LONGITUDE
ONE TWENTY°
ENGINEERING & DESIGN

Structural Package for:

Forest Creek Estates Lot 2

5214 Forest Ave SE
Mercer Island, WA 98040

Project No: S22201

April 25, 2023

BDC Response:

See pages 70-72 for revised retaining wall calulcation

STRUCTURAL ENGINEER
L120 ENGINEERING & DESIGN
13150 91ST PL NE
KIRKLAND, WA 98034
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Project Number:	Plan Name:	Sheet Number:
S22201	Forest Creek Estates Lot 2	DC
Engineer: HK	Specifics: Design Criteria	Date: 11/10/2022

Gravity Criteria: Code: IBC 2018

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/Mech	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
Miscellaneous	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	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 7-16

R = **6.5** Bearing Wall System, Wood Structural Panel Walls

Mapped Spectral Acceleration, Ss = **1.45**

Mapped Spectral Acceleration, S1 = **0.503**

Soil Site Class = **D**

WIND PARAMETERS:

Code Reference: ASCE 7-16

Basic Wind Speed (3 second Gust) = **100** mph

Exposure : **B**

Kzt = **1.00**

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 = **250** pcf

Soil Friction Coeff. = **0.35**

This is a beta release of the new ATC Hazards by Location website. Please contact us with feedback.

The ATC Hazards by Location website will not be updated to support ASCE 7-22. Find out why.

ATC Hazards by Location

Search Information

Address: 5214 Forest Ave SE, Mercer Island, WA 98040, USA

Coordinates: 47.55590489999999, -122.227624

Elevation: 119 ft

Timestamp: 2022-11-01T21:44:10.159Z

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.

Please note that the ATC Hazards by Location website will not be updated to support ASCE 7-22. Find out why.

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.

While the information presented on this website is believed to be correct, ATC and its sponsors and contributors assume no responsibility

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ATC Hazards by Location

Search Information

Address: 5214 Forest Ave SE, Mercer Island, WA 98040, USA

Coordinates: 47.55590489999999, -122.227624

Elevation: 119 ft

Timestamp: 2022-11-01T21:46:18.271Z

Hazard Type: Seismic

Reference Document: ASCE7-16

Risk Category: II

Site Class: D-default

Basic Parameters

Name	Value	Description
S_S	1.45	MCE_R ground motion (period=0.2s)
S_1	0.503	MCE_R ground motion (period=1.0s)
S_{MS}	1.741	Site-modified spectral acceleration value
S_{M1}	* null	Site-modified spectral acceleration value
S_{DS}	1.16	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	1.2	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	0.898	Coefficient of risk (1.0s)
PGA	0.621	MCE_G - peak ground acceleration

PGA	0.621	MCE_G peak ground acceleration
F_{PGA}	1.2	Site amplification factor at PGA
PGA_M	0.745	Site modified peak ground acceleration
T_L	6	Long-period transition period (s)
SsRT	1.45	Probabilistic risk-targeted ground motion (0.2s)
SsUH	1.608	Factored uniform-hazard spectral acceleration (2% probability of exceedance in 50 years)
SsD	4.088	Factored deterministic acceleration value (0.2s)
S1RT	0.503	Probabilistic risk-targeted ground motion (1.0s)
S1UH	0.561	Factored uniform-hazard spectral acceleration (2% probability of exceedance in 50 years)
S1D	1.593	Factored deterministic acceleration value (1.0s)
PGAd	1.372	Factored deterministic acceleration value (PGA)

* See Section 11.4.8

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Disclaimer

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

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LONGITUDE
ONE TWENTY°
ENGINEERING & DESIGN

FRAMING CALCULATIONS

BEAM REFERENCE PER PLAN

FORTEWEB™ JOB SUMMARY REPORT

Forest Creek Lot 2

Roof			
Member Name	Results	Current Solution	Comments
RJ-1	Passed	1 piece(s) 11 7/8" TJI® 210 @ 24" OC	
RJ-2	Passed	1 piece(s) 11 7/8" TJI® 210 @ 16" OC	Cantilever Reinforcement (PB1) Required
RJ-3	Passed	1 piece(s) 11 7/8" TJI® 210 @ 24" OC	Cantilever Reinforcement (PB1) Required
2nd Floor			
Member Name	Results	Current Solution	Comments
2J-1 (Deck Joist)	Passed	1 piece(s) 2 x 10 DF No.2 @ 16" OC	
2B-1	Passed	1 piece(s) 5 1/4" x 14" 2.2E Parallam® PSL	
2B-2	Passed	1 piece(s) 5 1/4" x 18" 2.2E Parallam® PSL	
2B-3	Passed	1 piece(s) 3 1/2" x 18" 2.2E Parallam® PSL	
2B-4	Passed	1 piece(s) 5 1/4" x 18" 2.2E Parallam® PSL	
2B-4.1	Passed	1 piece(s) 3 1/2" x 18" 2.2E Parallam® PSL	
2B-5	Passed	1 piece(s) 5 1/4" x 18" 2.2E Parallam® PSL	
2B-6	Passed	1 piece(s) 5 1/4" x 18" 2.2E Parallam® PSL	
2B-6.1	Passed	1 piece(s) 5 1/4" x 9 1/2" 2.2E Parallam® PSL	
2B-7	Passed	1 piece(s) 5 1/2" x 16 1/2" 24F-V4 DF Glulam	
2H-1	Passed	1 piece(s) 4 x 10 DF No.1	
2H-1.1 (4x6 Check)	Failed	1 piece(s) 4 x 6 DF No.1 (Plank) 6x6 used on plan	
2H-1.1 (6x6 Check)	Passed	1 piece(s) 6 x 6 DF No.1	
2H-2	Passed	1 piece(s) 2 x 8 DF No.2	
2H-3 (High)	Passed	1 piece(s) 6 x 8 DF No.1	
2H-3 (Low)	Passed	1 piece(s) 6 x 6 DF No.1	
2H-4	Passed	1 piece(s) 4 x 6 DF No.2	
2H-5	Passed	2 piece(s) 2 x 8 DF No.2	
2H-6	Passed	1 piece(s) 5 1/2" x 7 1/2" 24F-V4 DF Glulam	
2H-7	Passed	2 piece(s) 2 x 8 DF No.2	
2H-8	Passed	3 piece(s) 2 x 8 DF No.2	
2B-8	Passed	1 piece(s) 5 1/4" x 18" 2.2E Parallam® PSL	
2B-9	Passed	1 piece(s) 3 1/2" x 18" 2.2E Parallam® PSL	
2B-10	Passed	1 piece(s) 5 1/4" x 18" 2.2E Parallam® PSL	
Low Roof			
Member Name	Results	Current Solution	Comments
LRJ-1	Passed	1 piece(s) 2 x 8 DF No.2 @ 16" OC	
LRB-1	Passed	1 piece(s) 4 x 8 DF No.2	
1st Floor			
Member Name	Results	Current Solution	Comments
1B-1 (Garage Header)	Passed	1 piece(s) 5 1/2" x 10 1/2" 24F-V4 DF Glulam	
1H-1	Passed	1 piece(s) 5 1/2" x 9" 24F-V4 DF Glulam	
1H-2	Passed	1 piece(s) 6 x 10 DF No.2	
1H-3	Passed	2 piece(s) 2 x 8 DF No.2	
1H-4	Passed	3 piece(s) 2 x 10 DF No.2	
1H-5	Passed	1 piece(s) 5 1/2" x 10 1/2" 24F-V4 DF Glulam	
Basement			
Member Name	Results	Current Solution	Comments
BB-1	Passed	1 piece(s) 5 1/4" x 18" 2.2E Parallam® PSL	
BH-1	Passed	1 piece(s) 3 1/2" x 7 1/2" 24F-V4 DF Glulam	
BH-2	Passed	2 piece(s) 2 x 8 DF No.2	

ForteWEB Software Operator	Job Notes
Harrison Kliegl L120 Engineering (425) 636-3313 hkliegl@l120engineering.com	

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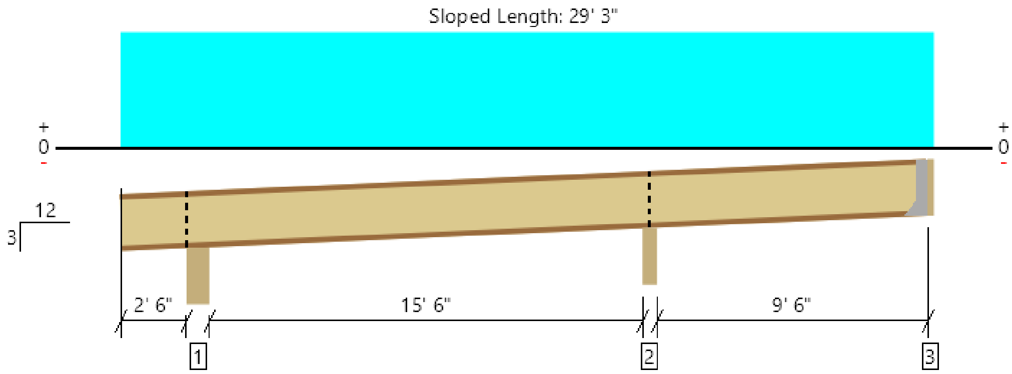
11/10/2022 4:12:44 PM UTC
ForteWEB v3.4
File Name: Forest Creek Lot 2

Roof, RJ-1

1 piece(s) 11 7/8" TJI® 210 @ 24" OC



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

Member Length : 29' 4 3/8"

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	1332 @ 18' 7 1/4"	2543 (3.50")	Passed (52%)	1.15	1.0 D + 1.0 S (Adj Spans)
Shear (lbs)	702 @ 18' 5 1/2"	1903	Passed (37%)	1.15	1.0 D + 1.0 S (Adj Spans)
Moment (Ft-lbs)	-1877 @ 18' 7 1/4"	4364	Passed (43%)	1.15	1.0 D + 1.0 S (Adj Spans)
Live Load Defl. (in)	0.164 @ 10' 9/16"	0.818	Passed (L/999+)	--	1.0 D + 1.0 S (Alt Spans)
Total Load Defl. (in)	0.256 @ 10' 5/16"	1.091	Passed (L/766)	--	1.0 D + 1.0 S (Alt Spans)

- Deflection criteria: LL (L/240) and TL (L/180).
- Overhang deflection criteria: LL (2L/240) and TL (2L/180).
- Allowed moment does not reflect the adjustment for the beam stability factor.

System : Roof
Member Type : Joist
Building Use : Residential
Building Code : IBC 2018
Design Methodology : ASD
Member Pitch : 3/12

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Snow	Factored	
1 - Beveled Plate - HF	5.50"	5.50"	3.50"	293	480	773	Blocking
2 - Beveled Plate - HF	3.50"	3.50"	3.50"	505	827	1332	Blocking
3 - Hanger on 11 7/8" HF ledgerOnMasonry	1.50"	Hanger[1]	1.75" / - [2]	80	180	260	See note [1]

- Blocking Panels are assumed to carry no loads applied directly above them and the full load is applied to the member being designed.
- At hanger supports, the Total Bearing dimension is equal to the width of the material that is supporting the hanger
- [1] See Connector grid below for additional information and/or requirements.
- [2] Required Bearing Length / Required Bearing Length with Web Stiffeners

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

- TJI joists are only analyzed using Maximum Allowable bracing solutions.
- Maximum allowable bracing intervals based on applied load.

Connector: Simpson Strong-Tie						
Support	Model	Seat Length	Top Fasteners	Face Fasteners	Member Fasteners	Accessories
3 - Face Mount Hanger	Connector not found	N/A	N/A	N/A	N/A	

- Refer to manufacturer notes and instructions for proper installation and use of all connectors.

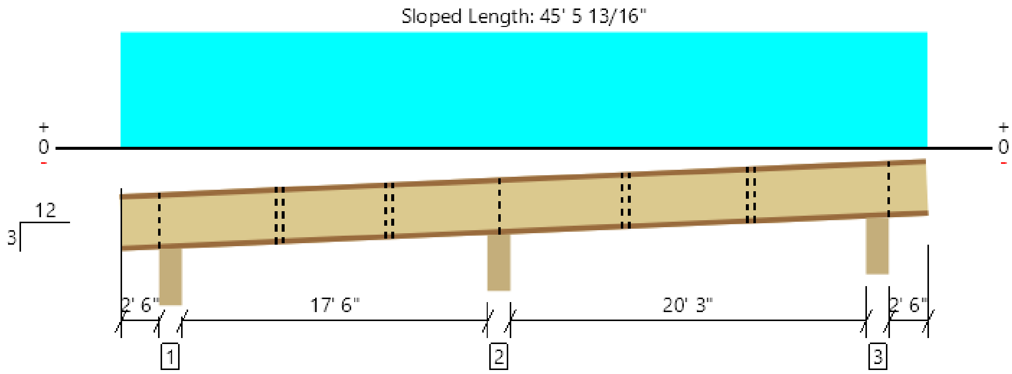
Vertical Load	Location	Spacing	Dead (0.90)	Snow (1.15)	Comments
1 - Uniform (PSF)	0 to 28' 4 1/2"	24"	15.0	25.0	Roof Load

Weyerhaeuser Notes

ForteWEB Software Operator	Job Notes
Harrison Kliegl L120 Engineering (425) 636-3313 hkliegl@l120engineering.com	

Roof, RJ-2

1 piece(s) 11 7/8" TJI® 210 @ 16" OC



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

Member Length : 45' 8 3/4"

System : Roof
Member Type : Joist
Building Use : Residential
Building Code : IBC 2018
Design Methodology : ASD
Member Pitch : 3/12

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	1287 @ 20' 8 1/4"	3041 (5.25")	Passed (42%)	1.15	1.0 D + 1.0 S (Adj Spans)
Shear (lbs)	640 @ 20' 11"	1903	Passed (34%)	1.15	1.0 D + 1.0 S (Adj Spans)
Moment (Ft-lbs)	-2489 @ 20' 8 1/4"	4364	Passed (57%)	1.15	1.0 D + 1.0 S (Adj Spans)
Live Load Defl. (in)	0.299 @ 31' 10 3/4"	1.067	Passed (L/857)	--	1.0 D + 1.0 S (Alt Spans)
Total Load Defl. (in)	0.450 @ 32' 1/8"	1.423	Passed (L/569)	--	1.0 D + 1.0 S (Alt Spans)

- Deflection criteria: LL (L/240) and TL (L/180).
- Overhang deflection criteria: LL (2L/240) and TL (2L/180).
- Allowed moment does not reflect the adjustment for the beam stability factor.
- Permanent bracing at third points in the back span or a direct applied ceiling over the entire back span length is required at the left and right span of the member. See literature detail (PB1) For clarification.

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Snow	Factored	
1 - Beveled Plate - HF	5.50"	5.50"	3.50"	193	339	533	Blocking
2 - Beveled Plate - HF	5.50"	5.50"	3.50"	488	799	1287	Blocking
3 - Beveled Plate - HF	5.50"	5.50"	3.50"	228	384	612	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)	5' 6" o/c	
Bottom Edge (Lu)	4' 8" o/c	

•TJI joists are only analyzed using Maximum Allowable bracing solutions.
•Maximum allowable bracing intervals based on applied load.

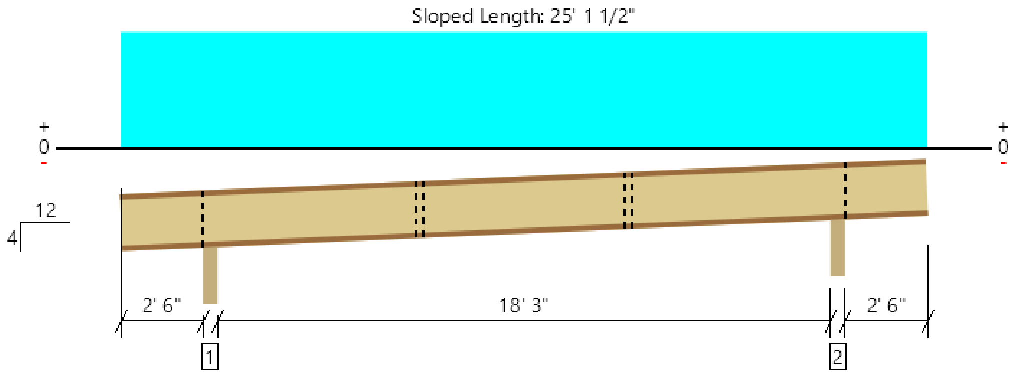
Vertical Load	Location	Spacing	Dead (0.90)	Snow (1.15)	Comments
1 - Uniform (PSF)	0 to 44' 1 1/2"	16"	15.0	25.0	Roof Load

Weyerhaeuser Notes

ForteWEB Software Operator	Job Notes
Harrison Kliegl L120 Engineering (425) 636-3313 hkliegl@l120engineering.com	

Roof, RJ-3

1 piece(s) 11 7/8" TJI® 210 @ 24" OC



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

Member Length : 25' 5 7/16"

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	977 @ 2' 7 3/4"	2600 (3.50")	Passed (38%)	1.15	1.0 D + 1.0 S (Adj Spans)
Shear (lbs)	717 @ 2' 9 1/2"	1903	Passed (38%)	1.15	1.0 D + 1.0 S (Adj Spans)
Moment (Ft-lbs)	3309 @ 11' 11"	4364	Passed (76%)	1.15	1.0 D + 1.0 S (Alt Spans)
Live Load Defl. (in)	0.485 @ 11' 11"	0.977	Passed (L/484)	--	1.0 D + 1.0 S (Alt Spans)
Total Load Defl. (in)	0.777 @ 11' 11"	1.303	Passed (L/302)	--	1.0 D + 1.0 S (Alt Spans)

- Deflection criteria: LL (L/240) and TL (L/180).
- Overhang deflection criteria: LL (2L/240) and TL (2L/180).
- Allowed moment does not reflect the adjustment for the beam stability factor.
- Permanent bracing at third points in the back span or a direct applied ceiling over the entire back span length is required at the left and right span of the member. See literature detail (PB1) For clarification.

System : Roof
Member Type : Joist
Building Use : Residential
Building Code : IBC 2018
Design Methodology : ASD
Member Pitch : 4/12

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Snow	Factored	
1 - Beveled Plate - HF	3.50"	3.50"	3.50"	377	601	977	Blocking
2 - Beveled Plate - HF	3.50"	3.50"	3.50"	377	601	977	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)	3' 11" o/c	
Bottom Edge (Lu)	8' 8" o/c	

•TJI joists are only analyzed using Maximum Allowable bracing solutions.
•Maximum allowable bracing intervals based on applied load.

Vertical Load	Location	Spacing	Dead (0.90)	Snow (1.15)	Comments
1 - Uniform (PSF)	0 to 23' 10"	24"	15.0	25.0	Roof Load

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

ForteWEB Software Operator	Job Notes
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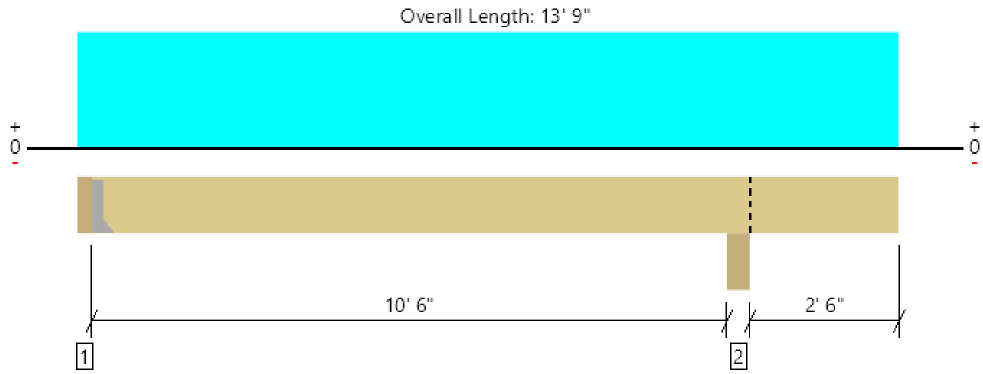
2nd Floor, 2J-1 (Deck Joist)

1 piece(s) 2 x 10 DF 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)	585 @ 3 1/2"	1406 (1.50")	Passed (42%)	--	1.0 D + 0.75 L + 0.75 S (Alt Spans)
Shear (lbs)	503 @ 10' 1/4"	1665	Passed (30%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	1486 @ 5' 6 13/16"	2029	Passed (73%)	1.00	1.0 D + 1.0 L (Alt Spans)
Live Load Defl. (in)	0.156 @ 5' 7 11/16"	0.268	Passed (L/823)	--	1.0 D + 0.75 L + 0.75 S (Alt Spans)
Total Load Defl. (in)	0.199 @ 5' 7 3/8"	0.536	Passed (L/647)	--	1.0 D + 0.75 L + 0.75 S (Alt Spans)
TJ-Pro™ Rating	N/A	N/A	N/A	--	N/A

System : Floor
Member Type : Joist
Building Use : Residential
Building Code : IBC 2018
Design Methodology : ASD

- Deflection criteria: LL (L/480) and TL (L/240).
- Overhang deflection criteria: LL (2L/480) and TL (2L/240).
- 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	Snow	Factored	
1 - Hanger on 9 1/4" DF beam	3.50"	Hanger[1]	1.50"	142	453/-4	183	618	See note [1]
2 - Beam - GLB	5.50"	5.50"	1.50"	225	675	281	943	Blocking

- Blocking Panels are assumed to carry no loads applied directly above them and the full load is applied to the member being designed.
- At hanger supports, the Total Bearing dimension is equal to the width of the material that is supporting the hanger
- [1] See Connector grid below for additional information and/or requirements.

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	Continuous	
Bottom Edge (Lu)	End Bearing Points	

Connector: Simpson Strong-Tie

Support	Model	Seat Length	Top Fasteners	Face Fasteners	Member Fasteners	Accessories
1 - Face Mount Hanger	LU28	1.50"	N/A	8-10dx1.5	6-10dx1.5	

- Refer to manufacturer notes and instructions for proper installation and use of all connectors.

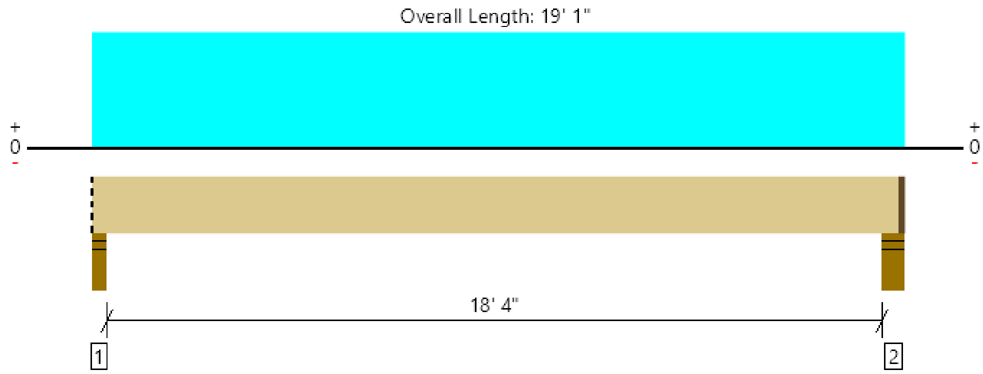
Vertical Load	Location (Side)	Spacing	Dead (0.90)	Floor Live (1.00)	Snow (1.15)	Comments
1 - Uniform (PSF)	0 to 13' 9"	16"	20.0	60.0	25.0	Deck Load

Weyerhaeuser Notes

ForteWEB Software Operator	Job Notes
Harrison Kliegl L120 Engineering (425) 636-3313 hkliegl@l120engineering.com	

2nd Floor, 2B-1

1 piece(s) 5 1/4" x 14" 2.2E Parallam® PSL



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)	5419 @ 2"	7442 (3.50")	Passed (73%)	--	1.0 D + 1.0 L (All Spans)
Shear (lbs)	4584 @ 1' 5 1/2"	14210	Passed (32%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	24733 @ 9' 5 1/2"	40743	Passed (61%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.431 @ 9' 5 1/2"	0.465	Passed (L/517)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.617 @ 9' 5 1/2"	0.929	Passed (L/361)	--	1.0 D + 1.0 L (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 2018
Design Methodology : ASD

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Floor Live	Factored	
1 - Stud wall - HF	3.50"	3.50"	2.55"	1636	3783	5419	Blocking
2 - Stud wall - HF	5.50"	4.00"	2.56"	1662	3850	5512	1 1/2" Rim Board

- Rim Board is assumed to carry all loads applied directly above it, bypassing the member being designed.
- 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)	18' 11" o/c	
Bottom Edge (Lu)	18' 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)	0 to 18' 11 1/2"	N/A	23.0	--	
1 - Uniform (PSF)	0 to 19' 1" (Front)	10'	15.0	40.0	Floor Load

Weyerhaeuser Notes

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

ForteWEB Software Operator	Job Notes
Harrison Kliegl L120 Engineering (425) 636-3313 hkliegl@l120engineering.com	

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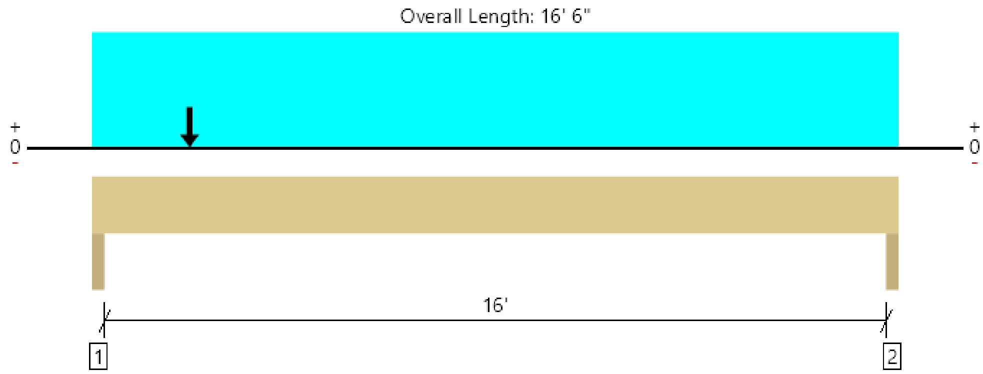
MEMBER REPORT

PASSED

2nd Floor, 2B-2

1 piece(s) 5 1/4" x 18" 2.2E Parallam® PSL



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)	7718 @ 1 1/2"	9844 (3.00")	Passed (78%)	--	1.0 D + 1.0 L (All Spans)
Shear (lbs)	7115 @ 1' 9"	18270	Passed (39%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	17127 @ 6' 4 7/8"	65252	Passed (26%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.072 @ 7' 7 1/4"	0.542	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.167 @ 7' 10 3/8"	0.813	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)

- Deflection criteria: LL (L/360) and TL (L/240).
- A 4.8% decrease in the moment capacity has been added to account for lateral stability.

System : Floor
Member Type : Drop Beam
Building Use : Residential
Building Code : IBC 2018
Design Methodology : ASD

Supports	Bearing Length			Loads to Supports (lbs)				Accessories
	Total	Available	Required	Dead	Floor Live	Snow	Factored	
1 - Trimmer - HF	3.00"	3.00"	2.35"	3488	4231	619	7718	None
2 - Trimmer - HF	3.00"	3.00"	1.50"	2209	1269	619	3625	None

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	End Bearing Points	
Bottom Edge (Lu)	End Bearing Points	

Vertical Loads	Location (Side)	Tributary Width	Dead (0.90)	Floor Live (1.00)	Snow (1.15)	Comments
0 - Self Weight (PLF)	0 to 16' 6"	N/A	29.5	--	--	
1 - Uniform (PSF)	0 to 16' 6" (Front)	1'	15.0	40.0	-	Floor Load
2 - Uniform (PSF)	0 to 16' 6" (Back)	1'	20.0	60.0	25.0	Deck Load
3 - Uniform (PLF)	0 to 16' 6" (Top)	N/A	150.0	-	-	Wall Load Above
4 - Uniform (PSF)	0 to 16' 6" (Top)	2'	15.0	-	25.0	Roof Load From Above
5 - Point (lb)	2' (Top)	N/A	1662	3850	-	Linked from: 2B-1, Support 2

Weyerhaeuser Notes

ForteWEB Software Operator	Job Notes
Harrison Kliegl L120 Engineering (425) 636-3313 hkliegl@l120engineering.com	

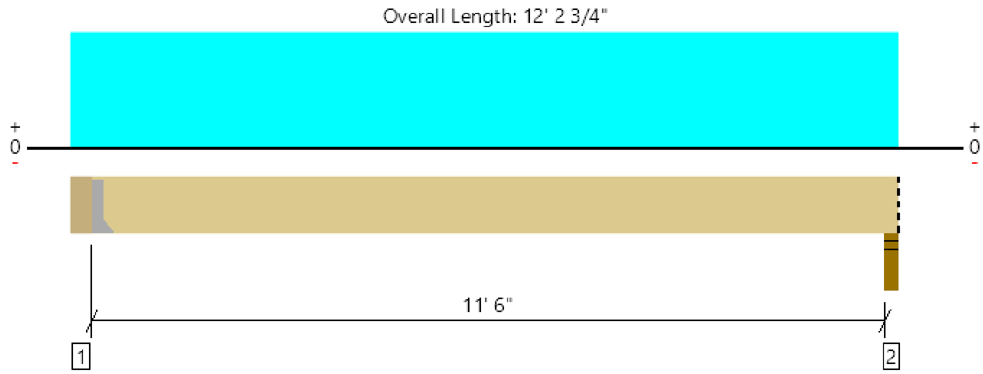
Weyerhaeuser

MEMBER REPORT

PASSED

2nd Floor, 2B-3

1 piece(s) 3 1/2" x 18" 2.2E Parallam® PSL



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)	1576 @ 5 1/4"	3281 (1.50")	Passed (48%)	--	1.0 D + 0.75 L + 0.75 S (All Spans)
Shear (lbs)	1094 @ 1' 11 1/4"	12180	Passed (9%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	4285 @ 6' 3"	43665	Passed (10%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.013 @ 6' 3"	0.291	Passed (L/999+)	--	1.0 D + 0.75 L + 0.75 S (All Spans)
Total Load Defl. (in)	0.037 @ 6' 3"	0.581	Passed (L/999+)	--	1.0 D + 0.75 L + 0.75 S (All Spans)

System : Floor
Member Type : Flush Beam
Building Use : Residential
Building Code : IBC 2018
Design Methodology : ASD

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

Supports	Bearing Length			Loads to Supports (lbs)				Accessories
	Total	Available	Required	Dead	Floor Live	Snow	Factored	
1 - Hanger on 18" PSL beam	5.25"	Hanger[1]	1.50"	1077	500	313	1686	See note [1]
2 - Stud wall - HF	3.50"	3.50"	1.50"	1039	478	299	1621	Blocking

- Blocking Panels are assumed to carry no loads applied directly above them and the full load is applied to the member being designed.
- At hanger supports, the Total Bearing dimension is equal to the width of the material that is supporting the hanger
- [1] See Connector grid below for additional information and/or requirements.

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	11' 10" o/c	
Bottom Edge (Lu)	11' 10" o/c	

- Maximum allowable bracing intervals based on applied load.

Connector: Simpson Strong-Tie						
Support	Model	Seat Length	Top Fasteners	Face Fasteners	Member Fasteners	Accessories
1 - Face Mount Hanger	THA413	1.75"	N/A	14-10d	4-10d	

- Refer to manufacturer notes and instructions for proper installation and use of all connectors.

Vertical Loads	Location (Side)	Tributary Width	Dead (0.90)	Floor Live (1.00)	Snow (1.15)	Comments
0 - Self Weight (PLF)	5 1/4" to 12' 2 3/4"	N/A	19.7	--	--	
1 - Uniform (PSF)	0 to 12' 2 3/4" (Front)	2'	12.0	40.0	-	Floor Load
2 - Uniform (PLF)	0 to 12' 2 3/4" (Top)	N/A	100.0	-	-	Wall Load Above
3 - Uniform (PSF)	0 to 12' 2 3/4" (Top)	2'	15.0	-	25.0	Roof Load

Weyerhaeuser Notes

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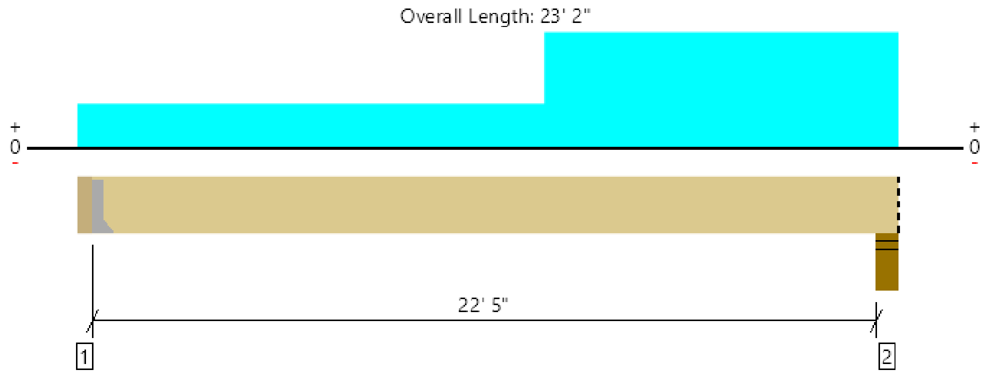
The product application, input design loads, dimensions and support information have been provided by ForteWEB Software Operator

ForteWEB Software Operator	Job Notes
Harrison Kliegl L120 Engineering (425) 636-3313 hkliegl@l120engineering.com	

Weyerhaeuser

2nd Floor, 2B-4

1 piece(s) 5 1/4" x 18" 2.2E Parallam® PSL



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)	5734 @ 3 1/2"	5734 (1.75")	Passed (100%)	--	1.0 D + 0.75 L + 0.75 S (All Spans)
Shear (lbs)	7624 @ 21' 2 1/2"	21011	Passed (36%)	1.15	1.0 D + 0.75 L + 0.75 S (All Spans)
Moment (Ft-lbs)	41991 @ 13' 9 9/16"	75322	Passed (56%)	1.15	1.0 D + 0.75 L + 0.75 S (All Spans)
Live Load Defl. (in)	0.394 @ 11' 10 1/2"	0.564	Passed (L/686)	--	1.0 D + 0.75 L + 0.75 S (All Spans)
Total Load Defl. (in)	0.711 @ 12'	1.127	Passed (L/380)	--	1.0 D + 0.75 L + 0.75 S (All Spans)

System : Floor
Member Type : Flush Beam
Building Use : Residential
Building Code : IBC 2018
Design Methodology : ASD

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

Supports	Bearing Length			Loads to Supports (lbs)				Accessories
	Total	Available	Required	Dead	Floor Live	Snow	Factored	
1 - Hanger on 18" PSL beam	3.50"	Hanger¹	1.75"	2344	3374	1285	5839	See note ¹
2 - Stud wall - HF	5.50"	5.50"	4.53"	4639	1860	4807	9639	Blocking

- Blocking Panels are assumed to carry no loads applied directly above them and the full load is applied to the member being designed.
- At hanger supports, the Total Bearing dimension is equal to the width of the material that is supporting the hanger
- ¹ See Connector grid below for additional information and/or requirements.

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	22' 11" o/c	
Bottom Edge (Lu)	22' 11" o/c	

•Maximum allowable bracing intervals based on applied load.

Connector: Simpson Strong-Tie						
Support	Model	Seat Length	Top Fasteners	Face Fasteners	Member Fasteners	Accessories
1 - Face Mount Hanger	HGUS5.50/14	4.00"	N/A	66-10d	22-10d	

- Refer to manufacturer notes and instructions for proper installation and use of all connectors.

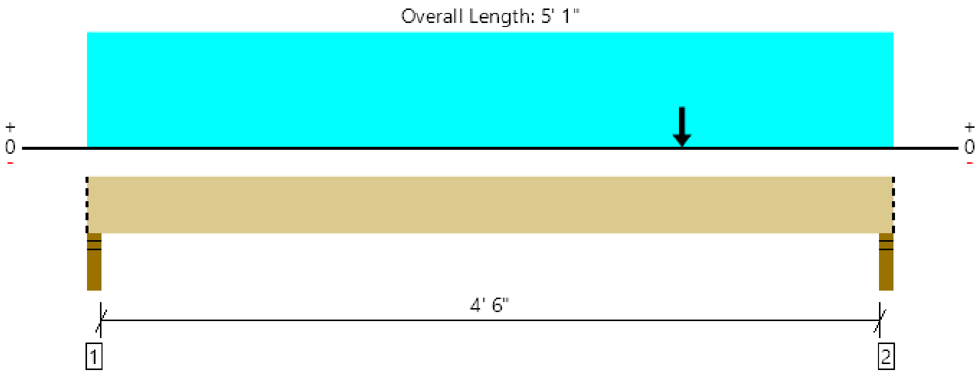
Vertical Loads	Location (Side)	Tributary Width	Dead (0.90)	Floor Live (1.00)	Snow (1.15)	Comments
0 - Self Weight (PLF)	3 1/2" to 23' 2"	N/A	29.5	--	--	
1 - Uniform (PSF)	0 to 13' (Front)	8' 6"	12.0	40.0	-	Floor Load
2 - Uniform (PSF)	13' to 23' 2" (Back)	2'	12.0	40.0	-	Floor Load
3 - Uniform (PLF)	13' to 23' 2" (Top)	N/A	100.0	-	-	Wall Load Above
4 - Uniform (PLF)	13' to 23' 2" (Top)	N/A	366.0	-	599.3	Linked from: RJ-2, Support 2

Weyerhaeuser Notes

ForteWEB Software Operator	Job Notes
Harrison Kliegl L120 Engineering (425) 636-3313 hkliegl@l120engineering.com	

2nd Floor, 2B-4.1

1 piece(s) 3 1/2" x 18" 2.2E Parallam® PSL



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)	4668 @ 4' 11"	4961 (3.50")	Passed (94%)	--	1.0 D + 0.75 L + 0.75 S (All Spans)
Shear (lbs)	2659 @ 3' 3 1/2"	12180	Passed (22%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	5291 @ 3' 9"	43665	Passed (12%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.007 @ 3' 9"	0.119	Passed (L/999+)	--	1.0 D + 0.75 L + 0.75 S (All Spans)
Total Load Defl. (in)	0.012 @ 3' 9"	0.237	Passed (L/999+)	--	1.0 D + 0.75 L + 0.75 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 2018
Design Methodology : ASD

Supports	Bearing Length			Loads to Supports (lbs)				Accessories
	Total	Available	Required	Dead	Floor Live	Snow	Factored	
1 - Stud wall - HF	3.50"	3.50"	1.50"	687	1032	316	1719	Blocking
2 - Stud wall - HF	3.50"	3.50"	3.29"	1879	2749	969	4668	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)	5' 1" o/c	
Bottom Edge (Lu)	5' 1" o/c	

•Maximum allowable bracing intervals based on applied load.

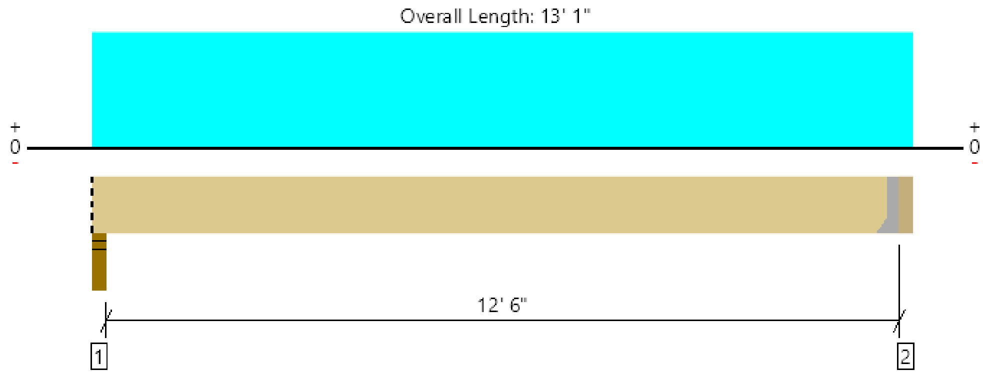
Vertical Loads	Location (Side)	Tributary Width	Dead (0.90)	Floor Live (1.00)	Snow (1.15)	Comments
0 - Self Weight (PLF)	0 to 5' 1"	N/A	19.7	--	--	
1 - Uniform (PSF)	0 to 5' 1" (Front)	2'	12.0	40.0	-	Floor Load
2 - Point (lb)	3' 9" (Front)	N/A	2344	3374	1285	Linked from: 2B-4, Support 1

Weyerhaeuser Notes

ForteWEB Software Operator	Job Notes
Harrison Kliegl L120 Engineering (425) 636-3313 hkliegl@l120engineering.com	

2nd Floor, 2B-5

1 piece(s) 5 1/4" x 18" 2.2E Parallam® PSL



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)	4073 @ 12' 9 1/2"	4922 (1.50")	Passed (83%)	--	1.0 D + 0.75 L + 0.75 S (All Spans)
Shear (lbs)	3105 @ 11' 3 1/2"	21011	Passed (15%)	1.15	1.0 D + 0.75 L + 0.75 S (All Spans)
Moment (Ft-lbs)	12854 @ 6' 5 3/4"	75322	Passed (17%)	1.15	1.0 D + 0.75 L + 0.75 S (All Spans)
Live Load Defl. (in)	0.042 @ 6' 5 3/4"	0.316	Passed (L/999+)	--	1.0 D + 0.75 L + 0.75 S (All Spans)
Total Load Defl. (in)	0.080 @ 6' 5 3/4"	0.631	Passed (L/999+)	--	1.0 D + 0.75 L + 0.75 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 2018
Design Methodology : ASD

Supports	Bearing Length			Loads to Supports (lbs)				Accessories
	Total	Available	Required	Dead	Floor Live	Snow	Factored	
1 - Stud wall - HF	3.50"	3.50"	1.97"	2005	1361	1539	4180	Blocking
2 - Hanger on 18" PSL beam	3.50"	Hanger¹	1.50"	2036	1387	1568	4252	See note ¹

- Blocking Panels are assumed to carry no loads applied directly above them and the full load is applied to the member being designed.
- At hanger supports, the Total Bearing dimension is equal to the width of the material that is supporting the hanger
- ¹ See Connector grid below for additional information and/or requirements.

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

•Maximum allowable bracing intervals based on applied load.

Connector: Simpson Strong-Tie

Support	Model	Seat Length	Top Fasteners	Face Fasteners	Member Fasteners	Accessories
2 - Face Mount Hanger	HU616	2.50"	N/A	26-16d	12-16d	

- Refer to manufacturer notes and instructions for proper installation and use of all connectors.

Vertical Loads	Location (Side)	Tributary Width	Dead (0.90)	Floor Live (1.00)	Snow (1.15)	Comments
0 - Self Weight (PLF)	0 to 12' 9 1/2"	N/A	29.5	--	--	
1 - Uniform (PSF)	0 to 13' 1" (Back)	5' 3"	12.0	40.0	-	Floor Load
2 - Uniform (PSF)	0 to 13' 1" (Front)	1'	15.0	-	25.0	Low Roof Load
3 - Uniform (PLF)	0 to 13' 1" (Top)	N/A	100.0	-	-	Wall Load Above
4 - Uniform (PSF)	0 to 13' 1" (Top)	8' 6"	12.0	-	25.0	Roof Load

Weyerhaeuser Notes

ForteWEB Software Operator	Job Notes
Harrison Kliegl L120 Engineering (425) 636-3313 hkliegl@l120engineering.com	

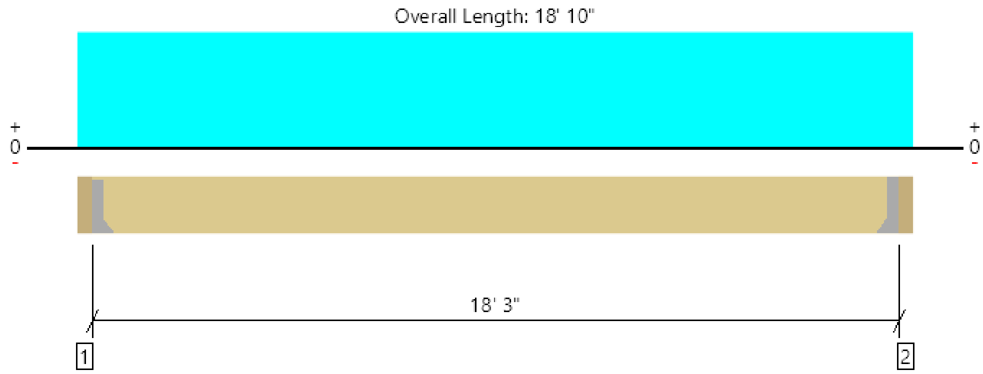
FORTEWEB

MEMBER REPORT

PASSED

2nd Floor, 2B-6

1 piece(s) 5 1/4" x 18" 2.2E Parallam® PSL



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)	4474 @ 3 1/2"	4922 (1.50")	Passed (91%)	--	1.0 D + 0.75 L + 0.75 S (All Spans)
Shear (lbs)	3443 @ 1' 9 1/2"	18270	Passed (19%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	18798 @ 9' 5"	65497	Passed (29%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.127 @ 9' 5"	0.456	Passed (L/999+)	--	1.0 D + 0.75 L + 0.75 S (All Spans)
Total Load Defl. (in)	0.241 @ 9' 5"	0.913	Passed (L/910)	--	1.0 D + 0.75 L + 0.75 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 2018
Design Methodology : ASD

Supports	Bearing Length			Loads to Supports (lbs)				Accessories
	Total	Available	Required	Dead	Floor Live	Snow	Factored	
1 - Hanger on 18" PSL beam	3.50"	Hanger[1]	1.50"	2172	2072	1177	4608	See note [1]
2 - Hanger on 18" PSL beam	3.50"	Hanger[1]	1.50"	2172	2072	1177	4608	See note [1]

- At hanger supports, the Total Bearing dimension is equal to the width of the material that is supporting the hanger
- [1] See Connector grid below for additional information and/or requirements.

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	18' 3" o/c	
Bottom Edge (Lu)	18' 3" o/c	

•Maximum allowable bracing intervals based on applied load.

Connector: Simpson Strong-Tie						
Support	Model	Seat Length	Top Fasteners	Face Fasteners	Member Fasteners	Accessories
1 - Face Mount Hanger	HGUS5.50/14	4.00"	N/A	66-10d	22-10d	
2 - Face Mount Hanger	HGUS5.50/14	4.00"	N/A	66-10d	22-10d	

- Refer to manufacturer notes and instructions for proper installation and use of all connectors.

Vertical Loads	Location (Side)	Tributary Width	Dead (0.90)	Floor Live (1.00)	Snow (1.15)	Comments
0 - Self Weight (PLF)	3 1/2" to 18' 6 1/2"	N/A	29.5	--	--	
1 - Uniform (PSF)	0 to 18' 10" (Front)	1'	12.0	40.0	-	Floor Load
2 - Uniform (PSF)	0 to 18' 10" (Back)	3'	20.0	60.0	25.0	Deck Load
3 - Uniform (PLF)	0 to 18' 10" (Top)	N/A	100.0	-	-	Wall Load Above
4 - Uniform (PSF)	0 to 18' 10" (Top)	2'	15.0	-	25.0	Roof Load

Weyerhaeuser Notes

ForteWEB Software Operator	Job Notes
Harrison Kliegl L120 Engineering (425) 636-3313 hkliegl@l120engineering.com	

Weyerhaeuser

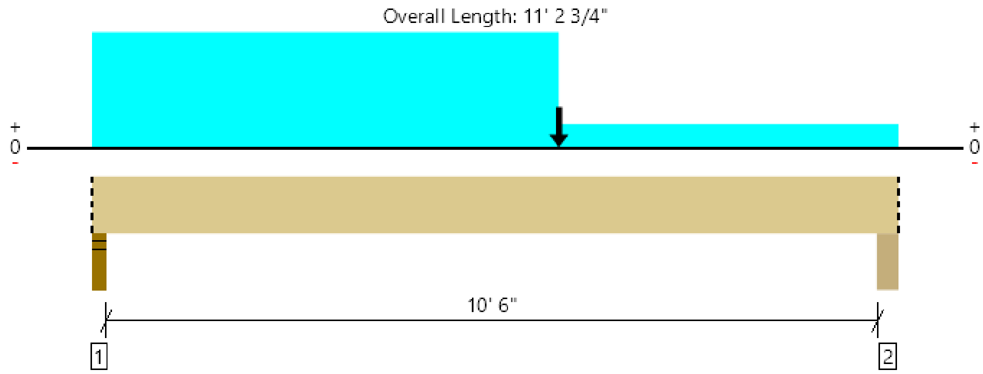
11/10/2022 4:12:44 PM UTC

ForteWEB v3.4, Engine: V8.2.2.122, Data: V8.1.3.0

File Name: Forest Creek Lot 2

2nd Floor, 2B-6.1

1 piece(s) 5 1/4" x 9 1/2" 2.2E Parallam® PSL



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)	4312 @ 2"	7442 (3.50")	Passed (58%)	--	1.0 D + 1.0 L (All Spans)
Shear (lbs)	3747 @ 10'	9643	Passed (39%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	16006 @ 6' 6"	19585	Passed (82%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.227 @ 5' 6 3/4"	0.269	Passed (L/569)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.389 @ 5' 7 3/8"	0.538	Passed (L/332)	--	1.0 D + 1.0 L (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 2018
Design Methodology : ASD

Supports	Bearing Length			Loads to Supports (lbs)				Accessories
	Total	Available	Required	Dead	Floor Live	Snow	Factored	
1 - Stud wall - HF	3.50"	3.50"	2.03"	1553	2759	506	4312	Blocking
2 - Beam - HF	5.25"	5.25"	1.84"	1669	2196	789	3908	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)	11' 3" o/c	
Bottom Edge (Lu)	11' 3" o/c	

- Maximum allowable bracing intervals based on applied load.

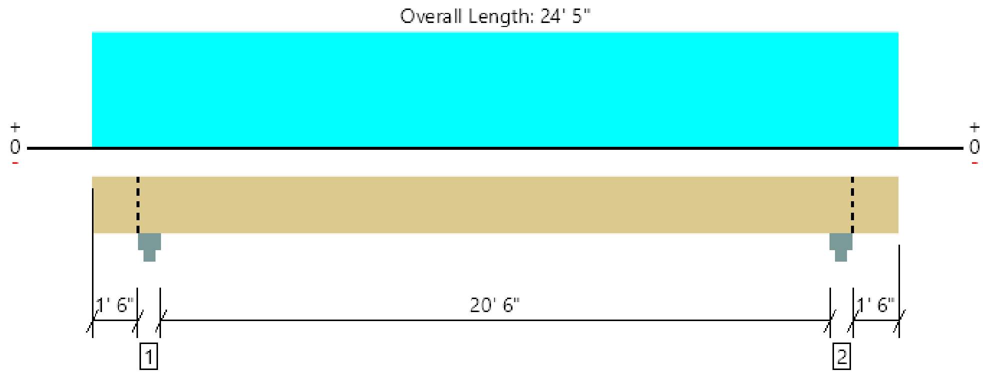
Vertical Loads	Location (Side)	Tributary Width	Dead (0.90)	Floor Live (1.00)	Snow (1.15)	Comments
0 - Self Weight (PLF)	0 to 11' 2 3/4"	N/A	15.6	--	--	
1 - Uniform (PSF)	0 to 6' 6" (Front)	10'	12.0	40.0	-	Floor Load
2 - Uniform (PSF)	6' 6" to 11' 2 3/4" (Front)	1'	20.0	60.0	25.0	Deck Load
3 - Point (lb)	6' 6" (Front)	N/A	2172	2072	1177	Linked from: 2B-6, Support 1

Weyerhaeuser Notes

ForteWEB Software Operator	Job Notes
Harrison Kliegl L120 Engineering (425) 636-3313 hkliegl@l120engineering.com	

2nd Floor, 2B-7

1 piece(s) 5 1/2" x 16 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)	6427 @ 1' 8 3/4"	19663 (5.50")	Passed (33%)	--	1.0 D + 0.75 L + 0.75 S (Adj Spans)
Shear (lbs)	4481 @ 3' 4"	16033	Passed (28%)	1.00	1.0 D + 1.0 L (Adj Spans)
Pos Moment (Ft-lbs)	27354 @ 12' 2 1/2"	48036	Passed (57%)	1.00	1.0 D + 1.0 L (Alt Spans)
Neg Moment (Ft-lbs)	-751 @ 1' 8 3/4"	38474	Passed (2%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.446 @ 12' 2 1/2"	0.524	Passed (L/564)	--	1.0 D + 0.75 L + 0.75 S (Alt Spans)
Total Load Defl. (in)	0.607 @ 12' 2 1/2"	1.048	Passed (L/414)	--	1.0 D + 0.75 L + 0.75 S (Alt Spans)

System : Floor
Member Type : Flush Beam
Building Use : Residential
Building Code : IBC 2018
Design Methodology : ASD

- Deflection criteria: LL (L/480) and TL (L/240).
- Overhang deflection criteria: LL (2L/480) and TL (2L/240).
- Allowed moment does not reflect the adjustment for the beam stability factor.
- Critical positive moment adjusted by a volume factor of 0.96 that was calculated using length L = 20' 10 1/2".
- Critical negative moment adjusted by a volume factor of 1.00 that was calculated using length L = 1' 10 1/2".
- 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	Snow	Factored	
1 - Column Cap - steel	5.50"	5.50"	1.80"	1734	4421	1837	6427	Blocking
2 - Column Cap - steel	5.50"	5.50"	1.80"	1734	4421	1837	6427	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)	24' 5" o/c	
Bottom Edge (Lu)	24' 5" 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 24' 5"	N/A	22.1	--	--	
1 - Uniform (PSF)	0 to 24' 5" (Front)	6'	20.0	60.0	25.0	Deck Load

Weyerhaeuser Notes

ForteWEB Software Operator	Job Notes
Harrison Kliegl L120 Engineering (425) 636-3313 hkliegl@l120engineering.com	

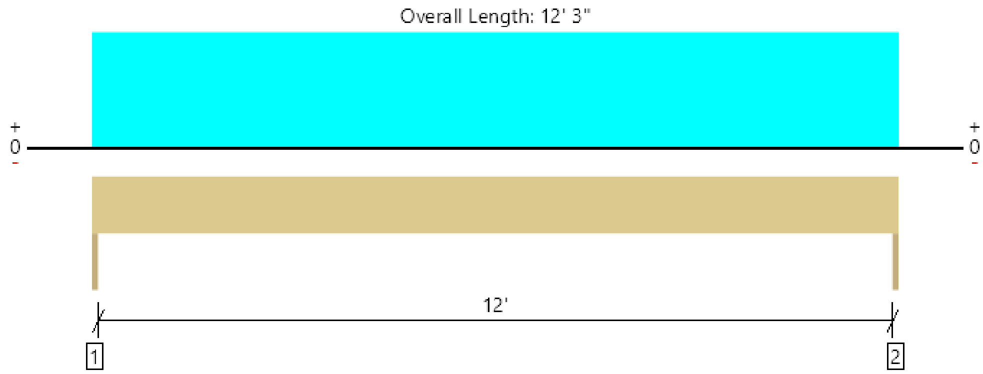
FORTEWEB

MEMBER REPORT

PASSED

2nd Floor, 2H-1

1 piece(s) 4 x 10 DF No.1



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)	1030 @ 0	3281 (1.50")	Passed (31%)	--	1.0 D + 1.0 S (All Spans)
Shear (lbs)	880 @ 10 3/4"	4468	Passed (20%)	1.15	1.0 D + 1.0 S (All Spans)
Moment (Ft-lbs)	3155 @ 6' 1 1/2"	5577	Passed (57%)	1.15	1.0 D + 1.0 S (All Spans)
Vert Live Load Defl. (in)	0.129 @ 6' 1 1/2"	0.408	Passed (L/999+)	--	1.0 D + 1.0 S (All Spans)
Vert Total Load Defl. (in)	0.217 @ 6' 1 1/2"	0.613	Passed (L/677)	--	1.0 D + 1.0 S (All Spans)
Lat Member Reaction (lbs)	176 @ 12' 3"	N/A	Passed (N/A)	1.60	1.0 D + 0.6 W
Lat Shear (lbs)	164 @ 5"	6216	Passed (3%)	1.60	1.0 D + 0.6 W
Lat Moment (Ft-lbs)	539 @ mid-span	3324	Passed (16%)	1.60	1.0 D + 0.6 W
Lat Deflection (in)	0.182 @ mid-span	1.225	Passed (L/810)	--	1.0 D + 0.6 W
Bi-Axial Bending	0.48	1.00	Passed (48%)	1.60	1.0 D + 0.45 W + 0.75 L + 0.75 S

System : Wall
Member Type : Header
Building Use : Residential
Building Code : IBC 2018
Design Methodology : ASD

- Deflection criteria: LL (L/360) and TL (L/240).
- Lateral deflection criteria: Wind (L/120)
- A 2.8% decrease in the moment capacity has been added to account for lateral stability.
- Applicable calculations are based on NDS.

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Snow	Factored	
1 - Trimmer - HF	1.50"	1.50"	1.50"	418	613	1030	None
2 - Trimmer - HF	1.50"	1.50"	1.50"	418	613	1030	None

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	End Bearing Points	
Bottom Edge (Lu)	End Bearing Points	

Lateral Connections

Supports	Plate Size	Plate Material	Connector	Type/Model	Quantity	Nailing
Left	2X	Hem Fir	Nails	10d (0.128" x 3") (End)	3	
Right	2X	Hem Fir	Nails	10d (0.128" x 3") (End)	3	

Vertical Loads	Location	Tributary Width	Dead (0.90)	Snow (1.15)	Comments
0 - Self Weight (PLF)	0 to 12' 3"	N/A	8.2	--	
1 - Uniform (PSF)	0 to 12' 3"	4'	15.0	25.0	Roof Load

Lateral Load	Location	Tributary Width	Wind (1.60)	Comments
1 - Uniform (PSF)	Full Length	2'	24.0	

• ASCE/SEI 7 Sec. 30.4: Exposure Category (B), Mean Roof Height (33'), Topographic Factor (1.0), Wind Directionality Factor (0.85), Basic Wind Speed (115), Risk Category(II), Effective Wind Area determined using full member span and trib. width.
• IBC Table 1604.3, footnote f: Deflection checks are performed using 42% of this lateral wind load.

ForteWEB Software Operator	Job Notes
Harrison Kliegl L120 Engineering (425) 636-3313 hkliegl@l120engineering.com	

Weyerhaeuser

11/10/2022 4:12:44 PM UTC

ForteWEB v3.4, Engine: V8.2.2.122, Data: V8.1.3.0

File Name: Forest Creek Lot 2

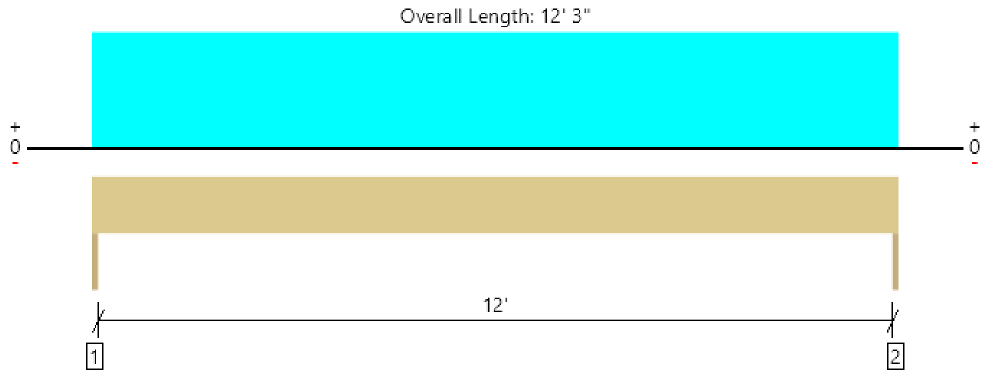
FORTEWEB

MEMBER REPORT

FAILED

2nd Floor, 2H-1.1 (4x6 Check)

1 piece(s) 4 x 6 DF No.1 (Plank)



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)	152 @ 0	5156 (1.50")	Passed (3%)	--	1.0 D (All Spans)
Shear (lbs)	142 @ 5"	2079	Passed (7%)	0.90	1.0 D (All Spans)
Moment (Ft-lbs)	467 @ 6' 1 1/2"	1150	Passed (41%)	0.90	1.0 D (All Spans)
Vert Live Load Defl. (in)	0.000 @ 0	0.408	Passed (2L/999+)	--	1.0 D (All Spans)
Vert Total Load Defl. (in)	0.377 @ 6' 1 1/2"	0.313	Failed (L/390)	--	1.0 D (All Spans)
Lat Member Reaction (lbs)	352 @ 12' 3"	N/A	Passed (N/A)	1.60	1.0 D + 0.6 W
Lat Shear (lbs)	319 @ 7"	3696	Passed (9%)	1.60	1.0 D + 0.6 W
Lat Moment (Ft-lbs)	1079 @ mid-span	2990	Passed (36%)	1.60	1.0 D + 0.6 W
Lat Deflection (in)	0.247 @ mid-span	1.225	Passed (L/595)	--	1.0 D + 0.6 W
Bi-Axial Bending	0.59	1.00	Passed (59%)	1.60	1.0 D + 0.6 W

System : Wall
Member Type : Header
Building Use : Residential
Building Code : IBC 2018
Design Methodology : ASD

- Deflection criteria: LL (L/360) and TL (5/16").
- Lateral deflection criteria: Wind (L/120)
- Member has been designed in flat (plank) orientation.
- Applicable calculations are based on NDS.

Supports	Bearing Length			Loads to Supports (lbs)		Accessories
	Total	Available	Required	Dead	Factored	
1 - Trimmer - HF	1.50"	1.50"	1.50"	152	152	None
2 - Trimmer - HF	1.50"	1.50"	1.50"	152	152	None

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	End Bearing Points	
Bottom Edge (Lu)	End Bearing Points	

Lateral Connections

Supports	Plate Size	Plate Material	Connector	Type/Model	Quantity	Nailing
Left	2X	Hem Fir	Nails	10d (0.128" x 3") (End)	5	
Right	2X	Hem Fir	Nails	10d (0.128" x 3") (End)	5	

Vertical Loads	Location	Tributary Width	Dead (0.90)	Comments
0 - Self Weight (PLF)	0 to 12' 3"	N/A	4.9	
1 - Uniform (PLF)	0 to 12' 3"	N/A	20.0	Clerestory Window Load

Lateral Load	Location	Tributary Width	Wind (1.60)	Comments
1 - Uniform (PSF)	Full Length	4'	24.0	

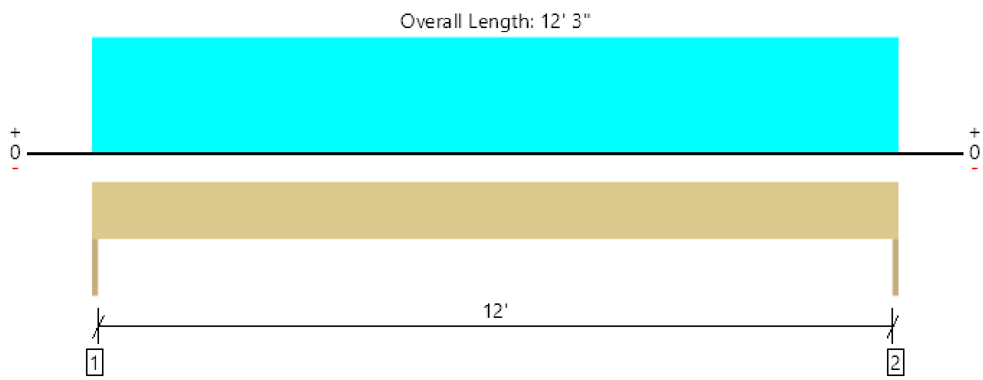
• ASCE/SEI 7 Sec. 30.4: Exposure Category (B), Mean Roof Height (33'), Topographic Factor (1.0), Wind Directionality Factor (0.85), Basic Wind Speed (115), Risk Category(II), Effective Wind Area determined using full member span and trib. width.
• IBC Table 1604.3, footnote f: Deflection checks are performed using 42% of this lateral wind load.

ForteWEB Software Operator	Job Notes
Harrison Kliegl L120 Engineering (425) 636-3313 hkliegl@l120engineering.com	

Weyerhaeuser

2nd Floor, 2H-1.1 (6x6 Check)

1 piece(s) 6 x 6 DF No.1



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)	169 @ 0	5156 (1.50")	Passed (3%)	--	1.0 D (All Spans)
Shear (lbs)	153 @ 7"	3086	Passed (5%)	0.90	1.0 D (All Spans)
Moment (Ft-lbs)	519 @ 6' 1 1/2"	2496	Passed (21%)	0.90	1.0 D (All Spans)
Vert Live Load Defl. (in)	0.000 @ 0	0.408	Passed (2L/999+)	--	1.0 D (All Spans)
Vert Total Load Defl. (in)	0.115 @ 6' 1 1/2"	0.313	Passed (L/999+)	--	1.0 D (All Spans)
Lat Member Reaction (lbs)	352 @ 12' 3"	N/A	Passed (N/A)	1.60	1.0 D + 0.6 W
Lat Shear (lbs)	319 @ 7"	5485	Passed (6%)	1.60	1.0 D + 0.6 W
Lat Moment (Ft-lbs)	1079 @ mid-span	4437	Passed (24%)	1.60	1.0 D + 0.6 W
Lat Deflection (in)	0.167 @ mid-span	1.225	Passed (L/879)	--	1.0 D + 0.6 W
Bi-Axial Bending	0.36	1.00	Passed (36%)	1.60	1.0 D + 0.6 W

System : Wall
Member Type : Header
Building Use : Residential
Building Code : IBC 2018
Design Methodology : ASD

- Deflection criteria: LL (L/360) and TL (5/16").
- Lateral deflection criteria: Wind (L/120)
- Applicable calculations are based on NDS.
- This product has a square cross section. The analysis engine has checked both edge and plank orientations to allow for either installation.

Supports	Bearing Length			Loads to Supports (lbs)		Accessories
	Total	Available	Required	Dead	Factored	
1 - Trimmer - HF	1.50"	1.50"	1.50"	169	169	None
2 - Trimmer - HF	1.50"	1.50"	1.50"	169	169	None

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	End Bearing Points	
Bottom Edge (Lu)	End Bearing Points	

Lateral Connections						
Supports	Plate Size	Plate Material	Connector	Type/Model	Quantity	Nailing
Left	2X	Hem Fir	Nails	10d (0.128" x 3") (End)	5	
Right	2X	Hem Fir	Nails	10d (0.128" x 3") (End)	5	

Vertical Loads	Location	Tributary Width	Dead (0.90)	Comments
0 - Self Weight (PLF)	0 to 12' 3"	N/A	7.7	
1 - Uniform (PLF)	0 to 12' 3"	N/A	20.0	Clerestory Window Load

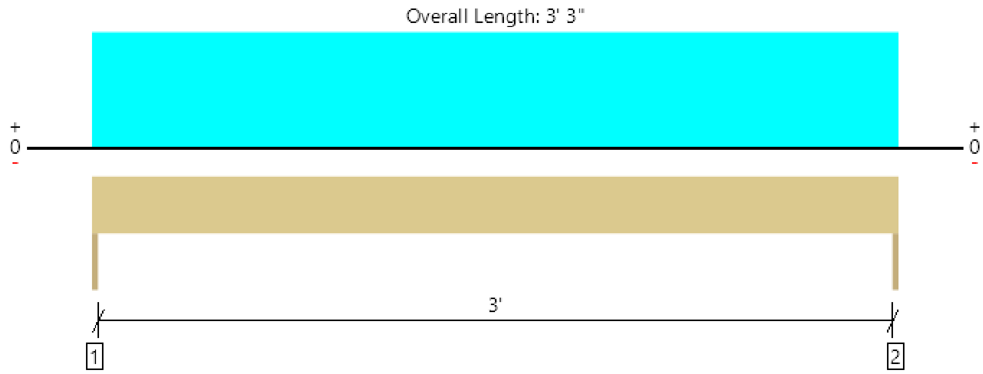
Lateral Load	Location	Tributary Width	Wind (1.60)	Comments
1 - Uniform (PSF)	Full Length	4'	24.0	

- ASCE/SEI 7 Sec. 30.4: Exposure Category (B), Mean Roof Height (33'), Topographic Factor (1.0), Wind Directionality Factor (0.85), Basic Wind Speed (115), Risk Category(II), Effective Wind Area determined using full member span and trib. width.
- IBC Table 1604.3, footnote f: Deflection checks are performed using 42% of this lateral wind load.

ForteWEB Software Operator	Job Notes
Harrison Kliegl L120 Engineering (425) 636-3313 hkliegl@l120engineering.com	

2nd Floor, 2H-2

1 piece(s) 2 x 8 DF 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)	728 @ 0	1406 (1.50")	Passed (52%)	--	1.0 D + 1.0 S (All Spans)
Shear (lbs)	401 @ 8 3/4"	1501	Passed (27%)	1.15	1.0 D + 1.0 S (All Spans)
Moment (Ft-lbs)	591 @ 1' 7 1/2"	1308	Passed (45%)	1.15	1.0 D + 1.0 S (All Spans)
Vert Live Load Defl. (in)	0.009 @ 1' 7 1/2"	0.108	Passed (L/999+)	--	1.0 D + 1.0 S (All Spans)
Vert Total Load Defl. (in)	0.015 @ 1' 7 1/2"	0.162	Passed (L/999+)	--	1.0 D + 1.0 S (All Spans)
Lat Member Reaction (lbs)	52 @ 3' 3"	N/A	Passed (N/A)	1.60	1.0 D + 0.6 W
Lat Shear (lbs)	44 @ 3"	2088	Passed (2%)	1.60	1.0 D + 0.6 W
Lat Moment (Ft-lbs)	42 @ mid-span	450	Passed (9%)	1.60	1.0 D + 0.6 W
Lat Deflection (in)	0.017 @ mid-span	0.325	Passed (L/999+)	--	1.0 D + 0.6 W
Bi-Axial Bending	0.36	1.00	Passed (36%)	1.60	1.0 D + 0.45 W + 0.75 L + 0.75 S

System : Wall
Member Type : Header
Building Use : Residential
Building Code : IBC 2018
Design Methodology : ASD

- Deflection criteria: LL (L/360) and TL (L/240).
- Lateral deflection criteria: Wind (L/120)
- A 3.8% decrease in the moment capacity has been added to account for lateral stability.
- Applicable calculations are based on NDS.

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Snow	Factored	
1 - Trimmer - HF	1.50"	1.50"	1.50"	281	447	728	None
2 - Trimmer - HF	1.50"	1.50"	1.50"	281	447	728	None

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	End Bearing Points	
Bottom Edge (Lu)	End Bearing Points	

Lateral Connections						
Supports	Plate Size	Plate Material	Connector	Type/Model	Quantity	Nailing
Left	2X	Hem Fir	Nails	10d (0.128" x 3") (End)	2	
Right	2X	Hem Fir	Nails	10d (0.128" x 3") (End)	2	

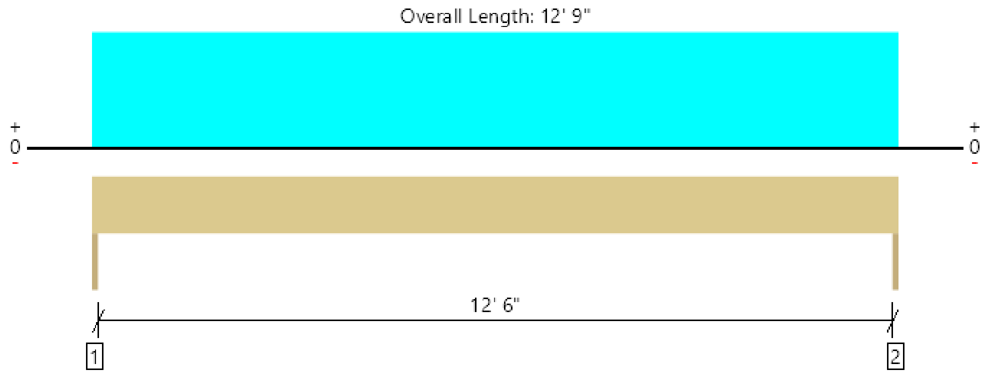
Vertical Loads	Location	Tributary Width	Dead (0.90)	Snow (1.15)	Comments
0 - Self Weight (PLF)	0 to 3' 3"	N/A	2.8	--	
1 - Uniform (PSF)	0 to 3' 3"	11'	15.5	25.0	Roof Load

Lateral Load	Location	Tributary Width	Wind (1.60)	Comments
1 - Uniform (PSF)	Full Length	2'	26.5	

- ASCE/SEI 7 Sec. 30.4: Exposure Category (B), Mean Roof Height (33'), Topographic Factor (1.0), Wind Directionality Factor (0.85), Basic Wind Speed (115), Risk Category(II), Effective Wind Area determined using full member span and trib. width.
- IBC Table 1604.3, footnote f: Deflection checks are performed using 42% of this lateral wind load.

ForteWEB Software Operator	Job Notes
Harrison Kliegl L120 Engineering (425) 636-3313 hkliegl@l120engineering.com	

2nd Floor, 2H-3 (High)
1 piece(s) 6 x 8 DF No.1



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)	577 @ 0	5156 (1.50")	Passed (11%)	--	1.0 D + 1.0 S (All Spans)
Shear (lbs)	509 @ 9"	5376	Passed (9%)	1.15	1.0 D + 1.0 S (All Spans)
Moment (Ft-lbs)	1838 @ 6' 4 1/2"	5930	Passed (31%)	1.15	1.0 D + 1.0 S (All Spans)
Vert Live Load Defl. (in)	0.096 @ 6' 4 1/2"	0.425	Passed (L/999+)	--	1.0 D + 1.0 S (All Spans)
Vert Total Load Defl. (in)	0.174 @ 6' 4 1/2"	0.637	Passed (L/880)	--	1.0 D + 1.0 S (All Spans)
Lat Member Reaction (lbs)	273 @ 12' 9"	N/A	Passed (N/A)	1.60	1.0 D + 0.6 W
Lat Shear (lbs)	248 @ 7"	7480	Passed (3%)	1.60	1.0 D + 0.6 W
Lat Moment (Ft-lbs)	872 @ mid-span	6050	Passed (14%)	1.60	1.0 D + 0.6 W
Lat Deflection (in)	0.107 @ mid-span	1.275	Passed (L/999+)	--	1.0 D + 0.6 W
Bi-Axial Bending	0.30	1.00	Passed (30%)	1.60	1.0 D + 0.45 W + 0.75 L + 0.75 S

System : Wall
Member Type : Header
Building Use : Residential
Building Code : IBC 2018
Design Methodology : ASD

- Deflection criteria: LL (L/360) and TL (L/240).
- Lateral deflection criteria: Wind (L/120)
- A 0.8% decrease in the moment capacity has been added to account for lateral stability.
- Applicable calculations are based on NDS.

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Snow	Factored	
1 - Trimmer - HF	1.50"	1.50"	1.50"	258	319	577	None
2 - Trimmer - HF	1.50"	1.50"	1.50"	258	319	577	None

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	End Bearing Points	
Bottom Edge (Lu)	End Bearing Points	

Lateral Connections						
Supports	Plate Size	Plate Material	Connector	Type/Model	Quantity	Nailing
Left	2X	Hem Fir	Nails	8d (0.113" x 2 1/2") (Toe)	4	
Right	2X	Hem Fir	Nails	8d (0.113" x 2 1/2") (Toe)	4	

Vertical Loads	Location	Tributary Width	Dead (0.90)	Snow (1.15)	Comments
0 - Self Weight (PLF)	0 to 12' 9"	N/A	10.4	--	
1 - Uniform (PSF)	0 to 12' 9"	2'	15.0	25.0	Roof Load

Lateral Load	Location	Tributary Width	Wind (1.60)	Comments
1 - Uniform (PSF)	Full Length	3'	23.8	

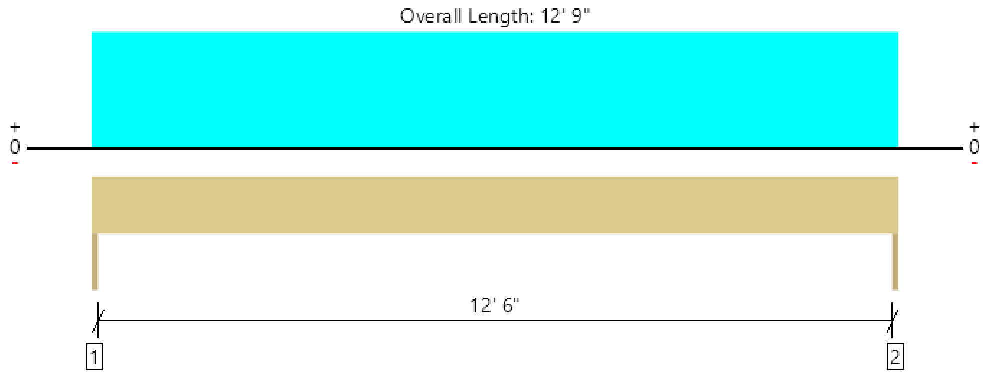
• ASCE/SEI 7 Sec. 30.4: Exposure Category (B), Mean Roof Height (33'), Topographic Factor (1.0), Wind Directionality Factor (0.85), Basic Wind Speed (115), Risk Category(II), Effective Wind Area determined using full member span and trib. width.
• IBC Table 1604.3, footnote f: Deflection checks are performed using 42% of this lateral wind load.

ForteWEB Software Operator	Job Notes
Harrison Kliegl L120 Engineering (425) 636-3313 hkliegl@l120engineering.com	

MEMBER REPORT

PASSED

2nd Floor, 2H-3 (Low)
1 piece(s) 6 x 6 DF No.1



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)	368 @ 0	5156 (1.50")	Passed (7%)	--	1.0 D (All Spans)
Shear (lbs)	334 @ 7"	3086	Passed (11%)	0.90	1.0 D (All Spans)
Moment (Ft-lbs)	1172 @ 6' 4 1/2"	2496	Passed (47%)	0.90	1.0 D (All Spans)
Vert Live Load Defl. (in)	0.000 @ 0	0.425	Passed (2L/999+)	--	1.0 D (All Spans)
Vert Total Load Defl. (in)	0.281 @ 6' 4 1/2"	0.313	Passed (L/544)	--	1.0 D (All Spans)
Lat Member Reaction (lbs)	534 @ 12' 9"	N/A	Passed (N/A)	1.60	1.0 D + 0.6 W
Lat Shear (lbs)	486 @ 7"	5485	Passed (9%)	1.60	1.0 D + 0.6 W
Lat Moment (Ft-lbs)	1703 @ mid-span	4437	Passed (38%)	1.60	1.0 D + 0.6 W
Lat Deflection (in)	0.286 @ mid-span	1.275	Passed (L/535)	--	1.0 D + 0.6 W
Bi-Axial Bending	0.65	1.00	Passed (65%)	1.60	1.0 D + 0.6 W

System : Wall
Member Type : Header
Building Use : Residential
Building Code : IBC 2018
Design Methodology : ASD

- Deflection criteria: LL (L/360) and TL (5/16").
- Lateral deflection criteria: Wind (L/120)
- Applicable calculations are based on NDS.
- This product has a square cross section. The analysis engine has checked both edge and plank orientations to allow for either installation.

Supports	Bearing Length			Loads to Supports (lbs)		Accessories
	Total	Available	Required	Dead	Factored	
1 - Trimmer - HF	1.50"	1.50"	1.50"	368	368	None
2 - Trimmer - HF	1.50"	1.50"	1.50"	368	368	None

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	End Bearing Points	
Bottom Edge (Lu)	End Bearing Points	

Lateral Connections

Supports	Plate Size	Plate Material	Connector	Type/Model	Quantity	Nailing
Left	2X	Hem Fir	Nails	10d (0.128" x 3") (End)	7	
Right	2X	Hem Fir	Nails	10d (0.128" x 3") (End)	7	

Vertical Loads	Location	Tributary Width	Dead (0.90)	Comments
0 - Self Weight (PLF)	0 to 12' 9"	N/A	7.7	
1 - Uniform (PLF)	0 to 12' 9"	N/A	50.0	Window Load

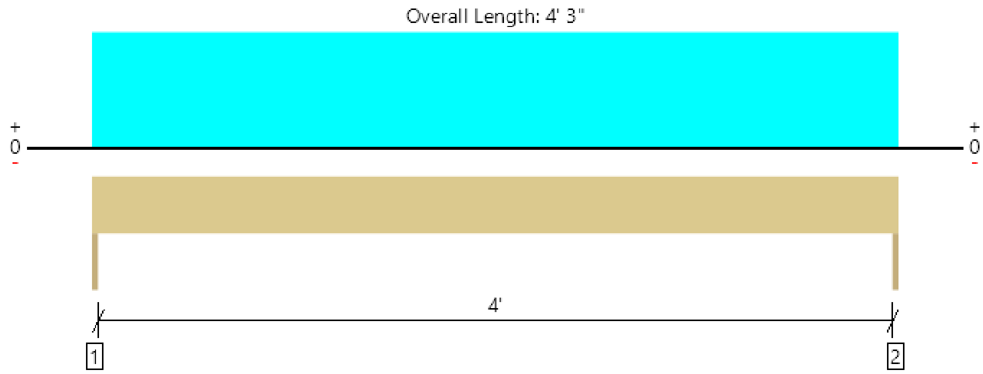
Lateral Load	Location	Tributary Width	Wind (1.60)	Comments
1 - Uniform (PSF)	Full Length	6'	23.3	

• ASCE/SEI 7 Sec. 30.4: Exposure Category (B), Mean Roof Height (33'), Topographic Factor (1.0), Wind Directionality Factor (0.85), Basic Wind Speed (115), Risk Category(II), Effective Wind Area determined using full member span and trib. width.
• IBC Table 1604.3, footnote f: Deflection checks are performed using 42% of this lateral wind load.

ForteWEB Software Operator	Job Notes
Harrison Kliegl L120 Engineering (425) 636-3313 hkliegl@l120engineering.com	

2nd Floor, 2H-4

1 piece(s) 4 x 6 DF 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)	858 @ 0	3281 (1.50")	Passed (26%)	--	1.0 D + 1.0 S (All Spans)
Shear (lbs)	623 @ 7"	2657	Passed (23%)	1.15	1.0 D + 1.0 S (All Spans)
Moment (Ft-lbs)	912 @ 2' 1 1/2"	1969	Passed (46%)	1.15	1.0 D + 1.0 S (All Spans)
Vert Live Load Defl. (in)	0.024 @ 2' 1 1/2"	0.142	Passed (L/999+)	--	1.0 D + 1.0 S (All Spans)
Vert Total Load Defl. (in)	0.038 @ 2' 1 1/2"	0.213	Passed (L/999+)	--	1.0 D + 1.0 S (All Spans)
Lat Member Reaction (lbs)	68 @ 4' 3"	N/A	Passed (N/A)	1.60	1.0 D + 0.6 W
Lat Shear (lbs)	54 @ 5"	3696	Passed (1%)	1.60	1.0 D + 0.6 W
Lat Moment (Ft-lbs)	72 @ mid-span	1839	Passed (4%)	1.60	1.0 D + 0.6 W
Lat Deflection (in)	0.005 @ mid-span	0.425	Passed (L/999+)	--	1.0 D + 0.6 W
Bi-Axial Bending	0.31	1.00	Passed (31%)	1.60	1.0 D + 0.45 W + 0.75 L + 0.75 S

System : Wall
Member Type : Header
Building Use : Residential
Building Code : IBC 2018
Design Methodology : ASD

- Deflection criteria: LL (L/360) and TL (L/240).
- Lateral deflection criteria: Wind (L/120)
- A 0.5% decrease in the moment capacity has been added to account for lateral stability.
- Applicable calculations are based on NDS.

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Snow	Factored	
1 - Trimmer - HF	1.50"	1.50"	1.50"	318	540	858	None
2 - Trimmer - HF	1.50"	1.50"	1.50"	318	540	858	None

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	End Bearing Points	
Bottom Edge (Lu)	End Bearing Points	

Lateral Connections

Supports	Plate Size	Plate Material	Connector	Type/Model	Quantity	Nailing
Left	2X	Hem Fir	Nails	8d (0.113" x 2 1/2") (Toe)	2	
Right	2X	Hem Fir	Nails	8d (0.113" x 2 1/2") (Toe)	2	

Vertical Loads	Location	Tributary Width	Dead (0.90)	Snow (1.15)	Comments
0 - Self Weight (PLF)	0 to 4' 3"	N/A	4.9	--	
1 - Uniform (PLF)	0 to 4' 3"	N/A	144.8	254.3	Linked from: RJ-2, Support 1

Lateral Load	Location	Tributary Width	Wind (1.60)	Comments
1 - Uniform (PSF)	Full Length	2'	26.5	

• ASCE/SEI 7 Sec. 30.4: Exposure Category (B), Mean Roof Height (33'), Topographic Factor (1.0), Wind Directionality Factor (0.85), Basic Wind Speed (115), Risk Category(II), Effective Wind Area determined using full member span and trib. width.
• IBC Table 1604.3, footnote f: Deflection checks are performed using 42% of this lateral wind load.

ForteWEB Software Operator	Job Notes
Harrison Kliegl L120 Engineering (425) 636-3313 hkliegl@l120engineering.com	

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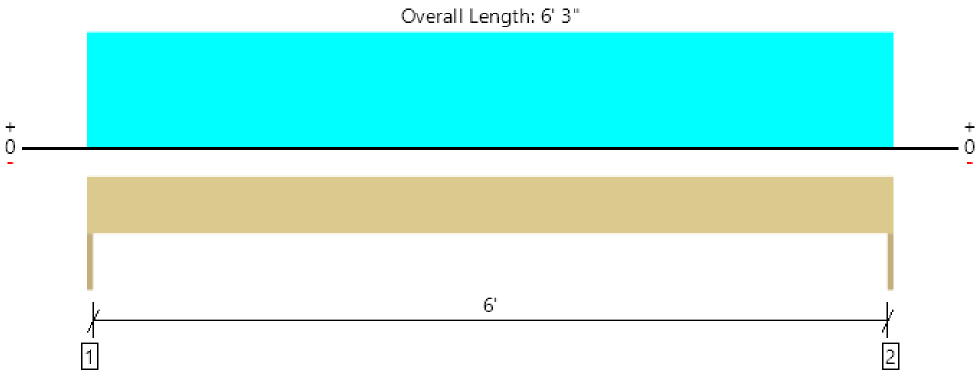
MEMBER REPORT

PASSED

2nd Floor, 2H-5

2 piece(s) 2 x 8 DF 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)	892 @ 0	2813 (1.50")	Passed (32%)	--	1.0 D + 1.0 L (All Spans)
Shear (lbs)	684 @ 8 3/4"	2610	Passed (26%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	1394 @ 3' 1 1/2"	2277	Passed (61%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.039 @ 3' 1 1/2"	0.208	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.064 @ 3' 1 1/2"	0.313	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)

- Deflection criteria: LL (L/360) and TL (L/240).
- A 3.7% decrease in the moment capacity has been added to account for lateral stability.
- Applicable calculations are based on NDS.

System : Wall
Member Type : Header
Building Use : Residential
Building Code : IBC 2018
Design Methodology : ASD

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Floor Live	Factored	
1 - Trimmer - HF	1.50"	1.50"	1.50"	345	547	892	None
2 - Trimmer - HF	1.50"	1.50"	1.50"	345	547	892	None

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	End Bearing Points	
Bottom Edge (Lu)	End Bearing Points	

Vertical Loads	Location	Tributary Width	Dead (0.90)	Floor Live (1.00)	Comments
0 - Self Weight (PLF)	0 to 6' 3"	N/A	5.5	--	
1 - Uniform (PSF)	0 to 6' 3"	7'	15.0	25.0	Roof Load

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
Harrison Kliegl L120 Engineering (425) 636-3313 hkliegl@l120engineering.com	

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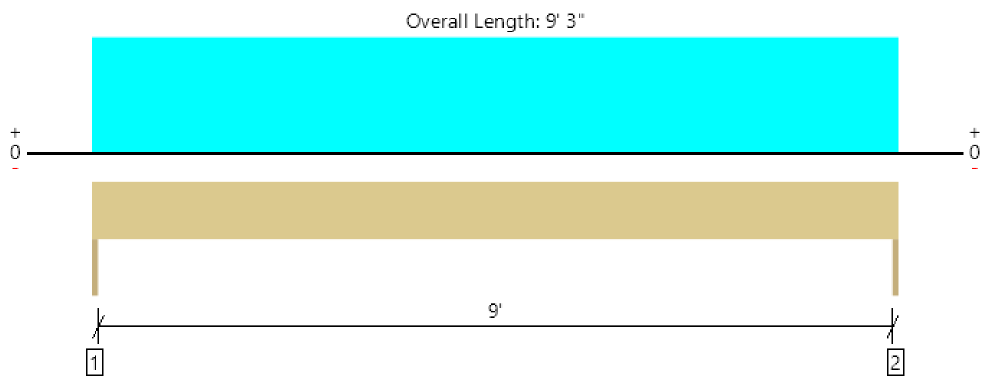
2nd Floor, 2H-6

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)	2174 @ 0	5363 (1.50")	Passed (41%)	--	1.0 D + 1.0 L (All Spans)
Shear (lbs)	1821 @ 9"	7288	Passed (25%)	1.00	1.0 D + 1.0 L (All Spans)
Pos Moment (Ft-lbs)	5027 @ 4' 7 1/2"	10252	Passed (49%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.136 @ 4' 7 1/2"	0.308	Passed (L/816)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.222 @ 4' 7 1/2"	0.463	Passed (L/499)	--	1.0 D + 1.0 L (All Spans)

System : Wall
Member Type : Header
Building Use : Residential
Building Code : IBC 2018
Design Methodology : ASD

- Deflection criteria: LL (L/360) and TL (L/240).
- A 0.6% decrease in the moment capacity has been added to account for lateral stability.
- Critical positive moment adjusted by a volume factor of 1.00 that was calculated using length L = 9' 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	Factored	
1 - Trimmer - HF	1.50"	1.50"	1.50"	844	1330	2174	None
2 - Trimmer - HF	1.50"	1.50"	1.50"	844	1330	2174	None

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	End Bearing Points	
Bottom Edge (Lu)	End Bearing Points	

Vertical Loads	Location	Tributary Width	Dead (0.90)	Floor Live (1.00)	Comments
0 - Self Weight (PLF)	0 to 9' 3"	N/A	10.0	--	
1 - Uniform (PSF)	0 to 9' 3"	11' 6"	15.0	25.0	Roof Load

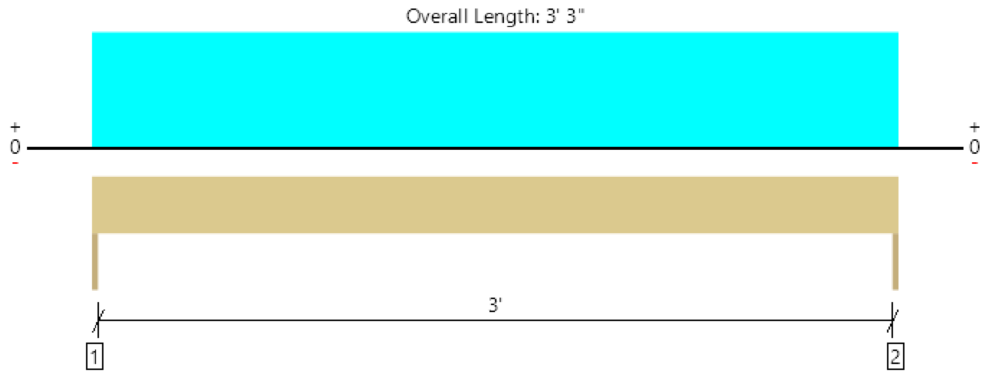
Weyerhaeuser Notes

ForteWEB Software Operator	Job Notes
Harrison Kliegl L120 Engineering (425) 636-3313 hkliegl@l120engineering.com	

MEMBER REPORT

PASSED

2nd Floor, 2H-7
2 piece(s) 2 x 8 DF 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)	1049 @ 0	2813 (1.50")	Passed (37%)	--	1.0 D + 1.0 L (All Spans)
Shear (lbs)	578 @ 8 3/4"	2610	Passed (22%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	852 @ 1' 7 1/2"	2327	Passed (37%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.007 @ 1' 7 1/2"	0.108	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.011 @ 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 2018
Design Methodology : ASD

- Deflection criteria: LL (L/360) and TL (L/240).
- A 1.6% decrease in the moment capacity has been added to account for lateral stability.
- Applicable calculations are based on NDS.

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Floor Live	Factored	
1 - Trimmer - HF	1.50"	1.50"	1.50"	399	650	1049	None
2 - Trimmer - HF	1.50"	1.50"	1.50"	399	650	1049	None

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	End Bearing Points	
Bottom Edge (Lu)	End Bearing Points	

Vertical Loads	Location	Tributary Width	Dead (0.90)	Floor Live (1.00)	Comments
0 - Self Weight (PLF)	0 to 3' 3"	N/A	5.5	--	
1 - Uniform (PSF)	0 to 3' 3"	16'	15.0	25.0	Roof Load

Weyerhaeuser Notes

ForteWEB Software Operator	Job Notes
Harrison Kliegl L120 Engineering (425) 636-3313 hkliegl@l120engineering.com	

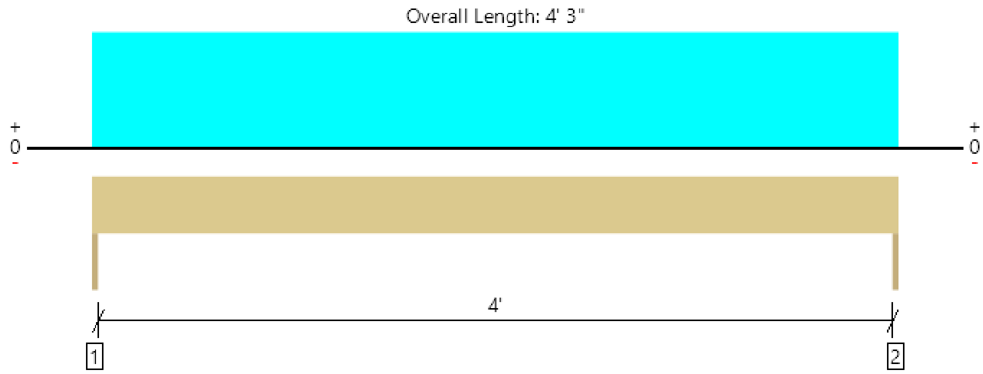
FORTEWEB

MEMBER REPORT

PASSED

2nd Floor, 2H-8

3 piece(s) 2 x 8 DF 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)	2511 @ 0	4219 (1.50")	Passed (60%)	--	1.0 D + 1.0 L (All Spans)
Shear (lbs)	1649 @ 8 3/4"	3915	Passed (42%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	2668 @ 2' 1 1/2"	3490	Passed (76%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.023 @ 2' 1 1/2"	0.142	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.038 @ 2' 1 1/2"	0.213	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)

System : Wall
Member Type : Header
Building Use : Residential
Building Code : IBC 2018
Design Methodology : ASD

- Deflection criteria: LL (L/360) and TL (L/240).
- A 1.6% decrease in the moment capacity has been added to account for lateral stability.
- Applicable calculations are based on NDS.

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Floor Live	Factored	
1 - Trimmer - HF	1.50"	1.50"	1.50"	970	1541	2511	None
2 - Trimmer - HF	1.50"	1.50"	1.50"	970	1541	2511	None

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	End Bearing Points	
Bottom Edge (Lu)	End Bearing Points	

Vertical Loads	Location	Tributary Width	Dead (0.90)	Floor Live (1.00)	Comments
0 - Self Weight (PLF)	0 to 4' 3"	N/A	8.3	--	
1 - Uniform (PSF)	0 to 4' 3"	29'	15.5	25.0	Roof Load

Weyerhaeuser Notes

ForteWEB Software Operator	Job Notes
Harrison Kliegl L120 Engineering (425) 636-3313 hkliegl@l120engineering.com	

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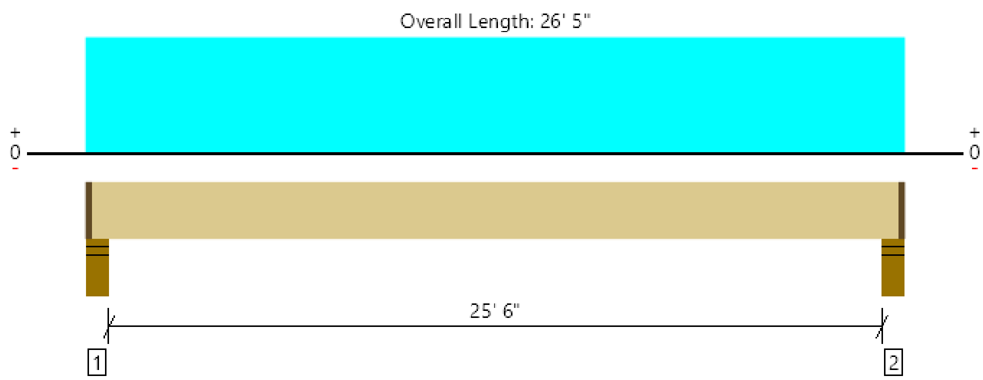
PASSED

2nd Floor, 2B-8

1 piece(s) 5 1/4" x 18" 2.2E Parallam® PSL



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)	3677 @ 4"	8505 (4.00")	Passed (43%)	--	1.0 D + 0.75 L + 0.75 S (All Spans)
Shear (lbs)	2965 @ 1' 11 1/2"	18270	Passed (16%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	21842 @ 13' 2 1/2"	65497	Passed (33%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.181 @ 13' 2 1/2"	0.644	Passed (L/999+)	--	1.0 D + 0.75 L + 0.75 S (All Spans)
Total Load Defl. (in)	0.521 @ 13' 2 1/2"	1.288	Passed (L/593)	--	1.0 D + 0.75 L + 0.75 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 2018
Design Methodology : ASD

Supports	Bearing Length			Loads to Supports (lbs)				Accessories
	Total	Available	Required	Dead	Floor Live	Snow	Factored	
1 - Stud wall - HF	5.50"	4.00"	1.73"	2420	1057	660	3708	1 1/2" Rim Board
2 - Stud wall - HF	5.50"	4.00"	1.73"	2420	1057	660	3708	1 1/2" 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)	26' 2" o/c	
Bottom Edge (Lu)	26' 2" o/c	

- Maximum allowable bracing intervals based on applied load.

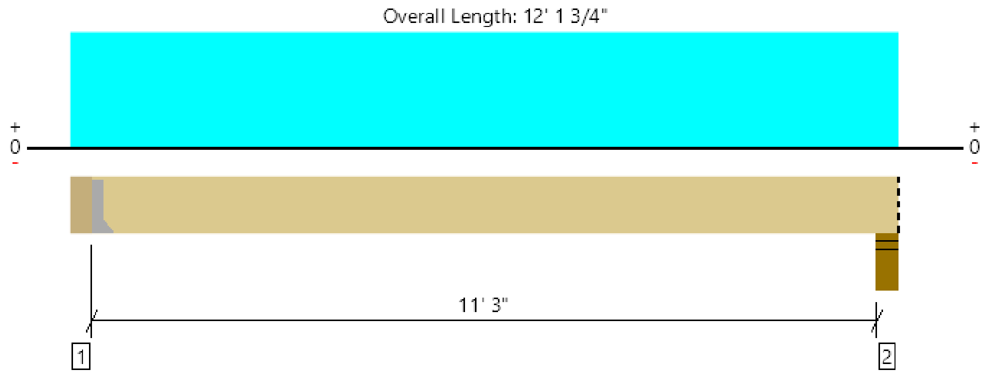
Vertical Loads	Location (Side)	Tributary Width	Dead (0.90)	Floor Live (1.00)	Snow (1.15)	Comments
0 - Self Weight (PLF)	1 1/2" to 26' 3 1/2"	N/A	29.5	--	--	
1 - Uniform (PSF)	0 to 26' 5" (Front)	2'	15.0	40.0	-	Floor Load
2 - Uniform (PLF)	0 to 26' 5" (Top)	N/A	100.0	-	-	Wall Load Above
3 - Uniform (PSF)	0 to 26' 5" (Front)	2'	12.0	-	25.0	Roof Load

Weyerhaeuser Notes

ForteWEB Software Operator	Job Notes
Harrison Kliegl L120 Engineering (425) 636-3313 hkliegl@l120engineering.com	

2nd Floor, 2B-9

1 piece(s) 3 1/2" x 18" 2.2E Parallam® PSL



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)	1687 @ 5 1/4"	3281 (1.50")	Passed (51%)	--	1.0 D + 0.75 L + 0.75 S (All Spans)
Shear (lbs)	1242 @ 1' 11 1/4"	14007	Passed (9%)	1.15	1.0 D + 0.75 L + 0.75 S (All Spans)
Moment (Ft-lbs)	4799 @ 6' 1 1/2"	50215	Passed (10%)	1.15	1.0 D + 0.75 L + 0.75 S (All Spans)
Live Load Defl. (in)	0.013 @ 6' 1 1/2"	0.284	Passed (L/999+)	--	1.0 D + 0.75 L + 0.75 S (All Spans)
Total Load Defl. (in)	0.038 @ 6' 1 1/2"	0.569	Passed (L/999+)	--	1.0 D + 0.75 L + 0.75 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 2018
Design Methodology : ASD

Supports	Bearing Length			Loads to Supports (lbs)				Accessories
	Total	Available	Required	Dead	Floor Live	Snow	Factored	
1 - Hanger on 18" PSL beam	5.25"	Hanger[1]	1.50"	1165	245	613	1809	See note [1]
2 - Stud wall - HF	5.50"	5.50"	1.50"	1154	241	602	1786	Blocking

- Blocking Panels are assumed to carry no loads applied directly above them and the full load is applied to the member being designed.
- At hanger supports, the Total Bearing dimension is equal to the width of the material that is supporting the hanger
- [1] See Connector grid below for additional information and/or requirements.

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	11' 9" o/c	
Bottom Edge (Lu)	11' 9" o/c	

•Maximum allowable bracing intervals based on applied load.

Connector: Simpson Strong-Tie						
Support	Model	Seat Length	Top Fasteners	Face Fasteners	Member Fasteners	Accessories
1 - Face Mount Hanger	IUS3.56/14	2.00"	N/A	14-10d	2-10dx1.5	

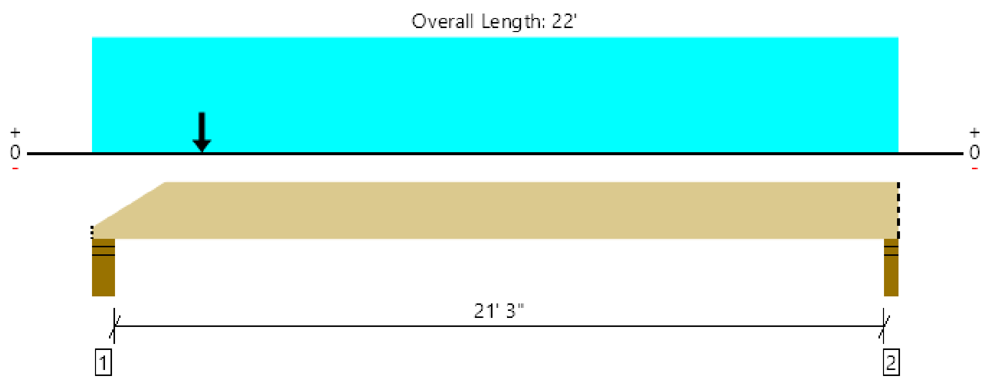
- Refer to manufacturer notes and instructions for proper installation and use of all connectors.

Vertical Loads	Location (Side)	Tributary Width	Dead (0.90)	Floor Live (1.00)	Snow (1.15)	Comments
0 - Self Weight (PLF)	5 1/4" to 12' 1 3/4"	N/A	19.7	--	--	
1 - Uniform (PSF)	0 to 12' 1 3/4" (Front)	1'	12.0	40.0	-	Floor Load
2 - Uniform (PSF)	0 to 12' 1 3/4" (Back)	2'	15.0	-	25.0	Low Roof Load
3 - Uniform (PLF)	0 to 12' 1 3/4" (Top)	N/A	100.0	-	-	Wall Load Above
4 - Uniform (PSF)	0 to 12' 1 3/4" (Top)	2'	15.0	-	25.0	Roof Load

ForteWEB Software Operator	Job Notes
Harrison Kliegl L120 Engineering (425) 636-3313 hkliegl@l120engineering.com	

2nd Floor, 2B-10

1 piece(s) 5 1/4" x 18" 2.2E Parallam® PSL



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)	6218 @ 21' 10"	7442 (3.50")	Passed (84%)	--	1.0 D + 1.0 L (All Spans)
Shear (lbs)	7116 @ 1' 4 7/8"	14432	Passed (49%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	33891 @ 10' 9 3/16"	65497	Passed (52%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.356 @ 11' 3/4"	0.538	Passed (L/725)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.544 @ 10' 11 15/16"	1.075	Passed (L/474)	--	1.0 D + 1.0 L (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 2018
Design Methodology : ASD

Supports	Bearing Length			Loads to Supports (lbs)				Accessories
	Total	Available	Required	Dead	Floor Live	Snow	Factored	
1 - Stud wall - HF	5.50"	5.50"	3.47"	2944	4426	1091	7370	Blocking
2 - Stud wall - HF	3.50"	3.50"	2.92"	2039	4179	622	6218	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)	22' o/c	
Bottom Edge (Lu)	22' 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 22'	N/A	29.5	--	--	
1 - Uniform (PSF)	0 to 22' (Front)	9' 6"	12.0	40.0	-	Floor Load
2 - Uniform (PSF)	0 to 22' (Back)	2'	15.0	-	25.0	Low Roof Load
3 - Point (lb)	3' (Front)	N/A	1165	245	613	Linked from: 2B-9, Support 1

Tapered End	Heel Height	Cut Length	Cut Slope	Location	Shear (lbs)			Comments
					Actual	Allowed	Result	
Left End	10"	2' 8"	3/12	1' 4 7/8"	7116	14432	Passed (49%)	

Weyerhaeuser Notes

ForteWEB Software Operator	Job Notes
Harrison Kliegl L120 Engineering (425) 636-3313 hkliegl@l120engineering.com	

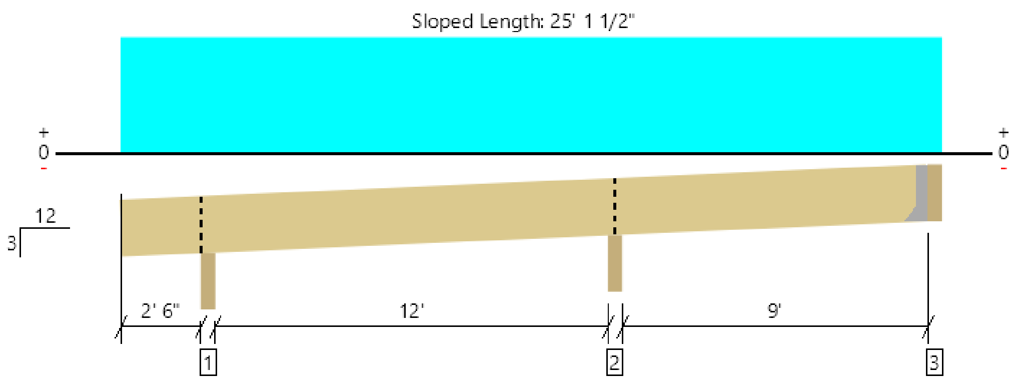
FORTEWEB

MEMBER REPORT

PASSED

Low Roof, LRJ-1

1 piece(s) 2 x 8 DF No.2 @ 16" OC



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

Member Length : 24' 11 11/16"

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	718 @ 14' 11 1/4"	3382 (3.50")	Passed (21%)	--	1.0 D + 1.0 S (Adj Spans)
Shear (lbs)	346 @ 14' 2 7/16"	1501	Passed (23%)	1.15	1.0 D + 1.0 S (Adj Spans)
Moment (Ft-lbs)	-787 @ 14' 11 1/4"	1564	Passed (50%)	1.15	1.0 D + 1.0 S (Adj Spans)
Live Load Defl. (in)	0.131 @ 8' 4 1/8"	0.633	Passed (L/999+)	--	1.0 D + 1.0 S (Alt Spans)
Total Load Defl. (in)	0.196 @ 8' 3 13/16"	0.845	Passed (L/775)	--	1.0 D + 1.0 S (Alt Spans)

System : Roof
Member Type : Joist
Building Use : Residential
Building Code : IBC 2018
Design Methodology : ASD
Member Pitch : 3/12

- Deflection criteria: LL (L/240) and TL (L/180).
- Overhang deflection criteria: LL (2L/240) and TL (2L/180).
- 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.

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Snow	Factored	
1 - Beveled Plate - DF	3.50"	3.50"	1.50"	163	270	433	Blocking
2 - Beveled Plate - DF	3.50"	3.50"	1.50"	271	447	718	Blocking
3 - Hanger on 7 1/4" DF ledgerOnMasonry	3.50"	Hanger[1]	1.50"	68	130	198	See note [1]

- Blocking Panels are assumed to carry no loads applied directly above them and the full load is applied to the member being designed.
- At hanger supports, the Total Bearing dimension is equal to the width of the material that is supporting the hanger
- [1] See Connector grid below for additional information and/or requirements.

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	16' 9" o/c	
Bottom Edge (Lu)	12' 11" o/c	

•Maximum allowable bracing intervals based on applied load.

Connector: Simpson Strong-Tie						
Support	Model	Seat Length	Top Fasteners	Face Fasteners	Member Fasteners	Accessories
3 - Face Mount Hanger	LRU26Z	1.94"	N/A	4-10dx1.5	5-10d	

- Refer to manufacturer notes and instructions for proper installation and use of all connectors.

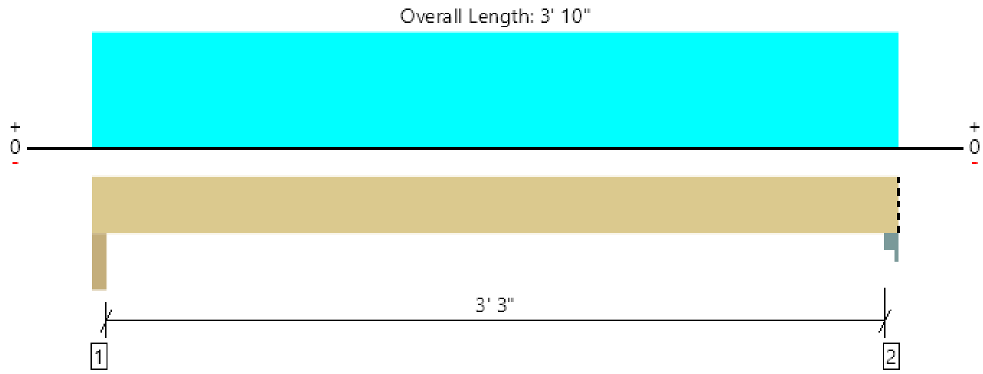
Vertical Load	Location (Side)	Spacing	Dead (0.90)	Snow (1.15)	Comments
1 - Uniform (PSF)	0 to 24' 4 1/2"	16"	15.0	25.0	Roof Load

ForteWEB Software Operator	Job Notes
Harrison Kliegl L120 Engineering (425) 636-3313 hkliegl@l120engineering.com	

Weyerhaeuser

Low Roof, LRB-1

1 piece(s) 4 x 8 DF 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)	1044 @ 2"	7656 (3.50")	Passed (14%)	--	1.0 D + 1.0 S (All Spans)
Shear (lbs)	556 @ 10 3/4"	3502	Passed (16%)	1.15	1.0 D + 1.0 S (All Spans)
Moment (Ft-lbs)	834 @ 1' 11"	3438	Passed (24%)	1.15	1.0 D + 1.0 S (All Spans)
Live Load Defl. (in)	0.006 @ 1' 11"	0.175	Passed (L/999+)	--	1.0 D + 1.0 S (All Spans)
Total Load Defl. (in)	0.010 @ 1' 11"	0.233	Passed (L/999+)	--	1.0 D + 1.0 S (All Spans)

System : Roof
Member Type : Drop Beam
Building Use : Residential
Building Code : IBC 2018
Design Methodology : ASD
Member Pitch : 0/12

- Deflection criteria: LL (L/240) and TL (L/180).
- 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	Factored	
1 - Trimmer - HF	3.50"	3.50"	1.50"	402	643	1044	None
2 - Column Cap - steel	3.50"	3.50"	1.50"	402	643	1044	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)	3' 10" o/c	
Bottom Edge (Lu)	3' 10" o/c	

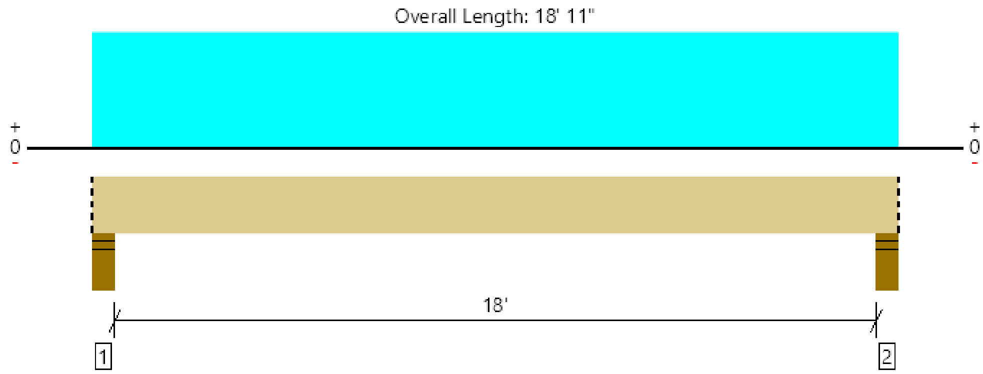
- Maximum allowable bracing intervals based on applied load.

Vertical Loads	Location (Side)	Tributary Width	Dead (0.90)	Snow (1.15)	Comments
0 - Self Weight (PLF)	0 to 3' 10"	N/A	6.4	--	
1 - Uniform (PLF)	0 to 3' 10" (Top)	N/A	203.3	335.3	Linked from: LRJ-1, Support 2

Weyerhaeuser Notes

ForteWEB Software Operator	Job Notes
Harrison Kliegl L120 Engineering (425) 636-3313 hkliegl@l120engineering.com	

1st Floor, 1B-1 (Garage Header)
1 piece(s) 5 1/2" x 10 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)	833 @ 4"	12251 (5.50")	Passed (7%)	--	1.0 D + 1.0 S (All Spans)
Shear (lbs)	715 @ 1' 4"	11733	Passed (6%)	1.15	1.0 D + 1.0 S (All Spans)
Pos Moment (Ft-lbs)	3665 @ 9' 5 1/2"	23244	Passed (16%)	1.15	1.0 D + 1.0 S (All Spans)
Live Load Defl. (in)	0.131 @ 9' 5 1/2"	0.456	Passed (L/999+)	--	1.0 D + 1.0 S (All Spans)
Total Load Defl. (in)	0.230 @ 9' 5 1/2"	0.913	Passed (L/952)	--	1.0 D + 1.0 S (All Spans)

System : Floor
Member Type : Flush Beam
Building Use : Residential
Building Code : IBC 2018
Design Methodology : ASD

- Deflection criteria: LL (L/480) 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 = 18' 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	Factored	
1 - Stud wall - HF	5.50"	5.50"	1.50"	360	473	833	Blocking
2 - Stud wall - HF	5.50"	5.50"	1.50"	360	473	833	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)	18' 11" o/c	
Bottom Edge (Lu)	18' 11" o/c	

•Maximum allowable bracing intervals based on applied load.

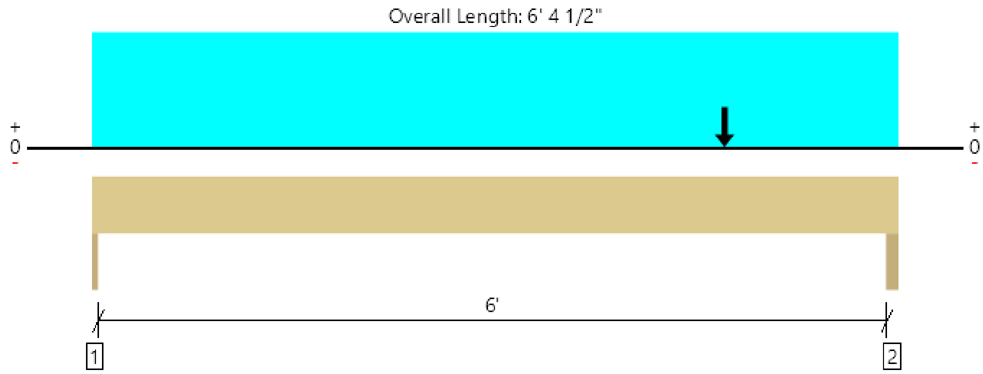
Vertical Loads	Location (Side)	Tributary Width	Dead (0.90)	Snow (1.15)	Comments
0 - Self Weight (PLF)	0 to 18' 11"	N/A	14.0	--	
1 - Uniform (PSF)	0 to 18' 11" (Top)	2'	12.0	25.0	Low Roof Load

Weyerhaeuser Notes

ForteWEB Software Operator	Job Notes
Harrison Kliegl L120 Engineering (425) 636-3313 hkliegl@l120engineering.com	

1st Floor, 1H-1

1 piece(s) 5 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)	7300 @ 6' 3"	10725 (3.00")	Passed (68%)	--	1.0 D + 1.0 L (All Spans)
Shear (lbs)	6868 @ 5' 4 1/2"	8745	Passed (79%)	1.00	1.0 D + 1.0 L (All Spans)
Pos Moment (Ft-lbs)	8720 @ 5'	14774	Passed (59%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.052 @ 3' 4 9/16"	0.208	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.087 @ 3' 4 5/8"	0.313	Passed (L/865)	--	1.0 D + 1.0 L (All Spans)

System : Wall
Member Type : Header
Building Use : Residential
Building Code : IBC 2018
Design Methodology : ASD

- Deflection criteria: LL (L/360) and TL (L/240).
- A 0.5% decrease in the moment capacity has been added to account for lateral stability.
- 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	Floor Live	Snow	Factored	
1 - Trimmer - HF	1.50"	1.50"	1.50"	1119	1706	218	2824	None
2 - Trimmer - HF	3.00"	3.00"	2.04"	2906	4394	873	7300	None

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	End Bearing Points	
Bottom Edge (Lu)	End Bearing Points	

Vertical Loads	Location	Tributary Width	Dead (0.90)	Floor Live (1.00)	Snow (1.15)	Comments
0 - Self Weight (PLF)	0 to 6' 4 1/2"	N/A	12.0	--	--	
1 - Uniform (PSF)	0 to 6' 4 1/2"	10' 6"	15.0	25.0	-	Low Roof Load
2 - Point (lb)	5'	N/A	2944	4426	1091	Linked from: 2B-10, Support 1

Weyerhaeuser Notes

ForteWEB Software Operator	Job Notes
Harrison Kliegl L120 Engineering (425) 636-3313 hkliegl@l120engineering.com	

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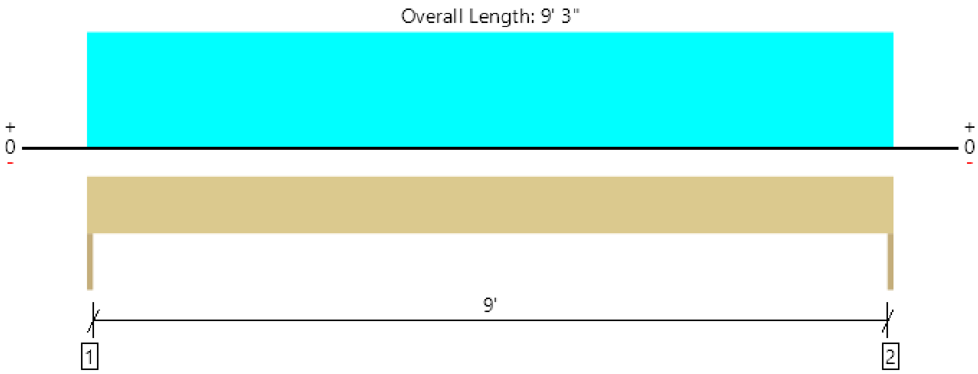
MEMBER REPORT

PASSED

1st Floor, 1H-2

1 piece(s) 6 x 10 DF 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)	2226 @ 0	5156 (1.50")	Passed (43%)	--	1.0 D + 1.0 L (All Spans)
Shear (lbs)	1785 @ 11"	5922	Passed (30%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	5147 @ 4' 7 1/2"	5998	Passed (86%)	1.00	1.0 D + 1.0 L (All Spans)
Vert Live Load Defl. (in)	0.116 @ 4' 7 1/2"	0.308	Passed (L/956)	--	1.0 D + 1.0 L (All Spans)
Vert Total Load Defl. (in)	0.155 @ 4' 7 1/2"	0.463	Passed (L/715)	--	1.0 D + 1.0 L (All Spans)
Lat Member Reaction (lbs)	207 @ 9' 3"	N/A	Passed (N/A)	1.60	1.0 D + 0.6 W
Lat Shear (lbs)	181 @ 7"	9475	Passed (2%)	1.60	1.0 D + 0.6 W
Lat Moment (Ft-lbs)	478 @ mid-span	5588	Passed (9%)	1.60	1.0 D + 0.6 W
Lat Deflection (in)	0.030 @ mid-span	0.925	Passed (L/999+)	--	1.0 D + 0.6 W
Bi-Axial Bending	0.50	1.00	Passed (50%)	1.60	1.0 D + 0.45 W + 0.75 L + 0.75 Lr

System : Wall
Member Type : Header
Building Use : Residential
Building Code : IBC 2018
Design Methodology : ASD

- Deflection criteria: LL (L/360) and TL (L/240).
- Lateral deflection criteria: Wind (L/120)
- A 0.6% decrease in the moment capacity has been added to account for lateral stability.
- Lumber grading provisions must be extended over the length of the member per NDS 4.2.5.5.
- Applicable calculations are based on NDS.

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Floor Live	Factored	
1 - Trimmer - HF	1.50"	1.50"	1.50"	561	1665	2226	None
2 - Trimmer - HF	1.50"	1.50"	1.50"	561	1665	2226	None

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	End Bearing Points	
Bottom Edge (Lu)	End Bearing Points	

Lateral Connections

Supports	Plate Size	Plate Material	Connector	Type/Model	Quantity	Nailing
Left	2X	Hem Fir	Nails	8d (0.113" x 2 1/2") (Toe)	3	
Right	2X	Hem Fir	Nails	8d (0.113" x 2 1/2") (Toe)	3	

Vertical Loads	Location	Tributary Width	Dead (0.90)	Floor Live (1.00)	Comments
0 - Self Weight (PLF)	0 to 9' 3"	N/A	13.2	--	
1 - Uniform (PSF)	0 to 9' 3"	9'	12.0	40.0	Floor Load

Lateral Load	Location	Tributary Width	Wind (1.60)	Comments
1 - Uniform (PSF)	Full Length	3'	24.9	

- ASCE/SEI 7 Sec. 30.4: Exposure Category (B), Mean Roof Height (33'), Topographic Factor (1.0), Wind Directionality Factor (0.85), Basic Wind Speed (115), Risk Category(II), Effective Wind Area determined using full member span and trib. width.
- IBC Table 1604.3, footnote f: Deflection checks are performed using 42% of this lateral wind load.

ForteWEB Software Operator	Job Notes
Harrison Kliegl L120 Engineering (425) 636-3313 hkliegl@l120engineering.com	

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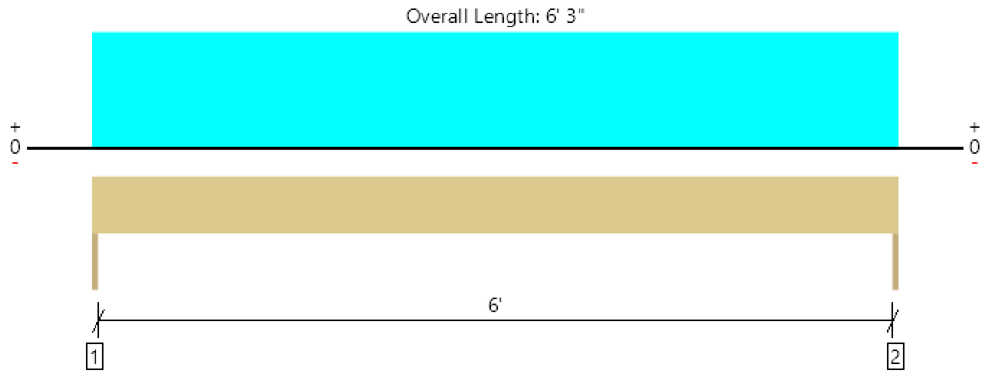
MEMBER REPORT

PASSED

1st Floor, 1H-3

2 piece(s) 2 x 8 DF 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)	251 @ 0	2813 (1.50")	Passed (9%)	--	1.0 D + 1.0 S (All Spans)
Shear (lbs)	192 @ 8 3/4"	3002	Passed (6%)	1.15	1.0 D + 1.0 S (All Spans)
Moment (Ft-lbs)	392 @ 3' 1 1/2"	2592	Passed (15%)	1.15	1.0 D + 1.0 S (All Spans)
Live Load Defl. (in)	0.011 @ 3' 1 1/2"	0.208	Passed (L/999+)	--	1.0 D + 1.0 S (All Spans)
Total Load Defl. (in)	0.018 @ 3' 1 1/2"	0.313	Passed (L/999+)	--	1.0 D + 1.0 S (All Spans)

System : Wall
Member Type : Header
Building Use : Residential
Building Code : IBC 2018
Design Methodology : ASD

- Deflection criteria: LL (L/360) and TL (L/240).
- A 4.7% decrease in the moment capacity has been added to account for lateral stability.
- Applicable calculations are based on NDS.

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Snow	Factored	
1 - Trimmer - HF	1.50"	1.50"	1.50"	95	156	251	None
2 - Trimmer - HF	1.50"	1.50"	1.50"	95	156	251	None

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	End Bearing Points	
Bottom Edge (Lu)	End Bearing Points	

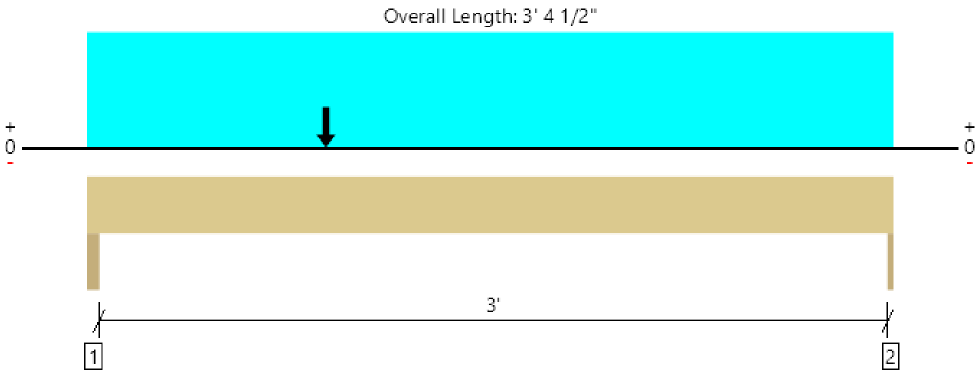
Vertical Loads	Location	Tributary Width	Dead (0.90)	Snow (1.15)	Comments
0 - Self Weight (PLF)	0 to 6' 3"	N/A	5.5	--	
1 - Uniform (PSF)	0 to 6' 3"	2'	12.4	25.0	Low Roof Load

Weyerhaeuser Notes

ForteWEB Software Operator	Job Notes
Harrison Kliegl L120 Engineering (425) 636-3313 hkliegl@l120engineering.com	

1st Floor, 1H-4

3 piece(s) 2 x 10 DF 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)	2770 @ 3' 4 1/2"	4219 (1.50")	Passed (66%)	--	1.0 D + 1.0 L (All Spans)
Shear (lbs)	3270 @ 1' 1/4"	4995	Passed (65%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	3396 @ 1'	5219	Passed (65%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.007 @ 1' 8 9/16"	0.108	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.013 @ 1' 8 3/8"	0.162	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)

System : Wall
Member Type : Header
Building Use : Residential
Building Code : IBC 2018
Design Methodology : ASD

- Deflection criteria: LL (L/360) and TL (L/240).
- A 1.4% decrease in the moment capacity has been added to account for lateral stability.
- Applicable calculations are based on NDS.

Supports	Bearing Length			Loads to Supports (lbs)				Accessories
	Total	Available	Required	Dead	Floor Live	Snow	Factored	
1 - Trimmer - HF	3.00"	3.00"	1.61"	2238	2277	482	4516	None
2 - Trimmer - HF	1.50"	1.50"	1.50"	1088	1682	178	2770	None

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	End Bearing Points	
Bottom Edge (Lu)	End Bearing Points	

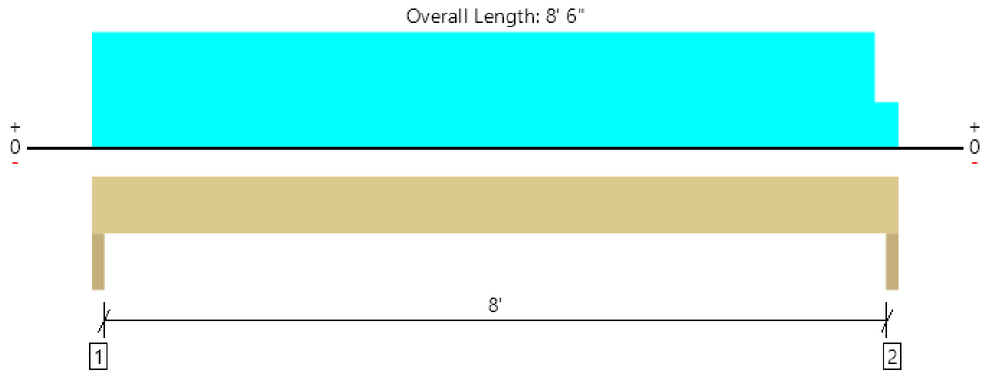
Vertical Loads	Location	Tributary Width	Dead (0.90)	Floor Live (1.00)	Snow (1.15)	Comments
0 - Self Weight (PLF)	0 to 3' 4 1/2"	N/A	10.6	--	--	
1 - Uniform (PSF)	0 to 3' 4 1/2"	21' 6"	12.0	40.0	-	Floor Load
2 - Point (lb)	1'	N/A	2420	1057	660	Linked from: 2B-8, Support 1

Weyerhaeuser Notes

ForteWEB Software Operator	Job Notes
Harrison Kliegl L120 Engineering (425) 636-3313 hkliegl@l120engineering.com	

1st Floor, 1H-5

1 piece(s) 5 1/2" x 10 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)	5734 @ 1 1/2"	10725 (3.00")	Passed (53%)	--	1.0 D + 0.75 L + 0.75 S (All Spans)
Shear (lbs)	4218 @ 7' 4 1/2"	11733	Passed (36%)	1.15	1.0 D + 0.75 L + 0.75 S (All Spans)
Pos Moment (Ft-lbs)	11478 @ 4' 3"	23024	Passed (50%)	1.15	1.0 D + 0.75 L + 0.75 S (All Spans)
Live Load Defl. (in)	0.082 @ 4' 3"	0.275	Passed (L/999+)	--	1.0 D + 0.75 L + 0.75 S (All Spans)
Total Load Defl. (in)	0.147 @ 4' 3"	0.412	Passed (L/672)	--	1.0 D + 0.75 L + 0.75 S (All Spans)

System : Wall
Member Type : Header
Building Use : Residential
Building Code : IBC 2018
Design Methodology : ASD

- Deflection criteria: LL (L/360) and TL (L/240).
- A 0.9% decrease in the moment capacity has been added to account for lateral stability.
- Critical positive moment adjusted by a volume factor of 1.00 that was calculated using length L = 8' 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	Snow	Factored	
1 - Trimmer - HF	3.00"	3.00"	1.60"	2550	1700	2546	5734	None
2 - Trimmer - HF	3.00"	3.00"	1.55"	2459	1700	2398	5532	None

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	End Bearing Points	
Bottom Edge (Lu)	End Bearing Points	

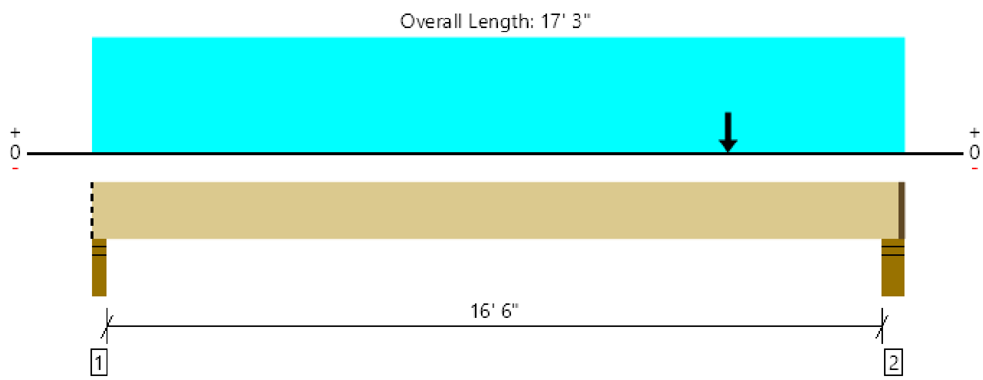
Vertical Loads	Location	Tributary Width	Dead (0.90)	Floor Live (1.00)	Snow (1.15)	Comments
0 - Self Weight (PLF)	0 to 8' 6"	N/A	14.0	--	--	
1 - Uniform (PSF)	0 to 8' 6"	10'	12.0	40.0	-	Floor Load
2 - Uniform (PLF)	0 to 8' 6"	N/A	100.0	-	-	Wall Load Above
3 - Uniform (PLF)	0 to 8' 3"	N/A	366.0	-	599.3	Linked from: RJ-2, Support 2

Weyerhaeuser Notes

ForteWEB Software Operator	Job Notes
Harrison Kliegl L120 Engineering (425) 636-3313 hkliegl@l120engineering.com	

Basement, BB-1

1 piece(s) 5 1/4" x 18" 2.2E Parallam® PSL



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)	5290 @ 16' 11"	8505 (4.00")	Passed (62%)	--	1.0 D + 0.75 L + 0.75 S (All Spans)
Shear (lbs)	5159 @ 15' 3 1/2"	21011	Passed (25%)	1.15	1.0 D + 0.75 L + 0.75 S (All Spans)
Moment (Ft-lbs)	17605 @ 13' 6"	75322	Passed (23%)	1.15	1.0 D + 0.75 L + 0.75 S (All Spans)
Live Load Defl. (in)	0.066 @ 9' 5 1/8"	0.419	Passed (L/999+)	--	1.0 D + 0.75 L + 0.75 S (All Spans)
Total Load Defl. (in)	0.142 @ 9' 4 7/8"	0.837	Passed (L/999+)	--	1.0 D + 0.75 L + 0.75 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 2018
Design Methodology : ASD

Supports	Bearing Length			Loads to Supports (lbs)				Accessories
	Total	Available	Required	Dead	Floor Live	Snow	Factored	
1 - Stud wall - HF	3.50"	3.50"	1.50"	982	722	381	1809	Blocking
2 - Stud wall - HF	5.50"	4.00"	2.49"	2806	1833	1486	5295	1 1/2" Rim Board

- Rim Board is assumed to carry all loads applied directly above it, bypassing the member being designed.
- 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)	17' 2" o/c	
Bottom Edge (Lu)	17' 2" 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 17' 1 1/2"	N/A	29.5	--	--	
1 - Uniform (PSF)	0 to 17' 3" (Front)	1'	12.0	40.0	-	Floor Load
2 - Point (lb)	13' 6" (Top)	N/A	1039	478	299	Linked from: 2B-3, Support 2
3 - Point (lb)	13' 6" (Top)	N/A	2036	1387	1568	Linked from: 2B-5, Support 2

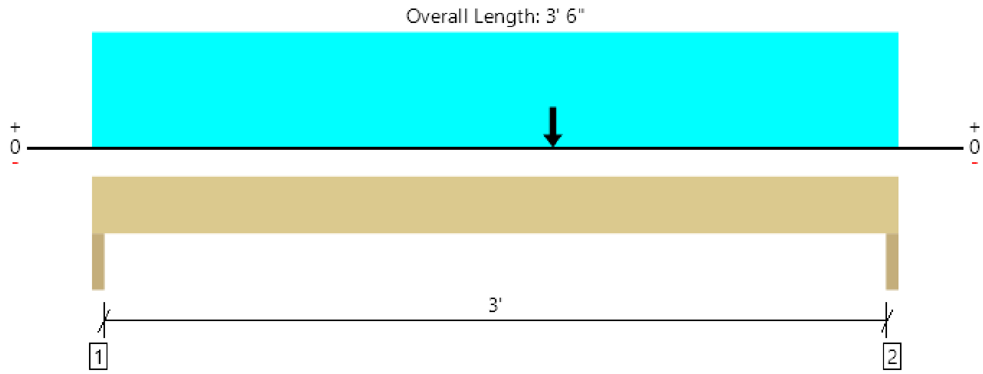
Weyerhaeuser Notes

ForteWEB Software Operator	Job Notes
Harrison Kliegl L120 Engineering (425) 636-3313 hkliegl@l120engineering.com	

Weyerhaeuser

Basement, BH-1

1 piece(s) 3 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)	4685 @ 3' 4 1/2"	6825 (3.00")	Passed (69%)	--	1.0 D + 1.0 L (All Spans)
Shear (lbs)	3905 @ 2' 7 1/2"	4638	Passed (84%)	1.00	1.0 D + 1.0 L (All Spans)
Pos Moment (Ft-lbs)	5446 @ 2'	6525	Passed (83%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.028 @ 1' 9 3/16"	0.108	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.039 @ 1' 9 3/16"	0.162	Passed (L/991)	--	1.0 D + 1.0 L (All Spans)

System : Wall
Member Type : Header
Building Use : Residential
Building Code : IBC 2018
Design Methodology : ASD

- Deflection criteria: LL (L/360) and TL (L/240).
- A 0.6% decrease in the moment capacity has been added to account for lateral stability.
- Critical positive moment adjusted by a volume factor of 1.00 that was calculated using length L = 3' 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	Factored	
1 - Trimmer - HF	3.00"	3.00"	1.69"	1060	2791	3851	None
2 - Trimmer - HF	3.00"	3.00"	2.06"	1312	3373	4685	None

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	End Bearing Points	
Bottom Edge (Lu)	End Bearing Points	

Vertical Loads	Location	Tributary Width	Dead (0.90)	Floor Live (1.00)	Comments
0 - Self Weight (PLF)	0 to 3' 6"	N/A	6.4	--	
1 - Uniform (PSF)	0 to 3' 6"	17'	12.0	40.0	Floor Load
2 - Point (lb)	2'	N/A	1636	3783	Linked from: 2B-1, Support 1

ForteWEB Software Operator	Job Notes
Harrison Kliegl L120 Engineering (425) 636-3313 hkliegl@l120engineering.com	

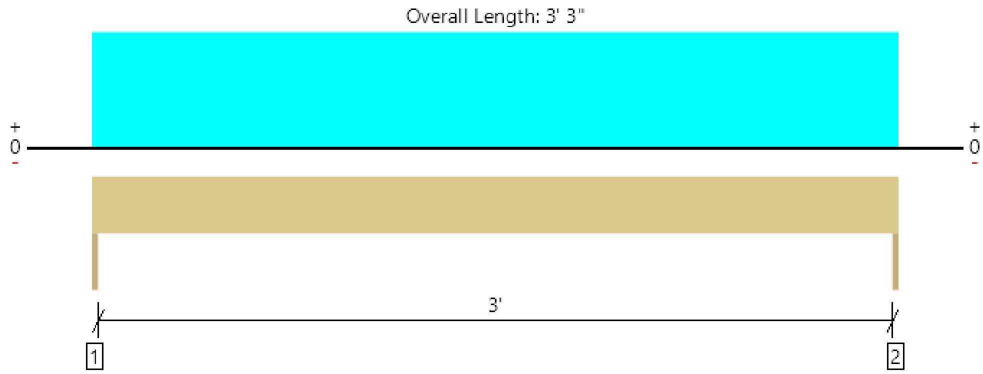
FORTEWEB

MEMBER REPORT

PASSED

Basement, BH-2

2 piece(s) 2 x 8 DF 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)	981 @ 0	2813 (1.50")	Passed (35%)	--	1.0 D + 1.0 L (All Spans)
Shear (lbs)	541 @ 8 3/4"	2610	Passed (21%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	797 @ 1' 7 1/2"	2327	Passed (34%)	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)	0.010 @ 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 2018
Design Methodology : ASD

- Deflection criteria: LL (L/360) and TL (L/240).
- A 1.6% decrease in the moment capacity has been added to account for lateral stability.
- Applicable calculations are based on NDS.

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Floor Live	Factored	
1 - Trimmer - HF	1.50"	1.50"	1.50"	233	748	981	None
2 - Trimmer - HF	1.50"	1.50"	1.50"	233	748	981	None

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	End Bearing Points	
Bottom Edge (Lu)	End Bearing Points	

Vertical Loads	Location	Tributary Width	Dead (0.90)	Floor Live (1.00)	Comments
0 - Self Weight (PLF)	0 to 3' 3"	N/A	5.5	--	
1 - Uniform (PSF)	0 to 3' 3"	11' 6"	12.0	40.0	Floor Load

Weyerhaeuser Notes

ForteWEB Software Operator	Job Notes
Harrison Kliegl L120 Engineering (425) 636-3313 hkliegl@l120engineering.com	

Weyerhaeuser

11/10/2022 4:12:44 PM UTC

ForteWEB v3.4, Engine: V8.2.2.122, Data: V8.1.3.0

File Name: Forest Creek Lot 2

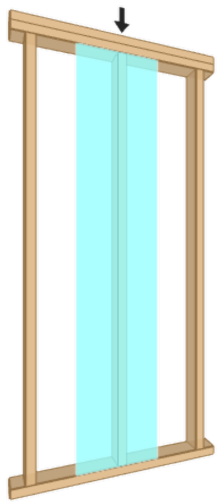
FORTEWEB

MEMBER REPORT

PASSED

1st Floor, Balloon Framed Wall Check
1 piece(s) 2 x 6 HF No.2 @ 12" OC

Wall Height: 20'   Member Height: 19' 7 1/2"   O. C. Spacing: 12.00"



Drawing is Conceptual

Design Results	Actual	Allowed	Result	LDF	Load: Combination
Slenderness	43	50	Passed (86%)	--	--
Compression (lbs)	440	1690	Passed (26%)	1.15	1.0 D + 1.0 S
Plate Bearing (lbs)	440	4177	Passed (11%)	--	1.0 D + 1.0 S
Lateral Reaction (lbs)	132	--	--	1.60	1.0 D + 0.6 W
Lateral Shear (lbs)	126	1320	Passed (10%)	1.60	1.0 D + 0.6 W
Lateral Moment (ft-lbs)	649 @ mid-span	1264	Passed (51%)	1.60	1.0 D + 0.6 W
Total Deflection (in)	1.16 @ mid-span	1.96	Passed (L/202)	--	1.0 D + 0.6 W
Bending/Compression	0.58	1	Passed (58%)	1.60	1.0 D + 0.6 W

- Lateral deflection criteria: Wind (L/120)
- Input axial load eccentricity for the design is zero
- Applicable calculations are based on NDS.
- A bearing area factor of 1.25 has been applied to base plate bearing capacity.
- A 15% increase in the moment capacity has been added to account for repetitive member usage.

Supports	Type	Material
Top	Dbl 2X	Hem Fir
Base	2X	Hem Fir

System : Wall
Member Type : Stud
Building Code : IBC 2018
Design Methodology : ASD

Max Unbraced Length	Comments
1'	

Lateral Connections

Supports	Connector	Type/Model	Quantity	Connector Nailing
Top	Nails	8d (0.113" x 2 1/2") (Toe)	2	N/A
Base	Nails	8d (0.113" x 2 1/2") (Toe)	2	N/A

- Nailed connection at the top of the member is assumed to be nailed through the bottom 2x plate prior to placement of the top 2x of the double top plate assembly.

Vertical Load	Spacing	Dead (0.90)	Snow (1.15)	Comments
1 - Point (lb)	N/A	165	275	Roof Load DL= 15psf * 11 ft SL= 25psf * 11 ft

Lateral Load	Location	Spacing	Wind (1.60)	Comments
1 - Uniform (PSF)	Full Length	12.00"	22.5	

- ASCE/SEI 7 Sec. 30.4: Exposure Category (B), Mean Roof Height (33'), Topographic Factor (1.0), Wind Directionality Factor (0.85), Basic Wind Speed (115), Risk Category(II), Effective Wind Area determined using full member span and trib. width.
- IBC Table 1604.3, footnote f: Deflection checks are performed using 42% of this lateral wind load.

Weyerhaeuser Notes

ForteWEB Software Operator	Job Notes
Harrison Kliegl L120 Engineering (425) 636-3313 hkliegl@l120engineering.com	

Weyerhaeuser

11/10/2022 4:26:41 PM UTC
ForteWEB v3.4, Engine: V8.2.2.122, Data: V8.1.3.0
File Name: Forest Creek Lot 2

LONGITUDE
ONE TWENTY°
ENGINEERING & DESIGN

LATERAL CALCULATIONS

SHEAR-WALL REFERENCE PER PLAN

Project Number: S22201	Plan Name: Forest Creek Estates Lot 2	Sheet Number: DC
Engineer: HK	Specifics: Design Criteria	Date: 11/10/2022

Gravity Criteria: Code: IBC 2018

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/Mech	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
Miscellaneous	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	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 7-16

R = **6.5** Bearing Wall System, Wood Structural Panel Walls

Mapped Spectral Acceleration, Ss = **1.45**

Mapped Spectral Acceleration, S1 = **0.503**

Soil Site Class = **D**

WIND PARAMETERS:

Code Reference: ASCE 7-16

Basic Wind Speed (3 second Gust) = **100** mph

Exposure : **B**

Kzt = **1.00**

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 = **250** pcf

Soil Friction Coeff. = **0.35**

Project Number: S22201	Plan: Forest Creek Estates Lot 2	Sheet Number: L1
Engineer: HK	Specifics: WIND FORCES	Date 11/10/2022

IBC 2018 Section 1609 → ASCE 7-16 Section 28.5 - Simplified Procedure → Main Wind-Force Resisting System

LOAD CRITERIA:

Basic Wind Speed, V_s =	100 mph		(ASCE 7-16, Section 26.5)
Exposure =	B		(ASCE 7-16, Section 26.7)

BUILDING GEOMETRY:

Roof Slope =	3.00 :12	= 14.04 degrees	
Loads From Front/Back - Width (ft)=	71.00 ft	Roof: Gable	
Loads From Side - Width (ft) =	65.00 ft	Roof: Gable	
Average Eave Height =	25.00 ft		
Mean Roof Ht. , h =	30.00 ft		(ASCE 7-16, Figure 27.5-2)
Edge Strip Width, a =	6.5 ft		(ASCE 7-16, Figure 28.5-1)
End Zone Width, 2a =	13.00 ft		(ASCE 7-16, Figure 28.5-1)

DESIGN:

Topographic Factor , Kzt =	**1.00**	(ASCE 7-16, Section 26.8)
Adjustment Factor, λ =	1.05	(ASCE 7-16, Figure 28.5-1)

WIND LOAD SUMMARY:

Front / Back Direction

Roof	6.84 k
2nd Floor	8.61 k
1st Floor	3.71 k
Basement (Base Shear)	**19.16 k**

Side / Side Direction

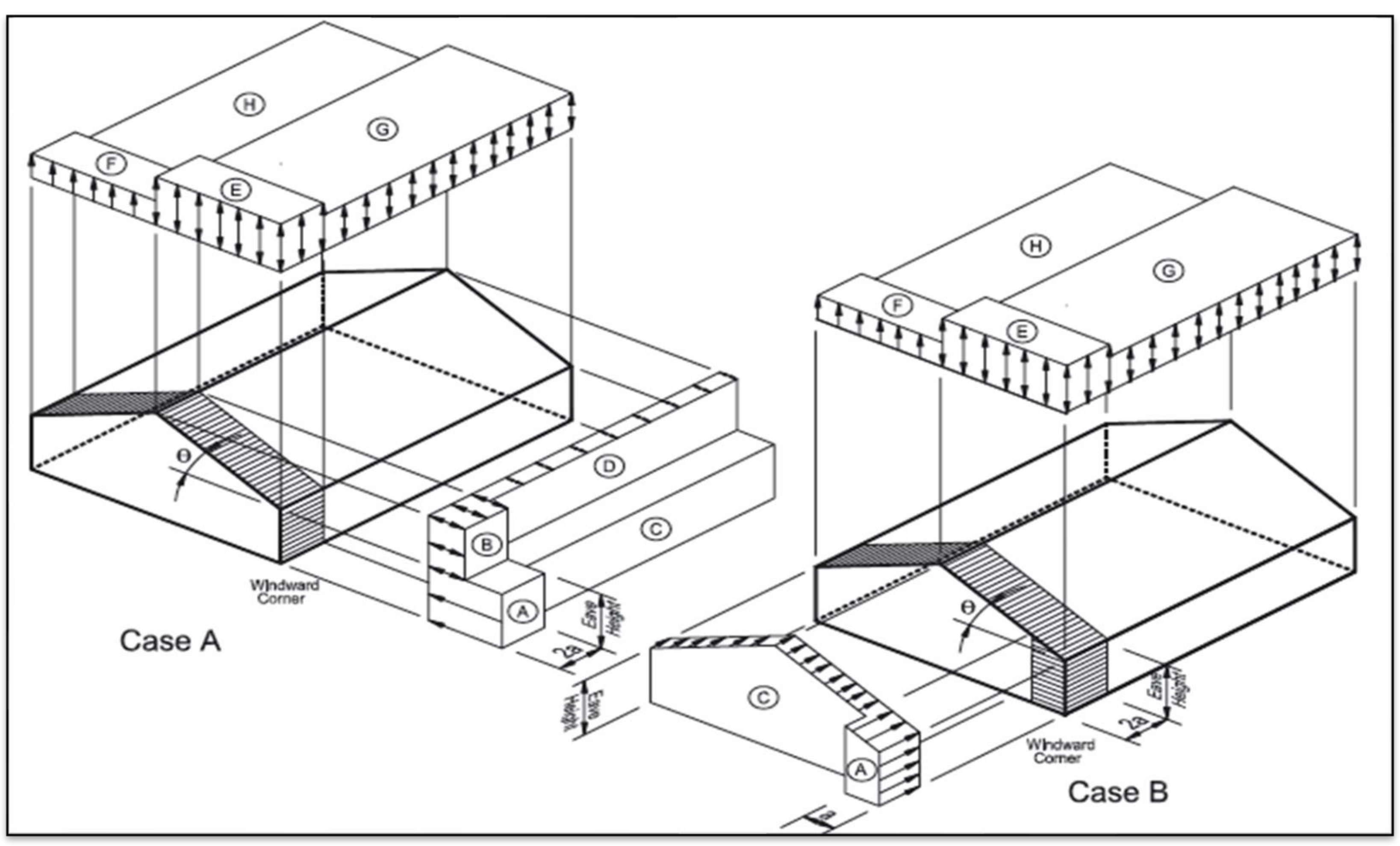
Roof	6.23 k
2nd Floor	8.11 k
1st Floor	1.48 k
Basement (Base Shear)	**15.82 k**

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 A	B	C	D	Vertical Presssure E	F	G	H	Overhang E_{OH}	G_{OH}
100	14.04	A	19.90	-6.60	13.30	-3.80	-19.10	-12.40	-13.30	-9.50	-26.70	-20.90

* *Values Interpolated from Figure 28.5-1 ASCE 7 - 16*



Project Number: S22201	Plan: Forest Creek Estates Lot 2	Sheet Number: L1
Engineer: HK	Specifics: WIND FORCES	Date 11/10/2022

IBC 2018 Section 1609 → ASCE 7-16 Section 28.5 - Simplified Procedure → Main Wind-Force Resisting System

HORIZONTAL LOADS (psf) $p_s = \lambda * Kzt*Ps30$				MIN. LOADS (psf) Per ASCE 7-16, 28.6.3	
End zone		Interior zone		Roof	Wall
A (Wall)	B (Roof)	C (Wall)	D (Roof)		
20.90	-6.93	13.97	-3.99	8.0	16.0

Full Impact at Basement? YES (No = 1/4 Impact)

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)	72.0	3.00	(roof)	13.0	20.90	59.0	13.97	2.56	1.35
	Plate to Mid 2nd LVL	72.0	5.00	(wall)	13.0	20.90	59.0	13.97	4.27	4.49
								Σ =	6.84	5.84
2nd FLOOR	Mid 2nd LVL to Floor	72.0	5.00	(wall)	13.0	20.90	59.0	13.97	4.27	4.49
	"Height" Low-Roof to Plate (see note)	0.0	0.00	(roof)	13.0	20.90	-13.0	13.97	0.00	0.00
	Floor to Mid 1st LVL	66.0	5.00	(wall)	13.0	20.90	53.0	13.97	3.95	4.12
								Σ =	8.22	8.61
1st FLOOR	Mid 1st LVL to Floor	53.0	5.00	(wall)	13.0	20.90	40.0	13.97	3.24	3.31
	"Height" Low-Roof to Plate (see note)	0.0	0.00	(roof)	13.0	20.90	-13.0	13.97	0.00	0.00
	Floor to Mid Basement LVL	66.0	5.00	(wall)	13.0	20.90	53.0	13.97	3.95	4.12
								Σ =	3.59	3.71
								Total Wind Base Shear (kips)	18.65	18.16

Full Impact at Basement? NO (No = 1/4 Impact)

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)	65.0	3.00	(roof)	13.0	20.90	52.0	13.97	2.33	1.22
	Plate to Mid 2nd LVL	65.0	5.00	(wall)	13.0	20.90	52.0	13.97	3.89	4.06
								Σ =	6.23	5.27
2nd FLOOR	Mid 2nd LVL to Floor	65.0	5.00	(wall)	13.0	20.90	52.0	13.97	3.89	4.06
	"Height" Low-Roof to Plate (see note)	0.0	0.00	(roof)	13.0	20.90	-13.0	13.97	0.00	0.00
	Floor to Mid 1st LVL	65.0	5.00	(wall)	13.0	20.90	52.0	13.97	3.89	4.06
								Σ =	7.78	8.11
1st FLOOR	Mid 1st LVL to Floor	65.0	5.00	(wall)	13.0	20.90	52.0	13.97	3.89	4.06
	"Height" Low-Roof to Plate (see note)	0.0	0.00	(roof)	13.0	20.90	-13.0	13.97	0.00	0.00
	Floor to Mid Basement LVL	30.0	5.00	(wall)	13.0	20.90	17.0	13.97	1.99	1.87
								Σ =	1.47	1.48
								Total Wind Base Shear (kips)	15.48	14.87

Project Number:	Plan Name:	Sheet Number:
S22201	**Forest Creek Estates Lot 2**	**L2**
Engineer:	Specifics:	Date:
HK	**SEISMIC WEIGHTS**	**11/10/2022**

Unit Weights (psf)

Roof:	15	psf
Floor:	12	psf
Exterior Wall:	12	psf
Interior Wall:	8	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)	UNIT WEIGHT (psf)		Item Total Weight. (lbs)	Sub-Total (kips)	Average Pressure (psf)
ROOF:								
	Roof	3,855	1.05	15	=	60,854		
	Ext. Wall Below	285	5.00	12	=	17,100		
	Corridor Wall Below	125	5.00	8	=	5,000		
							83	**22**
2nd FLOOR:								
	Floor	2,670	1.00	12	=	32,040		
	Low Roof	730	1.05	15	=	11,524		
	Ext. Wall Above	285	5.00	12	=	17,100		
	Corridor Wall Above	125	5.00	8	=	5,000		
	Ext. Wall Below	285	4.50	12	=	15,390		
	Corridor Wall Below	100	4.50	8	=	3,600		
							85	**25**
1st FLOOR:								
	Floor	1,930	1.00	12	=	23,160		
	Low Roof	0	1.05	15	=	0		
	Ext. Wall Above	285	4.50	12	=	15,390		
	Corridor Wall Above	100	4.50	8	=	3,600		
	Ext. Wall Below	115	4.50	12	=	6,210		
	Corridor Wall Below	75	4.50	8	=	2,700		
							51	**26**
BASEMENT:								
	Ext. Wall Above	115	4.50	12	=	6,210		
	Corridor Wall Above	75	4.50	8	=	2,700		
							9	

STRUCTURE WEIGHT FOR SEISMIC BASE SHEAR: 219 kips

TOTAL WEIGHT OF STRUCTURE: 228 kips
(Includes Basement Dead Load)

Project Number:	Plan Name:	Sheet Number:
S22201	**Forest Creek Estates Lot 2**	**L3**
Engineer:	Specifics:	Date:
HK	**SEISMIC FORCES**	**11/10/2022**

Equivelant Lateral Force Analysis per IBC 2018 1613.1 →ASCE 7-16 Table 12.6-1 →Sec 12.8

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

S_1 =	0.503		Maps
S_{DS} =	1.16		(ASCE 7 EQ 11.4.-3)
S_{D1} =	0.4024		(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 =	30.0	ft	
Building Period Coefficient, C_T =	0.020		(ASCE 7 Table 12.8.-2)
Approx. Fundamental Period, T_a =	0.256	$(C_{T*}(h_n^{0.75})$	(ASCE 7 EQ 12.8.-7)
Approx. Fundamental Period, T_L =	6.0	sec	(ASCE 7 11.4.6)

Seismic Response Coefficient

$C_s = S_{DS}/(R/I)$	C_s =	0.178		(ASCE 7 EQ 12.8.-2)

Seismic Response Coefficient, Maximum

$C_{s,\ MAX} = S_{D1}/(T{*}R/I)$	$C_{s,\ MAX}$ =	0.241	$T \leq T_L$	(ASCE 7 EQ 12.8.-3)
$C_{s,\ MAX} = S_{D1}\ T_L/(T^2{*}R/I)$	$C_{s,\ MAX}$ =	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 =	**0.178**		
Dead Load W =	219	kips	
V = Cs W =	39.0	kips	(ASCE 7 EQ 12.8.-1)
Q_E = V =	39.0	kips	(ASCE 7 EQ 12.4-3)
ρ =	1.0		(ASCE 7 12.3.4.2)
$E_H = \rho\ Q_E$	39.0	kips	(ASCE 7 EQ 12.4-3)
Ev = .2 S_{DS} D =	0.23	x D kips	

Factor for Alternate Basic Load conbinations - 2018 IBC

E_H/1.4 =	**27.9**	**kips**	IBC 2018 1605.3.2
k =	1		(ASCE 7 12.8.3)

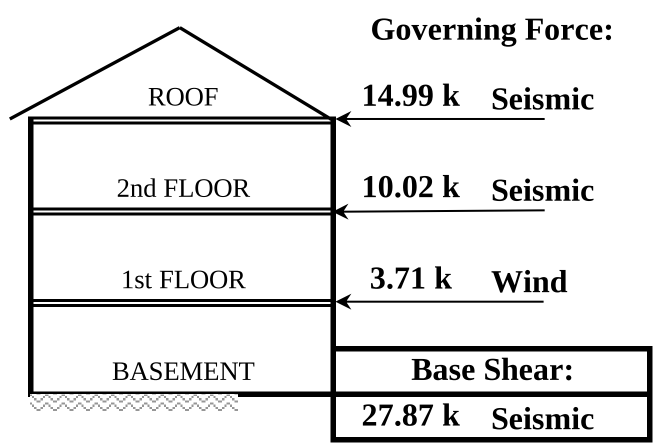
VERTICAL DISTRIBUTION (Per ASCE 7 - 12.8.3)

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	3,855	10.00	29.00	83	2,406	0.54	21.0	**15.0**
2nd	2,670	10.00	19.00	85	1,608	0.36	14.0	**10.0**
1st	1,930	9.00	9.00	51	460	0.10	4.0	**2.9**
				Sum =	4,474	1.000	39.0	**27.9**

Project Number:	Plan Name:	Sheet Number:
S22201	**Forest Creek Estates Lot 2**	**L4**
Engineer:	**Specifics:**	**Date:**
HK	**DESIGN LOADS**	**11/10/2022**

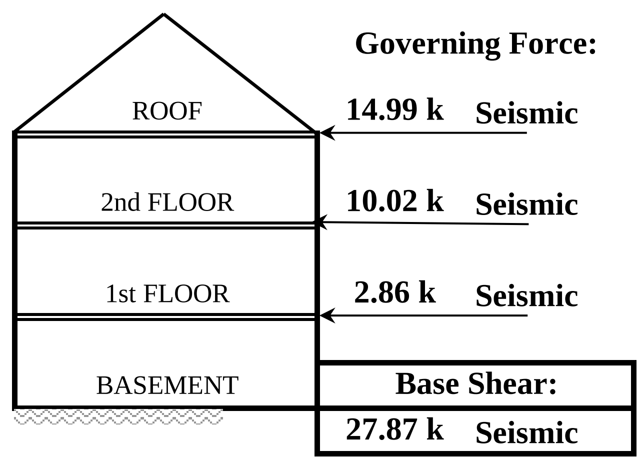
FRONT / BACK DIRECTION

Wind Force *0.6 ω * $W_{F/B}$ (kips)*		Seismic Force *E/1.4 (kips)*	
Per Level	Sum	Per Level	Sum
6.84		14.99	
8.61	6.84	10.02	14.99
3.71	15.45	2.86	25.01
	19.16		27.87



SIDE / SIDE DIRECTION

Wind Force *0.6 ω * W_S (kips)*		Seismic Force *E/1.4 (kips)*	
Per Level	Sum	Per Level	Sum
6.23		14.99	
8.11	6.23	10.02	14.99
1.48	14.34	2.86	25.01
	15.82		27.87



Project Number:	Plan Name:	Sheet Number:
S22201	Forest Creek Estates Lot 2	L5
Engineer:	Specifics:	Date:
HK	Shear walls (front/back)	11/10/2022

Notes:

* All walls designed with Force-Transfer should meet a minimum height to width ratio of 2:1 at Pier (SDPWS 2018, Table 4.3.4)

* Maximum allowed height to width ratio 3.5:1 for walls w/o openings (increased shear design values per SDPWS 2018, Table 4.3.4)

* 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)

Stud Species HF

Story shear(kips) = **14.99**
Story height (ft) = 10.00
Shear Panel height (ft) = 9.00
Total Diaphragm Area (sq ft) = **3855.00**

100% story shear **YES**

Governing Force (F/B Direction) = **Seismic**
Dead load factor (F/B Direction) = **0.90** IBC 2018 Equation 16-22
Shear panel capacity (Wind or Seismic) = **Seismic**
load balance check = OK

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. Area (sq 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)	Sum 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	19.00	5.00	5.00	3.00	2.00	14.00	400.00	1.00	400.00	1.56	1.56	111	1.00	111	SW6	2.00	0.14	0.14	15.6	22.4	-0.37	flr-flr	HF	Edge	No HD	0.75	CS16
2	2.1	12.50					12.50	1355.00	0.51	691.33	2.69	2.69	215	1.00	215	SW6	3.00	0.15	0.15	26.9	10.8	1.34	flr-flr	HF	Edge	MST37	1.34	CS16
2	2.2	26.00	14.00	6.00	2.50	2.00	12.00	1355.00	0.49	663.67	2.58	2.58	215	1.00	215	SW6	4.00	0.17	0.17	25.8	51.1	-0.99	flr-beam	HF	Edge	No HD	0.00	No strap
2	3.1	17.50					17.50	800.00	1.00	800.00	3.11	3.11	178	1.00	178	SW6	5.00	0.18	0.18	31.1	25.2	0.35	flr-beam	HF	Edge	No HD	0.00	No strap
2	4.1	15.33					15.33	1300.00	0.52	679.47	2.64	2.64	172	1.00	172	SW6	6.00	0.20	0.20	26.4	20.9	0.37	flr-beam	HF	Edge	No HD	0.00	No strap
2	4.2	26.00	12.00	6.00	2.50	2.00	14.00	1300.00	0.48	620.53	2.41	2.41	172	1.00	172	SW6	7.00	0.21	0.21	24.1	64.8	-1.59	flr-beam	HF	Edge	No HD	1.08	CS16
	S =	116.33				Total OSB wall length = (feet)	38.50		S =	3855.00	14.99	14.99	OK	Total OSB Capacity (kips)	14.99													

1st Story Walls (Front - Back Direction)

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

Story shear(kips) = **10.02**
Story height (ft) = 10.08
Shear Panel height (ft) = 9.08
Total Diaphragm width (ft) = **3400.00**

Accumulated Shear = **25.01**
load balance check = OK

1st 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	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	1.1	7.25					7.25	690.00	0.46	317.62	0.94	1.65	228	1.00	228	SW6	2.00	0.14	YES	0.28	16.7	3.3	1.98	flr-flr	HF	Edge	MST37	0.00	No strap
1	1.2	14.50	6.00	5.50	2.00	1.50	8.50	690.00	0.54	372.38	1.10	1.94	228	1.00	228	SW6	2.00	0.14	NO	0.14	19.5	13.1	0.46	flr-flr	HF	Edge	No HD	1.29	CS16
1	2.1	12.00					12.00	680.00	0.71	480.00	1.41	4.10	342	1.00	342	SW4	3.00	0.15	YES	0.32	41.4	9.9	2.73	flr-flr	HF	Edge	MST48	0.00	No strap
1	2.2	5.00					5.00	680.00	0.29	200.00	0.59	3.17	634	1.00	634	2W4	4.00	0.17	NO	0.17	32.0	1.9	6.68	flr-conc	HF	Edge	HDU11	0.00	No strap
1	3.1	12.00					12.00	900.00	0.51	459.57	1.35	2.94	245	1.00	245	SW4	5.00	0.18	NO	0.18	29.7	11.9	1.55	flr-flr	HF	Edge	MST37	0.00	No strap
1	3.2	11.50					11.50	900.00	0.49	440.43	1.30	2.82	245	1.00	245	SW4	6.00	0.20	NO	0.20	28.4	11.8	1.51	flr-flr	HF	Edge	MST37	0.00	No strap
1	4.1	12.67					12.67	1130.00	0.55	617.92	1.82	4.46	352	1.00	352	SW3	7.00	0.21	NO	0.21	45.0	15.4	2.43	flr-conc	HF	Edge	STHD14	0.00	No strap
1	4.2	10.50					10.50	1130.00	0.45	512.08	1.51	3.92	374	1.00	374	SW3	8.00	0.23	NO	0.23	39.5	11.3	2.82	flr-conc	HF	Edge	STHD14	0.00	No strap
	S =	85.42				Total OSB wall length = (feet)	27.75		S =	3400.00	8.51	25.01	OK	Total OSB Capacity (kips)	10.02														

Basement Walls (Front - Back Direction)

Story shear(kips) = **3.71**
Story height (ft) = 10.08
Shear Panel height (ft) = 9.08
Total Diaphragm width (ft) = **1930.00**

Accumulated Shear = **3.71** **The rest of the story shear from above has been transferred into foundation**
load balance check = OK

Basement 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	Story DL(klf)	Sum DL(klf)	OTM (k-ft)	RM (k-ft)	Resultant HD(kips)	HD TYPE	Resultant HD	Force at Window (Kips)	Window Strap
B	1.1	7.25					7.25	315.00	0.28	88.69	0.17	1.82	251	1.00	251	SW4	0.25	0.39	5.5	9.2	-0.55	flr-conc	No HD	0.00	No strap
B	1.2	18.50					18.50	315.00	0.72	226.31	0.44	2.37	128	1.00	128	CONCRETE FOUNDATION WALL									
B	2.1	9.67					9.67	475.00	1.00	475.00	0.91	5.02	519	1.00	519	SW2	0.25	0.40	45.6	17.0	3.11	flr-conc	STHD14	0.00	No strap
B	3.1	8.75					8.75	600.00	0.57	344.26	0.66	3.61	412	1.00	412	SW3	0.25	0.43	32.7	14.9	2.16	flr-conc	STHD14	0.00	No strap
B	3.2	6.50					6.50	600.00	0.43	255.74	0.49	3.31	510	1.00	510	SW2	0.25	0.45	30.1	8.5	3.59	flr-conc	HDU5	0.00	No strap
B	4.1	18.00					18.00	540.00	1.00	540.00	1.04	1.04	58	1.00	58	CONCRETE FOUNDATION WALL									
	S =	68.67							S =	1930.00	3.71	17.17	OK												

Notes:

* All walls designed with Force-Transfer should meet a minimum height to width ratio of 2:1 at Pier (SDPWS 2018, Table 4.3.4)

* Maximum allowed height to width ratio 3.5:1 for walls w/o openings (increased shear design values per SDPWS 2018, Table 4.3.4)

* Shear panel height is height to underside or roof or floor framing.

Project Number: S22201	Plan Name: Forest Creek Estates Lot 2	Sheet Number: L6
Engineer: HK	Specifics: Shear walls (side/side)	Date: 11/10/2022

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

2nd Story Walls (Side / Side Direction)

Stud Species HF

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

Story shear(kips) = 14.99
Story height (ft) = 9.08
Shear Panel height (ft) = 8.08
Total Diaphragm width (ft) = 3855.00

100% story shear YES

Governing Force (F/B Direction) = Seismic
Dead load factor (F/B Direction) = 0.90 IBC 2018 Equation 16-22
Shear panel capacity (Wind or Seismic) = Seismic
load balance check = OK

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. Area (sq 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	20.75	5.00	4.00	4.00	1.08	15.75	1100.00	0.70	770.00	2.99	2.99	190	1.00	190	SW6	2.00	0.13	0.13	27.2	24.6	0.13	flr-flr	HF	Edge	No HD	2.17	CS14
2	A2	12.75	6.00	4.50	3.00	1.50	6.75	1100.00	0.30	330.00	1.28	1.28	190	1.00	190	SW6	3.00	0.14	0.14	11.7	10.4	0.10	flr-conc	HF	Edge	No HD	1.43	CS16
2	B1	19.00	9.00	5.00	3.50	3.00	10.00	955.00	0.34	323.73	1.26	1.26	126	1.00	126	SW6	4.00	0.16	0.16	11.4	25.5	-0.76	flr-beam	HF	Edge	No HD	0.81	CS16
2	B2	19.50					19.50	955.00	0.66	631.27	2.45	2.45	126	1.00	126	SW6	5.00	0.17	0.17	22.3	29.4	-0.38	flr-beam	HF	Edge	No HD	0.00	No strap
2	C1	11.00					11.00	1100.00	0.50	550.00	2.14	2.14	194	1.00	194	SW6	6.00	0.19	0.19	19.4	10.2	0.88	flr-flr	HF	Edge	MST37	0.00	No strap
2	C2	11.00					11.00	1100.00	0.50	550.00	2.14	2.14	194	1.00	194	SW6	7.00	0.20	0.20	19.4	11.0	0.80	flr-flr	HF	Edge	MST37	0.00	No strap
2	D1	34.50	6.00	5.00	6.00	1.08	28.50	700.00	1.00	700.00	2.72	2.72	96	1.00	96	SW6	8.00	0.22	0.22	24.7	116.2	-2.69	flr-flr	HF	Edge	No HD	1.90	CS14

S = 128.50 | Total OSB wall length = (feet) 102.50 | S = 3855.00 | 14.99 | **14.99** | **OK** | Total OSB Capacity (kips) **14.99**

1st Story Walls (Side / Side Direction)

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

Shear panel capacity (Wind or Seismic) = Seismic

Story shear(kips) = 10.02
Story height (ft) = 10.08
Shear Panel height (ft) = 9.08
Total Diaphragm width (ft) = 3400.00

Accumulated Shear = 25.01
load balance check = OK

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. Area (sq 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	A1	20.75	9.00	5.00	2.00	1.08	11.75	1000.00	0.75	746.03	2.20	5.19	442	1.00	442	SW3	2.00	0.14	YES	0.27	52.3	52.5	-0.01	flr-flr	HF	Edge	No HD	2.93	CMSTC16
1	A2	13.00	9.00	5.00	2.00	2.00	4.00	1000.00	0.25	253.97	0.75	2.03	508	1.00	508	SW2	3.00	0.16	YES	0.30	20.5	22.7	-0.18	flr-flr	HF	Edge	No HD	2.29	CS14
1	B1	13.00	5.00	5.50	2.00	2.00	8.00	800.00	1.00	800.00	2.36	6.07	759	1.00	759	2W3	4.00	0.17	NO	0.17	61.2	12.8	3.87	flr-conc	HF	Edge	HDU5	3.60	CMSTC16
1	C1	34.00					34.00	800.00	1.00	800.00	2.36	6.64	195	1.00	195	SW6	5.00	0.18	YES	0.35	66.9	183.2	-3.47	flr-conc	HF	Edge	No HD	0.00	No strap
1	D1	25.00					25.00	800.00	1.00	800.00	2.36	5.08	203	1.00	203	SW6	6.00	0.19	YES	0.38	51.2	106.7	-2.26	flr-conc	HF	Edge	No HD	0.00	No strap

S = 105.75 | Total OSB wall length = (feet) 82.75 | S = 3400.00 | 10.02 | **25.01** | **OK** | Total OSB Capacity (kips) **10.02**

2.72 kips of shear from story above is transferred directly into foundation. Accumulated Shear to walls below is OK!

Basement Walls (Side / Side Direction)

Basement Walls (Side / Side Direction)
Hold downs and window straps

Shear panel capacity (Wind or Seismic) = Seismic

Story shear(kips) = 2.86
Story height (ft) = 10.08
Shear Panel height (ft) = 9.08
Total Diaphragm width (ft) = 1930.00

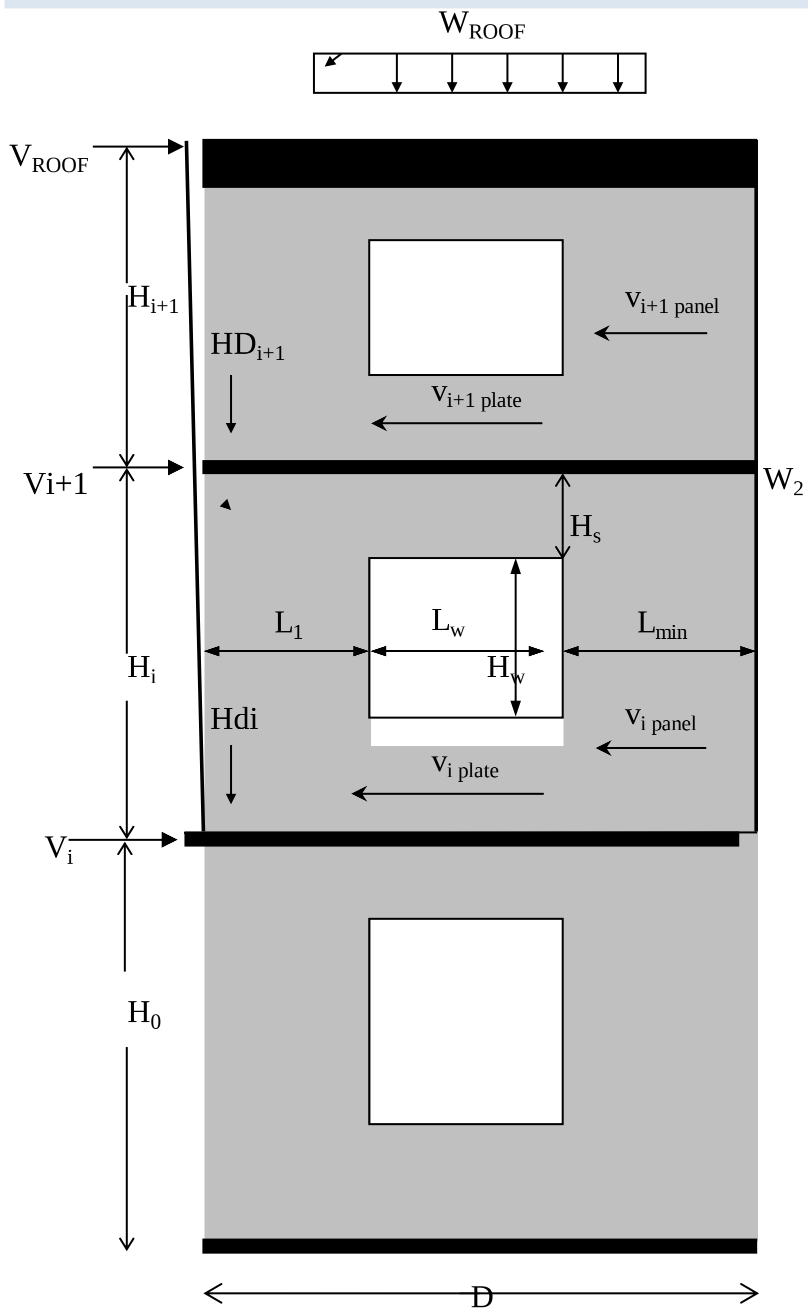
Accumulated Shear = 11.15 **----> Appx. 60% accumulated shear from levels above transferred directly into foundation**
load balance check = OK

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. Area (sq 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
B	A1	8.83					8.83	900.00	0.36	324.37	0.48	2.96	335	1.00	335	SW4	2.00	0.14	NO	0.14	14.8	5.1	1.17	flr-conc	HF	Edge	STHD14	0.00	No strap
B	A2	15.67					15.67	900.00	0.64	575.63	0.85	2.76	176	1.00	176	SW6	3.00	0.16	NO	0.16	34.3	17.3	1.12	flr-conc	HF	Edge	STHD14	0.00	No strap
B	B1	13.00					13.00	740.00	1.00	740.00	1.10	3.55	273	1.00	273	CONCRETE FOUNDATION WALL													
B	C1	34.00					34.00	290.00	1.00	290.00	0.43	1.89	56	1.00	56	CONCRETE FOUNDATION WALL													

S = 71.50 | Total OSB wall length = (feet) 24.50 | S = 1930.00 | 2.86 | **11.16** | **OK** | Total OSB Capacity (kips) **2.86**

Project	Sheet number:
Forest Creek Estates Lot 2	**L7**
Subject	Date
SHEAR WALL EQUATION DIAGRAM	**11/10/2022**

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

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

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

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

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

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

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

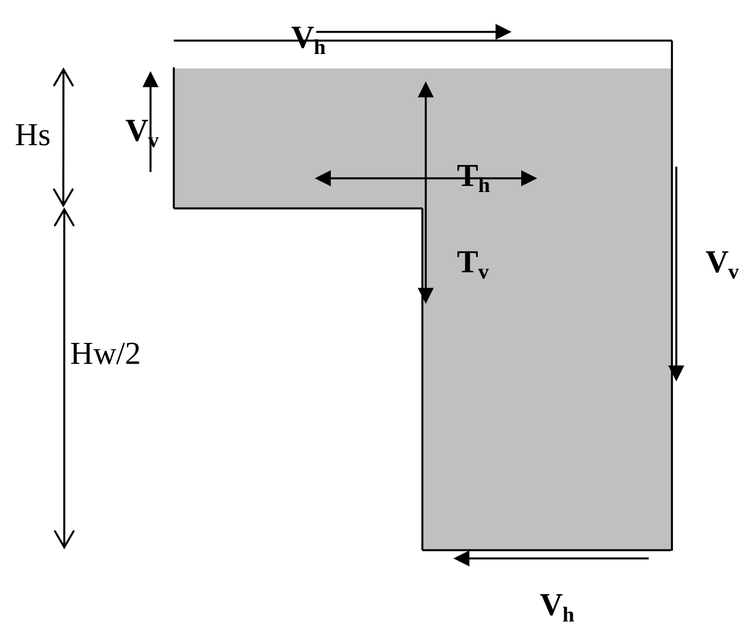
$V_{i+1\ panel} = V_{ROOF} / (L_1 + L_{max})$

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

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

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

FORCE TRANSFER AROUND WINDOW CALCULATION (CANTILEVER PIER METHOD)



$V_h = v_{i\ panel} \times L_{max}$

$V_v = HD_i$

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

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

LONGITUDE
ONE TWENTY°
ENGINEERING & DESIGN

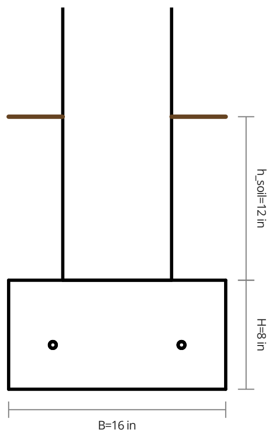
FOUNDATION CALCULATIONS

FOOTING REFERENCE PER PLAN

Client:		**Date:**	Jun 3, 2022
Author:	Harrison Kliegl	**Job #:**	
Project:	Footing Checks	**Subject:**	**16" Strip Footing - Dist Load - 2000 PSF** PASS
References:	ACI 318-14		

Summary

98%	Allowable Gross Soil Bearing Stress	$q_a =$	2000 psf
16%	Factored Moment Capacity	$\phi M_n =$	$900\ \mathrm{lb \cdot ft/ft}$
0%	Factored One-Way Shear Capacity	$\phi V_n =$	2880 plf
	Stability	Status $=$	Footing in Total Compression



Footing Properties

Footing Width	$B =$	1.33 ft
Footing Thickness	$H =$	8 in
Wall Type		Concrete
Wall Width	$b =$	8 in
Concrete Strength	$f'_c =$	2500 psi
Volume of Concrete	$V_c =$	$0.0328\ \mathrm{yd^3/ft}$

Soil Properties

Allowable Soil Gross Bearing Capacity	$q_a =$	2000 psf
Lateral Sliding Coefficient of Friction	$\mu =$	0.3

Bottom Reinforcement

Concrete Cover	cover $=$	3 in
Reinforcement Yield Strength	$f_y =$	60 000 psi

Design Criteria

Design Code for Load Combinations		International Building Code (IBC) 2018
Sliding and Overturning Minimum Factor of Safety	$FS_{\min} =$	1.5

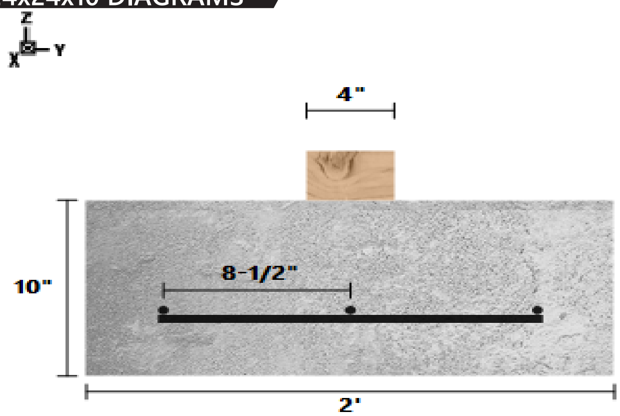
Comments

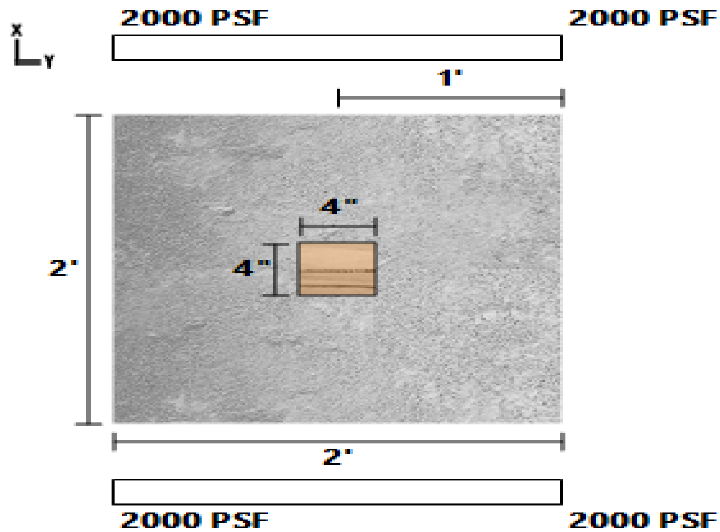
PASS

DATE:	9/29/2021	COMPANY:	L120 Engineering & Design, LLC
VITRUVIUS BUILD:	StruCalc	DESIGNED BY:	Mans Thurfjell
CUSTOMER:		REVIEWED BY:	Mans Thurfjell
PROJ. ADDRESS:	-- --	PROJECT NAME:	2018 foundation 2000psf
LEVEL:	Basement	LOADING:	
MEMBER NAME:	24x24x10	CODE:	2018 International Building Code
MEMBER TYPE:	ISOLATED FOOTING	ACI:	ACI 318-14
MATERIAL:	Concrete		

2 (ft) X 2 (ft) X 10 (in)	Soil Depth TOF: 0 (ft)	(3) #4 Long, (3) #4 Short

24x24x10 DIAGRAMS





MATERIAL PROPERTIES

FOOTING

fc' (psi)	Ec (psi)	Density (lbf/ft^3)	Width (ft)	Length (ft)	Depth (in)	Volume (ft^3)
2500	2880952	145	2	2	10	3.33

CALCULATION VARIABLES

Bo (in)	Φ-X	Φ-Y
42	0	0

COLUMN

Width (in)	Length (in)	Material	Offset (in)
4	4	Wood	0

SOIL

Bearing Strength (lbf/ft^2)	Density (lbf/ft^3)	Cohesion	Friction Angle	Depth (ft)	Rankine Coefficient (Kp)
2000	140	0	30	0	3

REBAR

Bar Size #	# Bars Long	# Bars Short	fy (psi)	Es (psi)
4	3	3	40000	2.9E+07

PASS-FAIL

	PASS/FAIL	MAGNITUDE	STRENGTH	LOAD COMBO
Soil Bearing Pressure (lbf/ft^2)	**PASS (0.0%)**	2000.0	2000.0	D+L
Two-Way Shear (Punching) (lbf)	**PASS (72.6%)**	11200.0	40950.0	1.2D+1.6L+0.5Lr
One-Way Shear X (lbf)	**PASS (86.0%)**	1633.3	11700.0	1.2D+1.6L+0.5Lr
Moment X (lbf-ft)	**PASS (39.2%)**	1944.4	3200.0	1.2D+1.6L+0.5Lr
One-Way Shear Y (lbf)	**PASS (86.0%)**	1633.3	11700.0	1.2D+1.6L+0.5Lr
Moment Y (lbf-ft)	**PASS (39.2%)**	1944.4	3200.0	1.2D+1.6L+0.5Lr
Crushing (psi)	**PASS (49.3%)**	700.0	1381.3	1.2D+1.6L+0.5Lr

LOAD LIST

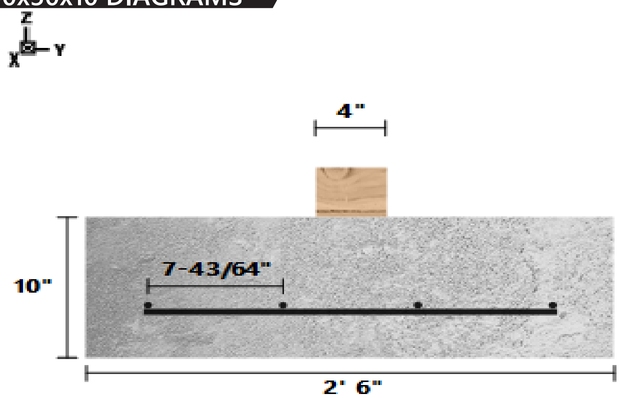
Type	Left Magnitude	Right Magnitude	Load Start (ft)	Load End (ft)	Load Type	Direction
Point (lbf)	4000	-	0	-	Dead	Z
Point (lbf)	4000	-	0	-	Live	Z

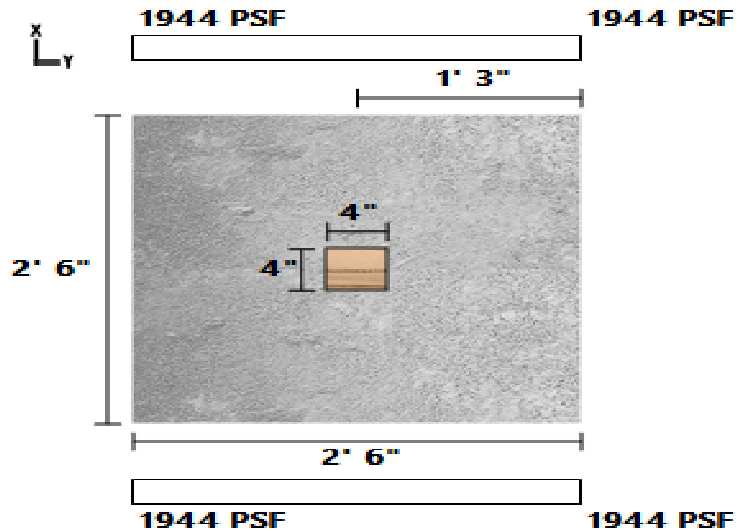
PASS

DATE:	9/29/2021	COMPANY:	L120 Engineering & Design, LLC
VITRUVIUS BUILD:	StruCalc	DESIGNED BY:	Mans Thurfjell
CUSTOMER:		REVIEWED BY:	Mans Thurfjell
PROJ. ADDRESS:	-- --	PROJECT NAME:	2018 foundation 2000psf
LEVEL:	Basement	LOADING:	
MEMBER NAME:	30x30x10	CODE:	2018 International Building Code
MEMBER TYPE:	ISOLATED FOOTING	ACI:	ACI 318-14
MATERIAL:	Concrete		

2.5 (ft) X 2.5 (ft) X 10 (in)	Soil Depth TOF: 0 (ft)	(4) #4 Long, (4) #4 Short

30x30x10 DIAGRAMS





MATERIAL PROPERTIES

FOOTING

fc' (psi)	Ec (psi)	Density (lbf/ft³)	Width (ft)	Length (ft)	Depth (in)	Volume (ft³)
2500	2880952	145	2.5	2.5	10	5.21

CALCULATION VARIABLES

Bo (in)	Φ-X	Φ-Y
42	0	0

COLUMN

Width (in)	Length (in)	Material	Offset (in)
4	4	Wood	0

SOIL

Bearing Strength (lbf/ft²)	Density (lbf/ft³)	Cohesion	Friction Angle	Depth (ft)	Rankine Coefficient (Kp)
2000	140	0	30	0	3

REBAR

Bar Size #	# Bars Long	# Bars Short	fy (psi)	Es (psi)
4	4	4	40000	2.9E+07

PASS-FAIL

	PASS/FAIL	MAGNITUDE	STRENGTH	LOAD COMBO
Soil Bearing Pressure (lbf/ft²)	**PASS (2.8%)**	1944.0	2000.0	D+L
Two-Way Shear (Punching) (lbf)	**PASS (58.4%)**	17040.0	40950.0	1.2D+1.6L+0.5Lr
One-Way Shear X (lbf)	**PASS (74.8%)**	3692.0	14625.0	1.2D+1.6L+0.5Lr
Moment X (lbf-ft)	**PASS (0.0%)**	3999.7	4000.0	1.2D+1.6L+0.5Lr
One-Way Shear Y (lbf)	**PASS (74.8%)**	3692.0	14625.0	1.2D+1.6L+0.5Lr
Moment Y (lbf-ft)	**PASS (0.0%)**	3999.7	4000.0	1.2D+1.6L+0.5Lr
Crushing (psi)	**PASS (22.9%)**	1065.0	1381.3	1.2D+1.6L+0.5Lr

LOAD LIST

Type	Left Magnitude	Right Magnitude	Load Start (ft)	Load End (ft)	Load Type	Direction
Point (lbf)	6000	-	0	-	Dead	Z
Point (lbf)	6150	-	0	-	Live	Z

Created with ClearCalcs.com

LONGITUDE
ONE TWENTY°
ENGINEERING & DESIGN

Client:		**Date:**	Jul 29, 2022
Author:	Harrison Kliegl	**Job #:**	
Project:	2000 PSF Retaining Walls	**Subject:**	**Member Schedule**

	Calculation	Member	Quantity	Comments
78%	**4'-0" Max Retaining Wall**			
77%	**6'-0" Max Retaining Wall**			
79%	**8'-0" Max Retaining Wall**			
79%	**8'-0" Max Retaining Wall with 40 psf Surcharge**			
80%	**9'-6" Max Retaining Wall**			
80%	**10'-6" Max Retaining Wall**			
93%	**13'-0" Max Retaining Wall**			

SEE NEW CALC AT END FOR 13'-0" RETAINING WALL WITH SURCHARGE LOADING

Client:		Date:	Jul 29, 2022
Author:	Harrison Kliegl	Job #:	
Project:	2000 PSF Retaining Walls	Subject:	Project Defaults

Design Criteria

Design Code Full Name $code =$ International Building Code (IBC) 2018

Additionally Include Simplified DL+LL Service Load Combination? Yes

Deflection Span Limits $\Delta_{span} =$

Member Type $type$	Short-Term (L, Lr, S, or W) D_{ST} (L/)	Long-Term (kD+L) D_{LT} (L/)	Simplified DL+LL (D+L) D_{DL+LL} (L/)
Roof	180	120	100
Ceiling	240	180	120
Floor	360	240	180
Wall	240	1	100

Absolute Deflection Limit $\Delta_{lim} = 1$ in

Default Bearing Length $l_b = 3$ in

Building Geometry

Number of Stories $n_{story} = 2$

Roof Slope $\alpha = 6:12$

Default Member Spacings $spacings =$

Rafters s_{raft} (in)	Joists s_{joist} (in)	Wall Studs s_{studs} (in)
16	16	16

Top Floor Height Dimensions $h_{top.floor} =$

Story Height (Floor to Eave) h_{story} (ft)	Headroom (Floor to Ceiling) h_{head} (ft)	Window Height (Floor to Top of Window) h_{window} (ft)
12	10	8

Lower Floors Height Dimensions $h_{lower.floors} =$

Story Height (Floor to Floor) h_{story} (ft)	Headroom (Floor to Ceiling) h_{head} (ft)	Window Height (Floor to Top of Window) h_{window} (ft)
12	10	8

Maximum Roof Beam Depth $d_{max,R} = 24$ in

Floor Beam Depth Limits $d_{min/max} =$

Minimum Beam Depth d_{min} (in)	Maximum Beam Depth d_{max} (in)
0	24

Default Roof Loads

Default Roof Loads $loads_{roof} =$

Superimposed Dead Load w_D (psf)	Roof Live Load w_{Lr} (psf)	Alternative Minimum Live Load P_{Lr2} (lb)	Snow Load w_S (psf)	Ultimate Wind Uplift (C&C) w_{Wu} (psf)	Ultimate Wind Downward (C&C) w_{Wd} (psf)
15	20	0	30	30	30

Default Ceiling Loads

Default Ceiling Loads $loads_{ceiling} =$

Superimposed Dead Load w_D (psf)	Live Load w_L (psf)	Alternative Minimum Live Load P_{L2} (lb)
5	20	0

Default Floor Loads

Default Floor Loads $loads_{floor} =$

Superimposed Dead Load w_D (psf)	Live Load w_L (psf)	Alternative Minimum Live Load P_{L2} (lb)
10	40	0

Default Wall & Window Loads

Default Total Wall & Window Dead Loads $w_{D,wall+window} =$

Total Weight of Interior Wall $w_{D,IW}$ (psf)	Total Weight of Exterior Wall $w_{D,EW}$ (psf)	Total Weight of Window $w_{D,window}$ (psf)
5	30	1.5

Default Ultimate Wall & Window Wind Loads $w_{W,wall+window} =$

Ultimate Inward Wind Load (C&C) w_{Wd} (psf)	Ultimate Outward Wind Load (C&C) w_{Wu} (psf)
30	30

Default Railing Properties

Railing Height $h_{railing} = 4$ ft

Railing Total Weight $w_{D,railing} = 20$ plf

Comments

LONGITUDE ONE TWENTY° ENGINEERING & DESIGN

Client:		**Date:**	Jul 29, 2022
Author:	Harrison Kliegl	**Job #:**	
Project:	2000 PSF Retaining Walls	**Subject:**	**4'-0" Max Retaining Wall** PASS
References:	IBC 2018, ASCE 7-16		

Stability Summary

	Lateral Force Transmitted to Footing Restraint	$F_{restraint} =$	0 kip/ft
78%	Overturning Factor of Safety	$FS_{overturn} =$	1.91
	Maximum Bearing Pressure	$q_{max} =$	1310 psf
66%	Soil Allowable Bearing Capacity	$q_a =$	2000 psf

Stem Summary

26%	Moment Capacity of Wall Stem	$\phi M_{n,stem} =$	3.66 kip · ft/ft
11%	Shear Capacity of Wall Stem	$\phi V_{n,stem} =$	5.63 kip/ft

Heel Summary

4%	Moment Capacity of Heel	$\phi M_{n,heel} =$	9.85 kip · ft/ft
11%	Shear Capacity of Wall Base	$\phi V_{n,heel} =$	7.88 kip/ft

Toe Summary

7%	Moment Capacity of Toe	$\phi M_{n,toe} =$	9.85 kip · ft/ft
5%	Shear Capacity of Toe	$\phi V_{n,toe} =$	7.88 kip/ft

Key Dimensions

Wall Height	$H =$	6 ft
Thickness of Wall Stem at Base	$t_{stem} =$	8 in
Thickness of Wall Stem at Top	$t_{stem,top} =$	8 in
Length of Heel	$L_{heel} =$	0.92 ft
Thickness of Footing	$t_{footing} =$	12 in

Surcharge

Dead Load Surcharge is Directly Above Heel?		No

Soil Properties

Height of Backfill	$h_{bf} =$	5 ft
Depth of Soil Cover to Bottom of Footing	$h_{cov} =$	1.5 ft
Lateral Pressure Method		Equivalent Fluid Pressure - Custom Values
Soil Unit Weight	$\gamma_{input} =$	125 pcf

Base Soil Properties

Source of Soil Properties		Same as Backfill

Concrete Properties

Concrete Strength	$f'_c =$	2500 psi
Reinforcement Yield Strength	$f_y =$	$60\,000$ psi
Volume of Concrete	$V_c =$	0.216 yd³/ft

Stem Reinforcement

Stem Concrete Cover	$c_{stem} =$	1.5 in

Heel Reinforcement (Top Bars)

Heel Concrete Cover	$c_{heel} =$	3 in

Toe Reinforcement (Bottom Bars)

Include Toe Reinforcement?		Yes
Toe Concrete Cover	$c_{toe} =$	3 in

Heel Reinforcement Depth & Spacing

Area of Heel Reinforcement	$A_{s,heel} =$	0.259 in²/ft

Toe Reinforcement Depth & Spacing

Area of Toe Reinforcement	$A_{s,toe} =$	0.259 in²/ft

Design Criteria

Design Code for Load Combinations	$code =$	International Building Code (IBC) 2018
Retaining Wall Movement Condition		Active Case (Ka)
Footing Restrained Against Sliding?		Yes
Consider Resisting Soil Pressures for Stability Checks?		No
Consider Soil Above Toe for Stability Checks?		No
Consider Resisting Pressure from Soil Above Toe for Strength Design?		No

Soil Unit Weight

Soil Unit Weight	$\gamma =$	125 pcf

Client:		**Date:**	Jul 29, 2022
Author:	Harrison Kliegl	**Job #:**	
Project:	2000 PSF Retaining Walls	**Subject:**	**6'-0" Max Retaining Wall** PASS
References:	IBC 2018, ASCE 7-16		

Stability Summary

	Lateral Force Transmitted to Footing Restraint	$F_{restraint} =$	0 kip/ft
62%	Overturning Factor of Safety	$FS_{overturn} =$	2.41
	Maximum Bearing Pressure	$q_{max} =$	1120 psf
56%	Soil Allowable Bearing Capacity	$q_a =$	2000 psf

Stem Summary

77%	Moment Capacity of Wall Stem	$\phi M_{n,stem} =$	3.66 kip · ft/ft
23%	Shear Capacity of Wall Stem	$\phi V_{n,stem} =$	5.63 kip/ft

Heel Summary

14%	Moment Capacity of Heel	$\phi M_{n,heel} =$	9.85 kip · ft/ft
24%	Shear Capacity of Wall Base	$\phi V_{n,heel} =$	7.88 kip/ft

Toe Summary

24%	Moment Capacity of Toe	$\phi M_{n,toe} =$	9.85 kip · ft/ft
20%	Shear Capacity of Toe	$\phi V_{n,toe} =$	7.88 kip/ft

Key Dimensions

Wall Height	$H =$	8 ft
Thickness of Wall Stem at Base	$t_{stem} =$	8 in
Thickness of Wall Stem at Top	$t_{stem,top} =$	8 in
Length of Heel	$L_{heel} =$	1.5 ft
Thickness of Footing	$t_{footing} =$	12 in

Surcharge

Dead Load Surcharge is Directly Above Heel?		No

Soil Properties

Height of Backfill	$h_{bf} =$	7 ft
Depth of Soil Cover to Bottom of Footing	$h_{cov} =$	1.5 ft
Lateral Pressure Method		Equivalent Fluid Pressure - Custom Values
Soil Unit Weight	$\gamma_{input} =$	125 pcf

Base Soil Properties

Source of Soil Properties		Same as Backfill

Concrete Properties

Concrete Strength	$f'_c =$	2500 psi
Reinforcement Yield Strength	$f_y =$	$60\,000$ psi
Volume of Concrete	$V_c =$	0.321 yd³/ft

Stem Reinforcement

Stem Concrete Cover	$c_{stem} =$	1.5 in

Heel Reinforcement (Top Bars)

Heel Concrete Cover	$c_{heel} =$	3 in

Toe Reinforcement (Bottom Bars)

Include Toe Reinforcement?		Yes
Toe Concrete Cover	$c_{toe} =$	3 in

Heel Reinforcement Depth & Spacing

Area of Heel Reinforcement	$A_{s,heel} =$	0.259 in²/ft

Toe Reinforcement Depth & Spacing

Area of Toe Reinforcement	$A_{s,toe} =$	0.259 in²/ft

Design Criteria

Design Code for Load Combinations	$code =$	International Building Code (IBC) 2018
Retaining Wall Movement Condition		Active Case (Ka)
Footing Restrained Against Sliding?		Yes
Consider Resisting Soil Pressures for Stability Checks?		No
Consider Soil Above Toe for Stability Checks?		No
Consider Resisting Pressure from Soil Above Toe for Strength Design?		No

Soil Unit Weight

Soil Unit Weight	$\gamma =$	125 pcf

Client:		**Date:**	Jul 29, 2022
Author:	Harrison Kliegl	**Job #:**	
Project:	2000 PSF Retaining Walls	**Subject:**	8'-0" Max Retaining Wall PASS
References:	IBC 2018, ASCE 7-16		

Stability Summary

	Lateral Force Transmitted to Footing Restraint	$F_{restraint} =$	0.136 kip/ft
79%	Overturning Factor of Safety	$FS_{overturn} =$	1.91
	Maximum Bearing Pressure	$q_{max} =$	1420 psf
71%	Soil Allowable Bearing Capacity	$q_a =$	2000 psf

Stem Summary

70%	Moment Capacity of Wall Stem	$\phi M_{n,stem} =$	7.12 kip · ft/ft
33%	Shear Capacity of Wall Stem	$\phi V_{n,stem} =$	5.63 kip/ft

Heel Summary

11%	Moment Capacity of Heel	$\phi M_{n,heel} =$	10.1 kip · ft/ft
22%	Shear Capacity of Wall Base	$\phi V_{n,heel} =$	7.88 kip/ft

Toe Summary

46%	Moment Capacity of Toe	$\phi M_{n,toe} =$	10.1 kip · ft/ft
35%	Shear Capacity of Toe	$\phi V_{n,toe} =$	7.88 kip/ft

Key Dimensions

Wall Height	$H =$	10 ft
Thickness of Wall Stem at Base	$t_{stem} =$	8 in
Thickness of Wall Stem at Top	$t_{stem,top} =$	8 in
Length of Heel	$L_{heel} =$	1.25 ft
Thickness of Footing	$t_{footing} =$	12 in

Surcharge

Dead Load Surcharge is Directly Above Heel?		No

Soil Properties

Height of Backfill	$h_{bf} =$	9 ft
Depth of Soil Cover to Bottom of Footing	$h_{cov} =$	1.5 ft
Lateral Pressure Method		Equivalent Fluid Pressure - Custom Values
Soil Unit Weight	$\gamma_{input} =$	125 pcf

Base Soil Properties

Source of Soil Properties		Same as Backfill

Concrete Properties

Concrete Strength	$f'_c =$	2500 psi
Reinforcement Yield Strength	$f_y =$	$60\,000$ psi
Volume of Concrete	$V_c =$	0.38 yd³/ft

Stem Reinforcement

Stem Concrete Cover	$c_{stem} =$	1.5 in

Heel Reinforcement (Top Bars)

Heel Concrete Cover	$c_{heel} =$	3 in

Toe Reinforcement (Bottom Bars)

Include Toe Reinforcement?		Yes
Toe Concrete Cover	$c_{toe} =$	3 in

Heel Reinforcement Depth & Spacing

Area of Heel Reinforcement	$A_{s,heel} =$	0.267 in²/ft

Toe Reinforcement Depth & Spacing

Area of Toe Reinforcement	$A_{s,toe} =$	0.267 in²/ft

Design Criteria

Design Code for Load Combinations	$code =$	International Building Code (IBC) 2018
Retaining Wall Movement Condition		Active Case (Ka)
Footing Restrained Against Sliding?		Yes
Consider Resisting Soil Pressures for Stability Checks?		No
Consider Soil Above Toe for Stability Checks?		No
Consider Resisting Pressure from Soil Above Toe for Strength Design?		No

Soil Unit Weight

Soil Unit Weight	$\gamma =$	125 pcf

Client:		**Date:**	Jul 29, 2022
Author:	Harrison Kliegl	**Job #:**	
Project:	2000 PSF Retaining Walls	**Subject:**	**8'-0" Max Retaining Wall with 40 psf Surcharge** PASS
References:	IBC 2018, ASCE 7-16		

Stability Summary

	Lateral Force Transmitted to Footing Restraint	$F_{restraint} =$	0.182 kip/ft
79%	Overturning Factor of Safety	$FS_{overturn} =$	1.9
	Maximum Bearing Pressure	$q_{max} =$	1390 psf
69%	Soil Allowable Bearing Capacity	$q_a =$	2000 psf

Stem Summary

75%	Moment Capacity of Wall Stem	$\phi M_{n,stem} =$	7.12 kip · ft/ft
34%	Shear Capacity of Wall Stem	$\phi V_{n,stem} =$	5.63 kip/ft

Heel Summary

11%	Moment Capacity of Heel	$\phi M_{n,heel} =$	10.1 kip · ft/ft
23%	Shear Capacity of Wall Base	$\phi V_{n,heel} =$	7.88 kip/ft

Toe Summary

51%	Moment Capacity of Toe	$\phi M_{n,toe} =$	10.1 kip · ft/ft
37%	Shear Capacity of Toe	$\phi V_{n,toe} =$	7.88 kip/ft

Key Dimensions

Wall Height	$H =$	10 ft
Thickness of Wall Stem at Base	$t_{stem} =$	8 in
Thickness of Wall Stem at Top	$t_{stem,top} =$	8 in
Length of Heel	$L_{heel} =$	1.25 ft
Thickness of Footing	$t_{footing} =$	12 in

Surcharge

Dead Load Surcharge is Directly Above Heel?		No

Soil Properties

Height of Backfill	$h_{bf} =$	9 ft
Depth of Soil Cover to Bottom of Footing	$h_{cov} =$	1.5 ft
Lateral Pressure Method		Equivalent Fluid Pressure - Custom Values
Soil Unit Weight	$\gamma_{input} =$	125 pcf

Base Soil Properties

Source of Soil Properties		Same as Backfill

Concrete Properties

Concrete Strength	$f'_c =$	2500 psi
Reinforcement Yield Strength	$f_y =$	$60\,000$ psi
Volume of Concrete	$V_c =$	0.386 yd³/ft

Stem Reinforcement

Stem Concrete Cover	$c_{stem} =$	1.5 in

Heel Reinforcement (Top Bars)

Heel Concrete Cover	$c_{heel} =$	3 in

Toe Reinforcement (Bottom Bars)

Include Toe Reinforcement?		Yes
Toe Concrete Cover	$c_{toe} =$	3 in

Heel Reinforcement Depth & Spacing

Area of Heel Reinforcement	$A_{s,heel} =$	0.267 in²/ft

Toe Reinforcement Depth & Spacing

Area of Toe Reinforcement	$A_{s,toe} =$	0.267 in²/ft

Design Criteria

Design Code for Load Combinations	$code =$	International Building Code (IBC) 2018
Retaining Wall Movement Condition		Active Case (Ka)
Footing Restrained Against Sliding?		Yes
Consider Resisting Soil Pressures for Stability Checks?		No
Consider Soil Above Toe for Stability Checks?		No
Consider Resisting Pressure from Soil Above Toe for Strength Design?		No

Soil Unit Weight

Soil Unit Weight	$\gamma =$	125 pcf

LONGITUDE ONE TWENTY° ENGINEERING & DESIGN

Client:		**Date:**	Jul 29, 2022
Author:	Harrison Kliegl	**Job #:**	
Project:	2000 PSF Retaining Walls	**Subject:**	**9'-6" Max Retaining Wall** PASS
References:	IBC 2018, ASCE 7-16		

Stability Summary

	Lateral Force Transmitted to Footing Restraint	$F_{restraint} =$	0.385 kip/ft
80%	Overturning Factor of Safety	$FS_{overturn} =$	1.88
	Maximum Bearing Pressure	$q_{max} =$	1350 psf
68%	Soil Allowable Bearing Capacity	$q_a =$	2000 psf

Stem Summary

80%	Moment Capacity of Wall Stem	$\phi M_{n,stem} =$	10.4 kip · ft/ft
46%	Shear Capacity of Wall Stem	$\phi V_{n,stem} =$	5.63 kip/ft

Heel Summary

13%	Moment Capacity of Heel	$\phi M_{n,heel} =$	10.1 kip · ft/ft
26%	Shear Capacity of Wall Base	$\phi V_{n,heel} =$	7.88 kip/ft

Toe Summary

58%	Moment Capacity of Toe	$\phi M_{n,toe} =$	14.9 kip · ft/ft
49%	Shear Capacity of Toe	$\phi V_{n,toe} =$	7.88 kip/ft

Key Dimensions

Wall Height	$H =$	11.5 ft
Thickness of Wall Stem at Base	$t_{stem} =$	8 in
Thickness of Wall Stem at Top	$t_{stem,top} =$	8 in
Length of Heel	$L_{heel} =$	1.25 ft
Thickness of Footing	$t_{footing} =$	12 in

Surcharge

Dead Load Surcharge is Directly Above Heel?		No

Soil Properties

Height of Backfill	$h_{bf} =$	10.5 ft
Depth of Soil Cover to Bottom of Footing	$h_{cov} =$	1.5 ft
Lateral Pressure Method		Equivalent Fluid Pressure - Custom Values
Soil Unit Weight	$\gamma_{input} =$	125 pcf

Base Soil Properties

Source of Soil Properties		Same as Backfill

Concrete Properties

Concrete Strength	$f'_c =$	2500 psi
Reinforcement Yield Strength	$f_y =$	$60\,000$ psi
Volume of Concrete	$V_c =$	0.454 yd³/ft

Stem Reinforcement

Stem Concrete Cover	$c_{stem} =$	1.5 in

Heel Reinforcement (Top Bars)

Heel Concrete Cover	$c_{heel} =$	3 in

Toe Reinforcement (Bottom Bars)

Include Toe Reinforcement?		Yes
Toe Concrete Cover	$c_{toe} =$	3 in

Heel Reinforcement Depth & Spacing

Area of Heel Reinforcement	$A_{s,heel} =$	0.267 in²/ft

Toe Reinforcement Depth & Spacing

Area of Toe Reinforcement	$A_{s,toe} =$	0.4 in²/ft

Design Criteria

Design Code for Load Combinations	$code =$	International Building Code (IBC) 2018
Retaining Wall Movement Condition		Active Case (Ka)
Footing Restrained Against Sliding?		Yes
Consider Resisting Soil Pressures for Stability Checks?		No
Consider Soil Above Toe for Stability Checks?		No
Consider Resisting Pressure from Soil Above Toe for Strength Design?		No

Soil Unit Weight

Soil Unit Weight	$\gamma =$	125 pcf

LONGITUDE ONE TWENTY° ENGINEERING & DESIGN

Client:		**Date:**	Jul 29, 2022
Author:	Harrison Kliegl	**Job #:**	
Project:	2000 PSF Retaining Walls	**Subject:**	**10'-6" Max Retaining Wall** PASS
References:	IBC 2018, ASCE 7-16		

Stability Summary

	Lateral Force Transmitted to Footing Restraint	$F_{restraint} =$	0.587 kip/ft
80%	Overturning Factor of Safety	$FS_{overturn} =$	1.87
	Maximum Bearing Pressure	$q_{max} =$	1300 psf
65%	Soil Allowable Bearing Capacity	$q_a =$	2000 psf

Stem Summary

75%	Moment Capacity of Wall Stem	$\phi M_{n,stem} =$	15 kip · ft/ft
56%	Shear Capacity of Wall Stem	$\phi V_{n,stem} =$	5.63 kip/ft

Heel Summary

14%	Moment Capacity of Heel	$\phi M_{n,heel} =$	10.1 kip · ft/ft
28%	Shear Capacity of Wall Base	$\phi V_{n,heel} =$	7.88 kip/ft

Toe Summary

57%	Moment Capacity of Toe	$\phi M_{n,toe} =$	21.7 kip · ft/ft
58%	Shear Capacity of Toe	$\phi V_{n,toe} =$	7.88 kip/ft

Key Dimensions

Wall Height	$H =$	12.5 ft
Thickness of Wall Stem at Base	$t_{stem} =$	8 in
Thickness of Wall Stem at Top	$t_{stem,top} =$	8 in
Length of Heel	$L_{heel} =$	1.25 ft
Thickness of Footing	$t_{footing} =$	12 in

Surcharge

Dead Load Surcharge is Directly Above Heel?		No

Soil Properties

Height of Backfill	$h_{bf} =$	11.5 ft
Depth of Soil Cover to Bottom of Footing	$h_{cov} =$	1.5 ft
Lateral Pressure Method		Equivalent Fluid Pressure - Custom Values
Soil Unit Weight	$\gamma_{input} =$	125 pcf

Base Soil Properties

Source of Soil Properties		Same as Backfill

Concrete Properties

Concrete Strength	$f'_c =$	2500 psi
Reinforcement Yield Strength	$f_y =$	$60\,000$ psi
Volume of Concrete	$V_c =$	0.506 yd³/ft

Stem Reinforcement

Stem Concrete Cover	$c_{stem} =$	1.5 in

Heel Reinforcement (Top Bars)

Heel Concrete Cover	$c_{heel} =$	3 in

Toe Reinforcement (Bottom Bars)

Include Toe Reinforcement?		Yes
Toe Concrete Cover	$c_{toe} =$	3 in

Heel Reinforcement Depth & Spacing

Area of Heel Reinforcement	$A_{s,heel} =$	0.267 in²/ft

Toe Reinforcement Depth & Spacing

Area of Toe Reinforcement	$A_{s,toe} =$	0.6 in²/ft

Design Criteria

Design Code for Load Combinations	$code =$	International Building Code (IBC) 2018
Retaining Wall Movement Condition		Active Case (Ka)
Footing Restrained Against Sliding?		Yes
Consider Resisting Soil Pressures for Stability Checks?		No
Consider Soil Above Toe for Stability Checks?		No
Consider Resisting Pressure from Soil Above Toe for Strength Design?		No

Soil Unit Weight

Soil Unit Weight	$\gamma =$	125 pcf

LONGITUDE ONE TWENTY° ENGINEERING & DESIGN

Client:		**Date:**	Jul 29, 2022
Author:	Harrison Kliegl	**Job #:**	
Project:	2000 PSF Retaining Walls	**Subject:**	**13'-0" Max Retaining Wall** PASS
References:	IBC 2018, ASCE 7-16		

Stability Summary

	Lateral Force Transmitted to Footing Restraint	$F_{restraint} =$	0.891 kip/ft
72%	Overturning Factor of Safety	$FS_{overturn} =$	2.09
	Maximum Bearing Pressure	$q_{max} =$	1210 psf
61%	Soil Allowable Bearing Capacity	$q_a =$	2000 psf

Stem Summary

76%	Moment Capacity of Wall Stem	$\phi M_{n,stem} =$	26.8 kip · ft/ft
64%	Shear Capacity of Wall Stem	$\phi V_{n,stem} =$	7.37 kip/ft

Heel Summary

10%	Moment Capacity of Heel	$\phi M_{n,heel} =$	24.3 kip · ft/ft
33%	Shear Capacity of Wall Base	$\phi V_{n,heel} =$	9.68 kip/ft

Toe Summary

93%	Moment Capacity of Toe	$\phi M_{n,toe} =$	24.3 kip · ft/ft
65%	Shear Capacity of Toe	$\phi V_{n,toe} =$	9.68 kip/ft

Key Dimensions

Wall Height	$H =$	15 ft
Thickness of Wall Stem at Base	$t_{stem} =$	10 in
Thickness of Wall Stem at Top	$t_{stem,top} =$	10 in
Length of Heel	$L_{heel} =$	1.5 ft
Thickness of Footing	$t_{footing} =$	14 in

Surcharge

Dead Load Surcharge is Directly Above Heel?		No

Soil Properties

Height of Backfill	$h_{bf} =$	14 ft
Depth of Soil Cover to Bottom of Footing	$h_{cov} =$	1.5 ft
Lateral Pressure Method		Equivalent Fluid Pressure - Custom Values
Soil Unit Weight	$\gamma_{input} =$	125 pcf

Base Soil Properties

Source of Soil Properties		Same as Backfill

Concrete Properties

Concrete Strength	$f'_c =$	2500 psi
Reinforcement Yield Strength	$f_y =$	$60\,000$ psi
Volume of Concrete	$V_c =$	0.773 yd³/ft

Stem Reinforcement

Stem Concrete Cover	$c_{stem} =$	1.5 in

Heel Reinforcement (Top Bars)

Heel Concrete Cover	$c_{heel} =$	3 in

Toe Reinforcement (Bottom Bars)

Include Toe Reinforcement?		Yes
Toe Concrete Cover	$c_{toe} =$	3 in

Heel Reinforcement Depth & Spacing

Area of Heel Reinforcement	$A_{s,heel} =$	0.533 in²/ft

Toe Reinforcement Depth & Spacing

Area of Toe Reinforcement	$A_{s,toe} =$	0.533 in²/ft

Design Criteria

Design Code for Load Combinations	$code =$	International Building Code (IBC) 2018
Retaining Wall Movement Condition		Active Case (Ka)
Footing Restrained Against Sliding?		Yes
Consider Resisting Soil Pressures for Stability Checks?		No
Consider Soil Above Toe for Stability Checks?		No
Consider Resisting Pressure from Soil Above Toe for Strength Design?		No

Soil Unit Weight

Soil Unit Weight	$\gamma =$	125 pcf

Client:		Date:	Jul 29, 2022
Author:	Harrison Kliegl	Job #:	
Project:	2000 PSF Retaining Walls	Subject:	13'-0" Max Retaining Wall PASS
References:	IBC 2018, ASCE 7-16		W/SURCHARGE LOADING

Stability Summary

	Total Sliding Forces	$F_{sliding} = 3.78$ kip/ft	
	Total Resistance to Sliding	$F_{resist} = 2.63$ kip/ft	IBC 2018, Cl 1806.3
	Lateral Force Transmitted to Footing Restraint	$F_{restraint} = 1.15$ kip/ft	
	Total Overturning Moment	$M_{overturn} = 18.5$ kip · ft/ft	
	Total Restoring Moment	$M_{restore} = 34.8$ kip · ft/ft	
80%	Overturning Factor of Safety	$FS_{overturn} = 1.88$	
	Maximum Bearing Pressure	$q_{max} = 1390$ psf	
70%	Soil Allowable Bearing Capacity	$q_a = 2000$ psf	

Stem Summary

	Moment Demand of Wall Stem	$M_{u,stem} = 23$ kip · ft/ft	
86%	Moment Capacity of Wall Stem	$\phi M_{n,stem} = 26.8$ kip · ft/ft	ACI 318-14, Cl 22.3
	Shear Demand of Wall Stem	$V_{u,stem} = 5.13$ kip/ft	ACI 318-14, Cl 9.4.3
70%	Shear Capacity of Wall Stem	$\phi V_{n,stem} = 7.37$ kip/ft	ACI 318-14, Cl 22.5

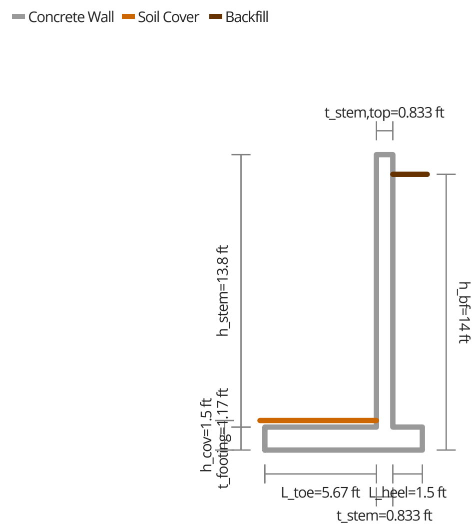
Heel Summary

	Moment Demand of Heel	$M_{u,heel} = 2.54$ kip · ft/ft	ACI 318-14, Cl 13.2.7.1
10%	Moment Capacity of Heel	$\phi M_{n,heel} = 24.3$ kip · ft/ft	ACI 318-14, Cl 22.3
	Shear Demand of Heel	$V_{u,heel} = 3.39$ kip/ft	ACI 318-14, Cl 9.4.3
35%	Shear Capacity of Wall Base	$\phi V_{n,heel} = 9.68$ kip/ft	ACI 318-14, Cl 22.5

Toe Summary

	Moment Demand of Toe	$M_{u,toe} = 25.1$ kip · ft/ft	ACI 318-14, Cl 13.2.7.1
90%	Moment Capacity of Toe	$\phi M_{n,toe} = 27.8$ kip · ft/ft	ACI 318-14, Cl 22.3
	Shear Demand of Toe	$V_{u,toe} = 6.88$ kip/ft	ACI 318-14, Cl 9.4.3
72%	Shear Capacity of Toe	$\phi V_{n,toe} = 9.62$ kip/ft	ACI 318-14, Cl 22.5

Key Dimensions



Wall Height	$H =$	$15\ \mathrm{ft}$
Thickness of Wall Stem at Base	$t_{stem} =$	$10\ \mathrm{in}$
Thickness of Wall Stem at Top	$t_{stem,top} =$	$10\ \mathrm{in}$
Length of Toe	$L_{toe} =$	$5.67\ \mathrm{ft}$
Length of Heel	$L_{heel} =$	$1.5\ \mathrm{ft}$
Thickness of Footing	$t_{footing} =$	$14\ \mathrm{in}$

Surcharge

Dead Load Surchage	$q_D =$	$50\ \mathrm{psf}$
Dead Load Surcharge is Directly Above Heel?		Yes
Live Load Surchage	$q_L =$	$40\ \mathrm{psf}$

Soil Properties

Height of Backfill	$h_{bf} =$	$14\ \mathrm{ft}$	
Depth of Soil Cover to Bottom of Footing	$h_{cov} =$	$1.5\ \mathrm{ft}$	
Lateral Pressure Method		Equivalent Fluid Pressure - Custom Values	IBC 2018
Soil Unit Weight	$\gamma_{input} =$	$125\ \mathrm{pcf}$	
Active Soil Pressure	$p_{a,input} =$	$35\ \mathrm{psf/ft}$	
Allowable Passive / Lateral Bearing Pressure	$p_{p,input} =$	$300\ \mathrm{psf/ft}$	

Base Soil Properties

Source of Soil Properties		Same as Backfill	
Allowable Bearing Capacity of Base Soil	$q_{a,input} =$	$2000\ \mathrm{psf}$	
Soil-Footing Friction Coefficient	$\mu_{input} =$	0.45	
Base Soil Cohesion	$c_{base,input} =$	$0\ \mathrm{psf}$	
Maximum Allowable Cohesion as Proportion of Dead Load	$c_{max} =$	0.5	
Soil-Footing Friction Coefficient	$\mu =$	0.45	If presumptive values are used: IBC 2018 Table 1806.2
Base Soil Cohesion	$c_{base} =$	$0\ \mathrm{psf}$	If presumptive values are used: IBC 2018 Table 1806.2

Water Table

Height of Water Table	$h_{water} =$	0 ft	
Unit Weight of Water	$\gamma_{water} =$	62.4 pcf	

Concrete Properties

Concrete Strength	$f'_c =$	2500 psi	ACI 318-14 Table 19.2.1.1
Concrete Weight Classification		Normalweight	ACI 318-14, Cl 19.2.4.2
Reinforcement Yield Strength	$f_y =$	$60\,000$ psi	ACI 318-14 Table 20.2.2.4a

Stem Reinforcement

Stem Concrete Cover	$c_{stem} =$	1.5 in	ACI 318-14 Table 20.6.1.3.1
Main Reinforcement Size		#5	
Main Reinforcement Spacing	$s_{stem} =$	4.5 in	ACI 318-14, Cl 25.2.1 (minimum spacing) and Cl 7.7.2.3 (maximum spacing)

Heel Reinforcement (Top Bars)

Heel Concrete Cover	$c_{heel} =$	3 in	ACI 318-14 Table 20.6.1.3.1
Heel Reinforcement Size		#4	
Heel Reinforcement Spacing	$s_{heel} =$	4.5 in	ACI 318-14, Cl 25.2.1 (minimum spacing) and Cl 7.7.2.3 (maximum spacing)

Toe Reinforcement (Bottom Bars)

Include Toe Reinforcement?		Yes	
Toe Concrete Cover	$c_{toe} =$	3 in	ACI 318-14 Table 20.6.1.3.1
Toe Reinforcement Size		#5	
Toe Reinforcement Spacing	$s_{toe} =$	6 in	ACI 318-14, Cl 25.2.1 (minimum spacing) and Cl 7.7.2.3 (maximum spacing)

Increase in toe reinforcement

Shrinkage / Temperature Reinforcement

Shrinkage/Temperature Reinforcement Size		#4	
Stem Shrinkage/Temperature Bar Spacing	$s_{\ell,stem} =$	10 in	ACI 318-14, Cl 7.7.2.3
Footing Shrinkage/Temperature Bar Spacing	$s_{\ell,footing} =$	6 in	ACI 318-14, Cl 7.7.2.3

Stem Reinforcement Depth & Spacing

Depth to Stem Reinforcement	$d_{stem} =$	8.19 in	
Area of Vertical Tension Reinforcement	$A_{s,stem} =$	$0.827\ \mathrm{in^2/ft}$	

Heel Reinforcement Depth & Spacing

Heel Depth to Reinforcement	$d_{heel} =$	10.7 in	ACI 318-14, Cl 13.3.1.2
Area of Heel Reinforcement	$A_{s,heel} =$	$0.533\ \mathrm{in^2/ft}$	

Toe Reinforcement Depth & Spacing

Toe Depth to Reinforcement	$d_{toe} =$	10.7 in	ACI 318-14, Cl 13.3.1.2
Area of Toe Reinforcement	$A_{s,toe} =$	$0.62\ \mathrm{in^2/ft}$	

Design Criteria

Design Code for Load Combinations	$code =$	International Building Code (IBC) 2018	
Retaining Wall Movement Condition		Active Case (Ka)	
Footing Restrained Against Sliding?		Yes	
Consider Resisting Soil Pressures for Stability Checks?		No	

Consider Soil Above Toe for Stability Checks?		No	
Consider Resisting Pressure from Soil Above Toe for Strength Design?		No	
Sliding Minimum Factor of Safety	$FS_{\min,sliding}$	1.5	IBC 2018, Cl 1807.2.3
Overturning Minimum Factor of Safety	$FS_{\min,ovt}$	$= 1.5$	IBC 2018, Cl 1807.2.3

Unfactored Vertical and Horizontal Loads for Stability Design

Backfill Soil Width	$w_s =$	$1\ \mathrm{ft},\ 6\ \mathrm{in}$	
Weight of Wall Stem	$W_{stem} =$	$1.73\ \mathrm{kip/ft}$	
Weight of Heel	$W_{heel} =$	$0.263\ \mathrm{kip/ft}$	
Weight of Toe	$W_{toe} =$	$0.992\ \mathrm{kip/ft}$	
Weight of Backfill Soil	$W_{bf} =$	$2.41\ \mathrm{kip/ft}$	
Lateral Force Due to Dead Load Surcharge	$P_D =$	$0.196\ \mathrm{kip/ft}$	
Lateral Force Due to Live Load Surcharge	$P_L =$	$0.157\ \mathrm{kip/ft}$	
Lateral Force Due to Backfill	$P_{bf} =$	$3.43\ \mathrm{kip/ft}$	
Passive Force of Soil on Footing	$P_{p,footing} =$	$0.321\ \mathrm{kip/ft}$	IBC 2018, Cl 1806.3.3
Passive Force of Soil Above Toe on Stem	$P_{p,stem} =$	$0.0167\ \mathrm{kip/ft}$	
Active Force of Soil on Footing	$P_{a,footing} =$	$0.0374\ \mathrm{kip/ft}$	IBC 2018, Cl 1806.3.3
Active Force of Soil Above Toe on Stem	$P_{a,stem} =$	$0.00194\ \mathrm{kip/ft}$	

Tabulated Soil Loads

Vertical Loads (Resisting) $\mathbf{W} =$

Element	Unfactored Forces $W_{unfactored}$ (kip/ft)	Load Factor ξ	Weight W (kip/ft)	Moment Arm y (ft)	Restoring Moment $M_{restore}$ (kip · ft/ft)
Dead Load Surcharge	0.075	1	0.075	7.25	0.544
Wall Stem	1.73	1	1.73	6.09	10.5
Wall Footing	1.4	1	1.4	4	5.6
Soil Cover Above Toe	0.236	1	0.236	2.84	0.67
Backfill Above Water Table	2.41	1	2.41	7.25	17.5

Live Load Surcharge Vertical Loads (Soil Bearing) $\mathbf{W}_L =$

Element	Unfactored Forces $W_{unfactored}$ (kip/ft)	Load Factor ξ	Weight W (kip/ft)	Moment Arm y (ft)	Restoring Moment $M_{restore}$ (kip · ft/ft)
Live Load Surcharge	0.06	1	0.06	7.25	0.435

Lateral Loads $\mathbf{H} =$ IBC 2018, Cl 1605.2

Element	Unfactored Forces $H_{unfactored}$ (kip/ft)	Load Factor ξ	Lateral Load H (kip/ft)	Moment Arm y (ft)	Overturning Moment $M_{overturn}$ (kip · ft/ft)
Dead Load Surcharge	0.196	1	0.196	7	1.37
Live Load Surcharge	0.157	1	0.157	7	1.1
Backfill	3.43	1	3.43	4.67	16

Passive Soil Loads (Resisting Sliding) $\mathbf{Fp} =$

Element	Unfactored Passive Forces $F_{p,unfactored}$ (kip/ft)	Load Factor ξ	Passive Lateral Resisting Load F_p (kip/ft)
Soil Against Toe Face	0.321	0.6	0.193

Active Soil Loads (Resisting Overturning) $\mathbf{Fa} =$

Element	Unfactored Active Forces $F_{a,unfactored}$ (kip/ft)	Load Factor ξ	Active Lateral Resisting Load F_a (kip/ft)	Moment Arm y (ft)	Active Resisting Moment M_a (kip · ft/ft)
Soil Against Toe Face	0.0374	0.6	0.0225	0.46	0.0103

Stability Analysis - Sliding Loads

Total Horizontal Loads (Sliding) $H_{total} = 3.78\ \mathrm{kip/ft}$

Total Vertical Loads (Resisting) $W_{total} = 5.85\ \mathrm{kip/ft}$

Total Passive Loads (Resisting) $F_{p,total} = 0.193\ \mathrm{kip/ft}$

Stability Analysis - Overturning Loads

Total Overturning Moment $M_{over} = 18.5\ \mathrm{kip \cdot ft/ft}$

Total Restoring Moment from Gravity $M_{res,grav} = 34.8\ \mathrm{kip \cdot ft/ft}$

Stability Analysis - Soil Bearing Check

Eccentricity (Live Load Not Over Heel) $e = 1\ \mathrm{ft},\ 2.5\ \mathrm{in}$

Eccentricity (Live Load Over Heel) $e_L = 1\ \mathrm{ft},\ 2\ \mathrm{in}$

Bearing Pressures $\mathbf{BP} =$ IBC 2018, Cl 1605.2

Location	Live Load Not Over Heel q (psf)	Live Load Over Heel q_L (psf)
Toe	1390	1380
At d from stem face	602	613
Stem face	454	469
Heel	67.6	93.3

Unfactored Vertical Loads for Structural Strength Design

Lateral Force on Stem Due to Dead Load Surcharge $P_{D,stem} = 0.18\ \mathrm{kip/ft}$

Lateral Force on Stem Due to Live Load Surcharge $P_{L,stem} = 0.144\ \mathrm{kip/ft}$

Lateral Force on Stem Due to Backfill $P_{bf,stem} = 2.88\ \mathrm{kip/ft}$

Structural Strength Design Loads

Lateral Stem Loads $\mathbf{SL} =$ IBC 2018, Cl 1605.2

Element	Unfactored Forces H (kip/ft)	Load Factor ξ	Factored Horizontal Loads H_u (kip/ft)	Moment Arm y (ft)	Stem Moment $M_{u,stem}$ (kip · ft/ft)
Dead Load Surcharge	0.18	1.6	0.287	6.42	1.84
Live Load Surcharge	0.144	1.6	0.23	6.42	1.48
Backfill	2.88	1.6	4.61	4.28	19.7

Heel Loads $\mathbf{HL} =$

Element	Unfactored Forces W (kip/ft)	Load Factor ξ	Factored Weight W_u (kip/ft)	Moment Arm y (ft)	Heel Moment $M_{u,heel}$ (kip · ft/ft)
Dead Load Surcharge	0.075	1.2	0.09	0.75	0.0675
Live Load Surcharge	0.06	1.6	0.096	0.75	0.072
Heel Weight	0.263	1.2	0.315	0.75	0.236
Backfill Above Water Table	2.41	1.2	2.89	0.75	2.17

Toe Loads (Shear) $\mathbf{TL}_V =$

Element	Unfactored Shear Load at d V_d (kip/ft)	Load Factor ξ	Factored Shear Load V_u (kip/ft)
Upwards Soil Pressure	4.77	1.6	7.63
Toe Weight	-0.836	0.9	-0.753

Toe Loads (Moment) $\mathbf{TL}_M =$

Element	Unfactored Toe Loads P (kip/ft)	Load Factor ξ	Moment Arm y (ft)	Toe Moment M_u (kip · ft/ft)
Upwards Soil Pressure	5.24	1.6	3.3	27.7
Toe Weight	-0.992	0.9	2.84	-2.53

Stem Flexural Analysis (ACI 318-14, Cl 22.2)

Tension Reinforcement Strain $\varepsilon_t = 0.00773$ ACI 318-14, Cl 22.2.2.4.1 and Cl 7.3.3.1 for strain limit

Resistance Factor in Bending	$\phi_b = 0.9$	ACI 318-14. Table 21.2.2
Factored Moment Capacity	$\phi M_n = 26\,800\ \mathrm{lb \cdot ft/ft}$	ACI 318-14, 8.5.1.1a

Heel Flexural Analysis (ACI 318-14, Cl 22.2)

Tension Reinforcement Strain	$\varepsilon_t = 0.0188$	ACI 318-14, Cl 22.2.2.4.1 and Cl 7.3.3.1 for strain limit
Resistance Factor in Bending	$\phi_b = 0.9$	ACI 318-14. Table 21.2.2
Factored Moment Capacity	$\phi M_{n,heel} = 24\,300\ \mathrm{lb \cdot ft/ft}$	ACI 318-14, 8.5.1.1a

Toe Flexural Analysis (ACI 318-14, Cl 22.2)

Tension Reinforcement Strain	$\varepsilon_t = 0.0157$	ACI 318-14, Cl 22.2.2.4.1 and Cl 7.3.3.1 for strain limit
Resistance Factor in Bending	$\phi_b = 0.9$	ACI 318-14. Table 21.2.1
Factored Moment Capacity	$\phi M_{n,toe} = 27\,800\ \mathrm{lb \cdot ft/ft}$	ACI 318-14, 8.5.1.1a

Shear in Stem (ACI 318-14, Cl 22.5)

Resistance Factor in Shear	$\phi_v = 0.75$	ACI 318-14 Table 21.2.1
Factored Stem Shear Capacity	$\phi V_{n,stem} = 7370\ \mathrm{plf}$	ACI 318-14, Cl 22.5.5

Shear in Heel (ACI 318-14, Cl 22.5)

Factored Base Shear Capacity	$\phi V_{n,base} = 9670\ \mathrm{plf}$	ACI 318-14, Cl 22.5.5

Shear in Toe (ACI 318-14, Cl 22.5)

Resistance Factor in Shear (Toe)	$\phi_{v,toe} = 0.75$	ACI 318-14 Table 21.2.1
Factored Toe Shear Capacity	$\phi V_{n,toe} = 9620\ \mathrm{plf}$	ACI 318-14, Cl 22.5.5

Comments

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***

- ***Typical Posts***

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LONGITUDE
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Hold-down anchor design calculations

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

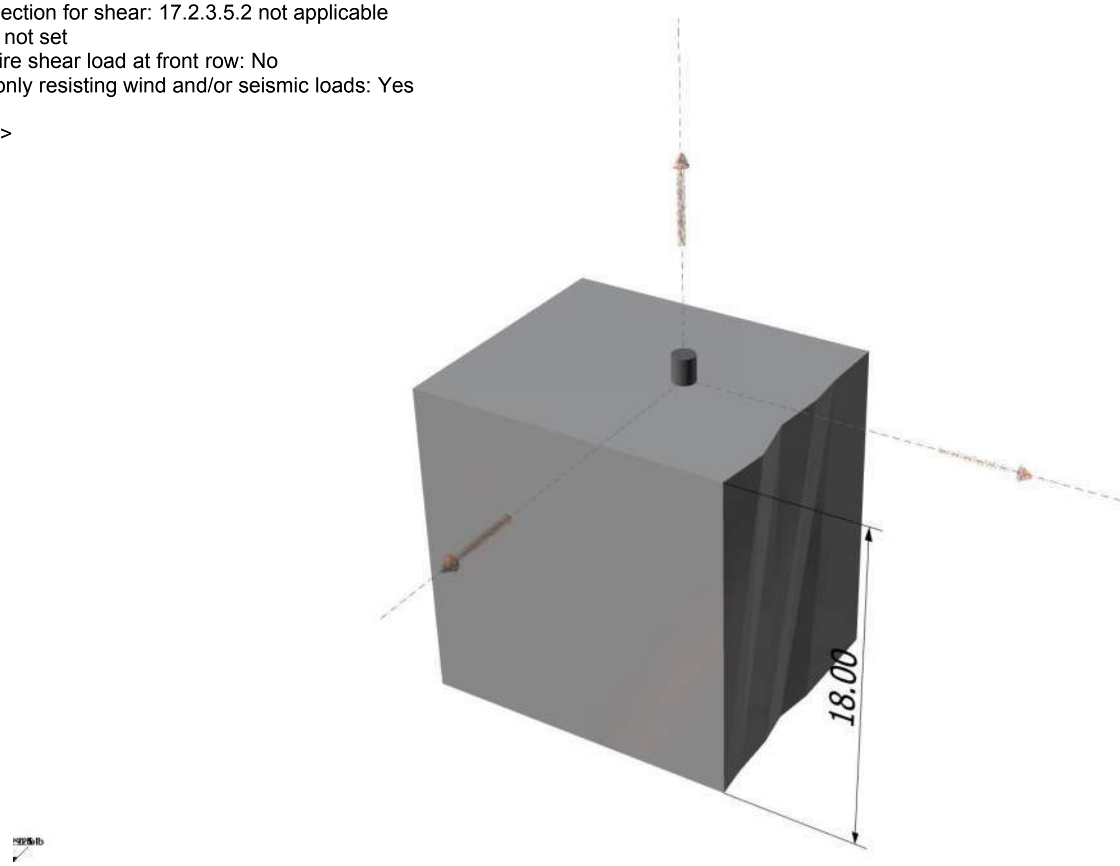
General
Design method:ACI 318-14
Units: Imperial units

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

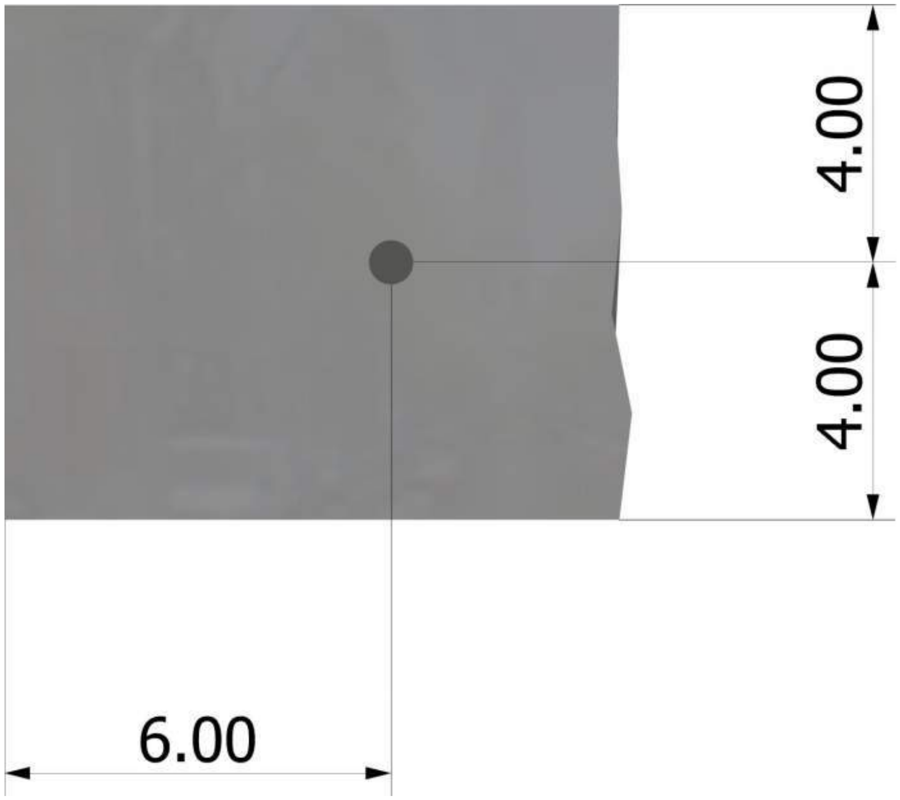
Base Material
Concrete: Normal-weight
Concrete thickness, h (inch): 18.00
State: Cracked
Compressive strength, f'_c (psi): 2500
$\Psi_{c,V}$: 1.0
Reinforcement condition: A tension, A shear
Supplemental reinforcement: Not applicable
Reinforcement provided at corners: Yes
Ignore concrete breakout in tension: 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>



<Figure 2>



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

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

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.

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

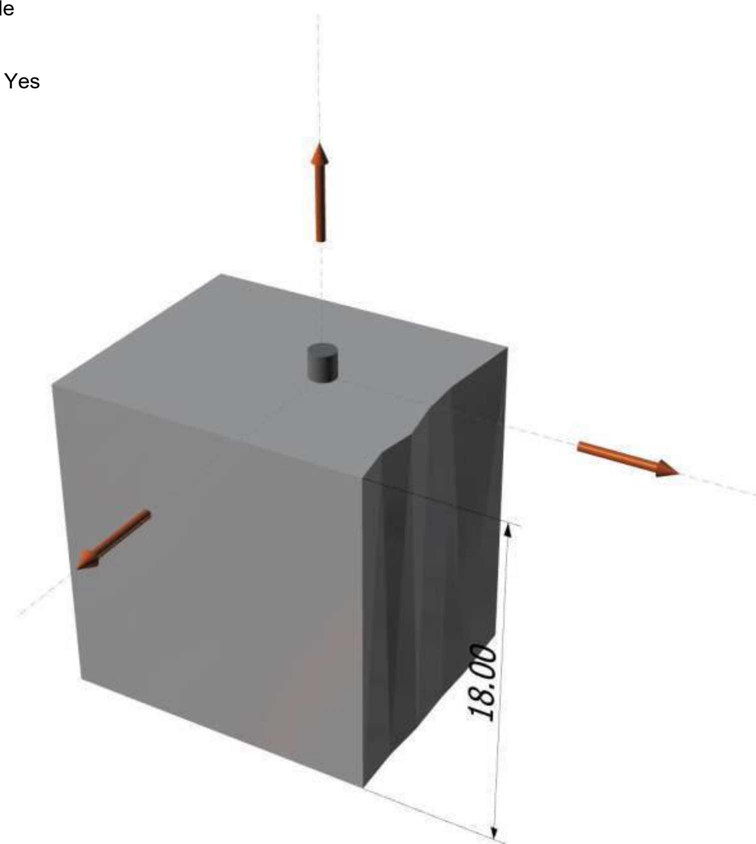
General
Design method:ACI 318-14
Units: Imperial units

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

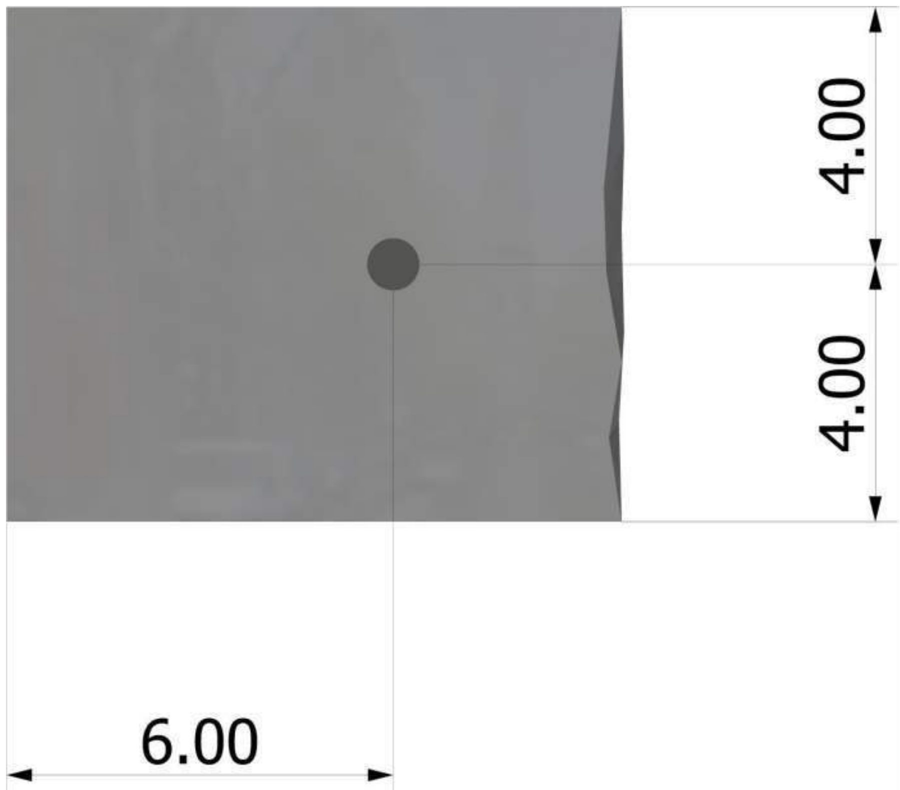
Load and Geometry
Load factor source: ACI 318 Section 5.3
Load combination: U = 0.9D + 1.0E
Seismic design: Yes
Anchors subjected to sustained tension: Not applicable
Ductility section for tension: 17.2.3.4.3 (a) (iii)-(vi) is satisfied
Ductility section for shear: 17.2.3.5.2 not applicable
Ω_0 factor: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

<Figure 1>



3050 lb

<Figure 2>



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

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)

$0.75\phi N_{pn} = 0.75\phi\Psi_{c,P}N_p = 0.75\phi\Psi_{c,P}8A_{brg}f'_c$ (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	3.53	2500	0.70	37107

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, ϕN_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.

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

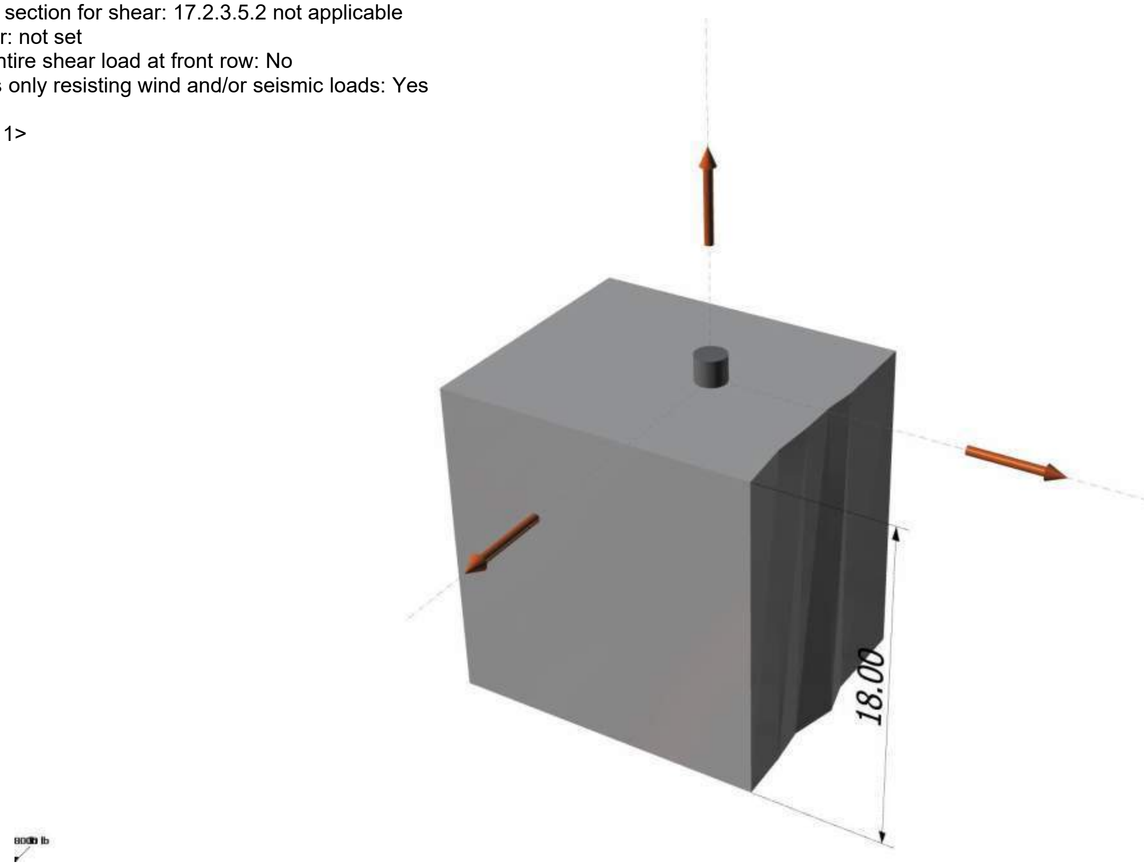
General
Design method:ACI 318-14
Units: Imperial units

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>



<Figure 2>

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

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

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, ϕN_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.

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.

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

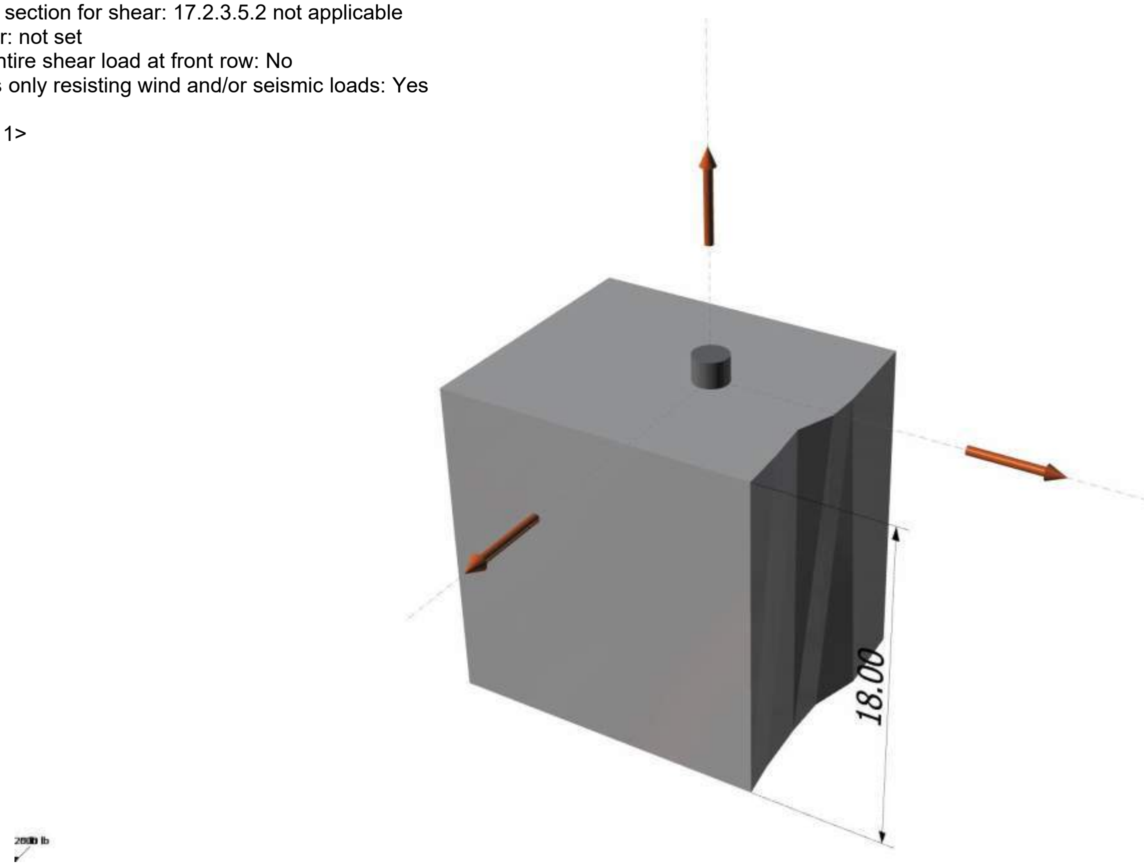
General
Design method:ACI 318-14
Units: Imperial units

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

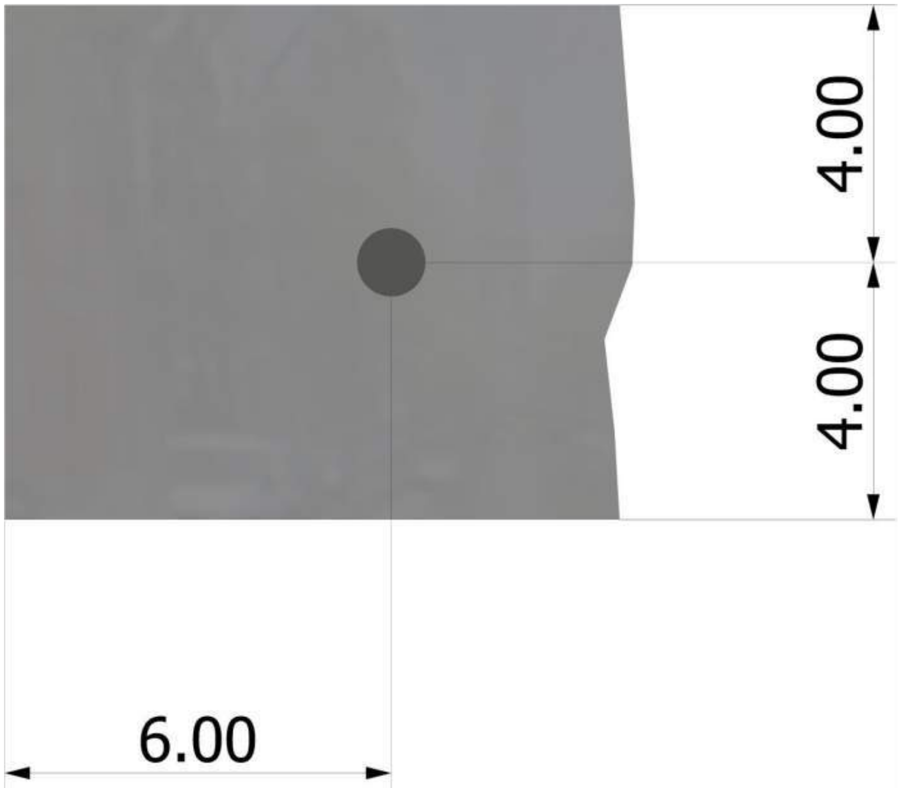
Base Material
Concrete: Normal-weight
Concrete thickness, h (inch): 18.00
State: Cracked
Compressive strength, f'_c (psi): 2500
$\Psi_{c,V}$: 1.0
Reinforcement condition: A tension, A shear
Supplemental reinforcement: Not applicable
Reinforcement provided at corners: Yes
Ignore concrete breakout in tension: 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>



<Figure 2>



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

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

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

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

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.

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

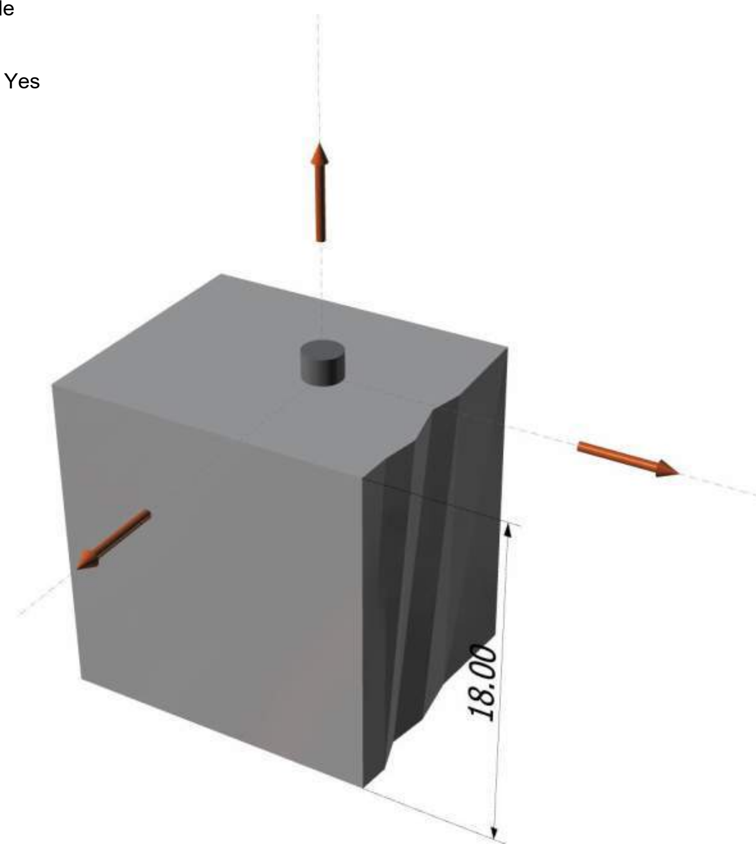
General
Design method:ACI 318-14
Units: Imperial units

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

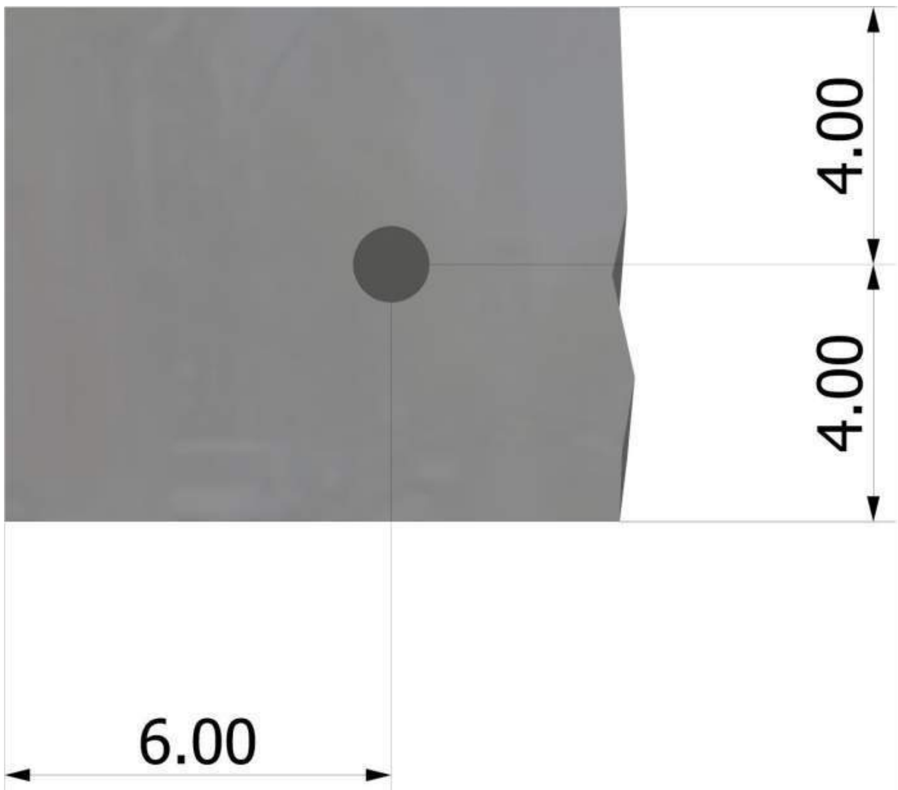
Base Material
Concrete: Normal-weight
Concrete thickness, h (inch): 18.00
State: Cracked
Compressive strength, f'_c (psi): 2500
$\Psi_{c,V}$: 1.0
Reinforcement condition: A tension, A shear
Supplemental reinforcement: Not applicable
Reinforcement provided at corners: Yes
Ignore concrete breakout in tension: 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>



<Figure 2>



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

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

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.

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.

Anchor Designer™
Software
Version 2.5.6582.0

Company:	L120 Engineering & Design	Date:	1/14/2018
Engineer:	MRT	Page:	1/5
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 1/4" DIA Anchor

2. Input Data & Anchor Parameters

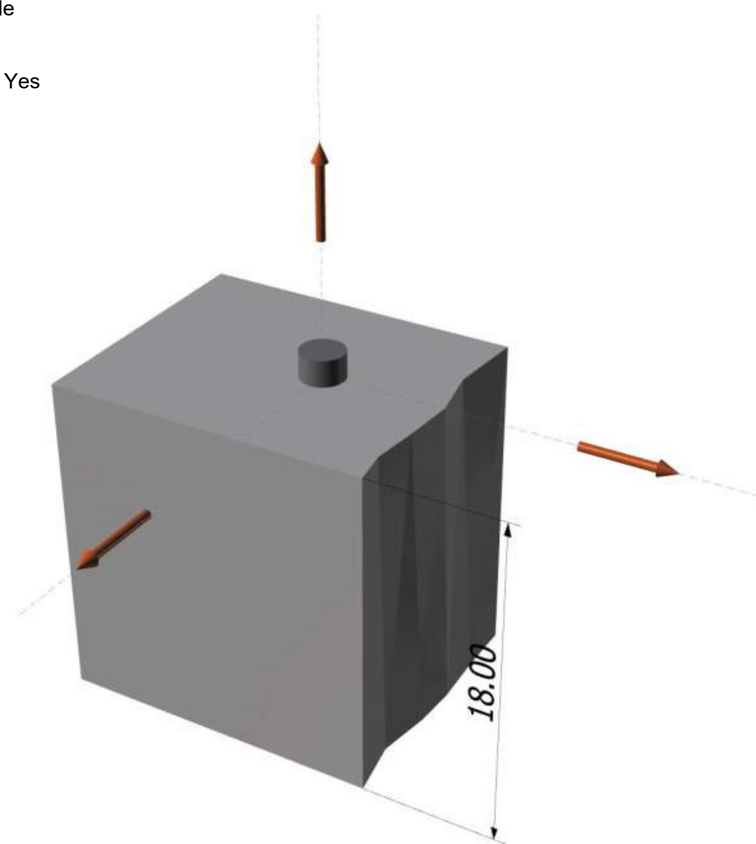
General
Design method:ACI 318-14
Units: Imperial units

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>



<Figure 2>

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

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

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, ϕN_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.

Hand-rail Calculations

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End Post Anchor Bolt Design:

Pv = 25 lbs

Ph = 200 lbs

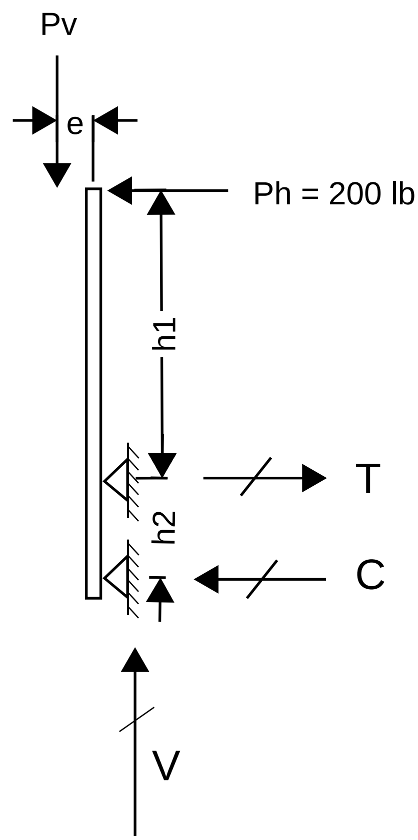
h1 = 46"

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 #"

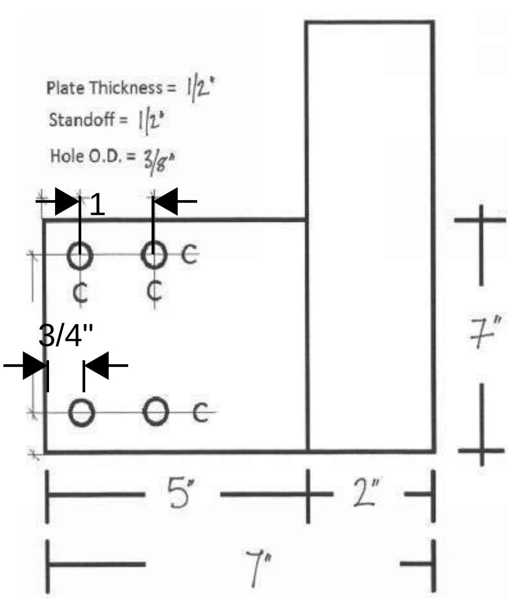
My = 200# x 4.5" = 900 #"



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.

Middle Post Anchor Bolt Design:

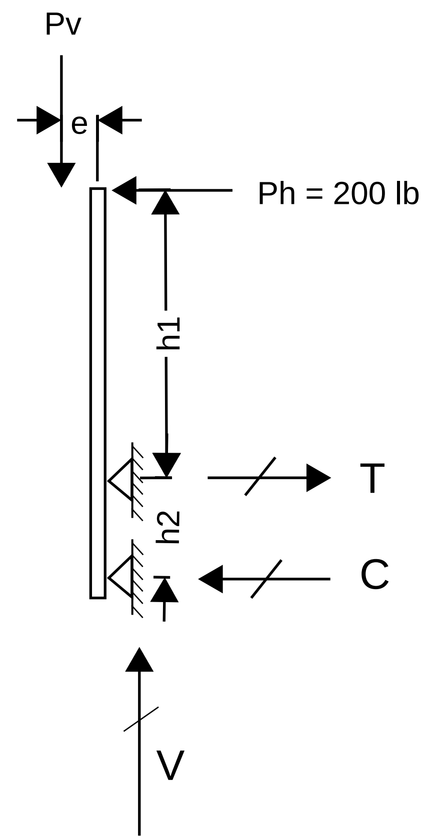
Pv = 25 lbs

Ph = 250 lbs

h1 = 46"

h2 = 5.5"

e = 1.5"

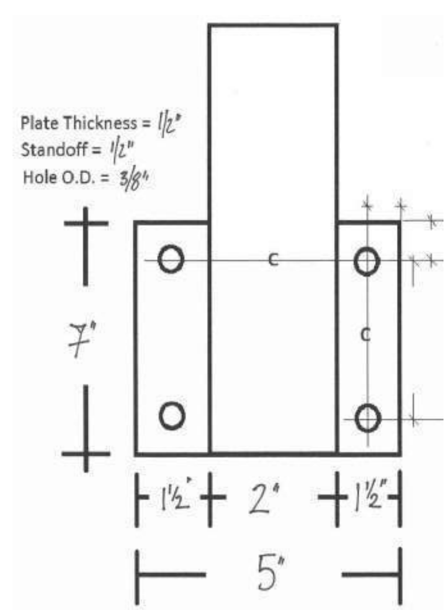


Anchor Moment M = Pv(e) + Ph (h1 + h2/2)
= 25 x 1.5 + 250 (46+5.5/2)
= 12,250

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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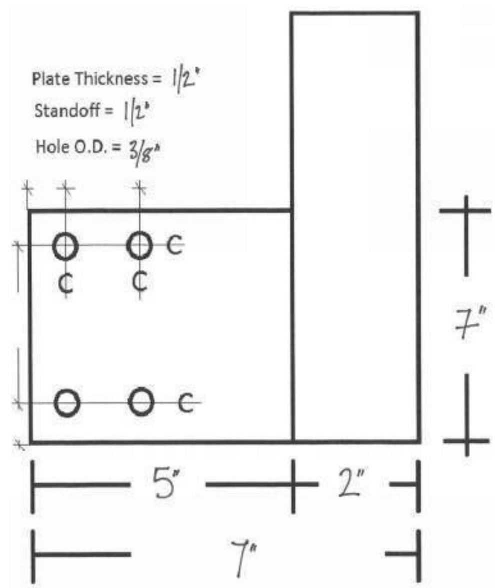
Mounting Plate Design:



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

Try 1/2" thick Plate

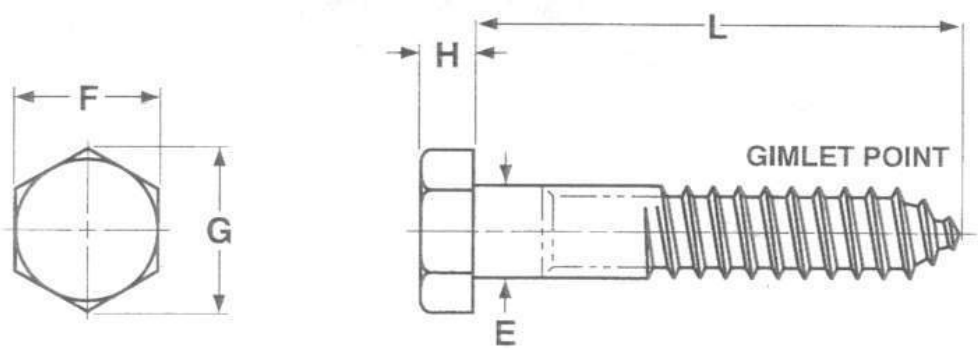
Plate Bending Stress: fbx = Mx/2/Sx
= 9788/2/(1/4 x 5" x (1/2)^2)
= 15,660 psi
fby = My/Sy
= 900/(1/4 x 7" x (1/2)^2)
= 2,057 psi

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.

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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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		
		$T \leq 100°F$	$100°F < T \leq 125°F$	$125°F < T \leq 150°F$
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.

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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BY ______________________ DATE ___ / ___ / ___

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.

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

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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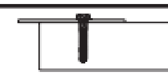
BY ______________________ DATE ___/___/___

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 in.	D in.	$Z_{\parallel}$ lbs.	$Z_{\perp}$ lbs.	$Z_{\parallel}$ lbs.	$Z_{\perp}$ lbs.	$Z_{\parallel}$ lbs.	$Z_{\perp}$ lbs.	$Z_{\parallel}$ lbs.	$Z_{\perp}$ lbs.	$Z_{\parallel}$ lbs.	$Z_{\perp}$ lbs.	$Z_{\parallel}$ lbs.	$Z_{\perp}$ lbs.	$Z_{\parallel}$ lbs.	$Z_{\perp}$ lbs.	$Z_{\parallel}$ lbs.	$Z_{\perp}$ lbs.	$Z_{\parallel}$ lbs.	$Z_{\perp}$ lbs.	$Z_{\parallel}$ 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}.

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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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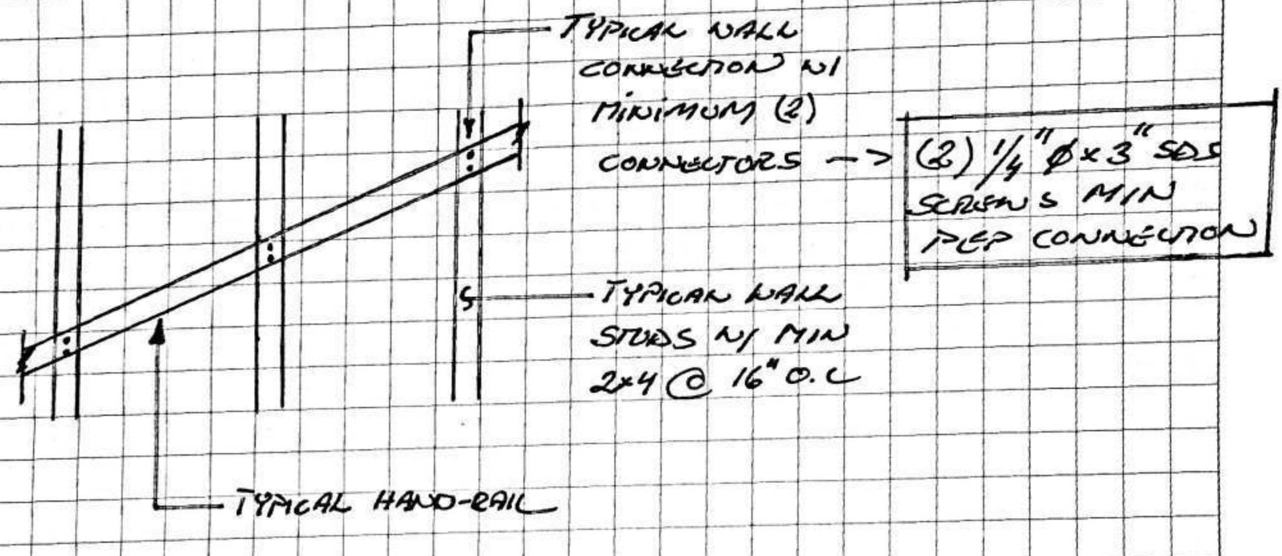
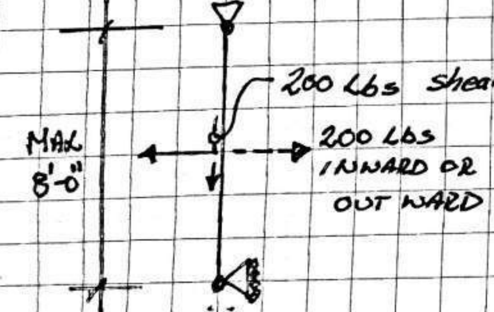
CALCULATIONS

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 $= 2 \times 160 = 320\ LBS$

$\boxed{200\ Lbs\ demand < 320\ Lbs\ CAPACITY\ \checkmark}$

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

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

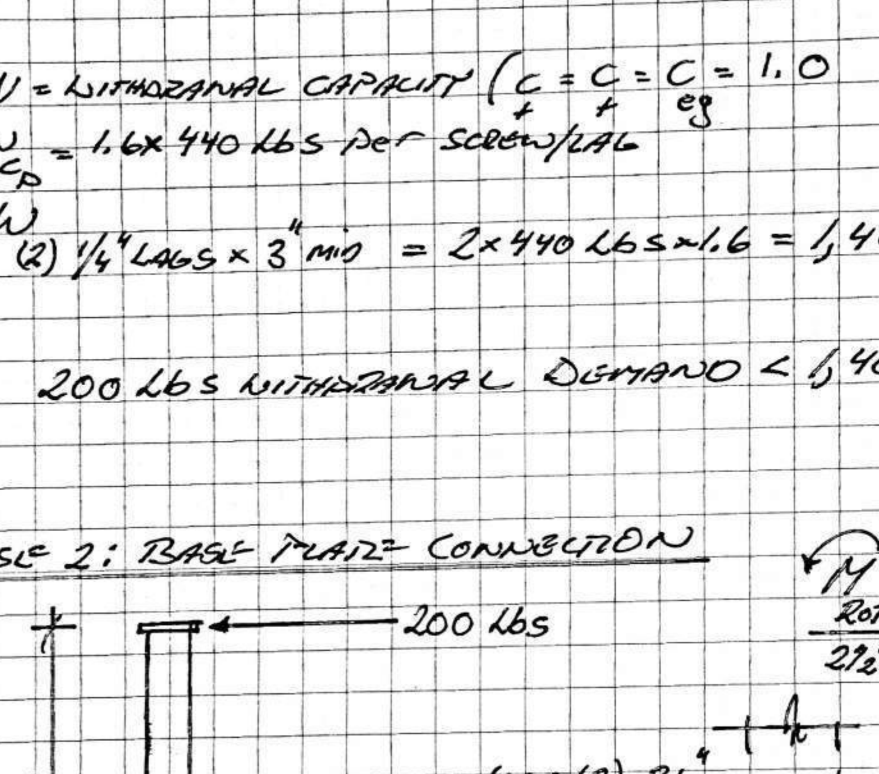
W_{c_D} = 1.6× 440 Lbs per SCREW/LAG = 179 Lbs × 2½" ≅ 446 Lbs PER ¼"ϕ LAG×3"

$W_{(2)\ 1/4" \text{ LAGS} \times 3" \text{ min}}$ = 2×440 Lbs×1.6 = 1,408 Lbs

200 Lbs WITHDRAWAL DEMAND < 1,408 Lbs CAPACITY ☑

CASE 2: BASE PLATE CONNECTION

$$\frac{M_{ROT}}{2\tfrac{1}{2}"} = \frac{200 \text{ Lbs} \times 36"}{2\tfrac{1}{2}"} = 2,880 \text{ Lbs}$$



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

W = 179 Lbs × 4" × 1.6 (2×¼"ϕ × 4½" LAGS) = 1,145 Lbs per ¼ϕ LAG×4½"

"NOT WORKING" ~~$W_{(2)\ 1/4" \times 4\frac{1}{2}}$ ϕ × 2 = 1,145 Lbs~~

$W_{(2)\ 3/8" \times 4\frac{1}{2}}$ × 2 = 243 × 4" × 2 × 1.6 = 3,110

2,880 Lbs demand < 3,110 CAPACITY ✓

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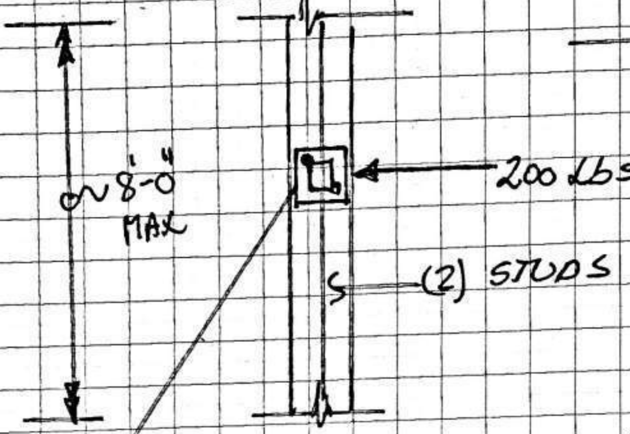
CASE 3: HORIZONTAL END-PLATE CONNECTIONS



→ SEE ATTACHED CALCULATIONS OF STUD CALCULATIONS.

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

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

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

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

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

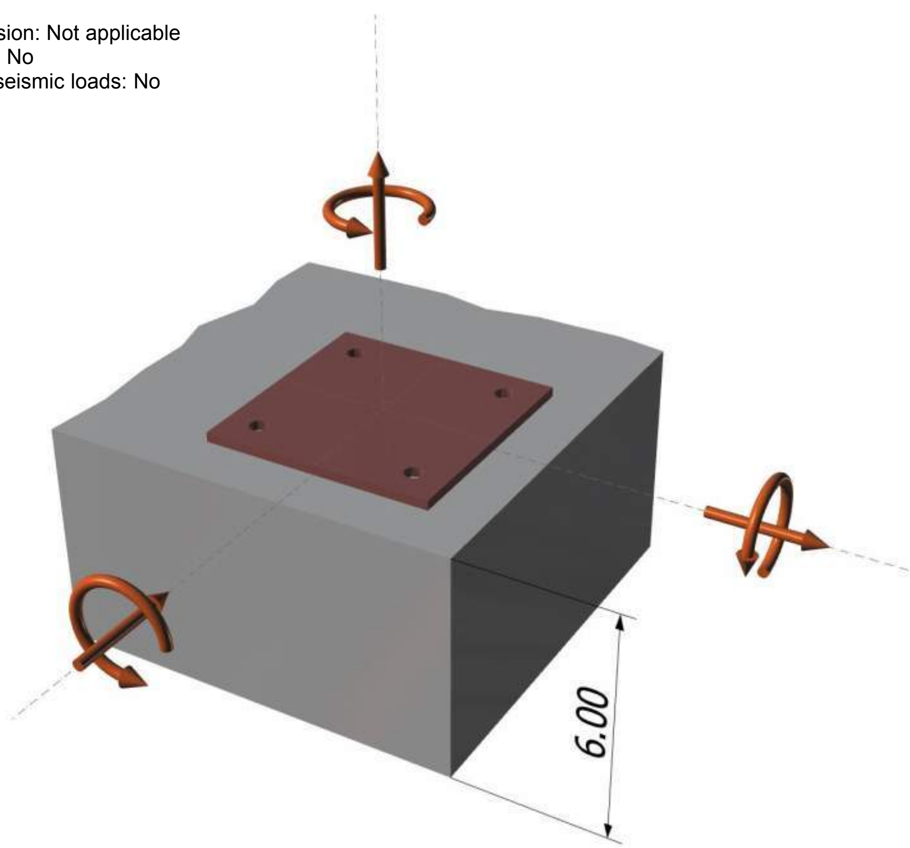
Anchor Information:
Anchor type: Concrete screw
Material: Carbon Steel
Diameter (inch): 0.375
Nominal Embedment depth (inch): 3.250
Effective Embedment depth, h_{ef} (inch): 2.400
Code report: ICC-ES ESR-2713
Anchor category: 1
Anchor ductility: No
h_{min} (inch): 5.00
c_{ac} (inch): 3.63
C_{min} (inch):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

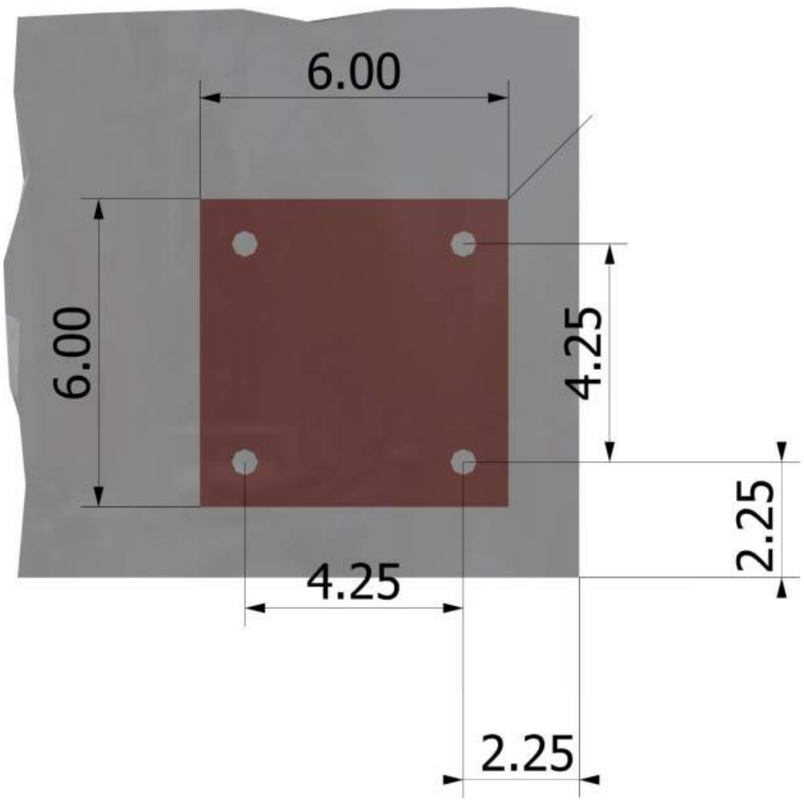
Base Plate
Length x Width x Thickness (inch): 6.00 x 6.00 x 0.25

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

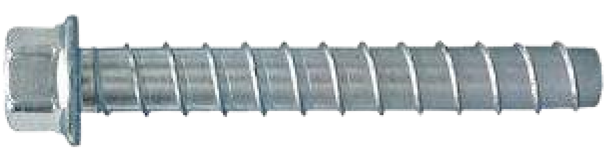
<Figure 1>



<Figure 2>



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

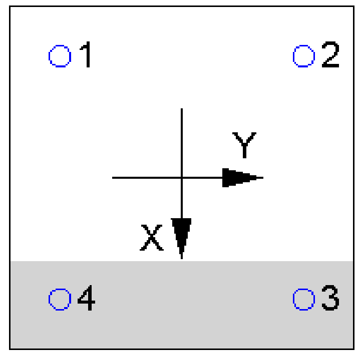


3. Resulting Anchor Forces

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

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.

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

StruCalc 9.0

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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	

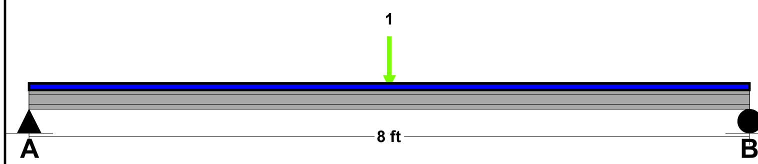
MATERIAL PROPERTIES
#2 - Hem-Fir

	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

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

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

StruCalc 9.0

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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	

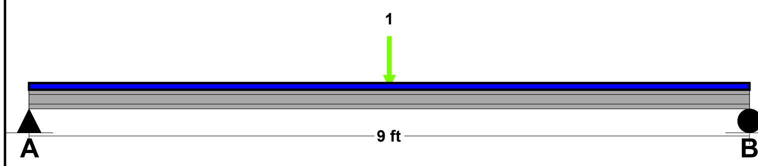
MATERIAL PROPERTIES
#2 - Hem-Fir

	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

LOADING DIAGRAM



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

StruCalc 9.0

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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	

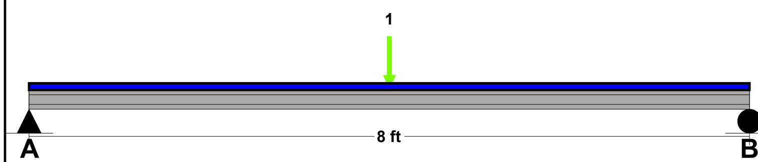
MATERIAL PROPERTIES
#2 - Hem-Fir

	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

LOADING DIAGRAM



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

PASS

DATE:	3/3/2021	COMPANY:	L120 Engineering & Design, LLC
VITRUVIUS BUILD:	StruCalc	DESIGNED BY:	Mans Thurfjell
CUSTOMER:		REVIEWED BY:	Mans Thurfjell
PROJECT LOCATION:	,		
LEVEL:	Roof	LOADING:	ASD
LOCATION:	2x6 Balloon Frame (12" o.c.) (wind load	CODE:	2018 International Building Code
TYPE:	COLUMN	NDS:	2018 NDS
MATERIAL:	SOLID SAWN		

Hem-Fir	No. 2	(1) 1.5 X 5.5	DRY		

COLUMN PROPERTIES

Start (ft): 0 End (ft): 17.25 Member Slope: 0/12 Actual Length (ft): 17.25

Area	Ix	Iy	BSW	Lams	G	Kcr
(in^2)	(in^4)	(in^4)	(lbf/ft)			Creep Factor
8.25	20.8	1.55	1.63	1	0.43	1

STRENGTH PROPERTIES

	Fb (psi)	Ft (psi)	Fv (psi)	Fc (psi)	Fc⊥(psi)	E (psi) x10^3	Emin (psi) x10^3
Base Values	850	525	150	1300	405	1300	470
Adjusted Values	1105	682	150	1430	405	1300	470
C_M	1	1	1	1	1	1	1
C_T	1	1	1	1	1	1	1
C_i	1	1	1	1	1	1	1
C_F	1.3	1.3	1	1.1	1	1	1

Bending Adjustment Factors C_{fu} = 1 C_r = 1

COLUMN DATA

		Unbraced Length (ft)		Column End					
Span	Length (ft)	X	Y	Offset	CP	Ke(X Axis)	Ke(Y Axis)	KeL/d (X Axis)	KeL/d (Y Axis)
1	17.25	17.25	1	0	0.18	1.00	1.00	37.64	8

PASS-FAIL

	PASS/FAIL	MAGNITUDE	STRENGTH	LOCATION (ft)	LOAD COMBO	DURATION FACTOR CD
Shear Stress Y (psi)	**PASS (89.5%)**	15.7	150.0	17.25	D+L	1
Bending Stress Y (psi)	**PASS (46.3%)**	590.2	1099.4	8.62	D+L	1
Deflection (in)	**PASS (35.9%)**	0.737 (=L/281)	1.150 (=L/180)	8.62	L	
Compressive Stress (psi)	**PASS (61.6%)**	100.4	261.1	0	D+L	1
Bearing Stress (psi)	**PASS (98.9%)**	16.4	1430.0	0	D+L	1
Bending-Compression (Unit)	**PASS (1.6%)**	0.98	1.00	8.62	D+L	1

REACTIONS

Units for V: lbf Units for M: lbf-ft

Z axis	DEAD	LIVE	LIVE ROOF	SNOW	WIND +	WIND -	SEISMIC +	SEISMIC -	ICE	RAIN	EARTH
A	328	500	0	0	0	0	0	0	0	0	0
B	0	0	0	0	0	0	0	0	0	0	0
Y axis											
A	0	86	0	0	0	0	0	0	0	0	0
B	0	86	0	0	0	0	0	0	0	0	0

Reaction Location

A B

LOAD LIST

Type	Left Magnitude	Right Magnitude	Load Start (ft)	Load End (ft)	Load Type	Direction
Uniform (lbf/ft)	10	10	0	17.25	Live	Y
Point (lbf)	-500	-	17.25	-	Live	Z
Point (lbf)	-300	-	17.25	-	Dead	Z
Self Weight (lbf/ft)	1.63	1.63	0	17.25	Dead	Z

NOTES

PASS

DATE:	3/3/2021	COMPANY:	L120 Engineering & Design, LLC
VITRUVIUS BUILD:	StruCalc	DESIGNED BY:	Mans Thurfjell
CUSTOMER:		REVIEWED BY:	Mans Thurfjell
PROJECT LOCATION:	,		
LEVEL:	Roof	LOADING:	ASD
LOCATION:	1.75x5.5 LSL Balloon Frame (@12") (win[illegible]	CODE:	2018 International Building Code
TYPE:	COLUMN	NDS:	2018 NDS
MATERIAL:	STRUCTURAL COMPOSITE LUMBER		

Weyerhaeuser	1.55E TimberStrand LSL	(1) 1.75 X 5.5	DRY		

COLUMN PROPERTIES

Start (ft): 0 End (ft): 17.25 Member Slope: 0/12 Actual Length (ft): 17.25

Area	Ix	Iy	BSW	Lams	Cfn	Kcr
(in^2)	(in^4)	(in^4)	(lbf/ft)			Creep Factor
9.62	24.26	2.46	3.01	1	10.87	1

STRENGTH PROPERTIES

	Fb (psi)	Ft (psi)	Fv (psi)	Fc (psi)	Fc⊥(psi)	E (psi) $x10^3$	Emin (psi) $x10^3$
Base Values	2325	1290	310	2170	900	1550	787.815
Adjusted Values	2325	1290	310	2170	900	1550	788
C_M	1	1	1	1	1	1	1
C_T	1	1	1	1	1	1	1

Bending Adjustment Factors C_V = 1.07 C_r = 1 Volume factor Is applied on a load combination basis And Is Not reflected in the adjusted values

COLUMN DATA

Span	Length (ft)	Unbraced Length (ft) X	Unbraced Length (ft) Y	Column End Offset	CP	Ke(X Axis)	Ke(Y Axis)	KeL/d (X Axis)	KeL/d (Y Axis)
1	17.25	17.25	1	0	0.21	1.00	1.00	37.64	6.86

PASS-FAIL

	PASS/FAIL	MAGNITUDE	STRENGTH	LOCATION (ft)	LOAD COMBO	DURATION FACTOR CD
Shear Stress Y (psi)	**PASS (95.7%)**	13.4	310.0	17.25	D+L	1
Bending Stress Y (psi)	**PASS (79.7%)**	505.9	2486.4	8.62	D+L	1
Deflection (in)	**PASS (53.9%)**	0.530 (=L/391)	1.150 (=L/180)	8.62	L	
Compressive Stress (psi)	**PASS (40.5%)**	265.1	445.7	0	D+L	1
Bearing Stress (psi)	**PASS (99.4%)**	14.1	2170.0	0	D+L	1
Bending-Compression (Unit)	**PASS (17.5%)**	0.82	1.00	8.45	D+L	1

REACTIONS

Units for V: lbf Units for M: lbf-ft

Z axis	DEAD	LIVE	LIVE ROOF	SNOW	WIND +	WIND -	SEISMIC +	SEISMIC -	ICE	RAIN	EARTH
A	1052	1500	0	0	0	0	0	0	0	0	0
B	0	0	0	0	0	0	0	0	0	0	0
Y axis											
A	0	86	0	0	0	0	0	0	0	0	0
B	0	86	0	0	0	0	0	0	0	0	0

Reaction Location

A B

LOAD LIST

Type	Left Magnitude	Right Magnitude	Load Start (ft)	Load End (ft)	Load Type	Direction
Point (lbf)	-1500	-	17.25	-	Live	Z
Point (lbf)	-1000	-	17.25	-	Dead	Z
Uniform (lbf/ft)	10	10	0	17.25	Live	Y
Self Weight (lbf/ft)	3.01	3.01	0	17.25	Dead	Z

NOTES

PASS

DATE:	3/3/2021	COMPANY:	L120 Engineering & Design, LLC
VITRUVIUS BUILD:	StruCalc	DESIGNED BY:	Mans Thurfjell
CUSTOMER:		REVIEWED BY:	Mans Thurfjell
PROJECT LOCATION:	,		
LEVEL:	Roof	LOADING:	ASD
LOCATION:	2x6 Balloon Frame (8" o.c.) (Wind load factor applied to stud wall)	CODE:	2018 International Building Code
TYPE:	COLUMN	NDS:	2018 NDS
MATERIAL:	SOLID SAWN		

Hem-Fir	No. 2	(1) 1.5 X 5.5	DRY		

COLUMN PROPERTIES

Start (ft): 0 End (ft): 17.25 Member Slope: 0/12 Actual Length (ft): 17.25

Area	Ix	Iy	BSW	Lams	G	Kcr
(in^2)	(in^4)	(in^4)	(lbf/ft)			Creep Factor
8.25	20.8	1.55	1.63	1	0.43	1

STRENGTH PROPERTIES

	Fb (psi)	Ft (psi)	Fv (psi)	Fc (psi)	Fc⊥(psi)	E (psi) x10^3	Emin (psi) x10^3
Base Values	850	525	150	1300	405	1300	470
Adjusted Values	1105	682	150	1430	405	1300	470
C_M	1	1	1	1	1	1	1
C_T	1	1	1	1	1	1	1
C_i	1	1	1	1	1	1	1
C_F	1.3	1.3	1	1.1	1	1	1

Bending Adjustment Factors C_{fu} = 1 C_r = 1

COLUMN DATA

		Unbraced Length (ft)		Column End					
Span	Length (ft)	X	Y	Offset	CP	Ke(X Axis)	Ke(Y Axis)	KeL/d (X Axis)	KeL/d (Y Axis)
1	17.25	17.25	1	0	0.18	1.00	1.00	37.64	8

PASS-FAIL

	PASS/FAIL	MAGNITUDE	STRENGTH	LOCATION (ft)	LOAD COMBO	DURATION FACTOR CD
Shear Stress Y (psi)	**PASS (92.7%)**	11.0	150.0	17.25	D+L	1
Bending Stress Y (psi)	**PASS (62.4%)**	413.1	1099.4	8.62	D+L	1
Deflection (in)	**PASS (55.1%)**	0.516 (=L/401)	1.150 (=L/180)	8.62	L	
Compressive Stress (psi)	**PASS (49.9%)**	130.7	261.1	0	D+L	1
Bearing Stress (psi)	**PASS (99.2%)**	11.5	1430.0	0	D+L	1
Bending-Compression (Unit)	**PASS (4.3%)**	0.96	1.00	8.45	D+L	1

REACTIONS

Units for V: lbf Units for M: lbf-ft

Z axis	DEAD	LIVE	LIVE ROOF	SNOW	WIND +	WIND -	SEISMIC +	SEISMIC -	ICE	RAIN	EARTH
A	528	550	0	0	0	0	0	0	0	0	0
B	0	0	0	0	0	0	0	0	0	0	0
Y axis											
A	0	60	0	0	0	0	0	0	0	0	0
B	0	60	0	0	0	0	0	0	0	0	0

Reaction Location

A B

LOAD LIST

Type	Left Magnitude	Right Magnitude	Load Start (ft)	Load End (ft)	Load Type	Direction
Point (lbf)	-550	-	17.25	-	Live	Z
Point (lbf)	-500	-	17.25	-	Dead	Z
Uniform (lbf/ft)	7	7	0	17.25	Live	Y
Self Weight (lbf/ft)	1.63	1.63	0	17.25	Dead	Z

NOTES

PASS

DATE:	3/3/2021	COMPANY:	L120 Engineering & Design, LLC
VITRUVIUS BUILD:	StruCalc	DESIGNED BY:	Mans Thurfjell
CUSTOMER:		REVIEWED BY:	Mans Thurfjell
PROJECT LOCATION:	,		

LEVEL:	Roof	LOADING:	ASD
LOCATION:	175x5.5 LSL Balloon Frame (@ 8") (Wind load factor applied and building LC)	CODE:	2018 International Building Code
TYPE:	COLUMN	NDS:	2018 NDS
MATERIAL:	STRUCTURAL COMPOSITE LUMBER		

Weyerhaeuser	1.55E TimberStrand LSL	(1) 1.75 X 5.5	DRY		

COLUMN PROPERTIES

Start (ft): 0 End (ft): 17.25 Member Slope: 0/12 Actual Length (ft): 17.25

Area	Ix	Iy	BSW	Lams	Cfn	Kcr
(in^2)	(in^4)	(in^4)	(lbf/ft)			Creep Factor
9.62	24.26	2.46	3.01	1	10.87	1

STRENGTH PROPERTIES

	Fb (psi)	Ft (psi)	Fv (psi)	Fc (psi)	Fc⊥(psi)	E (psi) $x10^3$	Emin (psi) $x10^3$
Base Values	2325	1290	310	2170	900	1550	787.815
Adjusted Values	2325	1290	310	2170	900	1550	788
C_M	1	1	1	1	1	1	1
C_T	1	1	1	1	1	1	1

Bending Adjustment Factors C_V = 1.07 C_r = 1 Volume factor Is applied on a load combination basis And Is Not reflected in the adjusted values

COLUMN DATA

Span	Length (ft)	Unbraced Length (ft) X	Unbraced Length (ft) Y	Column End Offset	CP	Ke(X Axis)	Ke(Y Axis)	KeL/d (X Axis)	KeL/d (Y Axis)
1	17.25	17.25	1	0	0.21	1.00	1.00	37.64	6.86

PASS-FAIL

	PASS/FAIL	MAGNITUDE	STRENGTH	LOCATION (ft)	LOAD COMBO	DURATION FACTOR CD
Shear Stress Y (psi)	**PASS (97.0%)**	9.4	310.0	17.25	D+L	1
Bending Stress Y (psi)	**PASS (85.8%)**	354.1	2486.4	8.62	D+L	1
Deflection (in)	**PASS (67.7%)**	0.371 (=L/558)	1.150 (=L/180)	8.62	L	
Compressive Stress (psi)	**PASS (28.9%)**	317.1	445.7	0	D+L	1
Bearing Stress (psi)	**PASS (99.5%)**	9.9	2170.0	0	D+L	1
Bending-Compression (Unit)	**PASS (4.6%)**	0.95	1.00	8.45	D+L	1

REACTIONS

Units for V: lbf Units for M: lbf-ft

Z axis	DEAD	LIVE	LIVE ROOF	SNOW	WIND +	WIND -	SEISMIC +	SEISMIC -	ICE	RAIN	EARTH
A	1052	2000	0	0	0	0	0	0	0	0	0
B	0	0	0	0	0	0	0	0	0	0	0
Y axis											
A	0	60	0	0	0	0	0	0	0	0	0
B	0	60	0	0	0	0	0	0	0	0	0

Reaction Location

A B

LOAD LIST

Type	Left Magnitude	Right Magnitude	Load Start (ft)	Load End (ft)	Load Type	Direction
Point (lbf)	-1000	-	17.25	-	Dead	Z
Point (lbf)	-2000	-	17.25	-	Live	Z
Uniform (lbf/ft)	7	7	0	17.25	Live	Y
Self Weight (lbf/ft)	3.01	3.01	0	17.25	Dead	Z

NOTES

Ledger Calculations

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Table 12.3.3A Assigned Specific Gravities

BEAMS (DF #2, and Engineered Lumber)

Joists and 2x members (HF #2)

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

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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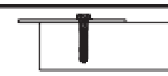
LAG SCREWS

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.	G=0.67 Red Oak $Z_{\perp}$ lbs.	G=0.55 Mixed Maple Southern Pine $Z_{\parallel}$ lbs.	G=0.55 Mixed Maple Southern Pine $Z_{\perp}$ lbs.	G=0.5 Douglas Fir-Larch $Z_{\parallel}$ lbs.	G=0.5 Douglas Fir-Larch $Z_{\perp}$ lbs.	G=0.49 Douglas Fir-Larch (N) $Z_{\parallel}$ lbs.	G=0.49 Douglas Fir-Larch (N) $Z_{\perp}$ lbs.	G=0.46 Douglas Fir(S) Hem-Fir(N) $Z_{\parallel}$ lbs.	G=0.46 Douglas Fir(S) Hem-Fir(N) $Z_{\perp}$ lbs.	G=0.43 Hem-Fir $Z_{\parallel}$ lbs.	G=0.43 Hem-Fir $Z_{\perp}$ lbs.	G=0.42 Spruce-Pine-Fir $Z_{\parallel}$ lbs.	G=0.42 Spruce-Pine-Fir $Z_{\perp}$ lbs.	G=0.37 Redwood (open grain) $Z_{\parallel}$ lbs.	G=0.37 Redwood (open grain) $Z_{\perp}$ lbs.	G=0.36 Eastern Softwoods Spruce-Pine-Fir(S) Western Cedars Western Woods $Z_{\parallel}$ lbs.	G=0.36 Eastern Softwoods Spruce-Pine-Fir(S) Western Cedars Western Woods $Z_{\perp}$ lbs.	G=0.35 Northern Species $Z_{\parallel}$ lbs.	G=0.35 Northern Species $Z_{\perp}$ 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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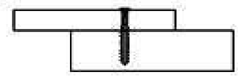
Table 12L **WOOD SCREWS: Reference Lateral Design Values, Z, for Single Shear (two member) Connections[1,2,3]**

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)



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	$\boxed{163}$	159	151	$\boxed{141}$	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.

WOOD SCREWS

DOWEL-TYPE FASTENERS

12

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.

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'.

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

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.

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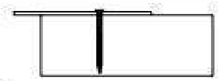
Table 12P (Cont.) **COMMON, BOX, or SINKER STEEL WIRE NAILS: Reference Lateral Design Values, Z, for Single Shear (two member) Connections[1,2,3]**

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	Box Nail	Sinker Nail	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.
		Pennyweight												
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.

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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.

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".

LONGITUDE
ONE TWENTY°
ENGINEERING & DESIGN

PROJECT NO.	SHEET NO.

PROJECT ______________________

SUBJECT ______________________

BY ______________________ DATE ___/___/___

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 $s < 9"$.

LONGITUDE
ONE TWENTY°
ENGINEERING & DESIGN

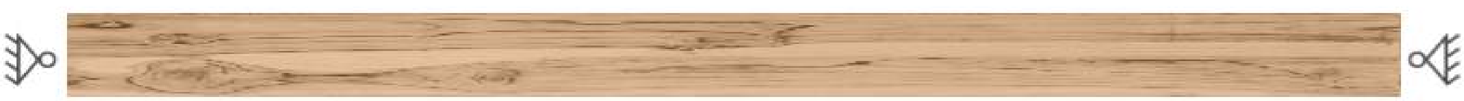
TYPICAL POSTS

PASS

DATE:	3/3/2021	COMPANY:	L120 Engineering & Design, LLC
VITRUVIUS BUILD:	StruCalc	DESIGNED BY:	Mans Thurfjell
CUSTOMER:		REVIEWED BY:	Mans Thurfjell
PROJECT LOCATION:	,		
LEVEL:	Roof	LOADING:	ASD
LOCATION:	2X4 STUD @ 16"	CODE:	2018 International Building Code
TYPE:	COLUMN	NDS:	2018 NDS
MATERIAL:	SOLID SAWN		

Hem-Fir	No. 2	(1) 1.5 X 3.5	DRY		

2X4 STUD @ 16" DIAGRAM



COLUMN PROPERTIES

Start (ft): 0 End (ft): 8 Member Slope: 0/12 Actual Length (ft): 8

Area	Ix	Iy	BSW	Lams	G	Kcr
(in^2)	(in^4)	(in^4)	(lbf/ft)			Creep Factor
5.25	5.36	0.98	1.04	1	0.43	1

STRENGTH PROPERTIES

	Fb (psi)	Ft (psi)	Fv (psi)	Fc (psi)	Fc⊥(psi)	E (psi) $x10^3$	Emin (psi) $x10^3$
Base Values	850	525	150	1300	405	1300	470
Adjusted Values	1275	788	150	1495	405	1300	470
C_M	1	1	1	1	1	1	1
C_T	1	1	1	1	1	1	1
C_i	1	1	1	1	1	1	1
C_F	1.5	1.5	1	1.15	1	1	1

Bending Adjustment Factors C_{fu} = 1 C_r = 1

COLUMN DATA

		Unbraced Length (ft)		Column End					
Span	Length (ft)	X	Y	Offset	CP	Ke(X Axis)	Ke(Y Axis)	KeL/d (X Axis)	KeL/d (Y Axis)
1	8	8	4	0	0.24	1.00	1.00	27.43	32

PASS-FAIL

	PASS/FAIL	MAGNITUDE	STRENGTH	LOCATION (ft)	LOAD COMBO	DURATION FACTOR CD
Deflection (in)	**PASS (90.5%)**	0.025 (=L/3795)	0.267 (=L/360)	8	L	
Compressive Stress (psi)	**PASS (3.0%)**	344.4	355.2	0	D+L	1

REACTIONS

Units for V: lbf Units for M: lbf-ft

Z axis	DEAD	LIVE	LIVE ROOF	SNOW	WIND +	WIND -	SEISMIC +	SEISMIC -	ICE	RAIN	EARTH
A	8	1800	0	0	0	0	0	0	0	0	0
B	0	0	0	0	0	0	0	0	0	0	0

Reaction Location

A B

LOAD LIST

Type	Left Magnitude	Right Magnitude	Load Start (ft)	Load End (ft)	Load Type	Direction
Point (lbf)	-1800	-	8	-	Live	Z
Self Weight (lbf/ft)	1.04	1.04	0	8	Dead	Z

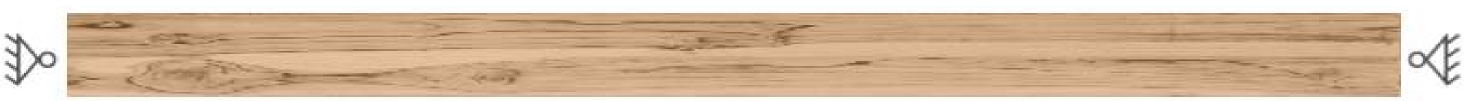
NOTES

PASS

DATE:	10/8/2020	COMPANY:	L120 Engineering & Design, LLC
VITRUVIUS BUILD:	StruCalc	DESIGNED BY:	Mans Thurfjell
CUSTOMER:		REVIEWED BY:	Mans Thurfjell
PROJECT LOCATION:	,		
LEVEL:	Main Floor	LOADING:	ASD
LOCATION:	2x4 @ 12" o.c.	CODE:	2018 International Building Code
TYPE:	COLUMN	NDS:	2018 NDS
MATERIAL:	SOLID SAWN		

Hem-Fir	No. 2	(1) 1.5 X 3.5	DRY		

2x4 @ 12" o.c. DIAGRAM



COLUMN PROPERTIES

Start (ft): 0 End (ft): 9 Member Slope: 0/12 Actual Length (ft): 9

Area	Ix	Iy	BSW	Lams	G	Kcr
(in^2)	(in^4)	(in^4)	(lbf/ft)			Creep Factor
5.25	5.36	0.98	1.04	1	0.43	1

STRENGTH PROPERTIES

	Fb (psi)	Ft (psi)	Fv (psi)	Fc (psi)	Fc⊥(psi)	E (psi) x10^3	Emin (psi) x10^3
Base Values	850	525	150	1300	405	1300	470
Adjusted Values	1275	788	150	1495	405	1300	470
C_M	1	1	1	1	1	1	1
C_T	1	1	1	1	1	1	1
C_i	1	1	1	1	1	1	1
C_F	1.5	1.5	1	1.15	1	1	1

Bending Adjustment Factors $C_{fu} = 1$ $C_r = 1$

COLUMN DATA

		Unbraced Length (ft)		Column End					
Span	Length (ft)	X	Y	Offset	CP	Ke(X Axis)	Ke(Y Axis)	KeL/d (X Axis)	KeL/d (Y Axis)
1	9	9	2	0	0.25	1.00	1.00	30.86	16

PASS-FAIL

	PASS/FAIL	MAGNITUDE	STRENGTH	LOCATION (ft)	LOAD COMBO	DURATION FACTOR CD
Deflection (in)	**PASS (89.7%)**	0.031 (=L/3495)	0.300 (=L/360)	9	L	
Compressive Stress (psi)	**PASS (1.8%)**	373.2	379.9	0	D+L	1
Tensile Stress (psi)	**PASS (100.0%)**	0.0	708.8	9	D	0.9

REACTIONS

V-(lbf) M-(lbf-ft)

Z axis	DEAD	LIVE	LIVE ROOF	SNOW	WIND +	WIND -	SEISMIC +	SEISMIC -	ICE	RAIN	EARTH
A	9	1950	0	0	0	0	0	0	0	0	0
B	0	0	0	0	0	0	0	0	0	0	0

Reaction Location

A B

LOAD LIST

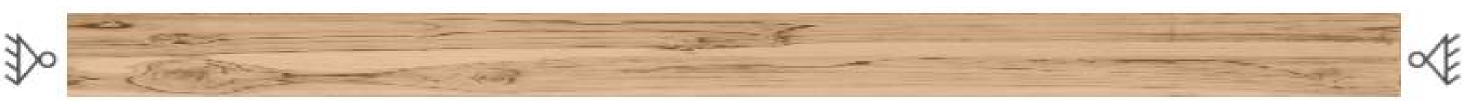
Type	Left Magnitude	Right Magnitude	Load Start (ft)	Load End (ft)	Load Type	Direction
Point (lbf)	-1950	-	9	-	Live	Z
Self Weight (lbf/ft)	1.04	1.04	0	9	Dead	Z

NOTES

PASS

DATE:	3/3/2021	COMPANY:	L120 Engineering & Design, LLC
VITRUVIUS BUILD:	StruCalc	DESIGNED BY:	Mans Thurfjell
CUSTOMER:		REVIEWED BY:	Mans Thurfjell
PROJECT LOCATION:	,		
LEVEL:	Roof	LOADING:	ASD
LOCATION:	(2) 2x4 (unbraced)	CODE:	2018 International Building Code
TYPE:	COLUMN	NDS:	2018 NDS
MATERIAL:	SOLID SAWN		

Hem-Fir	No. 2	(2) 1.5 X 3.5	DRY		



COLUMN PROPERTIES

Start (ft): 0 End (ft): 8 Member Slope: 0/12 Actual Length (ft): 8

Area	Ix	Iy	BSW	Lams	G	Kcr
(in^2)	(in^4)	(in^4)	(lbf/ft)			Creep Factor
10.5	10.72	1.97	2.07	2	0.43	1

STRENGTH PROPERTIES

	Fb (psi)	Ft (psi)	Fv (psi)	Fc (psi)	Fc⊥(psi)	E (psi) $x10^3$	Emin (psi) $x10^3$
Base Values	850	525	150	1300	405	1300	470
Adjusted Values	1275	788	150	1495	405	1300	470
C_M	1	1	1	1	1	1	1
C_T	1	1	1	1	1	1	1
C_i	1	1	1	1	1	1	1
C_F	1.5	1.5	1	1.15	1	1	1

Bending Adjustment Factors C_{fu} = 1 C_r = 1

COLUMN DATA

		Unbraced Length (ft)		Column End					
Span	Length (ft)	X	Y	Offset	CP	Ke(X Axis)	Ke(Y Axis)	KeL/d (X Axis)	KeL/d (Y Axis)
1	8	8	8	0	0.14	1.00	1.00	27.43	32

PASS-FAIL

	PASS/FAIL	MAGNITUDE	STRENGTH	LOCATION (ft)	LOAD COMBO	DURATION FACTOR CD
Deflection (in)	**PASS (96.0%)**	0.011 (=L/9144)	0.267 (=L/360)	8	L	
Compressive Stress (psi)	**PASS (0.9%)**	211.1	213.1	0	D+L	1

REACTIONS

Units for V: lbf Units for M: lbf-ft

Z axis	DEAD	LIVE	LIVE ROOF	SNOW	WIND +	WIND -	SEISMIC +	SEISMIC -	ICE	RAIN	EARTH
A	717	1500	0	0	0	0	0	0	0	0	0
B	0	0	0	0	0	0	0	0	0	0	0

Reaction Location

A B

LOAD LIST

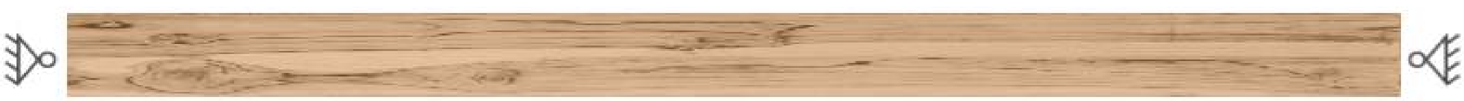
Type	Left Magnitude	Right Magnitude	Load Start (ft)	Load End (ft)	Load Type	Direction
Point (lbf)	-1500	-	8	-	Live	Z
Point (lbf)	-700	-	8	-	Dead	Z
Self Weight (lbf/ft)	2.07	2.07	0	8	Dead	Z

NOTES

PASS

DATE:	3/3/2021	COMPANY:	L120 Engineering & Design, LLC
VITRUVIUS BUILD:	StruCalc	DESIGNED BY:	Mans Thurfjell
CUSTOMER:		REVIEWED BY:	Mans Thurfjell
PROJECT LOCATION:	,		
LEVEL:	Roof	LOADING:	ASD
LOCATION:	(3) 2x4 (unbraced)	CODE:	2018 International Building Code
TYPE:	COLUMN	NDS:	2018 NDS
MATERIAL:	SOLID SAWN		

Hem-Fir	No. 2	(3) 1.5 X 3.5	DRY		



COLUMN PROPERTIES

Start (ft): 0 End (ft): 8 Member Slope: 0/12 Actual Length (ft): 8

Area	Ix	Iy	BSW	Lams	G	Kcr
(in^2)	(in^4)	(in^4)	(lbf/ft)			Creep Factor
15.75	16.08	2.95	3.11	3	0.43	1

STRENGTH PROPERTIES

	Fb (psi)	Ft (psi)	Fv (psi)	Fc (psi)	Fc⊥(psi)	E (psi) x10^3	Emin (psi) x10^3
Base Values	850	525	150	1300	405	1300	470
Adjusted Values	1275	788	150	1495	405	1300	470
C_M	1	1	1	1	1	1	1
C_T	1	1	1	1	1	1	1
C_i	1	1	1	1	1	1	1
C_F	1.5	1.5	1	1.15	1	1	1

Bending Adjustment Factors C_{fu} = 1 C_r = 1

COLUMN DATA

		Unbraced Length (ft)		Column End					
Span	Length (ft)	X	Y	Offset	CP	Ke(X Axis)	Ke(Y Axis)	KeL/d (X Axis)	KeL/d (Y Axis)
1	8	8	8	0	0.29	1.00	1.00	27.43	21.33

PASS-FAIL

	PASS/FAIL	MAGNITUDE	STRENGTH	LOCATION (ft)	LOAD COMBO	DURATION FACTOR CD
Deflection (in)	**PASS (93.0%)**	0.019 (=L/5107)	0.267 (=L/360)	8	L	
Compressive Stress (psi)	**PASS (3.7%)**	414.3	430.1	0	D+L	1

REACTIONS

Units for V: lbf Units for M: lbf-ft

Z axis	DEAD	LIVE	LIVE ROOF	SNOW	WIND +	WIND -	SEISMIC +	SEISMIC -	ICE	RAIN	EARTH
A	2525	4000	0	0	0	0	0	0	0	0	0
B	0	0	0	0	0	0	0	0	0	0	0

Reaction Location

A B

LOAD LIST

Type	Left Magnitude	Right Magnitude	Load Start (ft)	Load End (ft)	Load Type	Direction
Point (lbf)	-4000	-	8	-	Live	Z
Point (lbf)	-2500	-	8	-	Dead	Z
Self Weight (lbf/ft)	3.11	3.11	0	8	Dead	Z

NOTES

PASS

DATE:	3/3/2021	COMPANY:	L120 Engineering & Design, LLC
VITRUVIUS BUILD:	StruCalc	DESIGNED BY:	Mans Thurfjell
CUSTOMER:		REVIEWED BY:	Mans Thurfjell
PROJECT LOCATION:	,		
LEVEL:	Roof	LOADING:	ASD
LOCATION:	(4) 2x4 (Unbraced)	CODE:	2018 International Building Code
TYPE:	COLUMN	NDS:	2018 NDS
MATERIAL:	SOLID SAWN		

Hem-Fir	No. 2	(4) 1.5 X 3.5	DRY		

COLUMN PROPERTIES

Start (ft): 0 End (ft): 8 Member Slope: 0/12 Actual Length (ft): 8

Area	Ix	Iy	BSW	Lams	G	Kcr
(in^2)	(in^4)	(in^4)	(lbf/ft)			Creep Factor
21	21.44	3.94	4.14	4	0.43	1

STRENGTH PROPERTIES

	Fb (psi)	Ft (psi)	Fv (psi)	Fc (psi)	Fc⊥(psi)	E (psi) x10^3	Emin (psi) x10^3
Base Values	850	525	150	1300	405	1300	470
Adjusted Values	1275	788	150	1495	405	1300	470
C_M	1	1	1	1	1	1	1
C_T	1	1	1	1	1	1	1
C_i	1	1	1	1	1	1	1
C_F	1.5	1.5	1	1.15	1	1	1

Bending Adjustment Factors C_{fu} = 1 C_r = 1

COLUMN DATA

		Unbraced Length (ft)		Column End					
Span	Length (ft)	X	Y	Offset	CP	Ke(X Axis)	Ke(Y Axis)	KeL/d (X Axis)	KeL/d (Y Axis)
1	8	8	8	0	0.32	1.00	1.00	27.43	16

PASS-FAIL

	PASS/FAIL	MAGNITUDE	STRENGTH	LOCATION (ft)	LOAD COMBO	DURATION FACTOR CD
Deflection (in)	**PASS (92.7%)**	0.019 (=L/4975)	0.267 (=L/360)	8	L	
Compressive Stress (psi)	**PASS (3.5%)**	454.0	470.3	0	D+L	1

REACTIONS

Units for V: lbf Units for M: lbf-ft

Z axis	DEAD	LIVE	LIVE ROOF	SNOW	WIND +	WIND -	SEISMIC +	SEISMIC -	ICE	RAIN	EARTH
A	4033	5500	0	0	0	0	0	0	0	0	0
B	0	0	0	0	0	0	0	0	0	0	0

Reaction Location

A B

LOAD LIST

Type	Left Magnitude	Right Magnitude	Load Start (ft)	Load End (ft)	Load Type	Direction
Point (lbf)	-4000	-	8	-	Dead	Z
Point (lbf)	-5500	-	8	-	Live	Z
Self Weight (lbf/ft)	4.14	4.14	0	8	Dead	Z

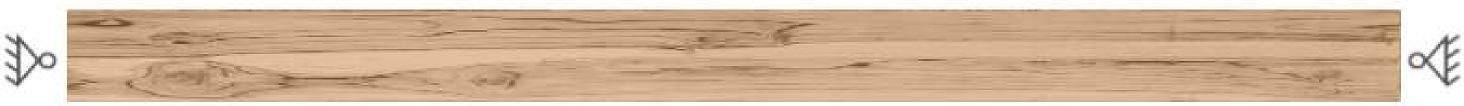
NOTES

PASS

DATE:	10/9/2020	COMPANY:	L120 Engineering & Design, LLC
VITRUVIUS BUILD:	StruCalc	DESIGNED BY:	Mans Thurfjell
CUSTOMER:		REVIEWED BY:	Mans Thurfjell
PROJECT LOCATION:	,		
LEVEL:	Main Floor	LOADING:	ASD
LOCATION:	2x6 stud	CODE:	2018 International Building Code
TYPE:	COLUMN	NDS:	2018 NDS
MATERIAL:	SOLID SAWN		

Hem-Fir	No. 2	(1) 1.5 X 5.5	DRY		

2x6 stud DIAGRAM



COLUMN PROPERTIES

Start (ft): 0 End (ft): 9 Member Slope: 0/12 Actual Length (ft): 9

Area	Ix	Iy	BSW	Lams	G	Kcr
(in^2)	(in^4)	(in^4)	(lbf/ft)			Creep Factor
8.25	20.8	1.55	1.63	1	0.43	1

STRENGTH PROPERTIES

	Fb (psi)	Ft (psi)	Fv (psi)	Fc (psi)	Fc⊥(psi)	E (psi) $x10^3$	Emin (psi) $x10^3$
Base Values	850	525	150	1300	405	1300	470
Adjusted Values	1105	682	150	1430	405	1300	470
C_M	1	1	1	1	1	1	1
C_T	1	1	1	1	1	1	1
C_i	1	1	1	1	1	1	1
C_F	1.3	1.3	1	1.1	1	1	1

Bending Adjustment Factors C_{fu} = 1 C_r = 1

COLUMN DATA

		Unbraced Length (ft)		Column End					
Span	Length (ft)	X	Y	Offset	CP	Ke(X Axis)	Ke(Y Axis)	KeL/d (X Axis)	KeL/d (Y Axis)
1	9	9	2	0	0.56	1.00	1.00	19.64	16

PASS-FAIL

	PASS/FAIL	MAGNITUDE	STRENGTH	LOCATION (ft)	LOAD COMBO	DURATION FACTOR CD
Deflection (in)	**PASS (88.3%)**	0.035 (=L/3068)	0.300 (=L/360)	9	L	
Compressive Stress (psi)	**PASS (1.2%)**	789.7	799.3	0	D+L	1

REACTIONS

V-(lbf) M-(lbf-ft)

Z axis	DEAD	LIVE	LIVE ROOF	SNOW	WIND +	WIND -	SEISMIC +	SEISMIC -	ICE	RAIN	EARTH
A	3015	3500	0	0	0	0	0	0	0	0	0
B	0	0	0	0	0	0	0	0	0	0	0

Reaction Location

A B

LOAD LIST

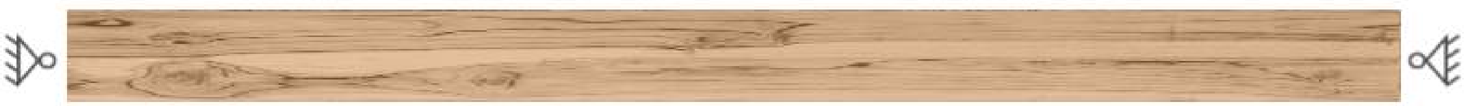
Type	Left Magnitude	Right Magnitude	Load Start (ft)	Load End (ft)	Load Type	Direction
Point (lbf)	-3500	-	9	-	Live	Z
Point (lbf)	-3000	-	9	-	Dead	Z
Self Weight (lbf/ft)	1.63	1.63	0	9	Dead	Z

NOTES

PASS

DATE:	3/3/2021	COMPANY:	L120 Engineering & Design, LLC
VITRUVIUS BUILD:	StruCalc	DESIGNED BY:	Mans Thurfjell
CUSTOMER:		REVIEWED BY:	Mans Thurfjell
PROJECT LOCATION:	,		
LEVEL:	Roof	LOADING:	ASD
LOCATION:	(2) 2x6 (Unbraced)	CODE:	2018 International Building Code
TYPE:	COLUMN	NDS:	2018 NDS
MATERIAL:	SOLID SAWN		

Hem-Fir	No. 2	(2) 1.5 X 5.5	DRY		



COLUMN PROPERTIES

Start (ft): 0   End (ft): 8   Member Slope: 0/12   Actual Length (ft): 8

Area	Ix	Iy	BSW	Lams	G	Kcr
(in^2)	(in^4)	(in^4)	(lbf/ft)			Creep Factor
16.5	41.59	3.09	3.26	2	0.43	1

STRENGTH PROPERTIES

	Fb (psi)	Ft (psi)	Fv (psi)	Fc (psi)	Fc⊥(psi)	E (psi) x10^3	Emin (psi) x10^3
Base Values	850	525	150	1300	405	1300	470
Adjusted Values	1105	682	150	1430	405	1300	470
C_M	1	1	1	1	1	1	1
C_T	1	1	1	1	1	1	1
C_i	1	1	1	1	1	1	1
C_F	1.3	1.3	1	1.1	1	1	1

Bending Adjustment Factors C_{fu} = 1 C_r = 1

COLUMN DATA

		Unbraced Length (ft)		Column End					
Span	Length (ft)	X	Y	Offset	CP	Ke(X Axis)	Ke(Y Axis)	KeL/d (X Axis)	KeL/d (Y Axis)
1	8	8	8	0	0.15	1.00	1.00	17.45	32

PASS-FAIL

	PASS/FAIL	MAGNITUDE	STRENGTH	LOCATION (ft)	LOAD COMBO	DURATION FACTOR CD
Deflection (in)	**PASS (96.6%)**	0.009 (=L/10668)	0.267 (=L/360)	8	L	
Compressive Stress (psi)	**PASS (2.2%)**	207.6	212.4	0	D+L	1

REACTIONS

Units for V: lbf   Units for M: lbf-ft

Z axis	DEAD	LIVE	LIVE ROOF	SNOW	WIND +	WIND -	SEISMIC +	SEISMIC -	ICE	RAIN	EARTH
A	1426	2000	0	0	0	0	0	0	0	0	0
B	0	0	0	0	0	0	0	0	0	0	0

Reaction Location

A   B

LOAD LIST

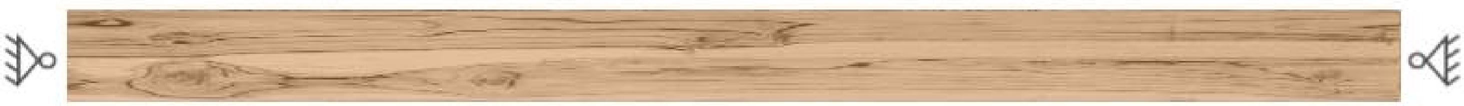
Type	Left Magnitude	Right Magnitude	Load Start (ft)	Load End (ft)	Load Type	Direction
Point (lbf)	-1400	-	8	-	Dead	Z
Point (lbf)	-2000	-	8	-	Live	Z
Self Weight (lbf/ft)	3.26	3.26	0	8	Dead	Z

NOTES

PASS

DATE:	3/3/2021	COMPANY:	L120 Engineering & Design, LLC
VITRUVIUS BUILD:	StruCalc	DESIGNED BY:	Mans Thurfjell
CUSTOMER:		REVIEWED BY:	Mans Thurfjell
PROJECT LOCATION:	,		
LEVEL:	Roof	LOADING:	ASD
LOCATION:	(3) 2x6 (Unbraced)	CODE:	2018 International Building Code
TYPE:	COLUMN	NDS:	2018 NDS
MATERIAL:	SOLID SAWN		

Hem-Fir	No. 2	(3) 1.5 X 5.5	DRY		



COLUMN PROPERTIES

Start (ft): 0 End (ft): 8 Member Slope: 0/12 Actual Length (ft): 8

Area	Ix	Iy	BSW	Lams	G	Kcr
(in^2)	(in^4)	(in^4)	(lbf/ft)			Creep Factor
24.75	62.39	4.64	4.88	3	0.43	1

STRENGTH PROPERTIES

	Fb (psi)	Ft (psi)	Fv (psi)	Fc (psi)	Fc⊥(psi)	E (psi) $x10^3$	Emin (psi) $x10^3$
Base Values	850	525	150	1300	405	1300	470
Adjusted Values	1105	682	150	1430	405	1300	470
C_M	1	1	1	1	1	1	1
C_T	1	1	1	1	1	1	1
C_i	1	1	1	1	1	1	1
C_F	1.3	1.3	1	1.1	1	1	1

Bending Adjustment Factors C_{fu} = 1 C_r = 1

COLUMN DATA

		Unbraced Length (ft)		Column End					
Span	Length (ft)	X	Y	Offset	CP	Ke(X Axis)	Ke(Y Axis)	KeL/d (X Axis)	KeL/d (Y Axis)
1	8	8	8	0	0.30	1.00	1.00	17.45	21.33

PASS-FAIL

	PASS/FAIL	MAGNITUDE	STRENGTH	LOCATION (ft)	LOAD COMBO	DURATION FACTOR CD
Deflection (in)	**PASS (93.3%)**	0.018 (=L/5364)	0.267 (=L/360)	8	L	
Compressive Stress (psi)	**PASS (4.7%)**	405.6	425.6	0	D+L	1

REACTIONS

Units for V: lbf Units for M: lbf-ft

Z axis	DEAD	LIVE	LIVE ROOF	SNOW	WIND +	WIND -	SEISMIC +	SEISMIC -	ICE	RAIN	EARTH
A	4039	6000	0	0	0	0	0	0	0	0	0
B	0	0	0	0	0	0	0	0	0	0	0

Reaction Location

A B

LOAD LIST

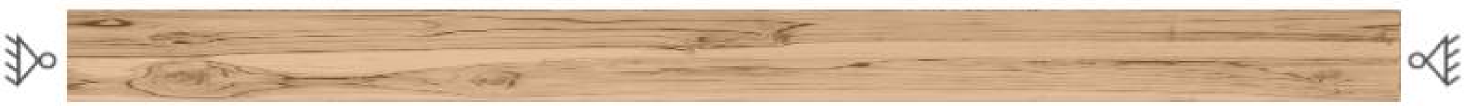
Type	Left Magnitude	Right Magnitude	Load Start (ft)	Load End (ft)	Load Type	Direction
Point (lbf)	-4000	-	8	-	Dead	Z
Point (lbf)	-6000	-	8	-	Live	Z
Self Weight (lbf/ft)	4.88	4.88	0	8	Dead	Z

NOTES

PASS

DATE:	3/3/2021	COMPANY:	L120 Engineering & Design, LLC
VITRUVIUS BUILD:	StruCalc	DESIGNED BY:	Mans Thurfjell
CUSTOMER:		REVIEWED BY:	Mans Thurfjell
PROJECT LOCATION:	,		
LEVEL:	Roof	LOADING:	ASD
LOCATION:	(4) 2x6 (Unbraced)	CODE:	2018 International Building Code
TYPE:	COLUMN	NDS:	2018 NDS
MATERIAL:	SOLID SAWN		

Hem-Fir	No. 2	(4) 1.5 X 5.5	DRY		



COLUMN PROPERTIES

Start (ft): 0 End (ft): 8 Member Slope: 0/12 Actual Length (ft): 8

Area	Ix	Iy	BSW	Lams	G	Kcr
(in^2)	(in^4)	(in^4)	(lbf/ft)			Creep Factor
33	83.19	6.19	6.51	4	0.43	1

STRENGTH PROPERTIES

	Fb (psi)	Ft (psi)	Fv (psi)	Fc (psi)	Fc⊥(psi)	E (psi) x10^3	Emin (psi) x10^3
Base Values	850	525	150	1300	405	1300	470
Adjusted Values	1105	682	150	1430	405	1300	470
C_M	1	1	1	1	1	1	1
C_T	1	1	1	1	1	1	1
C_i	1	1	1	1	1	1	1
C_F	1.3	1.3	1	1.1	1	1	1

Bending Adjustment Factors C_{fu} = 1 C_r = 1

COLUMN DATA

		Unbraced Length (ft)		Column End					
Span	Length (ft)	X	Y	Offset	CP	Ke(X Axis)	Ke(Y Axis)	KeL/d (X Axis)	KeL/d (Y Axis)
1	8	8	8	0	0.43	1.00	1.00	17.45	16

PASS-FAIL

	PASS/FAIL	MAGNITUDE	STRENGTH	LOCATION (ft)	LOAD COMBO	DURATION FACTOR CD
Deflection (in)	**PASS (91.6%)**	0.022 (=L/4286)	0.267 (=L/360)	8	L	
Compressive Stress (psi)	**PASS (10.1%)**	547.0	608.6	0	D+L	1

REACTIONS

Units for V: lbf Units for M: lbf-ft

Z axis	DEAD	LIVE	LIVE ROOF	SNOW	WIND +	WIND -	SEISMIC +	SEISMIC -	ICE	RAIN	EARTH
A	8052	10000	0	0	0	0	0	0	0	0	0
B	0	0	0	0	0	0	0	0	0	0	0

Reaction Location

A B

LOAD LIST

Type	Left Magnitude	Right Magnitude	Load Start (ft)	Load End (ft)	Load Type	Direction
Point (lbf)	-8000	-	8	-	Dead	Z
Point (lbf)	-10000	-	8	-	Live	Z
Self Weight (lbf/ft)	6.51	6.51	0	8	Dead	Z

NOTES