANTILEVER PIER METHOD)



$V_h = v_{i\ panel} \text{ x } L_{max}$

$V_v = HD_i$

$T_h = V_h (H_w / 2 + H_s) / H_s$

T_v = Is resisted by the continuous stud adjacent to the window.

LONGITUDE
ONE TWENTY°
ENGINEERING & DESIGN

Supplementary Calculations for the following:

- ~ ***Hold-down anchor design/calculations***
- ~ ***Hand-rail calculations (wood/concrete)***
- ~ ***Balloon framed stud design***

- ~ ***Ledger Calculations/Data***
- ~ ***Knee Brace***

LONGITUDE
ONE TWENTY°
ENGINEERING & DESIGN

Hold-down anchor design calculations

Anchor Designer™ Software
Version 2.5.6582.0

Company:	L120 Engineering & Design	Date:	5/3/2018
Engineer:	MRT	Page:	1/4
Project:	Hold-down Anchors		
Address:			
Phone:			
E-mail:			

1.Project information

Customer company:
Customer contact name:
Customer e-mail:
Comment:

Project description:
Location:
Fastening description:

5/8" DIA Anchor

2. Input Data & Anchor Parameters

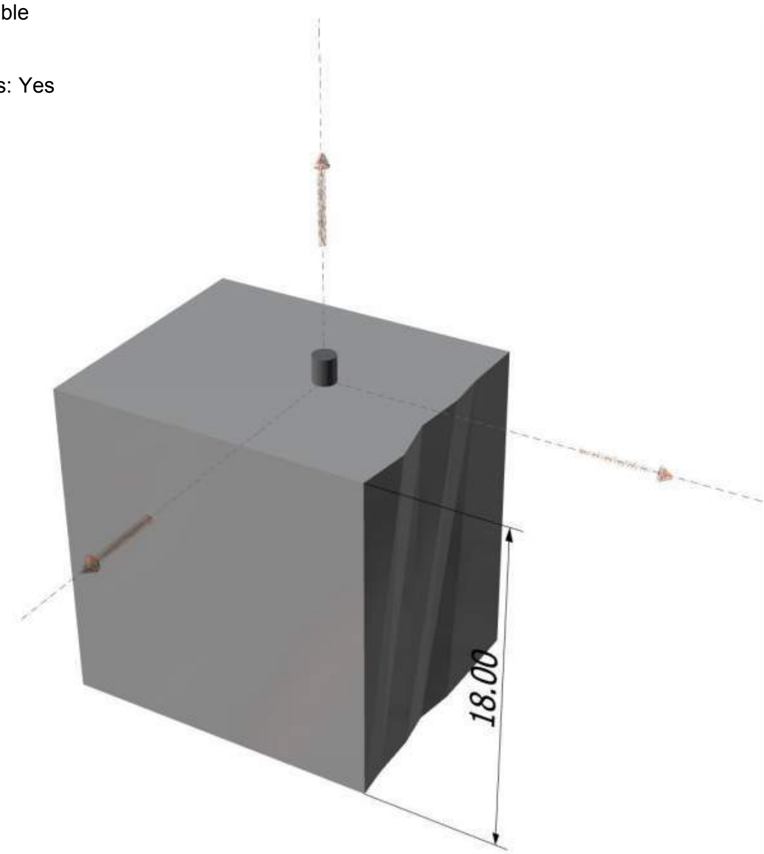
General
Design method:ACI 318-14
Units: Imperial units

Anchor Information:
Anchor type: Cast-in-place
Material: AB_H
Diameter (inch): 0.625
Effective Embedment depth, h_{ef} (inch): 4.000
Anchor category: -
Anchor ductility: Yes
h_{min} (inch): 6.13
C_{min} (inch): 1.38
S_{min} (inch): 2.50

Base Material
Concrete: Normal-weight
Concrete thickness, h (inch): 18.00
State: Cracked
Compressive strength, f'_c (psi): 2500
$\Psi_{c,V}$: 1.0
Reinforcement condition: A tension, A shear
Supplemental reinforcement: Not applicable
Reinforcement provided at corners: Yes
Ignore concrete breakout in tension: No
Ignore concrete breakout in shear: No
Ignore 6do requirement: Yes
Build-up grout pad: No

Load and Geometry
Load factor source: ACI 318 Section 5.3
Load combination: U = 0.9D + 1.0E
Seismic design: Yes
Anchors subjected to sustained tension: Not applicable
Ductility section for tension: 17.2.3.4.3 (a) (iii)-(vi) is satisfied
Ductility section for shear: 17.2.3.5.2 not applicable
Ω_0 factor: not set
Apply entire shear load at front row: No
Anchors only resisting wind and/or seismic loads: Yes

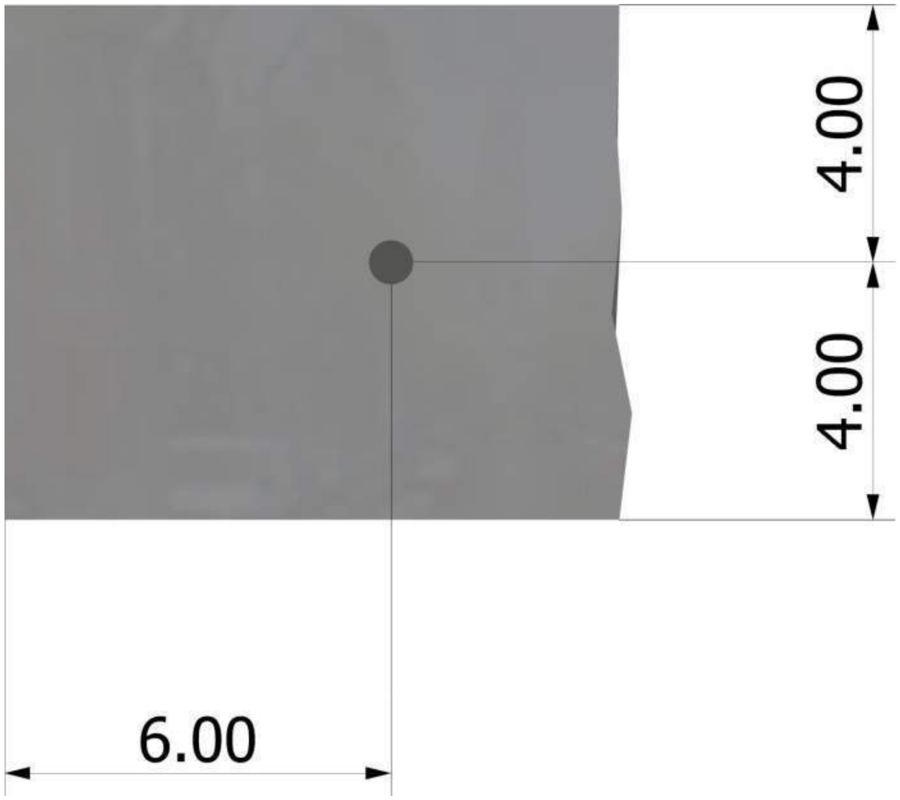
<Figure 1>



Anchor Designer™
Software
Version 2.5.6582.0

Company:	L120 Engineering & Design	Date:	5/3/2018
Engineer:	MRT	Page:	2/4
Project:	Hold-down Anchors		
Address:			
Phone:			
E-mail:			

<Figure 2>



Recommended Anchor
Anchor Name: PAB Pre-Assembled Anchor Bolt - PAB5H (5/8"Ø)

SIMPSON Strong-Tie® Anchor Designer™ Software
Version 2.5.6582.0

Company:	L120 Engineering & Design	Date:	5/3/2018
Engineer:	MRT	Page:	3/4
Project:	Hold-down Anchors		
Address:			
Phone:			
E-mail:			

3. Resulting Anchor Forces

Anchor	Tension load, N_{ua} (lb)	Shear load x, V_{uax} (lb)	Shear load y, V_{uay} (lb)	Shear load combined, $\sqrt{(V_{uax})^2+(V_{uay})^2}$ (lb)
1	2925.0	0.0	0.0	0.0
Sum	2925.0	0.0	0.0	0.0

Maximum concrete compression strain (‰): 0.00
Maximum concrete compression stress (psi): 0
Resultant tension force (lb): 2925
Resultant compression force (lb): 0
Eccentricity of resultant tension forces in x-axis, e'_{Nx} (inch): 0.00
Eccentricity of resultant tension forces in y-axis, e'_{Ny} (inch): 0.00

4. Steel Strength of Anchor in Tension (Sec. 17.4.1)

N_{sa} (lb)	ϕ	ϕN_{sa} (lb)
27120	0.75	20340

5. Concrete Breakout Strength of Anchor in Tension (Sec. 17.4.2)

$N_b = k_c \lambda_a \sqrt{f'_c} h_{ef}^{1.5}$ (Eq. 17.4.2.2a)

k_c	λ_a	f'_c (psi)	h_{ef} (in)	N_b (lb)
24.0	1.00	2500	4.000	9600

$0.75\phi N_{cb} = 0.75\phi\,(A_{Nc} / A_{Nco})\,\Psi_{ed,N}\Psi_{c,N}\Psi_{cp,N}N_b$ (Sec. 17.3.1 & Eq. 17.4.2.1a)

A_{Nc} (in^2)	A_{Nco} (in^2	$c_{a,min}$ (in)	$\Psi_{ed,N}$	$\Psi_{c,N}$	$\Psi_{cp,N}$	N_b (lb)	ϕ	$0.75\phi N_{cb}$ (lb)
103.00	144.00	4.00	0.900	1.00	1.000	9600	0.75	3476

6. Pullout Strength of Anchor in Tension (Sec. 17.4.3)

$0.75\phi N_{pn} = 0.75\phi\Psi_{c,P}N_p = 0.75\phi\Psi_{c,P}8A_{brg}f'_c$ (Sec. 17.3.1, Eq. 17.4.3.1 & 17.4.3.4)

$\Psi_{c,P}$	A_{brg} (in^2)	f'_c (psi)	ϕ	$0.75\phi N_{pn}$ (lb)
1.0	2.10	2500	0.70	22029

SIMPSON Strong-Tie®

Anchor Designer™
Software
Version 2.5.6582.0

Company:	L120 Engineering & Design	Date:	5/3/2018
Engineer:	MRT	Page:	4/4
Project:	Hold-down Anchors		
Address:			
Phone:			
E-mail:			

11. Results

11. Interaction of Tensile and Shear Forces (Sec. D.7)?

Tension	Factored Load, N_{ua} (lb)	Design Strength, $øN_n$ (lb)	Ratio	Status
Steel	2925	20340	0.14	Pass
Concrete breakout	**2925**	**3476**	**0.84**	**Pass (Governs)**
Pullout	2925	22029	0.13	Pass

PAB5H (5/8"Ø) with hef = 4.000 inch meets the selected design criteria.

ACI 318-14 Section 17.2.3.4.3(a) (i) & (ii) Calculations for Ductility requirement for tension load

Steel	Factored Load, N_{ua} (lb)	1.2 x Nominal Strength, N_n (lb)	Ratio	
Steel	2925	32544	9.0 %	

Concrete	Nominal Strength, N_n (lb)	Nominal Strength, N_n (lb)	Ratio	
Concrete breakout	**2925**	**6180**	**47.3 %**	**Governs**
Pullout	2925	41960	7.0 %	

ACI 318-14 Section 17.2.3.4.3(a) (i) & (ii) is not satisfied since steel ratio does not govern.

12. Warnings

- Minimum spacing and edge distance requirement of 6da per ACI 318 Sections 17.7.1 and 17.7.2 for torqued cast-in-place anchor is waived per designer option.

- Brittle failure governs for tension. Governing anchor failure mode is brittle failure. Attachment shall be designed to satisfy the requirements of ACI 318-14 Section 17.2.3.4.3 for structures assigned to Seismic Design Category C, D, E, or F when the component of the strength level earthquake force applied to anchors exceeds 20 percent of the total factored anchor force associated with the same load combination. In case when ACI 318-14 Sections 17.2.3.4.3 (a)(iii) to (vi), (b), (c) or (d) is satisfied for tension loading, select appropriate checkbox from Inputs tab to disable this message. Alternatively, Ω0 factor can be entered to satisfy ACI 318-14 Section 17.2.3.4.3(d) to increase the earthquake portion of the loads as required.

- Per designer input, the shear component of the strength-level earthquake force applied to anchors does not exceed 20 percent of the total factored anchor shear force associated with the same load combination. Therefore the ductility requirements of ACI 318 17.2.3.5.2 for shear need not be satisfied – designer to verify.

- Designer must exercise own judgement to determine if this design is suitable.

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Engineer:	MRT	Page:	1/4
Project:	Hold-down Anchors		
Address:			
Phone:			
E-mail:			

1.Project information

Customer company:
Customer contact name:
Customer e-mail:
Comment:

Project description:
Location:
Fastening description:

3/4" DIA Anchor

2. Input Data & Anchor Parameters

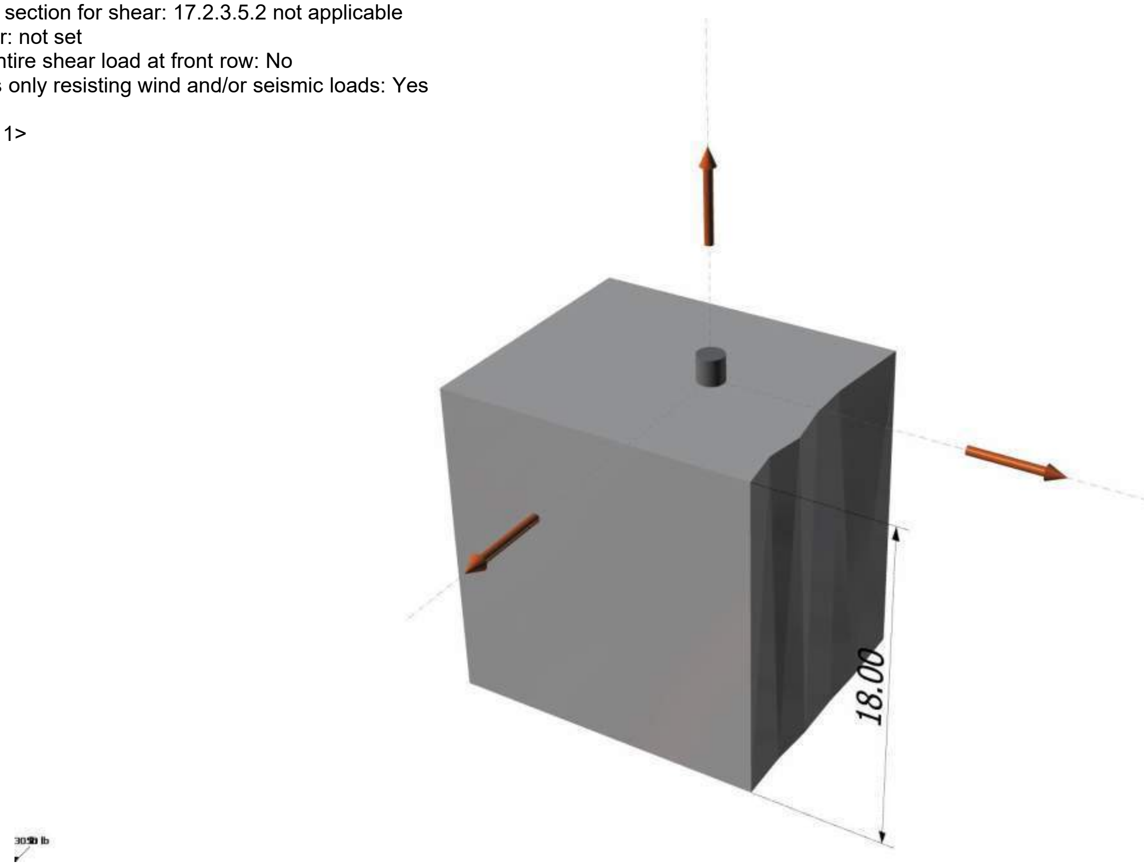
General
Design method:ACI 318-14
Units: Imperial units

Anchor Information:
Anchor type: Cast-in-place
Material: AB
Diameter (inch): 0.750
Effective Embedment depth, h_{ef} (inch): 12.000
Anchor category: -
Anchor ductility: Yes
h_{min} (inch): 14.25
C_{min} (inch): 1.63
S_{min} (inch): 3.00

Base Material
Concrete: Normal-weight
Concrete thickness, h (inch): 18.00
State: Cracked
Compressive strength, f'_c (psi): 2500
$\Psi_{c,V}$: 1.0
Reinforcement condition: A tension, A shear
Supplemental reinforcement: Not applicable
Reinforcement provided at corners: Yes
Ignore concrete breakout in tension: Yes
Ignore concrete breakout in shear: No
Ignore 6do requirement: Yes
Build-up grout pad: No

Load and Geometry
Load factor source: ACI 318 Section 5.3
Load combination: U = 0.9D + 1.0E
Seismic design: Yes
Anchors subjected to sustained tension: Not applicable
Ductility section for tension: 17.2.3.4.3 (a) (iii)-(vi) is satisfied
Ductility section for shear: 17.2.3.5.2 not applicable
Ω_0 factor: not set
Apply entire shear load at front row: No
Anchors only resisting wind and/or seismic loads: Yes

<Figure 1>



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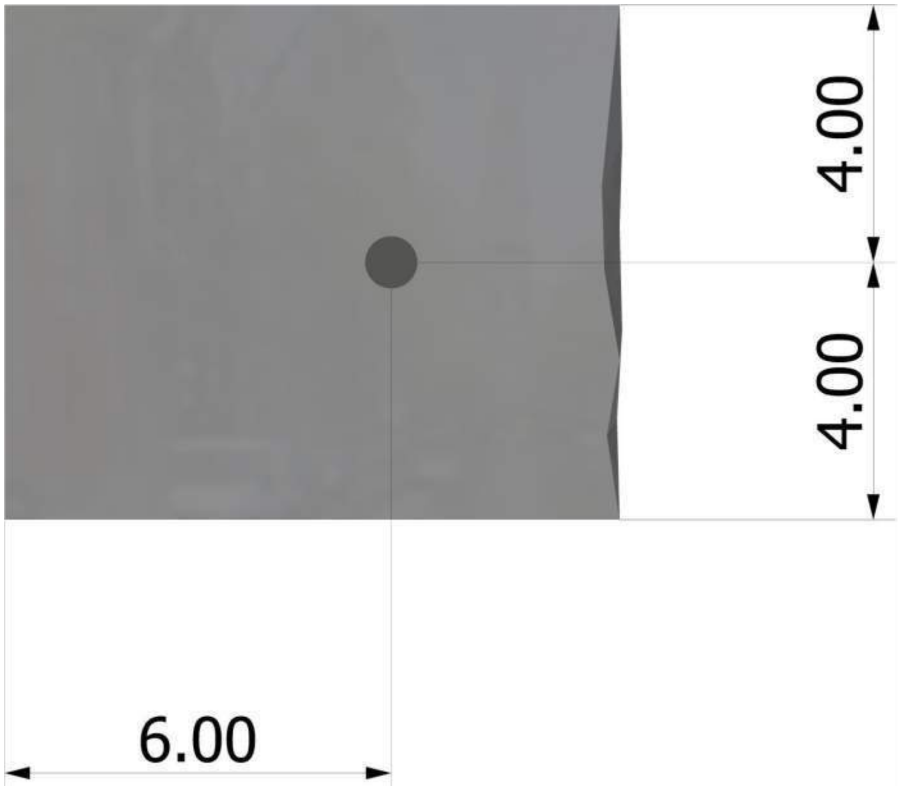
<Figure 2>



Recommended Anchor
Anchor Name: PAB Pre-Assembled Anchor Bolt - PAB6 (3/4"Ø)

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Phone:			
E-mail:			

3. Resulting Anchor Forces

Anchor	Tension load, N_{ua} (lb)	Shear load x, V_{uax} (lb)	Shear load y, V_{uay} (lb)	Shear load combined, $\sqrt{(V_{uax})^2+(V_{uay})^2}$ (lb)
1	13050.0	0.0	0.0	0.0
Sum	13050.0	0.0	0.0	0.0

Maximum concrete compression strain (‰): 0.00
Maximum concrete compression stress (psi): 0
Resultant tension force (lb): 0
Resultant compression force (lb): 0
Eccentricity of resultant tension forces in x-axis, e'_{Nx} (inch): 0.00
Eccentricity of resultant tension forces in y-axis, e'_{Ny} (inch): 0.00

4. Steel Strength of Anchor in Tension (Sec. 17.4.1)

N_{sa} (lb)	ϕ	ϕN_{sa} (lb)
19370	0.75	14528

6. Pullout Strength of Anchor in Tension (Sec. 17.4.3)

$0.75\phi N_{pn} = 0.75\phi \Psi_{c,P} N_p = 0.75\phi \Psi_{c,P} 8 A_{brg} f'_c$ (Sec. 17.3.1, Eq. 17.4.3.1 & 17.4.3.4)

$\Psi_{c,P}$	A_{brg} (in^2)	f'_c (psi)	ϕ	$0.75\phi N_{pn}$ (lb)
1.0	3.53	2500	0.70	37107

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7. Side-Face Blowout Strength of Anchor in Tension (Sec. 17.4.4)

$0.75\phi N_{sb} = 0.75\phi\{(1+c_{a2}/c_{a1})/4\}(160c_{a1}\sqrt{A_{brg}})\lambda\sqrt{f'_c}$ (Sec. 17.3.1 & Eq. 17.4.4.1)

c_{a1} (in)	c_{a2} (in)	A_{brg} (in^2)	λ_a	f'_c (psi)	ϕ	$0.75\phi N_{sbg}$ (lb)
4.00	6.00	3.53	1.00	2500	0.75	21149

11. Results

11. Interaction of Tensile and Shear Forces (Sec. D.7)?

Tension	Factored Load, N_{ua} (lb)	Design Strength, øN_n (lb)	Ratio	Status
Steel	**13050**	**14528**	**0.90**	**Pass (Governs)**
Pullout	13050	37107	0.35	Pass
Side-face blowout	13050	21149	0.62	Pass

PAB6 (3/4"Ø) with hef = 12.000 inch meets the selected design criteria.

ACI 318-14 Section 17.2.3.4.3(a) (i) & (ii) Calculations for Ductility requirement for tension load

Steel	Factored Load, N_{ua} (lb)	1.2 x Nominal Strength, N_n (lb)	Ratio	
Steel	**13050**	**23244**	**56.1%**	**Governs**

Concrete	Nominal Strength, N_n (lb)	Nominal Strength, N_n (lb)	Ratio
Pullout	13050	70680	18.5%
Side-face blowout	13050	37598	34.7%

ACI 318-14 Section 17.2.3.4.3(a) (i) & (ii) satisfied since steel ratio governs and the steel element is ductile.

12. Warnings

- Minimum spacing and edge distance requirement of 6da per ACI 318 Sections 17.7.1 and 17.7.2 for torqued cast-in-place anchor is waived per designer option.

- Concrete breakout strength in tension has not been evaluated against applied tension load(s) per designer option. Refer to ACI 318 Section 17.3.2.1 for conditions where calculations of the concrete breakout strength may not be required.

- Per designer input, the shear component of the strength-level earthquake force applied to anchors does not exceed 20 percent of the total factored anchor shear force associated with the same load combination. Therefore the ductility requirements of ACI 318 17.2.3.5.2 for shear need not be satisfied – designer to verify.

- Designer must exercise own judgement to determine if this design is suitable.

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Address:			
Phone:			
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1.Project information

Customer company:
Customer contact name:
Customer e-mail:
Comment:

Project description:
Location:
Fastening description:

7/8" DIA Anchor

2. Input Data & Anchor Parameters

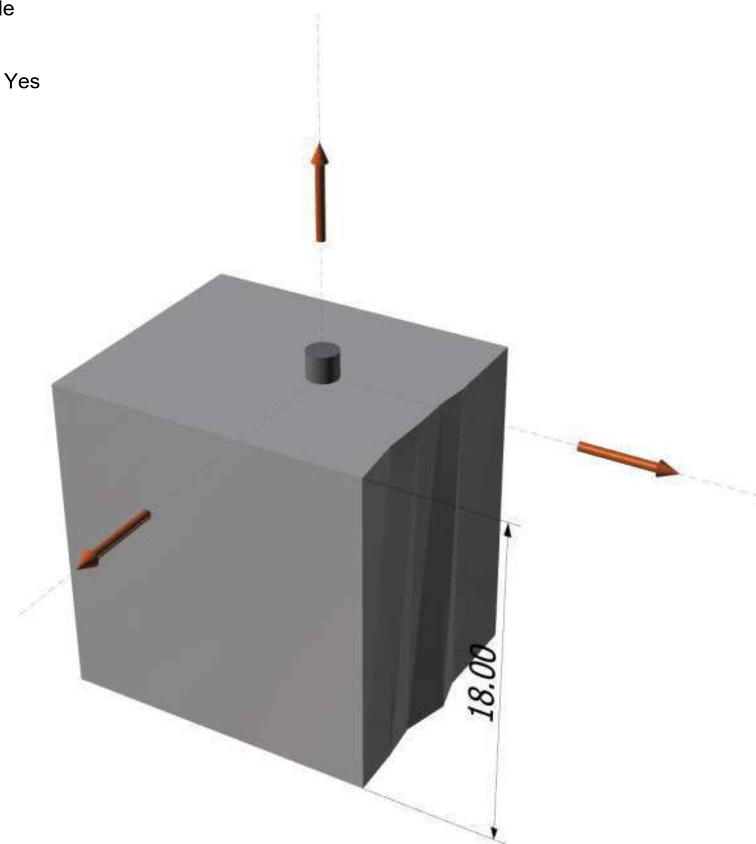
General
Design method:ACI 318-14
Units: Imperial units

Anchor Information:
Anchor type: Cast-in-place
Material: AB_H
Diameter (inch): 0.875
Effective Embedment depth, h_{ef} (inch): 12.000
Anchor category: -
Anchor ductility: Yes
h_{min} (inch): 14.38
C_{min} (inch): 1.75
S_{min} (inch): 3.50

Base Material
Concrete: Normal-weight
Concrete thickness, h (inch): 18.00
State: Cracked
Compressive strength, f'_c (psi): 2500
$\Psi_{c,V}$: 1.0
Reinforcement condition: A tension, A shear
Supplemental reinforcement: Not applicable
Reinforcement provided at corners: Yes
Ignore concrete breakout in tension: Yes
Ignore concrete breakout in shear: No
Ignore 6do requirement: Yes
Build-up grout pad: No

Load and Geometry
Load factor source: ACI 318 Section 5.3
Load combination: U = 0.9D + 1.0E
Seismic design: Yes
Anchors subjected to sustained tension: Not applicable
Ductility section for tension: 17.2.3.4.3 (a) (iii)-(vi) is satisfied
Ductility section for shear: 17.2.3.5.2 not applicable
Ω_0 factor: not set
Apply entire shear load at front row: No
Anchors only resisting wind and/or seismic loads: Yes

<Figure 1>



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<Figure 2>

Recommended Anchor
Anchor Name: PAB Pre-Assembled Anchor Bolt - PAB7H (7/8"Ø)

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Anchor Designer™ Software
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3. Resulting Anchor Forces

Anchor	Tension load, N_{ua} (lb)	Shear load x, V_{uax} (lb)	Shear load y, V_{uay} (lb)	Shear load combined, $\sqrt{(V_{uax})^2+(V_{uay})^2}$ (lb)
1	18000.0	0.0	0.0	0.0
Sum	18000.0	0.0	0.0	0.0

Maximum concrete compression strain (‰): 0.00
Maximum concrete compression stress (psi): 0
Resultant tension force (lb): 0
Resultant compression force (lb): 0
Eccentricity of resultant tension forces in x-axis, e'_{Nx} (inch): 0.00
Eccentricity of resultant tension forces in y-axis, e'_{Ny} (inch): 0.00

4. Steel Strength of Anchor in Tension (Sec. 17.4.1)

N_{sa} (lb)	ϕ	ϕN_{sa} (lb)
55440	0.75	41580

6. Pullout Strength of Anchor in Tension (Sec. 17.4.3)

$0.75\phi N_{pn} = 0.75\phi \Psi_{c,P} N_p = 0.75\phi \Psi_{c,P} 8 A_{brg} f'_c$ (Sec. 17.3.1, Eq. 17.4.3.1 & 17.4.3.4)

$\Psi_{c,P}$	A_{brg} (in^2)	f'_c (psi)	ϕ	$0.75\phi N_{pn}$ (lb)
1.0	4.07	2500	0.70	42683

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E-mail:			

7. Side-Face Blowout Strength of Anchor in Tension (Sec. 17.4.4)

$0.75\phi N_{sb} = 0.75\phi\{(1+c_{a2}/c_{a1})/4\}(160c_{a1}\sqrt{A_{brg}})\lambda\sqrt{f'_c}$ (Sec. 17.3.1 & Eq. 17.4.4.1)

c_{a1} (in)	c_{a2} (in)	A_{brg} (in^2)	λ_a	f'_c (psi)	ϕ	$0.75\phi N_{sbg}$ (lb)
4.00	6.00	4.07	1.00	2500	0.75	22682

11. Results

11. Interaction of Tensile and Shear Forces (Sec. D.7)?

Tension	Factored Load, N_{ua} (lb)	Design Strength, $øN_n$ (lb)	Ratio	Status
Steel	18000	41580	0.43	Pass
Pullout	18000	42683	0.42	Pass
Side-face blowout	**18000**	**22682**	**0.79**	**Pass (Governs)**

PAB7H (7/8"Ø) with hef = 12.000 inch meets the selected design criteria.

ACI 318-14 Section 17.2.3.4.3(a) (i) & (ii) Calculations for Ductility requirement for tension load

Steel	Factored Load, N_{ua} (lb)	1.2 x Nominal Strength, N_n (lb)	Ratio	
Steel	18000	66528	27.1%	

Concrete	Nominal Strength, N_n (lb)	Nominal Strength, N_n (lb)	Ratio	
Pullout	18000	81300	22.1%	
Side-face blowout	**18000**	**40324**	**44.6%**	**Governs**

ACI 318-14 Section 17.2.3.4.3(a) (i) & (ii) is not satisfied since steel ratio does not govern.

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12. Warnings

- Minimum spacing and edge distance requirement of 6da per ACI 318 Sections 17.7.1 and 17.7.2 for torqued cast-in-place anchor is waived per designer option.

- Concrete breakout strength in tension has not been evaluated against applied tension load(s) per designer option. Refer to ACI 318 Section 17.3.2.1 for conditions where calculations of the concrete breakout strength may not be required.

- Brittle failure governs for tension. Governing anchor failure mode is brittle failure. Attachment shall be designed to satisfy the requirements of ACI 318-14 Section 17.2.3.4.3 for structures assigned to Seismic Design Category C, D, E, or F when the component of the strength level earthquake force applied to anchors exceeds 20 percent of the total factored anchor force associated with the same load combination. In case when ACI 318-14 Sections 17.2.3.4.3 (a)(iii) to (vi), (b), (c) or (d) is satisfied for tension loading, select appropriate checkbox from Inputs tab to disable this message. Alternatively, Ω0 factor can be entered to satisfy ACI 318-14 Section 17.2.3.4.3(d) to increase the earthquake portion of the loads as required.

- Per designer input, the shear component of the strength-level earthquake force applied to anchors does not exceed 20 percent of the total factored anchor shear force associated with the same load combination. Therefore the ductility requirements of ACI 318 17.2.3.5.2 for shear need not be satisfied – designer to verify.

- Designer must exercise own judgement to determine if this design is suitable.

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Project:	Hold-down Anchors		
Address:			
Phone:			
E-mail:			

1.Project information

Customer company:
Customer contact name:
Customer e-mail:
Comment:

Project description:
Location:
Fastening description:

1" DIA Anchor

2. Input Data & Anchor Parameters

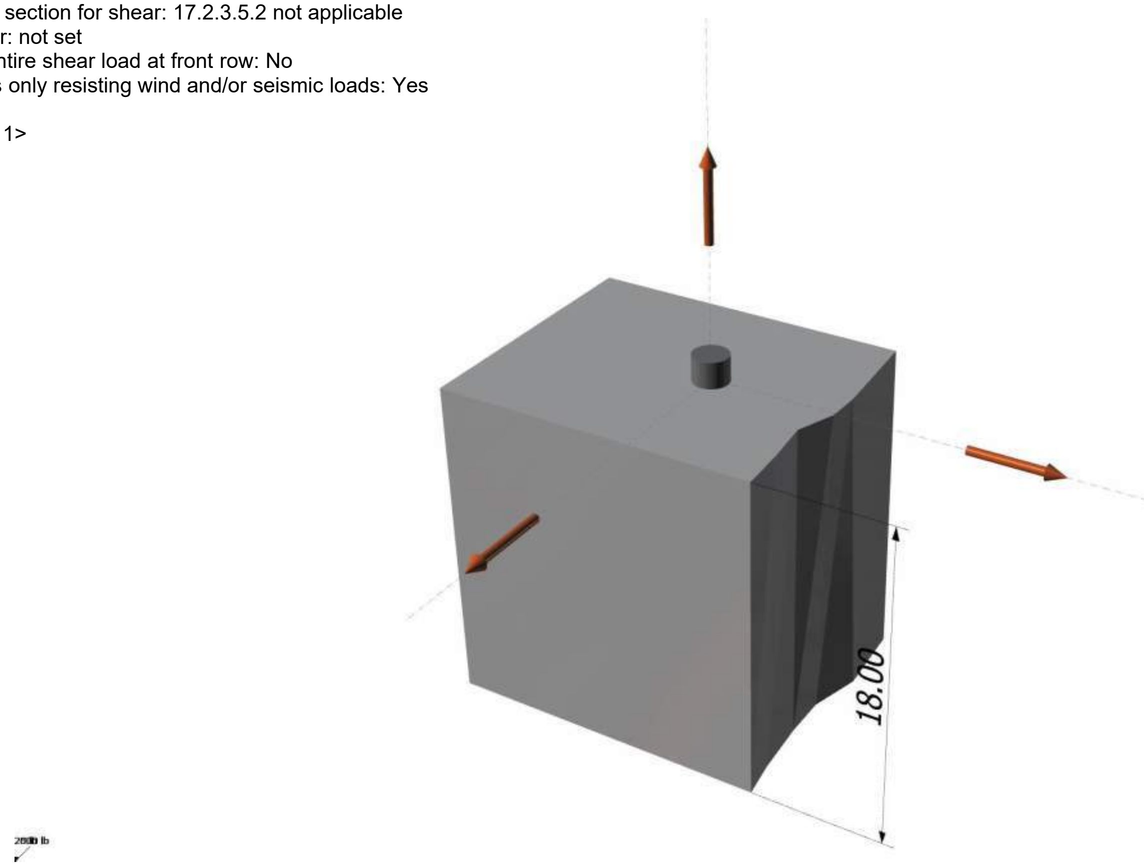
General
Design method:ACI 318-14
Units: Imperial units

Anchor Information:
Anchor type: Cast-in-place
Material: AB_H
Diameter (inch): 1.000
Effective Embedment depth, h_{ef} (inch): 15.000
Anchor category: -
Anchor ductility: Yes
h_{min} (inch): 17.63
C_{min} (inch): 1.88
S_{min} (inch): 4.00

Base Material
Concrete: Normal-weight
Concrete thickness, h (inch): 18.00
State: Cracked
Compressive strength, f'_c (psi): 2500
$\Psi_{c,V}$: 1.0
Reinforcement condition: A tension, A shear
Supplemental reinforcement: Not applicable
Reinforcement provided at corners: Yes
Ignore concrete breakout in tension: Yes
Ignore concrete breakout in shear: No
Ignore 6do requirement: Yes
Build-up grout pad: No

Load and Geometry
Load factor source: ACI 318 Section 5.3
Load combination: U = 0.9D + 1.0E
Seismic design: Yes
Anchors subjected to sustained tension: Not applicable
Ductility section for tension: 17.2.3.4.3 (a) (iii)-(vi) is satisfied
Ductility section for shear: 17.2.3.5.2 not applicable
Ω_0 factor: not set
Apply entire shear load at front row: No
Anchors only resisting wind and/or seismic loads: Yes

<Figure 1>



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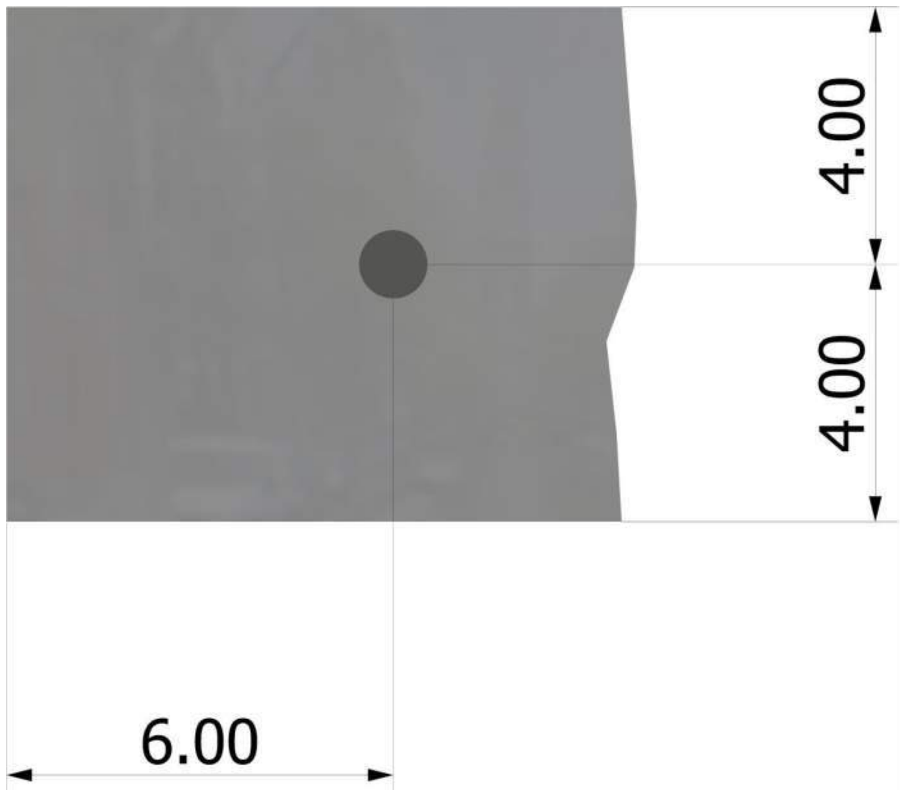
<Figure 2>



Recommended Anchor
Anchor Name: PAB Pre-Assembled Anchor Bolt - PAB8H (1"Ø)

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Company:	L120 Engineering & Design	Date:	1/14/2018
Engineer:	MRT	Page:	3/5
Project:	Hold-down Anchors		
Address:			
Phone:			
E-mail:			

3. Resulting Anchor Forces

Anchor	Tension load, N_{ua} (lb)	Shear load x, V_{uax} (lb)	Shear load y, V_{uay} (lb)	Shear load combined, $\sqrt{(V_{uax})^2+(V_{uay})^2}$ (lb)
1	22500.0	0.0	0.0	0.0
Sum	22500.0	0.0	0.0	0.0

Maximum concrete compression strain (‰): 0.00
Maximum concrete compression stress (psi): 0
Resultant tension force (lb): 0
Resultant compression force (lb): 0
Eccentricity of resultant tension forces in x-axis, e'_{Nx} (inch): 0.00
Eccentricity of resultant tension forces in y-axis, e'_{Ny} (inch): 0.00

4. Steel Strength of Anchor in Tension (Sec. 17.4.1)

N_{sa} (lb)	ϕ	ϕN_{sa} (lb)
72720	0.75	54540

6. Pullout Strength of Anchor in Tension (Sec. 17.4.3)

$0.75\phi N_{pn} = 0.75\phi \Psi_{c,P} N_p = 0.75\phi \Psi_{c,P} 8 A_{brg} f'_c$ (Sec. 17.3.1, Eq. 17.4.3.1 & 17.4.3.4)

$\Psi_{c,P}$	A_{brg} (in^2)	f'_c (psi)	ϕ	$0.75\phi N_{pn}$ (lb)
1.0	5.15	2500	0.70	54117

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Phone:			
E-mail:			

7. Side-Face Blowout Strength of Anchor in Tension (Sec. 17.4.4)

$0.75\phi N_{sb} = 0.75\phi\{(1+c_{a2}/c_{a1})/4\}(160c_{a1}\sqrt{A_{brg}})\lambda\sqrt{f'_c}$ (Sec. 17.3.1 & Eq. 17.4.4.1)

c_{a1} (in)	c_{a2} (in)	A_{brg} (in^2)	λ_a	f'_c (psi)	ϕ	$0.75\phi N_{sbg}$ (lb)
4.00	6.00	5.15	1.00	2500	0.75	25540

11. Results

11. Interaction of Tensile and Shear Forces (Sec. D.7)?

Tension	Factored Load, N_{ua} (lb)	Design Strength, ϕN_n (lb)	Ratio	Status
Steel	22500	54540	0.41	Pass
Pullout	22500	54117	0.42	Pass
Side-face blowout	**22500**	**25540**	**0.88**	**Pass (Governs)**

PAB8H (1"Ø) with hef = 15.000 inch meets the selected design criteria.

ACI 318-14 Section 17.2.3.4.3(a) (i) & (ii) Calculations for Ductility requirement for tension load

Steel	Factored Load, N_{ua} (lb)	1.2 x Nominal Strength, N_n (lb)	Ratio	
Steel	22500	87264	25.8%	

Concrete	Nominal Strength, N_n (lb)	Nominal Strength, N_n (lb)	Ratio	
Pullout	22500	103080	21.8%	
Side-face blowout	**22500**	**45405**	**49.6%**	**Governs**

ACI 318-14 Section 17.2.3.4.3(a) (i) & (ii) is not satisfied since steel ratio does not govern.

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Project:	Hold-down Anchors		
Address:			
Phone:			
E-mail:			

12. Warnings

- Minimum spacing and edge distance requirement of 6da per ACI 318 Sections 17.7.1 and 17.7.2 for torqued cast-in-place anchor is waived per designer option.

- Concrete breakout strength in tension has not been evaluated against applied tension load(s) per designer option. Refer to ACI 318 Section 17.3.2.1 for conditions where calculations of the concrete breakout strength may not be required.

- Brittle failure governs for tension. Governing anchor failure mode is brittle failure. Attachment shall be designed to satisfy the requirements of ACI 318-14 Section 17.2.3.4.3 for structures assigned to Seismic Design Category C, D, E, or F when the component of the strength level earthquake force applied to anchors exceeds 20 percent of the total factored anchor force associated with the same load combination. In case when ACI 318-14 Sections 17.2.3.4.3 (a)(iii) to (vi), (b), (c) or (d) is satisfied for tension loading, select appropriate checkbox from Inputs tab to disable this message. Alternatively, Ω0 factor can be entered to satisfy ACI 318-14 Section 17.2.3.4.3(d) to increase the earthquake portion of the loads as required.

- Per designer input, the shear component of the strength-level earthquake force applied to anchors does not exceed 20 percent of the total factored anchor shear force associated with the same load combination. Therefore the ductility requirements of ACI 318 17.2.3.5.2 for shear need not be satisfied – designer to verify.

- Designer must exercise own judgement to determine if this design is suitable.

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Version 2.5.6582.0

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Project:	Hold-down Anchors		
Address:			
Phone:			
E-mail:			

1.Project information

Customer company:
Customer contact name:
Customer e-mail:
Comment:

Project description:
Location:
Fastening description:

1 1/8" DIA Anchor

2. Input Data & Anchor Parameters

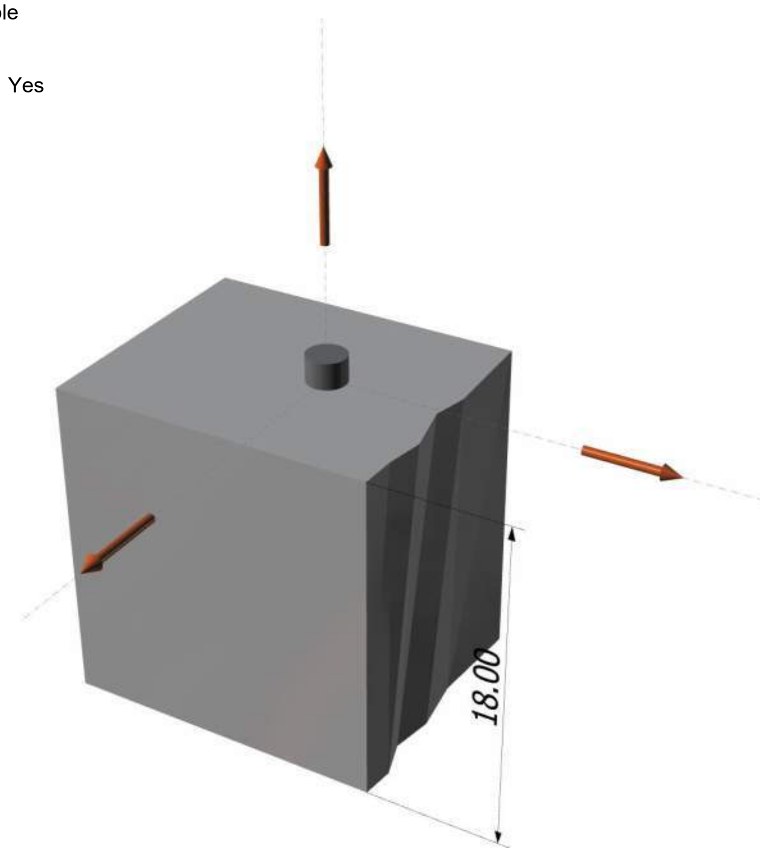
General
Design method:ACI 318-14
Units: Imperial units

Anchor Information:
Anchor type: Cast-in-place
Material: AB
Diameter (inch): 1.125
Effective Embedment depth, h_{ef} (inch): 15.000
Anchor category: -
Anchor ductility: Yes
h_{min} (inch): 17.75
C_{min} (inch): 2.13
S_{min} (inch): 4.50

Base Material
Concrete: Normal-weight
Concrete thickness, h (inch): 18.00
State: Cracked
Compressive strength, f'_c (psi): 2500
$\Psi_{c,V}$: 1.0
Reinforcement condition: A tension, A shear
Supplemental reinforcement: Not applicable
Reinforcement provided at corners: Yes
Ignore concrete breakout in tension: Yes
Ignore concrete breakout in shear: No
Ignore 6do requirement: Yes
Build-up grout pad: No

Load and Geometry
Load factor source: ACI 318 Section 5.3
Load combination: U = 0.9D + 1.0E
Seismic design: Yes
Anchors subjected to sustained tension: Not applicable
Ductility section for tension: 17.2.3.4.3 (a) (iii)-(vi) is satisfied
Ductility section for shear: 17.2.3.5.2 not applicable
Ω_0 factor: not set
Apply entire shear load at front row: No
Anchors only resisting wind and/or seismic loads: Yes

<Figure 1>



Anchor Designer™
Software
Version 2.5.6582.0

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Engineer:	MRT	Page:	2/5
Project:	Hold-down Anchors		
Address:			
Phone:			
E-mail:			

<Figure 2>

Recommended Anchor
Anchor Name: PAB Pre-Assembled Anchor Bolt - PAB9 (1 1/8"Ø)

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3. Resulting Anchor Forces

Anchor	Tension load, N_{ua} (lb)	Shear load x, V_{uax} (lb)	Shear load y, V_{uay} (lb)	Shear load combined, $\sqrt{(V_{uax})^2+(V_{uay})^2}$ (lb)
1	27900.0	0.0	0.0	0.0
Sum	27900.0	0.0	0.0	0.0

Maximum concrete compression strain (‰): 0.00
Maximum concrete compression stress (psi): 0
Resultant tension force (lb): 0
Resultant compression force (lb): 0
Eccentricity of resultant tension forces in x-axis, e'_{Nx} (inch): 0.00
Eccentricity of resultant tension forces in y-axis, e'_{Ny} (inch): 0.00

4. Steel Strength of Anchor in Tension (Sec. 17.4.1)

N_{sa} (lb)	ϕ	ϕN_{sa} (lb)
44255	0.75	33191

6. Pullout Strength of Anchor in Tension (Sec. 17.4.3)

$0.75\phi N_{pn} = 0.75\phi \Psi_{c,P} N_p = 0.75\phi \Psi_{c,P} 8 A_{brg} f'_c$ (Sec. 17.3.1, Eq. 17.4.3.1 & 17.4.3.4)

$\Psi_{c,P}$	A_{brg} (in^2)	f'_c (psi)	ϕ	$0.75\phi N_{pn}$ (lb)
1.0	6.37	2500	0.70	66885

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7. Side-Face Blowout Strength of Anchor in Tension (Sec. 17.4.4)

$0.75\phi N_{sb} = 0.75\phi\{(1+c_{a2}/c_{a1})/4\}(160c_{a1}\sqrt{A_{brg}})\lambda\sqrt{f'_c}$ (Sec. 17.3.1 & Eq. 17.4.4.1)

c_{a1} (in)	c_{a2} (in)	A_{brg} (in^2)	λ_a	f'_c (psi)	ϕ	$0.75\phi N_{sbg}$ (lb)
4.00	6.00	6.37	1.00	2500	0.75	28394

11. Results

11. Interaction of Tensile and Shear Forces (Sec. D.7)?

Tension	Factored Load, N_{ua} (lb)	Design Strength, øN_n (lb)	Ratio	Status
Steel	27900	33191	0.84	Pass
Pullout	27900	66885	0.42	Pass
Side-face blowout	**27900**	**28394**	**0.98**	**Pass (Governs)**

PAB9 (1 1/8"Ø) with hef = 15.000 inch meets the selected design criteria.

ACI 318-14 Section 17.2.3.4.3(a) (i) & (ii) Calculations for Ductility requirement for tension load

Steel	Factored Load, N_{ua} (lb)	1.2 x Nominal Strength, N_n (lb)	Ratio	
Steel	27900	53106	52.5%	

Concrete	Nominal Strength, N_n (lb)	Nominal Strength, N_n (lb)	Ratio	
Pullout	27900	127400	21.9%	
Side-face blowout	**27900**	**50478**	**55.3%**	**Governs**

ACI 318-14 Section 17.2.3.4.3(a) (i) & (ii) is not satisfied since steel ratio does not govern.

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12. Warnings

- Minimum spacing and edge distance requirement of 6da per ACI 318 Sections 17.7.1 and 17.7.2 for torqued cast-in-place anchor is waived per designer option.

- Concrete breakout strength in tension has not been evaluated against applied tension load(s) per designer option. Refer to ACI 318 Section 17.3.2.1 for conditions where calculations of the concrete breakout strength may not be required.

- Brittle failure governs for tension. Governing anchor failure mode is brittle failure. Attachment shall be designed to satisfy the requirements of ACI 318-14 Section 17.2.3.4.3 for structures assigned to Seismic Design Category C, D, E, or F when the component of the strength level earthquake force applied to anchors exceeds 20 percent of the total factored anchor force associated with the same load combination. In case when ACI 318-14 Sections 17.2.3.4.3 (a)(iii) to (vi), (b), (c) or (d) is satisfied for tension loading, select appropriate checkbox from Inputs tab to disable this message. Alternatively, Ω0 factor can be entered to satisfy ACI 318-14 Section 17.2.3.4.3(d) to increase the earthquake portion of the loads as required.

- Per designer input, the shear component of the strength-level earthquake force applied to anchors does not exceed 20 percent of the total factored anchor shear force associated with the same load combination. Therefore the ductility requirements of ACI 318 17.2.3.5.2 for shear need not be satisfied – designer to verify.

- Designer must exercise own judgement to determine if this design is suitable.

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1.Project information

Customer company:
Customer contact name:
Customer e-mail:
Comment:

Project description:
Location:
Fastening description:

1 1/4" DIA Anchor

2. Input Data & Anchor Parameters

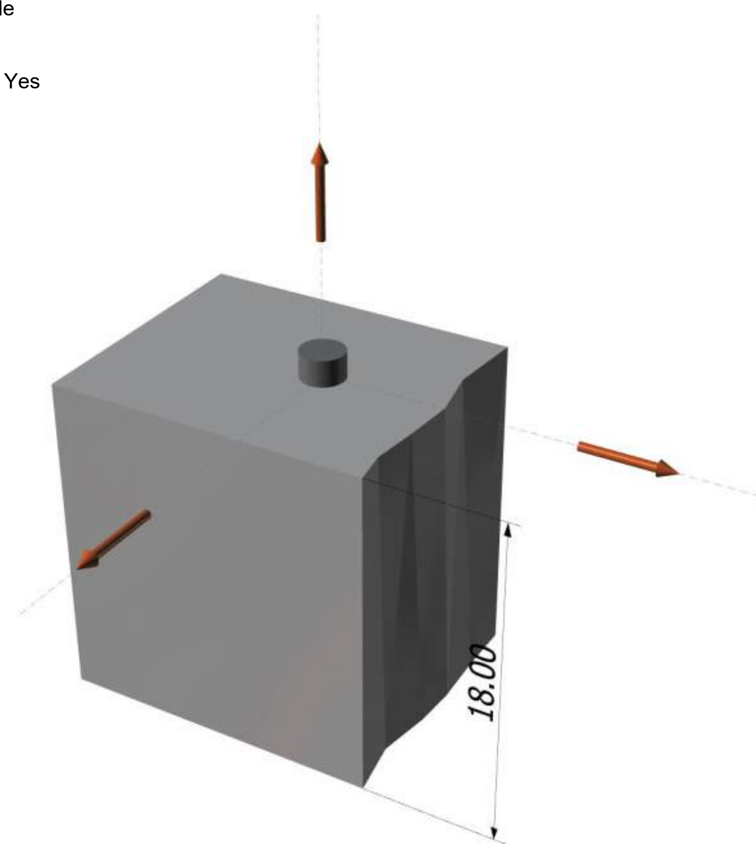
General
Design method:ACI 318-14
Units: Imperial units

Anchor Information:
Anchor type: Cast-in-place
Material: AB
Diameter (inch): 1.250
Effective Embedment depth, h_{ef} (inch): 15.000
Anchor category: -
Anchor ductility: Yes
h_{min} (inch): 18.00
C_{min} (inch): 2.25
S_{min} (inch): 5.00

Base Material
Concrete: Normal-weight
Concrete thickness, h (inch): 18.00
State: Cracked
Compressive strength, f'_c (psi): 2500
$\Psi_{c,V}$: 1.0
Reinforcement condition: A tension, A shear
Supplemental reinforcement: Not applicable
Reinforcement provided at corners: Yes
Ignore concrete breakout in tension: Yes
Ignore concrete breakout in shear: No
Ignore 6do requirement: Yes
Build-up grout pad: No

Load and Geometry
Load factor source: ACI 318 Section 5.3
Load combination: U = 0.9D + 1.0E
Seismic design: Yes
Anchors subjected to sustained tension: Not applicable
Ductility section for tension: 17.2.3.4.3 (a) (iii)-(vi) is satisfied
Ductility section for shear: 17.2.3.5.2 not applicable
Ω_0 factor: not set
Apply entire shear load at front row: No
Anchors only resisting wind and/or seismic loads: Yes

<Figure 1>



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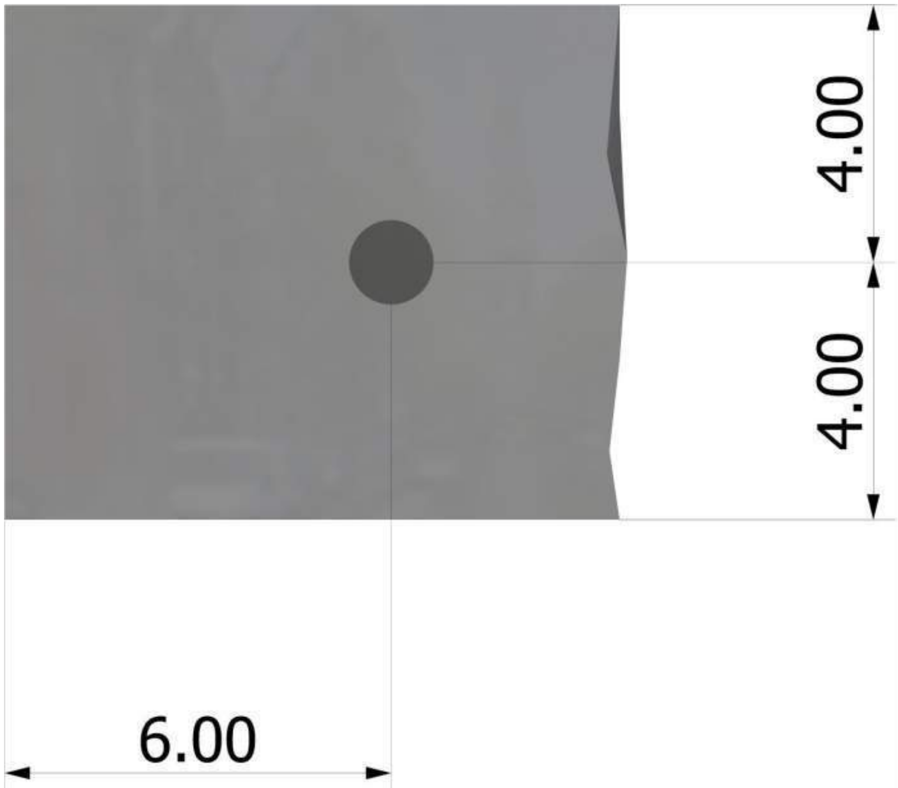
<Figure 2>



Recommended Anchor
Anchor Name: PAB Pre-Assembled Anchor Bolt - PAB10 (1 1/4"Ø)

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Anchor Designer™
Software
Version 2.5.6582.0

Company:	L120 Engineering & Design	Date:	1/14/2018
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Address:			
Phone:			
E-mail:			

3. Resulting Anchor Forces

Anchor	Tension load, N_{ua} (lb)	Shear load x, V_{uax} (lb)	Shear load y, V_{uay} (lb)	Shear load combined, $\sqrt{(V_{uax})^2+(V_{uay})^2}$ (lb)
1	31500.0	0.0	0.0	0.0
Sum	31500.0	0.0	0.0	0.0

Maximum concrete compression strain (‰): 0.00
Maximum concrete compression stress (psi): 0
Resultant tension force (lb): 0
Resultant compression force (lb): 0
Eccentricity of resultant tension forces in x-axis, e'_{Nx} (inch): 0.00
Eccentricity of resultant tension forces in y-axis, e'_{Ny} (inch): 0.00

4. Steel Strength of Anchor in Tension (Sec. 17.4.1)

N_{sa} (lb)	ϕ	ϕN_{sa} (lb)
56200	0.75	42150

6. Pullout Strength of Anchor in Tension (Sec. 17.4.3)

$0.75\phi N_{pn} = 0.75\phi \Psi_{c,P} N_p = 0.75\phi \Psi_{c,P} 8 A_{brg} f'_c$ (Sec. 17.3.1, Eq. 17.4.3.1 & 17.4.3.4)

$\Psi_{c,P}$	A_{brg} (in^2)	f'_c (psi)	ϕ	$0.75\phi N_{pn}$ (lb)
1.0	8.39	2500	0.70	88137

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Anchor Designer™
Software
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Company:	L120 Engineering & Design	Date:	1/14/2018
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Phone:			
E-mail:			

7. Side-Face Blowout Strength of Anchor in Tension (Sec. 17.4.4)

$0.75\phi N_{sb} = 0.75\phi\{(1+c_{a2}/c_{a1})/4\}(160c_{a1}\sqrt{A_{brg}})\lambda\sqrt{f'_c}$ (Sec. 17.3.1 & Eq. 17.4.4.1)

c_{a1} (in)	c_{a2} (in)	A_{brg} (in^2)	λ_a	f'_c (psi)	ϕ	$0.75\phi N_{sbg}$ (lb)
4.00	6.00	8.39	1.00	2500	0.75	32594

11. Results

11. Interaction of Tensile and Shear Forces (Sec. D.7)?

Tension	Factored Load, N_{ua} (lb)	Design Strength, $øN_n$ (lb)	Ratio	Status
Steel	31500	42150	0.75	Pass
Pullout	31500	88137	0.36	Pass
Side-face blowout	**31500**	**32594**	**0.97**	**Pass (Governs)**

PAB10 (1 1/4"Ø) with hef = 15.000 inch meets the selected design criteria.

ACI 318-14 Section 17.2.3.4.3(a) (i) & (ii) Calculations for Ductility requirement for tension load

Steel	Factored Load, N_{ua} (lb)	1.2 x Nominal Strength, N_n (lb)	Ratio	
Steel	31500	67440	46.7%	

Concrete	Nominal Strength, N_n (lb)	Nominal Strength, N_n (lb)	Ratio	
Pullout	31500	167880	18.8%	
Side-face blowout	**31500**	**57945**	**54.4%**	**Governs**

ACI 318-14 Section 17.2.3.4.3(a) (i) & (ii) is not satisfied since steel ratio does not govern.

12. Warnings

- Minimum spacing and edge distance requirement of 6da per ACI 318 Sections 17.7.1 and 17.7.2 for torqued cast-in-place anchor is waived per designer option.

- Concrete breakout strength in tension has not been evaluated against applied tension load(s) per designer option. Refer to ACI 318 Section 17.3.2.1 for conditions where calculations of the concrete breakout strength may not be required.

- Brittle failure governs for tension. Governing anchor failure mode is brittle failure. Attachment shall be designed to satisfy the requirements of ACI 318-14 Section 17.2.3.4.3 for structures assigned to Seismic Design Category C, D, E, or F when the component of the strength level earthquake force applied to anchors exceeds 20 percent of the total factored anchor force associated with the same load combination. In case when ACI 318-14 Sections 17.2.3.4.3 (a)(iii) to (vi), (b), (c) or (d) is satisfied for tension loading, select appropriate checkbox from Inputs tab to disable this message. Alternatively, Ω0 factor can be entered to satisfy ACI 318-14 Section 17.2.3.4.3(d) to increase the earthquake portion of the loads as required.

- Per designer input, the shear component of the strength-level earthquake force applied to anchors does not exceed 20 percent of the total factored anchor shear force associated with the same load combination. Therefore the ductility requirements of ACI 318 17.2.3.5.2 for shear need not be satisfied – designer to verify.

- Designer must exercise own judgement to determine if this design is suitable.

LONGITUDE
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Hand-rail Calculations

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PROJECT NO.	SHEET NO.

PROJECT

SUBJECT GuardRail Design

BY DATE / /

End Post Anchor Bolt Design:

Pv = 25 lbs

Ph = 200 lbs

h1 = 46"

h2 = 5.5"

e = 1.5"

Anchor Moment Mx = Pv(e) + Ph (h1 + h2/2)
= 25 x 1.5 + 200x (46+5.5/2)
= 9788 #"

My = 200# x 4.5" = 900 #"

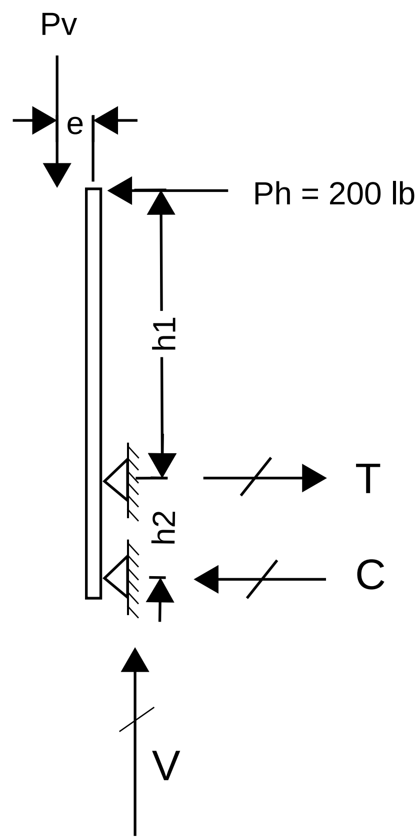
Anchor Forces T = [Pv (e) + Ph (h1+h2)] / h2 + My/1.5"
= 2480 #

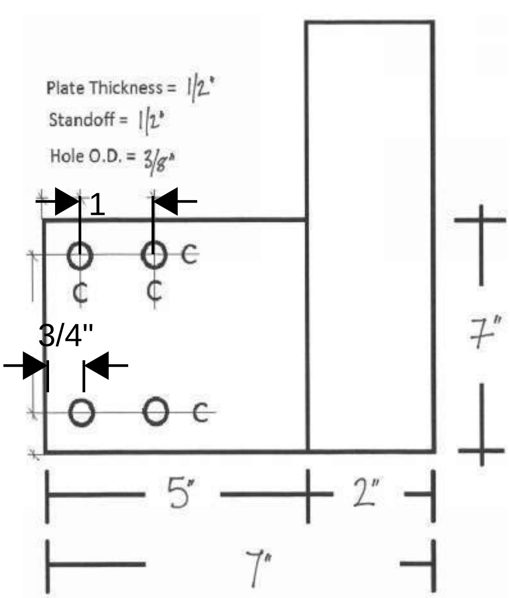
Anchor Forces C = T - Ph
= 2280 #

Each Bolt Force T = T / 2 = 1240 #
V = Pv / 4 + Pv x 4.5"/(4x2.85")= 16 #

Wood Lag Screw: 3/8" dia with 3" min. embed into DF beam.

Withdrawal Wa = 305 #/" x 1.6 x 3" = 1460 # > T O.K.
Shear Za = 180 # x 1.6 = 280 # O.K.





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PROJECT NO.	SHEET NO.

PROJECT

SUBJECT GuardRail Design

BY

DATE / /

Middle Post Anchor Bolt Design:

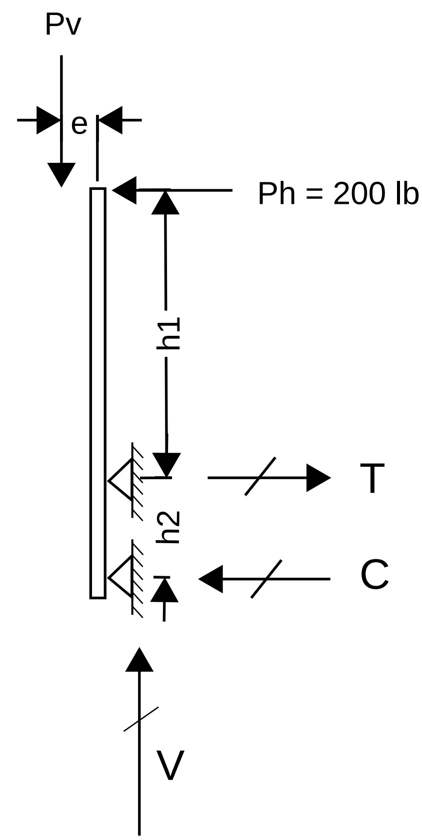
Pv = 25 lbs

Ph = 250 lbs

h1 = 46"

h2 = 5.5"

e = 1.5"

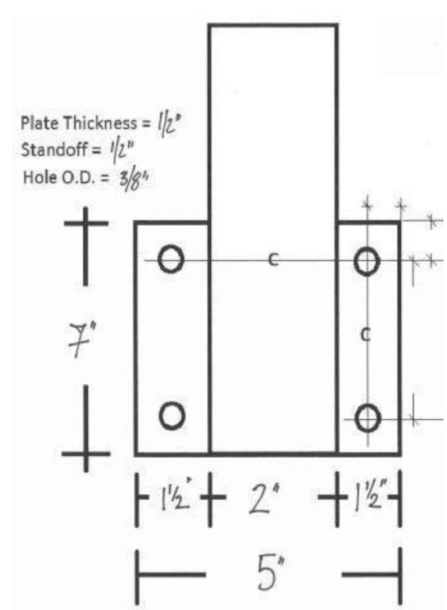


Anchor Moment M = Pv(e) + Ph (h1 + h2/2)
= 25 x 1.5 + 250 (46+5.5/2)
= 12,250

Anchor Forces T = [Pv (e) + Ph (h1+h2)] / h2
= 2347 #

Anchor Forces C = T - Ph
= 2147 #

Each Bolt Force T = T / 2 = 1174 #
V = Pv / 4 = 6 #



Wood Lag Screw: 3/8" dia with 3" min. embed into DF beam.

Withdrawal Wa = 305 #/" x 1.6 x 3" = 1460 # > T O.K.
Shear Za = 180 # x 1.6 = 280 # O.K.

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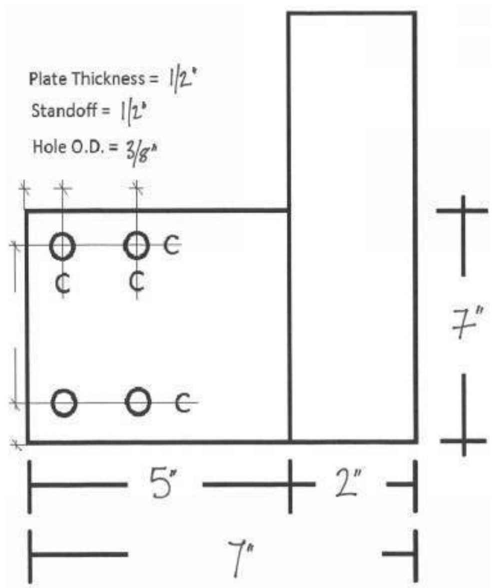
BY

DATE / /

Mounting Plate Design:

Apply Forces: Mx = 9788 #"
My = 900 #"
T = 200 #
V = 25 #



Try 1/2" thick Plate

Plate Bending Stress: fbx = Mx/2/Sx
= 9788/2/(1/4 x 5" x (1/2)^2)
= 15,660 psi
fby = My/Sy
= 900/(1/4 x 7" x (1/2)^2)
= 2,057 psi

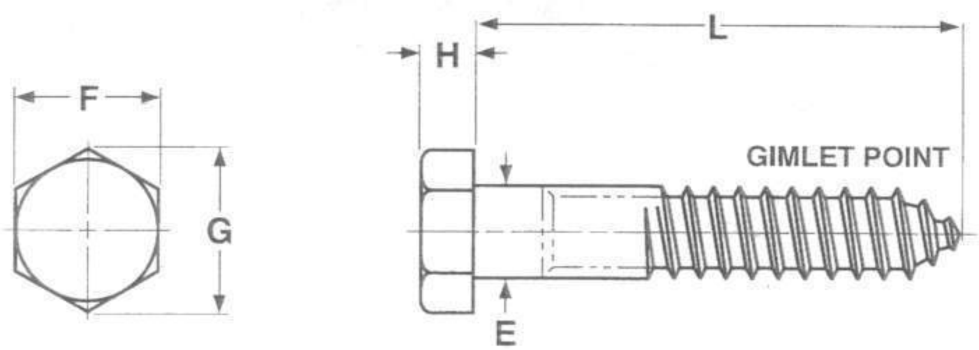
For Plate 6061-T6 Fb =35 ksi / 1.65
= 21,200 psi > fb O.K.

Plate Combined Stress
fbx/Fb + fby/Fb = 0.83 < 1.0 O.K.

Page 1 of 1	**Fastenal Product Standard**	REV-00
Date: January 11, 2012	FASTENAL	LAG.HDG

Hex Lag Screws, Hot Dipped Galvanized

The information below lists the required dimensional, chemical and physical characteristics of the products in this purchase order. If the order received does not meet these requirements, it may result in a supplier corrective action request, which could jeopardize your status as an approved vendor. Unless otherwise specified, all referenced consensus standards must be adhered to in their entirety.



Diameter	E		F		G		H	
	Body Diameter		Width Across Flats		Width Across Corners		Height	
	Max.	Min.	Max.	Min.	Max.	Min.	Max.	Min.
10	.199	.178	.281	.271	.323	.309	.140	.110
1/4	.260	.237	.438	.425	.505	.484	.188	.150
5/16	.324	.298	.500	.484	.577	.552	.235	.195
3/8	.388	.360	.562	.544	.650	.620	.268	.226
7/16	.452	.421	.625	.603	.722	.687	.316	.272
1/2	.515	.482	.750	.725	.866	.826	.364	.302
5/8	.642	.605	.938	.906	1.083	1.033	.444	.378
3/4	.768	.729	1.125	1.088	1.299	1.240	.524	.455
7/8	.895	.852	1.312	1.269	1.516	1.447	.604	.531
1	1.022	.976	1.500	1.450	1.732	1.653	.700	.591
1 1/8	1.149	1.098	1.688	1.631	1.949	1.859	.780	.658
1 1/4	1.277	1.223	1.875	1.812	2.165	2.066	.876	.749

Dimensions above are prior to coating

Specification Requirements:

- Dimensions: ASME B18.2.1.
- Material: Per ASTM A307, Grade A
- Thread requirements: The minimum thread length must be equal to one half the nominal Screw length plus ½", or 6 inch, whichever is shorter. Screws too short to conform to this formula must be threaded as close to the head as possible.
- Coating: Hot Dip Zinc per ASTM F2329 or in accordance with Class C of ASTM A153 and Class D for 3/8" diameter and less.

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PROJECT ____________________

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BY ____________________ DATE ___/___/___

Table 2.3.2 Frequently Used Load Duration Factors, C_D[1]

Load Duration	C_D	Typical Design Loads
Permanent	0.9	Dead Load
Ten years	1.0	Occupancy Live Load
Two months	1.15	Snow Load
Seven days	1.25	Construction Load
Ten minutes	1.6	Wind/Earthquake Load
Impact[2]	2.0	Impact Load

1. Load duration factors shall not apply to reference modulus of elasticity, E, reference modulus of elasticity for beam and column stability, E_{min}, nor to reference compression perpendicular to grain design values, $F_{c\perp}$, based on a deformation limit.
2. Load duration factors greater than 1.6 shall not apply to structural members pressure-treated with water-borne preservatives (see Reference 30), or fire retardant chemicals. The impact load duration factor shall not apply to connections.

2.3.3 Temperature Factor, C_t

Reference design values shall be multiplied by the temperature factors, C_t, in Table 2.3.3 for structural members that will experience sustained exposure to elevated temperatures up to 150°F (see Appendix C).

2.3.4 Fire Retardant Treatment

The effects of fire retardant chemical treatment on strength shall be accounted for in the design. Adjusted design values, including adjusted connection design values, for lumber and structural glued laminated timber pressure-treated with fire retardant chemicals shall be obtained from the company providing the treatment and redrying service. Load duration factors greater than 1.6 shall not apply to structural members pressure-treated with fire retardant chemicals (see Table 2.3.2).

2.3.5 Format Conversion Factor, K_F (LRFD Only)

For LRFD, reference design values shall be multiplied by the format conversion factor, K_F, specified in Table 2.3.5. The format conversion factor, K_F, shall not apply for designs in accordance with ASD methods specified herein.

2.3.6 Resistance Factor, ϕ (LRFD Only)

For LRFD, reference design values shall be multiplied by the resistance factor, ϕ, specified in Table 2.3.6. The resistance factor, ϕ, shall not apply for designs in accordance with ASD methods specified herein.

2.3.7 Time Effect Factor, λ (LRFD Only)

For LRFD, reference design values shall be multiplied by the time effect factor, λ, specified in Appendix N.3.3. The time effect factor, λ, shall not apply for designs in accordance with ASD methods specified herein.

Table 2.3.3 Temperature Factor, C_t

Reference Design Values	In-Service Moisture Conditions[1]	C_t		
		T≤100°F	100°F<T≤125°F	125°F<T≤150°F
F_t, E, E_{min}	Wet or Dry	1.0	0.9	0.9
F_b, F_v, F_c, and $F_{c\perp}$	Dry	1.0	0.8	0.7
	Wet	1.0	0.7	0.5

1. Wet and dry service conditions for sawn lumber, structural glued laminated timber, prefabricated wood I-joists, structural composite lumber, wood structural panels and cross-laminated timber are specified in 4.1.4, 5.1.4, 7.1.4, 8.1.4, 9.3.3, and 10.1.5 respectively.

LONGITUDE
ONE TWENTY°
ENGINEERING & DESIGN

PROJECT NO.	SHEET NO.

PROJECT ____________________

SUBJECT ____________________

BY ____________________ DATE ___/___/___

Table 11.3.1 Applicability of Adjustment Factors for Connections

		ASD Only	ASD and LRFD										LRFD Only		
		Load Duration Factor [1]	Wet Service Factor	Temperature Factor	Group Action Factor	Geometry Factor [3]	Penetration Depth Factor [3]	End Grain Factor [3]	Metal Side Plate Factor [3]	Diaphragm Factor [3]	Toe-Nail Factor [3]	Format Conversion Factor K_F	Resistance Factor ϕ	Time Effect Factor
Lateral Loads														
Dowel-type Fasteners (e.g. bolts, lag screws, wood screws, nails, spikes, drift bolts, & drift pins)	$Z' = Z$ x	C_D	C_M	C_t	C_g	C_Δ	-	C_{eg}	-	C_{di}	C_{tn}	3.32	0.65	λ
Split Ring and Shear Plate Connectors	$P' = P$ x	C_D	C_M	C_t	C_g	C_Δ	C_d	-	C_{st}	-	-	3.32	0.65	λ
	$Q' = Q$ x	C_D	C_M	C_t	C_g	C_Δ	C_d	-	-	-	-	3.32	0.65	λ
Timber Rivets	$P' = P$ x	C_D	C_M	C_t	-	-	-	-	C_{st} [4]	-	-	3.32	0.65	λ
	$Q' = Q$ x	C_D	C_M	C_t	-	C_Δ [5]	-	-	C_{st} [4]	-	-	3.32	0.65	λ
Spike Grids	$Z' = Z$ x	C_D	C_M	C_t	-	C_Δ	-	-	-	-	-	3.32	0.65	λ
Withdrawal Loads														
Nails, spikes, lag screws, wood screws, & drift pins	$W' = W$ x	C_D	C_M [2]	C_t	-	-	-	C_{eg}	-	-	C_{tn}	3.32	0.65	λ

1. The load duration factor, C_D, shall not exceed 1.6 for connections (see 11.3.2).
2. The wet service factor, C_M, shall not apply to toe-nails loaded in withdrawal (see 12.5.4.1).
3. Specific information concerning geometry factors C_Δ, penetration depth factors C_d, end grain factors, C_{eg}, metal side plate factors, C_{st}, diaphragm factors, C_{di}, and toe-nail factors, C_{tn}, is provided in Chapters 12, 13, and 14.
4. The metal side plate factor, C_{st}, is only applied when rivet capacity (P_r, Q_r) controls (see Chapter 14).
5. The geometry factor, C_Δ, is only applied when wood capacity, Q_w, controls (see Chapter 14).

11.3.2 Load Duration Factor, C_D (ASD Only)

Reference design values shall be multiplied by the load duration factors, $C_D \leq 1.6$, specified in 2.3.2 and Appendix B, except when the capacity of the connection is controlled by metal strength or strength of concrete/masonry (see 11.2.3, 11.2.4, and Appendix B.3). The impact load duration factor shall not apply to connections.

11.3.3 Wet Service Factor, C_M

Reference design values are for connections in wood seasoned to a moisture content of 19% or less and used under continuously dry conditions, as in most covered structures. For connections in wood that is unseasoned or partially seasoned, or when connections are exposed to wet service conditions in use, reference design values shall be multiplied by the wet service factors, C_M, specified in Table 11.3.3.

11.3.4 Temperature Factor, C_t

Reference design values shall be multiplied by the temperature factors, C_t, in Table 11.3.4 for connections that will experience sustained exposure to elevated temperatures up to 150°F (see Appendix C).

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Table 12.2A Lag Screw Reference Withdrawal Design Values, W[1]

Tabulated withdrawal design values (W) are in pounds per inch of thread penetration into side grain of wood member. Length of thread penetration in main member shall not include the length of the tapered tip (see 12.2.1.1).

Specific Gravity, G^2	Lag Screw Diameter, D										
	1/4"	5/16"	3/8"	7/16"	1/2"	5/8"	3/4"	7/8"	1"	1-1/8"	1-1/4"
0.73	397	469	538	604	668	789	905	1016	1123	1226	1327
0.71	381	450	516	579	640	757	868	974	1077	1176	1273
0.68	357	422	484	543	600	709	813	913	1009	1103	1193
0.67	349	413	473	531	587	694	796	893	987	1078	1167
0.58	281	332	381	428	473	559	641	719	795	869	940
0.55	260	307	352	395	437	516	592	664	734	802	868
0.51	232	274	314	353	390	461	528	593	656	716	775
0.50	225	266	305	342	378	447	513	576	636	695	752
0.49	218	258	296	332	367	434	498	559	617	674	730
0.47	205	242	278	312	345	408	467	525	580	634	686
0.46	199	235	269	302	334	395	453	508	562	613	664
0.44	186	220	252	283	312	369	423	475	525	574	621
0.43	179	212	243	273	302	357	409	459	508	554	600
0.42	173	205	235	264	291	344	395	443	490	535	579
0.41	167	198	226	254	281	332	381	428	473	516	559
0.40	161	190	218	245	271	320	367	412	455	497	538
0.39	155	183	210	236	261	308	353	397	438	479	518
0.38	149	176	202	227	251	296	340	381	422	461	498
0.37	143	169	194	218	241	285	326	367	405	443	479
0.36	137	163	186	209	231	273	313	352	389	425	460
0.35	132	156	179	200	222	262	300	337	373	407	441
0.31	110	130	149	167	185	218	250	281	311	339	367

1. Tabulated withdrawal design values, W, for lag screw connections shall be multiplied by all applicable adjustment factors (see Table 11.3.1).
2. Specific gravity, G, shall be determined in accordance with Table 12.3.3A.

12.2.3.2 For calculation of the fastener reference withdrawal design value in pounds, the unit reference withdrawal design value in lbs/in. of fastener penetration from 12.2.3.1 shall be multiplied by the length of fastener penetration, p_t, into the wood member.

12.2.3.3 The reference withdrawal design value, in lbs/in. of penetration, for a single post-frame ring shank nail driven in the side grain of the main member, with the nail axis perpendicular to the wood fibers, shall be determined from Table 12.2D or Equation 12.2-4, within the range of specific gravities and nail diameters given in Table 12.2D. Reference withdrawal design values, W, shall be multiplied by all applicable adjustment factors (see Table 11.3.1) to obtain adjusted withdrawal design values, W'.

$$W = 1800\, G^2 D \qquad (12.2\text{-}4)$$

12.2.3.4 For calculation of the fastener reference withdrawal design value in pounds, the unit reference withdrawal design value in lbs/in. of ring shank penetration from 12.2.3.3 shall be multiplied by the length of ring shank penetration, p_t, into the wood member.

12.2.3.5 Nails and spikes shall not be loaded in withdrawal from end grain of wood (C_{eg}=0.0).

12.2.3.6 Nails, and spikes shall not be loaded in withdrawal from end-grain of laminations in cross-laminated timber (C_{eg}=0.0).

12.2.4 Drift Bolts and Drift Pins

Reference withdrawal design values, W, for connections using drift bolt and drift pin connections shall be determined in accordance with 11.1.1.3.

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Table 12.3.3A Assigned Specific Gravities

Species Combination	Specific[1] Gravity, G	Species Combinations of MSR and MEL Lumber	Specific[1] Gravity, G
Alaska Cedar	0.47	Douglas Fir-Larch	
Alaska Hemlock	0.46	E=1,900,000 psi and lower grades of MSR	0.50
Alaska Spruce	0.41	E=2,000,000 psi grades of MSR	0.51
Alaska Yellow Cedar	0.46	E=2,100,000 psi grades of MSR	0.52
Aspen	0.39	E=2,200,000 psi grades of MSR	0.53
Balsam Fir	0.36	E=2,300,000 psi grades of MSR	0.54
Beech-Birch-Hickory	0.71	E=2,400,000 psi grades of MSR	0.55
Coast Sitka Spruce	0.39	Douglas Fir-Larch (North)	
Cottonwood	0.41	E=1,900,000 psi and lower grades of MSR and MEL	0.49
Douglas Fir-Larch	0.50	E=2,000,000 psi to 2,200,000 psi grades of MSR and MEL	0.53
Douglas Fir-Larch (North)	0.49	E=2,300,000 psi and higher grades of MSR and MEL	0.57
Douglas Fir-South	0.46	Douglas Fir-Larch (South)	
Eastern Hemlock	0.41	E=1,000,000 psi and higher grades of MSR	0.46
Eastern Hemlock-Balsam Fir	0.36	Engelmann Spruce-Lodgepole Pine	
Eastern Hemlock-Tamarack	0.41	E=1,400,000 psi and lower grades of MSR	0.38
Eastern Hemlock-Tamarack (North)	0.47	E=1,500,000 psi and higher grades of MSR	0.46
Eastern Softwoods	0.36	Hem-Fir	
Eastern Spruce	0.41	E=1,500,000 psi and lower grades of MSR	0.43
Eastern White Pine	0.36	E=1,600,000 psi grades of MSR	0.44
Engelmann Spruce-Lodgepole Pine	0.38	E=1,700,000 psi grades of MSR	0.45
Hem-Fir	0.43	E=1,800,000 psi grades of MSR	0.46
Hem-Fir (North)	0.46	E=1,900,000 psi grades of MSR	0.47
Mixed Maple	0.55	E=2,000,000 psi grades of MSR	0.48
Mixed Oak	0.68	E=2,100,000 psi grades of MSR	0.49
Mixed Southern Pine	0.51	E=2,200,000 psi grades of MSR	0.50
Mountain Hemlock	0.47	E=2,300,000 psi grades of MSR	0.51
Northern Pine	0.42	E=2,400,000 psi grades of MSR	0.52
Northern Red Oak	0.68	Hem-Fir (North)	
Northern Species	0.35	E=1,000,000 psi and higher grades of MSR and MEL	0.46
Northern White Cedar	0.31	Southern Pine	
Ponderosa Pine	0.43	E=1,700,000 psi and lower grades of MSR and MEL	0.55
Red Maple	0.58	E=1,800,000 psi and higher grades of MSR and MEL	0.57
Red Oak	0.67	Spruce-Pine-Fir	
Red Pine	0.44	E=1,700,000 psi and lower grades of MSR and MEL	0.42
Redwood, close grain	0.44	E=1,800,000 psi and 1,900,000 grades of MSR and MEL	0.46
Redwood, open grain	0.37	E=2,000,000 psi and higher grades of MSR and MEL	0.50
Sitka Spruce	0.43	Spruce-Pine-Fir (South)	
Southern Pine	0.55	E=1,100,000 psi and lower grades of MSR	0.36
Spruce-Pine-Fir	0.42	E=1,200,000 psi to1,900,000 psi grades of MSR	0.42
Spruce-Pine-Fir (South)	0.36	E=2,000,000 psi and higher grades of MSR	0.50
Western Cedars	0.36	Western Cedars	
Western Cedars (North)	0.35	E=1,000,000 psi and higher grades of MSR	0.36
Western Hemlock	0.47	Western Woods	
Western Hemlock (North)	0.46	E=1,000,000 psi and higher grades of MSR	0.36
Western White Pine	0.40		
Western Woods	0.36		
White Oak	0.73		
Yellow Poplar	0.43		

1. Specific gravity, G, based on weight and volume when oven-dry. Different specific gravities, G, are possible for different grades of MSR and MEL lumber (see Table 4C, Footnote 2).

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LAG SCREWS

Table 12K **LAG SCREWS: Reference Lateral Design Values, Z, for Single Shear (two member) Connections[1,2,3,4]**

for sawn lumber or SCL with ASTM A653, Grade 33 steel side plate (for t_s<1/4") or ASTM A 36 steel side plate (for t_s=1/4")
(tabulated lateral design values are calculated based on an assumed length of lag screw penetration, p, into the main member equal to 8D)

Side Member Thickness	Lag Screw Diameter	G=0.67 Red Oak		G=0.55 Mixed Maple Southern Pine		G=0.5 Douglas Fir-Larch		G=0.49 Douglas Fir-Larch (N)		G=0.46 Douglas Fir(S) Hem-Fir(N)		G=0.43 Hem-Fir		G=0.42 Spruce-Pine-Fir		G=0.37 Redwood (open grain)		G=0.36 Eastern Softwoods Spruce-Pine-Fir(S) Western Cedars Western Woods		G=0.35 Northern Species	
t_s	D	$Z_\parallel$	$Z_\perp$	$Z_\parallel$	$Z_\perp$	$Z_\parallel$	$Z_\perp$	$Z_\parallel$	$Z_\perp$	$Z_\parallel$	$Z_\perp$	$Z_\parallel$	$Z_\perp$	$Z_\parallel$	$Z_\perp$	$Z_\parallel$	$Z_\perp$	$Z_\parallel$	$Z_\perp$	$Z_\parallel$	$Z_\perp$
in.	in.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.
0.075	1/4	170	130	160	120	150	110	150	110	150	100	140	100	140	100	130	90	130	90	130	90
(14 gage)	5/16	220	160	200	140	190	130	190	130	190	130	180	120	180	120	170	110	170	110	160	100
	3/8	220	160	200	140	200	130	190	130	190	120	180	120	180	120	170	110	170	100	170	100
0.105	1/4	180	140	170	130	160	120	160	120	160	110	150	110	150	110	140	100	140	100	140	90
(12 gage)	5/16	230	170	210	150	200	140	200	140	190	130	190	130	190	120	180	110	170	110	170	110
	3/8	230	160	210	140	200	140	200	130	200	130	190	120	190	120	180	110	180	110	170	110
0.120	1/4	190	150	180	130	170	120	170	120	160	120	160	110	160	110	150	100	150	100	140	100
(11 gage)	5/16	230	170	210	150	210	140	200	140	200	140	190	130	190	130	180	120	180	120	180	110
	3/8	240	170	220	150	210	140	210	140	200	130	200	130	190	120	180	110	180	110	180	110
0.134	1/4	200	150	180	140	180	130	170	130	170	120	160	120	160	110	150	110	150	100	150	100
(10 gage)	5/16	240	180	220	160	210	150	210	140	200	140	200	130	200	130	190	120	180	120	180	120
	3/8	240	170	220	150	220	140	210	140	210	140	200	130	200	130	190	120	190	120	180	110
0.179	1/4	220	170	210	150	200	150	200	140	190	140	190	130	190	130	180	120	170	120	170	120
(7 gage)	5/16	260	190	240	170	230	160	230	160	230	150	220	150	220	150	210	130	200	130	200	130
	3/8	270	190	250	170	240	160	240	160	230	150	220	140	220	140	210	130	210	130	200	130
0.239	1/4	240	180	220	160	210	150	210	150	200	140	190	140	190	130	180	120	180	120	180	120
(3 gage)	5/16	300	220	280	190	270	180	260	180	260	170	250	160	250	160	230	150	230	150	230	140
	3/8	310	220	280	190	270	180	270	180	260	170	250	160	250	160	240	140	230	140	230	140
	7/16	420	290	390	260	380	240	370	240	360	230	350	220	350	220	330	200	330	200	320	190
	1/2	510	340	470	300	460	290	450	280	440	270	430	260	420	260	400	240	400	230	390	230
	5/8	770	490	710	430	680	400	680	400	660	380	640	370	630	360	600	330	590	330	580	320
	3/4	1110	670	1020	590	980	560	970	550	950	530	920	500	910	500	860	450	850	450	840	440
	7/8	1510	880	1390	780	1330	730	1320	710	1280	690	1250	650	1230	650	1170	590	1160	590	1140	570
	1	1940	1100	1780	960	1710	910	1700	890	1650	860	1600	820	1590	810	1500	740	1480	730	1460	710
1/4	1/4	240	180	220	160	210	150	210	150	200	140	200	140	190	130	180	120	180	120	180	120
	5/16	310	220	280	200	270	180	270	180	260	170	250	170	250	160	230	150	230	150	230	140
	3/8	320	220	290	190	280	180	270	180	270	170	260	160	250	160	240	150	240	140	230	140
	7/16	480	320	440	280	420	270	420	260	410	250	390	240	390	230	370	220	360	210	360	210
	1/2	580	390	540	340	520	320	510	320	500	310	480	290	480	290	460	270	450	260	440	260
	5/8	850	530	780	470	750	440	740	440	720	420	700	400	690	400	660	370	650	360	640	350
	3/4	1200	730	1100	640	1060	600	1050	590	1020	570	990	540	980	530	930	490	920	480	900	470
	7/8	1600	930	1470	820	1410	770	1400	750	1360	720	1320	690	1310	680	1240	630	1220	620	1200	600
	1	2040	1150	1870	1000	1800	950	1780	930	1730	900	1680	850	1660	840	1570	770	1550	760	1530	740

1. Tabulated lateral design values, Z, shall be multiplied by all applicable adjustment factors (see Table 11.3.1).
2. Tabulated lateral design values, Z, are for "reduced body diameter" lag screws (see Appendix Table L2) inserted in side grain with screw axis perpendicular to wood fibers; screw penetration, p, into the main member equal to 8D; dowel bearing strengths, F_e, of 61,850 psi for ASTM A653, Grade 33 steel and 87,000 psi for ASTM A36 steel and screw bending yield strengths, F_{yb}, of 70,000 psi for D = 1/4", 60,000 psi for D = 5/16", and 45,000 psi for D ≥3/8".
3. Where the lag screw penetration, p, is less than 8D but not less than 4D, tabulated lateral design values, Z, shall be multiplied by p/8D or lateral design values shall be calculated using the provisions of 12.3 for the reduced penetration.
4. The length of lag screw penetration, p, not including the length of the tapered tip, E (see Appendix Table L2), of the lag screw into the main member shall not be less than 4D. See 12.1.4.6 for minimum length of penetration, p_{min}.

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To determine the minimum required hand-rail connections, with a pre-manufactured hand-rail system provided by others. Our scope is limited to assess the minimum connection requirements of the hand-rail system as listed below. Our assumptions are that the base-plates, welds and metal member properties of the pre-manufactured complete system are sufficient in strength to support the code prescribed design loads, for which our design have been provided to comply with.

We have analyzed and verified the minimum connection requirements, for the following conditions:

- Wall connection (sloping wall @ stair)
  Result: minimum (2) ¼" DIA x 3" SDS screws to a minimum of (1) support studs at each connection

- Base-plate connection (vertical post application, typical)
  Result: The base-plate column connection to have a minimum of (4) 3/8" x 4 ½ lag-screws into full width support member/beams below

- *Wall connection (horizontal typical application)*
  Result: (2) ¼" DIA x 3" SDS screws to a minimum of (2) support studs at each connection

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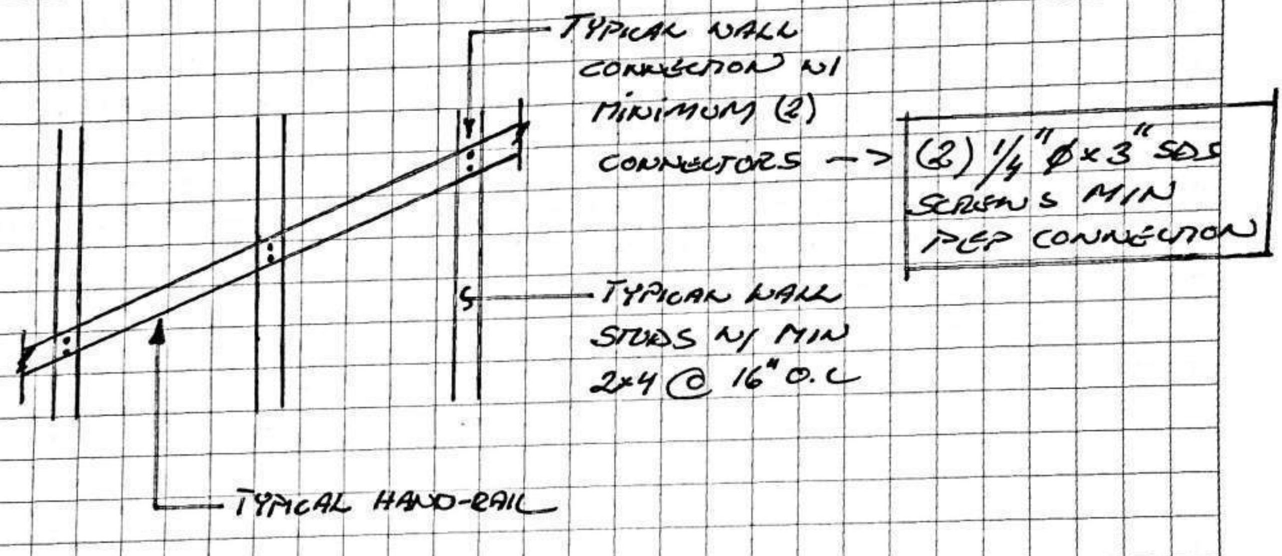
PROJECT

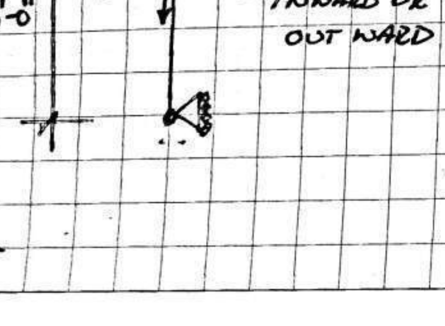
SUBJECT

BY MRT, P.E. DATE 12 / 4 / 2017

CALCULATIONS

CASE 1: SLOPING HAND-RAIL @ WALL / STAIR





SEE ATTACHED CALCULATION OF STUD MEMBER ANALYSIS

V = shear capacity ($C_t = C_i = 1.0$, $C_M = 1.0$, $C_g = 0.9$)

$V = C_D \times \underbrace{100\ Lbs}_{1/4"\phi\ w/\ 2x} = 1.6 \times 100\ Lbs \times 1.0 \times 0.9 \cong 160\ Lbs$

$V_{(2)\ 1/4"\phi}$ LAGS MIN INTO 2X HF #2 OR BETTER $= 2 \times 160 = 320\ LBS$

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200 Lbs demand < 320 Lbs CAPACITY ✓

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BY MRT, P.E. DATE 12 / 4 / 2017

CONT. CASE 1: SLOPING HAND-RAIL @ WALL/STAIR

W = WITHDRAWAL CAPACITY ($C_t = C_t = C_{eg} = 1.0$) = 179 Lbs/inch

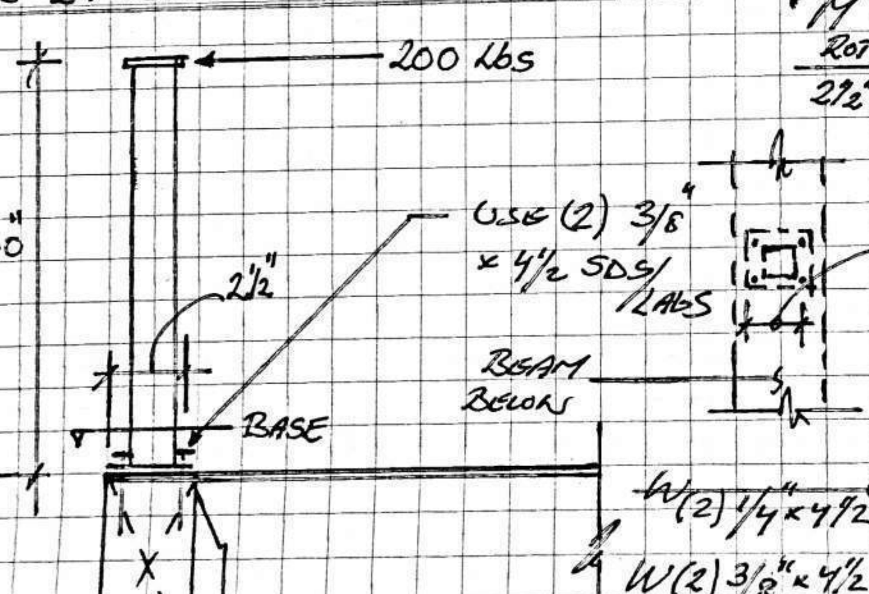
W_{cp} = 1.6x 440 Lbs per SCREW/LAG = 179 Lbs × 2½" ≅ 446 Lbs PER ¼"φ LAG×3"

$W_{(2)\ ¼"\ LAGS\ ×\ 3"\ min}$ = 2×440 Lbs×1.6 = 1,408 Lbs

200 Lbs WITHDRAWAL DEMAND < 1,408 Lbs CAPACITY ☑

CASE 2: BASE PLATE CONNECTION

$$M_{ROT} = \frac{200\ Lbs \times 36"}{2½"} = 2,880\ Lbs$$



WITHDRAWAL CAPACITY w/ ¼" × 4½ LAG-SCREWS = 179 Lbs/inch

W = 179 Lbs × 4" × 1.6 (2×¼"φ × 4½" LAGS) = 1145 Lbs per ¼"φ LAG×4½"

"NOT WORKING"

~~$W_{(2)\ ¼"×4½}$ φ × 2 = 1,145 Lbs~~

$W_{(2)\ 3/8"×4½}$ × 2 = 243 × 4" × 2 × 1.6 = 3,110

2,880 Lbs demand < 3,110 CAPACITY

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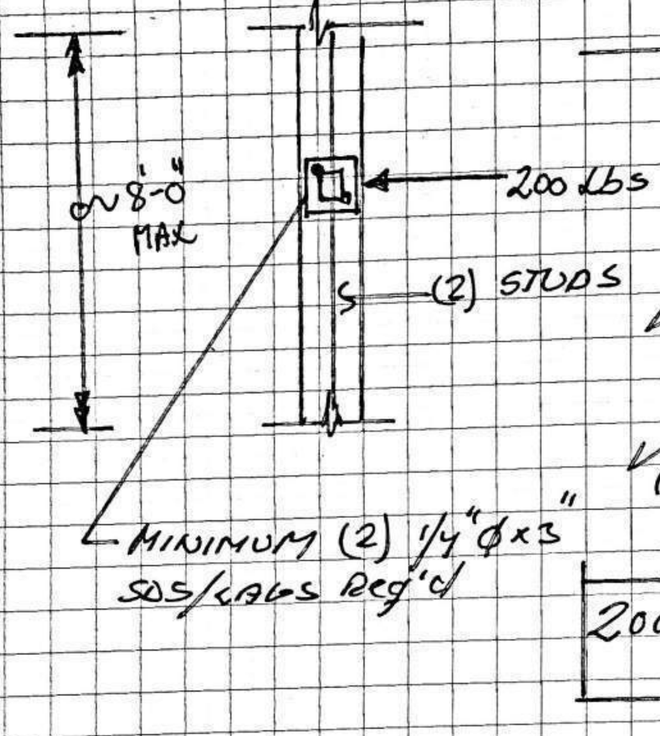
BY MRT, P.E

DATE 12/4/2014

CASE 3: HORIZONTAL END-PLATE CONNECTIONS



→ SEE ATTACHED CALCULATIONS OF STUD CALCULATIONS.

V = SHEAR CAPACITY ($C_t = C_i = C_M = 1.0$, $C_g = 0.9$)

$$V = C_D \times \underbrace{100\ Lbs}_{1/4"\phi\ w/2x} = 1.6 \times 0.9 \times 100\ Lbs = 144\ Lbs$$

$$V_{(2)\ 1/4"\phi \times 3"\ LAG\text{-}SCREWS} = 2 \times 144\ Lbs = 288\ Lbs$$

$$\boxed{200\ Lbs\ demand < 288\ Lbs\ CAPACITY}$$ ☑

SIMPSON Strong-Tie®

Anchor Designer™
Software
Version 2.5.6582.0

Company:	L120 Engineering & Design	Date:	5/3/2018
Engineer:	MRT	Page:	1/5
Project:	Hand-rail calculation		
Address:			
Phone:			
E-mail:			

1.Project information

Customer company:
Customer contact name:
Customer e-mail:
Comment:

Project description:
Location:
Fastening description:

2. Input Data & Anchor Parameters

General
Design method:ACI 318-14
Units: Imperial units

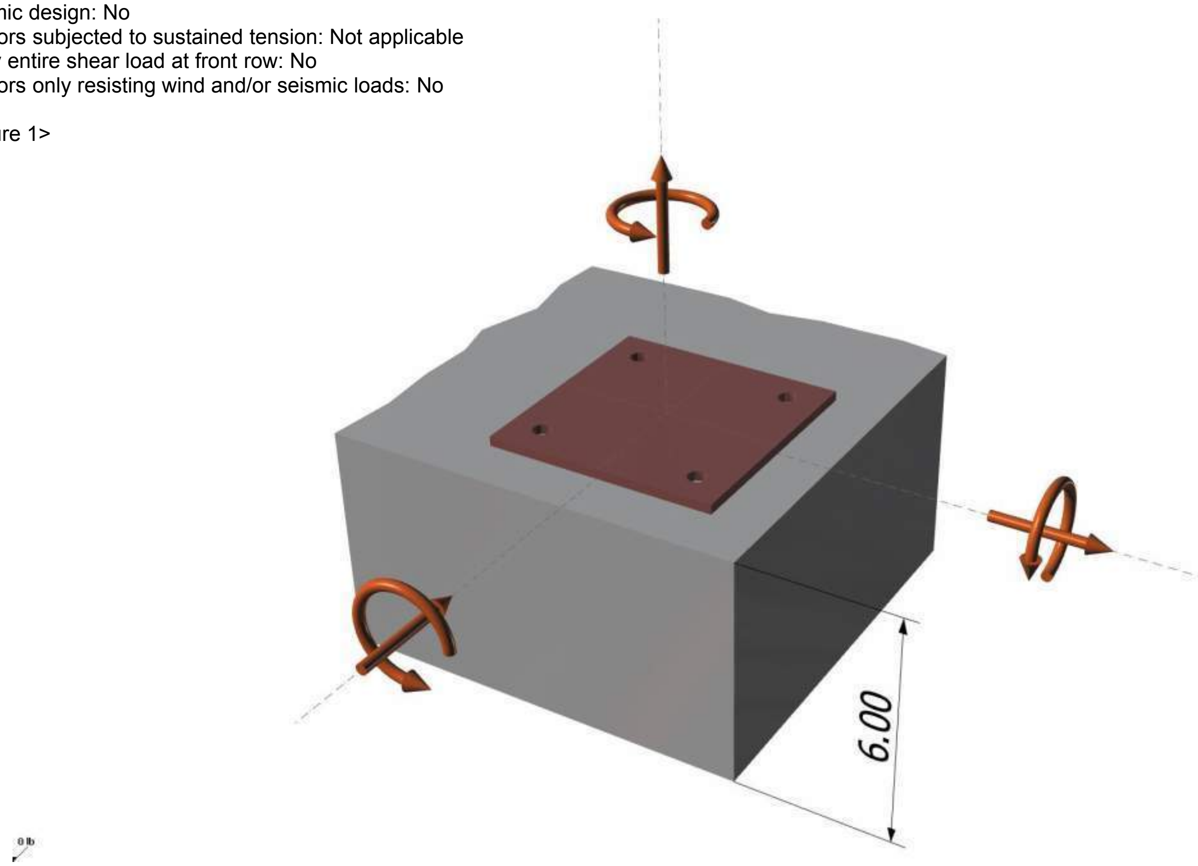
Anchor Information:
Anchor type: Concrete screw
Material: Carbon Steel
Diameter (inch): 0.375
Nominal Embedment depth (inch): 3.250
Effective Embedment depth, h_{ef} (inch): 2.400
Code report: ICC-ES ESR-2713
Anchor category: 1
Anchor ductility: No
h_{min} (inch): 5.00
c_{ac} (inch): 3.63
C_{min} (inch): 1.75
S_{min} (inch): 3.00

Base Material
Concrete: Normal-weight
Concrete thickness, h (inch): 6.00
State: Cracked
Compressive strength, f'_c (psi): 2500
$\Psi_{c,V}$: 1.0
Reinforcement condition: B tension, B shear
Supplemental reinforcement: Not applicable
Reinforcement provided at corners: No
Ignore concrete breakout in tension: No
Ignore concrete breakout in shear: No
Ignore 6do requirement: Not applicable
Build-up grout pad: No

Base Plate
Length x Width x Thickness (inch): 6.00 x 6.00 x 0.25

Load and Geometry
Load factor source: ACI 318 Section 5.3
Load combination: U = 1.2(D + F) + 1.6(L) + 0.5(Lr or S or R)
Seismic design: No
Anchors subjected to sustained tension: Not applicable
Apply entire shear load at front row: No
Anchors only resisting wind and/or seismic loads: No

<Figure 1>

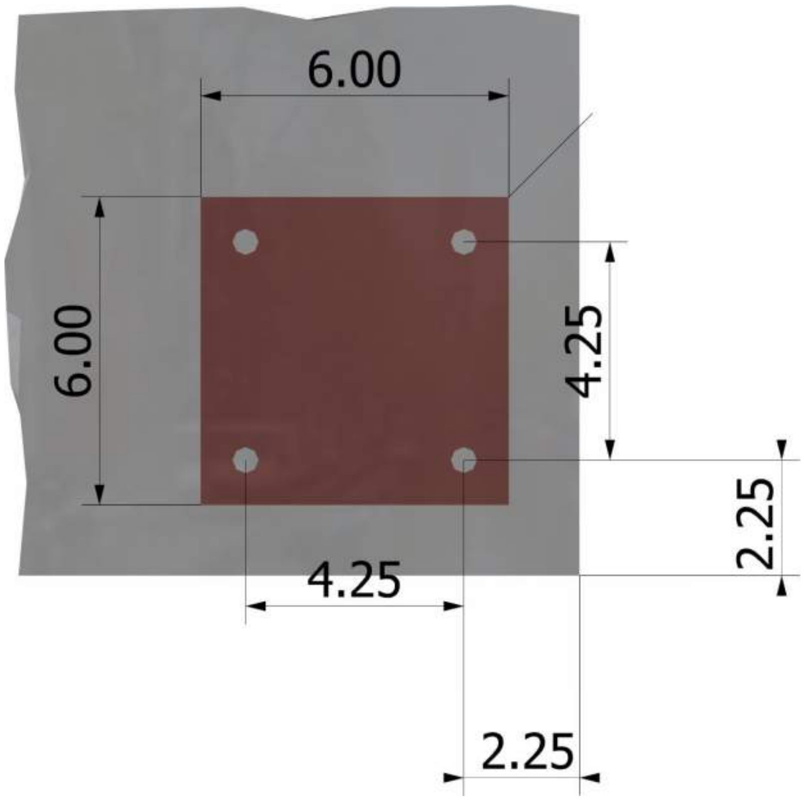


SIMPSON Strong-Tie®

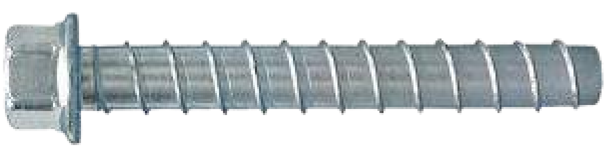
Anchor Designer™
Software
Version 2.5.6582.0

Company:	L120 Engineering & Design	Date:	5/3/2018
Engineer:	MRT	Page:	2/5
Project:	Hand-rail calculation		
Address:			
Phone:			
E-mail:			

<Figure 2>



Recommended Anchor
Anchor Name: Titen HD® - 3/8"Ø Titen HD, hnom:3.25" (83mm)
Code Report: ICC-ES ESR-2713

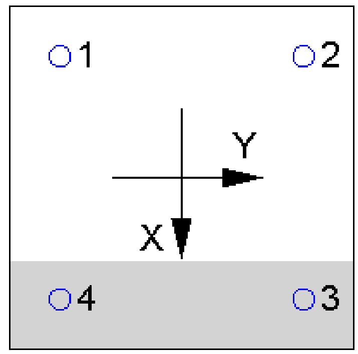


3. Resulting Anchor Forces

Anchor	Tension load, N_{ua} (lb)	Shear load x, V_{uax} (lb)	Shear load y, V_{uay} (lb)	Shear load combined, $\sqrt{(V_{uax})^2+(V_{uay})^2}$ (lb)
1	1250.4	-80.0	0.0	80.0
2	1250.4	-80.0	0.0	80.0
3	0.0	-80.0	0.0	80.0
4	0.0	-80.0	0.0	80.0
Sum	2500.7	-320.0	0.0	320.0

Maximum concrete compression strain (‰): 0.12
Maximum concrete compression stress (psi): 538
Resultant tension force (lb): 2501
Resultant compression force (lb): 2501
Eccentricity of resultant tension forces in x-axis, e'_{Nx} (inch): 0.00
Eccentricity of resultant tension forces in y-axis, e'_{Ny} (inch): 0.00
Eccentricity of resultant shear forces in x-axis, e'_{Vx} (inch): 0.00
Eccentricity of resultant shear forces in y-axis, e'_{Vy} (inch): 0.00

<Figure 3>



4. Steel Strength of Anchor in Tension (Sec. 17.4.1)

N_{sa} (lb)	ϕ	ϕN_{sa} (lb)
10890	0.65	7079

5. Concrete Breakout Strength of Anchor in Tension (Sec. 17.4.2)

$N_b = k_c \lambda_a \sqrt{f'_c} h_{ef}^{1.5}$ (Eq. 17.4.2.2a)

k_c	λ_a	f'_c (psi)	h_{ef} (in)	N_b (lb)
17.0	1.00	2500	2.400	3160

$\phi N_{cbg} = \phi\,(A_{Nc} / A_{Nco})\,\Psi_{ec,N}\,\Psi_{ed,N}\,\Psi_{c,N}\,\Psi_{cp,N} N_b$ (Sec. 17.3.1 & Eq. 17.4.2.1b)

A_{Nc} (in^2)	A_{Nco} (in^2)	$c_{a,min}$ (in)	$\Psi_{ec,N}$	$\Psi_{ed,N}$	$\Psi_{c,N}$	$\Psi_{cp,N}$	N_b (lb)	ϕ	ϕN_{cbg} (lb)
72.72	51.84	2.25	1.000	0.888	1.00	1.000	3160	0.65	2557

6. Pullout Strength of Anchor in Tension (Sec. 17.4.3)

$\phi N_{pn} = \phi \Psi_{c,P} \lambda_a N_p (f'_c / 2{,}500)^n$ (Sec. 17.3.1, Eq. 17.4.3.1 & Code Report)

$\Psi_{c,P}$	λ_a	N_p (lb)	f'_c (psi)	n	ϕ	ϕN_{pn} (lb)
1.0	1.00	2700	2500	0.50	0.65	1755

SIMPSON Strong-Tie®

Anchor Designer™ Software
Version 2.5.6582.0

Company:	L120 Engineering & Design	Date:	5/3/2018
Engineer:	MRT	Page:	4/5
Project:	Hand-rail calculation		
Address:			
Phone:			
E-mail:			

8. Steel Strength of Anchor in Shear (Sec. 17.5.1)

V_{sa} (lb)	ϕ_{grout}	ϕ	$\phi_{grout}\phi V_{sa}$ (lb)
4460	1.0	0.60	2676

9. Concrete Breakout Strength of Anchor in Shear (Sec. 17.5.2)

Shear parallel to edge in x-direction:

$V_{by} = \min|7(l_e / d_a)^{0.2}\sqrt{d_a}\lambda_a\sqrt{f'_c}c_{a1}^{1.5}; 9\lambda_a\sqrt{f'_c}c_{a1}^{1.5}|$ (Eq. 17.5.2.2a & Eq. 17.5.2.2b)

l_e (in)	d_a (in)	λ_a	f'_c (psi)	c_{a1} (in)	V_{by} (lb)
2.40	0.375	1.00	2500	2.25	1049

$\phi V_{cbgx} = \phi\ (2)(A_{Vc} / A_{Vco})\Psi_{ec,V}\Psi_{ed,V}\Psi_{c,V}\Psi_{h,V}V_{by}$ (Sec. 17.3.1, 17.5.2.1(c) & Eq. 17.5.2.1b)

A_{Vc} (in^2)	A_{Vco} (in^2)	$\Psi_{ec,V}$	$\Psi_{ed,V}$	$\Psi_{c,V}$	$\Psi_{h,V}$	V_{by} (lb)	ϕ	ϕV_{cbgx} (lb)
33.33	22.78	1.000	1.000	1.000	1.000	1049	0.70	2148

10. Concrete Pryout Strength of Anchor in Shear (Sec. 17.5.3)

$\phi V_{cpg} = \phi k_{cp}N_{cbg} = \phi k_{cp}(A_{Nc} / A_{Nco})\Psi_{ec,N}\Psi_{ed,N}\Psi_{c,N}\Psi_{cp,N}N_b$ (Sec. 17.3.1 & Eq. 17.5.3.1b)

k_{cp}	A_{Nc} (in^2)	A_{Nco} (in^2)	$\Psi_{ec,N}$	$\Psi_{ed,N}$	$\Psi_{c,N}$	$\Psi_{cp,N}$	N_b (lb)	ϕ	ϕV_{cpg} (lb)
1.0	102.01	51.84	1.000	0.888	1.000	1.000	3160	0.70	3863

11. Results

Interaction of Tensile and Shear Forces (Sec. 17.6.)

Tension	Factored Load, N_{ua} (lb)	Design Strength, $øN_n$ (lb)	Ratio	Status
Steel	1250	7079	0.18	Pass
Concrete breakout	**2501**	**2557**	**0.98**	**Pass (Governs)**
Pullout	1250	1755	0.71	Pass

Shear	Factored Load, V_{ua} (lb)	Design Strength, $øV_n$ (lb)	Ratio	Status
Steel	80	2676	0.03	Pass
|| Concrete breakout y+	160	2148	0.07	Pass
Pryout	**320**	**3863**	**0.08**	**Pass (Governs)**

Interaction check	$N_{ua}/\phi N_n$	$V_{ua}/\phi V_n$	Combined Ratio	Permissible	Status
Sec. 17.6..1	0.98	0.00	97.8 %	1.0	Pass

3/8"Ø Titen HD, hnom:3.25" (83mm) meets the selected design criteria.

Anchor Designer™
Software
Version 2.5.6582.0

Company:	L120 Engineering & Design	Date:	5/3/2018
Engineer:	MRT	Page:	5/5
Project:	Hand-rail calculation		
Address:			
Phone:			
E-mail:			

12. Warnings

- Minimum spacing and edge distance requirement of 6da per ACI 318 Sections 17.7.1 and 17.7.2 for torqued cast-in-place anchor is waived per designer option.

- Designer must exercise own judgement to determine if this design is suitable.

- Refer to manufacturer's product literature for hole cleaning and installation instructions.

Project:

Location: Single 2x4 stud (staircase)
Multi-Loaded Multi-Span Beam
[2015 International Building Code(2015 NDS)]
1.5 IN x 3.5 IN x 8.0 FT
#2 - Hem-Fir - Dry Use
Section Adequate By: 0.8%
Controlling Factor: Deflection

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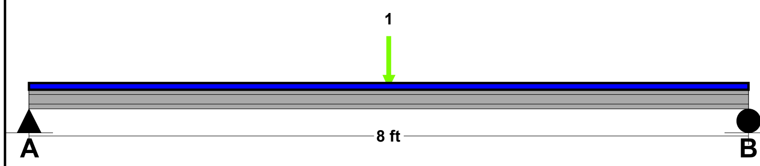
DEFLECTIONS	Center	
Live Load	0.53	IN L/181
Dead Load	0.01	in
Total Load	0.54	IN L/177

Live Load Deflection Criteria: L/180 Total Load Deflection Criteria: L/120

REACTIONS	A		B	
Live Load	100	lb	100	lb
Dead Load	4	lb	4	lb
Total Load	104	lb	104	lb
Bearing Length	0.17	in	0.17	in

BEAM DATA	Center	
Span Length	8	ft
Unbraced Length-Top	0	ft
Unbraced Length-Bottom	8	ft
Live Load Duration Factor	1.60	
Notch Depth	0.00	

LOADING DIAGRAM



UNIFORM LOADS	Center	
Uniform Live Load	0	plf
Uniform Dead Load	0	plf
Beam Self Weight	1	plf
Total Uniform Load	1	plf

POINT LOADS - CENTER SPAN	
Load Number	One
Live Load	200 lb
Dead Load	0 lb
Location	4 ft

MATERIAL PROPERTIES
#2 - Hem-Fir

	Base Values			Adjusted		
Bending Stress:	Fb =	850	psi	Fb' =	2040	psi
	Cd=1.60 CF=1.50					
Shear Stress:	Fv =	150	psi	Fv' =	240	psi
	Cd=1.60					
Modulus of Elasticity:	E =	1300	ksi	E' =	1300	ksi
Comp. ⊥ to Grain:	Fc - ⊥ =	405	psi	Fc - ⊥' =	405	psi

Controlling Moment: 408 ft-lb
4.0 Ft from left support of span 2 (Center Span)
Created by combining all dead loads and live loads on span(s) 2
Controlling Shear: -104 lb
At right support of span 2 (Center Span)
Created by combining all dead loads and live loads on span(s) 2

Comparisons with required sections:	Req'd	Provided
Section Modulus:	2.4 in3	3.06 in3
Area (Shear):	0.65 in2	5.25 in2
Moment of Inertia (deflection):	5.32 in4	5.36 in4
Moment:	408 ft-lb	521 ft-lb
Shear:	-104 lb	840 lb

Project:

Location: Single 2x6 stud (staircase)
Multi-Loaded Multi-Span Beam
[2015 International Building Code(2015 NDS)]
1.5 IN x 5.5 IN x 9.0 FT
#2 - Hem-Fir - Dry Use
Section Adequate By: 139.3%
Controlling Factor: Moment

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DEFLECTIONS	Center	
Live Load	0.19	IN L/556
Dead Load	0.01	in
Total Load	0.20	IN L/533

Live Load Deflection Criteria: L/180   Total Load Deflection Criteria: L/120

REACTIONS	A		B	
Live Load	100	lb	100	lb
Dead Load	7	lb	7	lb
Total Load	107	lb	107	lb
Bearing Length	0.18	in	0.18	in

BEAM DATA	Center	
Span Length	9	ft
Unbraced Length-Top	0	ft
Unbraced Length-Bottom	9	ft
Live Load Duration Factor	1.60	
Notch Depth	0.00	

MATERIAL PROPERTIES
#2 - Hem-Fir

	Base Values			Adjusted		
Bending Stress:	Fb =	850	psi	Fb' =	1768	psi
	Cd=1.60 CF=1.30					
Shear Stress:	Fv =	150	psi	Fv' =	240	psi
	Cd=1.60					
Modulus of Elasticity:	E =	1300	ksi	E' =	1300	ksi
Comp. ⊥ to Grain:	Fc - ⊥ =	405	psi	Fc - ⊥' =	405	psi

Controlling Moment: 466 ft-lb
4.5 Ft from left support of span 2 (Center Span)
Created by combining all dead loads and live loads on span(s) 2
Controlling Shear: -107 lb
At right support of span 2 (Center Span)
Created by combining all dead loads and live loads on span(s) 2

Comparisons with required sections:	Req'd	Provided
Section Modulus:	3.16 in3	7.56 in3
Area (Shear):	0.67 in2	8.25 in2
Moment of Inertia (deflection):	6.73 in4	20.8 in4
Moment:	466 ft-lb	1114 ft-lb
Shear:	-107 lb	1320 lb

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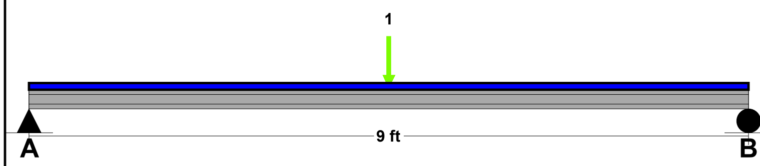

UNIFORM LOADS	Center	
Uniform Live Load	0	plf
Uniform Dead Load	0	plf
Beam Self Weight	2	plf
Total Uniform Load	2	plf

POINT LOADS - CENTER SPAN	
Load Number	One
Live Load	200 lb
Dead Load	0 lb
Location	4.5 ft

Project:

Location: Double 2x4 stud (flat orientation connection/top)
Multi-Loaded Multi-Span Beam
[2015 International Building Code(2015 NDS)]
(2) 1.5 IN x 3.5 IN x 8.0 FT
#2 - Hem-Fir - Dry Use
Section Adequate By: 101.6%
Controlling Factor: Deflection

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DEFLECTIONS	Center	
Live Load	0.26	IN L/363
Dead Load	0.01	in
Total Load	0.28	IN L/346

Live Load Deflection Criteria: L/180 Total Load Deflection Criteria: L/120

REACTIONS	A		B	
Live Load	100	lb	100	lb
Dead Load	8	lb	8	lb
Total Load	108	lb	108	lb
Bearing Length	0.09	in	0.09	in

BEAM DATA	Center	
Span Length	8	ft
Unbraced Length-Top	0	ft
Unbraced Length-Bottom	8	ft
Live Load Duration Factor	1.60	
Notch Depth	0.00	

MATERIAL PROPERTIES
#2 - Hem-Fir

	Base Values			Adjusted		
Bending Stress:	Fb =	850	psi	Fb' =	2040	psi
	Cd=1.60 CF=1.50					
Shear Stress:	Fv =	150	psi	Fv' =	240	psi
	Cd=1.60					
Modulus of Elasticity:	E =	1300	ksi	E' =	1300	ksi
Comp. ⊥ to Grain:	Fc - ⊥ =	405	psi	Fc - ⊥' =	405	psi

Controlling Moment: 416 ft-lb
4.0 Ft from left support of span 2 (Center Span)
Created by combining all dead loads and live loads on span(s) 2
Controlling Shear: 108 lb
At left support of span 2 (Center Span)
Created by combining all dead loads and live loads on span(s) 2

Comparisons with required sections:	Req'd	Provided
Section Modulus:	2.45 in3	6.13 in3
Area (Shear):	0.67 in2	10.5 in2
Moment of Inertia (deflection):	5.32 in4	10.72 in4
Moment:	416 ft-lb	1041 ft-lb
Shear:	108 lb	1680 lb

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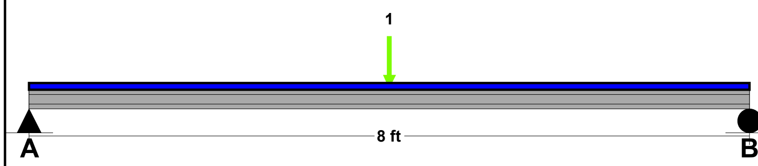

UNIFORM LOADS	Center	
Uniform Live Load	0	plf
Uniform Dead Load	0	plf
Beam Self Weight	2	plf
Total Uniform Load	2	plf

POINT LOADS - CENTER SPAN

Load Number	One
Live Load	200 lb
Dead Load	0 lb
Location	4 ft

LONGITUDE
ONE TWENTY°
ENGINEERING & DESIGN

Balloon Framed stud calculations

Project:

Location: Baloon Framed Stud Design (typical wind) - SS Column
[2015 International Building Code(2015 NDS)]
1.5 IN x 5.5 IN x 17.25 FT @ 12 O.C.
#2 - Hem-Fir - Dry Use
Section Adequate By: 4.0%

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DEFLECTIONS

Deflection due to lateral loads only:	Defl =	1.11	IN = L/187
Live Load Deflection Criteria:			L/180

VERTICAL REACTIONS

Live Load:	Vert-LL-Rxn =	500	lb
Dead Load:	Vert-DL-Rxn =	327	lb
Total Load:	Vert-TL-Rxn =	827	lb

HORIZONTAL REACTIONS

Total Reaction at Top of Column:	TL-Rxn-Top =	129	lb
Total Reaction at Bottom of Column:	TL-Rxn-Bottom =	129	lb

COLUMN DATA

Total Column Length:	17.25	ft
Unbraced Length (X-Axis) Lx:	17.25	ft
Unbraced Length (Y-Axis) Ly:	0	ft
Column End Condition-K (e):	1	
Axial Load Duration Factor	1.00	
Lateral Load Duration Factor (Wind/Seismic)	1.60	

STUD PROPERTIES

#2 - Hem-Fir

	Base Values	Adjusted
Compressive Stress:	Fc = 1300 psi	Fc' = 266 psi
	Cd=1.60 Cf=1.10 Cp=0.12	
Bending Stress (X-X Axis):	Fbx = 850 psi	Fbx' = 2033 psi
	Cd=1.60 CF=1.30 Cr=1.15 Cl=1.00	
Bending Stress (Y-Y Axis):	Fby = 850 psi	Fby' = 2033 psi
	Cd=1.60 CF=1.30 Cr=1.15	
Modulus of Elasticity:	E = 1300 ksi	E' = 1300 ksi

Stud Section (X-X Axis):	dx =	5.5	in
Stud Section (Y-Y Axis):	dy =	1.5	in
Area:	A =	8.25	in2
Section Modulus (X-X Axis):	Sx =	7.56	in3
Section Modulus (Y-Y Axis):	Sy =	2.06	in3
Slenderness Ratio:	Lex/dx =	37.64	
	Ley/dy =	0	

Stud Calculations (Controlling Case Only):

Controlling Load Case: Axial total Load and Lateral loads (D + 0.75[L + W]

Actual Compressive Stress:	Fc =	85	psi
Allowable Compressive Stress:	Fc' =	266	psi
Eccentricity Moment (X-X Axis):	Mx-ex =	0	ft-lb
Eccentricity Moment (Y-Y Axis):	My-ey =	0	ft-lb
Moment Due to Lateral Loads (X-X Axis):	Mx =	418	ft-lb
Moment Due to Lateral Loads (Y-Y Axis):	My =	0	ft-lb
Bending Stress Lateral Loads Only (X-X Axis):	Fbx =	664	psi
Allowable Bending Stress (X-X Axis):	Fbx' =	2033	psi
Bending Stress Lateral Loads Only (Y-Y Axis):	Fby =	0	psi
Allowable Bending Stress (Y-Y Axis):	Fby' =	2033	psi
Combined Stress Factor:	**CSF =**	**0.58**	

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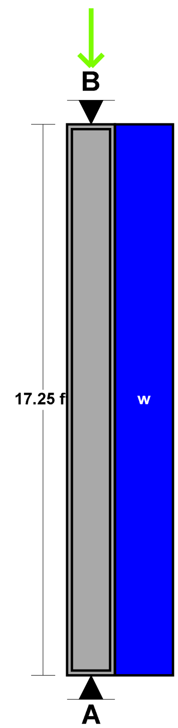

AXIAL LOADING

Live Load:	PL =	500 plf
Dead Load:	PD =	300 plf
Column Self Weight:	CSW =	27 plf
Total Axial Load:	PT =	827 plf

LATERAL LOADING (Dy Face)

Uniform Lateral Load:	wL-Lat =	15 psf

Project:

Location: Baloon Framed Stud Design (High Wind) - SS Column
[2015 International Building Code(2015 NDS)]
1.5 IN x 5.5 IN x 17.25 FT @ 8 O.C.
#2 - Hem-Fir - Dry Use
Section Adequate By: 6.4%

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DEFLECTIONS

Deflection due to lateral loads only:	Defl = 1.08 IN =	L/192
Live Load Deflection Criteria:		L/180

VERTICAL REACTIONS

Live Load:	Vert-LL-Rxn =	333	lb
Dead Load:	Vert-DL-Rxn =	227	lb
Total Load:	Vert-TL-Rxn =	560	lb

HORIZONTAL REACTIONS

Total Reaction at Top of Column:	TL-Rxn-Top =	127	lb
Total Reaction at Bottom of Column:	TL-Rxn-Bottom =	127	lb

COLUMN DATA

Total Column Length:	17.25	ft
Unbraced Length (X-Axis) Lx:	17.25	ft
Unbraced Length (Y-Axis) Ly:	0	ft
Column End Condition-K (e):	1	
Axial Load Duration Factor	1.00	
Lateral Load Duration Factor (Wind/Seismic)	1.60	

STUD PROPERTIES

#2 - Hem-Fir

	Base Values	Adjusted
Compressive Stress:	Fc = 1300 psi	Fc' = 266 psi
	Cd=1.60 Cf=1.10 Cp=0.12	
Bending Stress (X-X Axis):	Fbx = 850 psi	Fbx' = 2033 psi
	Cd=1.60 CF=1.30 Cr=1.15 Cl=1.00	
Bending Stress (Y-Y Axis):	Fby = 850 psi	Fby' = 2033 psi
	Cd=1.60 CF=1.30 Cr=1.15	
Modulus of Elasticity:	E = 1300 ksi	E' = 1300 ksi

Stud Section (X-X Axis):	dx =	5.5	in
Stud Section (Y-Y Axis):	dy =	1.5	in
Area:	A =	8.25	in2
Section Modulus (X-X Axis):	Sx =	7.56	in3
Section Modulus (Y-Y Axis):	Sy =	2.06	in3
Slenderness Ratio:	Lex/dx =	37.64	
	Ley/dy =	0	

Stud Calculations (Controlling Case Only):

Controlling Load Case: Axial Dead Load and Lateral loads (D + W or E)

Actual Compressive Stress:	Fc =	27	psi
Allowable Compressive Stress:	Fc' =	266	psi
Eccentricity Moment (X-X Axis):	Mx-ex =	0	ft-lb
Eccentricity Moment (Y-Y Axis):	My-ey =	0	ft-lb
Moment Due to Lateral Loads (X-X Axis):	Mx =	546	ft-lb
Moment Due to Lateral Loads (Y-Y Axis):	My =	0	ft-lb
Bending Stress Lateral Loads Only (X-X Axis):	Fbx =	866	psi
Allowable Bending Stress (X-X Axis):	Fbx' =	2033	psi
Bending Stress Lateral Loads Only (Y-Y Axis):	Fby =	0	psi
Allowable Bending Stress (Y-Y Axis):	Fby' =	2033	psi
Combined Stress Factor:	**CSF =**	**0.48**	

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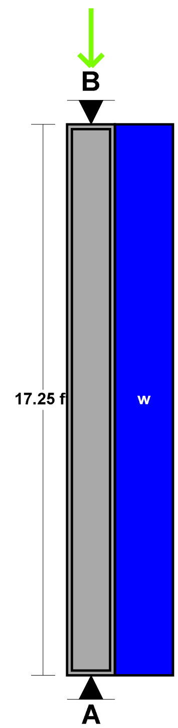

AXIAL LOADING

Live Load:	PL =	500 plf
Dead Load:	PD =	300 plf
Column Self Weight:	CSW =	27 plf
Total Axial Load:	PT =	827 plf

LATERAL LOADING (Dy Face)

Uniform Lateral Load:	wL-Lat =	22 psf

Project:

Location: Baloon Framed Stud Design (typical wind) - LSL Column
[2015 International Building Code(2015 NDS)]
1.75 IN x 5.5 IN x 17.25 FT @ 16 O.C.
1.55E Timberstrand LSL - iLevel Trus Joist
Section Adequate By: 8.5%

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DEFLECTIONS

Deflection due to lateral loads only:	Defl =	1.06	IN = L/195
Live Load Deflection Criteria:			L/180

VERTICAL REACTIONS

Live Load:	Vert-LL-Rxn =	667	lb
Dead Load:	Vert-DL-Rxn =	452	lb
Total Load:	Vert-TL-Rxn =	1119	lb

HORIZONTAL REACTIONS

Total Reaction at Top of Column:	TL-Rxn-Top =	173	lb
Total Reaction at Bottom of Column:	TL-Rxn-Bottom =	173	lb

COLUMN DATA

Total Column Length:	17.25	ft
Unbraced Length (X-Axis) Lx:	17.25	ft
Unbraced Length (Y-Axis) Ly:	0	ft
Column End Condition-K (e):	1	
Axial Load Duration Factor	1.00	
Lateral Load Duration Factor (Wind/Seismic)	1.60	

STUD PROPERTIES

1.55E Timberstrand LSL - iLevel Trus Joist

	Base Values	Adjusted
Compressive Stress:	Fc = 2170 psi	Fc' = 451 psi
	Cd=1.60 Cp=0.13	
Bending Stress (X-X Axis):	Fbx = 2325 psi	Fbx' = 3997 psi
	Cd=1.60 CF=1.07 Cl=1.00	
Bending Stress (Y-Y Axis):	Fby = 2325 psi	Fby' = 3997 psi
	Cd=1.60 CF=1.07	
Modulus of Elasticity:	E = 1550 ksi	E' = 1550 ksi

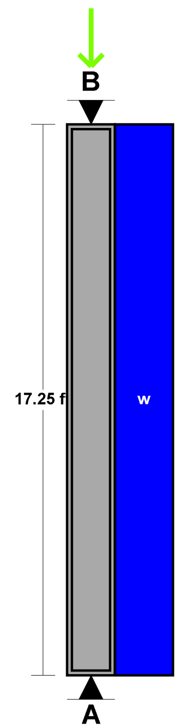
Stud Section (X-X Axis):	dx =	5.5	in
Stud Section (Y-Y Axis):	dy =	1.75	in
Area:	A =	9.63	in2
Section Modulus (X-X Axis):	Sx =	8.82	in3
Section Modulus (Y-Y Axis):	Sy =	2.81	in3
Slenderness Ratio:	Lex/dx =	37.64	
	Ley/dy =	0	

Stud Calculations (Controlling Case Only):

Controlling Load Case: Axial Dead Load and Lateral loads (D + W or E)

Actual Compressive Stress:	Fc =	47	psi
Allowable Compressive Stress:	Fc' =	451	psi
Eccentricity Moment (X-X Axis):	Mx-ex =	0	ft-lb
Eccentricity Moment (Y-Y Axis):	My-ey =	0	ft-lb
Moment Due to Lateral Loads (X-X Axis):	Mx =	744	ft-lb
Moment Due to Lateral Loads (Y-Y Axis):	My =	0	ft-lb
Bending Stress Lateral Loads Only (X-X Axis):	Fbx =	1012	psi
Allowable Bending Stress (X-X Axis):	Fbx' =	3997	psi
Bending Stress Lateral Loads Only (Y-Y Axis):	Fby =	0	psi
Allowable Bending Stress (Y-Y Axis):	Fby' =	3997	psi
Combined Stress Factor:	**CSF =**	**0.29**	

LOADING DIAGRAM



AXIAL LOADING

Live Load:	PL =	500 plf
Dead Load:	PD =	300 plf
Column Self Weight:	CSW =	52 plf
Total Axial Load:	PT =	852 plf

LATERAL LOADING (Dy Face)

Uniform Lateral Load:	wL-Lat =	15 psf

Project:

Location: Baloon Framed Stud Design (High Wind) - LSL Column
[2015 International Building Code(2015 NDS)]
1.75 IN x 5.5 IN x 17.25 FT @ 12 O.C.
1.55E Timberstrand LSL - iLevel Trus Joist
Section Adequate By: 1.0%

StruCalc 9.0

page of

StruCalc Version 10.0.1.6 3/24/2018 5:11:20 PM

DEFLECTIONS

Deflection due to lateral loads only:	Defl =	1.14	IN = L/182
Live Load Deflection Criteria:			L/180

VERTICAL REACTIONS

Live Load:	Vert-LL-Rxn =	500	lb
Dead Load:	Vert-DL-Rxn =	352	lb
Total Load:	Vert-TL-Rxn =	852	lb

HORIZONTAL REACTIONS

Total Reaction at Top of Column:	TL-Rxn-Top =	185	lb
Total Reaction at Bottom of Column:	TL-Rxn-Bottom =	185	lb

COLUMN DATA

Total Column Length:	17.25	ft
Unbraced Length (X-Axis) Lx:	17.25	ft
Unbraced Length (Y-Axis) Ly:	0	ft
Column End Condition-K (e):	1	
Axial Load Duration Factor	1.00	
Lateral Load Duration Factor (Wind/Seismic)	1.60	

STUD PROPERTIES

1.55E Timberstrand LSL - iLevel Trus Joist

	Base Values	Adjusted
Compressive Stress:	Fc = 2170 psi	Fc' = 451 psi
	Cd=1.60 Cp=0.13	
Bending Stress (X-X Axis):	Fbx = 2325 psi	Fbx' = 3997 psi
	Cd=1.60 CF=1.07 Cl=1.00	
Bending Stress (Y-Y Axis):	Fby = 2325 psi	Fby' = 3997 psi
	Cd=1.60 CF=1.07	
Modulus of Elasticity:	E = 1550 ksi	E' = 1550 ksi

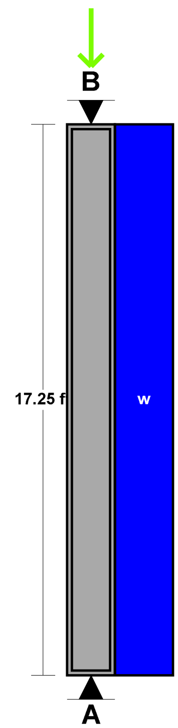
Stud Section (X-X Axis):	dx =	5.5	in
Stud Section (Y-Y Axis):	dy =	1.75	in
Area:	A =	9.63	in2
Section Modulus (X-X Axis):	Sx =	8.82	in3
Section Modulus (Y-Y Axis):	Sy =	2.81	in3
Slenderness Ratio:	Lex/dx =	37.64	
	Ley/dy =	0	

Stud Calculations (Controlling Case Only):

Controlling Load Case: Axial Dead Load and Lateral loads (D + W or E)

Actual Compressive Stress:	Fc =	37	psi
Allowable Compressive Stress:	Fc' =	451	psi
Eccentricity Moment (X-X Axis):	Mx-ex =	0	ft-lb
Eccentricity Moment (Y-Y Axis):	My-ey =	0	ft-lb
Moment Due to Lateral Loads (X-X Axis):	Mx =	800	ft-lb
Moment Due to Lateral Loads (Y-Y Axis):	My =	0	ft-lb
Bending Stress Lateral Loads Only (X-X Axis):	Fbx =	1088	psi
Allowable Bending Stress (X-X Axis):	Fbx' =	3997	psi
Bending Stress Lateral Loads Only (Y-Y Axis):	Fby =	0	psi
Allowable Bending Stress (Y-Y Axis):	Fby' =	3997	psi
Combined Stress Factor:	**CSF =**	**0.3**	

LOADING DIAGRAM



AXIAL LOADING

Live Load:	PL =	500 plf
Dead Load:	PD =	300 plf
Column Self Weight:	CSW =	52 plf
Total Axial Load:	PT =	852 plf

LATERAL LOADING (Dy Face)

Uniform Lateral Load:	wL-Lat =	22 psf

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Ledger Calculations

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Table 12.3.3A Assigned Specific Gravities

Species Combination	Specific[1] Gravity, G
Alaska Cedar	0.47
Alaska Hemlock	0.46
Alaska Spruce	0.41
Alaska Yellow Cedar	0.46
Aspen	0.39
Balsam Fir	0.36
Beech-Birch-Hickory	0.71
Coast Sitka Spruce	0.39
Cottonwood	0.41
Douglas Fir-Larch	0.50
Douglas Fir-Larch (North)	0.49
Douglas Fir-South	0.46
Eastern Hemlock	0.41
Eastern Hemlock-Balsam Fir	0.36
Eastern Hemlock-Tamarack	0.41
Eastern Hemlock-Tamarack (North)	0.47
Eastern Softwoods	0.36
Eastern Spruce	0.41
Eastern White Pine	0.36
Engelmann Spruce-Lodgepole Pine	0.38
Hem-Fir	0.43
Hem-Fir (North)	0.46
Mixed Maple	0.55
Mixed Oak	0.68
Mixed Southern Pine	0.51
Mountain Hemlock	0.47
Northern Pine	0.42
Northern Red Oak	0.68
Northern Species	0.35
Northern White Cedar	0.31
Ponderosa Pine	0.43
Red Maple	0.58
Red Oak	0.67
Red Pine	0.44
Redwood, close grain	0.44
Redwood, open grain	0.37
Sitka Spruce	0.43
Southern Pine	0.55
Spruce-Pine-Fir	0.42
Spruce-Pine-Fir (South)	0.36
Western Cedars	0.36
Western Cedars (North)	0.35
Western Hemlock	0.47
Western Hemlock (North)	0.46
Western White Pine	0.40
Western Woods	0.36
White Oak	0.73
Yellow Poplar	0.43

BEAMS (DF #2, and Engineered Lumber)

Joists and 2x members (HF #2)

Species Combinations of MSR and MEL Lumber	Specific[1] Gravity, G
Douglas Fir-Larch	
E=1,900,000 psi and lower grades of MSR	0.50
E=2,000,000 psi grades of MSR	0.51
E=2,100,000 psi grades of MSR	0.52
E=2,200,000 psi grades of MSR	0.53
E=2,300,000 psi grades of MSR	0.54
E=2,400,000 psi grades of MSR	0.55
Douglas Fir-Larch (North)	
E=1,900,000 psi and lower grades of MSR and MEL	0.49
E=2,000,000 psi to 2,200,000 psi grades of MSR and MEL	0.53
E=2,300,000 psi and higher grades of MSR and MEL	0.57
Douglas Fir-Larch (South)	
E=1,000,000 psi and higher grades of MSR	0.46
Engelmann Spruce-Lodgepole Pine	
E=1,400,000 psi and lower grades of MSR	0.38
E=1,500,000 psi and higher grades of MSR	0.46
Hem-Fir	
E=1,500,000 psi and lower grades of MSR	0.43
E=1,600,000 psi grades of MSR	0.44
E=1,700,000 psi grades of MSR	0.45
E=1,800,000 psi grades of MSR	0.46
E=1,900,000 psi grades of MSR	0.47
E=2,000,000 psi grades of MSR	0.48
E=2,100,000 psi grades of MSR	0.49
E=2,200,000 psi grades of MSR	0.50
E=2,300,000 psi grades of MSR	0.51
E=2,400,000 psi grades of MSR	0.52
Hem-Fir (North)	
E=1,000,000 psi and higher grades of MSR and MEL	0.46
Southern Pine	
E=1,700,000 psi and lower grades of MSR and MEL	0.55
E=1,800,000 psi and higher grades of MSR and MEL	0.57
Spruce-Pine-Fir	
E=1,700,000 psi and lower grades of MSR and MEL	0.42
E=1,800,000 psi and 1,900,000 grades of MSR and MEL	0.46
E=2,000,000 psi and higher grades of MSR and MEL	0.50
Spruce-Pine-Fir (South)	
E=1,100,000 psi and lower grades of MSR	0.36
E=1,200,000 psi to1,900,000 psi grades of MSR	0.42
E=2,000,000 psi and higher grades of MSR	0.50
Western Cedars	
E=1,000,000 psi and higher grades of MSR	0.36
Western Woods	
E=1,000,000 psi and higher grades of MSR	0.36

1. Specific gravity, G, based on weight and volume when oven-dry. Different specific gravities, G, are possible for different grades of MSR and MEL lumber (see Table 4C, Footnote 2).

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LAG SCREWS

Table 12K **LAG SCREWS: Reference Lateral Design Values, Z, for Single Shear (two member) Connections[1,2,3,4]**

for sawn lumber or SCL with ASTM A653, Grade 33 steel side plate (for t_s<1/4") or ASTM A 36 steel side plate (for t_s=1/4")
(tabulated lateral design values are calculated based on an assumed length of lag screw penetration, p, into the main member equal to 8D)

Side Member Thickness t_s in.	Lag Screw Diameter D in.	G=0.67 Red Oak $Z_{\parallel}$ lbs.	$Z_{\perp}$ lbs.	G=0.55 Mixed Maple Southern Pine $Z_{\parallel}$ lbs.	$Z_{\perp}$ lbs.	G=0.5 Douglas Fir-Larch $Z_{\parallel}$ lbs.	$Z_{\perp}$ lbs.	G=0.49 Douglas Fir-Larch (N) $Z_{\parallel}$ lbs.	$Z_{\perp}$ lbs.	G=0.46 Douglas Fir(S) Hem-Fir(N) $Z_{\parallel}$ lbs.	$Z_{\perp}$ lbs.	G=0.43 Hem-Fir $Z_{\parallel}$ lbs.	$Z_{\perp}$ lbs.	G=0.42 Spruce-Pine-Fir $Z_{\parallel}$ lbs.	$Z_{\perp}$ lbs.	G=0.37 Redwood (open grain) $Z_{\parallel}$ lbs.	$Z_{\perp}$ lbs.	G=0.36 Eastern Softwoods Spruce-Pine-Fir(S) Western Cedars Western Woods $Z_{\parallel}$ lbs.	$Z_{\perp}$ lbs.	G=0.35 Northern Species $Z_{\parallel}$ lbs.	$Z_{\perp}$ lbs.
0.075	1/4	170	130	160	120	150	110	150	110	150	100	140	100	140	100	130	90	130	90	130	90
(14 gage)	5/16	220	160	200	140	190	130	190	130	190	130	180	120	180	120	170	110	170	110	160	100
	3/8	220	160	200	140	200	130	190	130	190	120	180	120	180	120	170	110	170	100	170	100
0.105	1/4	180	140	170	130	160	120	160	120	160	110	150	110	150	110	140	100	140	100	140	90
(12 gage)	5/16	230	170	210	150	200	140	200	140	190	130	190	130	190	120	180	110	170	110	170	110
	3/8	230	160	210	140	200	140	200	130	200	130	190	120	190	120	180	110	180	110	170	110
0.120	1/4	190	150	180	130	170	120	170	120	160	120	160	110	160	110	150	100	150	100	140	100
(11 gage)	5/16	230	170	210	150	210	140	200	140	200	140	190	130	190	130	180	120	180	120	180	110
	3/8	240	170	220	150	210	140	210	140	200	130	200	130	190	120	180	110	180	110	180	110
0.134	1/4	200	150	180	140	180	130	170	130	170	120	160	120	160	110	150	110	150	100	150	100
(10 gage)	5/16	240	180	220	160	210	150	210	140	200	140	200	130	200	130	190	120	180	120	180	120
	3/8	240	170	220	150	220	140	210	140	210	140	200	130	200	130	190	120	190	120	180	110
0.179	1/4	220	170	210	150	200	150	200	140	190	140	190	130	190	130	180	120	170	120	170	120
(7 gage)	5/16	260	190	240	170	230	160	230	160	230	150	220	150	220	150	210	130	200	130	200	130
	3/8	270	190	250	170	240	160	240	160	230	150	220	140	220	140	210	130	210	130	200	130
0.239	1/4	240	180	220	160	210	150	210	150	200	140	190	140	190	130	180	120	180	120	180	120
(3 gage)	5/16	300	220	280	190	270	180	260	180	260	170	250	160	250	160	230	150	230	150	230	140
	3/8	310	220	280	190	270	180	270	180	260	170	250	160	250	160	240	140	230	140	230	140
	7/16	420	290	390	260	380	240	370	240	360	230	350	220	350	220	330	200	330	200	320	190
	1/2	510	340	470	300	460	290	450	280	440	270	430	260	420	260	400	240	400	230	390	230
	5/8	770	490	710	430	680	400	680	400	660	380	640	370	630	360	600	330	590	330	580	320
	3/4	1110	670	1020	590	980	560	970	550	950	530	920	500	910	500	860	450	850	450	840	440
	7/8	1510	880	1390	780	1330	730	1320	710	1280	690	1250	650	1230	650	1170	590	1160	590	1140	570
	1	1940	1100	1780	960	1710	910	1700	890	1650	860	1600	820	1590	810	1500	740	1480	730	1460	710
1/4	1/4	240	180	220	160	210	150	210	150	200	140	200	140	190	130	180	120	180	120	180	120
	5/16	310	220	280	200	270	180	270	180	260	170	250	170	250	160	230	150	230	150	230	140
	3/8	320	220	290	190	280	180	270	180	270	170	260	160	250	160	240	150	240	140	230	140
	7/16	480	320	440	280	420	270	420	260	410	250	390	240	390	230	370	220	360	210	360	210
	1/2	580	390	540	340	520	320	510	320	500	310	480	290	480	290	460	270	450	260	440	260
	5/8	850	530	780	470	750	440	740	440	720	420	700	400	690	400	660	370	650	360	640	350
	3/4	1200	730	1100	640	1060	600	1050	590	1020	570	990	540	980	530	930	490	920	480	900	470
	7/8	1600	930	1470	820	1410	770	1400	750	1360	720	1320	690	1310	680	1240	630	1220	620	1200	600
	1	2040	1150	1870	1000	1800	950	1780	930	1730	900	1680	850	1660	840	1570	770	1550	760	1530	740

1. Tabulated lateral design values, Z, shall be multiplied by all applicable adjustment factors (see Table 11.3.1).
2. Tabulated lateral design values, Z, are for "reduced body diameter" lag screws (see Appendix Table L2) inserted in side grain with screw axis perpendicular to wood fibers; screw penetration, p, into the main member equal to 8D; dowel bearing strengths, F_e, of 61,850 psi for ASTM A653, Grade 33 steel and 87,000 psi for ASTM A36 steel and screw bending yield strengths, F_{yb}, of 70,000 psi for D = 1/4", 60,000 psi for D = 5/16", and 45,000 psi for D ≥3/8".
3. Where the lag screw penetration, p, is less than 8D but not less than 4D, tabulated lateral design values, Z, shall be multiplied by p/8D or lateral design values shall be calculated using the provisions of 12.3 for the reduced penetration.
4. The length of lag screw penetration, p, not including the length of the tapered tip, E (see Appendix Table L2), of the lag screw into the main member shall not be less than 4D. See 12.1.4.6 for minimum length of penetration, p_{min}.

SDS connection of steel plate to wood, assuming HF, 100 lbs per 1/4" DIA SDS un-factored, without group action reduction, pending application/spacing.

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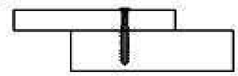
Table 12L **WOOD SCREWS: Reference Lateral Design Values, Z, for Single Shear (two member) Connections**[1,2,3]

for sawn lumber or SCL with both members of identical specific gravity (tabulated lateral design values are calculated based on an assumed length of wood screw penetration, p, into the main member equal to 10D)



WOOD SCREWS

Side Member Thickness t_s in.	Wood Screw Diameter D in.	Wood Screw Number	G=0.67 Red Oak lbs.	G=0.55 Mixed Maple Southern Pine lbs.	G=0.5 Douglas Fir-Larch lbs.	G=0.49 Douglas Fir-Larch(N) lbs.	G=0.46 Douglas Fir(S) Hem-Fir(N) lbs.	G=0.43 Hem-Fir lbs.	G=0.42 Spruce-Pine-Fir lbs.	G=0.37 Redwood (open grain) lbs.	G=0.36 Eastern Softwoods Spruce-Pine-Fir(S) Western Cedars Western Woods lbs.	G=0.35 Northern Species lbs.
1/2	0.138	6	88	67	59	57	53	49	47	41	40	38
	0.151	7	96	74	65	63	59	54	52	45	44	42
	0.164	8	107	82	73	71	66	61	59	51	50	48
	0.177	9	121	94	83	81	76	70	68	59	58	56
	0.190	10	130	101	90	87	82	75	73	64	63	60
	0.216	12	156	123	110	107	100	93	91	79	78	75
	0.242	14	168	133	120	117	110	102	99	87	86	83
5/8	0.138	6	94	76	66	64	59	53	52	44	43	41
	0.151	7	104	83	72	70	64	58	56	48	47	45
	0.164	8	120	92	80	77	72	65	63	54	53	51
	0.177	9	136	103	91	88	81	74	72	62	61	58
	0.190	10	146	111	97	94	88	80	78	67	65	63
	0.216	12	173	133	117	114	106	97	95	82	80	77
	0.242	14	184	142	126	123	115	106	103	89	87	84
3/4	0.138	6	94	79	72	71	65	58	57	47	46	44
	0.151	7	104	87	80	77	71	64	62	52	50	48
	0.164	8	120	101	88	85	78	71	69	58	56	54
	0.177	9	142	114	99	96	88	80	78	66	64	61
	0.190	10	153	122	107	103	95	86	83	71	69	66
	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	100	86	84	80
	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	108	93	91	87
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	61	55	54	51
	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	68	60	59	56
	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	78	67	65	62
	0.177	9	142	118	108	106	100	94	90	75	73	70
	0.190	10	153	128	117	114	108	101	97	81	78	75
	0.216	12	193	161	147	143	131	118	114	96	93	89
	0.242	14	213	178	157	152	139	126	122	102	100	95
1-1/4	0.138	6	94	79	72	71	67	63	61	55	54	52
	0.151	7	104	87	80	78	74	69	68	60	59	57
	0.164	8	120	101	92	90	85	80	78	70	68	66
	0.177	9	142	118	108	106	100	94	92	82	80	78
	0.190	10	153	128	117	114	108	101	99	88	87	84
	0.216	12	193	161	147	144	137	128	125	108	105	100
	0.242	14	213	178	163	159	151	141	138	115	111	106
1-1/2	0.138	6	94	79	72	71	67	63	61	55	54	52
	0.151	7	104	87	80	78	74	69	68	60	59	57
	0.164	8	120	101	92	90	85	80	78	70	68	66
	0.177	9	142	118	108	106	100	94	92	82	80	78
	0.190	10	153	128	117	114	108	101	99	88	87	84
	0.216	12	193	161	147	144	137	128	125	111	109	106
	0.242	14	213	178	163	159	151	141	138	123	120	117
1-3/4	0.138	6	94	79	72	71	67	63	61	55	54	52
	0.151	7	104	87	80	78	74	69	68	60	59	57
	0.164	8	120	101	92	90	85	80	78	70	68	66
	0.177	9	142	118	108	106	100	94	92	82	80	78
	0.190	10	153	128	117	114	108	101	99	88	87	84
	0.216	12	193	161	147	144	137	128	125	111	109	106
	0.242	14	213	178	163	159	151	141	138	123	120	117

Exterior: Typical Ledger connection w/ SDS, un-factored since typical Deck loading application with duration = 1. Minimum (3) SDSW screws into RIM @ 12" o.c stud. Assuming worst case with 12' deck framing with connections into RIM @ 12" o.c w/ 60 psf LL and 10 psf DL - loading on each connection, staggered, (and ignoring capacity of typical nailing of rim). Connection is 6' x 72 psf x 1.00 = 432# versus capacity into DF/Engineered lumber (LSL) - 489#, ok.

1. Tabulated lateral design values, Z, shall be multiplied by all applicable adjustment factors (see Table 11.3.1).
2. Tabulated lateral design values, Z, are for rolled thread wood screws (see Appendix Table L3) inserted in side grain with screw axis perpendicular to wood fibers; screw penetration, p, into the main member equal to 10D; and screw bending yield strengths, F_{yb}, of 100,000 psi for $0.099" \le D \le 0.142"$, 90,000 psi for $0.142" < D \le 0.177"$, 80,000 psi for $0.177" < D \le 0.236"$, and 70,000 psi for $0.236" < D \le 0.273"$.
3. Where the wood screw penetration, p, is less than 10D but not less than 6D, tabulated lateral design values, Z, shall be multiplied by p/10D or lateral design values shall be calculated using the provisions of 12.3 for the reduced penetration.

DOWEL-TYPE FASTENERS

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Interior: Typical Ledger connection w/ SDS, un-factored since typical floor loading application with duration = 1. Minimum (3) SDSW screws into studs/rim @ 16" o.c stud. Assuming worst case with 14' floor framing with connections into RIM @ 16" o.c w/ 40 psf LL and 12 psf DL - loading on each connection, staggered, (and ignoring capacity of typical nailing of rim). Connection is 7' x 52 psf x 1.00 = 364# versus capacity into HF lumber (SS) - 423#, ok.

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Table 12.2A   Lag Screw Reference Withdrawal Design Values, W[1]

Tabulated withdrawal design values (W) are in pounds per inch of thread penetration into side grain of wood member. Length of thread penetration in main member shall not include the length of the tapered tip (see 12.2.1.1).

Specific Gravity, G[2]	Lag Screw Diameter, D										
	1/4"	5/16"	3/8"	7/16"	1/2"	5/8"	3/4"	7/8"	1"	1-1/8"	1-1/4"
0.73	397	469	538	604	668	789	905	1016	1123	1226	1327
0.71	381	450	516	579	640	757	868	974	1077	1176	1273
0.68	357	422	484	543	600	709	813	913	1009	1103	1193
0.67	349	413	473	531	587	694	796	893	987	1078	1167
0.58	281	332	381	428	473	559	641	719	795	869	940
0.55	260	307	352	395	437	516	592	664	734	802	868
0.51	232	274	314	353	390	461	528	593	656	716	775
0.50	225	266	305	342	378	447	513	576	636	695	752
0.49	218	258	296	332	367	434	498	559	617	674	730
0.47	205	242	278	312	345	408	467	525	580	634	686
0.46	199	235	269	302	334	395	453	508	562	613	664
0.44	186	220	252	283	312	369	423	475	525	574	621
0.43	179	212	243	273	302	357	409	459	508	554	600
0.42	173	205	235	264	291	344	395	443	490	535	579
0.41	167	198	226	254	281	332	381	428	473	516	559
0.40	161	190	218	245	271	320	367	412	455	497	538
0.39	155	183	210	236	261	308	353	397	438	479	518
0.38	149	176	202	227	251	296	340	381	422	461	498
0.37	143	169	194	218	241	285	326	367	405	443	479
0.36	137	163	186	209	231	273	313	352	389	425	460
0.35	132	156	179	200	222	262	300	337	373	407	441
0.31	110	130	149	167	185	218	250	281	311	339	367

1. Tabulated withdrawal design values, W, for lag screw connections shall be multiplied by all applicable adjustment factors (see Table 11.3.1).
2. Specific gravity, G, shall be determined in accordance with Table 12.3.3A.

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12.2.3.2 For calculation of the fastener reference withdrawal design value in pounds, the unit reference withdrawal design value in lbs/in. of fastener penetration from 12.2.3.1 shall be multiplied by the length of fastener penetration, p_t, into the wood member.

12.2.3.3 The reference withdrawal design value, in lbs/in. of penetration, for a single post-frame ring shank nail driven in the side grain of the main member, with the nail axis perpendicular to the wood fibers, shall be determined from Table 12.2D or Equation 12.2-4, within the range of specific gravities and nail diameters given in Table 12.2D. Reference withdrawal design values, W, shall be multiplied by all applicable adjustment factors (see Table 11.3.1) to obtain adjusted withdrawal design values, W'.

$$W = 1800 G^2 D \quad (12.2\text{-}4)$$

Ledger withdrawal capacity - assuming minimum 1 1/2" embed (tip discounted) into SS/HF material = 179# x 1.5 x 3 = 805# per 16" of ledger connection (maximum utilized)

12.2.3.4 For calculation of the fastener reference withdrawal design value in pounds, the unit reference withdrawal design value in lbs/in. of ring shank penetration from 12.2.3.3 shall be multiplied by the length of ring shank penetration, p_t, into the wood member.

12.2.3.5 Nails and spikes shall not be loaded in withdrawal from end grain of wood (C_{eg}=0.0).

12.2.3.6 Nails, and spikes shall not be loaded in withdrawal from end-grain of laminations in cross-laminated timber (C_{eg}=0.0).

12.2.4 Drift Bolts and Drift Pins

Reference withdrawal design values, W, for connections using drift bolt and drift pin connections shall be determined in accordance with 11.1.1.3.

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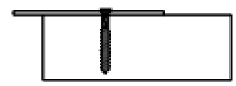
WOOD SCREWS

Table 12M **WOOD SCREWS: Reference Lateral Design Values, Z, for Single Shear (two member) Connections[1,2,3]**

for sawn lumber or SCL with ASTM 653, Grade 33 steel side plate (tabulated lateral design values are calculated based on an assumed length of wood screw penetration, p, into the main member equal to 10D)



Side Member Thickness t_s in.	Wood Screw Diameter D in.	Wood Screw Number	G=0.67 Red Oak lbs.	G=0.55 Mixed Maple Southern Pine lbs.	G=0.5 Douglas Fir-Larch lbs.	G=0.49 Douglas Fir-Larch(N) lbs.	G=0.46 Douglas Fir(S) Hem-Fir(N) lbs.	G=0.43 Hem-Fir lbs.	G=0.42 Spruce-Pine-Fir lbs.	G=0.37 Redwood (open grain) lbs.	G=0.36 Eastern Softwoods Spruce-Pine-Fir(S) Western Cedars Western Woods lbs.	G=0.35 Northern Species lbs.
0.036 (20 gage)	0.138	6	89	76	70	69	66	62	60	54	53	52
	0.151	7	99	84	78	76	72	68	67	60	59	57
	0.164	8	113	97	89	87	83	78	77	69	67	66
0.048 (18 gage)	0.138	6	90	77	71	70	67	63	61	55	54	53
	0.151	7	100	85	79	77	74	69	68	61	60	58
	0.164	8	114	98	90	89	84	79	78	70	69	67
0.060 (16 gage)	0.138	6	92	79	73	72	68	64	63	57	56	54
	0.151	7	101	87	81	79	75	71	70	63	61	60
	0.164	8	116	100	92	90	86	81	79	71	70	68
	0.177	9	136	116	107	105	100	94	93	83	82	79
	0.190	10	146	125	116	114	108	102	100	90	88	86
0.075 (14 gage)	0.138	6	95	82	76	75	71	67	66	59	58	57
	0.151	7	105	90	84	82	78	74	72	65	64	62
	0.164	8	119	103	95	93	89	84	82	74	73	71
	0.177	9	139	119	110	108	103	97	95	86	84	82
	0.190	10	150	128	119	117	111	105	103	92	91	88
	0.216	12	186	159	147	145	138	130	127	114	112	109
	0.242	14	204	175	162	158	151	142	139	125	123	120
0.105 (12 gage)	0.138	6	104	90	84	82	79	74	73	66	65	63
	0.151	7	114	99	92	90	86	81	80	72	71	69
	0.164	8	129	111	103	102	97	92	90	81	80	77
	0.177	9	148	128	119	116	111	105	103	93	91	89
	0.190	10	160	138	128	125	120	113	111	100	98	96
	0.216	12	196	168	156	153	146	138	135	122	120	116
	0.242	14	213	183	170	167	159	150	147	132	130	126
0.120 (11 gage)	0.138	6	110	95	89	87	83	79	77	70	68	67
	0.151	7	120	104	97	95	91	86	84	76	75	73
	0.164	8	135	117	109	107	102	96	94	85	84	82
	0.177	9	154	133	124	121	116	110	107	97	95	93
	0.190	10	166	144	133	131	125	118	116	104	103	100
	0.216	12	202	174	162	159	152	143	140	126	124	121
	0.242	14	219	189	175	172	164	155	152	137	134	131
0.134 (10 gage)	0.138	6	116	100	93	92	88	83	81	73	72	70
	0.151	7	126	110	102	100	96	91	89	80	79	77
	0.164	8	141	122	114	112	107	101	99	89	88	86
	0.177	9	160	139	129	127	121	114	112	101	100	97
	0.190	10	173	149	139	136	130	123	121	109	107	104
	0.216	12	209	180	167	164	157	148	145	131	129	126
	0.242	14	226	195	181	177	169	160	157	141	139	135
0.179 (7 gage)	0.138	6	126	107	99	97	92	86	84	76	74	72
	0.151	7	139	118	109	107	102	95	93	84	82	80
	0.164	8	160	136	126	123	117	110	108	96	95	92
	0.177	9	184	160	148	145	138	129	127	113	111	108
	0.190	10	198	172	159	156	149	140	137	122	120	117
	0.216	12	234	203	189	186	178	168	165	149	146	143
	0.242	14	251	217	202	198	190	179	176	159	156	152
0.239 (3 gage)	0.138	6	126	107	99	97	92	86	84	76	74	72
	0.151	7	139	118	109	107	102	95	93	84	82	80
	0.164	8	160	136	126	123	117	110	108	96	95	92
	0.177	9	188	160	148	145	138	129	127	113	111	108
	0.190	10	204	173	159	156	149	140	137	122	120	117
	0.216	12	256	218	201	197	187	176	172	154	151	147
	0.242	14	283	241	222	217	207	194	190	170	167	162

1. Tabulated lateral design values, Z, shall be multiplied by all applicable adjustment factors (see Table 11.3.1).
2. Tabulated lateral design values, Z, are for rolled thread wood screws (see Appendix L) inserted in side grain with screw axis perpendicular to wood fibers; screw penetration, p, into the main member equal to 10D; dowel bearing strength, F_e, of 61,850 psi for ASTM A653, Grade 33 steel and screw bending yield strengths, F_{yb}, of 100,000 psi for 0.099" $\leq$ D $\leq$ 0.142", 90,000 psi for 0.142" < D $\leq$ 0.177", 80,000 psi for 0.177" < D $\leq$ 0.236", 70,000 psi for 0.236" < D $\leq$ 0.273".
3. Where the wood screw penetration, p, is less than 10D but not less than 6D, tabulated lateral design values, Z, shall be multiplied by p/10D or lateral design values shall be calculated using the provisions of 12.3 for the reduced penetration.

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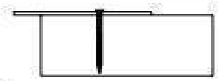
Table 12P (Cont.) **COMMON, BOX, or SINKER STEEL WIRE NAILS: Reference Lateral Design Values, Z, for Single Shear (two member) Connections[1,2,3]**

for sawn lumber or SCL with ASTM 653, Grade 33 steel side plate
(tabulated lateral design values are calculated based on an assumed length of nail penetration, p, into the main member equal to 10D)



Side Member Thickness t_s in.	Nail Diameter D in.	Common Wire Nail (Pennyweight)	Box Nail (Pennyweight)	Sinker Nail (Pennyweight)	G=0.67 Red Oak lbs.	G=0.55 Mixed Maple Southern Pine lbs.	G=0.5 Douglas Fir-Larch lbs.	G=0.49 Douglas Fir-Larch (N) lbs.	G=0.46 Douglas Fir(S) Hem-Fir(N) lbs.	G=0.43 Hem-Fir lbs.	G=0.42 Spruce-Pine-Fir lbs.	G=0.37 Redwood (open grain) lbs.	G=0.36 Eastern Softwoods Spruce-Pine-Fir(S) Western Cedars Western Woods lbs.	G=0.35 Northern Species lbs.
0.120 (11 gage)	0.099		6d	7d	90	78	72	71	68	64	63	57	56	53
	0.113	6d	8d	8d	110	95	89	87	83	79	77	70	68	66
	0.120			10d	121	105	97	96	91	86	85	76	75	73
	0.128		10d		134	116	108	106	101	96	94	85	83	81
	0.131	8d			140	121	112	110	105	99	97	88	86	84
	0.135		16d	12d	147	127	118	116	110	104	102	92	91	88
	0.148	10d	20d	16d	165	143	133	130	124	117	115	104	102	99
	0.162	16d	40d		193	166	154	152	145	137	134	121	119	115
	0.177			20d	218	188	174	171	163	154	151	136	134	130
	0.192	20d		30d	226	195	181	177	169	159	156	141	138	135
	0.207	30d		40d	244	210	194	191	182	172	168	151	149	145
	0.225	40d			265	228	211	207	198	186	183	164	161	157
	0.244	50d		60d	272	234	217	213	203	191	187	169	166	161
0.134 (10 gage)	0.099		6d	7d	95	82	76	74	71	66	65	58	56	54
	0.113	6d	8d	8d	116	100	93	92	88	83	81	73	72	69
	0.120			10d	127	110	102	100	96	91	89	80	79	76
	0.128		10d		140	122	113	111	106	100	98	89	87	85
	0.131	8d			146	126	117	115	110	104	102	92	90	88
	0.135		16d	12d	153	132	123	121	115	109	107	96	95	92
	0.148	10d	20d	16d	172	148	138	135	129	122	120	108	106	104
	0.162	16d	40d		199	172	160	157	150	142	139	125	123	120
	0.177			20d	224	194	180	176	169	159	156	141	138	135
	0.192	20d		30d	232	200	186	182	174	164	161	145	143	139
	0.207	30d		40d	249	215	199	196	187	176	173	156	153	149
	0.225	40d			270	233	216	212	202	191	187	168	165	161
	0.244	50d		60d	277	239	221	217	207	195	192	173	170	165
0.179 (7 gage)	0.099		6d	7d	97	82	76	74	71	66	65	58	56	54
	0.113	6d	8d	8d	126	107	99	97	92	86	84	76	74	70
	0.120			10d	142	121	111	109	104	97	95	85	83	79
	0.128		10d		161	137	126	124	118	111	108	97	94	90
	0.131	8d			168	144	132	130	123	116	114	102	99	94
	0.135		16d	12d	175	152	141	138	131	123	121	108	105	100
	0.148	10d	20d	16d	195	170	158	155	148	140	137	123	121	117
	0.162	16d	40d		224	194	180	177	169	160	157	142	140	136
	0.177			20d	249	215	200	197	188	178	174	157	155	151
	0.192	20d		30d	256	222	206	203	194	183	179	162	159	155
	0.207	30d		40d	272	236	219	215	205	194	190	172	169	164
	0.225	40d			292	252	234	230	220	207	203	184	180	176
	0.244	50d		60d	299	258	240	235	225	212	208	188	185	180
0.239 (3 gage)	0.099		6d	7d	97	82	76	74	71	66	65	58	56	54
	0.113	6d	8d	8d	126	107	99	97	92	86	84	76	74	70
	0.120			10d	142	121	111	109	104	97	95	85	83	79
	0.128		10d		161	137	126	124	118	111	108	97	94	90
	0.131	8d			169	144	132	130	123	116	114	102	99	94
	0.135		16d	12d	180	153	141	138	131	123	121	108	105	100
	0.148	10d	20d	16d	205	174	160	157	149	140	137	123	121	117
	0.162	16d	40d		245	209	192	188	179	168	165	147	145	140
	0.177			20d	284	241	222	218	207	195	191	170	167	162
	0.192	20d		30d	295	251	231	227	216	202	198	177	174	169
	0.207	30d		40d	310	270	251	246	236	222	217	194	191	185
	0.225	40d			328	285	265	260	249	235	231	209	205	200
	0.244	50d		60d	336	291	271	266	254	240	236	213	210	204

1. Tabulated lateral design values, Z, shall be multiplied by all applicable adjustment factors (see Table 11.3.1).
2. Tabulated lateral design values, Z, are for common, box, or sinker steel wire nails (see Appendix Table L4) inserted in side grain with nail axis perpendicular to wood fibers; nail penetration, p, into the main member equal to 10D; dowel bearing strength, F_e, of 61,850 psi for ASTM A653, Grade 33 steel and nail bending yield strengths, F_{yb}, of 100,000 psi for 0.099" $\leq$ D $\leq$ 0.142", 90,000 psi for 0.142" < D $\leq$ 0.177", 80,000 psi for 0.177" < D $\leq$ 0.236", 70,000 psi for 0.236" < D $\leq$ 0.273".
3. Where the nail or spike penetration, p, is less than 10D but not less than 6D, tabulated lateral design values, Z, shall be multiplied by p/10D or lateral design values shall be calculated using the provisions of 12.3 for the reduced penetration.

NAILS

DOWEL-TYPE FASTENERS

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Table 11.3.6A Group Action Factors, C_g, for Bolt or Lag Screw Connections with Wood Side Members[2]

For D = 1", s = 4", E = 1,400,000 psi

A_s/A_m[1]	A_s[1] in.^{2}	Number of fasteners in a row: 2	3	4	5	6	7	8	9	10	11	12
0.5	5	0.98	0.92	0.84	0.75	0.68	0.61	0.55	0.50	0.45	0.41	0.38
	12	0.99	0.96	0.92	0.87	0.81	0.76	0.70	0.65	0.61	0.57	0.53
	20	0.99	0.98	0.95	0.91	0.87	0.83	0.78	0.74	0.70	0.66	0.62
	28	1.00	0.98	0.96	0.93	0.90	0.87	0.83	0.79	0.76	0.72	0.69
	40	1.00	0.99	0.97	0.95	0.93	0.90	0.87	0.84	0.81	0.78	0.75
	64	1.00	0.99	0.98	0.97	0.95	0.93	0.91	0.89	0.87	0.84	0.82
1	5	1.00	0.97	0.91	0.85	0.78	0.71	0.64	0.59	0.54	0.49	0.45
	12	1.00	0.99	0.96	0.93	0.88	0.84	0.79	0.74	0.70	0.65	0.61
	20	1.00	0.99	0.98	0.95	0.92	0.89	0.86	0.82	0.78	0.75	0.71
	28	1.00	0.99	0.98	0.97	0.94	0.92	0.89	0.86	0.83	0.80	0.77
	40	1.00	1.00	0.99	0.98	0.96	0.94	0.92	0.90	0.87	0.85	0.82
	64	1.00	1.00	0.99	0.98	0.97	0.96	0.95	0.93	0.91	0.90	0.88

1. Where $A_s/A_m > 1.0$, use A_m/A_s and use A_m instead of A_s.
2. Tabulated group action factors (C_g) are conservative for D < 1", s < 4", or E > 1,400,000 psi.

Table 11.3.6B Group Action Factors, C_g, for 4" Split Ring or Shear Plate Connectors with Wood Side Members[2]

s = 9", E = 1,400,000 psi

A_s/A_m[1]	A_s[1] in.^{2}	Number of fasteners in a row: 2	3	4	5	6	7	8	9	10	11	12
0.5	5	0.90	0.73	0.59	0.48	0.41	0.35	0.31	0.27	0.25	0.22	0.20
	12	0.95	0.83	0.71	0.60	0.52	0.45	0.40	0.36	0.32	0.29	0.27
	20	0.97	0.88	0.78	0.69	0.60	0.53	0.47	0.43	0.39	0.35	0.32
	28	0.97	0.91	0.82	0.74	0.66	0.59	0.53	0.48	0.44	0.40	0.37
	40	0.98	0.93	0.86	0.79	0.72	0.65	0.59	0.54	0.49	0.45	0.42
	64	0.99	0.95	0.91	0.85	0.79	0.73	0.67	0.62	0.58	0.54	0.50
1	5	1.00	0.87	0.72	0.59	0.50	0.43	0.38	0.34	0.30	0.28	0.25
	12	1.00	0.93	0.83	0.72	0.63	0.55	0.48	0.43	0.39	0.36	0.33
	20	1.00	0.95	0.88	0.79	0.71	0.63	0.57	0.51	0.46	0.42	0.39
	28	1.00	0.97	0.91	0.83	0.76	0.69	0.62	0.57	0.52	0.47	0.44
	40	1.00	0.98	0.93	0.87	0.81	0.75	0.69	0.63	0.58	0.54	0.50
	64	1.00	0.98	0.95	0.91	0.87	0.82	0.77	0.72	0.67	0.62	0.58

1. Where $A_s/A_m > 1.0$, use A_m/A_s and use A_m instead of A_s.
2. Tabulated group action factors (C_g) are conservative for 2-1/2" split ring connectors, 2-5/8" shear plate connectors, s < 9", or E > 1,400,000 psi.

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Table 11.3.6C Group Action Factors, C_g, for Bolt or Lag Screw Connections with Steel Side Plates[1]

For D = 1", s = 4", E_{wood} = 1,400,000 psi, E_{steel} = 30,000,000 psi

A_m/A_s	A_m in.^{2}	Number of fasteners in a row: 2	3	4	5	6	7	8	9	10	11	12
12	5	0.97	0.89	0.80	0.70	0.62	0.55	0.49	0.44	0.40	0.37	0.34
	8	0.98	0.93	0.85	0.77	0.70	0.63	0.57	0.52	0.47	0.43	0.40
	16	0.99	0.96	0.92	0.86	0.80	0.75	0.69	0.64	0.60	0.55	0.52
	24	0.99	0.97	0.94	0.90	0.85	0.81	0.76	0.71	0.67	0.63	0.59
	40	1.00	0.98	0.96	0.94	0.90	0.87	0.83	0.79	0.76	0.72	0.69
	64	1.00	0.99	0.98	0.96	0.94	0.91	0.88	0.86	0.83	0.80	0.77
	120	1.00	0.99	0.99	0.98	0.96	0.95	0.93	0.91	0.90	0.87	0.85
	200	1.00	1.00	0.99	0.99	0.98	0.97	0.96	0.95	0.93	0.92	0.90
18	5	0.99	0.93	0.85	0.76	0.68	0.61	0.54	0.49	0.44	0.41	0.37
	8	0.99	0.95	0.90	0.83	0.75	0.69	0.62	0.57	0.52	0.48	0.44
	16	1.00	0.98	0.94	0.90	0.85	0.79	0.74	0.69	0.65	0.60	0.56
	24	1.00	0.98	0.96	0.93	0.89	0.85	0.80	0.76	0.72	0.68	0.64
	40	1.00	0.99	0.97	0.95	0.93	0.90	0.87	0.83	0.80	0.77	0.73
	64	1.00	0.99	0.98	0.97	0.95	0.93	0.91	0.89	0.86	0.83	0.81
	120	1.00	1.00	0.99	0.98	0.97	0.96	0.95	0.93	0.92	0.90	0.88
	200	1.00	1.00	0.99	0.99	0.98	0.98	0.97	0.96	0.95	0.94	0.92
24	40	1.00	0.99	0.97	0.95	0.93	0.89	0.86	0.83	0.79	0.76	0.72
	64	1.00	0.99	0.98	0.97	0.95	0.93	0.91	0.88	0.85	0.83	0.80
	120	1.00	1.00	0.99	0.98	0.97	0.96	0.95	0.93	0.91	0.90	0.88
	200	1.00	1.00	0.99	0.99	0.98	0.98	0.97	0.96	0.95	0.93	0.92
30	40	1.00	0.98	0.96	0.93	0.89	0.85	0.81	0.77	0.73	0.69	0.65
	64	1.00	0.99	0.97	0.95	0.93	0.90	0.87	0.83	0.80	0.77	0.73
	120	1.00	0.99	0.99	0.97	0.96	0.94	0.92	0.90	0.88	0.85	0.83
	200	1.00	1.00	0.99	0.98	0.97	0.96	0.95	0.94	0.92	0.90	0.89
35	40	0.99	0.97	0.94	0.91	0.86	0.82	0.77	0.73	0.68	0.64	0.60
	64	1.00	0.98	0.96	0.94	0.91	0.87	0.84	0.80	0.76	0.73	0.69
	120	1.00	0.99	0.98	0.97	0.95	0.92	0.90	0.88	0.85	0.82	0.79
	200	1.00	0.99	0.99	0.98	0.97	0.95	0.94	0.92	0.90	0.88	0.86
42	40	0.99	0.97	0.93	0.88	0.83	0.78	0.73	0.68	0.63	0.59	0.55
	64	0.99	0.98	0.95	0.92	0.88	0.84	0.80	0.76	0.72	0.68	0.64
	120	1.00	0.99	0.97	0.95	0.93	0.90	0.88	0.85	0.81	0.78	0.75
	200	1.00	0.99	0.98	0.97	0.96	0.94	0.92	0.90	0.88	0.85	0.83
50	40	0.99	0.96	0.91	0.85	0.79	0.74	0.68	0.63	0.58	0.54	0.51
	64	0.99	0.97	0.94	0.90	0.85	0.81	0.76	0.72	0.67	0.63	0.59
	120	1.00	0.98	0.97	0.94	0.91	0.88	0.85	0.81	0.78	0.74	0.71
	200	1.00	0.99	0.98	0.96	0.95	0.92	0.90	0.87	0.85	0.82	0.79

1. Tabulated group action factors (C_g) are conservative for D < 1" or s < 4".

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Table 11.3.6D Group Action Factors, C_g, for 4" Shear Plate Connectors with Steel Side Plates[1]

$s = 9"$, $E_{wood} = 1{,}400{,}000$ psi, $E_{steel} = 30{,}000{,}000$ psi

A_m/A_s	A_m in.^{2}	Number of fasteners in a row: 2	3	4	5	6	7	8	9	10	11	12
12	5	0.91	0.75	0.60	0.50	0.42	0.36	0.31	0.28	0.25	0.23	0.21
	8	0.94	0.80	0.67	0.56	0.47	0.41	0.36	0.32	0.29	0.26	0.24
	16	0.96	0.87	0.76	0.66	0.58	0.51	0.45	0.40	0.37	0.33	0.31
	24	0.97	0.90	0.82	0.73	0.64	0.57	0.51	0.46	0.42	0.39	0.35
	40	0.98	0.94	0.87	0.80	0.73	0.66	0.60	0.55	0.50	0.46	0.43
	64	0.99	0.96	0.91	0.86	0.80	0.74	0.69	0.63	0.59	0.55	0.51
	120	0.99	0.98	0.95	0.91	0.87	0.83	0.79	0.74	0.70	0.66	0.63
	200	1.00	0.99	0.97	0.95	0.92	0.89	0.85	0.82	0.79	0.75	0.72
18	5	0.97	0.83	0.68	0.56	0.47	0.41	0.36	0.32	0.28	0.26	0.24
	8	0.98	0.87	0.74	0.62	0.53	0.46	0.40	0.36	0.32	0.30	0.27
	16	0.99	0.92	0.82	0.73	0.64	0.56	0.50	0.45	0.41	0.37	0.34
	24	0.99	0.94	0.87	0.78	0.70	0.63	0.57	0.51	0.47	0.43	0.39
	40	0.99	0.96	0.91	0.85	0.78	0.72	0.66	0.60	0.55	0.51	0.47
	64	1.00	0.97	0.94	0.89	0.84	0.79	0.74	0.69	0.64	0.60	0.56
	120	1.00	0.99	0.97	0.94	0.90	0.87	0.83	0.79	0.75	0.71	0.67
	200	1.00	0.99	0.98	0.96	0.94	0.91	0.89	0.86	0.82	0.79	0.76
24	40	1.00	0.96	0.91	0.84	0.77	0.71	0.65	0.59	0.54	0.50	0.46
	64	1.00	0.98	0.94	0.89	0.84	0.78	0.73	0.68	0.63	0.58	0.54
	120	1.00	0.99	0.96	0.94	0.90	0.86	0.82	0.78	0.74	0.70	0.66
	200	1.00	0.99	0.98	0.96	0.94	0.91	0.88	0.85	0.82	0.78	0.75
30	40	0.99	0.93	0.86	0.78	0.70	0.63	0.57	0.52	0.47	0.43	0.40
	64	0.99	0.96	0.90	0.84	0.78	0.71	0.66	0.60	0.56	0.51	0.48
	120	0.99	0.98	0.94	0.90	0.86	0.81	0.76	0.71	0.67	0.63	0.59
	200	1.00	0.98	0.96	0.94	0.91	0.87	0.83	0.79	0.76	0.72	0.68
35	40	0.98	0.91	0.83	0.74	0.66	0.59	0.53	0.48	0.43	0.40	0.36
	64	0.99	0.94	0.88	0.81	0.73	0.67	0.61	0.56	0.51	0.47	0.43
	120	0.99	0.97	0.93	0.88	0.82	0.77	0.72	0.67	0.62	0.58	0.54
	200	1.00	0.98	0.95	0.92	0.88	0.84	0.80	0.76	0.71	0.68	0.64
42	40	0.97	0.88	0.79	0.69	0.61	0.54	0.48	0.43	0.39	0.36	0.33
	64	0.98	0.92	0.84	0.76	0.69	0.62	0.56	0.51	0.46	0.42	0.39
	120	0.99	0.95	0.90	0.85	0.78	0.72	0.67	0.62	0.57	0.53	0.49
	200	0.99	0.97	0.94	0.90	0.85	0.80	0.76	0.71	0.67	0.62	0.59
50	40	0.95	0.86	0.75	0.65	0.56	0.49	0.44	0.39	0.35	0.32	0.30
	64	0.97	0.90	0.81	0.72	0.64	0.57	0.51	0.46	0.42	0.38	0.35
	120	0.98	0.94	0.88	0.81	0.74	0.68	0.62	0.57	0.52	0.48	0.45
	200	0.99	0.96	0.92	0.87	0.82	0.77	0.71	0.66	0.62	0.58	0.54

1. Tabulated group action factors (C_g) are conservative for 2-5/8" shear plate connectors or $s < 9"$.

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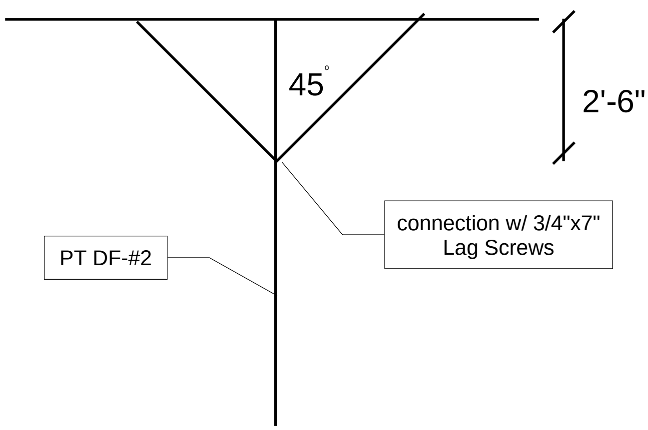
Knee Brace Calculation (Max Capacity)



M = V* h - T * d * 0.707

Shear (3/4" diam) = 887#
Withdrawal = 2873#
T total = 1355#

Therefore:

V = T/5.66

V Max - 1355/5.66 =

=239 lbs/brace