LNL Builds
317 4th Street
Kirkland, Washington 98003

May 18, 2024

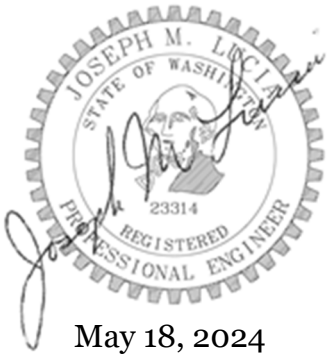
Attention: **Vann Lanz**

Reference: Lanz Residence
Soldier Pile & Timber Lagging Shoring Wall Design

Vann

Attached are the calculations and plans required for the soldier pile shoring wall.

Please contact me with any questions.



May 18, 2024
Joseph M. Lucia, PE

SHORING DESIGN

Geotechnical Design Criteria

Soldier Pile Wall Design

Soldier Pile Facia Wall Design

Soldier Pile Wall Plans

In our opinion, and consistent with the depiction on the referenced liquefaction susceptibility map, site susceptibility to liquefaction may be considered very low. The absence of a uniformly established shallow groundwater table and the relatively dense, fine-grained characteristics of the native soil were the primary bases for this opinion.

Retaining Walls

New retaining walls must be designed to resist earth pressures and applicable surcharge loads. The following parameters may be used for retaining wall design:

- Active earth pressure (unrestrained condition) — 42 pcf
- At-rest earth pressure (restrained condition) — 62 pcf
- Traffic surcharge (passenger vehicles) — 70 psf (rectangular distribution)
- Passive earth pressure — 200 pcf (level surface for at least 10 feet)
- Coefficient of friction — 0.40
- Seismic surcharge — 8H psf*

* *Where H equals the retained height (in feet).*

The passive earth pressure and coefficient of friction values include a safety factor of 1.5. Additional surcharge loading from adjacent foundations, sloped backfill, or other loads should be included in the retaining wall design.

Retaining walls should be backfilled with free-draining material that extends along the height of the wall and a distance of at least 18 inches behind the wall. The upper 12 inches of the wall backfill may consist of a less permeable soil, if desired.

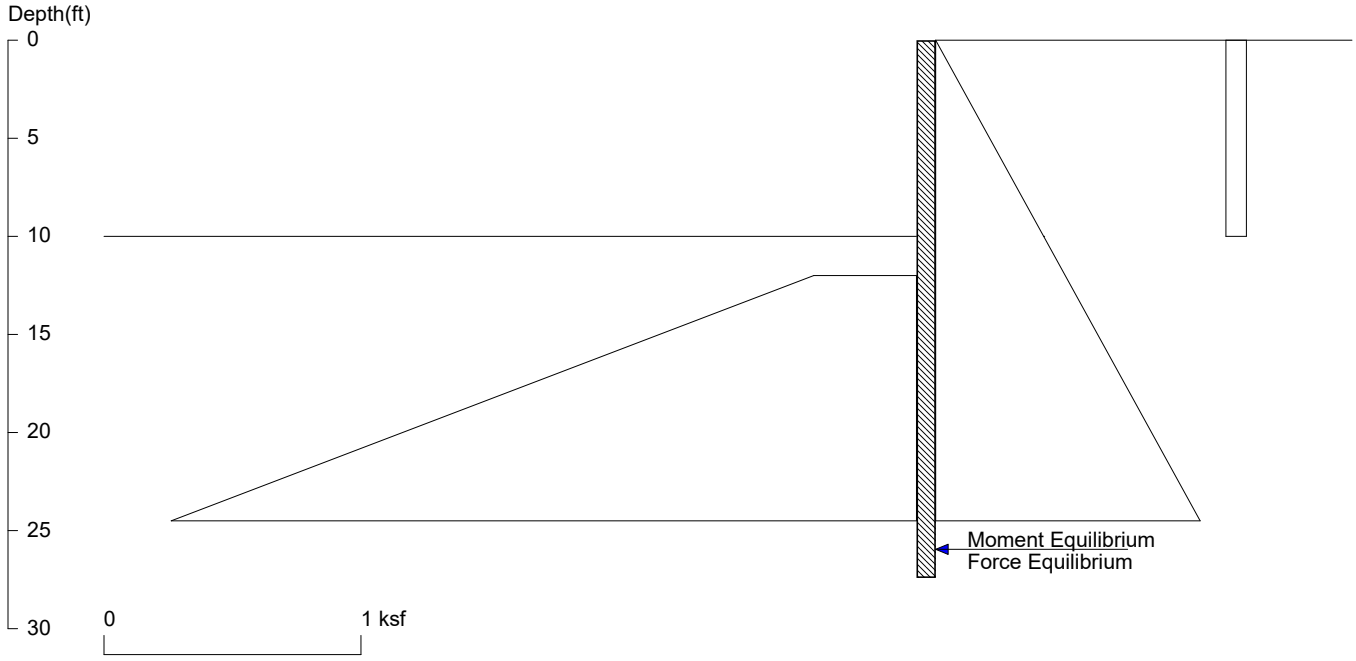
Drainage should be provided behind retaining walls such that hydrostatic pressures do not develop. If drainage is not provided, hydrostatic pressures should be included in the wall design. A perforated drainpipe should be placed along the base of the wall and connected to an approved discharge location. A typical retaining wall drainage detail is provided on Plate 3.

Drainage

Groundwater seepage will likely be encountered within site excavations, particularly during the wet season. Temporary measures to control surface water runoff and groundwater during construction would likely involve passive elements such as interceptor trenches, interceptor swales, and sumps. ESNW should be consulted during preliminary grading to identify areas of seepage and provide recommendations to reduce the potential for seepage-related instability.

Lanz Residence - Mercer Island, WA

Permanent Soldier Pile Shoring Wall



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File: C:\Lucia Engineering 2024\Clients\LNL Builds\4425 Fremont Aver N\10' Tall Soldier Pile Shoring Wall.sh8

Wall Height=10.0 Pile Diameter=2.5 Pile Spacing=6.0 Wall Type: 2. Soldier Pile, Drilled

PILE LENGTH: Min. Embedment=17.41 Min. Pile Length=27.41
MOMENT IN PILE: Max. Moment=175.82 per Pile Spacing=6.0 at Depth=17.86

PILE SELECTION:
Request Min. Section Modulus = 63.9 in3/pile=1047.68 cm3/pile, Fy= 50 ksi = 345 MPa, Fb/Fy=0.66
-> Piles meet Min. Section Requirements: Top Deflection is shown in (in)
W10X60 (1.60) HP12X53 (1.39) W12X50 (1.39) HP13X60 (1.08) HP14X73 (0.75)
W14X48 (1.13) W16X40 (1.05) HP16X88 (0.49) W16X89 (0.42) HP16X101 (0.42)
W16X100 (0.37) HP16X121 (0.35) W18X40 (0.89) HP18X135 (0.25)

DRIVING PRESSURES (ACTIVE, WATER, & SURCHARGE):

Z1	P1	Z2	P2	Slope
0.0	0.0	10.0	0.42	0.042000
10.0	0.42	30.0	1.26	0.042000
*	Seismic	Surch		
0.0	0.08	10.0	0.08	0.000000

PASSIVE PRESSURES:

Z1	P1	Z2	P2	Slope
12.0	0.40	30.0	4.0	0.2000

ACTIVE SPACING:

No.	Z depth	Spacing
1	0.00	6.00
2	10.00	2.50

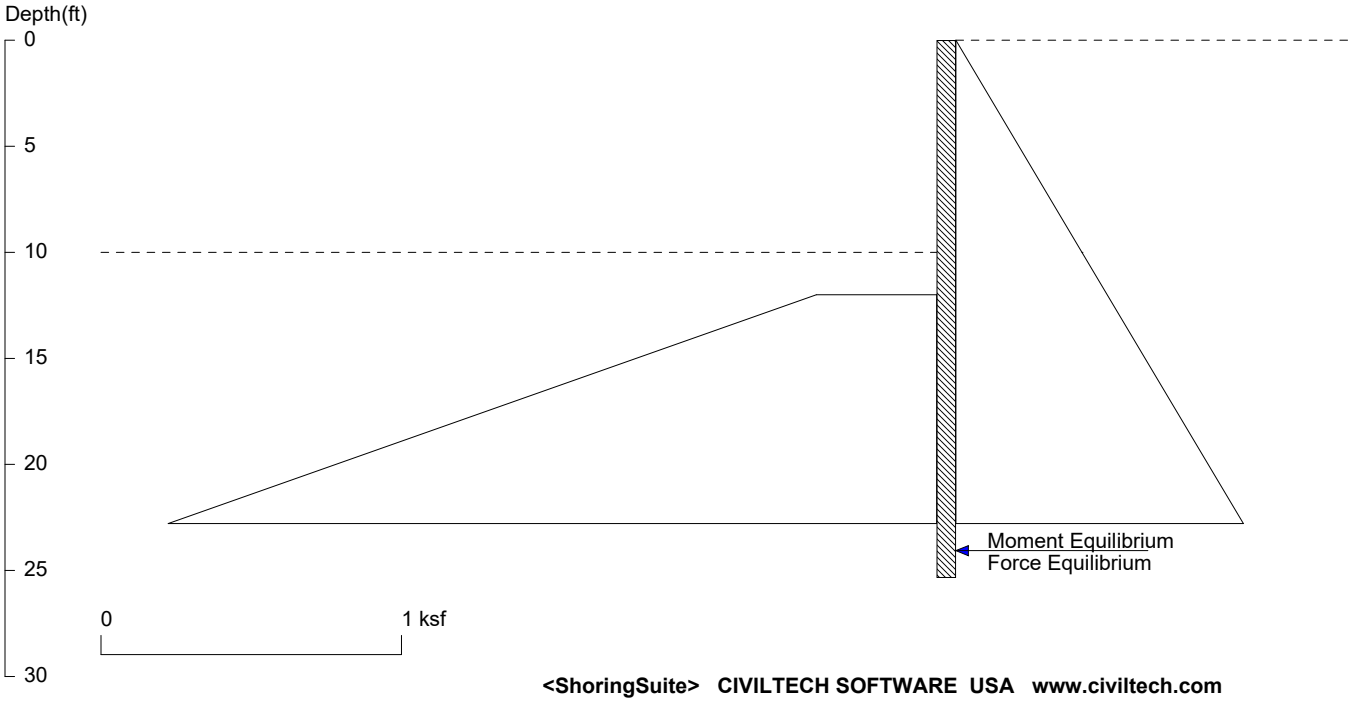
PASSIVE SPACING:

No.	Z depth	Spacing
1	10.00	5.00

UNITS: Width,Spacing,Diameter,Length,and Depth - ft; Force - kip; Moment - kip-ft
Friction,Bearing,and Pressure - ksf; Pres. Slope - kip/ft3; Deflection - in

Lanz Residence - Mercer Island, WA

Permanent Soldier Pile Shoring Wall



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File: C:\Lucia Engineering 2024\Clients\LNL Builds\4425 Fremont Aver N\10' Tall Soldier Pile Shoring Wall.sh8

Wall Height=10.0 Pile Diameter=2.5 Pile Spacing=6.0 Wall Type: 2. Soldier Pile, Drilled

PILE LENGTH: Min. Embedment=15.34 Min. Pile Length=25.34
MOMENT IN PILE: Max. Moment=116.09 per Pile Spacing=6.0 at Depth=17.01

PILE SELECTION:
Request Min. Section Modulus = 42.2 in3/pile=691.79 cm3/pile, Fy= 50 ksi = 345 MPa, Fb/Fy=0.66
W16X45 has Section Modulus = 72.7 in3/pile=1191.33 cm3/pile. It is greater than Min. Requirements!
Top Deflection = 0.53(in) based on E (ksi)=29000.00 and I (in4)/pile=586.0

DRIVING PRESSURES (ACTIVE, WATER, & SURCHARGE):

Z1	P1	Z2	P2	Slope
0.0	0.0	10.0	0.42	0.042000
10.0	0.42	30.0	1.26	0.042000

PASSIVE PRESSURES:

Z1	P1	Z2	P2	Slope
12.0	0.40	30.0	4.0	0.2000

ACTIVE SPACING:

No.	Z depth	Spacing
1	0.00	6.00
2	10.00	2.50

PASSIVE SPACING:

No.	Z depth	Spacing
1	10.00	5.00

UNITS: Width,Spacing,Diameter,Length,and Depth - ft; Force - kip; Moment - kip-ft
Friction,Bearing,and Pressure - ksf; Pres. Slope - kip/ft3; Deflection - in

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SHORING WALL CALCULATION SUMMARY
The leading shoring design and calculation software
Software Copyright by CivilTech Software
www.civiltech.com

****
ShoringSuite Software is developed by CivilTech Software, Bellevue, WA, USA.
The calculation method is based on the following references:

1. FHWA 98-011, FHWA-RD-97-130, FHWA SA 96-069, FHWA-IF-99-015
2. STEEL SHEET PILING DESIGN MANUAL by Pile Buck Inc., 1987
3. DESIGN MANUAL DM-7 (NAVFAC), Department of the Navy, May 1982
4. TRENCHING AND SHORING MANUAL Revision 12, California Department of Transportation, January 2000
6. EARTH SUPPORT SYSTEM & RETAINING STRUCTURES, Pile Buck Inc. 2002
5. DESIGN OF SHEET PILE WALLS, EM 1110-2-2504, U.S. Army Corps of Engineers, 31 March 1994
7. EARTH RETENTION SYSTEMS HANDBOOK, Alan Macnab, McGraw-Hill. 2002
8. Temporary Structures in Construction, Robert T. Ratay (Co-author of Chapter 7: John J. Peirce), McGraw-Hill. 2012
9. AASHTO HB-17, American Association of State and Highway Transportation Officials, 2 September 2002

UNITS: Width/Spacing/Diameter/Length/Depth - ft, Force - kip, Moment - kip-ft, Friction/Bearing/Pressure - ksf, Pres. Slope - kip/ft3, Deflection - in

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Title: Lanz Residence - Mercer Island, WA
Subtitle: Permanent Soldier Pile Shoring Wall

**********INPUT DATA**********
Wall Type: 2. Soldier Pile, Drilled
Wall Height: 10.00
Pile Diameter: 2.50
Pile Spacing: 6.00
Factor of Safety (F.S.): 1.00
Lateral Support Type (Braces): 1. No
Top Brace Increase (Multi-Bracing): Add 15%*
Embedment Option: 1. Yes
Friction at Pile Tip: No
Pile Properties:
Steel Strength, Fy: 50 ksi = 345 MPa
Allowable Fb/Fy: 0.66
Elastic Module, E: 29000.00
Moment of Inertia, I: 341
User Input Pile: W14X82

* DRIVING PRESSURE (ACTIVE, WATER, & SURCHARGE) *

No.	Z1 top	Top Pres.	Z2 bottom	Bottom Pres.	Slope
1	0.0	0.0	10.0	0.42	0.042000
2	10.0	0.42	30.0	1.26	0.042000
3	*	Seismic	Surch		
4	0.0	0.08	10.0	0.08	0.000000

* PASSIVE PRESSURE *

No.	Z1 top	Top Pres.	Z2 bottom	Bottom Pres.	Slope
1	12.0	0.40	30.0	4.0	0.2000

* ACTIVE SPACE *

No.	Z depth	Spacing
1	0.00	6.00
2	10.00	2.50

* PASSIVE SPACE *

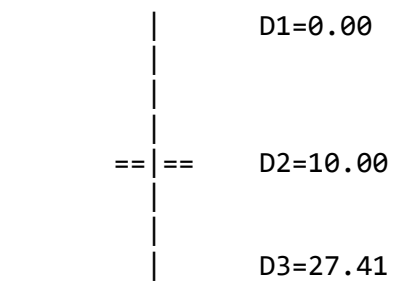
No.	Z depth	Spacing
1	10.00	5.00

*For Tieback: Input1 = Diameter; Input2 = Bond Strength
*For Plate: Input1 = Diameter; Input2 = Allowable Pressure
*For Deadman: Input1 = Horz. Width; Input2 = Passive Pressure;
*For Sheet Pile Anchor: Input1 = Horz. Width; Input2 = Passive Slope;

**********CALCULATION**********

The calculated moment and shear are per pile spacing. Sheet piles are per one foot or meter; Soldier piles are per pile.

Top Pressures start at depth = 0.00



D1 - TOP DEPTH
D2 - EXCAVATION BASE
D3 - PILE TIP

MOMENT equilibrium AT DEPTH=24.50 WITH EMBEDMENT OF 14.50
FORCE equilibrium AT DEPTH=27.41 WITH EMBEDMENT OF 17.41

The program calculates an embedment for moment equilibrium, then increase the embedment by 1.2

**********RESULTS**********

* EMBEDMENT Notes *
Based on USS Design Manual, first calculate embedment for moment equilibrium, then increased the embedment to get the design depth.
The embedment for moment equilibrium is 14.50
The program calculates an embedment for moment equilibrium, then increase the embedment by 1.2
The total desigh embedment is 17.41

Embedment Information:
If 20% increased, the total design embedment is 17.41
If 30% increased, the total design embedment is 18.86
If 40% increased, the total design embedment is 20.31
If 50% increased, the total design embedment is 21.76

* MOMENT IN PILE (per pile spacing)*
Pile Spacing: sheet piles are one foot or one meter; soldier piles are one pile.
Overall Maximum Moment = 175.82 at 17.86
Maximum Shear = 59.25
Moment and Shear are per pile spacing: 6.0 foot or meter

* VERTICAL LOADING *
Vertical Loading from Braces = 0.00
Vertical Loading from External Load = 0.00
Total Vertical Loading = 0.00

**********SOLDIER PILE SELECTION**********

Request Min. Section Modulus = 63.93 in3/pile = 1047.68 cm3/pile, Fy= 50 ksi = 345 MPa, Fb/Fy=0.66
The pile selection is based on the magnitude of the moment only. Axial force is neglected.

W10X60

(English Units):
Area= 17.6 in. Depth= 10.2 in. Width= 10.1 in. Height= 10 in.
Flange thickness= 0.68 in. Web thickness= 0.42 in.
Ix= 341 in4/pile Sx= 66.7 in3/pile Iy= 116 in4/pile Sy= 23 in3/pile
(Metric Units):
Ix= 141.92 x100cm4/pile Sx= 1093.01 cm3/pile Iy= 48.28 x100cm4/pile Sy= 376.90 cm3/pile
Top deflection = 1.598(in)
HP12X53
(English Units):
Area= 15.5 in. Depth= 11.8 in. Width= 12 in. Height= 12 in.
Flange thickness= 0.435 in. Web thickness= 0.435 in.
Ix= 393 in4/pile Sx= 66.7 in3/pile Iy= 127 in4/pile Sy= 21.1 in3/pile
(Metric Units):
Ix= 163.57 x100cm4/pile Sx= 1093.01 cm3/pile Iy= 52.86 x100cm4/pile Sy= 345.77 cm3/pile
Top deflection = 1.387(in)
W12X50
(English Units):
Area= 14.6 in. Depth= 12.2 in. Width= 8.08 in. Height= 12 in.
Flange thickness= 0.64 in. Web thickness= 0.37 in.
Ix= 391 in4/pile Sx= 64.2 in3/pile Iy= 56.3 in4/pile Sy= 13.9 in3/pile
(Metric Units):
Ix= 162.73 x100cm4/pile Sx= 1052.05 cm3/pile Iy= 23.43 x100cm4/pile Sy= 227.78 cm3/pile
Top deflection = 1.394(in)
HP13X60
(English Units):
Area= 17.5 in. Depth= 12.54 in. Width= 12.9 in. Height= 13 in.
Flange thickness= 0.46 in. Web thickness= 0.46 in.
Ix= 503 in4/pile Sx= 80.3 in3/pile Iy= 165 in4/pile Sy= 25.5 in3/pile
(Metric Units):
Ix= 209.35 x100cm4/pile Sx= 1315.88 cm3/pile Iy= 68.67 x100cm4/pile Sy= 417.87 cm3/pile
Top deflection = 1.083(in)
HP14X73
(English Units):
Area= 21.4 in. Depth= 13.6 in. Width= 14.6 in. Height= 14 in.
Flange thickness= 0.505 in. Web thickness= 0.505 in.
Ix= 729 in4/pile Sx= 107 in3/pile Iy= 261 in4/pile Sy= 35.8 in3/pile
(Metric Units):
Ix= 303.41 x100cm4/pile Sx= 1753.41 cm3/pile Iy= 108.63 x100cm4/pile Sy= 586.65 cm3/pile
Top deflection = 0.748(in)
W14X48
(English Units):
Area= 14.1 in. Depth= 13.8 in. Width= 8.03 in. Height= 14 in.
Flange thickness= 0.595 in. Web thickness= 0.34 in.
Ix= 484 in4/pile Sx= 70.2 in3/pile Iy= 51.4 in4/pile Sy= 12.8 in3/pile
(Metric Units):

Ix= 201.44 x100cm4/pile Sx= 1150.37 cm3/pile Iy= 21.39 x100cm4/pile Sy= 209.75 cm3/pile
Top deflection = 1.126(in)
W16X40
(English Units):
Area= 11.8 in. Depth= 16 in. Width= 7 in. Height= 16 in.
Flange thickness= 0.505 in. Web thickness= 0.305 in.
Ix= 518 in4/pile Sx= 64.7 in3/pile Iy= 28.9 in4/pile Sy= 8.25 in3/pile
(Metric Units):
Ix= 215.59 x100cm4/pile Sx= 1060.24 cm3/pile Iy= 12.03 x100cm4/pile Sy= 135.19 cm3/pile
Top deflection = 1.052(in)
HP16X88
(English Units):
Area= 25.8 in. Depth= 15.33 in. Width= 15.665 in. Height= 16 in.
Flange thickness= 0.54 in. Web thickness= 0.54 in.
Ix= 1112 in4/pile Sx= 145 in3/pile Iy= 347 in4/pile Sy= 44 in3/pile
(Metric Units):
Ix= 462.81 x100cm4/pile Sx= 2376.12 cm3/pile Iy= 144.42 x100cm4/pile Sy= 721.03 cm3/pile
Top deflection = 0.490(in)
W16X89
(English Units):
Area= 26.2 in. Depth= 16.8 in. Width= 10.4 in. Height= 16 in.
Flange thickness= 0.875 in. Web thickness= 0.525 in.
Ix= 1300 in4/pile Sx= 155 in3/pile Iy= 163 in4/pile Sy= 31.4 in3/pile
(Metric Units):
Ix= 541.06 x100cm4/pile Sx= 2539.99 cm3/pile Iy= 67.84 x100cm4/pile Sy= 514.55 cm3/pile
Top deflection = 0.419(in)
HP16X101
(English Units):
Area= 29.8 in. Depth= 15.5 in. Width= 15.75 in. Height= 16 in.
Flange thickness= 0.625 in. Web thickness= 0.625 in.
Ix= 1297 in4/pile Sx= 167 in3/pile Iy= 408 in4/pile Sy= 52.1 in3/pile
(Metric Units):
Ix= 539.81 x100cm4/pile Sx= 2736.63 cm3/pile Iy= 169.81 x100cm4/pile Sy= 853.76 cm3/pile
Top deflection = 0.420(in)
W16X100
(English Units):
Area= 29.5 in. Depth= 17 in. Width= 10.4 in. Height= 16 in.
Flange thickness= 0.985 in. Web thickness= 0.585 in.
Ix= 1490 in4/pile Sx= 175 in3/pile Iy= 186 in4/pile Sy= 35.7 in3/pile
(Metric Units):
Ix= 620.14 x100cm4/pile Sx= 2867.73 cm3/pile Iy= 77.41 x100cm4/pile Sy= 585.02 cm3/pile
Top deflection = 0.366(in)
HP16X121
(English Units):

Area= 35.7 in. Depth= 15.75 in. Width= 15.875 in. Height= 16 in.
Flange thickness= 0.75 in. Web thickness= 0.75 in.
Ix= 1578 in4/pile Sx= 200 in3/pile Iy= 501 in4/pile Sy= 63.1 in3/pile
(Metric Units):
Ix= 656.76 x100cm4/pile Sx= 3277.40 cm3/pile Iy= 208.52 x100cm4/pile Sy= 1034.02 cm3/pile
Top deflection = 0.345(in)
W18X40
(English Units):
Area= 11.8 in. Depth= 17.9 in. Width= 6.02 in. Height= 18 in.
Flange thickness= 0.525 in. Web thickness= 0.315 in.
Ix= 612 in4/pile Sx= 68.4 in3/pile Iy= 19.1 in4/pile Sy= 6.35 in3/pile
(Metric Units):
Ix= 254.71 x100cm4/pile Sx= 1120.87 cm3/pile Iy= 7.95 x100cm4/pile Sy= 104.06 cm3/pile
Top deflection = 0.890(in)
HP18X135
(English Units):
Area= 39.8 in. Depth= 17.5 in. Width= 17.75 in. Height= 18 in.
Flange thickness= 0.75 in. Web thickness= 0.75 in.
Ix= 2196 in4/pile Sx= 251 in3/pile Iy= 700 in4/pile Sy= 78.8 in3/pile
(Metric Units):
Ix= 913.98 x100cm4/pile Sx= 4113.14 cm3/pile Iy= 291.34 x100cm4/pile Sy= 1291.30 cm3/pile
Top deflection = 0.248(in)

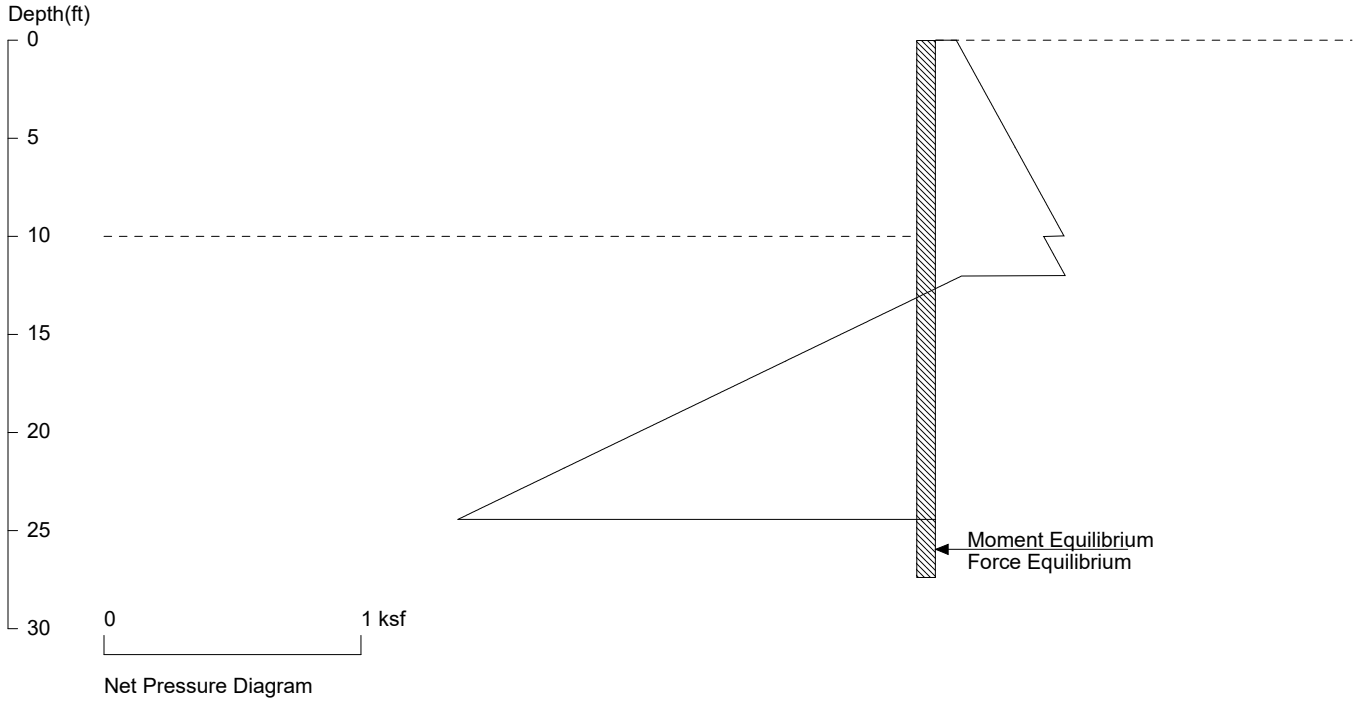
********** LAGGING SIZE ESTIMATION **********
Max. Pressure above base = 0.50
Piles are more rigid than timber lagging, due to arching, only portion of pressures are acting to lagging, 30-50% loading is suggested.
If 50% loading is used for lagging design, Design Pressure = 0.25
Pile Spacing =6.0, Max. Moment in lagging = 1.12
For 4"x12" Timber, Section Modules S=23.47 in3. The request allowable bending strength, fb=M/S=0.57
For 6"x12" Timber, Section Modules S=57.98 in3. The request allowable bending strength, fb=M/S=0.23

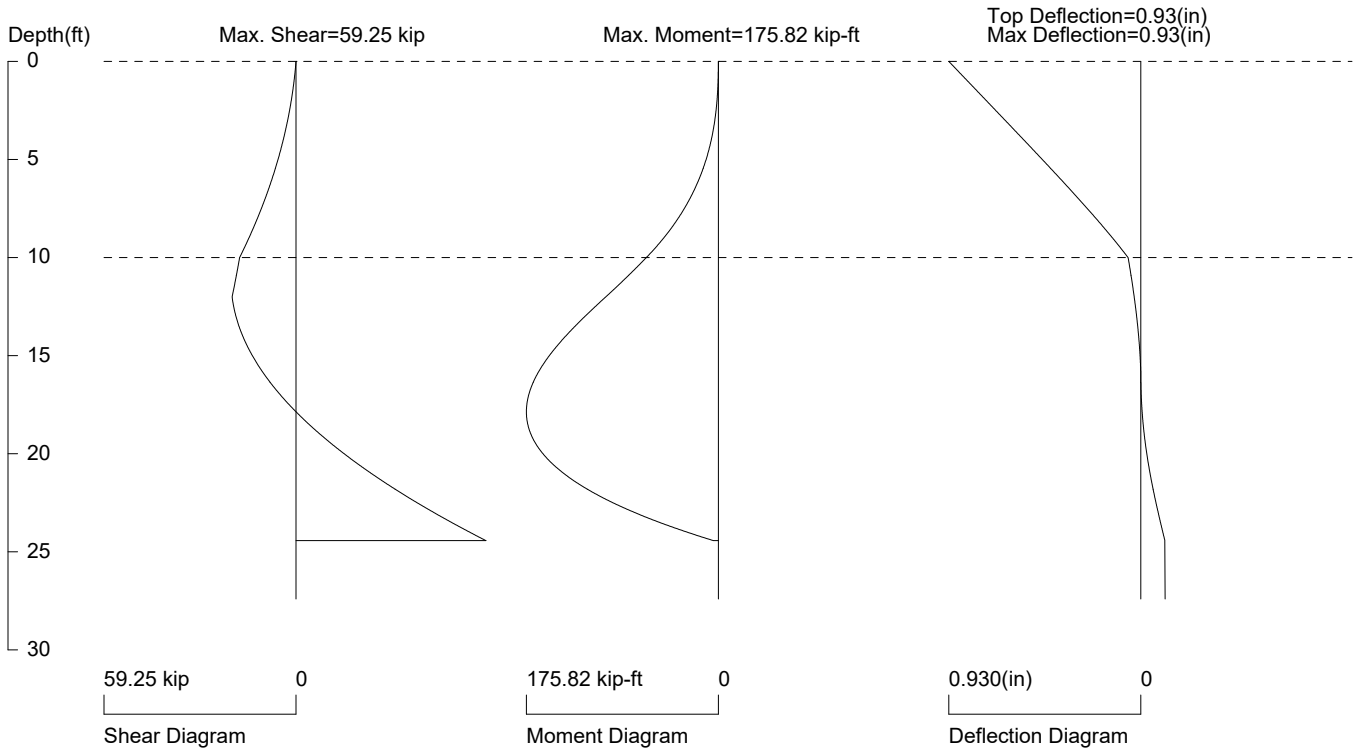
If 30% loading is used for lagging design, Design Pressure = 0.15
Pile Spacing =6.0, Max. Moment in lagging = 0.67
For 4"x12" Timber, Section Modules S=23.47 in3. The request allowable bending strength, fb=M/S=0.34
For 6"x12" Timber, Section Modules S=57.98 in3. The request allowable bending strength, fb=M/S=0.14

Unit: Pressure: ksf, Spacing: ft, Moment: kip-ft, Bending Strength, fb: ksi

Lanz Residence - Mercer Island, WA

Permanent Soldier Pile Shoring Wall





PRESSURE, SHEAR, MOMENT, AND DEFLECTION DIAGRAMS

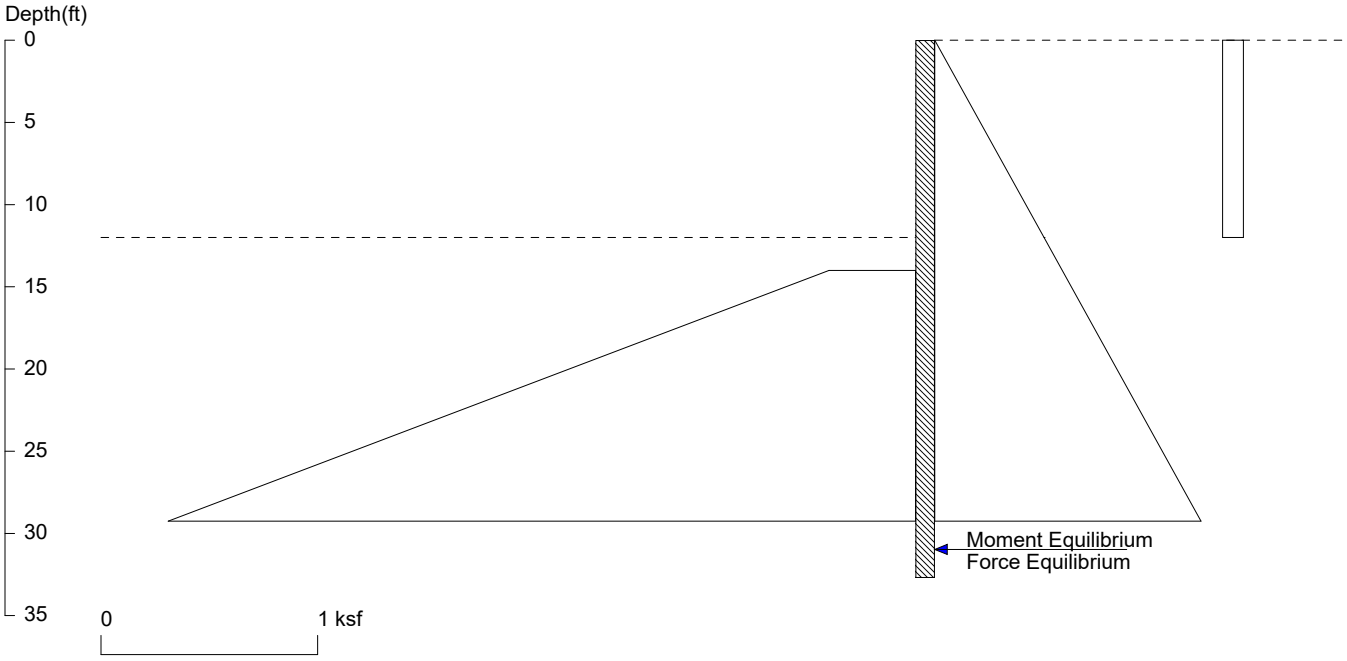
Based on pile spacing: 6.0 foot or meter

User Input Pile, W16X45: E (ksi)=29000.0, I (in4)/pile=586.0

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Lanz Residence - Mercer Island, WA

Permanent Soldier Pile Shoring Wall



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File: C:\Lucia Engineering 2024\Clients\LNL Builds\4425 Fremont Aver N\12' Tall Soldier Pile Shoring Wall.sh8

Wall Height=12.0 Pile Diameter=2.5 Pile Spacing=6.0 Wall Type: 2. Soldier Pile, Drilled

PILE LENGTH: Min. Embedment=20.70 Min. Pile Length=32.70
MOMENT IN PILE: Max. Moment=297.51 per Pile Spacing=6.0 at Depth=21.31

PILE SELECTION:
Request Min. Section Modulus = 108.2 in3/pile=1772.82 cm3/pile, Fy= 50 ksi = 345 MPa, Fb/Fy=0.66
-> Piles meet Min. Section Requirements: Top Deflection is shown in (in)
W10X100 (2.12) W12X87 (1.78) HP13X87 (1.75) HP14X89 (1.46) W14X74 (1.66)
W16X67 (1.38) HP16X88 (1.19) W16X89 (1.01) HP16X101 (1.02) W16X100 (0.88)
HP16X121 (0.84) W18X65 (1.23) HP18X135 (0.60) W18X130 (0.54)

DRIVING PRESSURES (ACTIVE, WATER, & SURCHARGE):

Z1	P1	Z2	P2	Slope
0.0	0.0	12.0	0.504	0.042000
12.0	0.504	40.0	1.68	0.042000
*	Seism	Surch		
0.0	0.096	12.0	0.096	0.000000

PASSIVE PRESSURES:

Z1	P1	Z2	P2	Slope
14.0	0.40	40.0	5.60	0.2000

ACTIVE SPACING:

No.	Z depth	Spacing
1	0.00	6.00
2	12.00	2.50

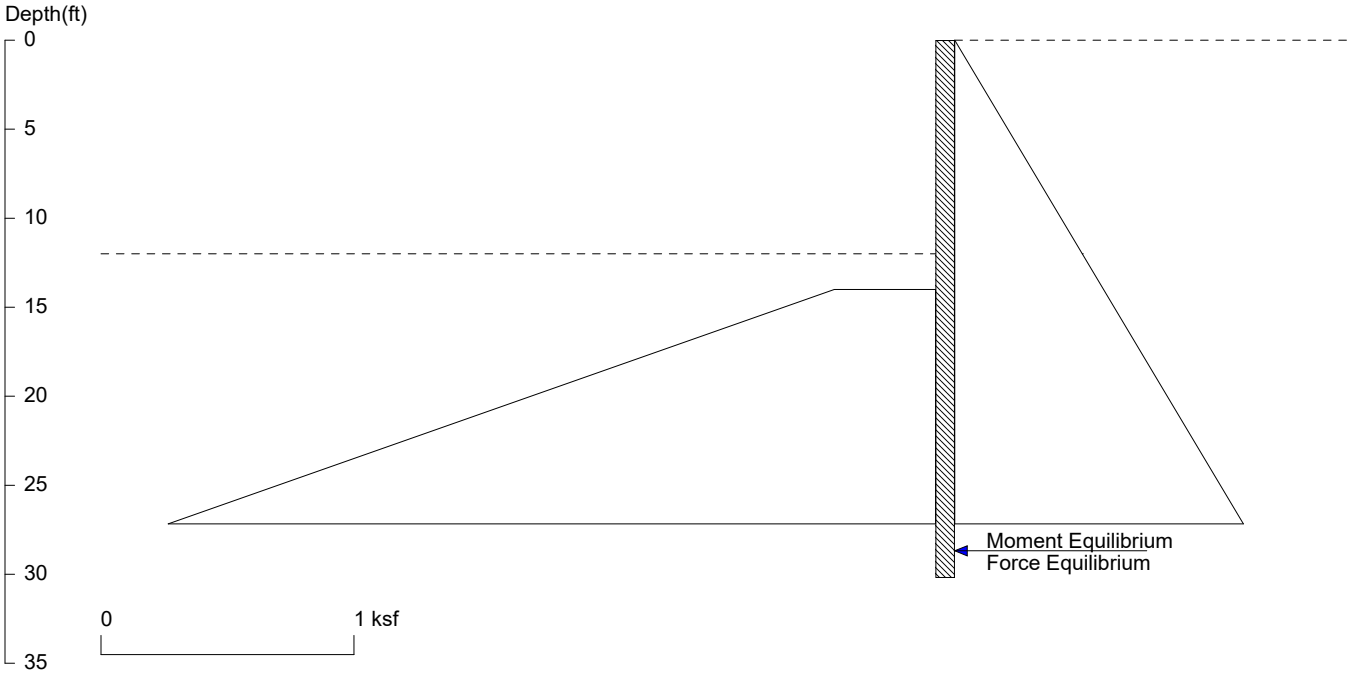
PASSIVE SPACING:

No.	Z depth	Spacing
1	12.00	5.00

UNITS: Width,Spacing,Diameter,Length,and Depth - ft; Force - kip; Moment - kip-ft
Friction,Bearing,and Pressure - ksf; Pres. Slope - kip/ft3; Deflection - in

Lanz Residence - Mercer Island, WA

Permanent Soldier Pile Shoring Wall



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File: C:\Lucia Engineering 2024\Clients\LNL Builds\4425 Fremont Aver N\12' Tall Soldier Pile Shoring Wall - DL.sh8

Wall Height=12.0 Pile Diameter=2.5 Pile Spacing=6.0 Wall Type: 2. Soldier Pile, Drilled

PILE LENGTH: Min. Embedment=18.20 Min. Pile Length=30.20
MOMENT IN PILE: Max. Moment=195.22 per Pile Spacing=6.0 at Depth=20.25

PILE SELECTION:
Request Min. Section Modulus = 71.0 in3/pile=1163.28 cm3/pile, Fy= 50 ksi = 345 MPa, Fb/Fy=0.66
W16X89 has Section Modulus = 155.0 in3/pile=2539.99 cm3/pile. It is greater than Min. Requirements!
Top Deflection = 0.58(in) based on E (ksi)=29000.00 and I (in4)/pile=1300.0

DRIVING PRESSURES (ACTIVE, WATER, & SURCHARGE):

Z1	P1	Z2	P2	Slope
0.0	0.0	12.0	0.504	0.042000
12.0	0.504	40.0	1.68	0.042000

PASSIVE PRESSURES:

Z1	P1	Z2	P2	Slope
14.0	0.40	40.0	5.60	0.2000

ACTIVE SPACING:

No.	Z depth	Spacing
1	0.00	6.00
2	12.00	2.50

PASSIVE SPACING:

No.	Z depth	Spacing
1	12.00	5.00

UNITS: Width,Spacing,Diameter,Length,and Depth - ft; Force - kip; Moment - kip-ft
Friction,Bearing,and Pressure - ksf; Pres. Slope - kip/ft3; Deflection - in

****

SHORING WALL CALCULATION SUMMARY
The leading shoring design and calculation software
Software Copyright by CivilTech Software
www.civiltech.com

****
ShoringSuite Software is developed by CivilTech Software, Bellevue, WA, USA.
The calculation method is based on the following references:

1. FHWA 98-011, FHWA-RD-97-130, FHWA SA 96-069, FHWA-IF-99-015
2. STEEL SHEET PILING DESIGN MANUAL by Pile Buck Inc., 1987
3. DESIGN MANUAL DM-7 (NAVFAC), Department of the Navy, May 1982
4. TRENCHING AND SHORING MANUAL Revision 12, California Department of Transportation, January 2000
6. EARTH SUPPORT SYSTEM & RETAINING STRUCTURES, Pile Buck Inc. 2002
5. DESIGN OF SHEET PILE WALLS, EM 1110-2-2504, U.S. Army Corps of Engineers, 31 March 1994
7. EARTH RETENTION SYSTEMS HANDBOOK, Alan Macnab, McGraw-Hill. 2002
8. Temporary Structures in Construction, Robert T. Ratay (Co-author of Chapter 7: John J. Peirce), McGraw-Hill. 2012
9. AASHTO HB-17, American Association of State and Highway Transportation Officials, 2 September 2002

UNITS: Width/Spacing/Diameter/Length/Depth - ft, Force - kip, Moment - kip-ft, Friction/Bearing/Pressure - ksf, Pres. Slope - kip/ft3, Deflection - in

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Date: 3/4/2024 File: C:\Lucia Engineering 2024\Clients\LNL Builds\4425 Fremont Aver N\12' Tall Soldier Pile Shoring Wall.sh8

Title: Lanz Residence - Mercer Island, WA
Subtitle: Permanent Soldier Pile Shoring Wall

**********INPUT DATA**********
Wall Type: 2. Soldier Pile, Drilled
Wall Height: 12.00
Pile Diameter: 2.50
Pile Spacing: 6.00
Factor of Safety (F.S.): 1.00
Lateral Support Type (Braces): 1. No
Top Brace Increase (Multi-Bracing): Add 15%*
Embedment Option: 1. Yes
Friction at Pile Tip: No
Pile Properties:
Steel Strength, Fy: 50 ksi = 345 MPa
Allowable Fb/Fy: 0.66
Elastic Module, E: 29000.00
Moment of Inertia, I: 623
User Input Pile: W16X45

* DRIVING PRESSURE (ACTIVE, WATER, & SURCHARGE) *

No.	Z1 top	Top Pres.	Z2 bottom	Bottom Pres.	Slope
1	0.0	0.0	12.0	0.504	0.042000
2	12.0	0.504	40.0	1.68	0.042000
3	*	Seism	Surch		
4	0.0	0.096	12.0	0.096	0.000000

* PASSIVE PRESSURE *

No.	Z1 top	Top Pres.	Z2 bottom	Bottom Pres.	Slope
1	14.0	0.40	40.0	5.60	0.2000

* ACTIVE SPACE *

No.	Z depth	Spacing
1	0.00	6.00
2	12.00	2.50

* PASSIVE SPACE *

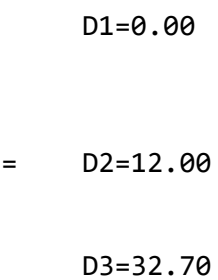
No.	Z depth	Spacing
1	12.00	5.00

*For Tieback: Input1 = Diameter; Input2 = Bond Strength
*For Plate: Input1 = Diameter; Input2 = Allowable Pressure
*For Deadman: Input1 = Horz. Width; Input2 = Passive Pressure;
*For Sheet Pile Anchor: Input1 = Horz. Width; Input2 = Passive Slope;

**********CALCULATION**********

The calculated moment and shear are per pile spacing. Sheet piles are per one foot or meter; Soldier piles are per pile.

Top Pressures start at depth = 0.00



D1 - TOP DEPTH
D2 - EXCAVATION BASE
D3 - PILE TIP

MOMENT equilibrium AT DEPTH=29.25 WITH EMBEDMENT OF 17.25
FORCE equilibrium AT DEPTH=32.70 WITH EMBEDMENT OF 20.70

The program calculates an embedment for moment equilibrium, then increase the embedment by 1.2

**********RESULTS**********

* EMBEDMENT Notes *
Based on USS Design Manual, first calculate embedment for moment equilibrium, then increased the embedment to get the design depth.
The embedment for moment equilibrium is 17.25
The program calculates an embedment for moment equilibrium, then increase the embedment by 1.2
The total desigh embedment is 20.70

Embedment Information:
If 20% increased, the total design embedment is 20.70
If 30% increased, the total design embedment is 22.42
If 40% increased, the total design embedment is 24.15
If 50% increased, the total design embedment is 25.87

* MOMENT IN PILE (per pile spacing)*
Pile Spacing: sheet piles are one foot or one meter; soldier piles are one pile.
Overall Maximum Moment = 297.51 at 21.31
Maximum Shear = 83.95
Moment and Shear are per pile spacing: 6.0 foot or meter

* VERTICAL LOADING *
Vertical Loading from Braces = 0.00
Vertical Loading from External Load = 0.00
Total Vertical Loading = 0.00

**********SOLDIER PILE SELECTION**********

Request Min. Section Modulus = 108.18 in3/pile = 1772.82 cm3/pile, Fy= 50 ksi = 345 MPa, Fb/Fy=0.66
The pile selection is based on the magnitude of the moment only. Axial force is neglected.

W10X100

(English Units):
Area= 29.4 in. Depth= 11.1 in. Width= 10.3 in. Height= 10 in.
Flange thickness= 1.12 in. Web thickness= 0.68 in.
Ix= 623 in4/pile Sx= 112 in3/pile Iy= 207 in4/pile Sy= 40 in3/pile
(Metric Units):
Ix= 259.29 x100cm4/pile Sx= 1835.34 cm3/pile Iy= 86.15 x100cm4/pile Sy= 655.48 cm3/pile
Top deflection = 2.116(in)
W12X87
(English Units):
Area= 25.6 in. Depth= 12.5 in. Width= 12.1 in. Height= 12 in.
Flange thickness= 0.81 in. Web thickness= 0.515 in.
Ix= 740 in4/pile Sx= 118 in3/pile Iy= 241 in4/pile Sy= 39.7 in3/pile
(Metric Units):
Ix= 307.99 x100cm4/pile Sx= 1933.67 cm3/pile Iy= 100.30 x100cm4/pile Sy= 650.56 cm3/pile
Top deflection = 1.782(in)
HP13X87
(English Units):
Area= 25.5 in. Depth= 12.95 in. Width= 13.105 in. Height= 13 in.
Flange thickness= 0.665 in. Web thickness= 0.665 in.
Ix= 755 in4/pile Sx= 117 in3/pile Iy= 250 in4/pile Sy= 38.1 in3/pile
(Metric Units):
Ix= 314.23 x100cm4/pile Sx= 1917.28 cm3/pile Iy= 104.05 x100cm4/pile Sy= 624.34 cm3/pile
Top deflection = 1.746(in)
HP14X89
(English Units):
Area= 26.1 in. Depth= 13.8 in. Width= 14.7 in. Height= 14 in.
Flange thickness= 0.615 in. Web thickness= 0.615 in.
Ix= 904 in4/pile Sx= 131 in3/pile Iy= 326 in4/pile Sy= 44.3 in3/pile
(Metric Units):
Ix= 376.24 x100cm4/pile Sx= 2146.70 cm3/pile Iy= 135.68 x100cm4/pile Sy= 725.94 cm3/pile
Top deflection = 1.459(in)
W14X74
(English Units):
Area= 21.8 in. Depth= 14.2 in. Width= 10.1 in. Height= 14 in.
Flange thickness= 0.785 in. Web thickness= 0.45 in.
Ix= 795 in4/pile Sx= 112 in3/pile Iy= 134 in4/pile Sy= 26.6 in3/pile
(Metric Units):
Ix= 330.88 x100cm4/pile Sx= 1835.34 cm3/pile Iy= 55.77 x100cm4/pile Sy= 435.89 cm3/pile
Top deflection = 1.659(in)
W16X67
(English Units):
Area= 19.7 in. Depth= 16.3 in. Width= 10.2 in. Height= 16 in.
Flange thickness= 0.665 in. Web thickness= 0.395 in.
Ix= 954 in4/pile Sx= 117 in3/pile Iy= 119 in4/pile Sy= 23.2 in3/pile
(Metric Units):

Ix= 397.05 x100cm4/pile Sx= 1917.28 cm3/pile Iy= 49.53 x100cm4/pile Sy= 380.18 cm3/pile
Top deflection = 1.382(in)
HP16X88
(English Units):
Area= 25.8 in. Depth= 15.33 in. Width= 15.665 in. Height= 16 in.
Flange thickness= 0.54 in. Web thickness= 0.54 in.
Ix= 1112 in4/pile Sx= 145 in3/pile Iy= 347 in4/pile Sy= 44 in3/pile
(Metric Units):
Ix= 462.81 x100cm4/pile Sx= 2376.12 cm3/pile Iy= 144.42 x100cm4/pile Sy= 721.03 cm3/pile
Top deflection = 1.186(in)
W16X89
(English Units):
Area= 26.2 in. Depth= 16.8 in. Width= 10.4 in. Height= 16 in.
Flange thickness= 0.875 in. Web thickness= 0.525 in.
Ix= 1300 in4/pile Sx= 155 in3/pile Iy= 163 in4/pile Sy= 31.4 in3/pile
(Metric Units):
Ix= 541.06 x100cm4/pile Sx= 2539.99 cm3/pile Iy= 67.84 x100cm4/pile Sy= 514.55 cm3/pile
Top deflection = 1.014(in)
HP16X101
(English Units):
Area= 29.8 in. Depth= 15.5 in. Width= 15.75 in. Height= 16 in.
Flange thickness= 0.625 in. Web thickness= 0.625 in.
Ix= 1297 in4/pile Sx= 167 in3/pile Iy= 408 in4/pile Sy= 52.1 in3/pile
(Metric Units):
Ix= 539.81 x100cm4/pile Sx= 2736.63 cm3/pile Iy= 169.81 x100cm4/pile Sy= 853.76 cm3/pile
Top deflection = 1.017(in)
W16X100
(English Units):
Area= 29.5 in. Depth= 17 in. Width= 10.4 in. Height= 16 in.
Flange thickness= 0.985 in. Web thickness= 0.585 in.
Ix= 1490 in4/pile Sx= 175 in3/pile Iy= 186 in4/pile Sy= 35.7 in3/pile
(Metric Units):
Ix= 620.14 x100cm4/pile Sx= 2867.73 cm3/pile Iy= 77.41 x100cm4/pile Sy= 585.02 cm3/pile
Top deflection = 0.885(in)
HP16X121
(English Units):
Area= 35.7 in. Depth= 15.75 in. Width= 15.875 in. Height= 16 in.
Flange thickness= 0.75 in. Web thickness= 0.75 in.
Ix= 1578 in4/pile Sx= 200 in3/pile Iy= 501 in4/pile Sy= 63.1 in3/pile
(Metric Units):
Ix= 656.76 x100cm4/pile Sx= 3277.40 cm3/pile Iy= 208.52 x100cm4/pile Sy= 1034.02 cm3/pile
Top deflection = 0.836(in)
W18X65
(English Units):

Area= 19.1 in. Depth= 18.4 in. Width= 7.59 in. Height= 18 in.
Flange thickness= 0.75 in. Web thickness= 0.45 in.
Ix= 1070 in4/pile Sx= 117 in3/pile Iy= 54.8 in4/pile Sy= 14.4 in3/pile
(Metric Units):
Ix= 445.33 x100cm4/pile Sx= 1917.28 cm3/pile Iy= 22.81 x100cm4/pile Sy= 235.97 cm3/pile
Top deflection = 1.232(in)
HP18X135
(English Units):
Area= 39.8 in. Depth= 17.5 in. Width= 17.75 in. Height= 18 in.
Flange thickness= 0.75 in. Web thickness= 0.75 in.
Ix= 2196 in4/pile Sx= 251 in3/pile Iy= 700 in4/pile Sy= 78.8 in3/pile
(Metric Units):
Ix= 913.98 x100cm4/pile Sx= 4113.14 cm3/pile Iy= 291.34 x100cm4/pile Sy= 1291.30 cm3/pile
Top deflection = 0.600(in)
W18X130
(English Units):
Area= 38.2 in. Depth= 19.3 in. Width= 11.2 in. Height= 18 in.
Flange thickness= 1.2 in. Web thickness= 0.67 in.
Ix= 2460 in4/pile Sx= 256 in3/pile Iy= 278 in4/pile Sy= 49.9 in3/pile
(Metric Units):
Ix= 1023.85 x100cm4/pile Sx= 4195.07 cm3/pile Iy= 115.70 x100cm4/pile Sy= 817.71 cm3/pile
Top deflection = 0.536(in)

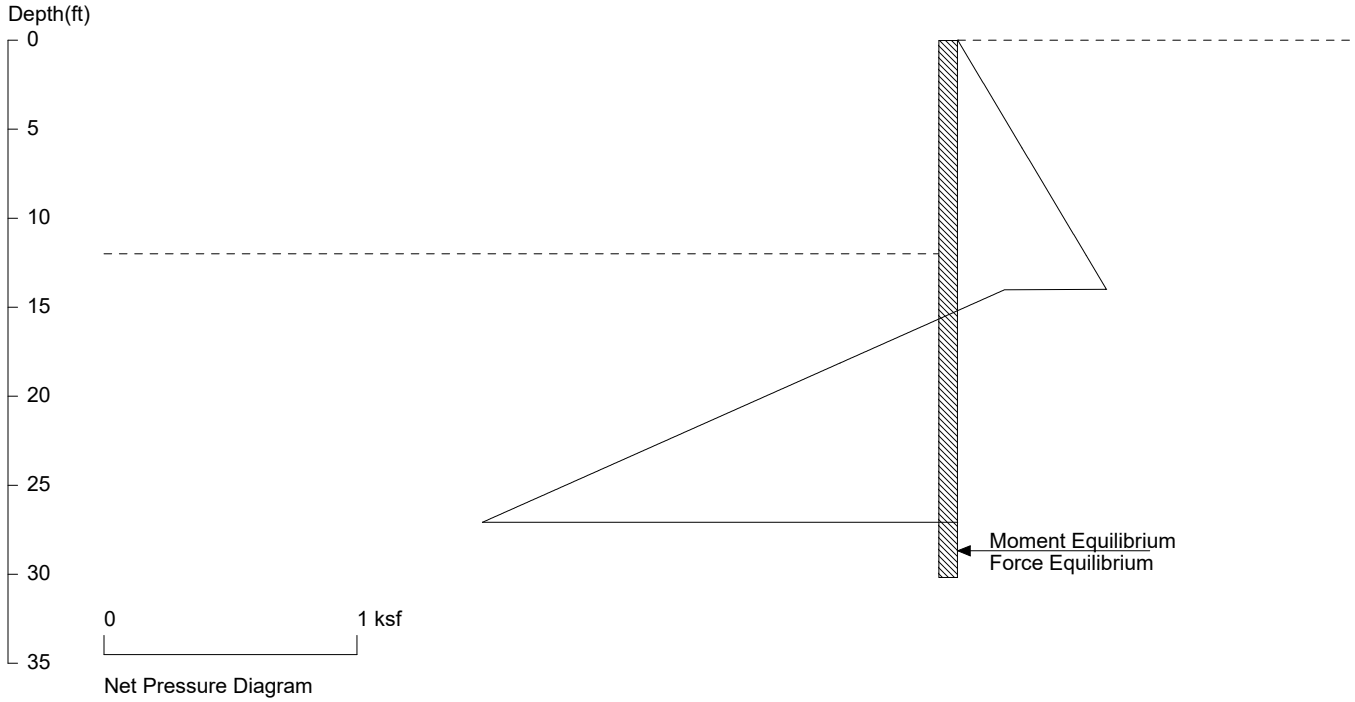
********** LAGGING SIZE ESTIMATION **********
Max. Pressure above base = 0.60
Piles are more rigid than timber lagging, due to arching, only portion of pressures are acting to lagging, 30-50% loading is suggested.
If 50% loading is used for lagging design, Design Pressure = 0.30
Pile Spacing =6.0, Max. Moment in lagging = 1.35
For 4"x12" Timber, Section Modules S=23.47 in3. The request allowable bending strength, fb=M/S=0.69
For 6"x12" Timber, Section Modules S=57.98 in3. The request allowable bending strength, fb=M/S=0.28

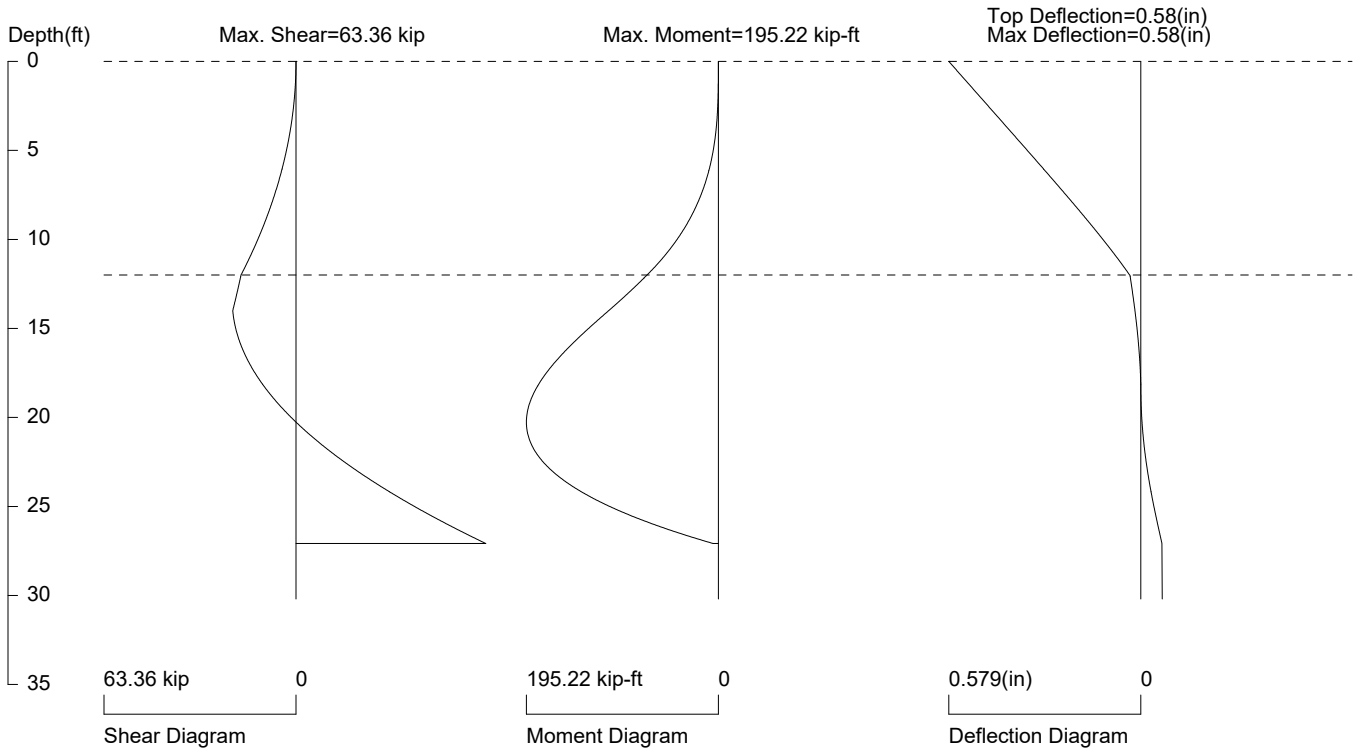
If 30% loading is used for lagging design, Design Pressure = 0.18
Pile Spacing =6.0, Max. Moment in lagging = 0.81
For 4"x12" Timber, Section Modules S=23.47 in3. The request allowable bending strength, fb=M/S=0.41
For 6"x12" Timber, Section Modules S=57.98 in3. The request allowable bending strength, fb=M/S=0.17

Unit: Pressure: ksf, Spacing: ft, Moment: kip-ft, Bending Strength, fb: ksi

Lanz Residence - Mercer Island, WA

Permanent Soldier Pile Shoring Wall





PRESSURE, SHEAR, MOMENT, AND DEFLECTION DIAGRAMS

Based on pile spacing: 6.0 foot or meter

User Input Pile, W16X89: E (ksi)=29000.0, I (in4)/pile=1300.0

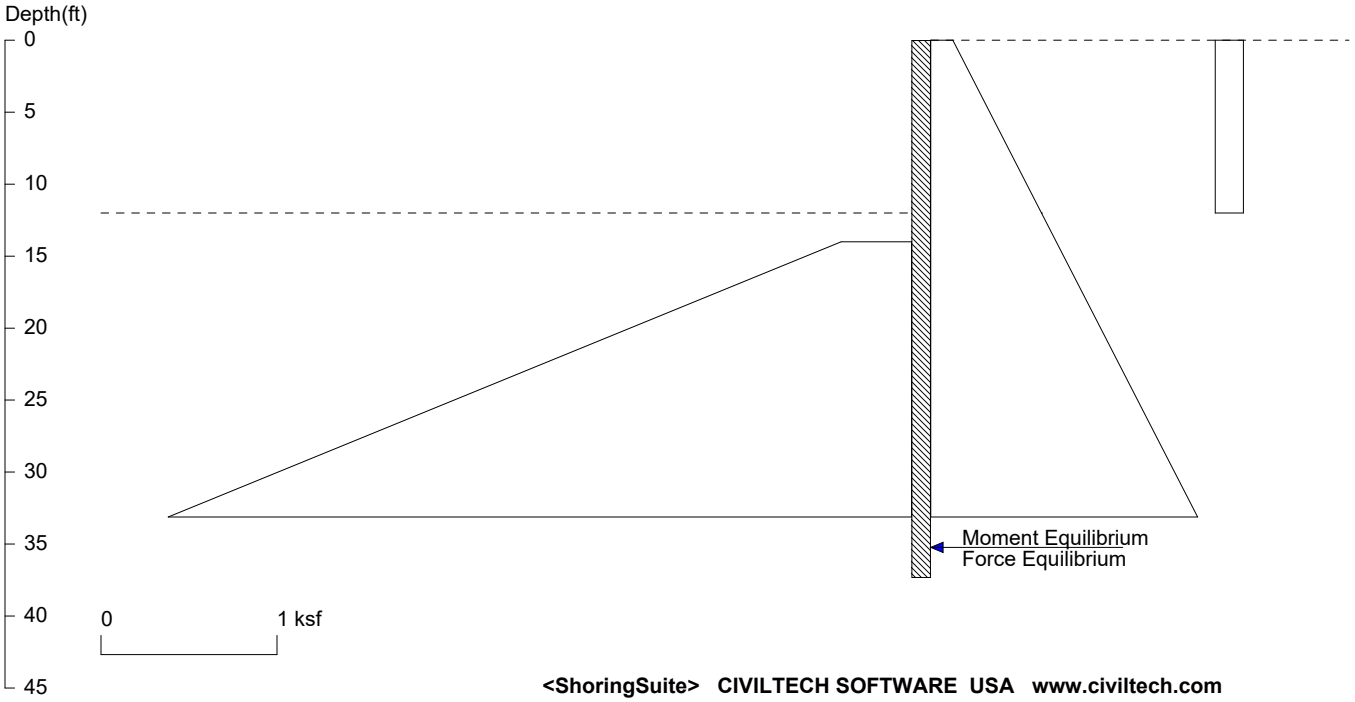
File: C:\Lucia Engineering 2024\Clients\LNL Builds\4425 Fremont Aver N\12' Tall Soldier Pile Shoring Wall - DL.sh8

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Lanz Residence - Mercer Island, WA

Permanent Soldier Pile Shoring Wall



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File: C:\Lucia Engineering 2024\Clients\LNL Builds\4425 Fremont Aver N\12' BS Tall Soldier Pile Shoring Wall.sh8

Wall Height=12.0 Pile Diameter=2.5 Pile Spacing=6.0 Wall Type: 2. Soldier Pile, Drilled

PILE LENGTH: Min. Embedment=25.34 Min. Pile Length=37.34
MOMENT IN PILE: Max. Moment=539.21 per Pile Spacing=6.0 at Depth=23.47

PILE SELECTION:
Request Min. Section Modulus = 196.1 in3/pile=3213.13 cm3/pile, Fy= 50 ksi = 345 MPa, Fb/Fy=0.66
-> Piles meet Min. Section Requirements: Top Deflection is shown in (in)
W12X152 (1.98) W14X132 (1.85) HP16X121 (1.80) W18X106 (1.49) HP18X135 (1.29)
W18X130 (1.15) HP18X157 (1.10) W18X158 (0.93) HP18X181 (0.94) W18X175J (0.82)
HP18X204 (0.82) W18X192J (0.73) W21X101 (1.17) W24X94 (1.05)

DRIVING PRESSURES (ACTIVE, WATER, & SURCHARGE):

Z1	P1	Z2	P2	Slope
0.0	0.126	12.0	0.63	0.042000
12.0	0.63	40.0	1.68	0.042000
*	Seism	Surch	(8x 20')	
0.0	0.160	12.0	0.160	0.000000

PASSIVE PRESSURES:

Z1	P1	Z2	P2	Slope
14.0	0.40	40.0	5.60	0.2000

ACTIVE SPACING:

No.	Z depth	Spacing
1	0.00	6.00
2	12.00	2.50

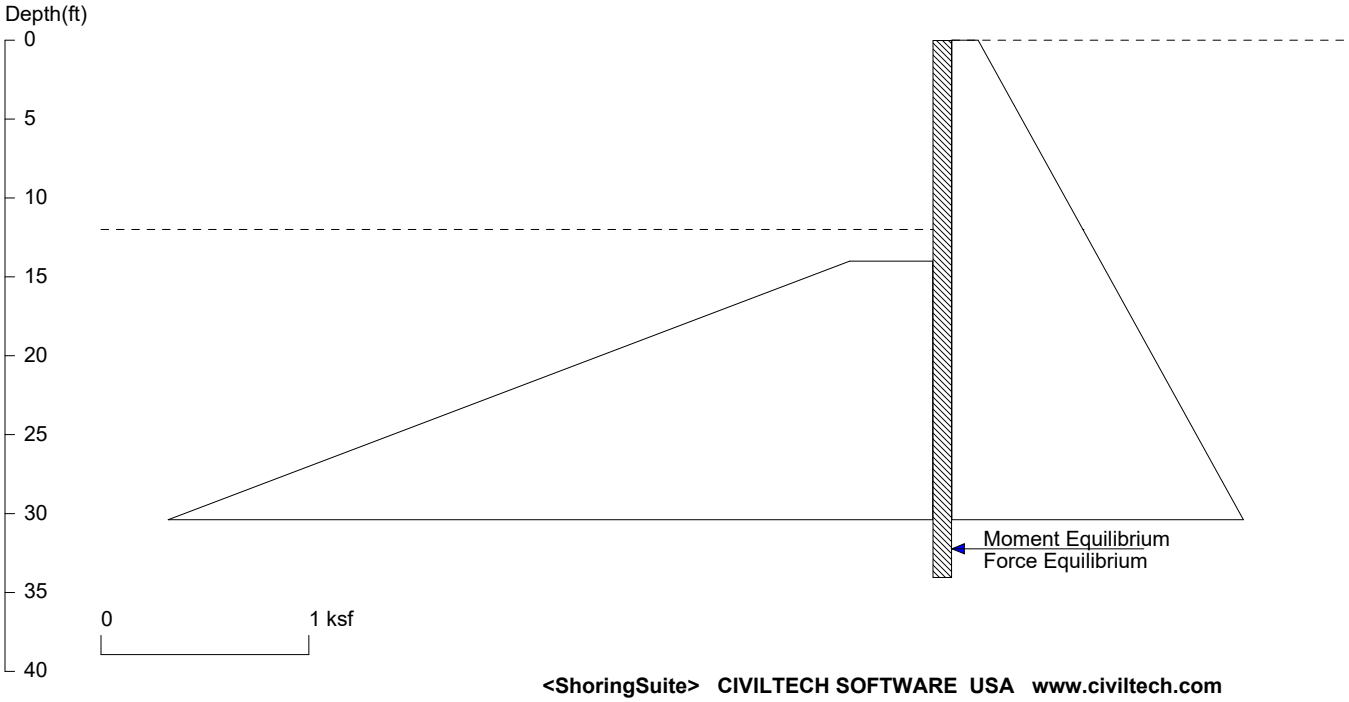
PASSIVE SPACING:

No.	Z depth	Spacing
1	12.00	5.00

UNITS: Width,Spacing,Diameter,Length,and Depth - ft; Force - kip; Moment - kip-ft
Friction,Bearing,and Pressure - ksf; Pres. Slope - kip/ft3; Deflection - in

Lanz Residence - Mercer Island, WA

Permanent Soldier Pile Shoring Wall



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File: C:\Lucia Engineering 2024\Clients\LNL Builds\4425 Fremont Aver N\12' BS Tall Soldier Pile Shoring Wall - DL.sh8

Wall Height=12.0 Pile Diameter=2.5 Pile Spacing=6.0 Wall Type: 2. Soldier Pile, Drilled

PILE LENGTH: Min. Embedment=22.07 Min. Pile Length=34.07
MOMENT IN PILE: Max. Moment=346.09 per Pile Spacing=6.0 at Depth=22.05

PILE SELECTION:
Request Min. Section Modulus = 125.8 in3/pile=2062.30 cm3/pile, Fy= 50 ksi = 345 MPa, Fb/Fy=0.66
W18X143 has Section Modulus = 282.0 in3/pile=4621.13 cm3/pile. It is greater than Min. Requirements!
Top Deflection = 0.58(in) based on E (ksi)=29000.00 and I (in4)/pile=2750.0

DRIVING PRESSURES (ACTIVE, WATER, & SURCHARGE):

Z1	P1	Z2	P2	Slope
0.0	0.126	12.0	0.63	0.042000
12.0	0.63	40.0	1.68	0.042000

PASSIVE PRESSURES:

Z1	P1	Z2	P2	Slope
14.0	0.40	40.0	5.60	0.2000

ACTIVE SPACING:

No.	Z depth	Spacing
1	0.00	6.00
2	12.00	2.50

PASSIVE SPACING:

No.	Z depth	Spacing
1	12.00	5.00

UNITS: Width,Spacing,Diameter,Length,and Depth - ft; Force - kip; Moment - kip-ft
Friction,Bearing,and Pressure - ksf; Pres. Slope - kip/ft3; Deflection - in

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SHORING WALL CALCULATION SUMMARY
The leading shoring design and calculation software
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ShoringSuite Software is developed by CivilTech Software, Bellevue, WA, USA.
The calculation method is based on the following references:

1. FHWA 98-011, FHWA-RD-97-130, FHWA SA 96-069, FHWA-IF-99-015
2. STEEL SHEET PILING DESIGN MANUAL by Pile Buck Inc., 1987
3. DESIGN MANUAL DM-7 (NAVFAC), Department of the Navy, May 1982
4. TRENCHING AND SHORING MANUAL Revision 12, California Department of Transportation, January 2000
6. EARTH SUPPORT SYSTEM & RETAINING STRUCTURES, Pile Buck Inc. 2002
5. DESIGN OF SHEET PILE WALLS, EM 1110-2-2504, U.S. Army Corps of Engineers, 31 March 1994
7. EARTH RETENTION SYSTEMS HANDBOOK, Alan Macnab, McGraw-Hill. 2002
8. Temporary Structures in Construction, Robert T. Ratay (Co-author of Chapter 7: John J. Peirce), McGraw-Hill. 2012
9. AASHTO HB-17, American Association of State and Highway Transportation Officials, 2 September 2002

UNITS: Width/Spacing/Diameter/Length/Depth - ft, Force - kip, Moment - kip-ft, Friction/Bearing/Pressure - ksf, Pres. Slope - kip/ft3, Deflection - in

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Date: 3/4/2024 File: C:\Lucia Engineering 2024\Clients\LNL Builds\4425 Fremont Aver N\12' BS Tall Soldier Pile Shoring Wall.sh8

Title: Lanz Residence - Mercer Island, WA
Subtitle: Permanent Soldier Pile Shoring Wall

**********INPUT DATA**********
Wall Type: 2. Soldier Pile, Drilled
Wall Height: 12.00
Pile Diameter: 2.50
Pile Spacing: 6.00
Factor of Safety (F.S.): 1.00
Lateral Support Type (Braces): 1. No
Top Brace Increase (Multi-Bracing): Add 15%*
Embedment Option: 1. Yes
Friction at Pile Tip: No
Pile Properties:
Steel Strength, Fy: 50 ksi = 345 MPa
Allowable Fb/Fy: 0.66
Elastic Module, E: 29000.00
Moment of Inertia, I: 1430
User Input Pile: W16X45

* DRIVING PRESSURE (ACTIVE, WATER, & SURCHARGE) *

No.	Z1 top	Top Pres.	Z2 bottom	Bottom Pres.	Slope
1	0.0	0.126	12.0	0.63	0.042000
2	12.0	0.63	40.0	1.68	0.042000
3	*	Seism	Surch	(8x 20')	
4	0.0	0.160	12.0	0.160	0.000000

* PASSIVE PRESSURE *

No.	Z1 top	Top Pres.	Z2 bottom	Bottom Pres.	Slope
1	14.0	0.40	40.0	5.60	0.2000

* ACTIVE SPACE *

No.	Z depth	Spacing
1	0.00	6.00
2	12.00	2.50

* PASSIVE SPACE *

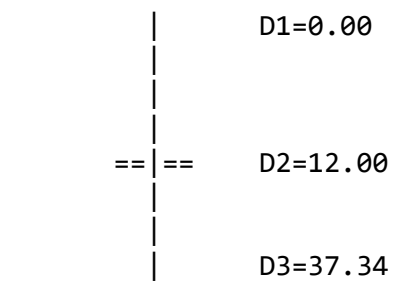
No.	Z depth	Spacing
1	12.00	5.00

*For Tieback: Input1 = Diameter; Input2 = Bond Strength
*For Plate: Input1 = Diameter; Input2 = Allowable Pressure
*For Deadman: Input1 = Horz. Width; Input2 = Passive Pressure;
*For Sheet Pile Anchor: Input1 = Horz. Width; Input2 = Passive Slope;

**********CALCULATION**********

The calculated moment and shear are per pile spacing. Sheet piles are per one foot or meter; Soldier piles are per pile.

Top Pressures start at depth = 0.00



D1 - TOP DEPTH
D2 - EXCAVATION BASE
D3 - PILE TIP

MOMENT equilibrium AT DEPTH=33.12 WITH EMBEDMENT OF 21.12
FORCE equilibrium AT DEPTH=37.34 WITH EMBEDMENT OF 25.34

The program calculates an embedment for moment equilibrium, then increase the embedment by 1.2

**********RESULTS**********

* EMBEDMENT Notes *
Based on USS Design Manual, first calculate embedment for moment equilibrium, then increased the embedment to get the design depth.
The embedment for moment equilibrium is 21.12
The program calculates an embedment for moment equilibrium, then increase the embedment by 1.2
The total desigh embedment is 25.34

Embedment Information:
If 20% increased, the total design embedment is 25.34
If 30% increased, the total design embedment is 27.45
If 40% increased, the total design embedment is 29.57
If 50% increased, the total design embedment is 31.68

* MOMENT IN PILE (per pile spacing)*
Pile Spacing: sheet piles are one foot or one meter; soldier piles are one pile.
Overall Maximum Moment = 539.21 at 23.47
Maximum Shear = 125.03
Moment and Shear are per pile spacing: 6.0 foot or meter

* VERTICAL LOADING *
Vertical Loading from Braces = 0.00
Vertical Loading from External Load = 0.00
Total Vertical Loading = 0.00

**********SOLDIER PILE SELECTION**********

Request Min. Section Modulus = 196.08 in3/pile = 3213.13 cm3/pile, Fy= 50 ksi = 345 MPa, Fb/Fy=0.66
The pile selection is based on the magnitude of the moment only. Axial force is neglected.

W12X152

(English Units):
Area= 44.7 in. Depth= 13.7 in. Width= 12.5 in. Height= 12 in.
Flange thickness= 1.4 in. Web thickness= 0.87 in.
Ix= 1430 in4/pile Sx= 209 in3/pile Iy= 454 in4/pile Sy= 72.8 in3/pile
(Metric Units):
Ix= 595.17 x100cm4/pile Sx= 3424.88 cm3/pile Iy= 188.95 x100cm4/pile Sy= 1192.97 cm3/pile
Top deflection = 1.984(in)
W14X132
(English Units):
Area= 38.8 in. Depth= 14.7 in. Width= 14.7 in. Height= 14 in.
Flange thickness= 1.03 in. Web thickness= 0.645 in.
Ix= 1530 in4/pile Sx= 209 in3/pile Iy= 548 in4/pile Sy= 74.5 in3/pile
(Metric Units):
Ix= 636.79 x100cm4/pile Sx= 3424.88 cm3/pile Iy= 228.08 x100cm4/pile Sy= 1220.83 cm3/pile
Top deflection = 1.855(in)
HP16X121
(English Units):
Area= 35.7 in. Depth= 15.75 in. Width= 15.875 in. Height= 16 in.
Flange thickness= 0.75 in. Web thickness= 0.75 in.
Ix= 1578 in4/pile Sx= 200 in3/pile Iy= 501 in4/pile Sy= 63.1 in3/pile
(Metric Units):
Ix= 656.76 x100cm4/pile Sx= 3277.40 cm3/pile Iy= 208.52 x100cm4/pile Sy= 1034.02 cm3/pile
Top deflection = 1.798(in)
W18X106
(English Units):
Area= 31.1 in. Depth= 18.7 in. Width= 11.2 in. Height= 18 in.
Flange thickness= 0.94 in. Web thickness= 0.59 in.
Ix= 1910 in4/pile Sx= 204 in3/pile Iy= 220 in4/pile Sy= 39.4 in3/pile
(Metric Units):
Ix= 794.94 x100cm4/pile Sx= 3342.95 cm3/pile Iy= 91.56 x100cm4/pile Sy= 645.65 cm3/pile
Top deflection = 1.486(in)
HP18X135
(English Units):
Area= 39.8 in. Depth= 17.5 in. Width= 17.75 in. Height= 18 in.
Flange thickness= 0.75 in. Web thickness= 0.75 in.
Ix= 2196 in4/pile Sx= 251 in3/pile Iy= 700 in4/pile Sy= 78.8 in3/pile
(Metric Units):
Ix= 913.98 x100cm4/pile Sx= 4113.14 cm3/pile Iy= 291.34 x100cm4/pile Sy= 1291.30 cm3/pile
Top deflection = 1.292(in)
W18X130
(English Units):
Area= 38.2 in. Depth= 19.3 in. Width= 11.2 in. Height= 18 in.
Flange thickness= 1.2 in. Web thickness= 0.67 in.
Ix= 2460 in4/pile Sx= 256 in3/pile Iy= 278 in4/pile Sy= 49.9 in3/pile
(Metric Units):

Ix= 1023.85 x100cm4/pile Sx= 4195.07 cm3/pile Iy= 115.70 x100cm4/pile Sy= 817.71 cm3/pile
Top deflection = 1.154(in)
HP18X157
(English Units):
Area= 46.2 in. Depth= 17.74 in. Width= 17.87 in. Height= 18 in.
Flange thickness= 0.87 in. Web thickness= 0.87 in.
Ix= 2583 in4/pile Sx= 291 in3/pile Iy= 829 in4/pile Sy= 93 in3/pile
(Metric Units):
Ix= 1075.04 x100cm4/pile Sx= 4768.62 cm3/pile Iy= 345.03 x100cm4/pile Sy= 1523.99 cm3/pile
Top deflection = 1.099(in)
W18X158
(English Units):
Area= 46.3 in. Depth= 19.7 in. Width= 11.3 in. Height= 18 in.
Flange thickness= 1.44 in. Web thickness= 0.81 in.
Ix= 3060 in4/pile Sx= 310 in3/pile Iy= 347 in4/pile Sy= 61.4 in3/pile
(Metric Units):
Ix= 1273.57 x100cm4/pile Sx= 5079.97 cm3/pile Iy= 144.42 x100cm4/pile Sy= 1006.16 cm3/pile
Top deflection = 0.927(in)
HP18X181
(English Units):
Area= 53.2 in. Depth= 18 in. Width= 18 in. Height= 18 in.
Flange thickness= 1 in. Web thickness= 1 in.
Ix= 3017 in4/pile Sx= 335 in3/pile Iy= 974 in4/pile Sy= 108.1 in3/pile
(Metric Units):
Ix= 1255.68 x100cm4/pile Sx= 5489.65 cm3/pile Iy= 405.38 x100cm4/pile Sy= 1771.43 cm3/pile
Top deflection = 0.941(in)
W18X175J
(English Units):
Area= 51.3 in. Depth= 20 in. Width= 11.4 in. Height= 18 in.
Flange thickness= 1.59 in. Web thickness= 0.89 in.
Ix= 3450 in4/pile Sx= 344 in3/pile Iy= 391 in4/pile Sy= 68.8 in3/pile
(Metric Units):
Ix= 1435.89 x100cm4/pile Sx= 5637.13 cm3/pile Iy= 162.73 x100cm4/pile Sy= 1127.43 cm3/pile
Top deflection = 0.823(in)
HP18X204
(English Units):
Area= 60 in. Depth= 18.25 in. Width= 18.125 in. Height= 18 in.
Flange thickness= 1.125 in. Web thickness= 1.125 in.
Ix= 3450 in4/pile Sx= 378 in3/pile Iy= 1119 in4/pile Sy= 123 in3/pile
(Metric Units):
Ix= 1435.89 x100cm4/pile Sx= 6194.29 cm3/pile Iy= 465.73 x100cm4/pile Sy= 2015.60 cm3/pile
Top deflection = 0.823(in)
W18X192J
(English Units):

Area= 56.4 in. Depth= 20.4 in. Width= 11.5 in. Height= 18 in.
Flange thickness= 1.75 in. Web thickness= 0.96 in.
Ix= 3870 in4/pile Sx= 380 in3/pile Iy= 440 in4/pile Sy= 76.8 in3/pile
(Metric Units):
Ix= 1610.69 x100cm4/pile Sx= 6227.06 cm3/pile Iy= 183.13 x100cm4/pile Sy= 1258.52 cm3/pile
Top deflection = 0.733(in)
W21X101
(English Units):
Area= 29.8 in. Depth= 21.4 in. Width= 12.3 in. Height= 21 in.
Flange thickness= 0.8 in. Web thickness= 0.5 in.
Ix= 2420 in4/pile Sx= 227 in3/pile Iy= 248 in4/pile Sy= 40.3 in3/pile
(Metric Units):
Ix= 1007.20 x100cm4/pile Sx= 3719.85 cm3/pile Iy= 103.22 x100cm4/pile Sy= 660.40 cm3/pile
Top deflection = 1.173(in)
W24X94
(English Units):
Area= 27.7 in. Depth= 24.3 in. Width= 9.07 in. Height= 24 in.
Flange thickness= 0.875 in. Web thickness= 0.515 in.
Ix= 2700 in4/pile Sx= 222 in3/pile Iy= 109 in4/pile Sy= 24 in3/pile
(Metric Units):
Ix= 1123.74 x100cm4/pile Sx= 3637.91 cm3/pile Iy= 45.37 x100cm4/pile Sy= 393.29 cm3/pile
Top deflection = 1.051(in)

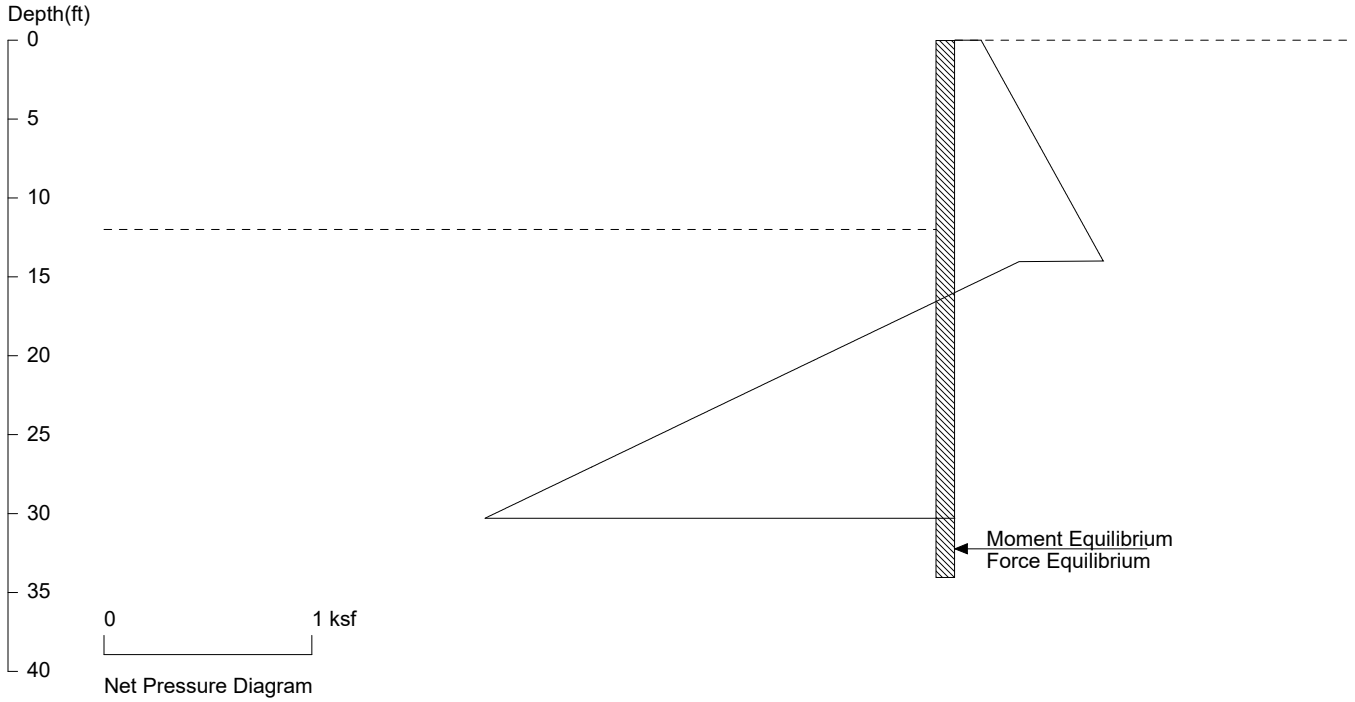
********** LAGGING SIZE ESTIMATION **********
Max. Pressure above base = 0.79
Piles are more rigid than timber lagging, due to arching, only portion of pressures are acting to lagging, 30-50% loading is suggested.
If 50% loading is used for lagging design, Design Pressure = 0.39
Pile Spacing =6.0, Max. Moment in lagging = 1.78
For 4"x12" Timber, Section Modules S=23.47 in3. The request allowable bending strength, fb=M/S=0.91
For 6"x12" Timber, Section Modules S=57.98 in3. The request allowable bending strength, fb=M/S=0.37

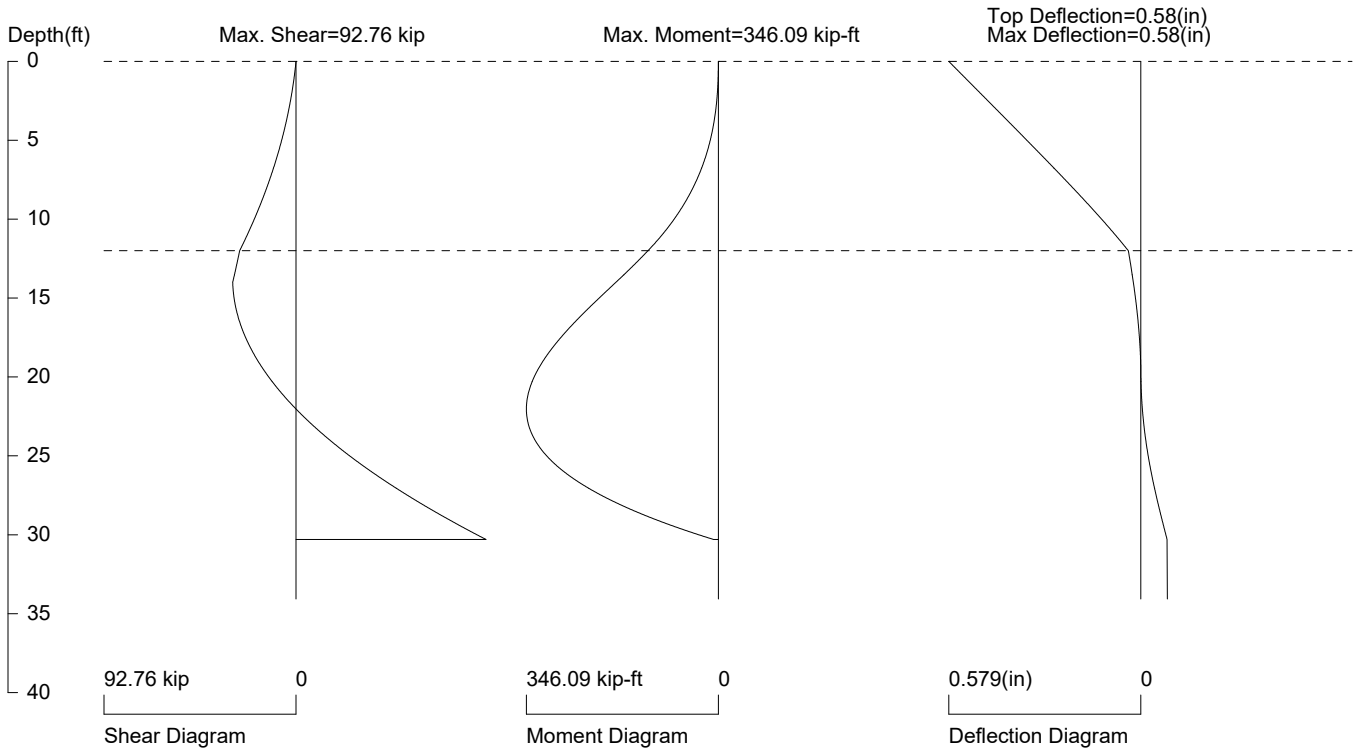
If 30% loading is used for lagging design, Design Pressure = 0.24
Pile Spacing =6.0, Max. Moment in lagging = 1.07
For 4"x12" Timber, Section Modules S=23.47 in3. The request allowable bending strength, fb=M/S=0.54
For 6"x12" Timber, Section Modules S=57.98 in3. The request allowable bending strength, fb=M/S=0.22

Unit: Pressure: ksf, Spacing: ft, Moment: kip-ft, Bending Strength, fb: ksi

Lanz Residence - Mercer Island, WA

Permanent Soldier Pile Shoring Wall





PRESSURE, SHEAR, MOMENT, AND DEFLECTION DIAGRAMS

Based on pile spacing: 6.0 foot or meter

User Input Pile, W18X143: E (ksi)=29000.0, I (in4)/pile=2750.0

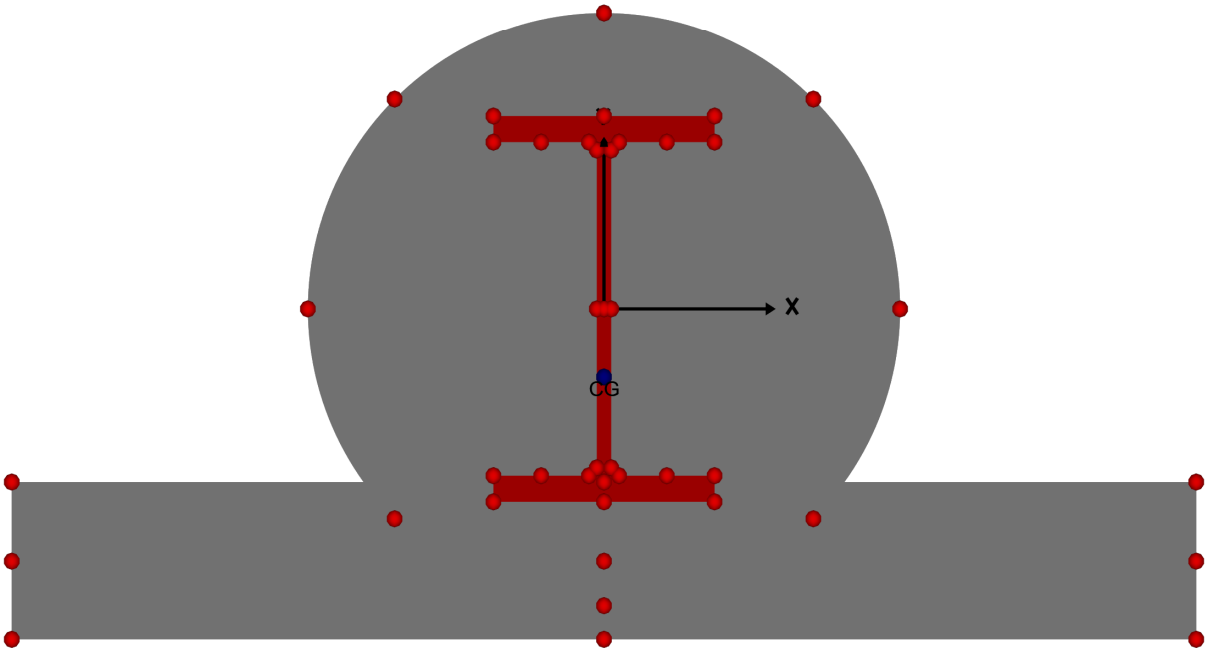
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<ShoringSuite> CIVILTECH SOFTWARE USA www.civiltech.com

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PILE INFORMATION

Pile No.	Wide Flange Section	Pile Spacing (FT)	Calculated Wide Flange Pile Length (FT)	Pile Weight (LBS)	Shored Height (FT)	Exist & Final Ground Eleve. At Back of Wall	Req'd Embedment Depth (FT)	Predicted Deflection (Inches)	Shaft Diameter (FT)	Lean Mix Concrete (CY Neat)	Timber Lagging	Lagging Area (SF)	Top of Pile Elev. (FT)	Excavation Grade Face of Wall Elev. (FT)	Bottom of Shaft Elev. (FT)
1	W16 x 45		31.50	1,417.50	8.00	113.00	17.50	< 1	2.50	5.72	4 x 12		114.00	100.00	82.50
2	W16 x 45	6.00	33.50	1,507.50	12.00	114.00	17.50	< 1	2.50	6.09	4 x 12	84.00	116.00	100.00	82.50
3	W18 x 143	6.00	42.50	6,077.50	12.00	115.50	25.50	< 1	2.50	7.72	4 x 12	96.00	117.00	100.00	74.50
4	W18 x 143	6.00	43.50	6,220.50	12.00	116.50	25.50	< 1	2.50	7.90	4 x 12	102.00	118.00	100.00	74.50
5	W18 x 143	6.00	44.50	6,363.50	12.00	118.00	25.50	< 1	2.50	8.09	4 x 12	108.00	119.00	100.00	74.50
6	W18 x 143	6.00	45.50	6,506.50	11.75	119.00	25.50	< 1	2.50	8.27	4 x 12	114.00	120.00	100.00	74.50
7	W18 x 143	6.00	45.50	6,506.50	11.50	120.00	25.50	< 1	2.50	8.27	4 x 12	120.00	120.00	100.00	74.50
8	W18 x 143	6.00	45.50	6,506.50	11.00	120.00	25.50	< 1	2.50	8.27	4 x 12	120.00	120.00	100.00	74.50
9	W18 x 143	6.00	45.50	6,506.50	8.50	120.00	25.50	< 1	2.50	8.27	4 x 12	120.00	120.00	100.00	74.50
10	W18 x 143	6.00	45.50	6,506.50	7.50	120.00	25.50	< 1	2.50	8.27	4 x 12	120.00	120.00	100.00	74.50
11	W18 x 143	6.00	45.50	6,506.50	6.00	120.00	25.50	< 1	2.50	8.27	4 x 12	120.00	120.00	100.00	74.50
12	W14 x 143	6.00	45.50	6,506.50	2.00	120.00	25.50	< 1	2.50	8.27	4 x 12	120.00	120.00	100.00	74.50
13	W14 x 143	6.00	45.50	6,506.50	2.25	120.00	25.50	< 1	2.50	8.27	4 x 12	120.00	120.00	100.00	74.50
14	W16 x 45	6.00	33.50	1,507.50	4.00	117.00	17.50	< 1	2.50	6.09	4 x 12	120.00	116.00	100.00	82.50
				75,146 LBS						108 CY		1,464 SF			


X
CG

Geometric Properties

Area*	153.769	in^2
Ix*	12089.924	in^4
Ixy*	0.000	in^4
Iy*	19695.796	in^4
Sx+*	655.604	in^3
Sx-*	908.395	in^3
Sy+*	656.527	in^3
Sy-*	656.527	in^3
Xc*	0.000	in
Yc*	-3.441	in
rx*	8.867	in
ry*	11.318	in

* means transformed by E/E_base

Overall Properties

Depth	31.750	in
E_base	29000.000	Ksi
Perimeter	177.443	in
Weight	1.224	K/ft
Width	60.000	in

Applied Loads

Ma	540.000	K-ft
Mb	0.000	K-ft
P	0.000	K
T	0.000	K-ft
Va	0.000	K
Vb	0.000	K

Loads applied to: Centroidal Axes

Maximum Results

Normal Stress σz	7.070	Ksi

Individual Part Properties

Name	Material	E Ksi	n	Area* in^2	I1* in^4	I2* in^4	Xc in	Yc in
Circle 1	Concrete (F'c = 2.5 ksi)	3122.000	0.108	76.084	4279.046	4279.046	0.000	0.000
Rectangle 1	Concrete (F'c = 3 ksi)	3122.000	0.108	51.674	15502.345	275.597	0.000	-12.750
W18X143 1	ASTM A992 Grade 50	29000.000	1.000	42.015	2748.646	309.660	0.000	0.000

* means transformed by E/E_base

Soldier Pile Lagging Design:

Assumptions:

fy =	60,000	PSI
f'c =	4,000	PSI

Shotcrete or CIP Wall Info:

d =	12.00	Inches
b =	8.00	Inch Width

Check Reinforcign Steel Required for Bending:

Active Soil pressure:

Shored Height:	24.00	FT
Active Soil Loading:	65.00	PCF
	1,560.00	PLF
Pile Spacing:	5.00	FT
M =	4.88	K-ft
	4.88	K-ft
M_u = M x 1.6 =	7.8	K-ft

$As_{(Req)} = M_u / \phi F_y jd$

Where:

b =	12.00	in
d =	8.00	in
ϕ =	0.90	
j =	0.92	

$As_{(Req)}$ = 0.24 in^2

Use (2) #5 bar A_s = 0.62 in^2 **Check - O.K.**

Check Shear:

Check Section Limitation: (ACI 11.4.7.9)

$V_{AL} = CA_{cv} + \upsilon (A_{vf}f_y + P_c)$

Where:

C =	0.075	
A_{cv} =	384.00	in^2
υ =	0.70	
A_{vf} =	0.62	in^2
f_y =	60.00	KSI
P_c =	-	Assumed

V_{AL} = 54.8 Kips

V_u </= $10\phi bd(f'_c)^{0.5}$

V_u </= $8bd(f'_c)^{0.5}$

Where:

ϕ = 0.75

V_{max} = 19,500 LBS

V_u = $1.6V_{max}$ = **31.2** Kips

$V_{AL} = CA_{cv} + \upsilon (A_{vf}f_y + P_c)$ = **54.8** Kips

31.2 kips < **54.8** Kips **Check - O.K.**

$10\phi bd(f'_c)^{0.5}$ = **91.1** Kips

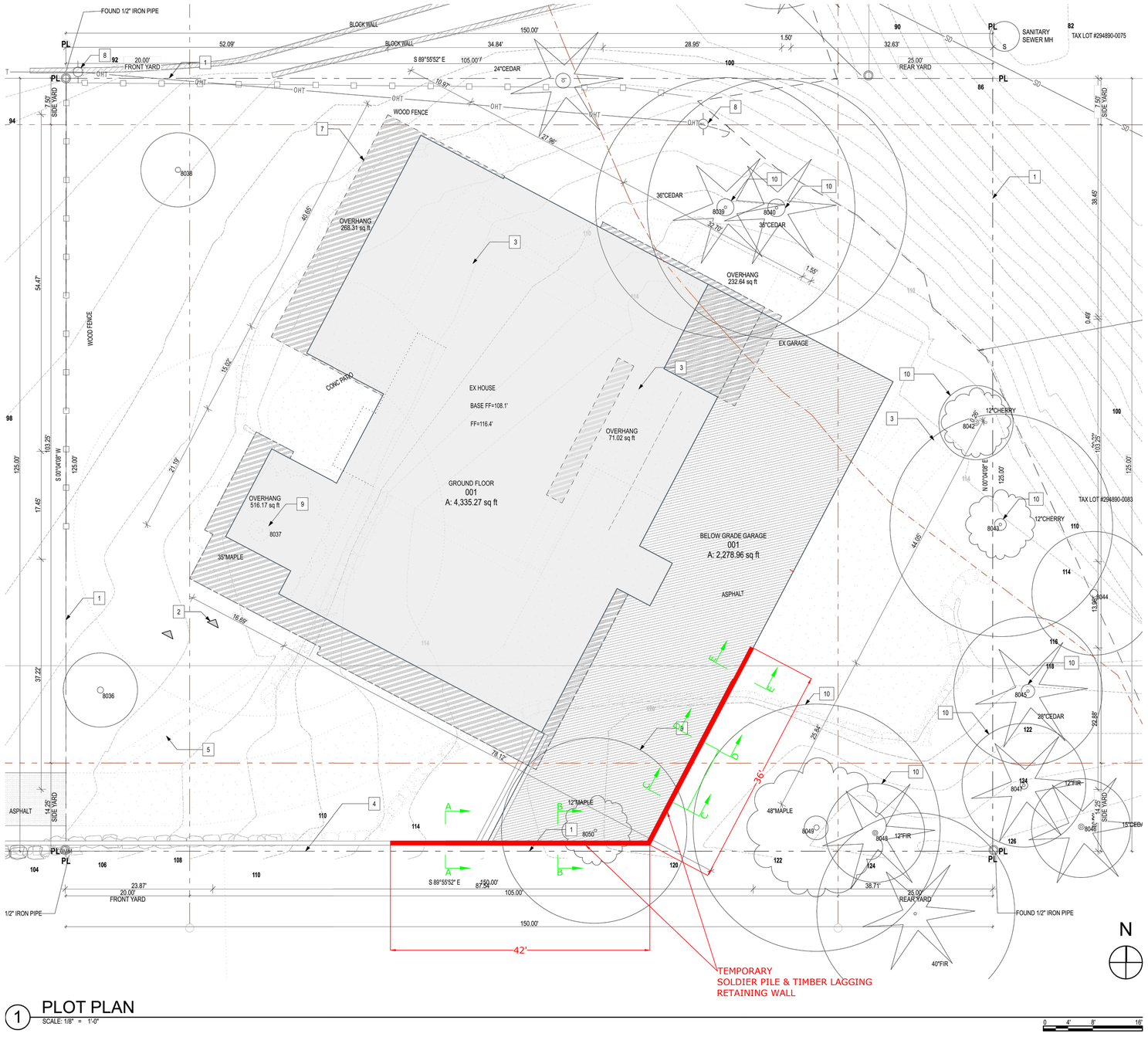
31.2 kips < **91.1** Kips **Check - O.K.**

$8bd(f'_c)^{0.5}$ = **97.1**

31.2 kips < **97.1** Kips **Check - O.K.**

LANZ RESIDENCE - SOLDER PILE RETAINING WALL

PERMANENT SOLDIER PILE & TIMBER LAGGING SHORING WALL



1 PLOT PLAN

SCALE: 1/8" = 1'-0"

OWNER:
Vann Lanz
8020 SE 57th Street
Mercer Island, WA 98040
(206)499-1277

SHORING DESIGNER:
Lucia Engineering, Inc.
Joseph M Lucia
12527 Huckleberry Lane
Arlington, WA 98223
(206)790-8039

GEOTECHNICAL ENGINEER:
Earth Solutions NW, LLC
15365 N.E. 90th Street, Suite 100
Redmond,WA 98052
(425)449-4704

ARCHITECT:
Bradley Khouri
610 2nd Avenue
Seattle, WA 98104
(206)297-1284

LANZ RESIDENCE
8020 SE 57th Street
Mercer Island, WA 98040

Permanent Soldier Pile & Timber Lagging Retaining Wall

LUCIA ENGINEERING, INC.
12527 Huckleberry Lane
Arlington, Washington 98223
PHONE: (206) 790-8039
E-MAIL: joe@luciaeng.com

JOSEPH M. LUCIA
STATE OF WASHINGTON
23314
REGISTERED
PROFESSIONAL ENGINEER

08-15-24

Number	Date	By	Description
5	08-15-24	JML	

SHEET
S-1.0

SOLDIER PILE - NOTES:

REFERENCE STANDARDS:

ACI 301-10 "STANDARD SPECIFICATIONS FOR STRUCTURAL CONCRETE"
2021 INTERNATIONAL BUILDING CODE
2018 NATIONAL DESIGN SPECIFICATIONS for WOOD CONSTRUCTION

DESIGN LOADING:
REF. SOIL REPORT
EARTH SOLUTIONS NW, LLC
Dated: October 4, 2023
Pa = 42 PCF
Pp = 200 PCF
Seismic loading = 8H

SEISMIC LOADING:
EQUIVALENT LATERAL FORCE PROCEDURE (ASCE 7-16, SECTION 12.8)
SITE CLASS: D
S_S: 1.462
S_1: 0.507
RISK CATEGORY: II
IMPORTANCE FACTOR: (I_E) 1.0
SEISMIC DESIGN CATEGORY: D

CONCRETE:

CONCRETE MIXTURES: CONFORM TO:
(1) ACI 301 SECTION 4 "CONCRETE MIXTURES"

MATERIALS: CONFORM TO:
(1) ACI 301 SECTION 4.2.1 "MATERIALS" FOR REQUIREMENTS FOR CEMENTITIOUS MATERIALS, AGGREGATES, MIXING WATER AND ADMIXTURES.

MIX DESIGN REQUIREMENTS:

PILE CONCRETE:
ABOVE EXCAVATION LINE (DREDGE LINE): LEAN MIX
BELOW EXCAVATION LINE (DREDGE LINE): LENA MIX

MIX DESIGN NOTES:
LEAN MIX SHALL HAVE A MINIMUM OF 1-1/2 SACKS (141 POUNDS) OF CEMENT AND 200 POUNDS OF FLY ASH PER CUBIC YARD OF CONCRETE.

PORTLAND CEMENT SHALL BE TYPE I, II, OR III CONFORMING TO ASTM C150 / AASHTO M85
FLY ASH SHALL BE TYPE F CONFORMING TO ASTM C618

FINE AGGREGATES SHALL CONFORM TO ASTM C88 / AASHTO M6
COARSE AGGREGATES SHALL CONFORM TO AASHTO M80. CLASS B

SLUMP FOR LEAN -MIX CONCRETE SHALL NOT BE LESS THAN 5 INCHES AND NOT MORE THAN 9 INCHES.

ADMIXTURES SHALL CONFORM TO ASTM C494 / AASHTO M194

MIX DESIGNS ARE TO BE SUBMITTED TO THE SHORING DESIGN ENGINEER FOR APPROVAL PRIOR TO USE

STRUCTURAL STEEL:

REFERENCED STANDARDS:
(1) AISC "MANUAL OF STEEL CONSTRUCTION - ALLOWABLE STRESS DESIGN"
(2) AISC "CODE OF STANDARD PRACTICE FOR STEEL BUILDINGS & BRIDGES"
(3) AWS D1.1 "STRUCTURAL WELDING CODE - STEEL"

MATERIALS: CONFORM TO:
STRUCTURAL WF SHAPES - ASTM A992-GR50
HEADED STUDS SHALL CONFORM TO ASTM A108

PAINT:

CORROSION PROTECTION IS NOT REQUIRED

WELDING:

WELDING AND REPAIR WELDING FOR ALL STEEL FABRICATION SHALL COMPLY WITH THE AWS D1.1/D1.1M, LATEST EDITION, STRUCTURAL WELDING CODE. THE REQUIREMENTS DESCRIBED IN THE REMAINDER OF THIS SECTION SHALL PREVAIL WHENEVER THEY DIFFER FROM EITHER OF THE ABOVE WELDING CODES.

THE CONTRACTOR SHALL WELD STRUCTURAL STEEL ONLY TO THE EXTENT SHOWN IN THE PLANS.
NO WELDING, INCLUDING TACK AND TEMPORARY WELDS SHALL BE DONE IN THE SHOP OR FIELD UNLESS THE LOCATION OF THE WELDS IS SHOWN ON THE APPROVED SHOP DRAWINGS OR APPROVED BY THE ENGINEER IN WRITING.
WELDING PROCEDURES SHALL BE SUBMITTED FOR APPROVAL WITH SHOP DRAWINGS . THE PROCEDURES SHALL SPECIFY THE TYPE OF EQUIPMENT TO BE USED, ELECTRODE SELECTION, PREHEAT REQUIREMENTS, BASE MATERIALS, AND JOINT DETAILS. WHEN THE PROCEDURES ARE NOT PREQUALIFIED BY AWS OR AASHTO, EVIDENCE OF QUALIFICATION TESTS SHALL BE SUBMITTED.

WELDING SHALL NOT BEGIN UNTIL AFTER THE CONTRACTOR HAS RECEIVED THE ENGINEER'S APPROVAL OF SHOP PLANS.
THESE PLANS SHALL INCLUDE PROCEDURES FOR WELDING, ASSEMBLY, AND ANY HEAT-STRAIGHTENING OR HEAT-CURVING.

IN SHIELDED METAL-ARC WELDING, THE CONTRACTOR SHALL USE LOW-HYDROGEN ELECTRODES.
IN SUBMERGED-ARC WELDING, FLUX SHALL BE OVEN-DRIED AT 550°F FOR AT LEAST 2-HOURS, THEN STORED IN OVENS HELD AT 250°F OR MORE. IF NOT USED WITHIN 4-HOURS AFTER REMOVAL FROM A DRYING OR STORAGE OVEN, FLUX SHALL BE REDRIED BEFORE USE.
PREHEAT AND INTERPASS TEMPERATURES SHALL CONFORM TO THE APPLICABLE WELDING CODE AS SPECIFIED IN THIS SECTION. REFER TO APPROVED WELDING PROCEDURES WHEN WELDING MAIN TO STEEL MEMBERS. IF GROOVE WELDS (WEB-TO-WEB OR FLANGE-TO-FLANGE) HAVE BEEN REJECTED, THEY MAY BE REPAIRED NO MORE THAN TWICE. IF A THIRD FAILURE OCCURS, THE CONTRACTOR SHALL:

1. TRIM THE MEMBERS, IF THE ENGINEER APPROVES, AT LEAST ½-INCH ON EACH SIDE OF THE WELD;
2. REPLACE THE MEMBERS AT NO EXPENSE TO THE CONTRACTING AGENCY.

BY USING EXTENSION BARS AND RUNOFF PLATES, THE CONTRACTOR SHALL TERMINATE GROOVE WELDS IN A WAY THAT ENSURES THE SOUNDNESS OF EACH WELD TO ITS ENDS. THE BARS AND PLATES SHALL BE REMOVED AFTER THE WELD IS FINISHED AND COOLED. THE WELD ENDS SHALL THEN BE GROUND SMOOTH AND FLUSH WITH THE EDGES OF ABUTTING PARTS.

THE CONTRACTOR SHALL NOT:

1. WELD WITH ELECTROGAS OR ELECTROSLAG METHODS,
2. WELD NOR FLAME CUT WHEN THE AMBIENT TEMPERATURE IS BELOW 20°F,
3. USE COPED HOLES IN THE WEB FOR WELDING BUTT SPLICES IN THE FLANGES UNLESS THE PLANS SHOW THEM.

TIMBER:

MATERIALS:
TIMBER LAGGING SHALL BE:
HEM FIR No. 1 OR BETTER
DESIGN PROPERTIES:
E = 1,500,000 PSI (NDS Table 4A)
$F_{v\ allowable}$ = 150 PSI (NDS Table 4A)
$F_{p\ allowable}$ = 405 PSI (NDS Table 4A)
$F_{b\ allowable}$ = 975 PSI (NDS Table 4A)
OR
DOUGLAS FIR - LARCH No. 2 OR BETTER
DESIGN PROPERTIES:
E = 1,600,000 PSI (NDS Table 4A)
$F_{v\ allowable}$ = 180 PSI (NDS Table 4A)
$F_{p\ allowable}$ = 625 PSI (NDS Table 4A)
$F_{b\ allowable}$ = 900 PSI (NDS Table 4A)
4x12 LAGGING (TYPICAL) (11.25" x 3.5")
A = 39.38 IN^2 (11.24" x 3.5")
S = 22.96 IN^3 ($11.25 \times 3.5^2 / 6$)
I = 160.78 IN^4 ($11.25 \times 3.5^3 / 3$)

PRESERVATIVE TREATMENT:
NONE REQUIRED

UTILITIES & INTERFERENCES:

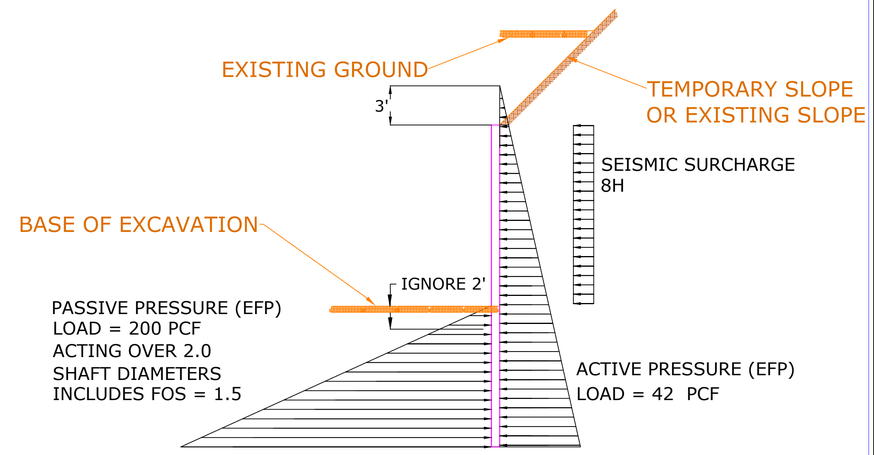
ALL EXISTING UTILITIES AND OTHER OBJECTS WHICH MAY INTERFERE WITH THE INSTALLATION OF THE SHORING SYSTEM ARE TO BE LOCATED PRIOR TO BEGINNING CONSTRUCTION.

POSSIBLE INTERFERENCES BETWEEN THE SHORING AND ANY UTILITY OR OTHER OBJECT(S) IS TO BE PROVIDED TO THE SHORING DESIGNER PRIOR TO THE START OF WORK.

SHORING INSTALLATION REVIEW:

SEE THE GEOTECHNICAL REPORT FOR REQUIRED GEOTECHNICAL INSPECTIONS & REVIEW
THE CITY REQUIRES CONTINUOUS MONITORING OF ALL SHORING INSTALLATION ACTIVITY BY THE GEOTECHNICAL ENGINEER.

SOLDIER PILE INSTALLATION - REQUIRES CONTINUOUS INSPECTION



LOADING DIAGRAM - CANTILEVERED CONDITION

LANZ RESIDENCE
8020 SE 57th Street
Mercer Island, WA 98040

Permanent Soldier Pile & Timber Lagging Retaining Wall

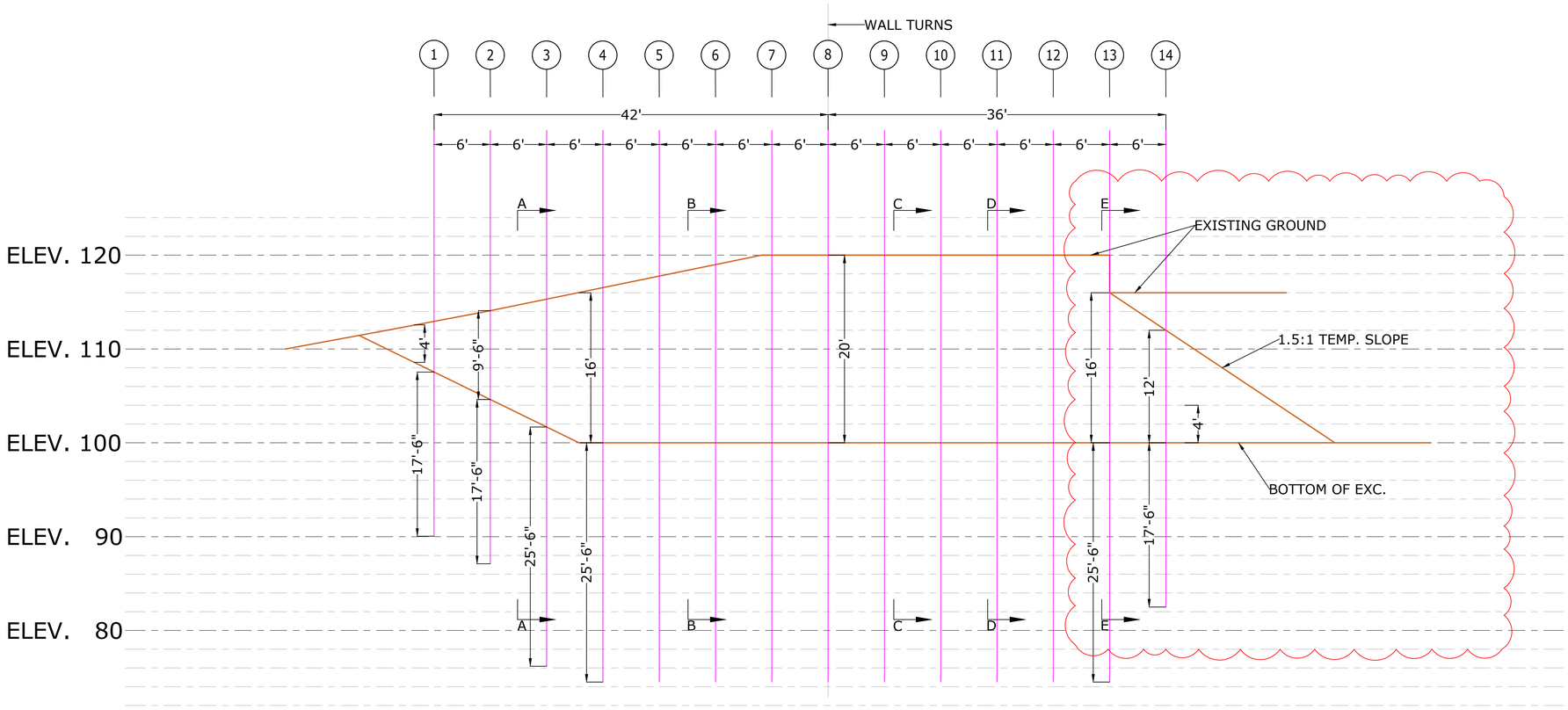
LUCIA ENGINEERING, INC.
12527 Huckleberry Lane
Arlington, Washington 98223
PHONE: (206) 790-8039
E-MAIL: joe@luciaeng.com

JOSEPH M. LUCIA
STATE OF WASHINGTON
23314
REGISTERED
PROFESSIONAL ENGINEER

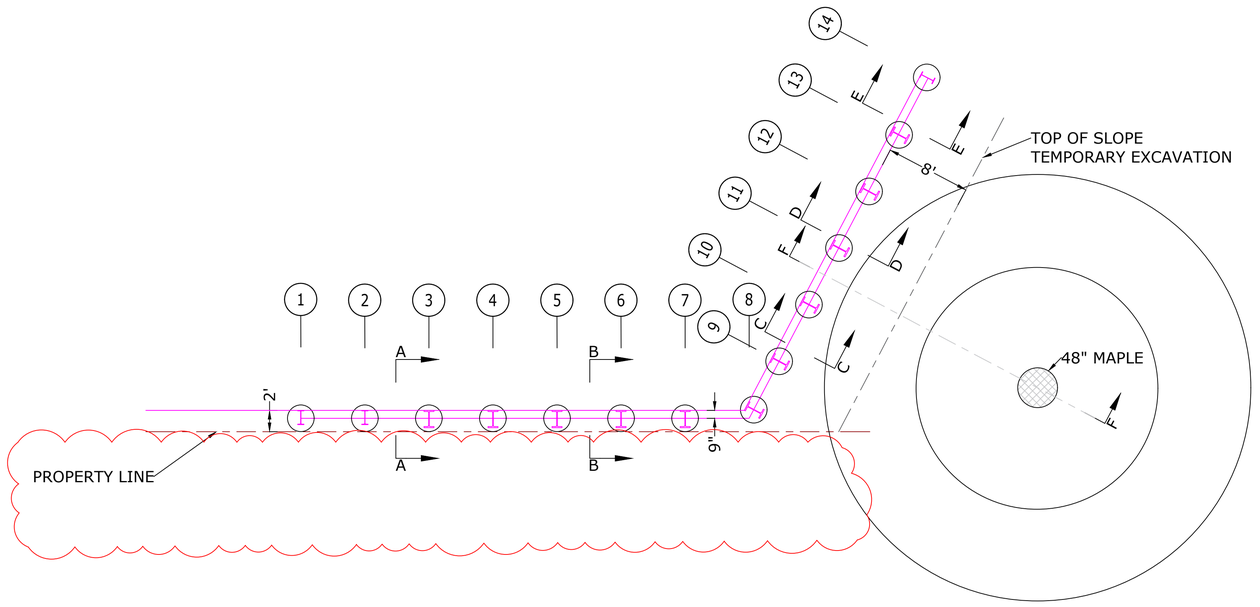
08-15-24

Number	Date	By	Description
5	08-15-24	JML	

SHEET
S-2.0



ELEVATION



PLAN

LANZ RESIDENCE
8020 SE 57th Street
Mercer Island, WA 98040

Permanent Soldier Pile
& Timber Lagging
Retaining Wall

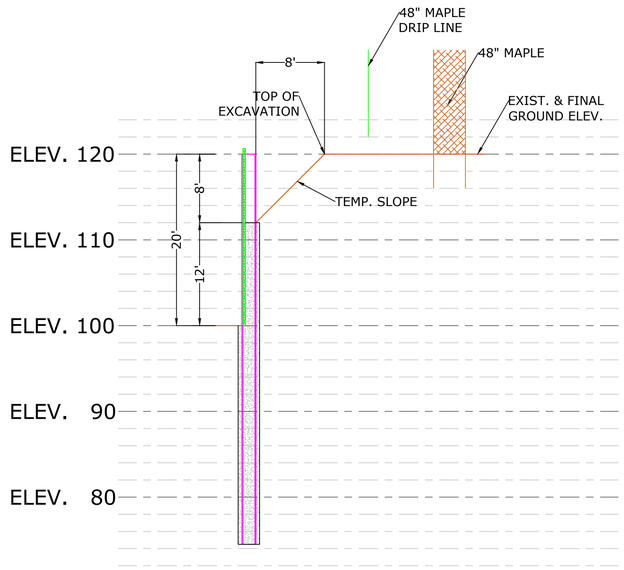
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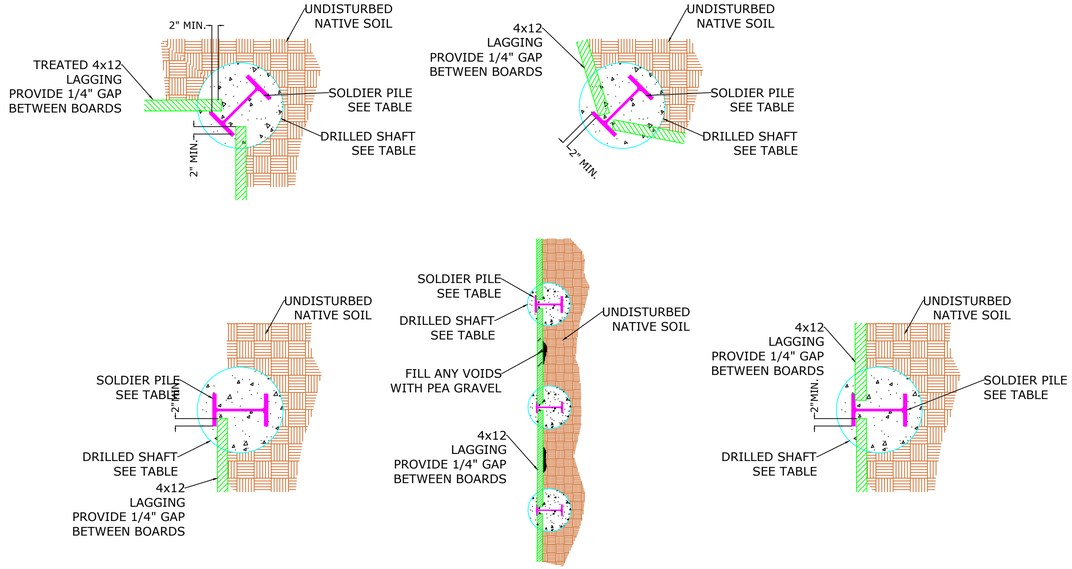
08-15-24

Number	Date	By	Description
5	08-15-24	JML	

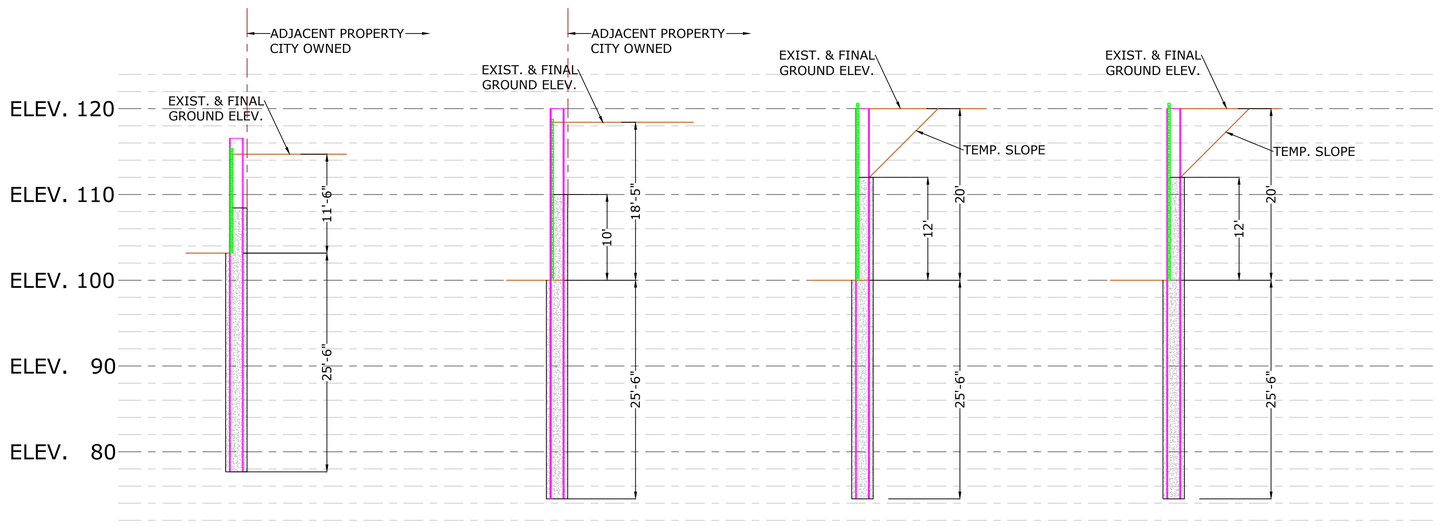
SHEET
S-3.0



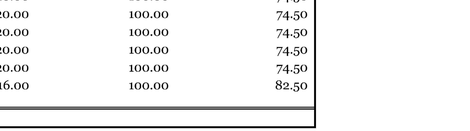
SECTION F-F



TYPICAL DETAILS - SOLDIER PILE & TIMBER LAGGING



SECTION A-A SECTION B-B SECTION C-C & D-D SECTION E-E



FUTURE FASCIA WALL DETAIL

PILE INFORMATION

Pile No.	Wide Flange Section	Pile Spacing (FT)	Calculated Wide Flange Pile Length (FT)	Pile Weight (LBS)	Shored Height (FT)	Exist & Final Ground Eleve. At Back of Wall	Req'd Embedment Depth (FT)	Predicted Deflection (Inches)	Shaft Diameter (FT)	Lean Mix Concrete (CY Neat)	Timber Lagging	Lagging Area (SF)	Top of Pile Elev. (FT)	Excavation Grade Face of Wall Elev. (FT)	Bottom of Shaft Elev. (FT)
1	W16 x 45		31.50	1,417.50	8.00	113.00	17.50	< 1	2.50	5.72	4 x 12		114.00	100.00	82.50
2	W16 x 45	6.00	33.50	1,507.50	12.00	114.00	17.50	< 1	2.50	6.09	4 x 12	84.00	116.00	100.00	82.50
3	W18 x 143	6.00	42.50	6,077.50	12.00	115.50	25.50	< 1	2.50	7.72	4 x 12	96.00	117.00	100.00	74.50
4	W18 x 143	6.00	43.50	6,220.50	12.00	116.50	25.50	< 1	2.50	7.90	4 x 12	102.00	118.00	100.00	74.50
5	W18 x 143	6.00	44.50	6,363.50	12.00	118.00	25.50	< 1	2.50	8.09	4 x 12	108.00	119.00	100.00	74.50
6	W18 x 143	6.00	45.50	6,506.50	11.75	119.00	25.50	< 1	2.50	8.27	4 x 12	114.00	120.00	100.00	74.50
7	W18 x 143	6.00	45.50	6,506.50	11.50	120.00	25.50	< 1	2.50	8.27	4 x 12	120.00	120.00	100.00	74.50
8	W18 x 143	6.00	45.50	6,506.50	11.00	120.00	25.50	< 1	2.50	8.27	4 x 12	120.00	120.00	100.00	74.50
9	W18 x 143	6.00	45.50	6,506.50	8.50	120.00	25.50	< 1	2.50	8.27	4 x 12	120.00	120.00	100.00	74.50
10	W18 x 143	6.00	45.50	6,506.50	7.50	120.00	25.50	< 1	2.50	8.27	4 x 12	120.00	120.00	100.00	74.50
11	W18 x 143	6.00	45.50	6,506.50	6.00	120.00	25.50	< 1	2.50	8.27	4 x 12	120.00	120.00	100.00	74.50
12	W14 x 143	6.00	45.50	6,506.50	2.00	120.00	25.50	< 1	2.50	8.27	4 x 12	120.00	120.00	100.00	74.50
13	W14 x 143	6.00	45.50	6,506.50	2.25	120.00	25.50	< 1	2.50	8.27	4 x 12	120.00	120.00	100.00	74.50
14	W16 x 45	6.00	33.50	1,507.50	4.00	117.00	17.50	< 1	2.50	6.09	4 x 12	120.00	116.00	100.00	82.50
				75,146 LBS						108 CY		1,464 SF			

LANZ RESIDENCE
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PROFESSIONAL ENGINEER

08-15-24

Number	Date	By	Description
5	08-15-24	JML	

SHEET
S-4.0

LATERAL MEMBER DESIGN

Cover Letter

Seismic Design Parameters

Wind & Seismic Loading Calculations

Shear Wall Design

Shear Wall Summary Table

Roof & Floor Sheathing Design

Simpson Tie & Holdown Information

Shear Wall Plans

LNL Builds
317 4th Street
Kirkland, Washington 98003

May 18, 2024

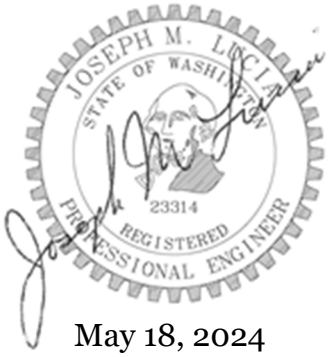
Attention: **Vann Lanz**

Reference: Lanz Residence
Vertical and lateral Member Design
Soldier Pile & Timber Lagging Shoring Wall Design

Vann

Attached are the calculations and plans required for the house and the soldier pile shoring wall.

Please contact me with any questions.



May 18, 2024
Joseph M. Lucia, PE

USGS web services were down for some period of time and as a result this tool wasn't operational, resulting in *timeout* error.
USGS web services are now operational so this tool should work as expected.

Lanz Residence

8015 SE 60th St, Mercer Island, WA 98040, USA

Latitude, Longitude: 47.549535, -122.2315926



Date	3/12/2024, 4:15:50 PM
Design Code Reference Document	ASCE7-16
Risk Category	I
Site Class	D - Default (See Section 11.4.3)

Type	Value	Description
S_S	1.466	MCE_R ground motion. (for 0.2 second period)
S_1	0.508	MCE_R ground motion. (for 1.0s period)
S_{MS}	1.759	Site-modified spectral acceleration value
S_{M1}	null -See Section 11.4.8	Site-modified spectral acceleration value
S_{DS}	1.173	Numeric seismic design value at 0.2 second SA
S_{D1}	null -See Section 11.4.8	Numeric seismic design value at 1.0 second SA

Type	Value	Description
SDC	null -See Section 11.4.8	Seismic design category
F_a	1.2	Site amplification factor at 0.2 second
F_v	null -See Section 11.4.8	Site amplification factor at 1.0 second
PGA	0.628	MCE_G peak ground acceleration
F_{PGA}	1.2	Site amplification factor at PGA
PGA_M	0.754	Site modified peak ground acceleration
T_L	6	Long-period transition period in seconds
SsRT	1.466	Probabilistic risk-targeted ground motion. (0.2 second)
SsUH	1.626	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration
SsD	4.245	Factored deterministic acceleration value. (0.2 second)
S1RT	0.508	Probabilistic risk-targeted ground motion. (1.0 second)
S1UH	0.566	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration.
S1D	1.643	Factored deterministic acceleration value. (1.0 second)

Type	Value	Description
PGAd	1.419	Factored deterministic acceleration value. (Peak Ground Acceleration)
PGA_{UH}	0.628	Uniform-hazard (2% probability of exceedance in 50 years) Peak Ground Acceleration
C_{RS}	0.902	Mapped value of the risk coefficient at short periods
C_{R1}	0.898	Mapped value of the risk coefficient at a period of 1 s
C_V	1.393	Vertical coefficient

LUCIA ENGINEERING, INC.
12527 Huckleberry Lane
Arlington, WA 98223
(206) 790-8039

Lateral Calcualtions: 2+ Story Timber Framed Structure

Project Name: **Lanz Residence**
Poject Location: 8020 Southeast 57th Street
Mercer Island, WA

Rev - 0 **3/8/2024**

Owner:	**Vann Lanz**	Architect:	**b9 Architects, Inc.**
Contact:	**Vann Lanz**	Contact:	**Bradley Khouri**
Phone:	(206) 499-1277	Phone:	(206) 297-1284

Referenced Design Standards:

IBC 2018
SBC 2018 **Wind:** Simplified Procedure ASCE Section C6.4 - Method 1
NDS 2018 **Seismic:** Equivalent Lateral Force Procedure ASCE 7-10

Input Data:

Address:
8020 Southeast 57th Street Mercer Island, WA

Building Length: "L" =	83.25	feet	X-Direction
Building Width: "B" =	82.67	feet	Y-Direction
Height to Eave: "he" =	34.00	feet	
Height to Ridge: "hr" =	36.00	feet	
Topographic Factor: "K_{zt}" =	1.38		Used, Exhibit A - Ordinance 16-0478 (See Seattle Wind Map, attached.)
Basic Wind speed:	110.00	mph	(V_{asd} = 85 mph, per SBC Table 1609.3.1)
Exposure Category:	B		
Importance Factor: "I" =	1.00		
Site Classification:	D		Default Site Class (Soil properties are not know in sufficient detail to determine site class)

Mapped Acceleration Parameters:

S_1 =	0.508	See the attached results from the USGS report
S_s =	1.466	See the attached results from the USGS report
S_{Ms} =	1.759	See the attached results from the USGS report
S_{M1} =	null	See the attached results from the USGS report (See USGS Section 11.4.2)
F_a =	1.20	See the attached results from the USGS report
F_v =	null	See the attached results from the USGS report (See USGS Section 11.4.2, Table 11.4-2)
S_{DS} =	1.173	See the attached results from the USGS report
S_{D1} =	null	See the attached results from the USGS report (See USGS Section 11.4.2)
R_w =	6.5	Wood Framed Structure - ASCE 7-10, Table 12.2-1 (A-13)
R_c =	5.0	Concrete Structure - ASCE 7-10, Table 12.2-1 (A-1)
$C_{d,w}$ =	4.00	Wood Framed Structure - ASCE 7-10, Table 12.2-1 (A-13)
$C_{d,c}$ =	5.00	Concrete Structure - ASCE 7-10, Table 12.2-1 (A-1)
$\Omega_{o,w}$ =	3.00	Wood Framed Structure - ASCE 7-10, Table 12.2-1 (A-13)
$\Omega_{o,c}$ =	2.50	Concrete Structure - ASCE 7-10, Table 12.2-1 (A-1)
ρ =	1.00	ASCE 7-10, Section 12.3.4.2 Redundancy Factor for Seismic Design Categories D through F
C_t =	0.02	ASCE 7-10, Table 12.8-2 Values of Approximate Period Parameters Ç & x
C_{t1} =	0.016	ASCE 7-10, Table 12.8-2 Values of Approximate Period Parameters Ç & x b (Concrete)
x =	0.75	ASCE 7-10, Table 12.8-2 Values of Approximate Period Parameters Ç & x
x_1 =	0.90	ASCE 7-10, Table 12.8-2 Values of Approximate Period Parameters Ç & x (Concrete)

Roof		Roof Area:	
	Roof Area:	1910.00	feet²
	Deck Area:	0.00	feet²
	Diaphragm Area:	1910.00	feet²

Floor		Floor Area:	
	Floor Area	1690.00	feet²
	Diaphragm Area:	1690.00	feet²

Second Floor		Floor Area:	
	Floor Area	4960.00	feet²
	Diaphragm Area:	4960.00	feet²

Bottom Floor		Floor Area:	
	Floor Area:	6,790.00	feet²
	Diaphragm Area:	6790.00	feet²

Basement Floor		Floor Area:	
	Floor Area:	0.00	feet²
	Diaphragm Area:	0.00	feet²

		Floor Area:	
	Floor Area:	0.00	feet²
	Diaphragm Area:	0.00	feet²

Roof Level:	23,493	lbs	Timber Framed	22%
Floor	17,745	lbs	Timber Framed	16%
Second Floor	52,080	lbs	Timber Framed	48%
Bottom Floor	14,805	lbs	Timber Framed	14%
Basement Floor		lbs	Concrete	0%
	108,123	lbs Total Floor Weight		

Roof Level:				
Thrid Floor	Level Wall Height:	8.00	feet	Timber Framed
Second Floor	Level Wall Height:	11.44	feet	Timber Framed
First Floor	Level Wall Height:	10.54	feet	Timber Framed
Basement Floor	Level Wall Height:	0.00	feet	Concrete

Roof Level:	Level Wall Weight:	10,221	lbs (Assumes full parameter wall lengths)
Thrid Floor	Level Wall Weight:	27,121	lbs (Assumes full parameter wall lengths + 10 PSF /SF of floor area)
Second Floor	Level Wall Weight:	78,831	lbs (Assumes full parameter wall lengths + 10 PSF /SF of floor area)
Bottom Floor	Level Wall Weight:	41,031	lbs (Assumes full parameter wall lengths + 10 PSF /SF of floor area)
Basement Floor	Level Wall Weight:	-	lbs (Assumes full parameter wall lengths + 10 PSF /SF of floor area)
		157,203	lbs Total Wall Weight
		265,326	lbs Total Building Weight, to Basement Slab

WIND & SEISMIC LOADING:

Building Weights:

Roof Dead Load:			
Roofing:	(Shingles)	1.50	psf
Deck Pavers		0.00	psf
30 lb Building Paper:		0.30	psf
1/2" Plywood Sheathing:		1.50	psf
Roof Framing: (Timber Framed)		5.00	psf
Batt Insulation:		0.50	psf
5/8 Gypsum Ceiling:		2.00	psf
HVAC Equipment & Ducting:		0.50	psf
Misc.:		1.00	psf
		12.3	psf

Floor Dead Load: (Timber Framed)			
Floor Covering: (Carpet/Linoleum)		1.50	psf
3/4" Plywood Sheeting:		1.50	psf
Floor Joists: (TJI's or 2x12's)		3.50	psf
Batt Insulation:		0.50	psf
5/8" Gypsum Ceiling:		2.00	psf
HVAC Equipment & Ducting:		0.50	psf
Misc.:		1.00	psf
		10.50	psf

Floor Dead Load: (Elevated Concrete Slab)			
Floor Covering: (Carpet/Linoleum)		1.50	psf
3/4" Plywood Sheeting:		0.00	psf
Concrete Slab:	(@ 150 pcf)	75.00	psf
Slab Depth =	6 inches		
Rigid Insulation:		0.00	psf
5/8" Gypsum Ceiling:		0.00	psf
HVAC Equipment & Ducting:		0.00	psf
Misc.:		0.00	psf
		76.50	psf

Wall Dead Load: (Timber Framed)			
1/2" Siding:		1.50	psf
1/2" Sheeting:		1.50	psf
2x6 Timber Framing:		1.00	psf
Batt Insulation:		0.50	psf
1/2" Gypsum Wallboard:		2.20	psf
Misc.:		1.00	psf
		7.7	psf
	Use in design:	10	psf of floor area

Wall Dead Load: (Concrete)			
Rigid Insulation:		0.00	psf
1/2" Gypsum Wallboard:		0.00	psf
Concrete Wall & 8" Thickness	(@ 150 pcf)	100.00	psf
Wall Thickness =	8 inches		
2x6 Timber Framing:		1.00	psf
Brick Facing:		0.0	psf
Misc.:		1.00	psf
		102	psf

WIND LOADING:

I	=	1.00	Importance Factor
V	=	110.00 MPH	Basic Wind Speed
K_{ZT}	=	1.38	Per City of Seattle
h_e	=	34.00 FT	Height to Eave
h_r	=	36.00 FT	Height to Ridge
B	=	82.67 FT	Building Length
L	=	83.25 FT	Building Width

Roof Level:	Ave. Elev. =	29.98	q_z =	24.21 psf *
Thrid Floor	8.00 feet	Elev. =	21.98	q_z =	24.21 psf *
Second Floor	11.44 feet	Elev. =	10.54	q_z =	24.21 psf *
Bottom Floor	10.54 feet	Elev. =	0.00	q_z =	24.21 psf *
Basement Floor	0.00 feet	Elev. =	0.00	q_z =	24.21 psf *
	0.00 feet	Elev. =	0.00	q_z =	psf

* See the attached Wind Loading Calculations

Wind Loading: Vx

		PSF		B		AREA		Wind Loading		
Roof	=	24.21	x	83.25	x	4.00	=	**2.34** KIPS	Timber Framed
Floor	=	24.21	x	83.25	x	9.72	=	**5.70** KIPS	Timber Framed
Second Floor	=	24.21	x	83.25	x	10.99	=	**6.44** KIPS	Timber Framed
Bottom Floor	=	24.21	x	83.25	x	5.27	=	**3.09** KIPS	Timber Framed
Basement Floor	=	24.21	x	83.25	x	-	=	**-** KIPS	Concrete
	=	0.00	x	83.25	x	-	=	**-** KIPS	

Wind Loading: Vy

		PSF		L		AREA		Wind Loading		
Roof	=	24.21	x	82.67	x	4.00	=	**8.01** KIPS	Timber Framed
Floor	=	24.21	x	82.67	x	9.72	=	**19.45** KIPS	Timber Framed
Second Floor	=	24.21	x	82.67	x	10.99	=	**22.00** KIPS	Timber Framed
Bottom Floor	=	24.21	x	82.67	x	5.27	=	**10.55** KIPS	Timber Framed
Basement Floor	=	24.21	x	82.67	x	-	=	**-** KIPS	Concrete
	=	0.00	x	82.67	x	-	=	**-** KIPS	

SEISMIC LOADING:

Mapped Acceleration Parameters:

S_1 =	0.508	See the attached results from the USGS report	R_c =	5.0	Concrete Structure - ASCE 7-10, Table 12.2-1 (A-1)
S_s =	1.466	See the attached results from the USGS report	$C_{d,w}$ =	4.00	Wood Framed Structure - ASCE 7-10, Table 12.2-1 (A-13)
S_{MS} =	1.759	See the attached results from the USGS report	$C_{d,c}$ =	5.00	Concrete Structure - ASCE 7-10, Table 12.2-1 (A-1)
S_{M1} =	0.000	See the attached results from the USGS report	$\Omega_{o,w}$ =	3.00	Wood Framed Structure - ASCE 7-10, Table 12.2-1 (A-13)
F_a =	1.20	See the attached results from the USGS report	$\Omega_{o,c}$ =	2.50	Concrete Structure - ASCE 7-10, Table 12.2-1 (A-1)
F_v =	0.00	See the attached results from the USGS report	ρ =	1.00	ASCE 7-10, Section 12.3.4.2 Redundancy Factor for Seismic Design Categories D through F
S_{DS} =	1.173	See the attached results from the USGS report	C_t =	0.02	ASCE 7-10, Table 12.8-2 Values of Approximate Period Parameters C & x
S_{D1} =	0	See the attached results from the USGS report	C_{t1} =	0.016	ASCE 7-10, Table 12.8-2 Values of Approximate Period Parameters C & x b (Concrete)
R_w =	6.5	Wood Framed Structure - ASCE 7-10, Table 12.2-1 (A-13)	x =	0.75	ASCE 7-10, Table 12.8-2 Values of Approximate Period Parameters C & x
I =	1.00	Importance Factor	x_1 =	0.90	ASCE 7-10, Table 12.8-2 Values of Approximate Period Parameters C & x (Concrete)

Seismic Base Shear:

For Wood Framed Story Levels:

$V = C_sW$ ASCE 7-10, Equation (12.8-1)

$C_s = S_{ds} / (R/I) \leq S_{D1} / T(R/I)$ or $S_{D1}T_L / T^2(R/I) \geq 0.01$ ASCE 7-10, Equations (12.8-2, 3 & 4)

Where: $T = T_a = C_th_n^x$ (Approximate Fundamental Period)

T_L = 6.00 ASCE 7-10, Section 11.4.5

Where:

C_t = 0.02 ASCE 7-10,Table 12.8-2

x = 0.75 ASCE 7-10,Table 12.8-2

T = 0.294 For h = 36.00 feet

Therefore k = 1.00 ASCE 7-10, 12.8.3

Cs = 0.1805 ≤ 0.000 or 0.000 ≥ 1.00

For Concrete Story Levels:

$V = C_sW$ ASCE 7-10, Equation (12.8-1)

$C_s = S_{ds} / (R/I) \leq S_{D1} / T(R/I)$ or $S_{D1}T_L / T^2(R/I) \geq 0.01$ ASCE 7-10, Equations (12.8-2, 3 & 4)

Where: $T = T_a = C_th_n^x$ (Approximate Fundamental Period)

T_L = 0.00 ASCE 7-10, Section 11.4.5 0.178172983

Where:

C_{t1} = 0.016 ASCE 7-10,Table 12.8-2 (For Concrete)

x_1 = 0.90 ASCE 7-10,Table 12.8-2 (For Concrete)

T = 0.403 For h = 36.00 feet

Therefore k = 0.00 ASCE 7-10, 12.8.3

Cs = 0.2346 ≤ 0.000 or 0.000 ≥ 1.00

Vertical Distribution of Shear forces

$F_x = C_{vx}V$ Where:

$C_{vx} = w_xh_x^k / \text{sum } w_ih_i^k$ ASCE 7-10, Equation (12.8-3)

V = **0.180** W **For Timber Framed Story Levels**

0.2346 W **For Concrete Story Level**

Level	Floor to Floor Height	Height	k*	hx^k	w_x (lbs)	$w_xh_x^k$	C_{vx}	F_x	Story Shear	V
Roof		29.98 feet	1.00	29.98	47,274.01	1,417,275	0.34	16.34	0	8.53
Floor	8.00	21.98 feet	1.00	21.98	70,720.90	1,554,445	0.37	17.92	16.34	12.76
Second Floor	11.44	10.54 feet	1.00	10.54	112,011.06	1,180,597	0.28	13.61	34.27	20.21
Bottom Floor	10.54	0.00 feet	1.00	-	35,320.50	-	-	0.00	47.88	6.37
Basement Floor	0.00	0.00 feet	1.00	-	-	-	-	0.00	47.88	-
	0.00	0.00 feet	-	-		-	-	0.00	0	-
Totals					**265,326**	4,152,317	100%	47.88	47.88	

$F_x = C_{vx}V$ 0.180 x 265,326 = 47,881.22 lbs

Base Shear = **47.88** KIPS

Diaphragm Forces

Sum F_i	Sum W_i	P_{px} (Kips)
16.34	47.27	16.34
17.92	70.72	17.92
13.61	112.01	13.61
0.00	35.32	-
0.00	-	-
0.00	-	-

Vx & Vy

		Seismic Load: (Kips)
Roof	Level:	- KIPS
Floor	Level:	16.34 KIPS
Second Floor	Level:	34.27 KIPS
Bottom Floor	Level:	47.88 KIPS
Basement Floor	Level:	47.88 KIPS
0	Level:	- KIPS

Comparison of Design Loads for Wind & Seismic

USE WORST CASE: WIND OR SEISMIC (Figures highlighted in **RED** indicate values used in the design.)

Vy Design Shear in the "Y" Direction

		Wind:	**Seismic:**	**DesignShear Load:**	
Roof	Level:	2.34 KIPS	**8.53** KIPS	**8.53** KIPS	Roof
Floor	Level:	5.70 KIPS	**12.76** KIPS	**21.29** KIPS	Floor
Second Floor	Level:	6.44 KIPS	**20.21** KIPS	**41.51** KIPS	Second Floor
Bottom Floor	Level:	3.09 KIPS	**6.37** KIPS	**47.88** KIPS	Bottom Floor
Basement Floor	Level:	- KIPS	**-** KIPS	**-** KIPS	Basement Floor

Vx Design Shear in the "X" Direction

Roof	Level:	8.01 KIPS	**8.53** KIPS	**8.53** KIPS	Roof
Floor	Level:	19.45 KIPS	**12.76** KIPS	**21.29** KIPS	Floor
Second Floor	Level:	22.00 KIPS	**20.21** KIPS	**41.51** KIPS	Second Floor
Bottom Floor	Level:	10.55 KIPS	**6.37** KIPS	**47.88** KIPS	Bottom Floor
Basement Floor	Level:	- KIPS	**-** KIPS	**-** KIPS	Basement Floor

Diaphragm Shear:

	Design Load:		Width (Feet)	Loading (Kips)	Length (Feet)	Loading (Kips)	Design Loading (PLF)	Allowable Diaphragm Loading	
	Vy	Design Shear in the "Y" Direction							
Roof Level:	**8.53**	KIPS	29.00	147.09	47.00	90.76	147	180	3/8 Structural 1 With 10d Nails @ 6" @ Edges & 6" Field
3rd Floor Level:	**12.76**	KIPS	29.00	220.04	47.00	135.77	220	280	15/32" Structural 1 With 10d Nails @ 6" @ Edges & 6" Field
2nd Floor Level:	**6.44**	KIPS	29.00	111.06	47.00	68.53	111	180	3/8 Structural 1 With 10d Nails @ 6" @ Edges & 6" Field
1st Floor Level:	**3.09**	KIPS	29.00	53.26	47.00	32.86	53	180	3/8 Structural 1 With 10d Nails @ 6" @ Edges & 6" Field
Basement Level:	**-**	KIPS	29.00	0.00	47.00	0.00	0		
									Attachment method
	Vx	Design Shear in the "X" Direction							
Roof Level:	**8.53**	KIPS	49.00	87.05	47.00	90.76	87	180	3/8 Structural 1 With 10d Nails @ 6" @ Edges & 6" Field
3rd Floor Level:	**19.45**	KIPS	49.00	198.51	47.00	206.96	199	280	15/32" Structural 1 With 10d Nails @ 6" @ Edges & 6" Field
2nd Floor Level:	**22.00**	KIPS	49.00	224.45	47.00	234.00	224	280	15/32" Structural 1 With 10d Nails @ 6" @ Edges & 6" Field
1st Floor Level:	**10.55**	KIPS	49.00	107.63	47.00	112.21	108	180	3/8 Structural 1 With 10d Nails @ 6" @ Edges & 6" Field
Basement Level:	**-**	KIPS	49.00	0.00	47.00	0.00	0		
									Attachment method

SHEAR WALL DESIGNATIONS, LENGTHS & SHEAR LOADING:

Main Level Walls

SHEAR WALL DESIGNATION		LENGTH	Height	Aspect Ratio	Use of Table 2306.4.1	Tributary Area Supported by Wall
SWX	301	17.00 FT	8.00 FT	0.5	</= 2.5 Check - O.K.	550 SQFT
SWX	302	4.83 FT	8.00 FT	1.7	</= 2.5 Check - O.K.	350 SQFT
SWX	303	10.00 FT	8.00 FT	0.8	</= 2.5 Check - O.K.	505 SQFT
SWX	304	17.00 FT	8.00 FT	0.5	</= 2.5 Check - O.K.	505 SQFT
		48.83			Total Floor Area:	1,910 SQFT Calculated
						1,910 SQFT Actual

TOTAL SHEAR WALL LENGTH IN "X" DIRECTION: 48.83 FT
TOTAL SHEAR WALL TRIBUTARY AREA IN "X" DIRECTION: 1,910 SQFT

SHEAR WALL DESIGNATION		LENGTH	Height	Aspect Ratio	Use of Table 2306.4.1	Tributary Area Supported by Wall
SWY	301	9.00 FT	8.00 FT	0.9	</= 2.5 Check - O.K.	510 SQFT
SWY	302	3.00 FT	8.00 FT	2.7	> 2.5 < 3.5 Use H2/8	150 SQFT
SWY	303	12.00 FT	8.00 FT	0.7	</= 2.5 Check - O.K.	750 SQFT
SWY	304	9.33 FT	8.00 FT	0.9	</= 2.5 Check - O.K.	500 SQFT
SWY	305	3.00 FT	8.00 FT	2.7	> 2.5 < 3.5 Use 2w/h	150 SQFT
		33.33			Total Floor Area:	1,910 SQFT Calculated
						1,910 SQFT Actual

TOTAL SHEAR WALL LENGTH IN "Y" DIRECTION: 33.33 FT
TOTAL SHEAR WALL TRIBUTARY AREA IN "Y" DIRECTION: 1,910 SQFT

Shear Wall Loading-Nailing Schedule				See Plans for Shear Wall Schedule Information
Capacity		Mark	x 1.4 for Wind	Per IBC Section 2306.4.1
270	PLF	P1-8-6	378	3/8" Structural 1 Panel with 8d nails @ 6" edges & 6" fiel
360	PLF	P1-8-4	504	3/8" Structural 1 Panel with 8d nails @ 4" edges & 6" fiel
530	PLF	P1-8-3	742	3/8" Structural 1 Panel with 8d nails @ 2.5" edges & 4" fiel
610	PLF	P1-8-2	854	3/8" Structural 1 Panel with 8d nails @ 2" edges & 3" fiel
720	PLF	P2-8-4	1008	(2) 3/8" Structural 1 Panels with 8d nails @ 4" enges & 6" field
1220	PLF	P2-8-2	1708	(2) 3/8" Structural 1 Panels with 8d nails @ 2" edges & 3" field

FORCE IN "X" DIRECTION TO TOP OF WALL

F_{max} = 8.53 kips

FT_{max} = 0.00 kips (F_{max} / Total Tributary Area)

								Wall Loading	Table 2306.4.1 x (2w/h)	Wall Designation
SWX	301	V_{max}	=	F_{max} x FT_{max}	=	2.46 kips	/ Wall Length =	145		P1-8-6
SWX	302	V_{max}	=	F_{max} x FT_{max}	=	1.56 kips	/ Wall Length =	324		P1-8-6
SWX	303	V_{max}	=	F_{max} x FT_{max}	=	2.26 kips	/ Wall Length =	226		P1-8-6
SWX	304	V_{max}	=	F_{max} x FT_{max}	=	2.26 kips	/ Wall Length =	133		P1-8-6
						8.53 kips				

FORCE IN "Y" DIRECTION TO TOP OF WALL

F_{max} = 21.29 kips

FT_{max} = 0.011 kips (F_{max} / Total Tributary Area)

								Wall Loading	Table 2306.4.1 x (2w/h)	Wall Designation
SWY	301	V_{max}	=	F_{max} x FT_{max}	=	5.69 kips	/ Wall Length =	632		P1-8-3
SWY	302	V_{max}	=	F_{max} x FT_{max}	=	1.67 kips	/ Wall Length =	557		P1-8-3
SWY	303	V_{max}	=	F_{max} x FT_{max}	=	8.36 kips	/ Wall Length =	697		P1-8-3
SWY	304	V_{max}	=	F_{max} x FT_{max}	=	5.57 kips	/ Wall Length =	597		P1-8-3
SWY	305	V_{max}	=	F_{max} x FT_{max}	=	1.67 kips	/ Wall Length =	557		P1-8-3
						22.97 kips				

Floor Weight Acting on Shear Wall

SHEAR WALL DESIGNATION		LENGTH	Tributary Area Supported by Wall	Dead Load x 60%	100% Dead Load Per Foot Of Wall	100% Live Load Per Foot Of Wall
SWX	301	17.00 FT	550 FT	3,465	339.7	809
SWX	302	4.83 FT	350 FT	2,205	760.9	1,812
SWX	303	10.00 FT	505 FT	3,182	530.3	1,263
SWX	304	17.00 FT	505 FT	3,182	311.9	743

SHEAR WALL DESIGNATION		LENGTH	Floor Length Supported by Wall	Dead Load x 60%	100% Dead Load	100% Live Load
SWY	301	9.00 FT	510 FT	3,213	595.0	1,417
SWY	302	3.00 FT	150 FT	945	525.0	1,250
SWY	303	12.00 FT	750 FT	4,725	656.3	1,563
SWY	304	9.33 FT	500 FT	3,150	562.7	1,340
SWY	305	3.00 FT	150 FT	945	525.0	1,250

SHEAR WALL DESIGNATIONS, LENGTHS & SHEAR LOADING:

Bottom Level Walls

SHEAR WALL DESIGNATION		LENGTH	Height	Aspect Ratio	Use of Table 2306.4.1	Tributary Area Supported by Wall
SWX	201	17.00 FT	11.44 FT	0.7	</= 2.5 Check - O.K.	285 SQFT
SWX	202	17.00 FT	11.44 FT	0.7	</= 2.5 Check - O.K.	285 SQFT
SWX	203	12.00 FT	11.44 FT	1.0	</= 2.5 Check - O.K.	275 SQFT
SWX	204	15.00 FT	11.44 FT	0.8	</= 2.5 Check - O.K.	280 SQFT
SWX	205	15.00 FT	11.44 FT	0.8	</= 2.5 Check - O.K.	280 SQFT
SWX	206	17.00 FT	11.44 FT	0.7	</= 2.5 Check - O.K.	285 SQFT
		93.00			Total Floor Area:	1,690 SQFT Calculated
						1,690.00 SQFT Actual

TOTAL SHEAR WALL LENGTH IN "X" DIRECTION: **93.00** FT

TOTAL SHEAR WALL TRIBUTARY AREA IN "X" DIRECTION: **1,690** SQFT

SHEAR WALL DESIGNATION		LENGTH	Height	Aspect Ratio	Use of Table 2306.4.1	Tributary Area Supported by Wall
SWY	201	12.00 FT	11.44 FT	1.0	</= 2.5 Check - O.K.	300 SQFT
SWY	202	2.50 FT	11.44 FT	4.6	> 2.5 < 3.5 Use 2w/h	100 SQFT
SWY	203	15.00 FT	11.44 FT	0.8	</= 2.5 Check - O.K.	425 SQFT
SWY	204	3.00 FT	11.44 FT	3.8	> 2.5 < 3.5 Use 2w/h	200 SQFT
SWY	205	12.50 FT	11.44 FT	0.9	</= 2.5 Check - O.K.	325 SQFT
SWY	206	15.00 FT	11.44 FT	0.8	</= 2.5 Check - O.K.	340 SQFT
		60.00			Total Floor Area:	1,690 SQFT Calculated
						1,690.00 SQFT Actual

TOTAL SHEAR WALL LENGTH IN "Y" DIRECTION: **60.00** FT
TOTAL SHEAR WALL TRIBUTARY AREA IN "Y" DIRECTION: **1,690** SQFT

Shear Wall Loading-Nailing Schedule				See Plans for Shear Wall Schedule Information
Capacity		Mark	x 1.4 for Wind	Per IBC Section 2306.4.1
270	PLF	P1-8-6	378	3/8" Structural 1 Panel with 8d nails @ 6" edges & 6" fiel
360	PLF	P1-8-4	504	3/8" Structural 1 Panel with 8d nails @ 4" enges & 6" field
530	PLF	P1-8-3	742	3/8" Structural 1 Panel with 8d nails @ 2.5" edges & 4" field
610	PLF	P1-8-2	854	3/8" Structural 1 Panel with 8d nails @ 2" edges & 3" field
720	PLF	P2-8-4	1008	(2) 3/8" Structural 1 Panels with 8d nails @ 4" enges & 6" field
1220	PLF	P2-8-2	1708	(2) 3/8" Structural 1 Panels with 8d nails @ 2" edges & 3" field

FORCE IN "X" DIRECTION TO TOP OF WALL

F_{max} = 21.29 kips
FT_{max} = 0.01 kips (F_{max} / Total Tributary Area)

								Wall Loading	Table 2306.4.1 x (2w/h)	Wall Designation
SWX	201	V_{max}	=	$F_{max} \times FT_{max}$	=	3.59 kips	/ Wall Length =	211		P1-8-6
SWX	202	V_{max}	=	$F_{max} \times FT_{max}$	=	3.59 kips	/ Wall Length =	211		P1-8-6
SWX	203	V_{max}	=	$F_{max} \times FT_{max}$	=	3.46 kips	/ Wall Length =	289		P1-8-6
SWX	204	V_{max}	=	$F_{max} \times FT_{max}$	=	3.53 kips	/ Wall Length =	235		P1-8-6
SWX	205	V_{max}	=	$F_{max} \times FT_{max}$	=	3.53 kips	/ Wall Length =	235		P1-8-6
SWX	206	V_{max}	=	$F_{max} \times FT_{max}$	=	3.59 kips	/ Wall Length =	211		P1-8-6
						21.29 kips				

FORCE IN "Y" DIRECTION TO TOP OF WALL

F_{max} = 41.51 kips
FT_{max} = 0.025 kips (F_{max} / Total Tributary Area)

								Wall Loading	Table 2306.4.1 x (2w/h)	Wall Designation
SWY	201	V_{max}	=	$F_{max} \times FT_{max}$	=	7.37 kips	/ Wall Length =	614		P1-8-3
SWY	201A	V_{max}	=	$F_{max} \times FT_{max}$	=	2.46 kips	/ Wall Length =	982		P2-8-4
SWY	203	V_{max}	=	$F_{max} \times FT_{max}$	=	10.44 kips	/ Wall Length =	696		P1-8-3
SWY	204	V_{max}	=	$F_{max} \times FT_{max}$	=	4.91 kips	/ Wall Length =	1637		P2-8-2
SWY	205	V_{max}	=	$F_{max} \times FT_{max}$	=	7.98 kips	/ Wall Length =	639		P1-8-3
SWY	206	V_{max}	=	$F_{max} \times FT_{max}$	=	8.35 kips	/ Wall Length =	557		P1-8-3
						41.51 kips				

Floor Weight Acting on Shear Wall

SHEAR WALL DESIGNATION		LENGTH	Tributary Area Supported by Wall	Dead Load x 60%	100% Dead Load Per Foot Of Wall	100% Live Load Per Foot Of Wall
SWX	201	17.00 FT	285 FT	1,796	176.0	419
SWX	202	17.00 FT	285 FT	1,796	176.0	419
SWX	203	12.00 FT	275 FT	1,733	240.6	573
SWX	204	15.00 FT	280 FT	1,764	196.0	467
SWX	205	15.00 FT	280 FT	1,764	196.0	467
SWX	206	17.00 FT	285 FT	1,796	176.0	419

SHEAR WALL DESIGNATION		LENGTH	Floor Length Supported by Wall	Dead Load x 60%	100% Dead Load	100% Live Load
SWY	201	12.00 FT	300 FT	1,890	262.5	625
SWY	201A	2.50 FT	100 FT	630	420.0	1,000
SWY	202	15.00 FT	425 FT	2,678	297.5	708
SWY	203	3.00 FT	200 FT	1,260	700.0	1,667
SWY	204	12.50 FT	325 FT	2,048	273.0	650
SWY	205	15.00 FT	340 FT	2,142	238.0	567
SWY	206	60.00 FT	1690 FT	10,647	295.8	704

SHEAR WALL DESIGNATIONS, LENGTHS & SHEAR LOADING:

Garage Level Walls

SHEAR WALL DESIGNATION		LENGTH	Height	Aspect Ratio	Use of Table 2306.4.1	Tributary Area Supported by Wall
SWX	101	22.50 FT	10.54 FT	0.5	</= 2.5 Check - O.K.	1785 SQFT
SWX	102	16.50 FT	10.54 FT	0.6	</= 2.5 Check - O.K.	525 SQFT
SWX	103	10.50 FT	10.54 FT	1.0	</= 2.5 Check - O.K.	450 SQFT
SWX	104	86.00 FT	10.54 FT	0.1	</= 2.5 Check - O.K.	2200 SQFT
		135.50			Total Floor Area:	4,960 SQFT Calculated
						4,960.00 SQFT Actual

TOTAL SHEAR WALL LENGTH IN "X" DIRECTION: **135.50** FT
TOTAL SHEAR WALL TRIBUTARY AREA IN "X" DIRECTION: **4,960** SQFT

SHEAR WALL DESIGNATION		LENGTH	Height	Aspect Ratio	Use of Table 2306.4.1	Tributary Area Supported by Wall
SWY	101	69.00 FT	10.54 FT	0.2	</= 2.5 Check - O.K.	2500 SQFT
SWY	102	42.00 FT	10.54 FT	0.3	</= 2.5 Check - O.K.	1910 SQFT
SWY	103	9.00 FT	10.54 FT	1.2	</= 2.5 Check - O.K.	550 SQFT
		120.00			Total Floor Area:	4,960 SQFT Calculated
						4,960.00 SQFT Actual

TOTAL SHEAR WALL LENGTH IN "Y" DIRECTION: **120.00** FT
TOTAL SHEAR WALL TRIBUTARY AREA IN "Y" DIRECTION: **4,960** SQFT

Shear Wall Loading-Nailing Schedule				See Plans for Shear Wall Schedule Information
Capacity		Mark	x 1.4 for Wind	Per IBC Section 2306.4.1
270	PLF	P1-8-6	378	3/8" Structural 1 Panel with 8d nails @ 6" edges & 6" field
360	PLF	P1-8-4	504	3/8" Structural 1 Panel with 8d nails @ 4" enges & 6" field
530	PLF	P1-8-3	742	3/8" Structural 1 Panel with 8d nails @ 2.5" edges & 4" fiel
610	PLF	P1-8-2	854	3/8" Structural 1 Panel with 8d nails @ 2" edges & 3" fiel
720	PLF	P2-8-4	1008	(2) 3/8" Structural 1 Panels with 8d nails @ 4" enges & 6" fiel
1220	PLF	P2-8-2	1708	(2) 3/8" Structural 1 Panels with 8d nails @ 2" edges & 3" fiel

FORCE IN "X" DIRECTION TO TOP OF WALL

F_{max} = 41.51 kips
FT_{max} = 0.01 kips (F_{max} / Total Tributary Area)

								Wall Loading	Table 2306.4.1 x (2w/h)	Wall Designation
SWX	101	V_{max}	=	$F_{max} \times FT_{max}$	=	14.94 kips	/ Wall Length =	664		CONCRETE
SWX	102	V_{max}	=	$F_{max} \times FT_{max}$	=	4.39 kips	/ Wall Length =	266		P1-8-6
SWX	103	V_{max}	=	$F_{max} \times FT_{max}$	=	3.77 kips	/ Wall Length =	359		P1-8-6
SWX	104	V_{max}	=	$F_{max} \times FT_{max}$	=	18.41 kips	/ Wall Length =	214		CONCRETE
						41.51 kips				

FORCE IN "Y" DIRECTION TO TOP OF WALL

F_{max} = 47.88 kips
FT_{max} = 0.010 kips (F_{max} / Total Tributary Area)

								Wall Loading	Table 2306.4.1 x (2w/h)	Wall Designation
SWY	101	V_{max}	=	$F_{max} \times FT_{max}$	=	24.13 kips	/ Wall Length =	350		CONCRETE
SWY	102	V_{max}	=	$F_{max} \times FT_{max}$	=	18.44 kips	/ Wall Length =	439		CONCRETE
SWY	103	V_{max}	=	$F_{max} \times FT_{max}$	=	5.31 kips	/ Wall Length =	590		CONCRETE
						47.88 kips				

Floor Weight Acting on Shear Wall

SHEAR WALL DESIGNATION		LENGTH	Tributary Area Supported by Wall	Dead Load x 60%	100% Dead Load Per Foot Of Wall	100% Live Load Per Foot Of Wall
SWX	101	22.50 FT	1785 FT	11,246	833.0	1,983
SWX	102	16.50 FT	525 FT	3,308	334.1	795
SWX	103	10.50 FT	450 FT	2,835	450.0	1,071
SWX	104	86.00 FT	2200 FT	13,860	268.6	640

SHEAR WALL DESIGNATION		LENGTH	Floor Length Supported by Wall	Dead Load x 60%	100% Dead Load	100% Live Load
SWY	101	69.00 FT	2500 FT	15,750	380.4	906
SWY	102	42.00 FT	1910 FT	12,033	477.5	1,137
SWY	103	9.00 FT	550 FT	3,465	641.7	1,528

Shear Wall Designation		Length	Height	Shear Loading	Shear Wall Type	Overturning Moment	60% Dead Load	Resisting Moment	Strap Load From Above	Strap/Holdown Load	Strap/Holdown Type	Uplift Resistance Provided	Notes:
SWX	301	17.00	8.00	145	**P1-8-6**	19,653	3,465	29,453		952	**MST37**	2,140	**(14) 16d NAILS x 2.5" - DBL 2 x 6 END STUDS**
SWX	302	4.83	8.00	324	**P1-8-6**	12,506	2,205	5,325		2,133	**MST37**	2,140	**(14) 16d NAILS x 2.5" - DBL 2 x 6 END STUDS**
SWX	303	10.00	8.00	226	**P1-8-6**	18,045	3,182	15,908		1,486	**MST37**	2,140	**(14) 16d NAILS x 2.5" - DBL 2 x 6 END STUDS**
SWX	304	17.00	8.00	133	**P1-8-6**	18,045	3,182	27,043		874	**MST37**	2,140	**(14) 16d NAILS x 2.5" - DBL 2 x 6 END STUDS**
SWY	301	9.00	8.00	632	**P1-8-3**	45,486	3,213	14,459		4,697	**MST60**	5,405	**(46) 16d NAILS x 2.5" - DBL 2 x 6 END STUDS**
SWY	302	3.00	8.00	557	**P1-8-3**	13,378	945	1,418		4,144	**MST60**	5,405	**(46) 16d NAILS x 2.5" - DBL 2 x 6 END STUDS**
SWY	303	12.00	8.00	697	**P1-8-3**	66,891	4,725	28,350		5,180	**MST60**	5,405	**(46) 16d NAILS x 2.5" - DBL 2 x 6 END STUDS**
SWY	304	9.33	8.00	597	**P1-8-3**	44,594	3,150	14,695		4,442	**MST60**	5,405	**(46) 16d NAILS x 2.5" - DBL 2 x 6 END STUDS**
SWY	305	3.00	8.00	557	**P1-8-3**	13,378	945	1,418		4,144	**MST60**	5,405	**(46) 16d NAILS x 2.5" - DBL 2 x 6 END STUDS**
SWX	201	17.00	11.44	211	**P1-8-6**	41,080	6,930	58,905	952	2,961	**MST60**	5,405	**(46) 16d NAILS x 2.5" - DBL 2 x 6 END STUDS**
SWX	202	17.00	11.44	211	**P1-8-6**	41,080	4,410	37,485	2,133	4,290	**MST60**	5,405	**(46) 16d NAILS x 2.5" - DBL 2 x 6 END STUDS**
SWX	203	12.00	11.44	289	**P1-8-6**	39,639	6,363	38,178	1,486	4,259	**MST60**	5,405	**(46) 16d NAILS x 2.5" - DBL 2 x 6 END STUDS**
SWX	204	15.00	11.44	235	**P1-8-6**	40,359	6,363	47,723	874	3,141	**MST60**	5,405	**(46) 16d NAILS x 2.5" - DBL 2 x 6 END STUDS**
SWX	205	15.00	11.44	235	**P1-8-6**	40,359	-	-	-	2,691	**MST60**	5,405	**(46) 16d NAILS x 2.5" - DBL 2 x 6 END STUDS**
SWX	206	17.00	11.44	211	**P1-8-6**	41,080	3,213	27,311	4,697	6,924	**MST72**	7,670	**(46) 16d NAILS x 2.5" - DBL 2 x 6 END STUDS**
SWY	201	12.00	11.44	614	**P1-8-3**	84,292	6,426	38,556	4,697	11,186	**MDCST48**	11,312	**(44) 0.25" Dia. x 3" SDS SCREWS - 4 x 6 END STUDS**
SWY	202	2.50	11.44	982	**P2-8-4**	28,097	4,158	5,198	4,697	14,273	**DBL MDCST48**	22,624	**(44) 0.25" Dia. x 3" SDS SCREWS - 4 x 6 END STUDS**
SWY	203	15.00	11.44	696	**P1-8-3**	119,413	5,670	42,525	4,144	11,727	**DBL MDCST48**	22,624	**(44) 0.25" Dia. x 3" SDS SCREWS - 4 x 6 END STUDS**
SWY	204	3.00	11.44	1637	**P2-8-2**	56,194	7,875	11,813	5,180	21,287	**DBL MDCST48**	22,624	**(44) 0.25" Dia. x 3" SDS SCREWS - 4 x 6 END STUDS**
SWY	205	12.50	11.44	639	**P1-8-3**	91,316	4,095	25,594	4,442	11,420	**DBL MDCST48**	22,624	**(44) 0.25" Dia. x 3" SDS SCREWS - 4 x 6 END STUDS**
SWY	206	15.00	11.44	557	**P1-8-3**	95,531	945	7,088	4,144	10,450	**MDCST48**	11,312	**(44) 0.25" Dia. x 3" SDS SCREWS - 4 x 6 END STUDS**
SWX	101	22.50	10.54	664	**CONCRETE**	157,442	10,395	116,944	2,961	9,496	**Concrete Stem Wall**		
SWX	102	16.50	10.54	266	**P1-8-6**	46,307	6,615	54,574	4,290	6,695	**HDU11**	8,030	**(30) 0.25" Dia. X 2.5" SDS SCREWS - 4 x 6 END STUDS**
SWX	103	10.50	10.54	359	**P1-8-6**	39,691	9,545	50,109	4,259	7,130	**HDU11**	8,030	**(30) 0.25" Dia. X 2.5" SDS SCREWS - 4 x 6 END STUDS**
SWX	104	86.00	10.54	214	**CONCRETE**	194,047	9,545	410,414	3,141	5,286	**Concrete Stem Wall**		
SWY	101	69.00	10.54	350	**CONCRETE**	254,369	9,639	332,546	11,186	14,733	**Concrete Stem Wall**		
SWY	102	42.00	10.54	439	**CONCRETE**	194,338	7,371	154,791	11,186	15,637	**Concrete Stem Wall**		
SWY	103	9.00	10.54	590	**CONCRETE**	55,961	6,615	29,768	14,273	19,756	**Concrete Stem Wall**		

ROOF SHEATHING DESIGN:

Assumptions:

1. Plywood shall be: **1/2 inch** APA Rated Sheathing Ext.[3]
2. 2 x _ Trusses spaced at: **24** inches o.c.. maximum

Therefore: L_1 = 24
L_2 = 22.5
L_3 = 22.75

3. Plywood spans over three or more supports.
4. Roof Design Live Load: **LL** = **16**
   (See UBC Table 16-C)
5. Roof Design Snow Load: **SL** = **25**

1/2 inch Plywood Properties:

		APA Design Values
t_s	=	**0.5** inches
A	=	**2.884** $inches^2$
I	=	**0.075** $inches^3$
KS	=	**0.267** $inches^4$
lb/Q	=	**4.891** $inches^2$
F_b	=	**1930** PSI
F_s	=	**72** PSI
E_e	=	**1,500,000** PSI
E	=	**1,650,000** PSI

Check Plywood Stresses:

A) Check Bending:

$KS_{Req'd}$ = $W(L_1)^2 / 120(F_b)$ = **0.133** $inches^4$

Where **W** =D.L +Roof L.L. + Roof Snow I 53.3 PSF

$KS_{Furnished}$ = **0.267** $inches^4$

Check - O.K.

B) Check Rolling Shear:

$Fs_{Req'd}$ = $W(L_2) / 20(lb/Q)$ = **8.58** PSI

$Fs_{Furnished}$ = $L_2/360$ = **72** PSI

Check - O.K.

C) Check Shear Deflection:

$D_{maximum}$ = $[WC(t_s)^2(L_2)^2] / (1270\ E_e\ I)$ = **0.0020** inches

$D_{Allowable}$ = $L_2/360$ = **0.0625** inches

Check - O.K.

D) Check Bending Deflection:

$D_{maximum}$ = $[W(L_3)^4] / 1743\ E\ I$ = **0.0039** inches

$D_{Allowable}$ = $L_3/360$ = **0.0632** inches

Check - O.K.

FIRST, SECOND & THIRD FLOOR SHEATHING DESIGN:

Assumptions:

1. Plywood shall be:	**3/4 inch**	APA Rated Strd-I-Floor Ext.1 or 2[3]		
2. 2 x _ floor joists spaced at:		**16**	inches o.c.. maximum	
Therefore:	L_1	=	16	
	L_2	=	12.5	
	L_3	=	12.75	
3. Plywood spans over three or more supports.				
4. Floor Design Live Load:	**LL**	=	**40**	
			(See UBC Table 16-A)	

3/4 inch Plywood Properties:

APA Design Values			
t_s	=	**0.568**	inches
A	=	**2.884**	$inches^2$
I	=	**0.199**	$inches^3$
KS	=	**0.455**	$inches^4$
lb/Q	=	**7.187**	$inches^2$
F_b	=	**1930**	PSI
F_s	=	**72**	PSI
E_e	=	**1,500,000**	PSI
E	=	**1,650,000**	PSI

Check Plywood Stresses:

A) Check Bending:

$KS_{Req'd}$	=	$W(L_1)^2 / 120(F_b)$	=	**0.056** $inches^4$
$KS_{Furnished}$	=			**0.455** $inches^4$
				Check - O.K.

B) Check Rolling Shear:

$Fs_{Req'd}$	=	$W(L_2) / 20(lb/Q)$	=	**4.39** PSI
$Fs_{Furnished}$	=	$L_2/360$	=	**72** PSI
				Check - O.K.

C) Check Shear Deflection:

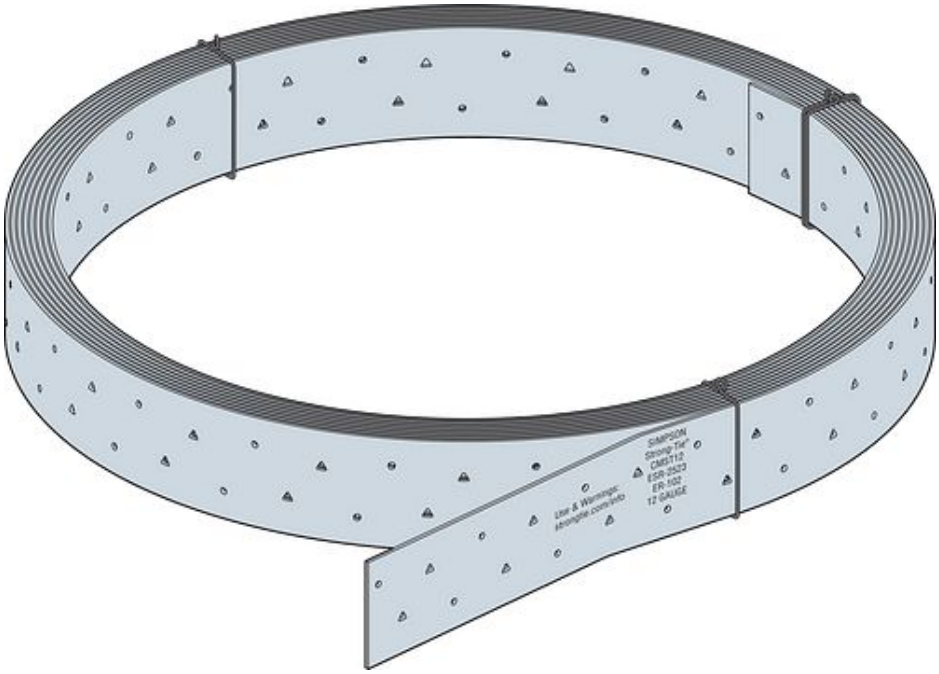
$D_{maximum}$	=	$[WC(t_s)^2(L_2)^2] / (1270\ E_e\ I)$	=	**0.0004**
$D_{Allowable}$	=	$L_2/360$	=	**0.0347**
				Check - O.K.

D) Check Bending Deflection:

$D_{maximum}$	=	$[W(L_3)^4] / 1743\ E\ I$	=	**0.0023** inches
$D_{Allowable}$	=	$L_3/360$	=	**0.0354** inches
				Check - O.K.

CMST Coiled Medium Strap Tie

Select Media



Model No.	Ga.	Length (ft.)	Coating/Material

Coiled Straps

Load Tables

This product's information may differ depending on the category of use. You are currently viewing details related to **Coiled Straps**. You can also view product information related to the category: Lateral Connectors, Ties and Straps for Cold-Formed Steel Construction

Product Details

The CMST is a continuous utility strap designed to resist tension and uplift loads. This coiled strap can be cut to length on the jobsite and used in a double 2x stud or post application. The CMST is ideal for many applications including resisting uplift in a floor-to-floor application and also resisting tension in a horizontal application.

Key Features

- CMSTC option available, which comes with countersunk nail slots for lower profile when installed with 0.148" x 3 1/4" sinkers
- Used for roof uplift and shearwall overturning resistance
- A cost-effective alternative to pre-cut straps

Material

- See table

Finish

- Galvanized (G90)

Installation

- Use all specified fasteners; see General Notes.
- Wood shrinkage after strap installation across horizontal wood members may cause strap to buckle outward.
- Refer to the applicable code for minimum nail penetration and minimum wood edge and end distances.
- The table shows the maximum allowable loads and the nails required to attain them. Fewer nails may be used; reduce the allowable load as shown in the Straps and Ties General Notes.
- For lap splice and alternative nailing information, see tables below.
- The cut length of the strap shall be equal to twice the “End Length” noted in the table plus the clear span dimension.
- Use only every other round hole if the wood tends to split.
- Fill round and triangle holes for loads shown. If wood tends to split, fill only round hole and double the end length listed for full load.

Lap Splicing

Lap splicing of coiled straps can be used to extend standard strap lengths for designing continuous drag elements and diaphragm chord members. The Strap Lap Splices table

Model No.	Ga.	Length (ft.)	Coating/Material

Load Tables

The Allowable Loads for Alternative Nailing table provides information for coiled straps when installed with different nailing schedules. The highest allowable load given for each model is limited by the steel capacity.

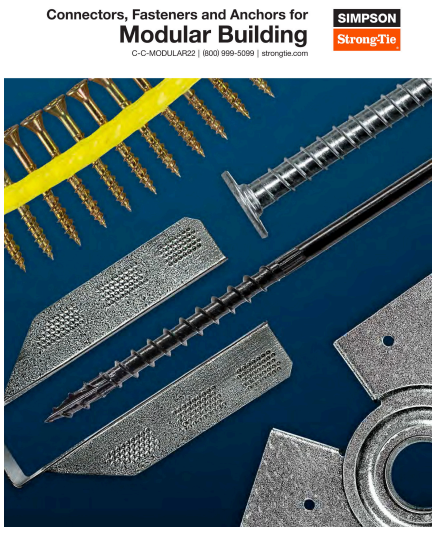
The Engineer/Designer of Record must evaluate and determine the adequacy of the coiled strap's lap splice and alternative nailing applications to meet their design loads.

Related Links

- Wood Construction Connectors Technical and Installation Notes
- Cold-Formed Steel Connectors Technical and Installation Notes
- General Corrosion Information
- Straps and Ties General Notes
- Coiled Strap Calculator

Catalog Pages

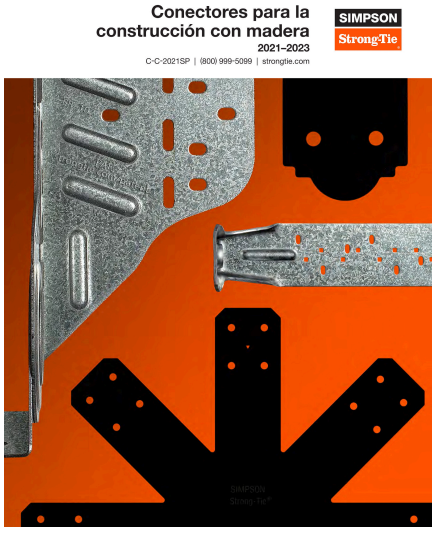
- C-C-2024 (Wood Construction Connectors), pages 283–285
- C-CF-2023 (Connectors for Cold-Formed Steel Construction), page 234



Connectors, Fasteners & Anchors for Modular Building

C-C-MODULAR22 — *Catalog*

A catalog featuring connectors, fasteners and mechanical anchors that are ideal for modular building.



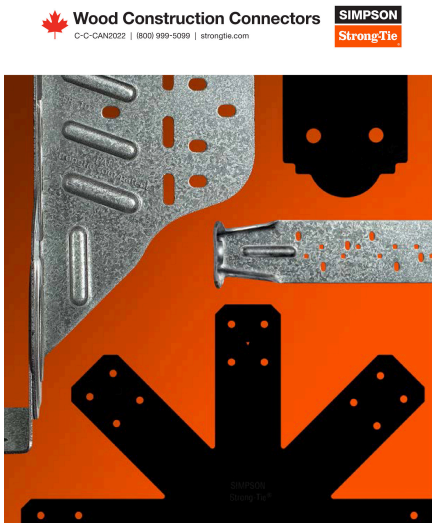
Conectores para la construcción con madera 2021–2023

C-C-2021SP — *Catalog*

El catálogo incluye información sobre nuevos productos, así como especificaciones e instrucciones de instalación aplicables para conectores

SIMPSON StrongTie Model No.	Ga.	Length (ft.)	Coating/Material

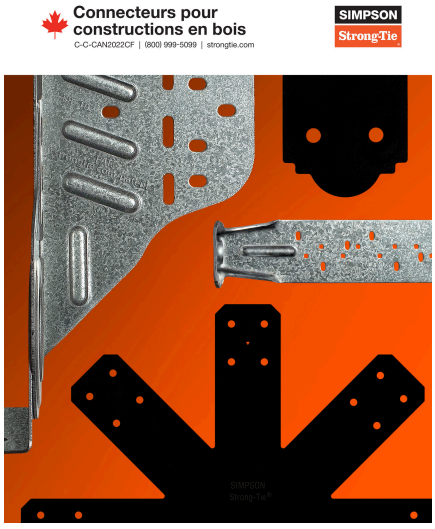
Load Tables



Wood Construction Connectors – Canadian Limit States Design

C-C-CAN2022 — *Catalog*

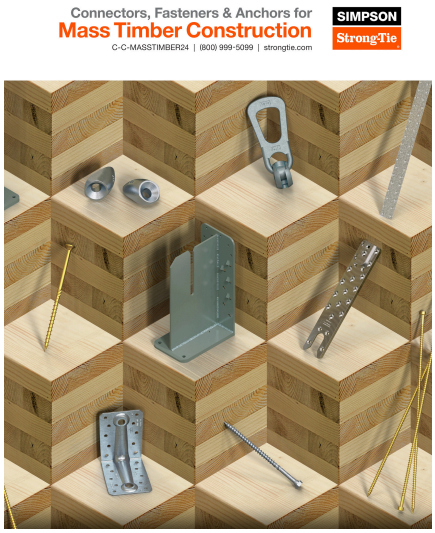
A complete product and application catalogue including load tables and installation information for wood-to-wood, wood-to-concrete and wood-to-masonry connectors.



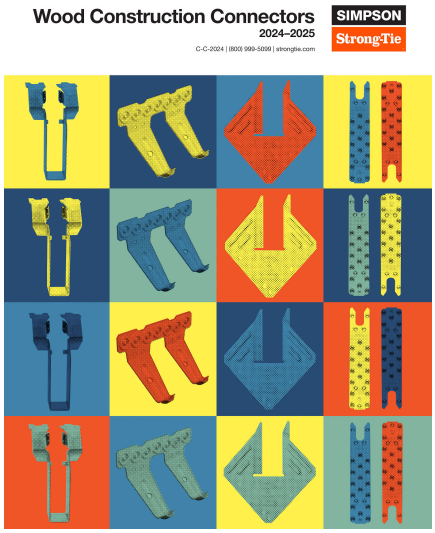
Connecteurs pour constructions en bois — Conception aux états limites canadiens

C-C-CAN2022CF — *Catalog*

Un catalogue complet sur les produits et les applications, y compris de l’information sur l’installation et les tableaux de charge pour les connecteurs bois-à-bois, bois-à-béton et bois-à-maçonnerie.



Connectors, Fasteners & Anchors for Mass Timber Construction



Wood Construction Connectors 2024-2025

C-C-2024 — *Catalog*

A catalog featuring connectors, fasteners and anchors for mass timber and CLT construction solutions.

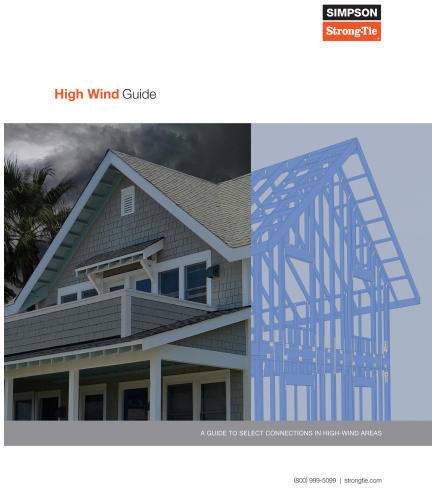
A catalog including new product information as well as any applicable specification and installation instructions for wood-to-wood and wood-to-concrete structural connectors.



Connectors for Cold-Formed Steel Construction Catalog

C-CF-2023 — *Catalog*

A catalog for Connectors for Cold-Formed Steel Construction products and solutions.



High Wind Guide

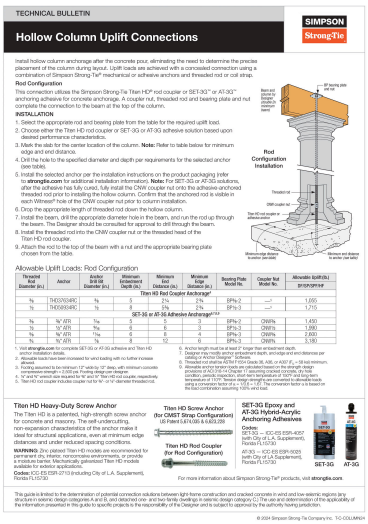
F-C-HWG23 — *Product Guide*

A comprehensive guide to help designers select the most appropriate connections for high-wind regions.



Installer's Pocket Guide

S-C-INSTALL22 — *Product Guide*



Hollow Column Uplift Connections

T-C-COLUMN24 — *Technical Bulletin*

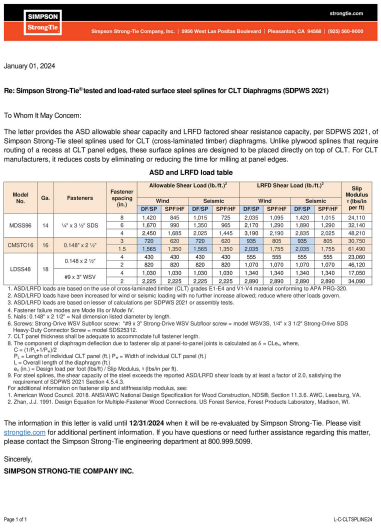
Model No.	Ga.	Length (ft.)	Coating/Material

A manual pocket installation guide for

A technical bulletin providing information on

Load Tables

with a concealed connection using a combination of Simpson Strong-Tie anchors and threaded rods or coil straps.



January 01, 2024

Re: Simpson Strong-Tie® tested and load-rated surface steel splines for CLT Diaphragms (SDPWS 2021)

To Whom It May Concern:

The letter provides the ASD allowable shear capacity and LRFD factored shear resistance capacity, per SDPWS 2021, of Simpson Strong-Tie steel splines used for CLT (cross-laminated timber) diaphragms. Unlike plywood splines that require routing of a recess at CLT panel edges, these surface splines are designed to be placed directly on top of CLT. For CLT manufacturers, it reduces costs by eliminating or reducing the time for milling at panel edges.

ASD and LRFD load table

Model No.	Ga.	Fasteners	Fastener spacing (in.)	Allowable Shear Load (lb./ft.)[2]				LRFD Shear Load (lb./ft.)[2]				Slip Modulus (lb/in per ft)
				Wind		Seismic		Wind		Seismic		
				DF/SP	SPF/HF	DF/SP	SPF/HF	DF/SP	SPF/HF	DF/SP	SPF/HF	
MDSS96	14	¼" x 3 ½" SDS	8	1,420	845	1,015	725	2,035	1,095	1,420	1,015	24,110
			6	1,870	990	1,350	965	2,170	1,295	1,890	1,295	32,140
			4	2,450	1,685	2,025	1,445	3,190	2,190	2,835	2,025	48,210
CMSTC16	16	0.148" x 2 ½"	3	720	630	720	630	935	815	935	815	30,750
			1.5	1,565	1,350	1,565	1,350	2,035	1,755	2,035	1,755	61,490
LDSS48	18	0.148 x 2 ½"	4	430	430	430	430	555	555	555	555	23,060
			2	820	820	820	820	1,070	1,070	1,070	1,070	46,120
		#9 x 3" WSV	4	1,030	1,030	1,030	1,030	1,340	1,340	1,340	1,340	17,050
			2	2,225	2,225	2,225	2,225	2,890	2,890	2,890	2,890	34,090

1. ASD/LRFD loads are based on the use of cross-laminated timber (CLT) grades E1-E4 and V1-V4 material conforming to APA PRG-320.
2. ASD/LRFD loads have been increased for wind or seismic loading with no further increase allowed; reduce where other loads govern.
3. ASD/LRFD loads are based on lesser of calculations per SDPWS 2021 or assembly tests.
4. Fastener failure modes are Mode IIIs or Mode IV.
5. Nails: 0.148" x 2 1/2" = Nail dimension listed diameter by length.
6. Screws: Strong-Drive WSV SubRoof screw: "#9 x 3" Strong-Drive WSV SubRoof screw = model WSV3S, 1/4" x 3 1/2" Strong-Drive SDS Heavy-Duty Connector Screw = model SDS25312.
7. CLT panel thickness shall be adequate to accommodate full fastener length.
8. The component of diaphragm deflection due to fastener slip at panel-to-panel joints is calculated as $\delta_s = CL_e$, where,
   $C = (1/P_x + 1/P_y)/2$
   P_x = Length of individual CLT panel (ft.) P_y = Width of individual CLT panel (ft.)
   L_e = Overall length of the diaphragm (ft.)
   e_n (in.) = Design load per foot (lbs/ft) / Slip Modulus, γ (lbs/in per ft).
9. For steel splines, the applied shear capacity or shear demand exceeds the reported ASD/LRFD shear loads by at least a factor of 2.0, satisfying the requirements of SDPWS 2021 Section 4.5.4.

For additional information on fastener slip and stiffness/slip modulus, see:
1. American Wood Council. 2018. ANSI/AWC National Design Specification for Wood Construction, NDS®, Section 11.3.6. AWC, Leesburg, VA.
2. Zhan, J.J. 1991. Design Equation for Multiple Fastener Wood Connections. US Forest Service, Forest Products Laboratory, Madison, WI.

The information in this letter is valid until **12/31/2024** when it will be re-evaluated by Simpson Strong-Tie. Please visit strongtie.com for additional pertinent information. If you have questions or need further assistance regarding this matter, please contact the Simpson Strong-Tie engineering department at 800.999.5099.

Sincerely,

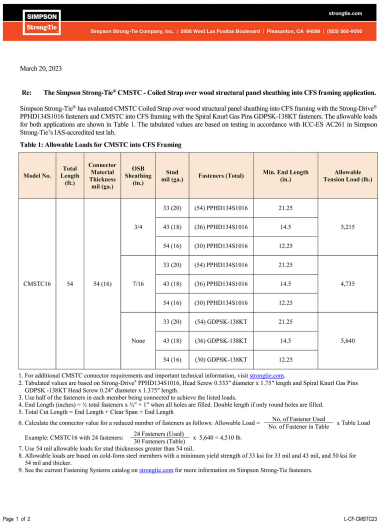
SIMPSON STRONG-TIE COMPANY INC.

Page 1 of 1 L-C-CLTSPLINE24

Simpson Strong-Tie Tested and Load-Rated Surface Steel Splines for CLT Diaphragms (SDPWS 2021)

L-C-CLTSPLINE24 — *Engineering Letter*

This engineering letter provides both LRFD and ASD values for the steel splines of CLT diaphragms satisfying the requirements of SDPWS 2021 Section 4.5.4.



March 20, 2023

Re: The Simpson Strong-Tie® CMSTC - Coiled Strap over wood structural panel sheathing into CFS framing application.

Simpson Strong-Tie® has evaluated CMSTC Coiled Strap over wood structural panel sheathing into CFS framing with the Strong-Drive® PPHD134S1016 fasteners and CMSTC into CFS framing with the Spiral Knurl Gas Pins GDPSK-138KT fasteners. The allowable loads for both applications are shown in Table 1. The tabulated values are based on testing in accordance with ICC-ES AC261 in Simpson Strong-Tie's IAS-accredited test lab.

Table 1: Allowable Loads for CMSTC into CFS Framing

Model No.	Total Length (ft.)	Connector Material Thickness mil (ga.)	OSB Sheathing (in.)	Stud mil (ga.)	Fasteners (Total)	Min. End Length (in.)	Allowable Tension Load (lb.)
CMSTC16	54	54 (16)	3/4	33 (20)	(54) PPHD134S1016	21.25	5,215
				43 (18)	(36) PPHD134S1016	14.5	
				54 (16)	(30) PPHD134S1016	12.25	
			7/16	33 (20)	(54) PPHD134S1016	21.25	4,735
				43 (18)	(36) PPHD134S1016	14.5	
				54 (16)	(30) PPHD134S1016	12.25	
			None	33 (20)	(54) GDPSK-138KT	21.25	5,640
				43 (18)	(36) GDPSK-138KT	14.5	
				54 (16)	(30) GDPSK-138KT	12.25	

1. For additional CMSTC connector requirements and important technical information, visit strongtie.com.
2. Tabulated values are based on Strong-Drive® PPHD134S1016, Head Screw 0.333" diameter x 1.75" length and Spiral Knurl Gas Pins GDPSK-138KT Head Screw 0.34" diameter x 1.375" length.
3. Use half of the fasteners in each member being connected to achieve the listed loads.
4. End Length (inches) = ½ total fasteners x ½" + 1" when all holes are filled. Double length if only round holes are filled.
5. Total Cut Length = End Length + Clear Span + End Length
6. Calculate the connector value for a reduced number of fasteners as follows: Allowable Load = $\frac{\text{No. of Fastener Used}}{\text{No. of Fastener in Table}}$ x Table Load
   Example: CMSTC16 with 24 fasteners: $\frac{24 \text{ Fasteners (Used)}}{30 \text{ Fasteners (Table)}}$ x 5,640 = 4,510 lb.
7. Use 54 mil allowable loads for stud thicknesses greater than 54 mil.
8. Allowable loads are based on cold-form steel members with a minimum yield strength of 33 ksi for 33 mil and 43 mil, and 50 ksi for 54 mil and thicker.
9. See the current Fastening Systems catalog on strongtie.com for more information on Simpson Strong-Tie fasteners.

Page 1 of 2 L-CF-CMSTC23

The Simpson Strong-Tie® CMSTC — Coiled Strap over Wood Structural Panel Sheathing into CFS Framing Application

L-CF-CMSTC23 — *Engineering Letter*

The engineering letter provides capacities of CMSTC coiled strap as installed over wood structural panel sheathing into CFS framing with Strong-Drive® PPSD134S1016 fasteners.

Product Information Table

Model No.	Ga.	Length (ft.)	Coating/Material	Packaging Qty.
CMST12	12	40	Zinc Galvanized, G90	1
CMST14	14	52 1/2	Zinc Galvanized, G90	1
CMSTC16	16	54	Zinc Galvanized, G90	1

Load Tables

Model No.	Total L	Ga.	DF/SP		SPF/HF		Allowable Tension Load (160)
			Fasteners (in.)	End Length (in.)	Fasteners (in.)	End Length (in.)	
CMST12	40'	12	(74) 0.162 x 2½	33	(84) 0.162 x 2½	38	9,215
			(86) 0.148 x 2½	39	(98) 0.148 x 2½	44	9,215
CMST14	52½'	14	(56) 0.162 x 2½	26	(66) 0.162 x 2½	30	6,475
			(66) 0.148 x 2½	30	(76) 0.148 x 2½	34	6,475
CMSTC16	54'	16	(50) 0.148 x 3¼	20	(58) 0.148 x 3¼	25	4,690

1. See Straps and Ties General Notes.
2. Calculate the connector value for a reduced number of nails as follows:

$$\text{Allowable Load} = \frac{\text{No. of Nails Used}}{\text{No. of Nails in Table}} \times \text{Table Load}$$

Example: CMSTC16 in DF/SP with 40 nails total.
(Half of the nails in each member being connected)

$$\text{Allowable Load} = \frac{40 \text{ nails (Used)}}{50 \text{ Nails (Table)}} \times 4{,}690 \text{ lb.} = 3{,}752 \text{ lb.}$$

3. See table below for alternate nailing and lap splice information.
4. Fasteners: Nail dimensions in the table are listed diameter by length. For additional information, see Fastener Types and Sizes Specified for Simpson Strong-Tie Connectors.

Strap Lap Splices

Model No.	Ga.	Strap Lap Splice	
		Minimum Fasteners per Splice	Min. Splice Length, L_{sp} (in.)
CMST12	12	(18) 0.162 x 2½	18
		(22) 0.148 x 2½	21
CMST14	14	(13) 0.162 x 2½	14
		(15) 0.148 x 2½	15
CMSTC16	16	(11) 0.162 x 2½	10
		(11) 0.148 x 2½	10

1. See Straps and Ties General Notes.

SIMPSON StrongTie
Model No. | Ga. | Length (ft.) | Coating/Material
Load Tables

2. 0.148" x 3 1/2" nails can be replaced by 0.148" x 3 1/4" nails. No other nail substitution is
3. Refer to the applicable code for minimum edge distance and minimum end distance.
4. No strap modification is allowed and the splice must meet both the minimum number of fasteners and the minimum splice length.

Allowable Loads for Alternative Nailing

Model No.	Ga.	Total Coil Length (ft.)	Fasteners (in.)	DF/SP Allowable Tension Loads	End Length (in.)	
				(160)	Nail Installed in Every Hole	Nail Installed in Every Other Hole
CMST12	12	40	(66) 0.162 x 2½	8,415	30	58
			(58) 0.162 x 2½	7,395	27	51
			(50) 0.162 x 2½	6,375	23	44
			(76) 0.148 x 2½	8,320	35	66
			(68) 0.148 x 2½	7,445	31	59
			(60) 0.148 x 2½	6,570	28	52
CMST14	14	52.5	(48) 0.162 x 2½	5,615	22	42
			(40) 0.162 x 2½	4,680	19	35
			(32) 0.162 x 2½	3,745	15	28
			(58) 0.148 x 2½	5,770	27	51
			(50) 0.148 x 2½	4,975	23	44
			(42) 0.148 x 2½	4,180	20	37
CMSTC16	16	54	(42) 0.162 x 2½	4,690	17	32
			(34) 0.162 x 2½	3,875	14	26
			(26) 0.162 x 2½	2,965	11	20
			(18) 0.162 x 2½	2,050	8	14
			(48) 0.148 x 2½	4,610	19	35
			(40) 0.148 x 2½	3,840	16	29
			(32) 0.148 x 2½	3,070	13	23
			(24) 0.148 x 2½	2,305	10	17
			(16) 0.148 x 2½	1,535	7	11

1. See Straps and Ties General Notes.

Model No.	Ga.	Length (ft.)	Coating/Material

Fasteners: Nail dimensions in the table are listed diameter x length. For additional information, see Fastener Types and Sizes Specified for Simpson Strong-Tie Connectors.

Load Tables

Code Reports & Compliance

Product Series	Compliance/Certification	Report/Approval
CMST	International Building Code International Residential Code City of Los Angeles Building Code City of Los Angeles Residential Code	ER-124 (CFS)
	International Building Code International Residential Code City of Los Angeles Building Code City of Los Angeles Residential Code	ESR-2105
	Florida Building Code	FL10456
	Florida Building Code	FL13872

Footnotes

1. Please review compliance documents for information about specific product models. In some cases, Compliance documents cover most but not all models associated with a product series.
2. For additional information regarding Florida's Statewide Product Approval System and Miami-Dade County Notice of Acceptance (NOA), click here.

Related Products



CS
Coiled Strap

Model No.	Ga.	Length (ft.)	Coating/Material	

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Product Use & Corrosion Info
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Value-Engineered Design Options for CLT

SIMPSON Strong-Tie®



Simpson Strong-Tie® load-rated tension straps and fasteners to minimize cost and maximize speed and precision — while providing the strength, versatility and reliability you depend on.

Straps are designed to carry tension loads in a wide variety of applications.

MDCST48 — A 14-gauge high-capacity strap specifically designed to carry tension forces across a CLT floor or wall panel joint. The MDCST48 installs with Simpson Strong-Tie Strong-Drive® SDS Heavy-Duty Connector screws.

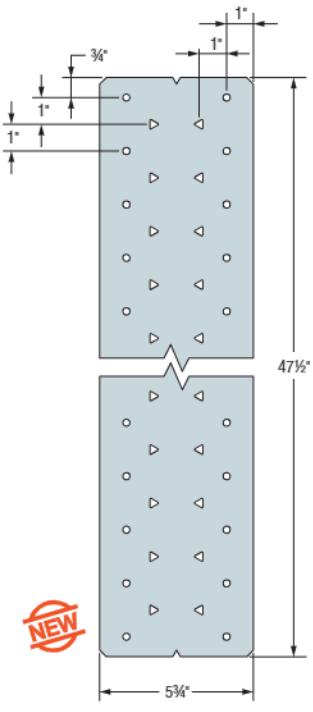
MSTC — A high-capacity strap that utilizes a staggered nail pattern to help minimize wood splitting. Nail slots have been countersunk to provide a lower nail head profile.

HRS — A 12-gauge strap with a nailing pattern designed for installation on the edge of 2x members, but also suitable for many CLT applications. The HRS416Z installs with Simpson Strong-Tie Strong-Drive SDS Heavy-Duty Connector screws.

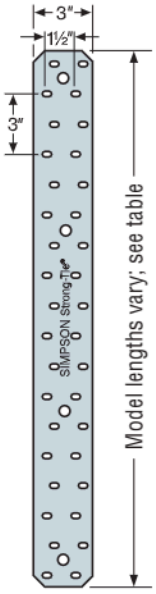
MST — A high capacity strap that uses nails and is suitable for a variety of CLT applications. Suitable for double 2x member connections or greater, and also for a variety of CLT applications.

Finish: Galvanized G90. Some products are available in ZMAX coating; see Corrosion Information at strongtie.com.

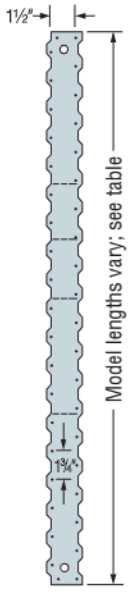
Installation: Use all specified fasteners. See General Notes.



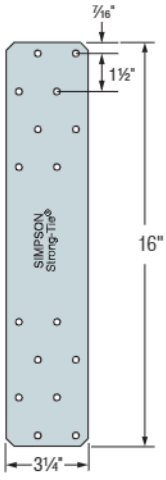
MDCST48



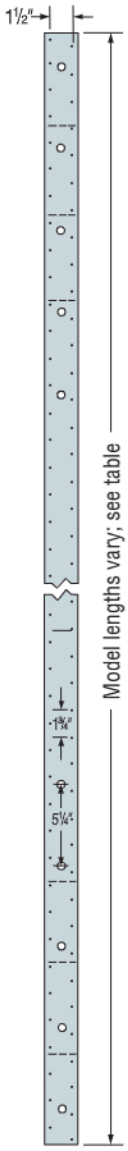
MSTC



ST



HRS416Z



MST

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Value-Engineered Design Options for CLT

Factored Tensile Resistance Table

Model No.	Ga.	Dimensions (in.)		Fasteners (Total)	Factored Tensile Resistance							
					D.Fir-L				S-P-F			
					K_D = 1.00		K_D = 1.15		K_D = 1.00		K_D = 1.15	
		W	L		lb.	kN	lb.	kN	lb.	kN	lb.	kN
ST6215	16	2 1/16	16 5/16	(16) 8d x 2 1/2"	1365	6.07	1565	6.98	1265	5.64	1455	6.49
ST6224		2 1/16	23 5/16	(24) 8d x 2 1/2"	2245	10.00	2580	11.50	2065	9.20	2375	10.58
MSTC28		3	28 1/4	(32) 10d x 2 1/2"	3850	17.14	4425	19.71	3430	15.28	3945	17.57
MSTC40		3	40 1/4	(48) 10d x 2 1/2"	5770	25.71	6640	29.57	5145	22.92	5915	26.36
MSTC52		3	52 1/4	(54) 10d x 2 1/2"	6495	28.93	6940	30.87	5790	25.79	6655	29.65
MSTC66	14	3	65 3/4	(66) 10d x 2 1/2"	8085	36.02	8570	38.11	7075	31.52	8135	36.24
MSTC78		3	77 3/4	(66) 10d x 2 1/2"	8085	36.02	8570	38.11	7075	31.52	8135	36.24
ST6236		2 1/16	33 13/16	(36) 8d x 2 1/2"	3550	15.82	4085	18.19	3105	13.84	3575	15.91
MDCST48		5 3/4	47 1/2	(44) 1/4" x 3" SDS	12350	55.00	14200	63.26	11320	50.43	13020	58.00
MDCST48 (Double/ Overlapped)		5 3/4	47 1/2	(88) 1/4" x 3" SDS	34670	154.42	35505	157.94	31110	138.57	35505	157.94
HRS416Z	12	3 1/4	16	(16) 1/4" x 1 1/2" SDS	3095	13.80	3560	15.87	2920	13.00	3355	14.95
MST27		2 1/16	27	(26) 8d x 2 1/2"	2555	11.38	2940	13.09	2235	9.97	2575	11.46
MST37		2 1/16	37 1/2	(38) 8d x 2 1/2"	3735	16.63	4295	19.13	3270	14.57	3760	16.75
MST48		2 1/16	48	(50) 8d x 2 1/2"	4915	21.90	5655	25.18	4305	19.17	4950	22.05
MST60	10	2 1/16	60	(64) 8d x 2 1/2"	6290	28.03	7235	32.23	5510	24.54	6335	28.22
MST72		2 1/16	72	(78) 8d x 2 1/2"	7670	34.16	8450	37.60	6715	29.91	7720	34.39

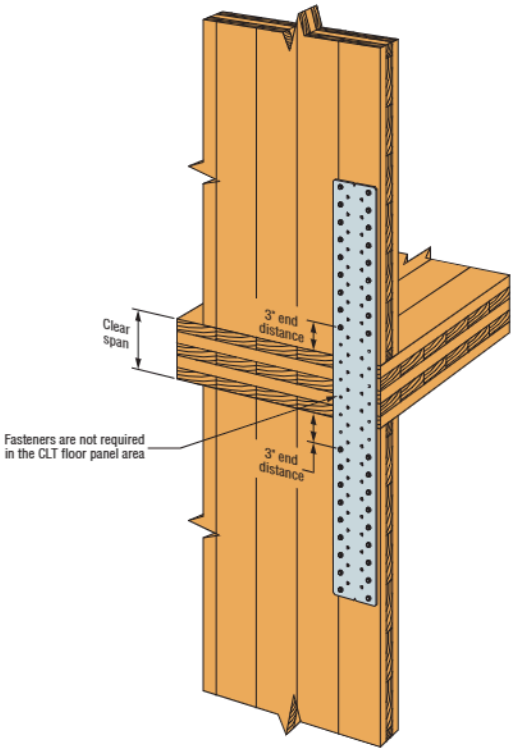
1. Factored resistances have been increased 15% for wind or seismic loading with no further increase allowed; reduce where other loads govern.
2. Install fasteners as specified by Designer.
3. Use half of required fasteners in each member being connected to achieve the listed resistances.
4. When using the MDCST48 as a single strap, fill only round holes. When using the MDCST48 as a double/overlapped strap, fill round and triangle holes.
5. Nails: 8d = 0.131" x 2 1/2" long, 10d = 0.148" x 2 1/2" long. Screws: 1/4" x 1 1/2" long SDS, 1/4" x 3" long SDS. All nails shown are common wire.

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Value-Engineered Design Options for CLT

Floor-to-Floor Clear Span Table

Model	CLT Plys	Clear Span (in.)	Fasteners (Total)	Factored Tensile Resistance							
				D.Fir-L				S-P-F			
				K_D = 1.00		K_D = 1.15		K_D = 1.00		K_D = 1.15	
				lb.	kN	lb.	kN	lb.	kN	lb.	kN
MSTC40	3	4.125	(44) 10d x 2½"	5290	23.53	6085	27.07	4715	20.97	5425	24.13
	5	6.875	(40) 10d x 2½"	4810	21.40	5530	24.60	4290	19.08	4930	21.93
	7	9.625	(36) 10d x 2½"	4330	19.26	4980	22.15	3860	17.17	4440	19.75
	9	12.375	(32) 10d x 2½"	3850	17.13	4425	19.68	3430	15.26	3945	17.55
MSTC52	3	4.125	(60) 10d x 2½"	6940	30.87	6940	30.87	6430	28.60	6940	30.87
	5	6.875	(56) 10d x 2½"	6735	29.96	6940	30.87	6005	26.71	6905	30.71
	7	9.625	(52) 10d x 2½"	6255	27.82	6940	30.87	5575	24.80	6410	28.51
	9	12.375	(48) 10d x 2½"	5770	25.67	6640	29.54	5145	22.89	5915	26.31
MSTC66	3	4.125	(76) 10d x 2½"	8570	38.12	8570	38.12	8145	36.23	8570	38.12
	5	6.875	(72) 10d x 2½"	8570	38.12	8570	38.12	7720	34.34	8570	38.12
	7	9.625	(68) 10d x 2½"	8330	37.05	8570	38.12	7290	32.43	8385	37.30
	9	12.375	(64) 10d x 2½"	7840	34.87	8570	38.12	6860	30.51	7890	35.10
MSTC78	3	4.125	(92) 10d x 2½"	8570	38.12	8570	38.12	8570	38.12	8570	38.12
	5	6.875	(88) 10d x 2½"	8570	38.12	8570	38.12	8570	38.12	8570	38.12
	7	9.625	(84) 10d x 2½"	8570	38.12	8570	38.12	8570	38.12	8570	38.12
	9	12.375	(80) 10d x 2½"	8570	38.12	8570	38.12	8570	38.12	8570	38.12
MST48	3	4.125	(48) 8d x 2½"	4720	21.00	5425	24.13	4130	18.37	4750	21.13
	5	6.875	(44) 8d x 2½"	4325	19.24	4975	22.13	3790	16.86	4355	19.37
	7	9.625	(40) 8d x 2½"	3930	17.48	4520	20.11	3445	15.32	3960	17.61
	9	12.375	(36) 8d x 2½"	3540	15.75	4070	18.10	3100	13.79	3565	15.86
MST60	3	4.125	(60) 8d x 2½"	5900	26.24	6785	30.18	5165	22.98	5940	26.42
	5	6.875	(56) 8d x 2½"	5505	24.49	6330	28.16	4820	21.44	5545	24.67
	7	9.625	(52) 8d x 2½"	5110	22.73	5880	26.16	4475	19.91	5150	22.91
	9	12.375	(52) 8d x 2½"	5110	22.73	5880	26.16	4475	19.91	5150	22.91
MST72	3	4.125	(72) 8d x 2½"	7080	31.49	8070	35.90	6200	27.58	7125	31.69
	5	6.875	(72) 8d x 2½"	7080	31.49	7895	35.12	6200	27.58	7125	31.69
	7	9.625	(68) 8d x 2½"	6685	29.74	7550	33.58	5855	26.04	6730	29.94
	9	12.375	(64) 8d x 2½"	6290	27.98	7235	32.18	5510	24.51	6335	28.18
MDCST48	3	4.125	(36) ¼" x 3" SDS	10105	44.95	11620	51.69	9265	41.21	10655	47.40
	5	6.875	(32) ¼" x 3" SDS	8980	39.95	10330	45.95	8235	36.63	9470	42.12
	7	9.625	(32) ¼" x 3" SDS	8980	39.95	10330	45.95	8235	36.63	9470	42.12
	9	12.375	(28) ¼" x 3" SDS	7860	34.96	9035	40.19	7205	32.05	8285	36.85
MDCST48 (Double/ Overlapped)	3	4.125	(72) ¼" x 3" SDS	28365	126.17	32620	145.10	25455	113.23	29270	130.20
	5	6.875	(64) ¼" x 3" SDS	25215	112.16	28995	128.98	22625	100.64	26020	115.74
	7	9.625	(60) ¼" x 3" SDS	23635	105.13	27185	120.92	21210	94.35	24390	108.49
	9	12.375	(56) ¼" x 3" SDS	22060	98.13	25370	112.85	19795	88.05	22765	101.26



Typical Floor-to-Floor Tie Installation with MDCST48 and SDS Heavy-Duty Connector Screws at Three-Ply CLT Walls and Five-Ply CLT Floor

(other strap options similar; other CLT ply combinations similar)

1. Factored resistances have been increased 15% for wind or seismic loading with no further increase allowed; reduce where other loads govern.
2. Install fasteners as specified by Designer.
3. Use half of required fasteners in each member being connected to achieve the listed resistances.
4. When nailing the strap over OSB/plywood, use a minimum 2-½" long nail.
5. When using the MDCST48 as a single strap, fill only round holes. When using the MDCST48 as a double/ overlapped strap, fill round and triangle holes.
6. Nails: 8d = 0.131" x 2½" long, 10d = 0.148" x 2½" long. Screws: ¼" x 3" long SDS. All nails shown are common wire.

This flier is effective until June 30, 2024, and reflects information available as of July 1, 2019.
This information is updated periodically and should not be relied upon after June 30, 2024.
Contact Simpson Strong-Tie for current information and limited warranty or see strongtie.com.

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F-C-CLTNSTCAN19 7/19 exp. 6/24

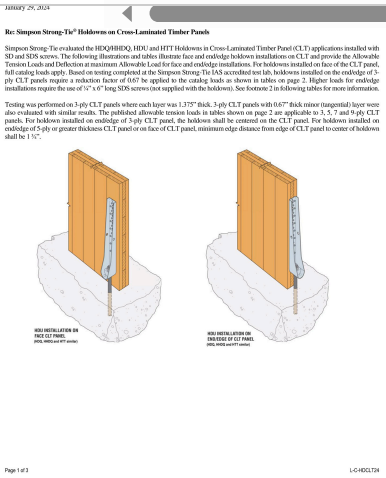
(800) 999-5099
strongtie.com

HDU Holdown

Select Media



HDU11-SDS2.5



January 29, 2024

Re: Simpson Strong-Tie® Holdowns on Cross-Laminated Timber Panels

Simpson Strong-Tie evaluated the HDQ/HHDQ, HDU and HTT Holdowns in Cross-Laminated Timber Panel (CLT) applications installed with SD and SDS screws. The following illustrations and tables illustrate face and endedge holdown installations on CLT and provide the Allowable Tension Loads and Deflection at maximum Allowable Load for face and endedge installations. For holdowns installed on face of the CLT panel, full catalog loads apply. Based on testing completed at the Simpson Strong-Tie IAS accredited test lab, holdowns installed on the endedge of 3-ply CLT panels require a reduction factor of 0.67 be applied to the catalog loads as shown in tables on page 2. Higher loads for endedge installations require the use of ¼" x 6" long SDS screws (not supplied with the holdown). See footnote 2 in following tables for more information.

Testing was performed on 3-ply CLT panels where each layer was 1.375" thick. 3-ply CLT panels with 0.67" thick minor (tangential) layer were also evaluated with similar results. The published allowable tension loads in tables shown on page 2 are applicable to 3, 5, 7 and 9-ply CLT panels. For holdown installed on endedge of 3-ply CLT panel, the holdown shall be centered on the CLT panel. For holdown installed on endedge of 5-ply or greater thickness CLT panel or on face of CLT panel, minimum edge distance from edge of CLT panel to center of holdown shall be 1 ½".

HDU INSTALLATION ON FACE CLT PANEL (HDQ, HHDQ and HTT similar)

HDU INSTALLATION ON ENDGRAIN OF CLT PANEL (HDQ, HHDQ and HTT similar)

Page 1 of 3

L-C-HDCLT24

Simpson Strong-Tie® Holdowns on Cross-Laminated Timber Panels

L-C-HDCLT24 — *Engineering Letter*

Simpson Strong-Tie evaluated the HDQ/HHDQ, HDU and HTT holdowns in cross-laminated timber (CLT) panel applications installed with SD and SDS screws.

Product Information Table

Model No.	Ga.	W (in.)	H (in.)	B (in.)	CL (in.)	SO (in.)	Coating/Material	Product Includes	Packaging Qty.
HDU11-SDS2.5	10	3	22 1/4	3 1/2	1 3/8	1 1/2	Zinc Galvanized, G90	(30) 1/4 in. x 2-1/2 in. Strong-Drive SDS Heavy-Duty Connector screws and (1) crescent washer	1
HDU11-SDS2.5HDG	10	3	22 1/4	3 1/2	1 3/8	1 1/2	Hot-Dip Galvanized	(30) 1/4 in. x 2-1/2 in. Strong-Drive SDS Heavy-Duty Connector screws and (1) crescent washer	1
HDU14-SDS2.5	7	3	25 11/16	3 1/2	1 9/16	1 9/16	Zinc Galvanized, G90	(36) 1/4 in. x 2-1/2 in. Strong-Drive SDS Heavy-Duty Connector screws, (1) crescent washer and (1) heavy-hex anchor nut	1
HDU14-SDS2.5HDG	7	3	25 11/16	3 1/2	1 9/16	1 9/16	Hot-Dip Galvanized	(36) 1/4 in. x 2-1/2 in. Strong-Drive SDS Heavy-Duty Connector screws, (1) crescent washer and (1) heavy-hex anchor nut	1
HDU2-SDS2.5	14	3	8 11/16	3 1/4	1 5/16	1 3/8	Zinc Galvanized, G90	(6) 1/4 in. x 2-1/2 in. Strong-Drive SDS Heavy-Duty Connector screws and (1) crescent washer	1

Model No.	Ga.	W (in.)	H (in.)	B (in.)	CL (in.)	SO (in.)	Coating/Material	Product Includes	Packaging Qty.
Load Tables HDU2-SDS2.5HDG	14	3	8 11/16	3 1/4	1 5/16	1 3/8	Hot-Dip Galvanized	(6) 1/4 in. x 2-1/2 in. Strong-Drive SDS Heavy-Duty Connector screws and (1) crescent washer	1
HDU4-SDS2.5	14	3	10 15/16	3 1/4	1 5/16	1 3/8	Zinc Galvanized, G90	(10) 1/4 in. x 2-1/2 in. Strong-Drive SDS Heavy-Duty Connector screws and (1) crescent washer	1
HDU4-SDS2.5HDG	14	3	10 15/16	3 1/4	1 5/16	1 3/8	Hot-Dip Galvanized	(10) 1/4 in. x 2-1/2 in. Strong-Drive SDS Heavy-Duty Connector screws and (1) crescent washer	1

Load Tables

These products are available with additional corrosion protection.

Model No.	Ga.	Dimensions (in.)					Fasteners (in.)		Minimum Wood Member Size (in.)	Allowable Tension Loads (160)		
		W	H	B	CL	SO	Anchor Bolt Dia. (in.)	Wood Fasteners		DF/SP	SPF/HF	Deflection at Allowable Loa (in.)
HDU2-SDS2.5	14	3	8 11/16	3¼	1 5/16	1⅜	⅝	(6) ¼ x 2½ SDS	3 x 3½	3,075	2,215	0.088
HDU4-SDS2.5	14	3	10 15/16	3¼	1 5/16	1⅜	⅝	(10) ¼ x 2½ SDS	3 x 3½	4,565	3,285	0.114
HDU5-SDS2.5	14	3	13 3/16	3¼	1 5/16	1⅜	⅝	(14) ¼ x 2½ SDS	3 x 3½	5,645	4,340	0.115
HDU8-SDS2.5	10	3	16⅝	3½	1⅜	1½	⅞	(20) ¼ x 2½ SDS	3 x 3½	6,765	5,820	0.11
									3½ x 3½	6,970	5,995	0.116
									3½ x 4½	7,870	6,580	0.113
HDU11-SDS2.5	10	3	22¼	3½	1⅜	1½	1	(30) ¼ x 2½ SDS	3½ x 5½	9,535	8,030	0.137
									3½ x 7¼	11,175	9,610	0.137
HDU14-SDS2.5	7	3	25 11/16	3½	1 9/16	1 9/16	1	(36) ¼ x 2½ SDS	3½ x 5½	10,770	9,260	0.122
									3½ x 7¼	14,390	12,375	0.177
									5½ x 5½	14,445	12,425	0.172

1. HDU14 requires heavy-hex anchor nut to achieve tabulated loads (supplied with holdown).
2. HDU14 loads on 4x6 post are applicable to installation on either the narrow or the wide face of the post.
3. Fasteners: Nail dimensions are listed diameter by length. SD and SDS screws are Simpson Strong-Tie® Strong-Drive SD Connector and SDS Heavy-Duty Connector screws. For additional information, see Fastener Types and Sizes Specified for Simpson Strong-Tie Connectors.

Code Reports & Compliance

SIMPSON Strong-Tie t Series	Model No.	Ga.	W (in.)	H (in.)	B (in.)	CL (in.)	SO (in.)	Report/Approval Coating/Material
Load Tables	International Residential Code City of Los Angeles Building Code City of Los Angeles Residential Code							ESR-2330
HDU	Florida Building Code							FL10441

Footnotes

1. Please review compliance documents for information about specific product models. In some cases, Compliance documents cover most but not all models associated with a product series.
2. For additional information regarding Florida's Statewide Product Approval System and Miami-Dade County Notice of Acceptance (NOA), click here.

Related Products



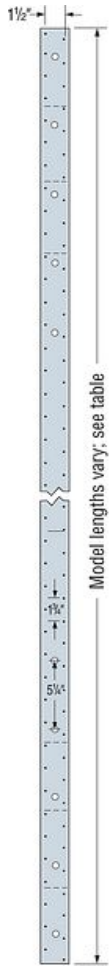
DTT™
Deck Tension Tie



Strong-Drive® SDS HEAVY-DUTY CONNECTOR Screw

MST Medium Strap Tie

Select Media



MST

Straight Straps

This product's information may differ depending on the category of use. You are currently viewing details related to **Straight Straps**. You can also view product information related to the category: Lateral Connectors, Ties and Straps for Cold-Formed Steel Construction

SIMPSON Strong-Tie high-wind-induced damage to existing
s in Hawaii.

Load Tables

Product Information Table

Model No.	Thickness (mil)	Ga.	W (in.)	L (in.)	Coating/Material	Packaging Qty.
MST27	97	12	2 1/16	27	Zinc Galvanized, G90	1
MST27HDG	97	12	2 1/16	27	Hot-Dip Galvanized	1
MST37	97	12	2 1/16	37	Zinc Galvanized, G90	1
MST37HDG	97	12	2 1/16	37	Hot-Dip Galvanized	1
MST48	97	12	2 1/16	48	Zinc Galvanized, G90	1
MST48HDG	97	12	2 1/16	48	Hot-Dip Galvanized	1
MST60	118	10	2 1/16	60	Zinc Galvanized, G90	1
MST60HDG	118	10	2 1/16	60	Hot-Dip Galvanized	1
MST72	118	10	2 1/16	72	Zinc Galvanized, G90	1

Load Tables

Floor-to-Floor Span Table

These products are available with additional corrosion protection. Additional products on this page may also be available with this option, check with Simpson Strong-Tie for details.

SD Many of these products are approved for installation with Strong-Drive® SD Connector screws.

Model No.	Clear Span (in.)	Fasteners (Total) (in.)	DF/SP Allowable Tension Loads (160)	SPF/HF Allowable Tension Loads (160)
MST37	24	(14) 0.162 x 2½	1,720	1,500
	18	(20) 0.162 x 2½	2,460	2,140
	16	(22) 0.162 x 2½	2,705	2,355
MST48	24	(26) 0.162 x 2½	3,210	2,780
	18	(32) 0.162 x 2½	3,950	3,425
	16	(34) 0.162 x 2½	4,200	3,640
MST60	30	(34) 0.162 x 2½	4,605	3,995
	24	(40) 0.162 x 2½	5,240	4,700
	18	(46) 0.162 x 2½	6,235	5,405
MST72	30	(48) 0.162 x 2½	6,505	5,640
	24	(54) 0.162 x 2½	6,730	6,345
	18	(62) 0.162 x 2½	6,730	6,475

SIMPSON Strong-Tie notes below.

Load Tables

Model No.	Ga.	Dimensions (in.)		Fasteners (Total)			DF/SP Allowable Tension Loads		SPF/HF Allowable Tension Loads	
		W	L	Nails (in.)	Bolts		Nails	Bolts	Nails	Bolts
					Qty.	Dia.	(160)	(160)	(160)	(160)
MST27	12	2 1⁄16	27	(30) 0.162 x 2 1⁄2	4	1⁄2	3,700	2,165	3,210	2,000
MST37		2 1⁄16	37 1⁄2	(42) 0.162 x 2 1⁄2	6	1⁄2	5,070	3,030	4,495	2,800
MST48		2 1⁄16	48	(50) 0.162 x 2 1⁄2	8	1⁄2	5,310	3,675	5,190	3,395
MST60	10	2 1⁄16	60	(68) 0.162 x 2 1⁄2	10	1⁄2	6,730	4,490	6,475	4,150
MST72		2 1⁄16	72	(68) 0.162 x 2 1⁄2	10	1⁄2	6,730	4,490	6,475	4,150

1. See General Notes for Straps and Ties.
2. Install bolts or nails as specified by designer. Bolt and nail values may not be combined.
3. Allowable bolt loads are based on parallel-to-grain loading and minimum member thickness: MST — 2 1/2".
4. Splitting may be a problem with installations on lumber smaller than 3 1/2"; either fill every nail hole with 0.148" x 1 1/2" nails or fill every other hole with 0.162" x 2 1/2" nails. Reduce the allowable load based on the size and quantity of fasteners used.
5. Fasteners: Nail dimensions in the table are listed diameter by length. For additional information, see Fastener Types and Sizes Specified for Simpson Strong-Tie Connectors.

Load Values with Strong-Drive® SD Connector Screws

Model No.	Fasteners (Total)	(DF/SP) Allowable Tension Loads	(SPF/HF) Allowable Tension Loads
		(160)	(160)
MST27	30-SD10112	4150	3310
MST37	40-SD10112	5070	4415
MST48	52-SD10112	5310	5035
MST60	68-SD10112	6765	6375
MST72	70-SD10112	6765	6375

These products are available with additional corrosion protection. Additional products on this page may also be available with this option, check with Simpson Strong-Tie for details.

Code Reports & Compliance

Product Series	Compliance/Certification	Report/Approval
MST	International Building Code International Residential Code City of Los Angeles Building Code City of Los Angeles Residential Code	ESR-3096
	International Building Code International Residential Code City of Los Angeles Building Code City of Los Angeles Residential Code	ESR-2105
	Florida Building Code	FL10456

Footnotes

1. Please review compliance documents for information about specific product models. In some cases, Compliance documents cover most but not all models associated with a product series.
2. For additional information regarding Florida's Statewide Product Approval System and Miami-Dade County Notice of Acceptance (NOA), click here.

LTT/HTT

Tension Ties

Tension ties offer a solution for resisting tension loads that are fastened with nails or Strong-Drive® SD Connector screws. The new LTTP2 light tension tie, designed for wood joist attachments to concrete or masonry walls, features two separate nailing patterns: obround holes spaced 3" apart for I-joist purlins and square holes spaced to accommodate the narrow face of 2x solid-sawn purlins. LTTP2 may also be installed vertically on the wide face of a minimum 2x4 stud for holdown application. It features an extruded anchor bolt hole to accommodate ¾", ⅝" and ½" bolt diameters.

The LTTI31 is designed for wood chord open-web truss attachments to concrete or masonry walls.

The HTT4 and HTT5 tension ties feature an optimized nailing pattern which results in better performance with less deflection. HTT5KT is sold as a kit with the holdown, bearing plate washer and Strong-Drive SD Connector screws.

The HTT5-¾ is designed to use a ¾"-diameter anchor bolt.

When using LTT or HTT tension ties with unreinforced concrete masonry, ¾" post-installed anchor bolts are commonly used.

Material: See table

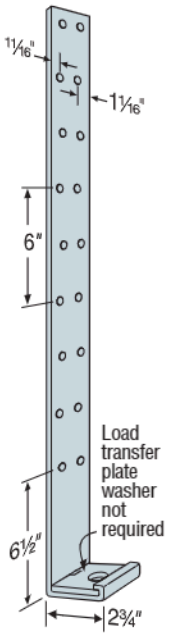
Finish: Galvanized. May be ordered HDG.

Installation:

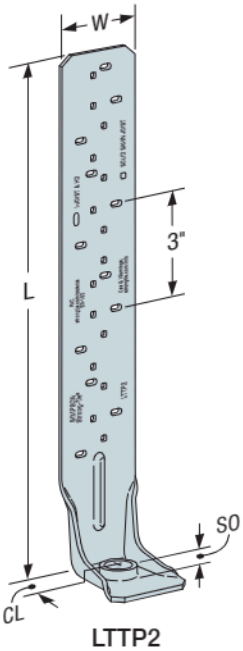
- See Holdown and Tension Tie General Notes on pp. 51–52.
- LTTP2 — one standard cut-washer is required when using ½" and ⅝" anchor bolts; and no additional washer is required for ¾" anchor bolts.
- LTTP2 — For installations on narrow edge of solid sawn (2x, 3x) joists use (15) square holes; for all other installations use (12) obround holes.
- For tension ties installed over wood structural panel sheathing, use a 2½"-long fastener minimum.
- For information about marriage strap at panelized roof applications, see **strongtie.com**.
- HTT5-KT requires BP 5/8-2 bearing plate and #10 x 2½" SD Strong-Drive screws (included in kit).

Codes: Codes: See p. 13 for Code Reference Key Chart

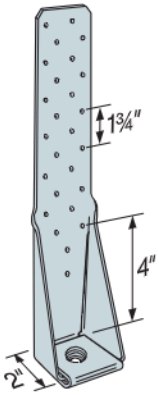
Web Applications: Visit **app.strongtie.com/pfd** to access our Post-to-Foundation Designer web application.



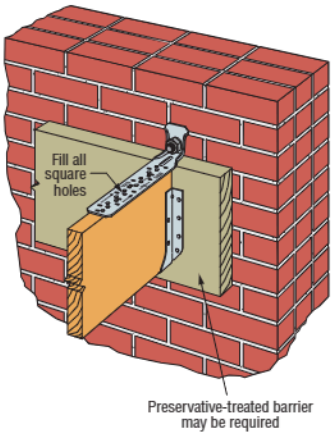
LTTI31



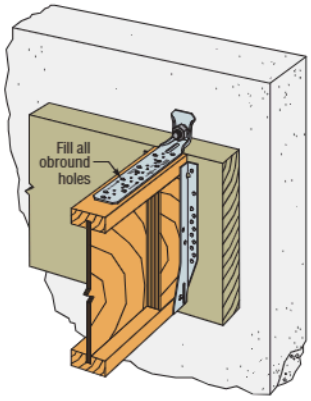
LTTP2
US Patent Pending



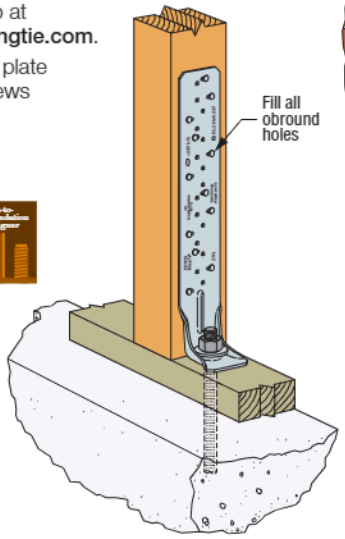
HTT5
(HTT4 similar)



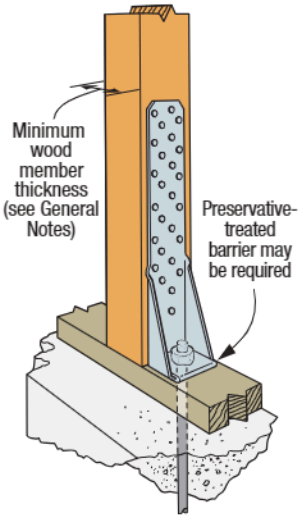
Typical LTTP2 Installation for Solid Sawn Joist



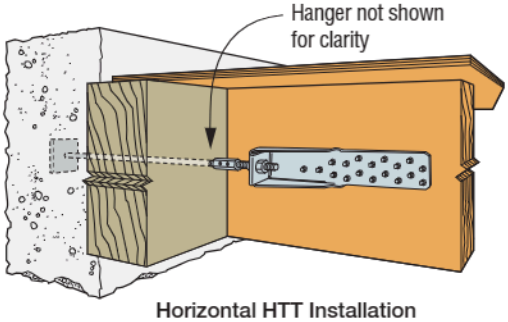
Typical LTTP2 Installation for I-joist



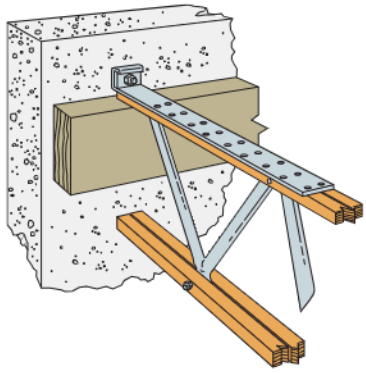
Typical LTTP2 Installation for Holdown Application



Vertical HTT5 Installation
(HTT4 similar)



Horizontal HTT Installation



Horizontal LTTI31 Installation

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Tension Ties (cont.)

These products are available with additional corrosion protection. For more information, see p. 16.

SD Many of these products are approved for installation with Strong-Drive® SD Connector screws. See pp. 362–366 for more information.

Model No.	Ga.	Dimensions (in.)			SO (in.)	Fasteners (in.)		Minimum Wood Member Size (in.)	Allowable Tension Loads (160)		Deflection at Highest Allowable Load (in.)	Code Ref.
		W	L	CL		Anchor Rod Diameter	Wood Fasteners		DF/SP	SPF/HF		
LTTP2	10	2⁹⁄₁₆	14 15⁄16	1⅛	7⁄16	½, ⅝, ¾	(15) 0.148 x 2½	1½ x 3½ (narrow edge)[4,5]	1,845	1,695	0.104	IBC®, FL, LA
						½	(12) 0.148 x 1½	1½ x 3½	1,680[6]	1,545[6]	0.138	
						⅝, ¾			2,135	1,965	0.112	
						½	(12) #9 x 1½" SD	1½ x 3½	2,320	1,970	0.112	
						⅝, ¾			2,570	2,045	0.136	
						½, ⅝, ¾	(12) 0.148 x 2½	3 x 3½	2,275	2,230	0.128	
LTTI31	18	3¾	31	1⅜	¼	⅝	(18) 0.148 x 1½	3 x 3½	1,350	1,160	0.193	
HTT4	11	2½	12⅜	1 5⁄16	7⁄16	⅝	(18) 0.148 x 1½	1½ x 3½	3,000	2,580	0.090	—
							(18) 0.148 x 1½	3 x 3½	3,610	3,105	0.086	IBC, FL, LA
							(18) 0.162 x 2½	3 x 3½	4,235	3,640	0.123	
							(18) #10 x 1½" SD	1½ x 5½	4,455	3,830	0.112	—
							(18) #10 x 1½" SD	3 x 3½	4,455	3,830	0.112	
HTT5	11	2½	16	1⅜	7⁄16	⅝	(26) 0.148 x 1½	3 x 3½	4,350	3,740	0.120	IBC, FL, LA
							(26) 0.148 x 3	3 x 3½	4,670	4,015	0.116	
							(26) 0.162 x 2½	3 x 3½	5,090	4,375	0.135	
							(26) #10 x 1½" SD	1½ x 5½	4,555	3,915	0.114	—
HTT5KT	11	2½	16	1⅜	7⁄16	⅝	(26) #10 x 2½" SD	3 x 3½	5,445	5,360	0.103	
HTT5-3/4	11	2½	16	1⅜	7⁄16	¾	(26) 0.148 x 1½	1½ x 5½	4,065	3,495	0.103	IBC, FL
							(26) 0.162 x 2½	3 x 3½	5,090	4,375	0.121	
							(26) #10 x 1½" SD	1½ x 7¼	4,830	4,155	0.100	

1. LTTI31 installed flush with concrete or masonry has an allowable load of 2,285 lb.
2. Allowable load for HTT5 with a BP5/8-2 bearing-plate washer installed in the seat of the holdown is 5,295 lb. for DF/SP and 4,555 lb. for SPF/HF.
3. For LTTP2, standard cut washer is required when using ½" and ⅝" anchor rods.
4. For (15) nail installations on narrow edge of 2x4 (minimum) joist, LTTP2 installed flush with concrete or masonry has an allowable load of 2,560 lb. for DF/SP and 2,355 lb. for SPF/HF.
5. LTTP2 installed with (15) #9 x 1½" SD screws on narrow edge of 2x joist has an allowable load of 2,105 lb. for DF/SP and 1,935 lb. for SPF/HF.
6. For (12) nail installations on I-joist or wide face of 2x member, LTTP2 installed flush with concrete or masonry has an allowable load of 1,950 lb. for DF/SP and 1,795 lb. for SPF/HF.
7. **Fasteners:** Nail dimensions are listed diameter by length. SD screws are Simpson Strong-Tie Strong-Drive SD Connector screws. See pp. 23–24 for fastener information.

Table 1 — Anchorage Selection Guide for Holdowns Attached to DF/SP Lumber

Holdown on DF/SP Lumber	Stemwall				Slab on Grade			
	Stemwall Width (in.)	Wind and Seismic Design Category A&B		Seismic Design Category C–F		Wind and Seismic Design Category A&B		Seismic Design Category C–F	
		Midwall/Corner	End Wall	Midwall/Corner	End Wall	Midwall/Corner	Garage Curb	Midwall/Corner	Garage Curb
HDU2	6	SSTB16		SSTB24		SSTB16		SSTB16	SSTB20* (2,960)
HDU4	6	SB5/8X24		SB5/8X24		SSTB16	SB5/8X24	SSTB20	SB5/8X24
HDU5	6	SB5/8X24		SB5/8X24		SSTB20	SB5/8X24	SSTB24	SB5/8X24
HDU8									
HDQ8									
HDU11									
HHDQ11									
HDU14									
HHDQ14									
LTTP2									
LTTI31									
HTT4									
HTT5									
HD3B									
HD5B									
HD7B									
HD9B									
HD12									

See footnotes be

Table 2 — Anchorage Selection Guide for Holdowns Attached to SPF/HF Lumber

Holdown on SPF/HF Lumber	Stemwall					Slab on Grade			
	Stemwall Width (in.)	Wind and Seismic Design Category A&B		Seismic Design Category C–F		Wind and Seismic Design Category A&B		Seismic Design Category C–F	
		Midwall/Corner	End Wall	Midwall/Corner	End Wall	Midwall/Corner	Garage Curb	Midwall/Corner	Garage Curb
HDU2	6	SSTB16		SSTB16		SSTB16		SSTB16	
HDU4	6	SSTB16		SSTB24		SSTB16		SSTB16	SSTB24
HDU5	6	SSTB24* (4,295)		SB5/8X24		SSTB16	SSTB24* (4,295)	SSTB20	SB5/8X24
HDU8	8	SSTB28		SSTB28	SSTB28* (6,395)	SSTB28		SSTB28	SSTB28
HDQ8	8	SSTB28		SSTB28	SSTB28* (6,395)	SSTB28		SSTB28	SSTB28
HDU11	8	SB1X30* (9,505)	PAB8	PAB8	PAB8	SB1x30		SB1x30	
HHDQ11	8	SB1X30	PAB8	PAB8					
HDU14	—	PAB8		PAB8		SB1x30		SB1x30	
HHDQ14	—								
LTTP2	6	SSTB16		SSTB16		SSTB16		SSTB16	
LTTI31	6								
HTT4	6	SSTB20		SB5/8X24		SSTB16	SSTB20	SSTB16* (3,780)	SB5/8X24
HTT5	6	SB5/8X24		SB5/8X24		SSTB20	SB5/8X24	SSTB24	SB5/8X24
HD3B	6	SSTB16		SSTB24		SSTB16		SSTB16	SSTB20* (2,960)

We've made selecting the right anchor bolt for the holdown easier; check out our Holdown Anchorage Solutions table on p. 46 or visit **app.strongtie.com/pfd** to access the Post-to-Foundation Designer web application.

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HHDQ Heavy Holdown, Quick-Install



On This Page

Product Details

The HHDQ is designed for extra-high loads and provides ease of installation. This heavy holdown comes with Strong-Drive® SDS Heavy-Duty Connector screws, which install easily, reduce fastener slip and provide a greater net section than bolts. The HHDQ is installed directly on the sill plate and is suitable for back-to-back applications where eccentricity is a concern.

Key Features

- Strong-Drive SDS Heavy-Duty Connector screws are supplied with the holdowns to ensure proper fasteners are used
- Screws may be installed either flush or raised off the mudsill without a reduction in load value
- No stud bolts to countersink at openings

Material

- Body: 7 gauge, washer: 1/2" plate

Finish

- Simpson Strong-Tie gray paint; HHDQ11 — available in stainless steel

Installation

- See General Notes for Holdowns and Tension Ties
- Strong-Drive SDS Heavy-Duty Connector screws install best with a low-speed, high-torque drill equipped with a 3/8" hex-head driver
- No additional washer is required
- HHDQ14 requires a heavy-hex anchor nut (supplied with holdown)

Related Links

- Wood Construction Connectors Technical and Installation Notes
- General Notes for Holdowns and Tension Ties
- Holdown Selector Software

Catalog Pages

- C-C-2021 (Wood Construction Connectors), page 51



Wood Construction Connectors 2021-2023

C-C-2021 — *Catalog*

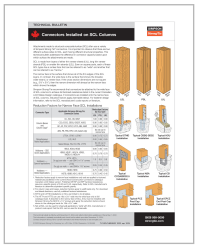
A catalog including new product information as well as any applicable specification and installation instructions for wood-to-wood and wood-to-concrete structural connectors.



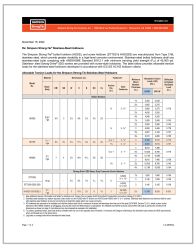
Conectores para la construcción con madera 2021–2023

C-C-2021SP — *Catalog*

El catálogo incluye información sobre nuevos productos, así como especificaciones e instrucciones de instalación aplicables para conectores estructurales de madera a madera y de madera a concreto.



Connectors Installed on SCL Columns



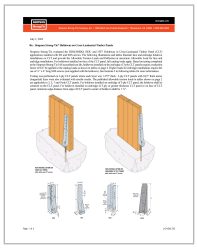
Simpson Strong-Tie® Stainless-Steel Holdowns

LMCAN23 — *Technical Bulletin*

A Canadian Limit States technical bulletin addressing typical installations of Simpson Strong-Tie® connectors on SCL columns as well as reduction factors for narrow-face SCL installations.

L-C-SSHD22 — *Engineering Letter*

The Simpson Strong-Tie bolted holdown (HDSS) and screw holdown (DTTSS & HHDQSS) are manufactured from Type 316L stainless steel, which provides superior durability in a highly corrosive environment.



Simpson Strong-Tie® Holdowns on Cross-Laminated Timber Panels

L-C-HDCLT23 — *Engineering Letter*

Simpson Strong-Tie evaluated the HDQ/HHDQ, HDU and HTT holdowns in cross-laminated timber (CLT) panel applications installed with SD and SDS screws.

Load Tables

These products are available with additional corrosion protection. Additional products on this page may also be available with this option, check with Simpson Strong-Tie for details.

SS For stainless-steel fasteners, see Fastener Types and Sizes Specified for Simpson Strong-Tie Connectors.

Model No.	Ga.	Dimensions (in.)					Fasteners		Minimum Wood Member Size (in.)	Allowable Tension Loads (160)		
		W	H	B	CL	SO	Anchor Bolt Dia. (in.)	SDS Screws		DF/SP	SPF/HF	Deflection at Allowable Load (in.)
HDQ8-SDS3	7	2⅞	14	2½	1¼	2⅜	⅞	(20) ¼" x 3"	3 x 3½	5,715	4,915	0.073
								(20) ¼" x 3"	3½ x 3½	7,630	6,560	0.091
								(20) ¼" x 3"	3½ x 4½	9,230	7,020	0.095
SS HHDQ11-SDS2.5	7	3	15⅛	3½	1½	⅞	1	(24) ¼" x 2½"	3½ x 5½	11,810	8,425	0.131
HHDQ14-SDS2.5	7	3	18¾	3½	1½	⅞	1	(30) ¼" x 2½"	3½ x 7¼	13,015	10,530	0.107
									5½ x 5½	13,710	10,530	0.107

1. HHDQ14 requires a heavy-hex anchor nut (supplied with holdown).
2. HDQ and HHDQ installed horizontally achieve compression loads with the addition of a standard nut on the underside of the load transfer plate. Refer to ICC-ES ESR-2330 for design values. HDQ8 requires a standard nut and BP 7/8-2 load washer (sold separately) on the underside of the holdown for compression load. Design of anchorage rods for compression force shall be per the Designer.

Code Reports & Compliance

Product Series	Compliance/Certification	Report/Approval
HHDQ	International Building Code International Residential Code City of Los Angeles Building Code City of Los Angeles Residential Code	ESR-2330
	Florida Building Code	FL10441

Footnotes

1. Please review compliance documents for information about specific product models. In some cases, Compliance documents cover most but not all models associated with a product series.
2. For additional information regarding Florida's Statewide Product Approval System and Miami-Dade County Notice of Acceptance (NOA), click here.

Related Products



GENERAL NOTES

1. ALL CONSTRUCTION MATERIALS AND WORKMANSHIP SHALL CONFORM TO THE REQUIREMENTS OF THE DRAWINGS, SPECIFICATIONS, AND THE CODES, RULES AND REGULATIONS OF INTERNATIONAL BUILDING CODE (IBC) 2021 EDITION.
2. THE CONTRACTOR SHALL VERIFY ALL DIMENSIONS PRIOR TO CONSTRUCTION. THE ARCHITECT SHALL BE NOTIFIED OF ANY DISCREPANCIES OR INCONSISTENCIES.
3. IF ANY ERRORS OR OMISSIONS APPEAR IN THESE DRAWINGS, SPECIFICATIONS, OR OTHER DOCUMENTS; THE CONTRACTOR SHALL NOTIFY THE STRUCTURAL ENGINEER OR ARCHITECT IN WRITING OF SUCH OMISSION OR ERROR BEFORE PROCEEDING WITH THE WORK.
4. MANUFACTURED MATERIALS SHALL BE APPROVED BY THE CHECKING AGENCY PRIOR TO THEIR USE. ALL REQUIREMENTS OF THOSE APPROVALS SHALL BE FOLLOWED.
5. ALL STRUCTURAL SYSTEMS THAT ARE TO BE COMPOSED OF MANUFACTURED COMPONENTS TO BE FIELD ERECTED SHALL BE APPROVED BY THE CHECKING AGENCY PRIOR TO THEIR USE AND SHALL BE SUPERVISED BY THE SUPPLIER DURING MANUFACTURING, DELIVERY, HANDLING, STORAGE, AND ERECTION IN ACCORDANCE WITH INSTRUCTIONS PREPARED BY THE SUPPLIER
6. FRAMING MEMBERS THAT ARE NOT DIMENSIONED SHALL BE EQUALLY SPACED BETWEEN DIMENSIONED POINT OR MEMBERS.
7. SEE ARCHITECTURAL DRAWINGS AND PROJECT SPECIFICATIONS FOR THE FOLLOWING:
   SIZE AND LOCATION OF ALL DOOR AND WINDOW OPENINGS AND THRESHOLD REQUIREMENTS.
   SIZE AND LOCATION OF ALL NON-BEARING PARTITIONS.
   SIZE AND LOCATION OF ROOF, FLOOR AND WALL OPENINGS.
   SIZE AND LOCATION OF DEPRESSED AREAS, CHANGES IN ELEVATION, FLOOR AND ROOF DRAINS,
   SLOPES, CONCRETE CURBS, LEDGES, PADS AND ISLANDS,
   CHAMFERS, GROOVES, INSERTS, ETC.
   DIMENSIONS NOT SHOWN ON THE STRUCTURAL DRAWINGS, SIZE, WEIGHT AND LOCATION OF MACHINES AND EQUIPMENT BASES.
8. THE CONTRACT DOCUMENTS REPRESENT THE FINISHED STRUCTURE, THEY DO NOT INDICATE THE METHOD OF CONSTRUCTION. THE CONTRACTOR SHALL PROVIDE ALL MEASURES NECESSARY TO PROTECT THE STRUCTURE DURING CONSTRUCTION. SUCH MEASURES SHALL INCLUDE, BUT NOT BE LIMITED TO, BRACING, SHORING FOR LOADS DUE TO CONSTRUCTION EQUIPMENT, ETC. OBSERVATION VISITS TO THE SITE BY THE STRUCTURAL ENGINEER SHALL NOT INCLUDE INSPECTION OF THE ABOVE ITEMS.
9. OPENINGS, POCKETS, ETC. SHALL NOT BE PLACED IN STRUCTURAL MEMBERS UNLESS SPECIFICALLY DETAILED ON THE STRUCTURAL DRAWINGS. NOTIFY THE STRUCTURAL ENGINEER WHEN DRAWINGS BY OTHERS SHOW OPENINGS, POCKETS, ETC., LARGER THAN 6 INCHES NOT SHOWN ON THE STRUCTURAL DRAWINGS, BUT WHICH ARE LOCATED IN STRUCTURAL MEMBERS.
10. SPECIFICATIONS, CODES, AND STANDARDS NOTED IN THE CONTRACT DOCUMENTS SHALL BE OF THE LATEST APPROVED ISSUE, INCLUDING SUPPLEMENTS, UNLESS OTHERWISE NOTED. MATERIAL SPECIFICATIONS ARE ASTM LATEST EDITION.
11. CONTRACTOR SHALL PROVIDE TEMPORARY BRACING FOR THE STRUCTURE AND STRUCTURAL COMPONENTS UNTIL ALL FINAL CONNECTIONS HAVE BEEN COMPLETED IN ACCORDANCE WITH THE PLANS.

DESIGN CRITERIA

LIVE LOADS

ROOF SNOW LOAD	25.0 PSF BASIC

DEAD LOADS

SUPERIMPOSED ROOF DEAD LOAD	
FRAMING, CEILING, ETC.	15 PSF
SUPERIMPOSED WALL DEAD LOAD	
EXTERIOR WALLS.	10 PSF

WIND DESIGN (PER 1615 -1622)	
BASIC WIND SPEED	110 MPH
EXPOSURE	B
IMPORTANCE FACTOR	1.0
TOPOGRAPHIC FACTOR	1.38

SEISMIC DESIGN (PER 1615 - 1633)
SEISMIC CATEGORY II
IMPORTANCE FACTOR= 1.0
MAPPED SPECTRAL RESPONSE ACCELERATION PARAMETERS:
Ss = 1.466
S1 = 0.508 SITE CLASS = D
Sds = 1.173 SEISMIC RISK CATEGORY = D
BASIC SEISMIC FORCE-RESISTING SYSTEMS:
LIGHT FRAMED WALLS SHEATHED WITH
WOOD STRUCTURAL PANELS RATED FOR SHEAR RESISTANCE.
DESIGN BASE SHEAR: 47.88 KIPS
R= 6.5 - Wood Framed
R = 5.0 - Concrete
ANALYSIS METHODS USED:
WIND; METHOD 2 - ANALYTICAL PROCEDURE
SEISMIC; METHOD 2 - EQUIVALENT LATERAL FORCE

MAPPED SPECTRAL RESPONSE
ACCELERATIONS OBTAINED FROM THE USGS - SEISMIC HAZARD MAPS & DATA

FOUNDATIONS

1. ALL FOUNDATIONS SHALL BE FOUNDED A MINIMUM OF 18" BELOW LOWEST ADJACENT FINAL FINISH FLOOR OR GRADE. EXPOSED SOIL SHALL BE INSPECTED FOR COMPLIANCE BY THE ENGINEER OR HIS REPRESENTATIVE PRIOR TO CONSTRUCTING CONCRETE FORMS AND/OR PLACING REINFORCING STEEL. ANY EXCESS OR NON-COMPLYING MATERIAL AS DETERMINED BY THE ENGINEER OR HIS REPRESENTATIVE SHALL BE REMOVED AND REPLACED AS DIRECTED.
2. THE ALLOWABLE SOIL BEARING LOAD IS PER THE GEOTECHNICAL REPORT.

REINFORCING STEEL

1. REINFORCING STEEL SHALL BE DETAILED, INCLUDING HOOKS AND BENDS, AND PLACED IN ACCORDANCE WITH ACI 315 AND ACI 318.
2. REINFORCING STEEL SHALL CONFORM TO ASTM A-615 OR A-706, GRADE 40 OR BETTER,
3. WELDED WIRE FABRIC SHALL CONFORM TO ASTM A-185,
4. ALL REINFORCING BAR BENDS SHALL BE MADE COLD,
5. REINFORCING SPLICES SHALL BE MADE AS INDICATED ON THE DRAWINGS,
6. DOWELS BETWEEN FOOTINGS AND WALLS OR COLUMNS SHALL BE THE SAME GRADE, SIZE AND SPACING AS THE VERTICAL REINFORCING, RESPECTIVELY, UON,
7. NO BARS PARTIALLY EMBEDDED IN HARDENED CONCRETE SHALL BE FIELD BENT UNLESS SPECIFICALLY SO DETAILED AND REVIEWED BY THE STRUCTURAL ENGINEER
8. WELDING OF REINFORCEMENT SHALL BE WITH LOW HYDROGEN ELECTRODES IN CONFORMANCE WITH ACI 318-95 AND THE RECOMMENDATIONS OF THE AMERICAN WELDING SOCIETY, AWS DI,4 AND WITH THE REVIEW OF THE STRUCTURAL ENGINEER

CONCRETE

1. ALL CONCRETE CONSTRUCTION SHALL CONFORM TO THE BUILDING CODE REQUIREMENTS FOR REINFORCED CONCRETE' ACI 318 AND ACI 301, WITH MODIFICATIONS AS NOTED IN THE CONTRACT DOCUMENTS,
2. PORTLAND CEMENT SHALL CONFORM TO ASTM C-150 TYPE 1 OR TYPE H,
3. COARSE AND FINE AGGREGATE FOR NORMAL WEIGHT CONCRETE SHALL CONFORM TO ASTM C-33,
4. WATER SHALL BE CLEAR AND SHALL CONFORM TO ASTM C-94,
5. CONCRETE MIXING OPERATION SHALL CONFORM TO ASTM C-94,
6. ADD TO ALL CONCRETE EXPOSED TO WEATHER MICROAIR OR MBVR AIR ENTRAINING AGENT TO ATTAIN 5 PERCENT +1- 1 PERCENT ENTRAINED AIR, BY VOLUME, CONFORMING TO ASTM C-260, ALL REFERENCE DATA USED FOR PAST PERFORMANCE DESIGN SHALL HAVE CONTAINED THE SAME ADMIXTURE BRAND AS THAT USED IN THE MIX SUBMITTED,
7. CONCRETE STRENGTHS SHALL BE VERIFIED BY 28-DAY CYLINDER TESTS, UNLESS OTHERWISE APPROVED, CONCRETE SHALL BE AS FOLLOWS:

ELEMENT TYPE	STRENGTH PSI CONCRETE
FOOTINGS, GRADE BEAMS	2,500 NORMAL WT
SLAB ON GRADE	2,500 NORMAL WT
FOUNDATION STEM WALLS	3,000 NORMAL WT
RETAINING WALLS	3,000 NORMAL WT

A MINIMUM 5 SACK MIX SHALL BE USED TO ACHIEVE THE DESIGN STRENGTHS LISTED ABOVE.

8. CONTRACTOR MAY USE AN ADMIXTURE SYSTEM TO PRODUCE FLOWABLE CONCRETE, MAXIMUM SLUMP SHALL NOT EXCEED 10 INCHES MEASURED AT THE PUMP, THE WATER/CEMENTIOUS MATERIAL RATIO OF THE APPROVED MIXES SHALL BE MAINTAINED OR LOWERED WHEN FLOWABLE CONCRETE IS USED,
9. THE FOLLOWING MINIMUM CONCRETE COVER SHALL BE PROVIDED FOR REINFORCEMENT PLACED IN CAST-IN-PLACE CONCRETE:

	CONCRETE COVER (MINIMUM)
A . CONCRETE CAST AGAINST AND PERMANENTLY EXPOSED TO EARTH	3"
B. CONCRETE EXPOSED TO EARTH OR WEATHER:	
#6 THROUGH #18 BARS	2"
#5 BAR, W31 OR D31 WIRE, A1413 SMALLER	1 1/2"
C. CONCRETE NOT EXPOSED TO WEATHER OR IN CONTACT WITH GROUND:	
SLABS, WALLS, JOISTS	
#14 AND #18 BARS	1 1/2"
#11 BARS AND SMALLER	3/4"
BEAMS, COLUMNS:	
PRIMARY REINFORCEMENT, TIES, STIRRUPS, SPIRALS	1 1/2"

10. PLACEMENT OF CONCRETE SHALL CONFORM TO ACI 304 AND THE CONTRACT DOCUMENTS. SANDBLAST ALL CONCRETE SURFACES AGAINST WHICH CONCRETE IS TO BE PLACED,
11. ALL REINFORCING BARS, ANCHOR BOLTS AND OTHER CONCRETE INSERTS SHALL BE WELL SECURED IN POSITION PRIOR TO PLACING CONCRETE,
12. PROVIDE SLEEVES FOR PLUMBING AND ELECTRICAL OPENINGS IN CONCRETE BEFORE PLACING, REINFORCING SHALL NOT BE CUT, CORING OF CONCRETE IS NOT PERMITTED EXCEPT AS INDICATED,
13. CURING COMPOUNDS USED ON CONCRETE TO RECEIVE A FINISH SHALL BE APPROVED BY THE FINISH APPLICATOR BEFORE USE.

DESIGN LOADING:
REF. SOIL REPORT
EARTH SOLUTIONS NW, LLC
Dated: October 4, 2023
Pa = 42 PCF
Pp = 200 PCF
Seismic loading = 8H
Allowable Bearing Pressure = 2,500 PSF

WOOD

1. FRAMING LUMBER SHALL BE GRADED AND MARKED IN CONFORMANCE WITH WCLB STANDARD GRADING AND DRESSING RULES FOR WEST COAST LUMBER NO, 16, LATEST EDITION. UNLESS OTHERWISE NOTED ON THE DRAWINGS, LUMBER GRADES SHALL BE AS FOLLOWS:
   A. JOISTS: 2" AND 3" THICKNESS, HEM FIR NO, 1,
   B. BEAMS AND STRINGERS: DOUGLAS FIR NO, 1,
   C. POST AND TIMBERS: DOUGLAS FIR NO, 1,
   D. PLATES AND MISCELLANEOUS LIGHT FRAMING: HEM FIR STANDARD,
   E. STUDS: HEM FIR STUD.
   F. ALL BOLTED CONNECTIONS TO BE 3/4"Ø A302 BOLTS

2. MINIMUM NAILING REQUIREMENTS:

   UNLESS OTHERWISE NOTED, MINIMUM NAILING SHALL CONFORM TO THE GOVERNING CODE AND AS FOLLOWS:
   A. JOISTS OR RAFTERS TO SIDES OF STUDS 8-INCH OR LESS 3-16DB
   B. FOR EACH ADDITIONAL 4-INCH IN DEPTH OF JOISTS 1-16DC
   C. JOISTS OR RAFTERS AT ALL BEARINGS - TOENAILS EACH SIDE 2-10DD
   D. STUDS TO BEARING - TOENAILS EACH SIDE 2-10DE
   E. BLOCKING BETWEEN JOISTS OR RAFTERS TO JOIST OR RAFTERS - TOENAILS EACH SIDE EACH END 2-10D TO JOIST OR RAFTER BEARINGS - TOENAILS EACH SIDE 2-10D
   F. CROSS-BRIDGING BETWEEN JOISTS OR RAFTERS TOE NAILS EACH END 2-8D
   G. BLOCKING BETWEEN STUDS - TOENAILS EACH END 2-10D
   H. DOUBLE TOP PLATES - LOWER PLATE TO TOP OF STUD 2-16D
   J. UPPER TO LOWER PLATE - STAGGERED 16D @ 16" O,C,
   K. MULTIPLE JOISTS - STAGGERED 16D @ 12" O,C,
   L. MULTIPLE JOISTS STAGGER FOR WIDTHS MORE THAN 4 INCHES 16D @ 12" O,C.

3. INDIVIDUAL MEMBERS OF BUILT-UP POSTS AND BEAMS SHALL EACH BE ATTACHED WITH 16D SPIKES AT 12" O,C. STAGGERED, MIN,
4. ALL NAILS SHALL BE COMMON WIRE NAILS, WHENEVER POSSIBLE, NAILS DRIVEN PERPENDICULAR TO THE GRAIN SHALL BE USED, THERE SHALL BE A MINIMUM OF 2 NAILS AT ALL WOOD CONTACTS AND JOINTS USING 8D NAILS FOR 1-INCH THICK MATERIAL, 16D NAILS FOR 2-INCH THICK MATERIAL, AND 40D NAILS FOR 3-INCH THICK MATERIAL, ALL CONTINUOUS CONTACTS PROVIDE MINIMUM NAILS AT 12" O,C, WITH NAIL SIZES AS CALLED ABOVE,
5. NOTATIONS ON DRAWINGS RELATING TO FRAMING CLIPS, JOIST HANGERS, AND OTHER CONNECTING DEVICES REFER TO CATALOG NUMBERS OF STRONG-TIE CONNECTORS MANUFACTURED BY THE SIMPSON COMPANY, EQUIVALENT DEVICES BY OTHER MANUFACTURERS MAY BE SUBSTITUTED PROVIDED THAT THEY HAVE ICBO APPROVAL FOR EQUAL OR GREATER LOAD CAPACITIES AND ARE REVIEWED BY THE STRUCTURAL ENGINEER,
6. AT SAWN TIMBER JOISTS WITH THICKNESS-TO-DEPTH RATIO OF 1:6 AND GREATER, PROVIDE CROSS-BRIDGING AT 8' 0" O,C, AND SOLID BLOCKING AT BEARING POINTS,
7. ALL WOOD FRAMING DETAILS NOT SHOWN OTHERWISE SHALL BE CONSTRUCTED TO THE MINIMUM STANDARDS OF THE GOVERNING CODE,
8. ALL BEARING AND EXTERIOR STUD WALLS SHALL BE 2X6 @6"O.C. BELOW SECOND FLOOR AND 2X4 @ 16" O.C. ELSEWHERE, UNLESS OTHERWISE NOTED.
9. PROVIDE CONTINUOUS SOLID BLOCKING AT MID-HEIGHTS AND AT INTERVALS NOT TO EXCEED 8 FEET OF ALL STUD-BEARING WALLS OVER 8 FEET IN HEIGHT,
10. SEE ARCHITECTURAL DRAWINGS FOR LOCATIONS OF INTERIOR NONBEARING STUD PARTITIONS FOR LOCATION AND SIZE OF OPENINGS IN STUD WALLS, AND FOR ALL WALL FINISH DETAILS,
11. ALL CANTS AND CRICKETS SHALL BE PLACED OVER BASIC ROOF SHEATHING. SEE ARCHITECTURAL DRAWINGS FOR DETAILS AND LOCATIONS,
12. ALL WOOD STUD WALL SILL PLATES SHALL BE ATTACHED TO CONCRETE OR MASONRY WITH 1/2-INCH DIAMETER ANCHOR BOLTS AT 48" O.C., UNLESS OTHERWISE NOTED ,
13. ALL WOOD STUD WALLS SHALL HAVE LOWER WOOD PLATE ATTACHED TO WOOD FRAMING BELOW WITH 16D NAILS AT 6" 0.C, STAGGERED UNLESS SHOWN OTHERWISE.
14. FASTEN ALL POSTS TO CONCRETE WITH "CB" COLUMN BASE OR EQUAL,
15. ALL WOOD PLATES AND BLOCKING IN DIRECT CONTACT WITH CONCRETE OR MASONRY SHALL BE PRESSURE TREATED WITH AN APPROVED PRESERVATIVE IN ACCORDANCE WITH AWPS-FDN, AND BEAR THAT QUALITY MARK,
16. PROVIDE STANDARD CUT WASHERS UNDER ALL BOLTS HEADS AND NUTS IN CONTACT WITH WOOD.
17. ATTACH TIMBER JOISTS TO FLUSH HEADERS AND BEAMS WITH "U" SERIES METAL JOIST HANGERS TO SUIT THE JOIST SIZE.
18. ALL PLYWOOD SHALL BE HEM FIR, STRUCTURAL 2 OR BETTER AND SHALL CONFORM TO APA C-D INTERIOR GRADE WITH EXTERIOR GLUE, WITH UBC STANDARD 23-2 AND WITH PRODUCT STANDARD PSI, WOOD-BASED STRUCTURAL-USE PANELS SHALL CONFORM WITH UBC STANDARD 23-3 AND WITH PRODUCT STANDARD PS2, TYPE AND THICKNESS SHALL BE AS SPECIFIED ON THE PLANS.
19. PLYWOOD NAILING, USE UNLESS OTHERWISE NOTED:

A. ROOF:	8D @ 6" O.C. AT SHEET EDGES
	8D @ 12" O.C. AT INTERMEDIATE BEARING POINTS
B. FLOOR:	10D @ 6" O.C. AT SHEET EDGES
	10D @ 10" O.C. AT INTERMEDIATE BEARING POINTS
C. WALLS:	8D @ 6" O.C. AT EDGES
	8D @ 12" O.C. AT INTERMEDIATE BEARING POINTS

PLYWOOD AND WOOD-BASED STRUCTURAL-USE PANELS USED FOR WALL SHEATHING SHALL HAVE SOLID BLOCKING AT ALL EDGES.
20. MACHINE APPLIED NAILING IS SUBJECT TO A SATISFACTORY DEMONSTRATION AND THE APPROVAL OF THE CHECKING AGENCY AND THE ARCHITECT, NAIL HEADS SHALL NOT PENETRATE THE OUTER PLY MORE THAN WOULD BE NORMAL FOR A HAND HAMMER. EDGE DISTANCES SHALL BE MAINTAINED, SHINERS SHALL BE REMOVED AND REPLACED, THE APPROVAL IS SUBJECT TO CONTINUED SATISFACTORY PERFORMANCE, MACHINE APPLIED NAILING ONLY ON PLYWOOD GREATER THAN 5/16".

STRUCTURAL STEEL, MISC. METAL

1. STRUCTURAL STEEL DETAILING, FABRICATION AND ERECTION SHALL BE BASED ON THE LATEST EDITION AND SUPPLEMENTS OF THE AISC "SPECIFICATION FOR STRUCTURAL STEEL FOR BUILDINGS - ALLOWABLE STRESS DESIGN AND PLASTIC DESIGN", STRUCTURAL STEEL SHALL CONFORM TO THE FOLLOWING REQUIREMENTS,

TYPE OF MEMBER	ASTM SPECIFICATION	FY
WIDE FLANGE SHAPES	A572 OR A992	50 KSI
PLATES, SHAPES, ANGLES, AND RODS	A36	36 KSI
HOLLOW STRUCTURAL SECTION (ROUND)	A53 (GRADE B)	36 KSI
HOLLOW STRUCTURAL SECTION (SQUARE OR RECTANGLE)	A500 (GRADE B)	46 KSI
ANCHOR RODS (EMBEDDED IN CONCRETE)	A307	

2. ALL WELDS SHALL BE PREQUALIFIED IN CONFORMANCE WITH AISC AND AWS STANDARDS AND SHALL BE PERFORMED BY WELDERS CERTIFIED IN THE JURISDICTION HAVING AUTHORITY OVER THIS PORTION OF THE WORK, USE E70XX ELECTRODES.3, WELD LENGTHS CALLED FOR ON THE PLANS ARE THE NET EFFECTIVE LENGTH REQUIRED, WELD SIZE SHALL BE AISC MINIMUM, UNLESS OTHERWISE NOTED.

ANCHORAGE

1. EXPANSION ANCHORS SHALL BE ZINC PLATED IN ACCORDANCE WITH ASTM B 633, AND CONFORM WITH FS FF-S-325, GROUP II, TYPE 4, CLASS 1,
2. SLEEVE ANCHORS SHALL BE ZINC PLATED IN ACCORDANCE WITH ASTM B 633, AND CONFORM WITH FS FF-S-325, GROUP II, TYPE 3, CLASS 3,
3. FLUSH SHELL ANCHORS SHALL ZINC PLATED IN ACCORDANCE WITH ASTM B 633, AND CONFORM WITH FS FF-S-325, GROUP VIII, TYPE 1,
4. ADHESIVE ANCHORS SHALL CONSIST OF ALL-THREAD ANCHOR ROD, NUT, WASHER AND EPOXY INJECTION GEL OR ADHESIVE CAPSULE SYSTEM, ANCHOR RODS SHALL BE MANUFACTURED FROM A-36 MATERIAL, ZINC PLATED IN ACCORDANCE WITH ASTM B 633,
5. ALL RELATED PRODUCTS, MATERIALS AND INSTALLATION SHALL BE IN ACCORDANCE WITH THE MANUFACTURER'S RECOMMENDATIONS,
6. NOTATIONS ON DRAWINGS RELATING TO EXPANSION, SLEEVE, FLUSH OR ADHESIVE ANCHORS AND OTHER CONNECTING DEVICES REFER TO CONNECTORS MANUFACTURED BY POWERS FASTENING, INC, EQUIVALENT DEVICES BY OTHER MANUFACTURERS MAY BE SUBSTITUTED PROVIDED THAT THEY HAVE ICBO APPROVAL FOR EQUAL OR GREATER LOAD CAPACITIES AND ARE REVIEWED BY THE STRUCTURAL ENGINEER

SPECIAL INSPECTION

1. SPECIAL INSPECTION BY A REGISTERED DEPUTY BUILDING INSPECTOR, APPROVED BY THE ARCHITECT AND THE CHECKING AGENCY SHALL BE REQUIRED FOR THE FOLLOWING TYPES OF WORK. SEE THE PROJECT SPECIFICATIONS FOR FURTHER REQUIREMENTS, SPECIAL INSPECTIONS SHALL NOT BE REQUIRED WHEN THE WORK IS DONE ON THE PREMISES OF A FABRICATOR REGISTERED AND APPROVED BY THE BUILDING OFFICIAL TO PERFORM SUCH WORK WITHOUT SPECIAL INSPECTION,
   SOIL
   EXCAVATION
   SOIL COMPACTION
   CONCRETE
   DESIGN STRENGTHS GREATER THAN 2,500 PSI PLACING OF REINFORCING STEEL
   WELDING
   STRUCTURAL STEEL
   REINFORCING STEEL
   FABRICATED TIMBER JOISTS
   EXPANSION TYPE ANCHOR BOLTS
   STRUCTURAL MASONRY CONSTRUCTION
   PILING, DRILLED OR DRIVEN
   STRUCTURAL STEEL FABRICATION

2. ALL PREPARED SOIL-BEARING SURFACES SHALL BE INSPECTED BY THE SOILS ENGINEER PRIOR TO PLACEMENT OF REINFORCING STEEL,
3. EXPANSION TYPE ANCHORS SHALL BE APPROVED BY THE CHECKING AGENCY FOR THEIR USE AND SHALL BE INSTALLED ACCORDING TO THE MANUFACTURER'S RECOMMENDATIONS,
4. THE OWNER, ARCHITECT, STRUCTURAL ENGINEER, AND BUILDING OFFICIAL SHALL BE FURNISHED WITH COPIES OF ALL TEST RESULTS.

LANZ RESIDENCE
8020 SE 57th Street
Mercer Island, WA 98040

House Structure
Notes

LUCIA ENGINEERING, INC.
12527 Huckleberry Lane
Arlington, Washington 98223
PHONE: (206) 790-8039
E-MAIL: joe@luciaeng.com

JOSEPH M. LUCIA
STATE OF WASHINGTON
23314
REGISTERED
PROFESSIONAL ENGINEER
08-15-24

Number	Date	By	Description
5	08-15-24	JML	

SHEET
S-5.0

SHEAR WALL SCHEDULE

MARK	SHEATHING	NAILING (5)		LUMBER			SHEAR TRANSFER					1.4 INCREASE FOR WIND
		EDGE (E.N.)	FIELD	ALLOWABLE SHEAR	SILL PL	TOP PL'S	"A" SILL PL TO CONC.	"B" BLKG TO TOP PL.	"C" SILL PL RIM/JST/BLKG (F.N.)	"D" SHEAR WALL INTERSECTIONS	CAPACITY	CAPACITY
P1-8-6	3/8" APA RATED SHEATHING, ONE SIDE	8d@6"	8d@ 6"	2x	2x	(2)2x	5/8 @ 48"	A35@20" OR LPT4 @ 30"	16d @ 5"	16d @ 8"	270 PLF	378 PLF
P1-8-4	3/8" APA RATED SHEATHING, ONE SIDE	8d@4"	8d@ 6"	2x	2x	(2)2x	5/8 @ 40"	A35@16" OR LPT4 @ 20"	16d @ 5"	16d @ 5"	360 PLF	504 PLF
P1-8-3	3/8" APA RATED SHEATHING, ONE SIDE	8d@2-1/2"	8d@4"	2x	3x	(2)2x	5/8 @ 36"	A35@12" OR LPT4 @ 15"	20d @ 4"	16d @ 3 1/2"	530 PLF	742 PLF
P1-8-2	3/8" APA RATED SHEATHING, ONE SIDE	8d@2"	8d@ 3"	3x(9)	3x	(2)2x	5/8 @ 24"	A35@9" OR LPT4 @ 11"	20d @ 3"	1/2" x4 1/2" LAG @ 9"	610 PLF	854 PLF
P2-8-4	3/8" APA RATED SHEATHING, TWO SIDE	8d@4"	8d@ 6"	3x(9)	3x	(2)2x	5/8 @ 12"	LPT4 @ 9"	(2)ROWS 20d @ 3"	1/2" x4 1/2" LAG @ 6"	720 PLF	1008 PLF
P2-8-3	3/8" APA RATED SHEATHING, TWO SIDE	8d@2"	8d@ 6"	3x(9)	3x	(2)2x	5/8 @ 12"	LPT4 @ 7"	(2)ROWS 20d @ 3"	1/2" x4 1/2" LAG @ 5"	980 PLF	1372 PLF
P2-8-2	3/8" APA RATED SHEATHING, TWO SIDE	8d@2"	8d@3"	3x(9)	3x	(2)2x	5/8 @ 12"	LPT4 @ 6"	(2)ROWS 20d @ 3"	1/2" x4 1/2" LAG @ 4 1/2"	1220 PLF	1708 PLF

ROOF & FLOOR DIAPHRAGM NAILING SCHEDULE

DIA. #	DIAPHRAGM SHEATHING	NAILING (INCHES o.c.) 15/32" SHEATHING W/ 10d COMMON			
		EDGE (E.N.)	FIELD	ALLOWABLE SHEAR (KLF)	NOTES
	UNBLOCKED, OTHER		6	0.20	2x
	UNBLOCKED CASE#1		6	0.28	2x
1	BLOCKED	6	6	0.32	3x
2	BLOCKED	4	6	0.43	3x
3	BLOCKED	2.5	4	0.67	3x
4	BLOCKED	2	3	0.73	3x
5	BLOCKED	2	3	0.82	3x

DIAPHRAGM NOTES:

1. APA RATED SHEATHING, STURD-I-FLOOR EXP1/EXP2/EXT OR C-C/C-D PLYWOOD
2. STRUCTURAL 1 APA RATED SHEATHINGEXP1/EXT OR STRUCT 1 PLYWOOD
3. PROVIDE 3x'S (76mm) AT ADJOINING PANEL EDGES W/NAILS STAGGERED.
4. ALL MEMBERS TO BE 4x MINIMUM W/2 LINES OF FASTENERS (ICBO ER 1952)
5. ALL MEMBERS TO BE 4x MINIMUM W/3 LINES OF FASTENERS (ICBO ER 1952)
6. SPECIAL INSPECTION REQUIRED IN ACCORDANCE WITH ICBO ER 1952
7. PROVIDE BOUNDARY NAILING @ ALL PANEL EDGES, CASES 3,4,5 & 6.
8. ALL MEMBERS TO BE 3x (76mm) MINIMUM.

SHEAR WALL FRAMING NOTES:

1. IN ADDITION TO THE TYPICAL WALL FRAMING REQUIREMENTS PROVIDE FRAMING AT SHEAR WALLS AS INDICATED.
2. SEE SCHEDULE FOR SHEATHING AND NAILING REQUIRED. SCHEDULE ASSUMES HEM-FIR OR BETTER LUMBER. STAGGER PANEL JOINTS EACH SIDE OF WALL WHERE SHEATHING IS REQUIRED BOTH SIDE OF WALL.
3. STUD BLOCKING THICKNESS SHOWN ARE MINIMUM SIZES BASED ON SHEAR WALL NAILING REQUIREMENT. PROVIDE LARGER STUD WHERE REQUIRED OTHERWISE.
4 BLOCK ALL PANEL EDGES.
5. 10d SHALL BE 0.148x3". 8d SHALL BE 0.131X2 ½". DRIVE ALL NAILS FLUSH WITH THE FACE OF . TOLERANCE IS +1/16 to -0
6. PLATES ON CONCRETE SHALL BE TREATED. SEE GENERAL STRUCTURAL NOTES.
7. NAIL OR LAG SHEATHING & STUD AT SHEAR WALL INTERSECTION AS INDICATED.
8. WHERE ONLY ONE HOLDOWN IS SPECIFIED LOCATE ON OPENING SIDE OF HOLDOWN STUDS. SEE WALL ELEVATION AT RIGHT.
9. (2)2x MAY BE USED IN LIEU OF 3x AT PANEL JOINTS. STITCH NAIL THE STUDS TOGETHER PER SHEAR TRANSFER 'C'. SEE 'PLAN VIEW 1'. REFER TO APA TECHNICAL PUBLICATION TT-076.

TYPICAL WALL FRAMING NOTES:

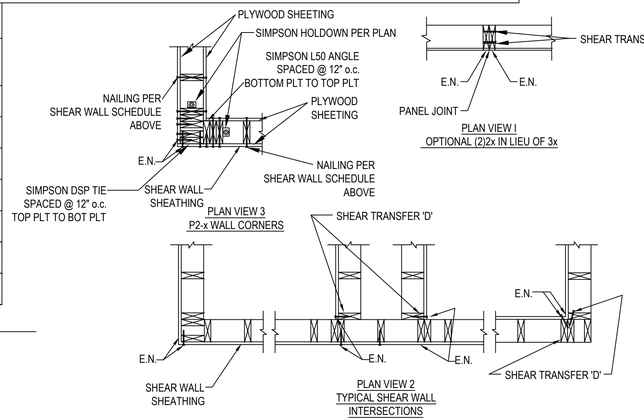
1. PROVIDE TYPICAL WALL FRAMING INDICATED, EXCEPT WHERE NOTED OTHERWISE.
2. SEE ARCHITECTURAL DRAWINGS FOR FIRE BLOCKING AND BACKING FOR FINISHES AND FURNISHINGS.

TYPICAL ROOF & FLOOR DIAPHRAGM FRAMING NOTES:

1. ROOF AND FLOOR DIAPHRAGMS ARE UNBLOCKED, U.L.N. AND NAILED ACCORDING TO THE FASTENING SCHEDULE OF IBC TABLE 2304.9.1.

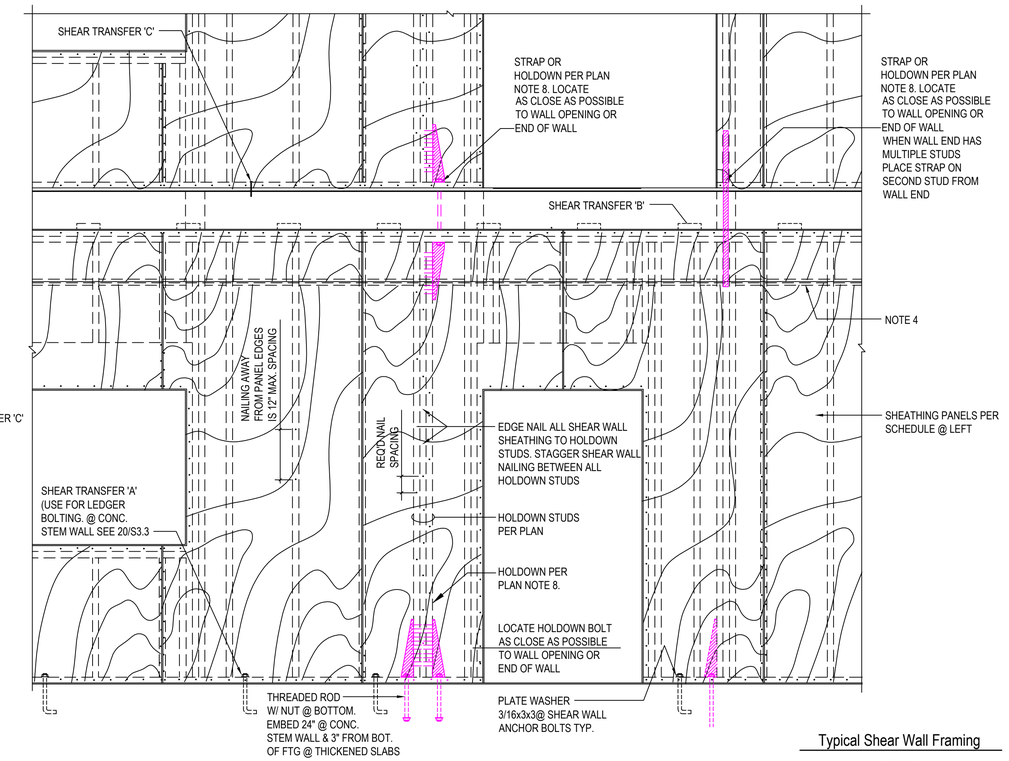
HEADER END NAILING	
NOMINAL DEPTH	END ATTACHMENT
4	(4)16d
6	(6)16d
8	(8)16d
10	(10)16d
12	(12)16d
14	(14)16d
16	(16)16d
18	(18)16d

ROUGH WINDOW SILL			
HORIZ ROUGH OPENING	NUMBER OF SILLS REQUIRED	END ATTACHMENT	REF.
0 TO 6'	1	(2)16d END NAIL	20/S6.1
> 6'	2	(2)16d END NAIL, +A35 EA END @ EA SILL	20/S6.1

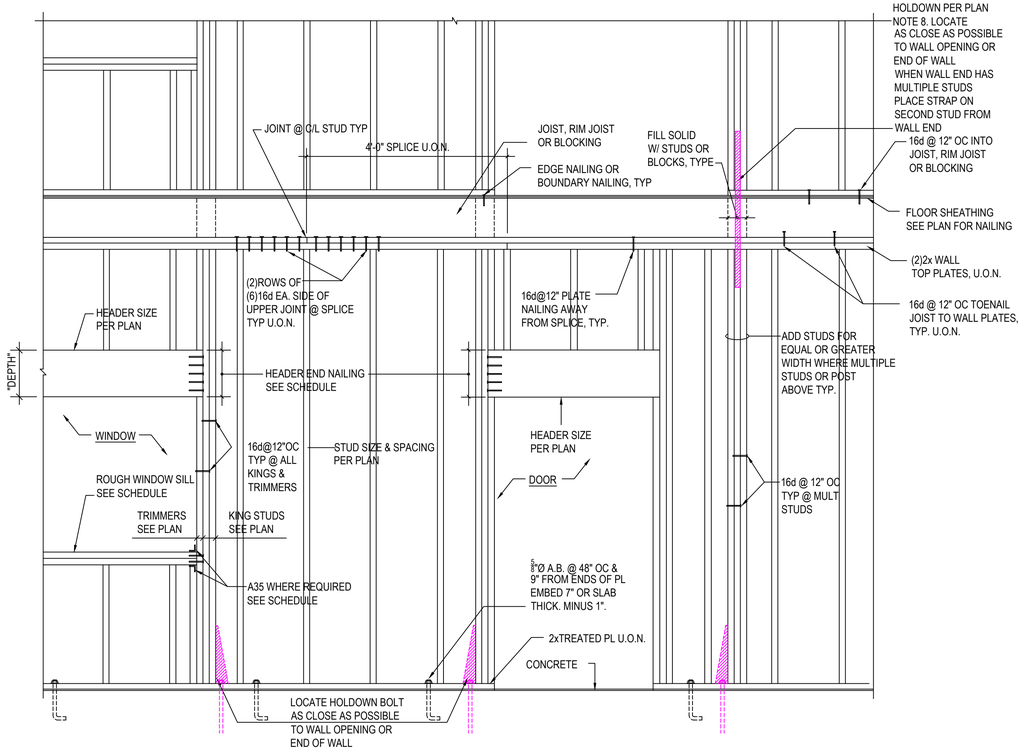


MINIMUM NAILING SCHEDULE

CONNECTION	NAILS
1. Joist to sill or girder, toenail	(3) 8d
2. Bridging to joist, toenail each end	(2) 8d
3. 1" x 6" sub floor or less to each joist, face nail	(2) 8d
4. Wider than 1"x6" sub floor to each joist, face nail	(3)8d
5. 2" subfloor to joist or girder, blind and face nail	(2)16d
6. Sole plate to joist or blocking, typical face nail	16d at 16" o.c.
Sole plate to joist or blocking, at braced wall panels	(3)16d per 16"
7. Top plates to stud, end nail	(4)16d
8. Stud to sole plate	(4)8d, toenail or (2) 16d, end nail
9. Double stud, face nail	16d at 24" o.c.
10. Double top plates, typical face nail	16d at 16" o.c.
Double top plates, lap splice	(8)16d
11. Blocking between joist or rafters to top plate, toenail	(3)8d
12. Rim joist to top plate, toenail	8d at 6" o.c.
13. Top plates, laps and intersections, face nail	(2)16d
14. Continuous header, two pieces	16d at 16" o.c. along each edge
15. Ceiling joist to plate, toenail	(3)8d
16. Continuous header to studs, toenail	(4)8d
17. Ceiling joist, lap over partitions face nail	(3)16d
18. Ceiling joist to parallel rafters, face nail	(3)16d
19. Rafter to plate, toenail	(3)8d
20. 1" brace to each stud and plate, face nail	(2)8d
21. 1"X8" sheathing or less to each bearing , face nail	(2)8d
22. Wider than 1"x8" sheathing to each bearing face nail	(5)8d
23. Built up corner studs	16d at 24" o.c.
24. Built up girder and beams	



Typical Shear Wall Framing



Typical Wall Framing
Scale: N.T.C.

LANZ RESIDENCE
8020 SE 57th Street
Mercer Island, WA 98040

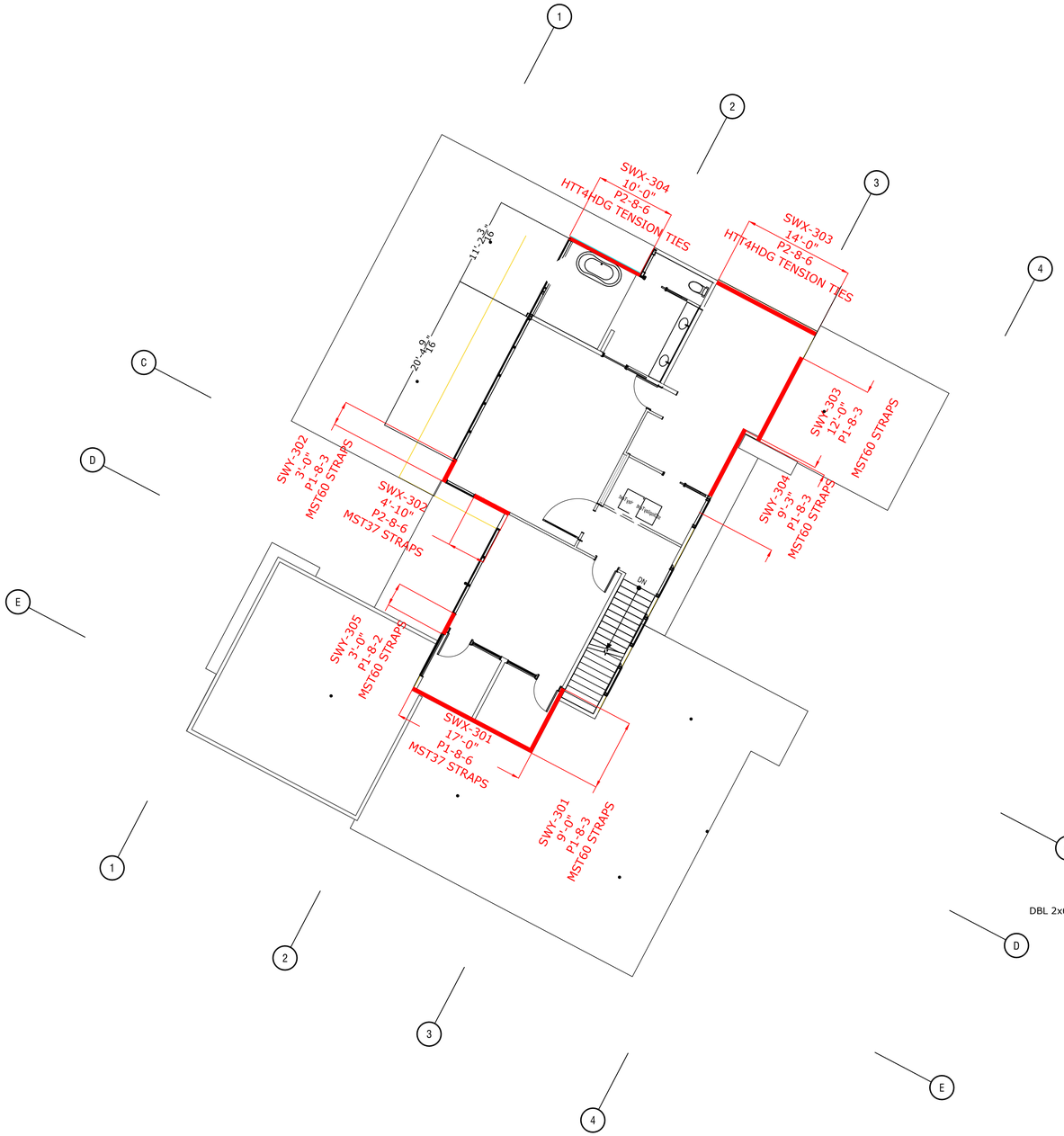
Shear Wall Details

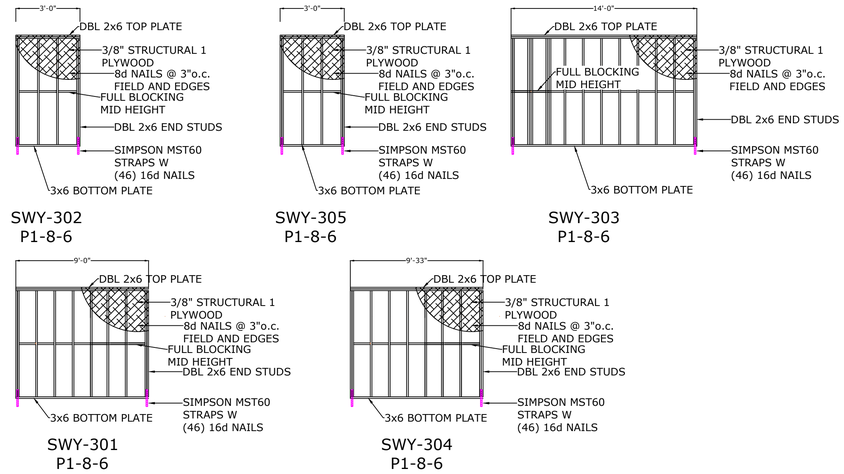
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12527 Huckleberry Lane
Arlington, Washington 98223
PHONE: (206) 790-8039
E-MAIL: joe@luciaeng.com

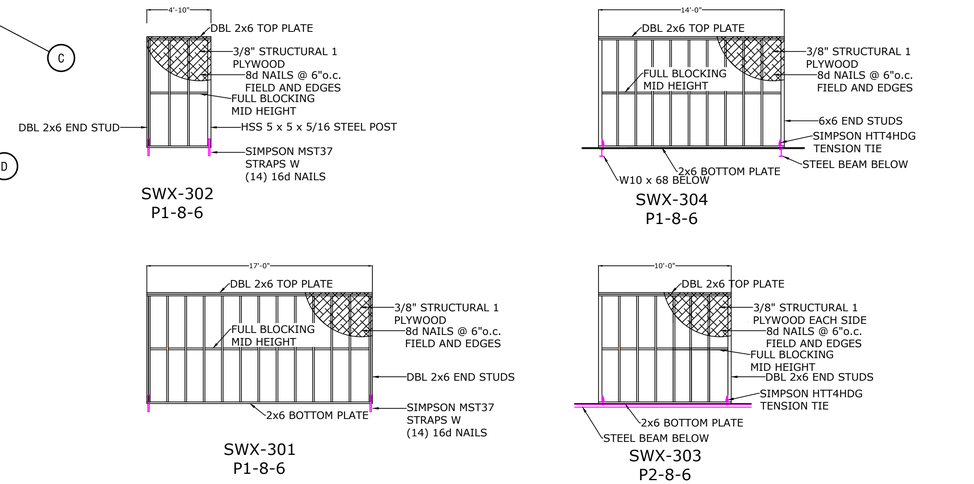
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08-15-24

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5	08-15-24	JML	

SHEET
S-6.0







SECOND FLOOR LEVEL - SHEAR WALLS

LANZ RESIDENCE
8020 SE 57th Street
Mercer Island, WA 98040

Second Level
Shear Walls

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E-MAIL: joe@luciaeng.com

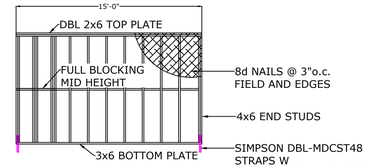
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08-15-24

Number	Date	By	Description
5	08-15-24	JML	

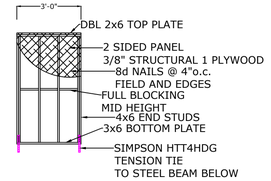
SHEET
S-7.0



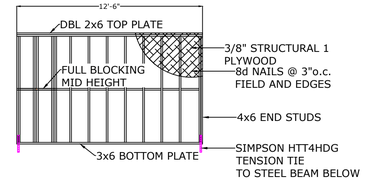
FIRST FLOOR LEVEL - SHEAR WALLS



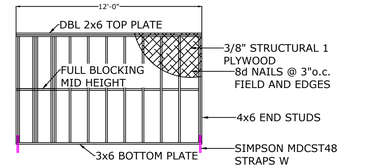
SWY-203
P1-8-6



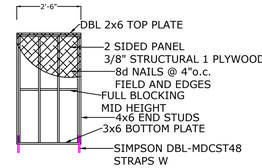
SWY-204
P1-8-6



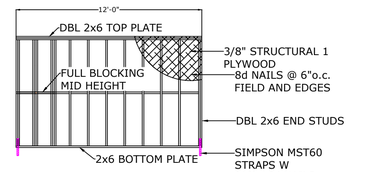
SWY-205
P1-8-6



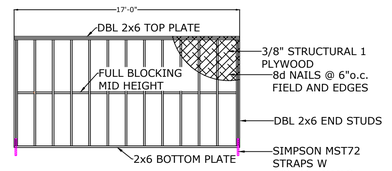
SWY-201
P1-8-6



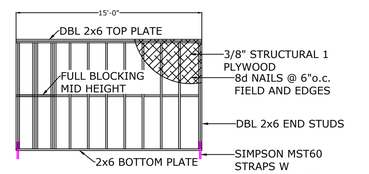
SWY-202
P1-8-6



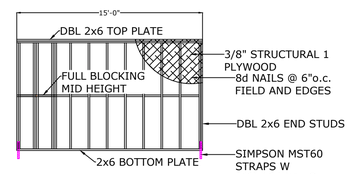
SWX-203
P1-8-6



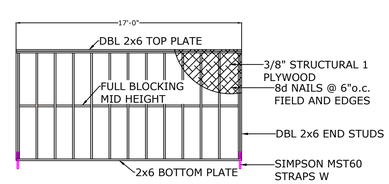
SWX-206
P1-8-6



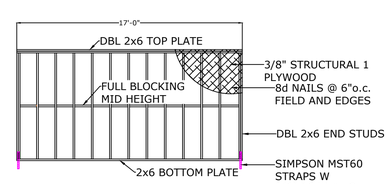
SWX-204
P1-8-6



SWX-205
P1-8-6



SWX-201
P1-8-6



SWX-202
P1-8-6

LANZ RESIDENCE
8020 SE 57th Street
Mercer Island, WA 98040

First Level
Shear Walls

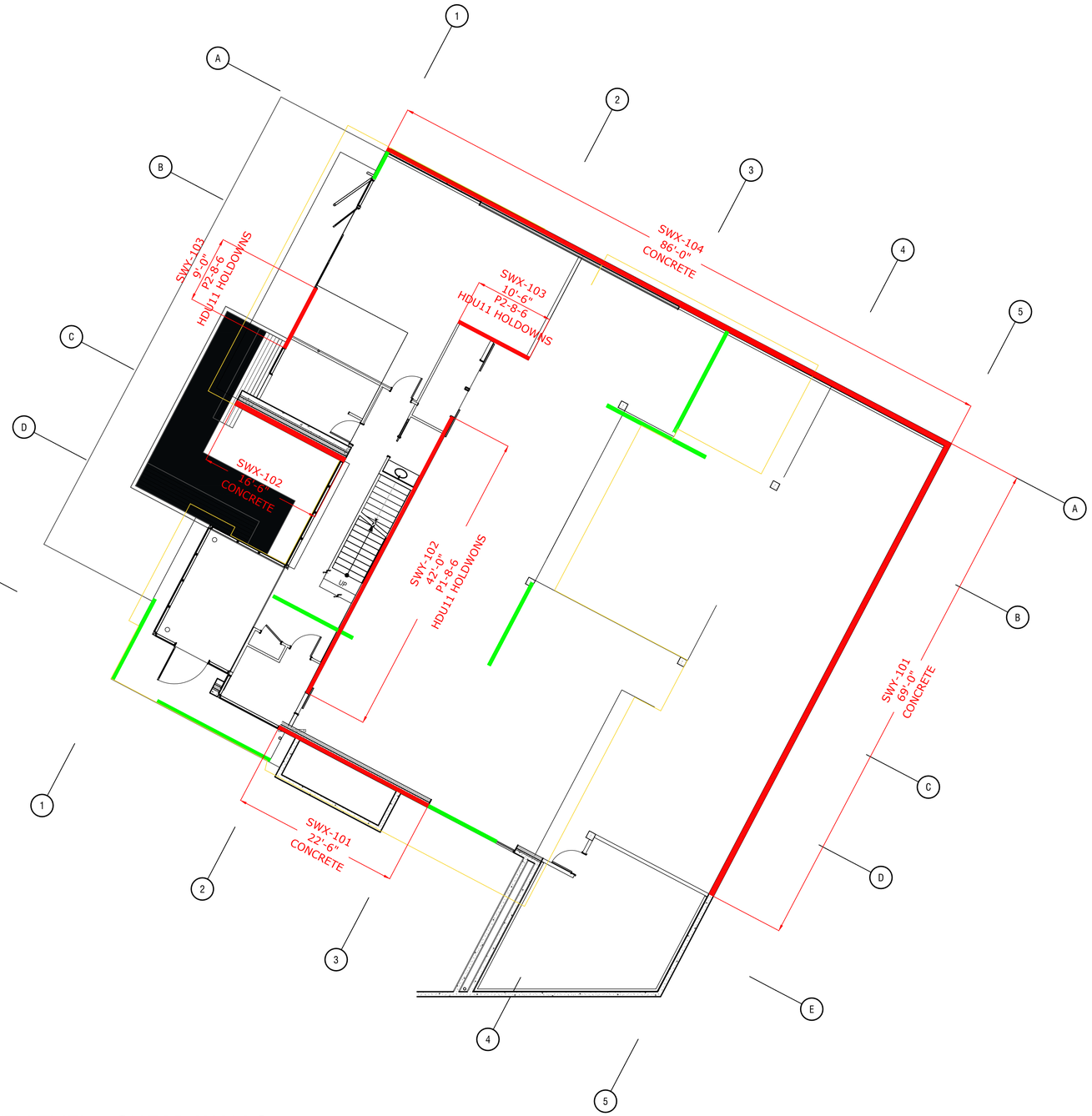
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12527 Huckleberry Lane
Arlington, Washington 98223
PHONE: (206) 790-8039
E-MAIL: joe@luciaeng.com

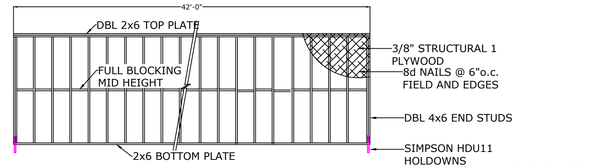
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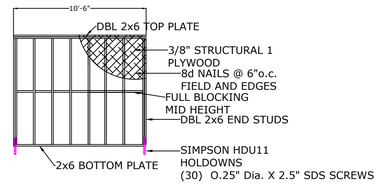
Number	Date	By	Description
5	08-15-24	JML	

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S-8.0

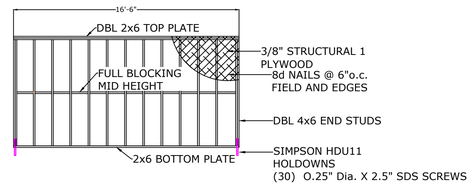




SWY-102
P1-8-6



SWX-103
P1-8-6



SWX-102
P1-8-6

GARAGE-BASEMENT LEVEL - SHEAR WALLS

LANZ RESIDENCE
8020 SE 57th Street
Mercer Island, WA 98040

Garage Level
Shear Walls

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08-15-24

Number	Date	By	Description
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FOUNDATION & RETAINING WALL DESIGN

Geotechnical Design Parameters

Foundation & Retaining Wall Designs

Foundation Plans

In our opinion, and consistent with the depiction on the referenced liquefaction susceptibility map, site susceptibility to liquefaction may be considered very low. The absence of a uniformly established shallow groundwater table and the relatively dense, fine-grained characteristics of the native soil were the primary bases for this opinion.

Retaining Walls

New retaining walls must be designed to resist earth pressures and applicable surcharge loads. The following parameters may be used for retaining wall design:

- Active earth pressure (unrestrained condition) — 42 pcf
- At-rest earth pressure (restrained condition) — 62 pcf
- Traffic surcharge (passenger vehicles) — 70 psf (rectangular distribution)
- Passive earth pressure — 200 pcf (level surface for at least 10 feet)
- Coefficient of friction — 0.40
- Seismic surcharge — 8H psf*

* *Where H equals the retained height (in feet).*

The passive earth pressure and coefficient of friction values include a safety factor of 1.5. Additional surcharge loading from adjacent foundations, sloped backfill, or other loads should be included in the retaining wall design.

Retaining walls should be backfilled with free-draining material that extends along the height of the wall and a distance of at least 18 inches behind the wall. The upper 12 inches of the wall backfill may consist of a less permeable soil, if desired.

Drainage should be provided behind retaining walls such that hydrostatic pressures do not develop. If drainage is not provided, hydrostatic pressures should be included in the wall design. A perforated drainpipe should be placed along the base of the wall and connected to an approved discharge location. A typical retaining wall drainage detail is provided on Plate 3.

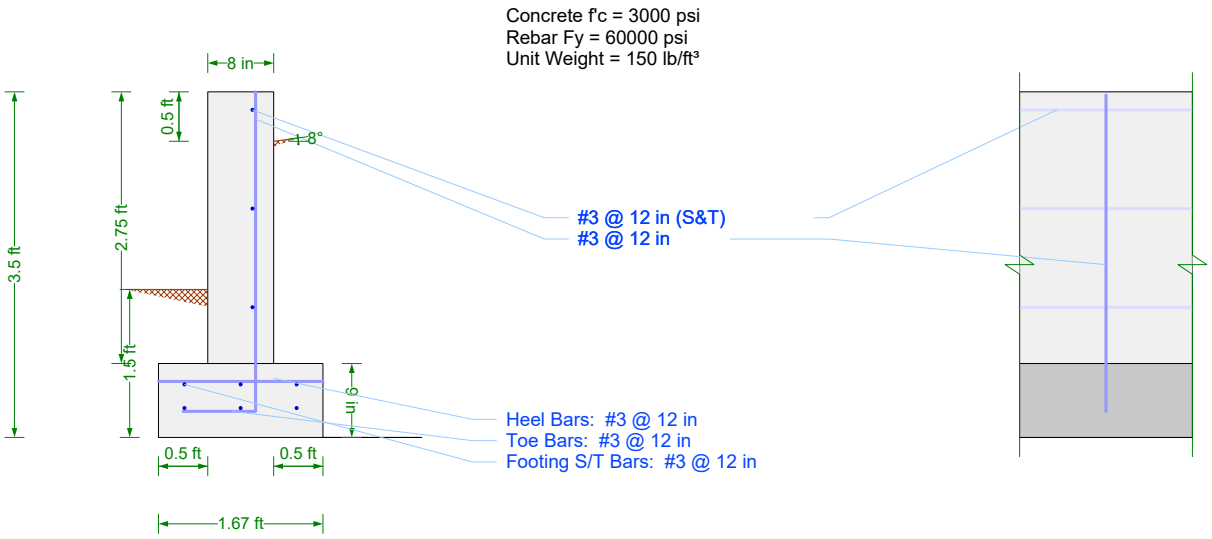
Drainage

Groundwater seepage will likely be encountered within site excavations, particularly during the wet season. Temporary measures to control surface water runoff and groundwater during construction would likely involve passive elements such as interceptor trenches, interceptor swales, and sumps. ESNW should be consulted during preliminary grading to identify areas of seepage and provide recommendations to reduce the potential for seepage-related instability.

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STANDARD FOOTING _STEM WALL

Design Detail



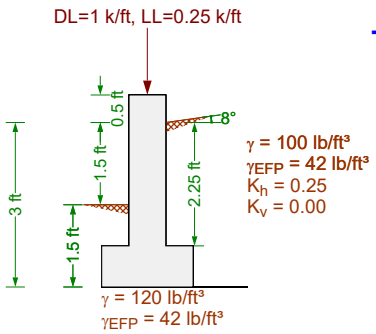
Check Summary

Ratio	Check	Provided	Required	Combination
----- Stability Checks -----				
✓ 0.317	Overturning	3.79	1.20	1.0D + 1.0L + 1.0H + 0.7E
✓ 0.554	Sliding	2.17	1.20	1.0D + 1.0H + 0.7E
✓ 0.763	Bearing Pressure	2500 psf	1909 psf	1.0D + 1.0L + 1.0H + 0.7E
✓ 0.220	Bearing Eccentricity	2.2 in	10 in	1.0D + 1.0L + 1.0H + 0.7E
----- Toe Checks -----				
✓ 0.006	Shear	5.73 k/ft	0.03 k/ft	1.2D + 1.6L + 1.6H
✓ 0.076	Moment	2.82 ft·k/ft	0.21 ft·k/ft	1.2D + 0.5L + 1.6H + 1.0E
✓ 0.061	Min Strain	0.0657	0.0040	1.2D + 1.6L + 1.6H
✓ 0.000	Min Steel	0.01 in²	0 in²	1.2D + 1.6L + 1.6H
✓ 1.000	Development	6 in	6 in	1.2D + 1.6L + 1.6H
✓ 0.667	S&T Max Spacing	12 in	18 in	1.2D + 1.6L + 1.6H
✓ 0.884	S&T Min Rho	0.0020	0.0018	1.2D + 1.6L + 1.6H
----- Heel Checks -----				
✓ 0.036	Shear	6.72 k/ft	0.24 k/ft	1.4D
✓ 0.016	Moment	3.32 ft·k/ft	0.05 ft·k/ft	1.2D + 1.6L + 1.6H
✓ 0.052	Min Strain	0.0775	0.0040	1.2D + 1.6L + 1.6H
✓ 0.000	Min Steel	0.01 in²	0 in²	1.2D + 1.6L + 1.6H
✓ 1.000	Development	12 in	12 in	1.2D + 1.6L + 1.6H
✓ 0.667	S&T Max Spacing	12 in	18 in	1.2D + 1.6L + 1.6H
✓ 0.884	S&T Min Rho	0.0020	0.0018	1.2D + 1.6L + 1.6H
----- Stem Checks -----				
✓ 0.079	Moment	2.82 ft·k/ft	0.22 ft·k/ft	1.2D + 0.5L + 1.6H + 1.0E
✓ 0.042	Shear	5.73 k/ft	0.24 k/ft	1.2D + 0.5L + 1.6H + 1.0E
✓ 0.061	Max Steel	0.0657	0.0040	1.2D + 1.6L + 1.6H
✓ 0.000	Min Steel	0.01 in²/in	0 in²/in	1.2D + 1.6L + 1.6H
✓ 1.000	Base Development	6 in	6 in	1.2D + 1.6L + 1.6H
✓ 0.000	Horz Bar Rho	0.0000	0.0000	1.2D + 1.6L + 1.6H
✓ 0.667	Horz Bar Spacing	12 in	18 in	1.2D + 1.6L + 1.6H

Criteria

Use basic criteria from common proje...	Yes
Building Code	IBC 2021
Concrete Load Combs	IBC 2021 (Strength)
Masonry Load Combs	ASCE 7-16 (ASD)
Stability Load Combs	IBC Retaining Wall St...
Apply Sds Factor to Seismic Combin...	No
Restrained Against Sliding	No
Neglect Bearing At Heel	Yes
Use Vert. Comp. for OT	No
Use Vert. Comp. for Sliding	No
Use Vert. Comp. for Bearing	Yes
Use Surcharge for Sliding & OT	Yes
Use Surcharge for Bearing	Yes
Neglect Soil Over Toe	No
Neglect Backfill Wt. for Coulomb	No
Factor Soil Weight As Dead	Yes
Use Passive Force for OT	Yes
Assume Pressure To Top	Yes
Extend Backfill Pressure To Key Bott...	No
Use Toe Passive Pressure for Bearing	No
Required F.S. for OT	1.50
Required F.S. for Sliding	1.50
Has Different Safety Factors for Seis...	Yes
Seismic F.S. for OT	1.20
Seismic F.S. for Sliding	1.20
Allowable Bearing Pressure	2500 psf
Req'd Bearing Location	Over footing
Wall Friction Angle	25°
Friction Coefficent	0.35
Soil Reaction Modulus	172800 lb/ft³

Loads



Loading Options/Assumptions

➔ Passive pressure neglects top 0 ft of soil.

Load Combinations

IBC 2018 (Strength)

1.2D + 1.6L + 1.6H
1.2D + 1.6L + 0.9H
1.2D + 0.5L + 1.6H + 1.0E
1.2D + 0.5L + 1.6H
1.2D + 0.5L + 0.9H + 1.0E
1.2D + 0.5L + 0.9H
1.2D + 1.6H + 1.0E
1.2D + 1.6H
1.2D + 0.9H + 1.0E
1.2D + 0.9H
0.9D + 1.6H + 1.0E
0.9D + 1.6H
0.9D + 0.9H + 1.0E
0.9D + 0.9H
1.4D

Strength Check Results Summary

Load Combination	Stem M-applied (ft·k/ft)	Stem M-allow (ft·k/ft)	Stem V-applied (k/ft)	Stem V-allow (k/ft)	Stem Min. ld (in)	Stem Actual ld (in)	Stem Min. strain	Stem Actual strain	Stem Min. steel (in²/in)
1.2D + 1.6L + 1.6H	0.13	2.82	0.17	5.73	6	6	0.0040	0.0657	0
1.2D + 1.6L + 0.9H	0.07	2.82	0.1	5.73	6	6	0.0040	0.0657	0
1.2D + 0.5L + 1.6H + 1.0E	0.22	2.82	0.24	5.73	6	6	0.0040	0.0657	0
1.2D + 0.5L + 1.6H	0.13	2.82	0.17	5.73	6	6	0.0040	0.0657	0
1.2D + 0.5L + 0.9H + 1.0E	0.17	2.82	0.17	5.73	6	6	0.0040	0.0657	0
1.2D + 0.5L + 0.9H	0.07	2.82	0.1	5.73	6	6	0.0040	0.0657	0
1.2D + 1.6H + 1.0E	0.22	2.82	0.24	5.73	6	6	0.0040	0.0657	0
1.2D + 1.6H	0.13	2.82	0.17	5.73	6	6	0.0040	0.0657	0
1.2D + 0.9H + 1.0E	0.17	2.82	0.17	5.73	6	6	0.0040	0.0657	0
1.2D + 0.9H	0.07	2.82	0.1	5.73	6	6	0.0040	0.0657	0
0.9D + 1.6H + 1.0E	0.22	2.82	0.24	5.73	6	6	0.0040	0.0657	0
0.9D + 1.6H	0.13	2.82	0.17	5.73	6	6	0.0040	0.0657	0
0.9D + 0.9H + 1.0E	0.17	2.82	0.17	5.73	6	6	0.0040	0.0657	0
0.9D + 0.9H	0.07	2.82	0.1	5.73	6	6	0.0040	0.0657	0
1.4D	0	0	0	0	6	6	0.0040	0.0657	0

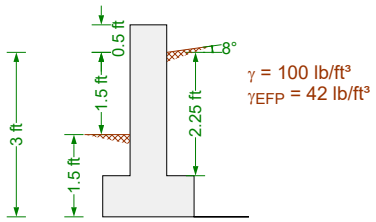
Load Combination	Stem Actual steel (in²/in)	Heel M-applied (ft·k/ft)	Heel M-allow (ft·k/ft)	Heel V-applied (k/ft)	Heel V-allow (k/ft)	Toe M-applied (ft·k/ft)	Toe M-allow (ft·k/ft)	Toe V-applied (k/ft)	Toe V-allow (k/ft)
1.2D + 1.6L + 1.6H	0.01	0.05	3.32	0.21	6.72	0.24	2.82	0.03	5.73
1.2D + 1.6L + 0.9H	0.01	0.05	3.32	0.21	6.72	0.24	2.82	0.03	5.73
1.2D + 0.5L + 1.6H + 1.0E	0.01	0.05	3.32	0.21	6.72	0.21	2.82	0.03	5.73
1.2D + 0.5L + 1.6H	0.01	0.05	3.32	0.21	6.72	0.21	2.82	0.03	5.73
1.2D + 0.5L + 0.9H + 1.0E	0.01	0.05	3.32	0.21	6.72	0.21	2.82	0.03	5.73
1.2D + 0.5L + 0.9H	0.01	0.05	3.32	0.21	6.72	0.21	2.82	0.03	5.73
1.2D + 1.6H + 1.0E	0.01	0.05	3.32	0.21	6.72	0.2	2.82	0.03	5.73
1.2D + 1.6H	0.01	0.05	3.32	0.21	6.72	0.19	2.82	0.03	5.73
1.2D + 0.9H + 1.0E	0.01	0.05	3.32	0.21	6.72	0.2	2.82	0.03	5.73
1.2D + 0.9H	0.01	0.05	3.32	0.21	6.72	0.19	2.82	0.03	5.73
0.9D + 1.6H + 1.0E	0.01	0.04	3.32	0.15	6.72	0.15	2.82	0.02	5.73
0.9D + 1.6H	0.01	0.04	3.32	0.15	6.72	0.14	2.82	0.02	5.73
0.9D + 0.9H + 1.0E	0.01	0.04	3.32	0.15	6.72	0.15	2.82	0.02	5.73
0.9D + 0.9H	0.01	0.04	3.32	0.15	6.72	0.14	2.82	0.02	5.73
1.4D	0.01	0.06	3.32	0.24	6.72	0.22	2.82	0.03	5.73

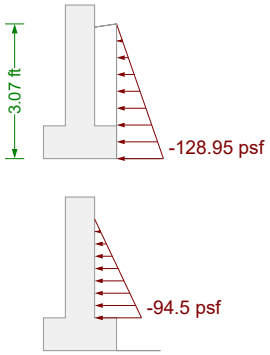
Stability Check Results Summary

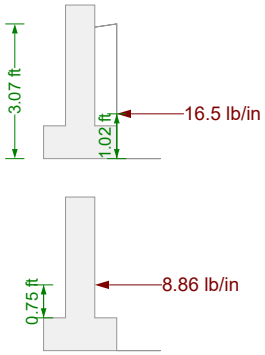
Load Combination	Overturning Moment (ft·k/ft)	Resisting Moment (ft·k/ft)	Overturning F.S.	Overturning F.S. Req'd	Overturning F.S. Req'd Seismic	Sliding Force (lb/in)	Resisting Force (lb/in)	Sliding F.S.
1.0D + 1.0L + 1.0H + 0.7E	0.2	1.42	6.988	1.500	1.200	16.5	59.79	3.625
1.0D + 1.0L + 1.0H	0.2	1.42	6.988	1.500	1.200	16.5	58.53	3.548
1.0D + 1.0H + 0.7E	0.2	1.42	6.988	1.500	1.200	16.5	52.5	3.183
1.0D + 1.0H	0.2	1.42	6.988	1.500	1.200	16.5	51.24	3.106

Load Combination	Sliding F.S. Req'd	Sliding F.S. Req'd Seismic	Bearing Pressure Actual (psf)	Bearing Pressure Allowable (psf)	Bearing Eccentricity Actual (in)	Bearing Eccentricity Allowable (in)	Wall Top Actual Deflection (in)
1.0D + 1.0L + 1.0H + 0.7E	1.500	1.200	1909	2500	2.2	10	0.18
1.0D + 1.0L + 1.0H	1.500	1.200	1865	2500	2.2	10	0.18
1.0D + 1.0H + 0.7E	1.500	1.200	1659	2500	2.2	10	0.18
1.0D + 1.0H	1.500	1.200	1616	2500	2.2	10	0.18

Backfill Pressure







Lateral Earth Pressure

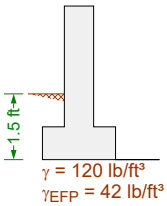
Equivalent Fluid Pressure

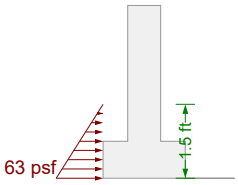
$\sigma_h = H\,\gamma_{fluid}$ = (3.07 ft)(42 lb / ft³) = 129 psf

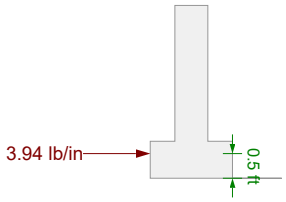
Lateral Earth Pressure (stem only)

$\sigma_h = H\,\gamma_{fluid}$ = (2.25 ft)(42 lb / ft³) = 94.5 psf

Passive Pressure





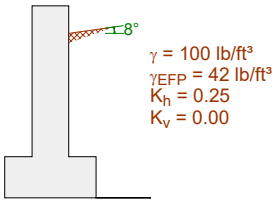


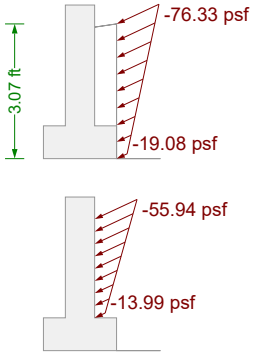
Lateral Earth Pressure

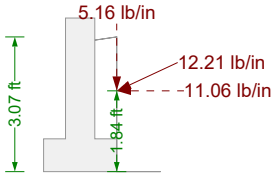
Equivalent Fluid Pressure

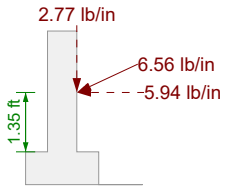
$\sigma_h = H\,\gamma_{fluid} = (1.5\text{ ft})(42\text{ lb / ft}^3) = 63\text{ psf}$

Seismic Pressure









Seismic Pressure

Dynamic + static force (Mononobe - Okabe equation)

$$\theta' = \text{atan}\left(\frac{k_h}{1 - k_v}\right) = \arctan\left[\frac{(0.250)}{1 - (0.0)}\right] = 14.04°$$

$$K_{ae} = \frac{\sin^2(\beta + \phi - \theta')}{\cos(\theta')\sin^2(\beta)\sin(\beta - \theta' - \delta)\left[1 + \sqrt{\frac{\sin(\phi + \delta)\sin(\phi - \theta' - \alpha)}{\sin(\beta - \delta - \theta')\sin(\alpha + \beta)}}\right]^{\wedge} 2}$$

$$= \frac{\sin^2[(90°) + (30°) - (14.04°)]}{\cos((14.04°))\sin^2((90°))\sin[(90°) - (14.04°) - (25°)]\left[1 + \sqrt{\frac{\sin[(30°) + (25°)]\sin[(30°) - (14.04°) - (8°)]}{\sin[(90°) - (25°) - (14.04°)]\sin[(8°) + (90°)]}}\right]^{\wedge} 2}$$

$$= 0.6403$$

$$P_{ae} = \frac{1}{2} K_{ae}\,\gamma\, H^{\wedge} 2\,(1 - k_v) = \frac{1}{2}(0.6403)(100\text{ lb / ft}^3)(3.07\text{ ft})^{\wedge} 2\,[1 - (0.0)] = 25.15\text{ lb / in}$$

Static - only force (Coulomb equation)

$$K_a = \frac{\sin^2(\beta + \phi)}{\sin^2(\beta)\sin(\beta - \delta)\left[1 + \sqrt{\frac{\sin(\phi + \delta)\sin(\phi - \alpha)}{\sin(\beta - \delta)\sin(\alpha + \beta)}}\right]^{\wedge} 2}$$

$$= \frac{\sin^2[(90°) + (30°)]}{\sin^2((90°))\sin[(90°) - (25°)]\left[1 + \sqrt{\frac{\sin[(30°) + (25°)]\sin[(30°) - (8°)]}{\sin[(90°) - (25°)]\sin[(8°) + (90°)]}}\right]^{\wedge} 2}$$

$$= 0.3295$$

$$P_a = \frac{1}{2} K_a\,\gamma\, H^{\wedge} 2 = \frac{1}{2}(0.3295)(100\text{ lb / ft}^3)(3.07\text{ ft})^{\wedge} 2 = 12.94\text{ lb / in}$$

Net dynamic force

$$\Delta P_{ae} = P_{ae} - P_a = (25.15\text{ lb / in}) - (12.94\text{ lb / in}) = 12.21\text{ lb / in}$$

$$\alpha_P = 90° - \beta + \delta = 90° - (90°) + (25°) = 25°$$ (resultant force angle with horizontal)

To arrive at the pressure distribution illustrated above (used to determine stem moments), apply inverted triangular pressure plus a uniform portion to bring resultant to 0.6H

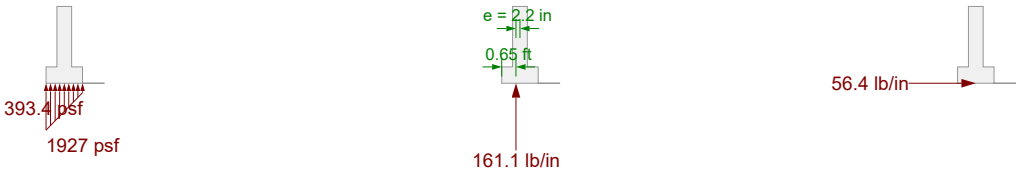
$$\sigma_{e_top} = \frac{8}{5}\frac{\Delta P_{ae}}{H} = \frac{8}{5}\frac{(12.21\text{ lb / in})}{(3.07\text{ ft})} = 76.33\text{ psf}$$

$$\sigma_{e_bot} = \frac{2}{5}\frac{\Delta P_{ae}}{H} = \frac{2}{5}\frac{(12.21\text{ lb / in})}{(3.07\text{ ft})} = 19.08\text{ psf}$$

Wall/Soil Weights



Bearing Pressure



Friction

$F = \mu R = (0.350)(161.1 \text{ lb / in}) = 56.4 \text{ lb / in}$

Bearing Pressure Calculation

Contributing Forces

	Vert Force	...offset	Horz Force	...offset	OT Moment
Backfill Pressure	-0 lb/in	-	-16.5 lb/in	1.02 ft	2431 in·lb/ft
Axial Dead Load	-83.33 lb/in	0.83 ft	0 lb/in	-	-10000 in·lb/ft
Axial Live Load	-20.83 lb/in	0.83 ft	0 lb/in	-	-2500 in·lb/ft
Seismic Force	-5.16 lb/in	1.67 ft	-11.06 lb/in	1.84 ft	1697 in·lb/ft
Footing Weight	-15.63 lb/in	0.83 ft	0 lb/in	-	-1875 in·lb/ft
Stem Weight	-22.92 lb/in	0.83 ft	0 lb/in	-	-2750 in·lb/ft
Backfill Weight	-9.38 lb/in	1.42 ft	0 lb/in	-	-1912.5 in·lb/ft
Backfill Weight	-0.15 lb/in	1.5 ft	0 lb/in	-	-31.62 in·lb/ft
Soil over toe Weight	-3.75 lb/in	0.25 ft	0 lb/in	-	-135 in·lb/ft
	-161.14 lb/in				-15076.4 in·lb/ft

$$\frac{-15076.4 \text{ in·lb / ft}}{-161.14 \text{ lb / in}} = 0.65 \text{ ft}$$

Stability Checks [1.0D + 1.0L + 1.0H + 0.7E]

Overturning Check

Overturning Moments

	Force	Distance	Moment
Backfill pressure (horz)	16.5 lb/in	1.02 ft	2431 in·lb/ft
Seismic force	7.74 lb/in	1.84 ft	2054 in·lb/ft
		Total:	4485 in·lb/ft

Resisting Moments

	Force	Distance	Moment
Passive pressure @ toe	3.94 lb/in	0.5 ft	283.5 in·lb/ft
Axial dead load	-83.33 lb/in	0.83 ft	10000 in·lb/ft
Footing Weight	-15.63 lb/in	0.83 ft	1875 in·lb/ft
Stem Weight	-22.92 lb/in	0.83 ft	2750 in·lb/ft
Backfill Weight	-9.38 lb/in	1.42 ft	1913 in·lb/ft
Backfill Weight	-0.15 lb/in	1.5 ft	31.62 in·lb/ft
Soil over toe Weight	-3.75 lb/in	0.25 ft	135 in·lb/ft
		Total:	16988 in·lb/ft

Without seismic loads:

$$F.S. = \frac{RM}{OTM} = \frac{16988\ \text{in·lb / ft}}{2431\ \text{in·lb / ft}} = 6.988 > 1.50\ (OK)$$

Including seismic loads:

$$F.S. = \frac{RM}{OTM} = \frac{16988\ \text{in·lb / ft}}{4485\ \text{in·lb / ft}} = 3.787 > 1.20\ (OK)$$

Sliding Check

Sliding Force(s)

Backfill pressure	16.5 lb/in
Seismic force	7.74 lb/in
Total:	24.24 lb/in

Resisting Force(s)

Passive pressure @ toe	3.94 lb/in
Friction	55.86 lb/in
Total:	59.79 lb/in

Without seismic loads:

$$F.S. = \frac{RF}{SF} = \frac{59.79\ \text{lb / in}}{16.5\ \text{lb / in}} = 3.625 > 1.50\ (OK)$$

Including seismic loads:

$$F.S. = \frac{RF}{SF} = \frac{59.79\ \text{lb / in}}{24.24\ \text{lb / in}} = 2.467 > 1.20\ (OK)$$

Bearing Capacity Check

Bearing pressure < allowable (1909 psf < 2500 psf) - OK
Bearing resultant eccentricity < allowable (2.2 in < 10 in) - OK

Wall Top Displacement

(based on unfactored service loads)

Deflection due to stem flexural displacement	0 in
Deflection due to rotation from settlement	0.176 in
Total deflection at top of wall (positive towards toe)	0.176 in

Stability Checks [1.0D + 1.0H + 0.7E]

Overturning Check

Overturning Moments

	Force	Distance	Moment
Backfill pressure (horz)	16.5 lb/in	1.02 ft	2431 in·lb/ft
Seismic force	7.74 lb/in	1.84 ft	2054 in·lb/ft
		Total:	4485 in·lb/ft

Resisting Moments

	Force	Distance	Moment
Passive pressure @ toe	3.94 lb/in	0.5 ft	283.5 in·lb/ft
Axial dead load	-83.33 lb/in	0.83 ft	10000 in·lb/ft
Footing Weight	-15.63 lb/in	0.83 ft	1875 in·lb/ft
Stem Weight	-22.92 lb/in	0.83 ft	2750 in·lb/ft
Backfill Weight	-9.38 lb/in	1.42 ft	1913 in·lb/ft
Backfill Weight	-0.15 lb/in	1.5 ft	31.62 in·lb/ft
Soil over toe Weight	-3.75 lb/in	0.25 ft	135 in·lb/ft
		Total:	16988 in·lb/ft

Without seismic loads:

$$F.S. = \frac{RM}{OTM} = \frac{16988 \text{ in·lb / ft}}{2431 \text{ in·lb / ft}} = 6.988 > 1.50 \text{ (OK)}$$

Including seismic loads:

$$F.S. = \frac{RM}{OTM} = \frac{16988 \text{ in·lb / ft}}{4485 \text{ in·lb / ft}} = 3.787 > 1.20 \text{ (OK)}$$

Sliding Check

Sliding Force(s)

Backfill pressure	16.5 lb/in
Seismic force	7.74 lb/in
Total:	24.24 lb/in

Resisting Force(s)

Passive pressure @ toe	3.94 lb/in
Friction	48.57 lb/in
Total:	52.5 lb/in

Without seismic loads:

$$F.S. = \frac{RF}{SF} = \frac{52.5 \text{ lb / in}}{16.5 \text{ lb / in}} = 3.183 > 1.50 \text{ (OK)}$$

Including seismic loads:

$$F.S. = \frac{RF}{SF} = \frac{52.5 \text{ lb / in}}{24.24 \text{ lb / in}} = 2.166 > 1.20 \text{ (OK)}$$

Bearing Capacity Check

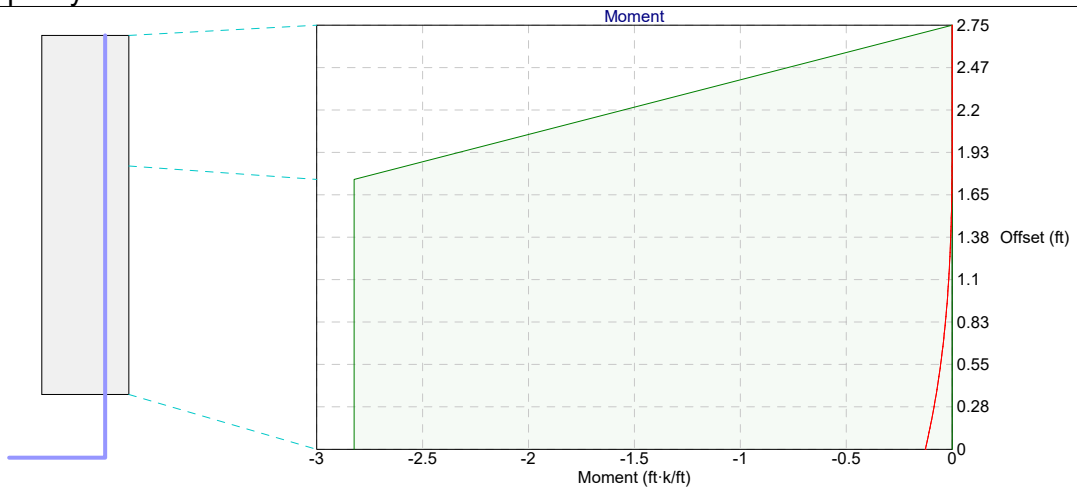
Bearing pressure < allowable (1659 psf < 2500 psf) - OK
Bearing resultant eccentricity < allowable (2.2 in < 10 in) - OK

Wall Top Displacement

(based on unfactored service loads)

Deflection due to stem flexural displacement	0 in
Deflection due to rotation from settlement	0.176 in
Total deflection at top of wall (positive towards toe)	0.176 in

Stem Flexural Capacity



Capacity (ACI 318-14 11.5.2.2, »22.3, »22.2) @ 0 ft from base

$$a = \frac{A_s f_y}{0.85 f'_c} = \frac{(0.01 \text{ in}^2 / \text{in})(60000 \text{ psi})}{0.85 (3000 \text{ psi})} = 0.22 \text{ in}$$

$$\phi M_n = \phi A_s f_y (d - a / 2) = (0.90)(0.01 \text{ in}^2 / \text{in})(60000 \text{ psi})[(5.81 \text{ in}) - (0.22 \text{ in}) / 2] = 2.82 \text{ ft·k / ft}$$

Capacity (ACI 318-14 11.5.2.2, »22.3, »22.2) @ 1.75 ft from base

$$a = \frac{A_s f_y}{0.85 f'_c} = \frac{(0.01 \text{ in}^2 / \text{in})(60000 \text{ psi})}{0.85 (3000 \text{ psi})} = 0.22 \text{ in}$$

$$\phi M_n = \phi A_s f_y (d - a / 2) = (0.90)(0.01 \text{ in}^2 / \text{in})(60000 \text{ psi})[(5.81 \text{ in}) - (0.22 \text{ in}) / 2] = 2.82 \text{ ft·k / ft}$$

Capacity (ACI 318-14 11.5.2.2, »22.3, »22.2) @ 2.75 ft from base

$$a = \frac{A_s f_y}{0.85 f'_c} = \frac{(0 \text{ in}^2 / \text{in})(60000 \text{ psi})}{0.85 (3000 \text{ psi})} = 0 \text{ in}$$

$$\phi M_n = \phi A_s f_y (d - a / 2) = (0.90)(0 \text{ in}^2 / \text{in})(60000 \text{ psi})[(5.81 \text{ in}) - (0 \text{ in}) / 2] = 0 \text{ ft·k / ft}$$

Stem Shear Capacity



Shear Capacity (ACI 318-14 11.5.5.1, 22.5.1.1, 22.5.5.1) @ 0 ft from base

λ = 1.0 (normal weight concrete)

$V_c = 2 \lambda \sqrt{F'_c}\, d$ = 2 (1.0) $\sqrt{3000 \text{ psi}}$ (5.81 in) = 7.64 k / ft

$\phi V_n = \phi V_c$ = (0.750) (7.64 k / ft) = 5.73 k / ft

Shear Capacity (ACI 318-14 11.5.5.1, 22.5.1.1, 22.5.5.1) @ 2.75 ft from base

λ = 1.0 (normal weight concrete)

$V_c = 2 \lambda \sqrt{F'_c}\, d$ = 2 (1.0) $\sqrt{3000 \text{ psi}}$ (5.81 in) = 7.64 k / ft

$\phi V_n = \phi V_c$ = (0.750) (7.64 k / ft) = 5.73 k / ft

Stem Development/Lap Length Calculations

Main vertical stem bars (bottom end) - Development Length Calculation (ACI 318-14 11.7.1.2, 25.4.2.3, 25.4.3)

ψ_e = 1.0 (uncoated hooked bars)

ψ_c = 0.70 (based on side cover and extension cover)

ψ_r = 1.0 (no confining reinforcement)

λ = 1.0 (normal weight concrete)

$$l_{dh} = \left(\frac{f_y \, \psi_e \, \psi_c \, \psi_r}{50 \, \lambda \sqrt{F'_c}} \right) d_b = \left[\frac{(60000 \text{ psi}) (1.0) (0.70) (1.0)}{50 (1.0) \sqrt{3000 \text{ psi}}} \right] (0.38 \text{ in}) = 5.75 \text{ in}$$

$8 \, d_b = 8 \, (0.38 \text{ in}) = 3.0$ (minimum limit, does not control)

6 inch minimum controls

Main vertical stem bars (top end) - Development Length Calculation (ACI 318-14 11.7.1.2, 25.4.2.3, 25.4.3)

ψ_t = 1.0 (bars are not horizontal)

ψ_e = 1.0 (bar not epoxy coated)

ψ_s = 0.80 (bars are #6 or smaller)

λ = 1.0 (normal weight concrete)

$s / 2 = (12 \text{ in}) / 2 = 6 \text{ in}$

$\text{cover} + d_b / 2 = (2 \text{ in}) + (0.38 \text{ in}) / 2 = 2.19 \text{ in}$

c_b = 2.19 in (lesser of half spacing, ctr to surface)

K_{tr} = 0.0 (no transverse reinforcement)

$$\frac{c_b + K_{tr}}{d_b} = \frac{(2.19 \text{ in}) + (0.0)}{(0.38 \text{ in})} = 5.8333$$

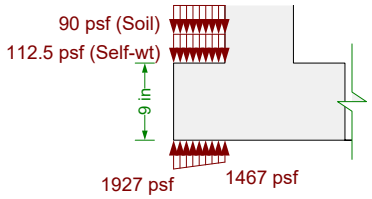
$$l_d = \left(\frac{3.}{40} \frac{f_y}{\lambda \sqrt{F'_c}} \frac{\psi_t \, \psi_e \, \psi_s}{2.5} \right) d_b = \left[\frac{3.}{40} \frac{(60000 \text{ psi})}{(1.0) \sqrt{3000 \text{ psi}}} \frac{(1.0) (1.0) (0.80)}{2.5} \right] (0.38 \text{ in}) = 9.86 \text{ in}$$

12 inch minimum controls

Toe Checks [1.2D + 1.6L + 1.6H]

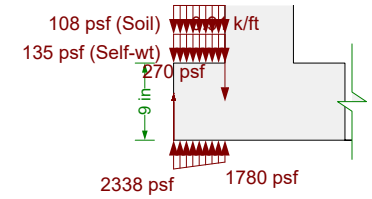
Toe Unfactored Loads

Unfactored Loads



Toe Factored Loads

1.2D + 1.6L + 1.6H



Controlling Moment

Design moment M_u for toe need not exceed moment at stem base:

M_{toe} = 0.24 ft·k / ft ≥ M_{stem} = 0.13 ft·k / ft

M_u = 0.13 ft·k / ft (stem base moment controls)

Flexure Check (ACI 318-14 13.3.2.1, 7.5.2.1, »22.3, »22.2, 7.5.1.1a)

$$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0.01 \text{ in}^2 / \text{in})(60000 \text{ psi})}{0.85 (3000 \text{ psi})} = 0.22 \text{ in}$$

$$\phi M_n = \phi A_s f_y (d - a / 2) = (0.90)(0.01 \text{ in}^2 / \text{in})(60000 \text{ psi})[(5.81 \text{ in}) - (0.22 \text{ in}) / 2] = 2.82 \text{ ft·k / ft}$$

ϕM_n = 2.82 ft·k / ft ≥ M_u = 0.13 ft·k / ft ✓

Shear Check (ACI 318-14 13.3.2.1, 7.5.3.1, »22.5.1, »22.5.5, 7.5.1.1b)

λ = 1.0 (normal weight concrete)

$$V_c = 2 \lambda \sqrt{F'_c} d = 2 (1.0) \sqrt{3000 \text{ psi}} (5.81 \text{ in}) = 7.64 \text{ k / ft}$$

$$\phi V_n = \phi V_c = (0.750)(7.64 \text{ k / ft}) = 5.73 \text{ k / ft}$$

ϕV_n = 5.73 k / ft ≥ V_u = 0.03 k / ft ✓

Minimum Strain Check (ACI 318-14 13.3.2.1, 7.3.3.1)

β_1 = 0.850 ($F'_c \le 4000$ psi)

$$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0.01 \text{ in}^2 / \text{in})(60000 \text{ psi})}{0.85 (3000 \text{ psi})} = 0.22 \text{ in}$$

$$\varepsilon_t = 0.003 \left(\frac{d}{a / \beta_1} - 1 \right) = 0.003 \left[\frac{(5.81 \text{ in})}{(0.22 \text{ in}) / (0.850)} - 1 \right] = 0.0657$$

ε_t = 0.0657 ≥ 0.004 ✓

Minimum Steel Check (ACI 318-14 13.3.2.1, 9.6.1)

ϕM_n = 2.82 ft·k / ft ≥ (4 / 3) M_u = [4 / 3] (0.13 ft·k / ft) = 0.17 ft·k / ft

Check is waived per ACI 9.6.1.3 ✓

Shrinkage and Temperature Steel (ACI 318-14 13.2.8.1, 7.6.4.1, 24.4.3.2, 24.4.3.3)

$$\rho_{ST_prov} = \frac{A_{ST}}{t\, s_{ST}} = \frac{(0.22 \text{ in}^2 / \text{in})}{(9 \text{ in})(12 \text{ in})} = 0.0020$$

$$\rho_{ST_prov} = \frac{A_{ST}}{t\, s_{ST}} = \frac{(0.22 \text{ in}^2 / \text{in})}{(9 \text{ in})(12 \text{ in})} = 0.0020$$

$$\frac{0.0018 (60000)}{f_y} = \frac{0.0018 (60000)}{(60000 \text{ psi})} = 0.0018$$

ρ_{ST_min} = 0.0018

ρ_{ST_prov} = 0.0020 ≥ ρ_{ST_min} = 0.0018 ✓

18 inch limit governs

s_{ST_max} = 18 in

s_{ST} = 12 in ≤ s_{ST_max} = 18 in ✓

Development Check (ACI 318-14 13.2.8.1, 25.4.2.3, 25.4.10)

$$\frac{M_u}{\phi M_n} = \frac{(0.13 \text{ ft·k / ft})}{(2.82 \text{ ft·k / ft})} = 0.0452$$ (ratio to represent excess reinforcement)

ψ_e = 1.0 (uncoated hooked bars)

ψ_c = 0.70 (based on side cover and extension cover)

ψ_r = 1.0 (no confining reinforcement)

λ = 1.0 (normal weight concrete)

$$l_{dh} = \left(\frac{f_y \psi_e \psi_c \psi_r}{50 \lambda \sqrt{F'_c}} \right) d_b = \left[\frac{(60000 \text{ psi})(1.0)(0.70)(1.0)}{50 (1.0) \sqrt{3000 \text{ psi}}} \right] (0.38 \text{ in}) = 5.75 \text{ in}$$

Factoring l_{dh} by the excess reinforcement ratio (0.0452) per 25.4.10: l_{dh} = 0.26 in

8 d_b = 8 (0.38 in) = 3.0 (minimum limit, does not control)

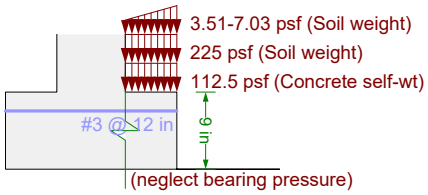
6 inch minimum controls

l_{dh_prov} = 6 in ≥ l_{dh} = 6 in ✓

Heel Checks [1.2D + 1.6L + 1.6H]

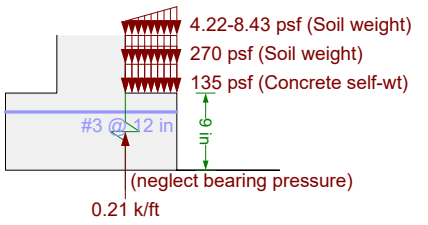
Heel Unfactored Loads

Unfactored Loads



Heel Factored Loads

1.2D + 1.6L + 1.6H



Controlling Moment

Design moment M_u for heel need not exceed moment at stem base:

M_{heel} = 0.05 ft·k / ft < M_{stem} = 0.13 ft·k / ft

M_u = 0.05 ft·k / ft (stem moment does not control)

Flexure Check (ACI 318-14 13.3.2.1, 7.5.2.1, »22.3, »22.2, 7.5.1.1a)

$$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0.01 \text{ in}^2 / \text{in})(60000 \text{ psi})}{0.85 (3000 \text{ psi})} = 0.22 \text{ in}$$

$\phi M_n = \phi A_s f_y (d - a / 2)$ = (0.90)(0.01 in² / in)(60000 psi)[(6.81 in) - (0.22 in) / 2] = 3.32 ft·k / ft

ϕM_n = 3.32 ft·k / ft ≥ M_u = 0.05 ft·k / ft ✓

Shear Check (ACI 318-14 13.3.2.1, 7.5.3.1, »22.5.1, »22.5.5, 7.5.1.1b)

λ = 1.0 (normal weight concrete)

$V_c = 2 \lambda \sqrt{F'_c}\, d$ = 2 (1.0) $\sqrt{3000 \text{ psi}}$ (6.81 in) = 8.96 k / ft

$\phi V_n = \phi V_c$ = (0.750)(8.96 k / ft) = 6.72 k / ft

ϕV_n = 6.72 k / ft ≥ V_u = 0.21 k / ft ✓

Minimum Strain Check (ACI 318-14 13.3.2.1, 7.3.3.1)

β_1 = 0.850 ($F'_c \le 4000$ psi)

$$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0.01 \text{ in}^2 / \text{in})(60000 \text{ psi})}{0.85 (3000 \text{ psi})} = 0.22 \text{ in}$$

$$\varepsilon_t = 0.003 \left(\frac{d}{a / \beta_1} - 1 \right) = 0.003 \left[\frac{(6.81 \text{ in})}{(0.22 \text{ in}) / (0.850)} - 1 \right] = 0.0775$$

ε_t = 0.0775 ≥ 0.004 ✓

Minimum Steel Check (ACI 318-14 13.3.2.1, 9.6.1)

ϕM_n = 3.32 ft·k / ft ≥ (4 / 3) M_u = [4 / 3](0.05 ft·k / ft) = 0.07 ft·k / ft

Check is waived per ACI 9.6.1.3 ✓

Shrinkage and Temperature Steel (ACI 318-14 13.2.8.1, 7.6.4.1, 24.4.3.2, 24.4.3.3)

$$\rho_{ST_prov} = \frac{A_{ST}}{t\, s_{ST}} = \frac{(0.22 \text{ in}^2 / \text{in})}{(9 \text{ in})(12 \text{ in})} = 0.0020$$

$$\rho_{ST_prov} = \frac{A_{ST}}{t\, s_{ST}} = \frac{(0.22 \text{ in}^2 / \text{in})}{(9 \text{ in})(12 \text{ in})} = 0.0020$$

$$\frac{0.0018 (60000)}{f_y} = \frac{0.0018 (60000)}{(60000 \text{ psi})} = 0.0018$$

ρ_{ST_min} = 0.0018

ρ_{ST_prov} = 0.0020 ≥ ρ_{ST_min} = 0.0018 ✓

18 inch limit governs

s_{ST_max} = 18 in

s_{ST} = 12 in ≤ s_{ST_max} = 18 in ✓

Development Check (ACI 318-14 13.2.8.1, 25.4.2.3, 25.4.10)

$$\frac{M_u}{\phi M_n} = \frac{(0.05 \text{ ft·k / ft})}{(3.32 \text{ ft·k / ft})} = 0.0155$$ (ratio to represent excess reinforcement)

ψ_t = 1.0 (12 inches or less cast below - 6.63 inches)

ψ_e = 1.0 (bar not epoxy coated)

ψ_s = 0.80 (bars are #6 or smaller)

λ = 1.0 (normal weight concrete)

s / 2 = (12 in) / 2 = 6 in

cover + d_b / 2 = (2 in) + (0.38 in) / 2 = 2.19 in

c_b = 2.19 in (lesser of half spacing, ctr to surface)

K_{tr} = 0.0 (no transverse reinforcement)

$$\frac{c_b + K_{tr}}{d_b} = \frac{(2.19 \text{ in}) + (0.0)}{(0.38 \text{ in})} = 5.8333$$

$$l_d = \left(\frac{3.}{40} \frac{f_y}{\lambda \sqrt{F'_c}} \frac{\psi_t \psi_e \psi_s}{2.5} \right) d_b = \left[\frac{3.}{40} \frac{(60000 \text{ psi})}{(1.0)\sqrt{3000 \text{ psi}}} \frac{(1.0)(1.0)(0.80)}{2.5} \right] (0.38 \text{ in}) = 9.86 \text{ in}$$

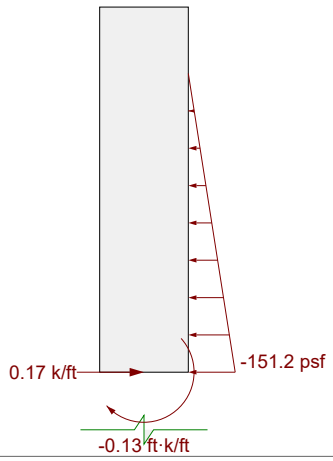
Factoring l_d by the excess reinforcement ratio (0.0155) per 25.4.10: l_d = 0.15 in

12 inch minimum controls

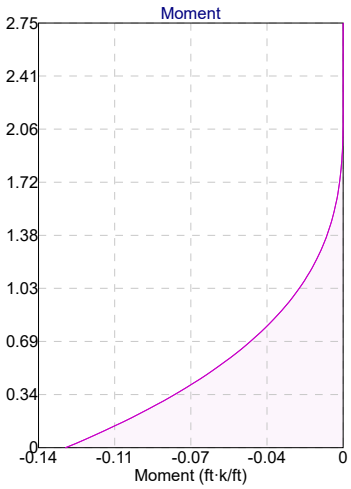
l_{d_prov} = 12 in ≥ l_d = 12 in ✓

Stem Forces [1.2D + 1.6L + 1.6H]

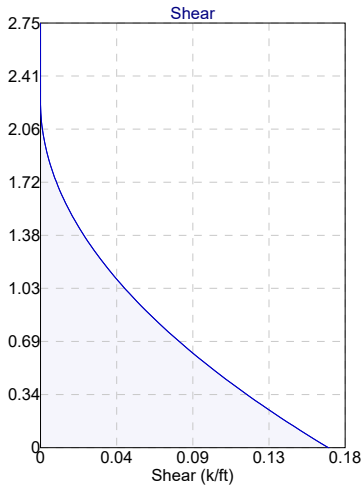
Stem Internal Forces



Stem Internal Forces



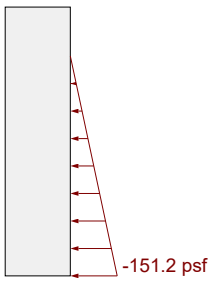
Stem Internal Forces



Stem Joint Force Transfer

Location	Force
@ stem base	0.17 k/ft

Stem Internal Forces



Stem Moment Checks [1.2D + 1.6L + 1.6H]



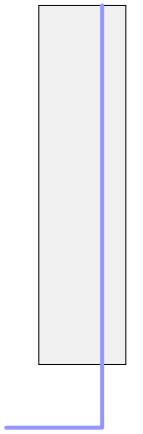
Check (ACI 318-14 11.5.5.1b) @ 0 ft from base

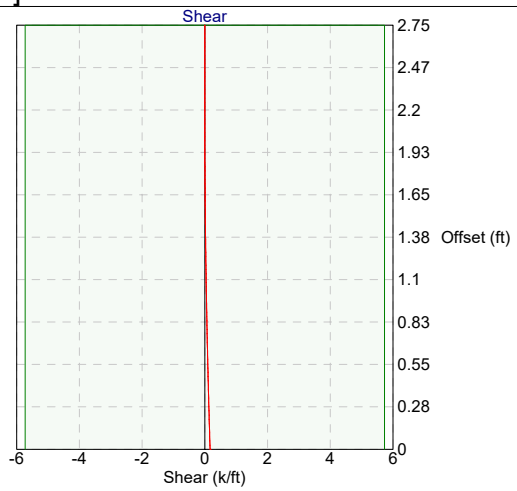
ϕM_n = 2.82 ft·k / ft ≥ M_u = 0.13 ft·k / ft ✓

Check (ACI 318-14 11.5.5.1b) @ 1.75 ft from base

ϕM_n = 2.82 ft·k / ft ≥ M_u = 0 ft·k / ft ✓

Stem Shear Checks [1.2D + 1.6L + 1.6H]





Shear Check (ACI 318-14 11.5.5.1c) @ 0 ft from base

ϕV_n = 5.73 k / ft ≥ V_u = 0.17 k / ft ✓

Stem Miscellaneous Checks [1.2D + 1.6L + 1.6H]

Minimum Steel Check (ACI 318-14 9.6.1) @ 0 ft from base [Stem in negative flexure]

ϕM_n = 2.82 ft·k / ft ≥ (4 / 3) M_u = [4 / 3] (0.13 ft·k / ft) = 0.17 ft·k / ft

Check is waived per ACI 9.6.1.3 ✓

Minimum Steel Check (ACI 318-14 9.6.1) @ 2.75 ft from base [Stem in negative flexure]

ϕM_n = 0 ft·k / ft ≥ (4 / 3) M_u = [4 / 3] (0 ft·k / ft) = 0 ft·k / ft

Check is waived per ACI 9.6.1.3 ✓

Maximum Steel Check (ACI 318-14 9.3.3.1) @ 0 ft from base [Stem in negative flexure]

β_1 = 0.850 ($F'_c \le 4000$ psi)

$$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0.01 \text{ in}^2 / \text{in})(60000 \text{ psi})}{0.85 (3000 \text{ psi})} = 0.22 \text{ in}$$

$$\varepsilon_t = 0.003 \left(\frac{d}{a / \beta_1} - 1 \right) = 0.003 \left[\frac{(5.81 \text{ in})}{(0.22 \text{ in}) / (0.850)} - 1 \right] = 0.0657$$

ε_t = 0.0657 ≥ 0.004 ✓

Maximum Steel Check (ACI 318-14 9.3.3.1) @ 2.75 ft from base [Stem in negative flexure]

β_1 = 0.850 ($F'_c \le 4000$ psi)

$$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0.01 \text{ in}^2 / \text{in})(60000 \text{ psi})}{0.85 (3000 \text{ psi})} = 0.22 \text{ in}$$

$$\varepsilon_t = 0.003 \left(\frac{d}{a / \beta_1} - 1 \right) = 0.003 \left[\frac{(5.81 \text{ in})}{(0.22 \text{ in}) / (0.850)} - 1 \right] = 0.0657$$

ε_t = 0.0657 ≥ 0.004 ✓

Wall Horizontal Steel (ACI 318-14 11.6.1, 11.7.3)

$$\rho_t = \frac{A_{s_horz} / s_{horz}}{t} = \frac{(0.11 \text{ in}^2) / (12 \text{ in})}{(8 \text{ in})} = 0.0011$$

ρ_{t_min} = 0.0020 (bars No. 5 or less, not less than 60 ksi)

ρ_t = 0.0011 < ρ_{t_min} = 0.0020 ✘

3 h = 3 (8 in) = 24 in

18 inch limit governs

s_{horz} = 12 in ≤ s_{horz_max} = 18 in ✓

Development Check (ACI 318-14 11.7.1.2, 25.4.2.3, 25.4.10)

$\frac{M_u}{\phi M_n} = \frac{(0.13 \text{ ft·k / ft})}{(2.82 \text{ ft·k / ft})} = 0.0452$ (ratio to represent excess reinforcement)

ψ_e = 1.0 (uncoated hooked bars)

ψ_c = 0.70 (based on side cover and extension cover)

ψ_r = 1.0 (no confining reinforcement)

λ = 1.0 (normal weight concrete)

$$l_{dh} = \left(\frac{f_y \psi_e \psi_c \psi_r}{50 \lambda \sqrt{F'_c}} \right) d_b = \left[\frac{(60000 \text{ psi})(1.0)(0.70)(1.0)}{50 (1.0) \sqrt{3000 \text{ psi}}} \right] (0.38 \text{ in}) = 5.75 \text{ in}$$

Factoring l_{dh} by the excess reinforcement ratio (0.0452) per 25.4.10: l_{dh} = 0.26 in

8 d_b = 8 (0.38 in) = 3.0 (minimum limit, does not control)

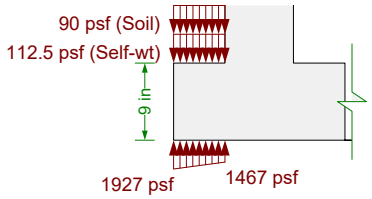
6 inch minimum controls

l_{dh_prov} = 6 in ≥ l_{dh} = 6 in ✓

Toe Checks [1.2D + 0.5L + 1.6H + 1.0E]

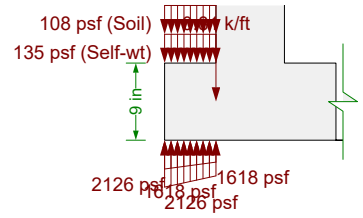
Toe Unfactored Loads

Unfactored Loads



Toe Factored Loads

1.2D + 0.5L + 1.6H + 1.0E



Controlling Moment

Design moment M_u for toe need not exceed moment at stem base:

M_{toe} = 0.21 ft·k / ft < M_{stem} = 0.22 ft·k / ft

M_u = 0.21 ft·k / ft (stem moment does not control)

Flexure Check (ACI 318-14 13.3.2.1, 7.5.2.1, »22.3, »22.2, 7.5.1.1a)

$$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0.01 \text{ in}^2 / \text{in})(60000 \text{ psi})}{0.85 (3000 \text{ psi})} = 0.22 \text{ in}$$

$$\phi M_n = \phi A_s f_y (d - a / 2) = (0.90)(0.01 \text{ in}^2 / \text{in})(60000 \text{ psi})[(5.81 \text{ in}) - (0.22 \text{ in}) / 2] = 2.82 \text{ ft·k / ft}$$

ϕM_n = 2.82 ft·k / ft ≥ M_u = 0.21 ft·k / ft ✓

Shear Check (ACI 318-14 13.3.2.1, 7.5.3.1, »22.5.1, »22.5.5, 7.5.1.1b)

λ = 1.0 (normal weight concrete)

$$V_c = 2 \lambda \sqrt{F'_c}\, d = 2 (1.0) \sqrt{3000 \text{ psi}}\, (5.81 \text{ in}) = 7.64 \text{ k / ft}$$

$$\phi V_n = \phi V_c = (0.750)(7.64 \text{ k / ft}) = 5.73 \text{ k / ft}$$

ϕV_n = 5.73 k / ft ≥ V_u = 0.03 k / ft ✓

Minimum Strain Check (ACI 318-14 13.3.2.1, 7.3.3.1)

β_1 = 0.850 ($F'_c \le 4000$ psi)

$$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0.01 \text{ in}^2 / \text{in})(60000 \text{ psi})}{0.85 (3000 \text{ psi})} = 0.22 \text{ in}$$

$$\varepsilon_t = 0.003 \left(\frac{d}{a / \beta_1} - 1 \right) = 0.003 \left[\frac{(5.81 \text{ in})}{(0.22 \text{ in}) / (0.850)} - 1 \right] = 0.0657$$

ε_t = 0.0657 ≥ 0.004 ✓

Minimum Steel Check (ACI 318-14 13.3.2.1, 9.6.1)

ϕM_n = 2.82 ft·k / ft ≥ (4 / 3) M_u = [4 / 3] (0.21 ft·k / ft) = 0.29 ft·k / ft

Check is waived per ACI 9.6.1.3 ✓

Shrinkage and Temperature Steel (ACI 318-14 13.2.8.1, 7.6.4.1, 24.4.3.2, 24.4.3.3)

$$\rho_{ST_prov} = \frac{A_{ST}}{t\, s_{ST}} = \frac{(0.22 \text{ in}^2 / \text{in})}{(9 \text{ in})(12 \text{ in})} = 0.0020$$

$$\rho_{ST_prov} = \frac{A_{ST}}{t\, s_{ST}} = \frac{(0.22 \text{ in}^2 / \text{in})}{(9 \text{ in})(12 \text{ in})} = 0.0020$$

$$\frac{0.0018 (60000)}{f_y} = \frac{0.0018 (60000)}{(60000 \text{ psi})} = 0.0018$$

ρ_{ST_min} = 0.0018

ρ_{ST_prov} = 0.0020 ≥ ρ_{ST_min} = 0.0018 ✓

18 inch limit governs

s_{ST_max} = 18 in

s_{ST} = 12 in ≤ s_{ST_max} = 18 in ✓

Development Check (ACI 318-14 13.2.8.1, 25.4.2.3, 25.4.10)

$$\frac{M_u}{\phi M_n} = \frac{(0.21 \text{ ft·k / ft})}{(2.82 \text{ ft·k / ft})} = 0.0759$$ (ratio to represent excess reinforcement)

ψ_e = 1.0 (uncoated hooked bars)

ψ_c = 0.70 (based on side cover and extension cover)

ψ_r = 1.0 (no confining reinforcement)

λ = 1.0 (normal weight concrete)

$$l_{dh} = \left(\frac{f_y \psi_e \psi_c \psi_r}{50 \lambda \sqrt{F'_c}} \right) d_b = \left[\frac{(60000 \text{ psi})(1.0)(0.70)(1.0)}{50 (1.0) \sqrt{3000 \text{ psi}}} \right] (0.38 \text{ in}) = 5.75 \text{ in}$$

Factoring l_{dh} by the excess reinforcement ratio (0.0759) per 25.4.10: l_{dh} = 0.44 in

8 d_b = 8 (0.38 in) = 3.0 (minimum limit, does not control)

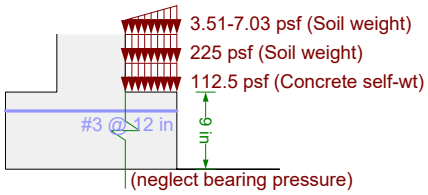
6 inch minimum controls

l_{dh_prov} = 6 in ≥ l_{dh} = 6 in ✓

Heel Checks [1.2D + 0.5L + 1.6H + 1.0E]

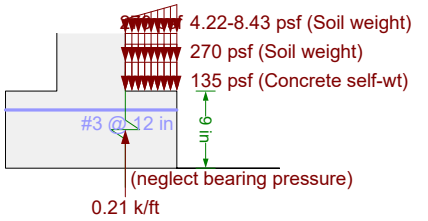
Heel Unfactored Loads

Unfactored Loads



Heel Factored Loads

1.2D + 0.5L + 1.6H + 1.0E



Controlling Moment

Design moment M_u for heel need not exceed moment at stem base:

M_{heel} = 0.05 ft·k / ft < M_{stem} = 0.22 ft·k / ft

M_u = 0.05 ft·k / ft (stem moment does not control)

Flexure Check (ACI 318-14 13.3.2.1, 7.5.2.1, »22.3, »22.2, 7.5.1.1a)

$$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0.01 \text{ in}^2 / \text{in})(60000 \text{ psi})}{0.85 (3000 \text{ psi})} = 0.22 \text{ in}$$

$\phi M_n = \phi A_s f_y (d - a / 2)$ = (0.90) (0.01 in² / in) (60000 psi) [(6.81 in) - (0.22 in) / 2] = 3.32 ft·k / ft

ϕM_n = 3.32 ft·k / ft ≥ M_u = 0.05 ft·k / ft ✓

Shear Check (ACI 318-14 13.3.2.1, 7.5.3.1, »22.5.1, »22.5.5, 7.5.1.1b)

λ = 1.0 (normal weight concrete)

$V_c = 2 \lambda \sqrt{F'_c}\, d$ = 2 (1.0) $\sqrt{3000 \text{ psi}}$ (6.81 in) = 8.96 k / ft

$\phi V_n = \phi V_c$ = (0.750) (8.96 k / ft) = 6.72 k / ft

ϕV_n = 6.72 k / ft ≥ V_u = 0.21 k / ft ✓

Minimum Strain Check (ACI 318-14 13.3.2.1, 7.3.3.1)

β_1 = 0.850 ($F'_c \le 4000$ psi)

$$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0.01 \text{ in}^2 / \text{in})(60000 \text{ psi})}{0.85 (3000 \text{ psi})} = 0.22 \text{ in}$$

$$\varepsilon_t = 0.003 \left(\frac{d}{a / \beta_1} - 1 \right) = 0.003 \left[\frac{(6.81 \text{ in})}{(0.22 \text{ in}) / (0.850)} - 1 \right] = 0.0775$$

ε_t = 0.0775 ≥ 0.004 ✓

Minimum Steel Check (ACI 318-14 13.3.2.1, 9.6.1)

ϕM_n = 3.32 ft·k / ft ≥ (4 / 3) M_u = [4 / 3] (0.05 ft·k / ft) = 0.07 ft·k / ft

Check is waived per ACI 9.6.1.3 ✓

Shrinkage and Temperature Steel (ACI 318-14 13.2.8.1, 7.6.4.1, 24.4.3.2, 24.4.3.3)

$$\rho_{ST_prov} = \frac{A_{ST}}{t\, s_{ST}} = \frac{(0.22 \text{ in}^2 / \text{in})}{(9 \text{ in})(12 \text{ in})} = 0.0020$$

$$\rho_{ST_prov} = \frac{A_{ST}}{t\, s_{ST}} = \frac{(0.22 \text{ in}^2 / \text{in})}{(9 \text{ in})(12 \text{ in})} = 0.0020$$

$$\frac{0.0018 (60000)}{f_y} = \frac{0.0018 (60000)}{(60000 \text{ psi})} = 0.0018$$

ρ_{ST_min} = 0.0018

ρ_{ST_prov} = 0.0020 ≥ ρ_{ST_min} = 0.0018 ✓

18 inch limit governs

s_{ST_max} = 18 in

s_{ST} = 12 in ≤ s_{ST_max} = 18 in ✓

Development Check (ACI 318-14 13.2.8.1, 25.4.2.3, 25.4.10)

$$\frac{M_u}{\phi M_n} = \frac{(0.05 \text{ ft·k / ft})}{(3.32 \text{ ft·k / ft})} = 0.0155$$ (ratio to represent excess reinforcement)

ψ_t = 1.0 (12 inches or less cast below - 6.63 inches)

ψ_e = 1.0 (bar not epoxy coated)

ψ_s = 0.80 (bars are #6 or smaller)

λ = 1.0 (normal weight concrete)

s / 2 = (12 in) / 2 = 6 in

cover + d_b / 2 = (2 in) + (0.38 in) / 2 = 2.19 in

c_b = 2.19 in (lesser of half spacing, ctr to surface)

K_{tr} = 0.0 (no transverse reinforcement)

$$\frac{c_b + K_{tr}}{d_b} = \frac{(2.19 \text{ in}) + (0.0)}{(0.38 \text{ in})} = 5.8333$$

$$l_d = \left(\frac{3.}{40} \frac{f_y}{\lambda \sqrt{F'_c}} \frac{\psi_t \psi_e \psi_s}{2.5} \right) d_b = \left[\frac{3.}{40} \frac{(60000 \text{ psi})}{(1.0) \sqrt{3000 \text{ psi}}} \frac{(1.0)(1.0)(0.80)}{2.5} \right] (0.38 \text{ in}) = 9.86 \text{ in}$$

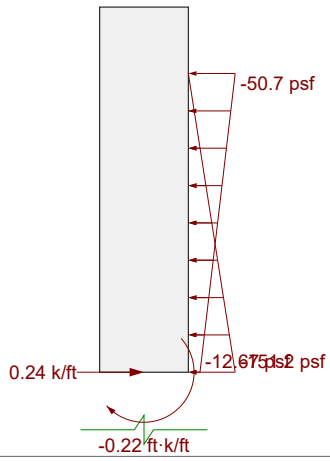
Factoring l_d by the excess reinforcement ratio (0.0155) per 25.4.10: l_d = 0.15 in

12 inch minimum controls

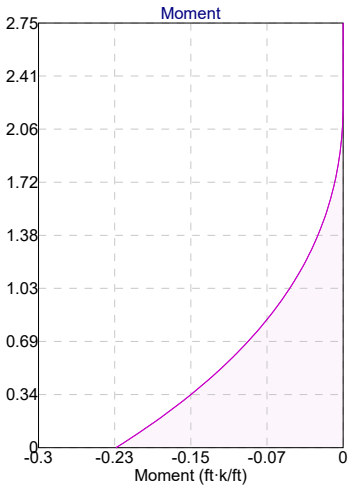
l_{d_prov} = 12 in ≥ l_d = 12 in ✓

Stem Forces [1.2D + 0.5L + 1.6H + 1.0E]

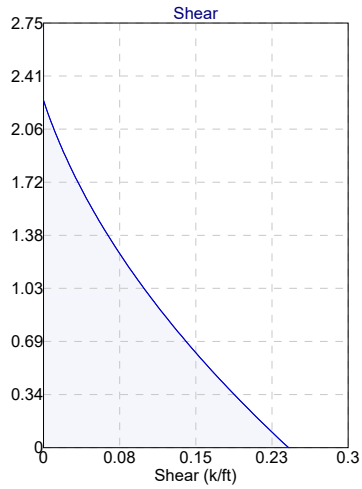
Stem Internal Forces



Stem Internal Forces



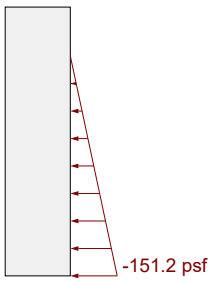
Stem Internal Forces

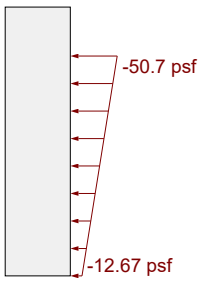


Stem Joint Force Transfer

Location	Force
@ stem base	0.24 k/ft

Stem Internal Forces





Stem Moment Checks [1.2D + 0.5L + 1.6H + 1.0E]



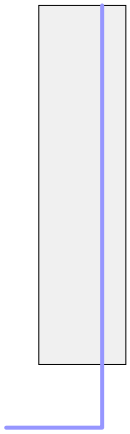
Check (ACI 318-14 11.5.5.1b) @ 0 ft from base

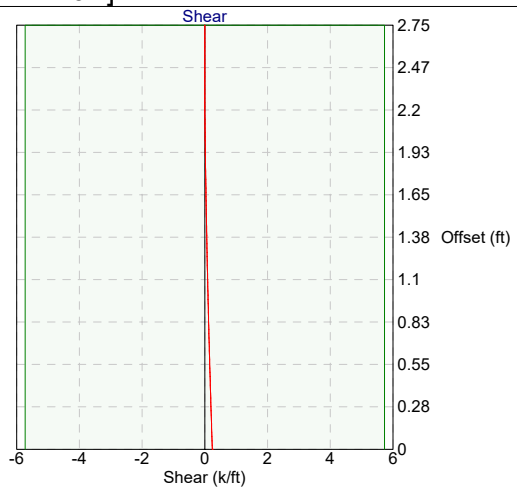
ϕM_n = 2.82 ft·k / ft ≥ M_u = 0.22 ft·k / ft ✓

Check (ACI 318-14 11.5.5.1b) @ 1.75 ft from base

ϕM_n = 2.82 ft·k / ft ≥ M_u = 0.01 ft·k / ft ✓

Stem Shear Checks [1.2D + 0.5L + 1.6H + 1.0E]





Shear Check (ACI 318-14 11.5.5.1c) @ 0 ft from base

ϕV_n = 5.73 k / ft ≥ V_u = 0.24 k / ft ✓

Stem Miscellaneous Checks [1.2D + 0.5L + 1.6H + 1.0E]

Minimum Steel Check (ACI 318-14 9.6.1) @ 0 ft from base [Stem in negative flexure]

$\phi M_n = 2.82$ ft·k / ft $\geq (4 / 3) M_u = [4 / 3] (0.22$ ft·k / ft$) = 0.3$ ft·k / ft

Check is waived per ACI 9.6.1.3 ✓

Minimum Steel Check (ACI 318-14 9.6.1) @ 2.75 ft from base [Stem in negative flexure]

$\phi M_n = 0$ ft·k / ft $\geq (4 / 3) M_u = [4 / 3] (0$ ft·k / ft$) = 0$ ft·k / ft

Check is waived per ACI 9.6.1.3 ✓

Maximum Steel Check (ACI 318-14 9.3.3.1) @ 0 ft from base [Stem in negative flexure]

$\beta_1 = 0.850$ ($F'_c \leq 4000$ psi)

$$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0.01 \text{ in}^2 / \text{in}) (60000 \text{ psi})}{0.85 (3000 \text{ psi})} = 0.22 \text{ in}$$

$$\varepsilon_t = 0.003 \left(\frac{d}{a / \beta_1} - 1 \right) = 0.003 \left[\frac{(5.81 \text{ in})}{(0.22 \text{ in}) / (0.850)} - 1 \right] = 0.0657$$

$\varepsilon_t = 0.0657 \geq 0.004$ ✓

Maximum Steel Check (ACI 318-14 9.3.3.1) @ 2.75 ft from base [Stem in negative flexure]

$\beta_1 = 0.850$ ($F'_c \leq 4000$ psi)

$$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0.01 \text{ in}^2 / \text{in}) (60000 \text{ psi})}{0.85 (3000 \text{ psi})} = 0.22 \text{ in}$$

$$\varepsilon_t = 0.003 \left(\frac{d}{a / \beta_1} - 1 \right) = 0.003 \left[\frac{(5.81 \text{ in})}{(0.22 \text{ in}) / (0.850)} - 1 \right] = 0.0657$$

$\varepsilon_t = 0.0657 \geq 0.004$ ✓

Wall Horizontal Steel (ACI 318-14 11.6.1, 11.7.3)

$$\rho_t = \frac{A_{s_horz} / s_{horz}}{t} = \frac{(0.11 \text{ in}^2) / (12 \text{ in})}{(8 \text{ in})} = 0.0011$$

$\rho_{t_min} = 0.0020$ (bars No. 5 or less, not less than 60 ksi)

$\rho_t = 0.0011 < \rho_{t_min} = 0.0020$ ✘

$3 h = 3 (8 \text{ in}) = 24$ in

18 inch limit governs

$s_{horz} = 12$ in $\leq s_{horz_max} = 18$ in ✓

Development Check (ACI 318-14 11.7.1.2, 25.4.2.3, 25.4.10)

$\frac{M_u}{\phi M_n} = \frac{(0.22 \text{ ft·k / ft})}{(2.82 \text{ ft·k / ft})} = 0.0793$ (ratio to represent excess reinforcement)

$\psi_e = 1.0$ (uncoated hooked bars)

$\psi_c = 0.70$ (based on side cover and extension cover)

$\psi_r = 1.0$ (no confining reinforcement)

$\lambda = 1.0$ (normal weight concrete)

$$l_{dh} = \left(\frac{f_y \psi_e \psi_c \psi_r}{50 \lambda \sqrt{F'_c}} \right) d_b = \left[\frac{(60000 \text{ psi}) (1.0) (0.70) (1.0)}{50 (1.0) \sqrt{3000 \text{ psi}}} \right] (0.38 \text{ in}) = 5.75 \text{ in}$$

Factoring l_{dh} by the excess reinforcement ratio (0.0793) per 25.4.10: $l_{dh} = 0.46$ in

$8 d_b = 8 (0.38 \text{ in}) = 3.0$ (minimum limit, does not control)

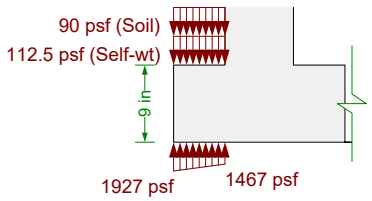
6 inch minimum controls

$l_{dh_prov} = 6$ in $\geq l_{dh} = 6$ in ✓

Toe Checks [1.4D]

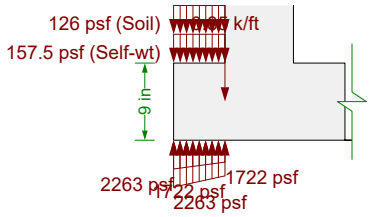
Toe Unfactored Loads

Unfactored Loads



Toe Factored Loads

1.4D



Controlling Moment

Design moment M_u for toe need not exceed moment at stem base:

M_{toe} = 0.22 ft·k / ft ≥ M_{stem} = - 0 ft·k / ft

M_u = - 0 ft·k / ft (stem base moment controls)

Flexure Check (ACI 318-14 13.3.2.1, 7.5.2.1, »22.3, »22.2, 7.5.1.1a)

$$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0.01 \text{ in}^2 / \text{in})(60000 \text{ psi})}{0.85 (3000 \text{ psi})} = 0.22 \text{ in}$$

$$\phi M_n = \phi A_s f_y (d - a / 2) = (0.90)(0.01 \text{ in}^2 / \text{in})(60000 \text{ psi})[(5.81 \text{ in}) - (0.22 \text{ in}) / 2] = 2.82 \text{ ft·k / ft}$$

ϕM_n = 2.82 ft·k / ft ≥ M_u = - 0 ft·k / ft ✓

Shear Check (ACI 318-14 13.3.2.1, 7.5.3.1, »22.5.1, »22.5.5, 7.5.1.1b)

λ = 1.0 (normal weight concrete)

$$V_c = 2 \lambda \sqrt{F'_c} d = 2 (1.0) \sqrt{3000 \text{ psi}} (5.81 \text{ in}) = 7.64 \text{ k / ft}$$

$$\phi V_n = \phi V_c = (0.750)(7.64 \text{ k / ft}) = 5.73 \text{ k / ft}$$

ϕV_n = 5.73 k / ft ≥ V_u = 0.03 k / ft ✓

Minimum Strain Check (ACI 318-14 13.3.2.1, 7.3.3.1)

β_1 = 0.850 ($F'_c \le 4000$ psi)

$$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0.01 \text{ in}^2 / \text{in})(60000 \text{ psi})}{0.85 (3000 \text{ psi})} = 0.22 \text{ in}$$

$$\varepsilon_t = 0.003 \left(\frac{d}{a / \beta_1} - 1 \right) = 0.003 \left[\frac{(5.81 \text{ in})}{(0.22 \text{ in}) / (0.850)} - 1 \right] = 0.0657$$

ε_t = 0.0657 ≥ 0.004 ✓

Minimum Steel Check (ACI 318-14 13.3.2.1, 9.6.1)

ϕM_n = 2.82 ft·k / ft ≥ (4 / 3) M_u = [4 / 3] (- 0 ft·k / ft) = - 0 ft·k / ft

Check is waived per ACI 9.6.1.3 ✓

Shrinkage and Temperature Steel (ACI 318-14 13.2.8.1, 7.6.4.1, 24.4.3.2, 24.4.3.3)

$$\rho_{ST_prov} = \frac{A_{ST}}{t\, s_{ST}} = \frac{(0.22 \text{ in}^2 / \text{in})}{(9 \text{ in})(12 \text{ in})} = 0.0020$$

$$\rho_{ST_prov} = \frac{A_{ST}}{t\, s_{ST}} = \frac{(0.22 \text{ in}^2 / \text{in})}{(9 \text{ in})(12 \text{ in})} = 0.0020$$

$$\frac{0.0018 (60000)}{f_y} = \frac{0.0018 (60000)}{(60000 \text{ psi})} = 0.0018$$

ρ_{ST_min} = 0.0018

ρ_{ST_prov} = 0.0020 ≥ ρ_{ST_min} = 0.0018 ✓

18 inch limit governs

s_{ST_max} = 18 in

s_{ST} = 12 in ≤ s_{ST_max} = 18 in ✓

Development Check (ACI 318-14 13.2.8.1, 25.4.2.3, 25.4.10)

$$\frac{M_u}{\phi M_n} = \frac{(- 0 \text{ ft·k / ft})}{(2.82 \text{ ft·k / ft})} = - 0.0$$ (ratio to represent excess reinforcement)

ψ_e = 1.0 (uncoated hooked bars)

ψ_c = 0.70 (based on side cover and extension cover)

ψ_r = 1.0 (no confining reinforcement)

λ = 1.0 (normal weight concrete)

$$l_{dh} = \left(\frac{f_y \psi_e \psi_c \psi_r}{50 \lambda \sqrt{F'_c}} \right) d_b = \left[\frac{(60000 \text{ psi})(1.0)(0.70)(1.0)}{50 (1.0) \sqrt{3000 \text{ psi}}} \right] (0.38 \text{ in}) = 5.75 \text{ in}$$

Factoring l_{dh} by the excess reinforcement ratio (- 0.0000) per 25.4.10: l_{dh} = - 0 in

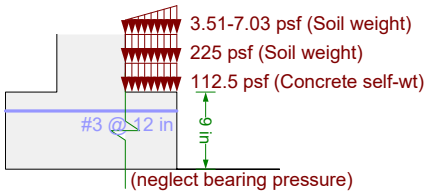
8 d_b = 8 (0.38 in) = 3.0 (minimum limit, does not control)

6 inch minimum controls

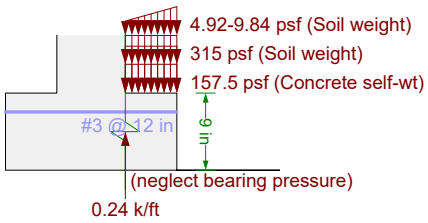
l_{dh_prov} = 6 in ≥ l_{dh} = 6 in ✓

Heel Checks [1.4D]

Heel Unfactored Loads



Heel Factored Loads



Controlling Moment

Design moment M_u for heel need not exceed moment at stem base:

M_{heel} = 0.06 ft·k / ft ≥ M_{stem} = - 0 ft·k / ft

M_u = - 0 ft·k / ft (stem base moment controls)

Flexure Check (ACI 318-14 13.3.2.1, 7.5.2.1, »22.3, »22.2, 7.5.1.1a)

$$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0.01 \text{ in}^2 / \text{in})(60000 \text{ psi})}{0.85 (3000 \text{ psi})} = 0.22 \text{ in}$$

$\phi M_n = \phi A_s f_y (d - a / 2)$ = (0.90) (0.01 in² / in) (60000 psi) [(6.81 in) - (0.22 in) / 2] = 3.32 ft·k / ft

ϕM_n = 3.32 ft·k / ft ≥ M_u = - 0 ft·k / ft ✓

Shear Check (ACI 318-14 13.3.2.1, 7.5.3.1, »22.5.1, »22.5.5, 7.5.1.1b)

λ = 1.0 (normal weight concrete)

$V_c = 2 \lambda \sqrt{F'_c}\, d$ = 2 (1.0) $\sqrt{3000 \text{ psi}}$ (6.81 in) = 8.96 k / ft

$\phi V_n = \phi V_c$ = (0.750) (8.96 k / ft) = 6.72 k / ft

ϕV_n = 6.72 k / ft ≥ V_u = 0.24 k / ft ✓

Minimum Strain Check (ACI 318-14 13.3.2.1, 7.3.3.1)

β_1 = 0.850 ($F'_c \le 4000$ psi)

$$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0.01 \text{ in}^2 / \text{in})(60000 \text{ psi})}{0.85 (3000 \text{ psi})} = 0.22 \text{ in}$$

$$\varepsilon_t = 0.003 \left(\frac{d}{a / \beta_1} - 1 \right) = 0.003 \left[\frac{(6.81 \text{ in})}{(0.22 \text{ in}) / (0.850)} - 1 \right] = 0.0775$$

ε_t = 0.0775 ≥ 0.004 ✓

Minimum Steel Check (ACI 318-14 13.3.2.1, 9.6.1)

ϕM_n = 3.32 ft·k / ft ≥ (4 / 3) M_u = [4 / 3] (- 0 ft·k / ft) = - 0 ft·k / ft

Check is waived per ACI 9.6.1.3 ✓

Shrinkage and Temperature Steel (ACI 318-14 13.2.8.1, 7.6.4.1, 24.4.3.2, 24.4.3.3)

$$\rho_{ST_prov} = \frac{A_{ST}}{t\, s_{ST}} = \frac{(0.22 \text{ in}^2 / \text{in})}{(9 \text{ in})(12 \text{ in})} = 0.0020$$

$$\rho_{ST_prov} = \frac{A_{ST}}{t\, s_{ST}} = \frac{(0.22 \text{ in}^2 / \text{in})}{(9 \text{ in})(12 \text{ in})} = 0.0020$$

$$\frac{0.0018 (60000)}{f_y} = \frac{0.0018 (60000)}{(60000 \text{ psi})} = 0.0018$$

ρ_{ST_min} = 0.0018

ρ_{ST_prov} = 0.0020 ≥ ρ_{ST_min} = 0.0018 ✓

18 inch limit governs

s_{ST_max} = 18 in

s_{ST} = 12 in ≤ s_{ST_max} = 18 in ✓

Development Check (ACI 318-14 13.2.8.1, 25.4.2.3, 25.4.10)

$$\frac{M_u}{\phi M_n} = \frac{(- 0 \text{ ft·k / ft})}{(3.32 \text{ ft·k / ft})} = - 0.0$$ (ratio to represent excess reinforcement)

ψ_t = 1.0 (12 inches or less cast below - 6.63 inches)

ψ_e = 1.0 (bar not epoxy coated)

ψ_s = 0.80 (bars are #6 or smaller)

λ = 1.0 (normal weight concrete)

s / 2 = (12 in) / 2 = 6 in

cover + d_b / 2 = (2 in) + (0.38 in) / 2 = 2.19 in

c_b = 2.19 in (lesser of half spacing, ctr to surface)

K_{tr} = 0.0 (no transverse reinforcement)

$$\frac{c_b + K_{tr}}{d_b} = \frac{(2.19 \text{ in}) + (0.0)}{(0.38 \text{ in})} = 5.8333$$

$$l_d = \left(\frac{3.}{40} \frac{f_y}{\lambda \sqrt{F'_c}} \frac{\psi_t \psi_e \psi_s}{2.5} \right) d_b = \left[\frac{3.}{40} \frac{(60000 \text{ psi})}{(1.0) \sqrt{3000 \text{ psi}}} \frac{(1.0)(1.0)(0.80)}{2.5} \right] (0.38 \text{ in}) = 9.86 \text{ in}$$

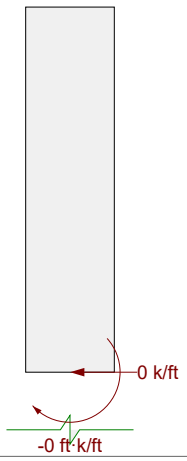
Factoring l_d by the excess reinforcement ratio (- 0.0000) per 25.4.10: l_d = - 0 in

12 inch minimum controls

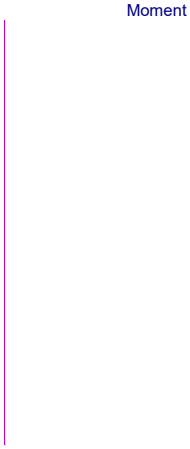
l_{d_prov} = 12 in ≥ l_d = 12 in ✓

Stem Forces [1.4D]

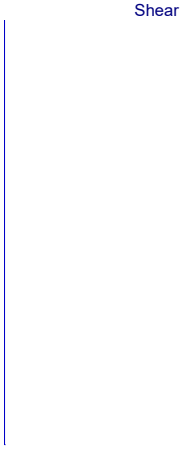
Stem Internal Forces



Stem Internal Forces



Stem Internal Forces

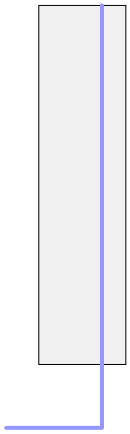


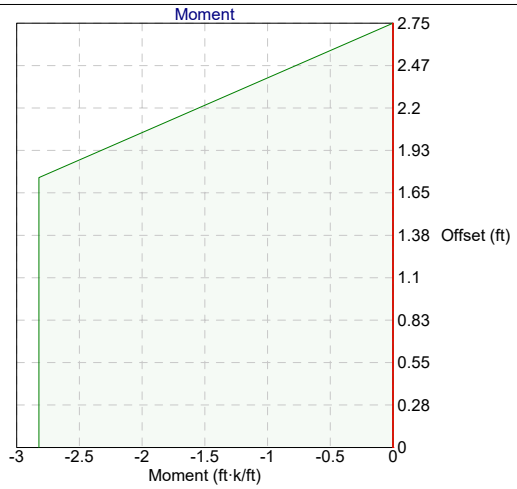
Stem Joint Force Transfer

Location	Force
@ stem base	0 k/ft

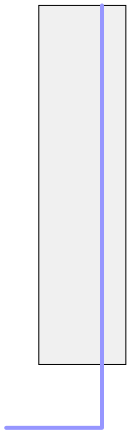
Stem Internal Forces

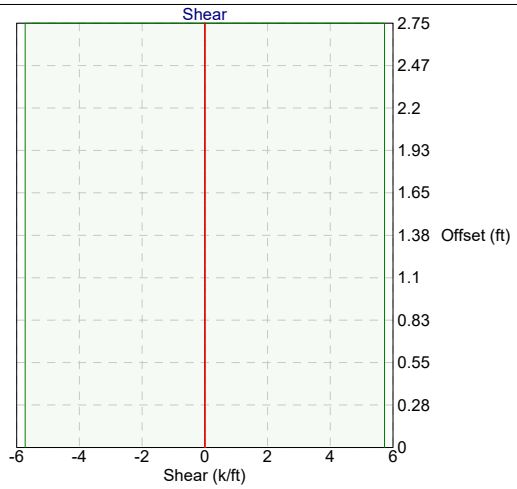
Stem Moment Checks [1.4D]





Stem Shear Checks [1.4D]





Stem Miscellaneous Checks [1.4D]

Minimum Steel Check (ACI 318-14 9.6.1) @ 0 ft from base [Stem in negative flexure]

ϕM_n = 2.82 ft·k / ft ≥ (4 / 3) M_u = [4 / 3] (0 ft·k / ft) = 0 ft·k / ft

Check is waived per ACI 9.6.1.3 ✓

Minimum Steel Check (ACI 318-14 9.6.1) @ 2.75 ft from base [Stem in negative flexure]

ϕM_n = 0 ft·k / ft ≥ (4 / 3) M_u = [4 / 3] (0 ft·k / ft) = 0 ft·k / ft

Check is waived per ACI 9.6.1.3 ✓

Maximum Steel Check (ACI 318-14 9.3.3.1) @ 0 ft from base [Stem in negative flexure]

β_1 = 0.850 ($F'_c \le 4000$ psi)

$$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0.01 \text{ in}^2 / \text{in})(60000 \text{ psi})}{0.85 (3000 \text{ psi})} = 0.22 \text{ in}$$

$$\varepsilon_t = 0.003 \left(\frac{d}{a / \beta_1} - 1 \right) = 0.003 \left[\frac{(5.81 \text{ in})}{(0.22 \text{ in}) / (0.850)} - 1 \right] = 0.0657$$

ε_t = 0.0657 ≥ 0.004 ✓

Maximum Steel Check (ACI 318-14 9.3.3.1) @ 2.75 ft from base [Stem in negative flexure]

β_1 = 0.850 ($F'_c \le 4000$ psi)

$$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0.01 \text{ in}^2 / \text{in})(60000 \text{ psi})}{0.85 (3000 \text{ psi})} = 0.22 \text{ in}$$

$$\varepsilon_t = 0.003 \left(\frac{d}{a / \beta_1} - 1 \right) = 0.003 \left[\frac{(5.81 \text{ in})}{(0.22 \text{ in}) / (0.850)} - 1 \right] = 0.0657$$

ε_t = 0.0657 ≥ 0.004 ✓

Wall Horizontal Steel (ACI 318-14 11.6.1, 11.7.3)

$$\rho_t = \frac{A_{s_horz} / s_{horz}}{t} = \frac{(0.11 \text{ in}^2) / (12 \text{ in})}{(8 \text{ in})} = 0.0011$$

ρ_{t_min} = 0.0020 (bars No. 5 or less, not less than 60 ksi)

ρ_t = 0.0011 < ρ_{t_min} = 0.0020 ✘

3 h = 3 (8 in) = 24 in

18 inch limit governs

s_{horz} = 12 in ≤ s_{horz_max} = 18 in ✓

Development Check (ACI 318-14 11.7.1.2, 25.4.2.3, 25.4.10)

$\frac{M_u}{\phi M_n} = \frac{(0 \text{ ft·k / ft})}{(2.82 \text{ ft·k / ft})} = 0.0$ (ratio to represent excess reinforcement)

ψ_e = 1.0 (uncoated hooked bars)

ψ_c = 0.70 (based on side cover and extension cover)

ψ_r = 1.0 (no confining reinforcement)

λ = 1.0 (normal weight concrete)

$$l_{dh} = \left(\frac{f_y \psi_e \psi_c \psi_r}{50 \lambda \sqrt{F'_c}} \right) d_b = \left[\frac{(60000 \text{ psi})(1.0)(0.70)(1.0)}{50 (1.0) \sqrt{3000 \text{ psi}}} \right] (0.38 \text{ in}) = 5.75 \text{ in}$$

Factoring l_{dh} by the excess reinforcement ratio (0.0000) per 25.4.10: l_{dh} = 0 in

8 d_b = 8 (0.38 in) = 3.0 (minimum limit, does not control)

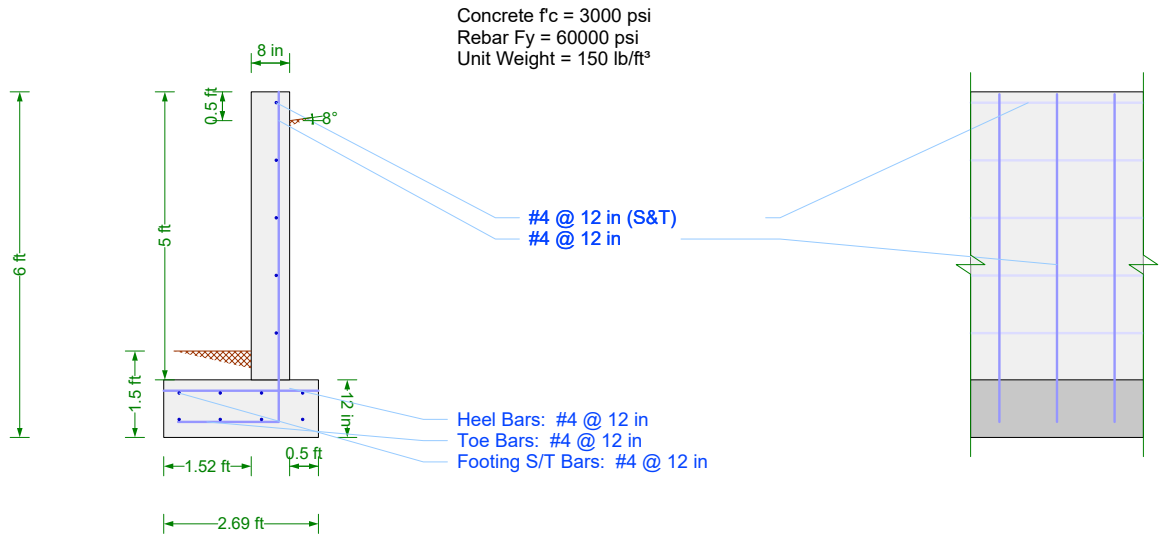
6 inch minimum controls

l_{dh_prov} = 6 in ≥ l_{dh} = 6 in ✓

JOSEPH LUCIA
LUCIA ENGINEERING, INC.

Cantilever Wall 4' Tall

Design Detail



Check Summary

Ratio	Check	Provided	Required	Combination
----- Stability Checks -----				
✓ 0.675	Overturning	1.78	1.20	1.0D + 1.0L + 1.0H + 0.7E
✓ 0.816	Bearing Pressure	2500 psf	2041 psf	1.0D + 1.0L + 1.0H + 0.7E
✓ 0.364	Bearing Eccentricity	5.87 in	16.12 in	1.0D + 1.0L + 1.0H + 0.7E
----- Toe Checks -----				
✓ 0.164	Shear	8.63 k/ft	1.41 k/ft	1.2D + 1.6L + 1.6H
✓ 0.233	Moment	7.7 ft·k/ft	1.79 ft·k/ft	1.2D + 0.5L + 1.6H + 1.0E
✓ 0.074	Min Strain	0.0539	0.0040	1.2D + 1.6L + 1.6H
✓ 0.000	Min Steel	0.02 in²	0 in²	1.2D + 1.6L + 1.6H
✓ 1.000	Development	6 in	6 in	1.2D + 1.6L + 1.6H
✓ 0.667	S&T Max Spacing	12 in	18 in	1.2D + 1.6L + 1.6H
✓ 0.648	S&T Min Rho	0.0028	0.0018	1.2D + 1.6L + 1.6H
----- Heel Checks -----				
✓ 0.044	Shear	9.61 k/ft	0.42 k/ft	1.4D
✓ 0.011	Moment	8.6 ft·k/ft	0.09 ft·k/ft	1.2D + 1.6L + 1.6H
✓ 0.066	Min Strain	0.0604	0.0040	1.2D + 1.6L + 1.6H
✓ 0.000	Min Steel	0.02 in²	0 in²	1.2D + 1.6L + 1.6H
✓ 0.495	Development	24.24 in	12 in	1.2D + 1.6L + 1.6H
✓ 0.667	S&T Max Spacing	12 in	18 in	1.2D + 1.6L + 1.6H
✓ 0.648	S&T Min Rho	0.0028	0.0018	1.2D + 1.6L + 1.6H
----- Stem Checks -----				
✓ 0.358	Moment	5 ft·k/ft	1.79 ft·k/ft	1.2D + 0.5L + 1.6H + 1.0E
✓ 0.170	Shear	5.67 k/ft	0.97 k/ft	1.2D + 0.5L + 1.6H + 1.0E
✓ 0.116	Max Steel	0.0344	0.0040	1.2D + 1.6L + 1.6H
✓ 0.000	Min Steel	0.02 in²/in	0 in²/in	1.2D + 1.6L + 1.6H
✓ 0.667	Base Development	9 in	6 in	1.2D + 1.6L + 1.6H
✓ 0.000	Horz Bar Rho	0.0000	0.0000	1.2D + 1.6L + 1.6H
✓ 0.667	Horz Bar Spacing	12 in	18 in	1.2D + 1.6L + 1.6H

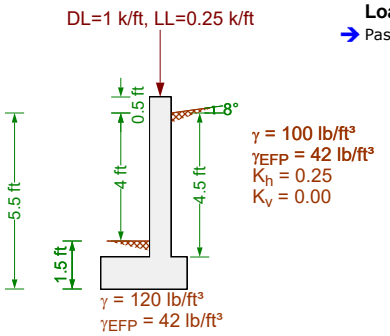
Criteria

Use basic criteria from common proje...	Yes
Building Code	IBC 2021
Concrete Load Combs	IBC 2021 (Strength)
Masonry Load Combs	ASCE 7-16 (ASD)
Stability Load Combs	IBC Retaining Wall St...
Apply Sds Factor to Seismic Combin...	No
Restrained Against Sliding	Yes
Neglect Bearing At Heel	Yes
Use Vert. Comp. for OT	No
Use Vert. Comp. for Sliding	No
Use Vert. Comp. for Bearing	Yes
Use Surcharge for Sliding & OT	Yes
Use Surcharge for Bearing	Yes
Neglect Soil Over Toe	No
Neglect Backfill Wt. for Coulomb	No
Factor Soil Weight As Dead	Yes
Use Passive Force for OT	Yes
Assume Pressure To Top	Yes
Extend Backfill Pressure To Key Bott...	No
Use Toe Passive Pressure for Bearing	No
Required F.S. for OT	1.50
Required F.S. for Sliding	1.50
Has Different Safety Factors for Seis...	Yes
Seismic F.S. for OT	1.20
Seismic F.S. for Sliding	1.20
Allowable Bearing Pressure	2500 psf
Req'd Bearing Location	Over footing
Wall Friction Angle	25°
Friction Coefficent	0.35
Soil Reaction Modulus	172800 lb/ft³

JOSEPH LUCIA
LUCIA ENGINEERING, INC.

Cantilever Wall 4' Tall

Loads



Loading Options/Assumptions

➔ Passive pressure neglects top 0 ft of soil.

Load Combinations

IBC 2018 (Strength)

1.2D + 1.6L + 1.6H
1.2D + 1.6L + 0.9H
1.2D + 0.5L + 1.6H + 1.0E
1.2D + 0.5L + 1.6H
1.2D + 0.5L + 0.9H + 1.0E
1.2D + 0.5L + 0.9H
1.2D + 1.6H + 1.0E
1.2D + 1.6H
1.2D + 0.9H + 1.0E
1.2D + 0.9H
0.9D + 1.6H + 1.0E
0.9D + 1.6H
0.9D + 0.9H + 1.0E
0.9D + 0.9H
1.4D

Strength Check Results Summary

Load Combination	Stem M-applied (ft·k/ft)	Stem M-allow (ft·k/ft)	Stem V-applied (k/ft)	Stem V-allow (k/ft)	Stem Min. ld (in)	Stem Actual ld (in)	Stem Min. strain	Stem Actual strain	Stem Min. steel (in²/in)
1.2D + 1.6L + 1.6H	1.02	5	0.68	5.67	6	9	0.0040	0.0344	0
1.2D + 1.6L + 0.9H	0.57	5	0.38	5.67	6	9	0.0040	0.0344	0
1.2D + 0.5L + 1.6H + 1.0E	1.79	5	0.97	5.67	6	9	0.0040	0.0344	0
1.2D + 0.5L + 1.6H	1.02	5	0.68	5.67	6	9	0.0040	0.0344	0
1.2D + 0.5L + 0.9H + 1.0E	1.34	5	0.67	5.67	6	9	0.0040	0.0344	0
1.2D + 0.5L + 0.9H	0.57	5	0.38	5.67	6	9	0.0040	0.0344	0
1.2D + 1.6H + 1.0E	1.79	5	0.97	5.67	6	9	0.0040	0.0344	0
1.2D + 1.6H	1.02	5	0.68	5.67	6	9	0.0040	0.0344	0
1.2D + 0.9H + 1.0E	1.34	5	0.67	5.67	6	9	0.0040	0.0344	0
1.2D + 0.9H	0.57	5	0.38	5.67	6	9	0.0040	0.0344	0
0.9D + 1.6H + 1.0E	1.79	5	0.97	5.67	6	9	0.0040	0.0344	0
0.9D + 1.6H	1.02	5	0.68	5.67	6	9	0.0040	0.0344	0
0.9D + 0.9H + 1.0E	1.34	5	0.67	5.67	6	9	0.0040	0.0344	0
0.9D + 0.9H	0.57	5	0.38	5.67	6	9	0.0040	0.0344	0
1.4D	0	0	0	0	6	9	0.0040	0.0344	0

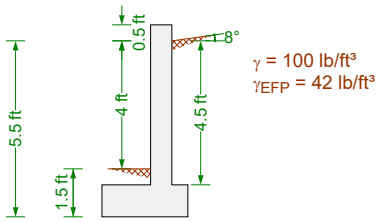
Load Combination	Stem Actual steel (in²/in)	Heel M-applied (ft·k/ft)	Heel M-allow (ft·k/ft)	Heel V-applied (k/ft)	Heel V-allow (k/ft)	Toe M-applied (ft·k/ft)	Toe M-allow (ft·k/ft)	Toe V-applied (k/ft)	Toe V-allow (k/ft)
1.2D + 1.6L + 1.6H	0.02	0.09	8.6	0.36	9.61	1.93	7.7	1.41	8.63
1.2D + 1.6L + 0.9H	0.02	0.09	8.6	0.36	9.61	1.93	7.7	1.41	8.63
1.2D + 0.5L + 1.6H + 1.0E	0.02	0.09	8.6	0.36	9.61	1.88	7.7	1.37	8.63
1.2D + 0.5L + 1.6H	0.02	0.09	8.6	0.36	9.61	1.73	7.7	1.27	8.63
1.2D + 0.5L + 0.9H + 1.0E	0.02	0.09	8.6	0.36	9.61	1.88	7.7	1.37	8.63
1.2D + 0.5L + 0.9H	0.02	0.09	8.6	0.36	9.61	1.73	7.7	1.27	8.63
1.2D + 1.6H + 1.0E	0.02	0.09	8.6	0.36	9.61	1.78	7.7	1.31	8.63
1.2D + 1.6H	0.02	0.09	8.6	0.36	9.61	1.64	7.7	1.2	8.63
1.2D + 0.9H + 1.0E	0.02	0.09	8.6	0.36	9.61	1.78	7.7	1.31	8.63
1.2D + 0.9H	0.02	0.09	8.6	0.36	9.61	1.64	7.7	1.2	8.63
0.9D + 1.6H + 1.0E	0.02	0.07	8.6	0.27	9.61	1.38	7.7	1.01	8.63
0.9D + 1.6H	0.02	0.07	8.6	0.27	9.61	1.23	7.7	0.9	8.63
0.9D + 0.9H + 1.0E	0.02	0.07	8.6	0.27	9.61	1.38	7.7	1.01	8.63
0.9D + 0.9H	0.02	0.07	8.6	0.27	9.61	1.23	7.7	0.9	8.63
1.4D	0.02	0.11	8.6	0.42	9.61	1.91	7.7	1.4	8.63

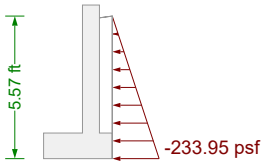
Stability Check Results Summary

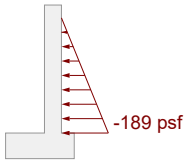
Load Combination	Overturning Moment (ft·k/ft)	Resisting Moment (ft·k/ft)	Overturning F.S.	Overturning F.S. Req'd	Overturning F.S. Req'd Seismic	Sliding Force (lb/in)	Resisting Force (lb/in)	Sliding F.S.
1.0D + 1.0L + 1.0H + 0.7E	1.21	3.97	3.279	1.500	1.200	54.3	80.17	1.476
1.0D + 1.0L + 1.0H	1.21	3.97	3.279	1.500	1.200	54.3	76.01	1.400
1.0D + 1.0H + 0.7E	1.21	3.97	3.279	1.500	1.200	54.3	72.88	1.342
1.0D + 1.0H	1.21	3.97	3.279	1.500	1.200	54.3	68.72	1.266

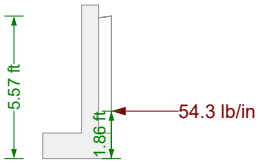
Load Combination	Sliding F.S. Req'd	Sliding F.S. Req'd Seismic	Bearing Pressure Actual (psf)	Bearing Pressure Allowable (psf)	Bearing Eccentricity Actual (in)	Bearing Eccentricity Allowable (in)	Wall Top Actual Deflection (in)
1.0D + 1.0L + 1.0H + 0.7E	1.500	1.200	2041	2500	5.87	16.12	0.28
1.0D + 1.0L + 1.0H	1.500	1.200	1929	2500	5.87	16.12	0.28
1.0D + 1.0H + 0.7E	1.500	1.200	1845	2500	5.87	16.12	0.28
1.0D + 1.0H	1.500	1.200	1734	2500	5.87	16.12	0.28

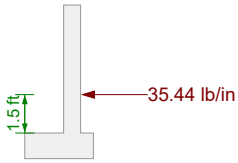
Backfill Pressure











Lateral Earth Pressure

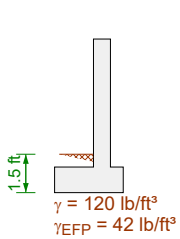
Equivalent Fluid Pressure

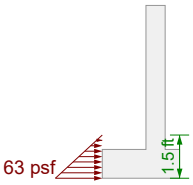
$\sigma_h = H\,\gamma_{fluid}$ = (5.57 ft)(42 lb / ft³) = 234 psf

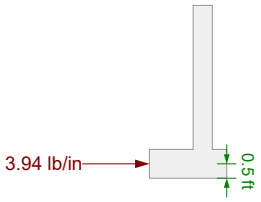
Lateral Earth Pressure (stem only)

$\sigma_h = H\,\gamma_{fluid}$ = (4.5 ft)(42 lb / ft³) = 189 psf

Passive Pressure





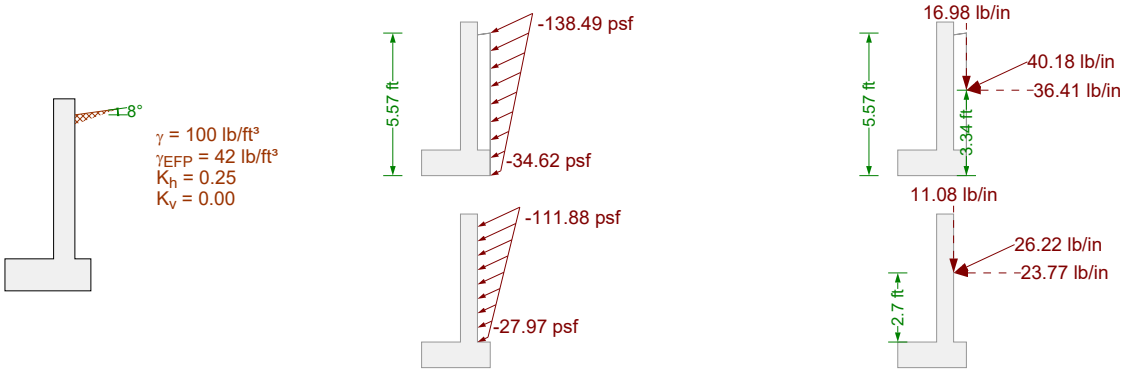


Lateral Earth Pressure

Equivalent Fluid Pressure

$\sigma_h = H\,\gamma_{fluid} = (1.5\text{ ft})(42\text{ lb / ft}^3) = 63\text{ psf}$

Seismic Pressure



Seismic Pressure

Dynamic + static force (Mononobe - Okabe equation)

$$\theta' = \text{atan}\left(\frac{k_h}{1 - k_v}\right) = \arctan\left[\frac{(0.250)}{1 - (0.0)}\right] = 14.04°$$

$$K_{ae} = \frac{\sin^2(\beta + \phi - \theta')}{\cos(\theta')\sin^2(\beta)\sin(\beta - \theta' - \delta)\left[1 + \sqrt{\frac{\sin(\phi + \delta)\sin(\phi - \theta' - \alpha)}{\sin(\beta - \delta - \theta')\sin(\alpha + \beta)}}\right]^{\wedge} 2}$$

$$= \frac{\sin^2[(90°) + (30°) - (14.04°)]}{\cos((14.04°))\sin^2((90°))\sin[(90°) - (14.04°) - (25°)]\left[1 + \sqrt{\frac{\sin[(30°) + (25°)]\sin[(30°) - (14.04°) - (8°)]}{\sin[(90°) - (25°) - (14.04°)]\sin[(8°) + (90°)]}}\right]^{\wedge} 2}$$

$$= 0.6403$$

$$P_{ae} = \frac{1}{2} K_{ae}\,\gamma\, H^{\wedge} 2\,(1 - k_v) = \frac{1}{2}(0.6403)(100\text{ lb / ft}^3)(5.57\text{ ft})^{\wedge} 2\,[1 - (0.0)] = 82.78\text{ lb / in}$$

Static - only force (Coulomb equation)

$$K_a = \frac{\sin^2(\beta + \phi)}{\sin^2(\beta)\sin(\beta - \delta)\left[1 + \sqrt{\frac{\sin(\phi + \delta)\sin(\phi - \alpha)}{\sin(\beta - \delta)\sin(\alpha + \beta)}}\right]^{\wedge} 2}$$

$$= \frac{\sin^2[(90°) + (30°)]}{\sin^2((90°))\sin[(90°) - (25°)]\left[1 + \sqrt{\frac{\sin[(30°) + (25°)]\sin[(30°) - (8°)]}{\sin[(90°) - (25°)]\sin[(8°) + (90°)]}}\right]^{\wedge} 2}$$

$$= 0.3295$$

$$P_a = \frac{1}{2} K_a\,\gamma\, H^{\wedge} 2 = \frac{1}{2}(0.3295)(100\text{ lb / ft}^3)(5.57\text{ ft})^{\wedge} 2 = 42.6\text{ lb / in}$$

Net dynamic force

$$\Delta P_{ae} = P_{ae} - P_a = (82.78\text{ lb / in}) - (42.6\text{ lb / in}) = 40.18\text{ lb / in}$$

$\alpha_P = 90° - \beta + \delta = 90° - (90°) + (25°) = 25°$ (resultant force angle with horizontal)

To arrive at the pressure distribution illustrated above (used to determine stem moments), apply inverted triangular pressure plus a uniform portion to bring resultant to 0.6H

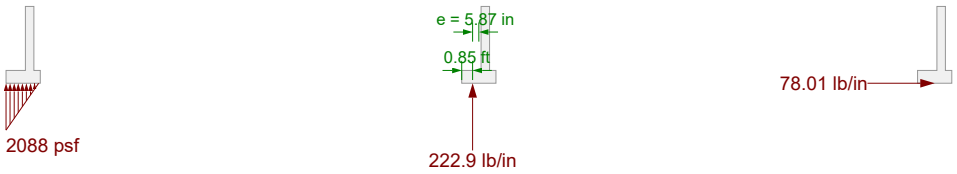
$$\sigma_{e_top} = \frac{8}{5}\frac{\Delta P_{ae}}{H} = \frac{8}{5}\frac{(40.18\text{ lb / in})}{(5.57\text{ ft})} = 138.5\text{ psf}$$

$$\sigma_{e_bot} = \frac{2}{5}\frac{\Delta P_{ae}}{H} = \frac{2}{5}\frac{(40.18\text{ lb / in})}{(5.57\text{ ft})} = 34.62\text{ psf}$$

Wall/Soil Weights



Bearing Pressure



Friction

$F = \mu R = (0.350)(222.9\ lb/in) = 78.01\ lb/in$

Bearing Pressure Calculation

Contributing Forces

	Vert Force	...offset	Horz Force	...offset	OT Moment
Backfill Pressure	-0 lb/in	-	-54.3 lb/in	1.86 ft	14518 in·lb/ft
Axial Dead Load	-83.33 lb/in	1.85 ft	0 lb/in	-	-22240 in·lb/ft
Axial Live Load	-20.83 lb/in	1.85 ft	0 lb/in	-	-5560 in·lb/ft
Seismic Force	-16.98 lb/in	2.69 ft	-36.41 lb/in	3.34 ft	10956 in·lb/ft
Footing Weight	-33.58 lb/in	1.34 ft	0 lb/in	-	-6496.36 in·lb/ft
Stem Weight	-41.67 lb/in	1.85 ft	0 lb/in	-	-11120 in·lb/ft
Backfill Weight	-18.75 lb/in	2.44 ft	0 lb/in	-	-6579 in·lb/ft
Backfill Weight	-0.15 lb/in	2.52 ft	0 lb/in	-	-53.12 in·lb/ft
Soil over toe Weight	-7.6 lb/in	0.76 ft	0 lb/in	-	-831.74 in·lb/ft
	-222.89 lb/in				-27406.43 in·lb/ft

$$\frac{-27406.43\ in{\cdot}lb/ft}{-222.89\ lb/in} = 0.85\ ft$$

Stability Checks [1.0D + 1.0L + 1.0H + 0.7E]

Overturning Check

Overturning Moments

	Force	Distance	Moment
Backfill pressure (horz)	54.3 lb/in	1.86 ft	14518 in·lb/ft
Seismic force	25.49 lb/in	3.34 ft	12268 in·lb/ft
		Total:	26786 in·lb/ft

Resisting Moments

	Force	Distance	Moment
Passive pressure @ toe	3.94 lb/in	0.5 ft	283.5 in·lb/ft
Axial dead load	-83.33 lb/in	1.85 ft	22240 in·lb/ft
Footing Weight	-33.58 lb/in	1.34 ft	6496 in·lb/ft
Stem Weight	-41.67 lb/in	1.85 ft	11120 in·lb/ft
Backfill Weight	-18.75 lb/in	2.44 ft	6579 in·lb/ft
Backfill Weight	-0.15 lb/in	2.52 ft	53.12 in·lb/ft
Soil over toe Weight	-7.6 lb/in	0.76 ft	831.7 in·lb/ft
		Total:	47604 in·lb/ft

Without seismic loads:

$$F.S. = \frac{RM}{OTM} = \frac{47604 \text{ in·lb / ft}}{14518 \text{ in·lb / ft}} = 3.279 > 1.50 \text{ (OK)}$$

Including seismic loads:

$$F.S. = \frac{RM}{OTM} = \frac{47604 \text{ in·lb / ft}}{26786 \text{ in·lb / ft}} = 1.777 > 1.20 \text{ (OK)}$$

Sliding Check

Check not performed; restrained against sliding.

Bearing Capacity Check

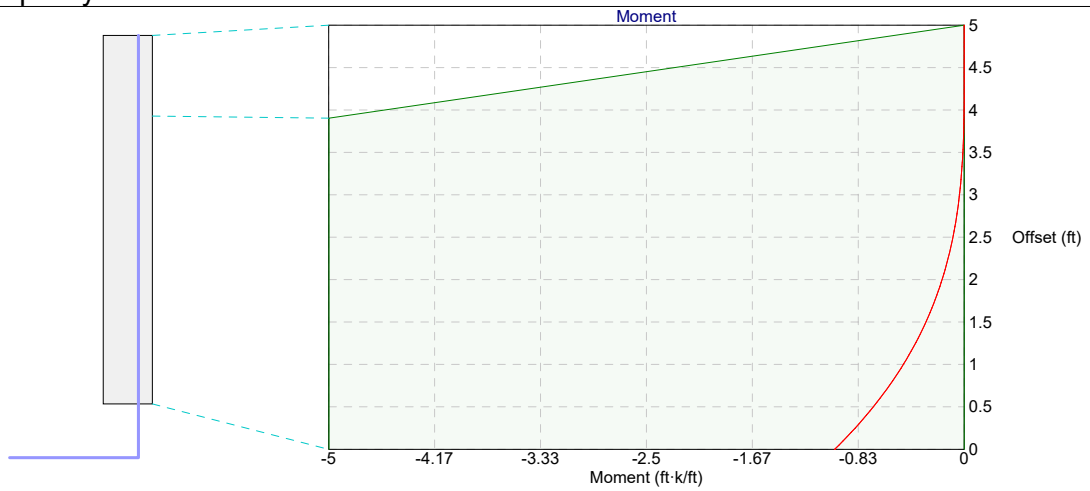
Bearing pressure < allowable (2041 psf < 2500 psf) - OK
Bearing resultant eccentricity < allowable (5.87 in < 16.12 in) - OK

Wall Top Displacement

(based on unfactored service loads)

Deflection due to stem flexural displacement	0.008 in
Deflection due to rotation from settlement	0.27 in
Total deflection at top of wall (positive towards toe)	0.278 in

Stem Flexural Capacity



Capacity (ACI 318-14 11.5.2.2, »22.3, »22.2) @ 0 ft from base

$$a = \frac{A_s f_y}{0.85 f'_c} = \frac{(0.02 \text{ in}^2 / \text{in})(60000 \text{ psi})}{0.85 (3000 \text{ psi})} = 0.39 \text{ in}$$

$$\phi M_n = \phi A_s f_y (d - a / 2) = (0.90)(0.02 \text{ in}^2 / \text{in})(60000 \text{ psi})[(5.75 \text{ in}) - (0.39 \text{ in}) / 2] = 5 \text{ ft·k / ft}$$

Capacity (ACI 318-14 11.5.2.2, »22.3, »22.2) @ 3.9 ft from base

$$a = \frac{A_s f_y}{0.85 f'_c} = \frac{(0.02 \text{ in}^2 / \text{in})(60000 \text{ psi})}{0.85 (3000 \text{ psi})} = 0.39 \text{ in}$$

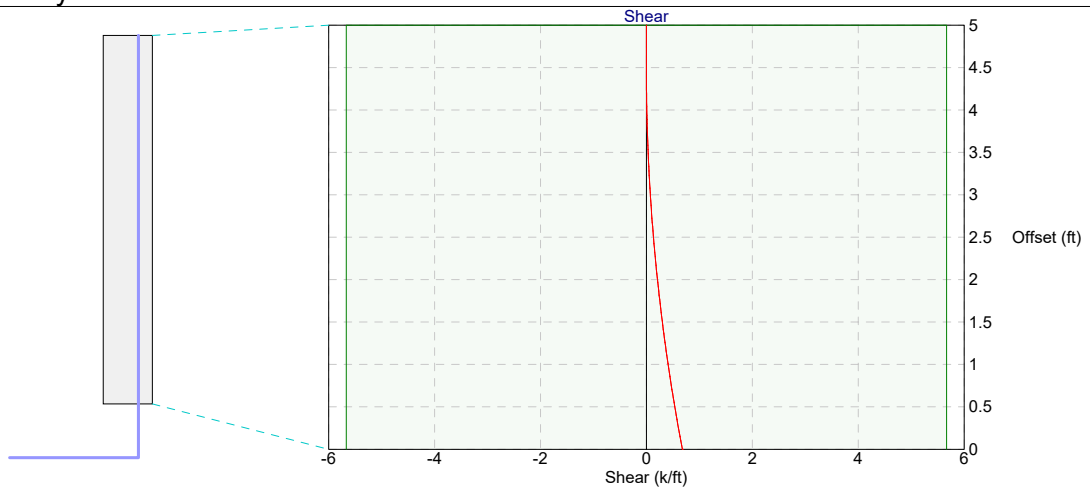
$$\phi M_n = \phi A_s f_y (d - a / 2) = (0.90)(0.02 \text{ in}^2 / \text{in})(60000 \text{ psi})[(5.75 \text{ in}) - (0.39 \text{ in}) / 2] = 5 \text{ ft·k / ft}$$

Capacity (ACI 318-14 11.5.2.2, »22.3, »22.2) @ 5 ft from base

$$a = \frac{A_s f_y}{0.85 f'_c} = \frac{(0 \text{ in}^2 / \text{in})(60000 \text{ psi})}{0.85 (3000 \text{ psi})} = 0 \text{ in}$$

$$\phi M_n = \phi A_s f_y (d - a / 2) = (0.90)(0 \text{ in}^2 / \text{in})(60000 \text{ psi})[(5.75 \text{ in}) - (0 \text{ in}) / 2] = 0 \text{ ft·k / ft}$$

Stem Shear Capacity



Shear Capacity (ACI 318-14 11.5.5.1, 22.5.1.1, 22.5.5.1) @ 0 ft from base

λ = 1.0 (normal weight concrete)

$V_c = 2 \lambda \sqrt{F'_c}\, d$ = 2 (1.0) $\sqrt{3000 \text{ psi}}$ (5.75 in) = 7.56 k / ft

$\phi V_n = \phi V_c$ = (0.750) (7.56 k / ft) = 5.67 k / ft

Shear Capacity (ACI 318-14 11.5.5.1, 22.5.1.1, 22.5.5.1) @ 5 ft from base

λ = 1.0 (normal weight concrete)

$V_c = 2 \lambda \sqrt{F'_c}\, d$ = 2 (1.0) $\sqrt{3000 \text{ psi}}$ (5.75 in) = 7.56 k / ft

$\phi V_n = \phi V_c$ = (0.750) (7.56 k / ft) = 5.67 k / ft

Stem Development/Lap Length Calculations

Main vertical stem bars (bottom end) - Development Length Calculation (ACI 318-14 11.7.1.2, 25.4.2.3, 25.4.3)

ψ_e = 1.0 (uncoated hooked bars)

ψ_c = 0.70 (based on side cover and extension cover)

ψ_r = 1.0 (no confining reinforcement)

λ = 1.0 (normal weight concrete)

$$l_{dh} = \left(\frac{f_y \, \psi_e \, \psi_c \, \psi_r}{50 \, \lambda \sqrt{F'_c}} \right) d_b = \left[\frac{(60000 \text{ psi}) \, (1.0) \, (0.70) \, (1.0)}{50 \, (1.0) \sqrt{3000 \text{ psi}}} \right] (0.5 \text{ in}) = 7.67 \text{ in}$$

$8 \, d_b = 8 \, (0.5 \text{ in}) = 4.0$ (minimum limit, does not control)

Main vertical stem bars (top end) - Development Length Calculation (ACI 318-14 11.7.1.2, 25.4.2.3, 25.4.3)

ψ_t = 1.0 (bars are not horizontal)

ψ_e = 1.0 (bar not epoxy coated)

ψ_s = 0.80 (bars are #6 or smaller)

λ = 1.0 (normal weight concrete)

$s / 2 = (12 \text{ in}) / 2 = 6 \text{ in}$

$\text{cover} + d_b / 2 = (2 \text{ in}) + (0.5 \text{ in}) / 2 = 2.25 \text{ in}$

c_b = 2.25 in (lesser of half spacing, ctr to surface)

K_{tr} = 0.0 (no transverse reinforcement)

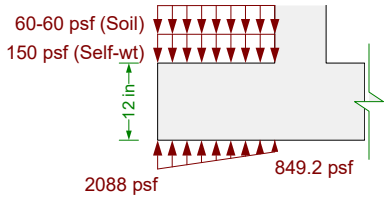
$$\frac{c_b + K_{tr}}{d_b} = \frac{(2.25 \text{ in}) + (0.0)}{(0.5 \text{ in})} = 4.50$$

$$l_d = \left(\frac{3.}{40} \frac{f_y}{\lambda \sqrt{F'_c}} \frac{\psi_t \, \psi_e \, \psi_s}{2.5} \right) d_b = \left[\frac{3.}{40} \frac{(60000 \text{ psi})}{(1.0) \sqrt{3000 \text{ psi}}} \frac{(1.0) \, (1.0) \, (0.80)}{2.5} \right] (0.5 \text{ in}) = 13.15 \text{ in}$$

Toe Checks [1.2D + 1.6L + 1.6H]

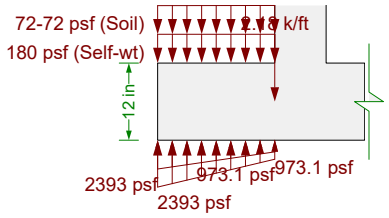
Toe Unfactored Loads

Unfactored Loads



Toe Factored Loads

1.2D + 1.6L + 1.6H



Controlling Moment

Design moment M_u for toe need not exceed moment at stem base:

M_{toe} = 1.93 ft·k / ft ≥ M_{stem} = 1.02 ft·k / ft

M_u = 1.02 ft·k / ft (stem base moment controls)

Flexure Check (ACI 318-14 13.3.2.1, 7.5.2.1, »22.3, »22.2, 7.5.1.1a)

$$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0.02 \text{ in}^2 / \text{in})(60000 \text{ psi})}{0.85 (3000 \text{ psi})} = 0.39 \text{ in}$$

$\phi M_n = \phi A_s f_y (d - a / 2)$ = (0.90) (0.02 in² / in) (60000 psi) [(8.75 in) - (0.39 in) / 2] = 7.7 ft·k / ft

ϕM_n = 7.7 ft·k / ft ≥ M_u = 1.02 ft·k / ft ✓

Shear Check (ACI 318-14 13.3.2.1, 7.5.3.1, »22.5.1, »22.5.5, 7.5.1.1b)

λ = 1.0 (normal weight concrete)

$V_c = 2 \lambda \sqrt{F'_c}\, d$ = 2 (1.0) $\sqrt{3000 \text{ psi}}$ (8.75 in) = 11.5 k / ft

$\phi V_n = \phi V_c$ = (0.750) (11.5 k / ft) = 8.63 k / ft

ϕV_n = 8.63 k / ft ≥ V_u = 1.41 k / ft ✓

Minimum Strain Check (ACI 318-14 13.3.2.1, 7.3.3.1)

β_1 = 0.850 ($F'_c \le 4000$ psi)

$$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0.02 \text{ in}^2 / \text{in})(60000 \text{ psi})}{0.85 (3000 \text{ psi})} = 0.39 \text{ in}$$

$$\varepsilon_t = 0.003 \left(\frac{d}{a / \beta_1} - 1 \right) = 0.003 \left[\frac{(8.75 \text{ in})}{(0.39 \text{ in}) / (0.850)} - 1 \right] = 0.0539$$

ε_t = 0.0539 ≥ 0.004 ✓

Minimum Steel Check (ACI 318-14 13.3.2.1, 9.6.1)

ϕM_n = 7.7 ft·k / ft ≥ (4 / 3) M_u = [4 / 3] (1.02 ft·k / ft) = 1.36 ft·k / ft

Check is waived per ACI 9.6.1.3 ✓

Shrinkage and Temperature Steel (ACI 318-14 13.2.8.1, 7.6.4.1, 24.4.3.2, 24.4.3.3)

$$\rho_{ST_prov} = \frac{A_{ST}}{t\, s_{ST}} = \frac{(0.4 \text{ in}^2 / \text{in})}{(12 \text{ in})(12 \text{ in})} = 0.0028$$

$$\rho_{ST_prov} = \frac{A_{ST}}{t\, s_{ST}} = \frac{(0.4 \text{ in}^2 / \text{in})}{(12 \text{ in})(12 \text{ in})} = 0.0028$$

$$\frac{0.0018 (60000)}{f_y} = \frac{0.0018 (60000)}{(60000 \text{ psi})} = 0.0018$$

ρ_{ST_min} = 0.0018

ρ_{ST_prov} = 0.0028 ≥ ρ_{ST_min} = 0.0018 ✓

18 inch limit governs

s_{ST_max} = 18 in

s_{ST} = 12 in ≤ s_{ST_max} = 18 in ✓

Development Check (ACI 318-14 13.2.8.1, 25.4.2.3, 25.4.10)

$$\frac{M_u}{\phi M_n} = \frac{(1.02 \text{ ft·k / ft})}{(7.7 \text{ ft·k / ft})} = 0.1326$$ (ratio to represent excess reinforcement)

ψ_e = 1.0 (uncoated hooked bars)

ψ_c = 0.70 (based on side cover and extension cover)

ψ_r = 1.0 (no confining reinforcement)

λ = 1.0 (normal weight concrete)

$$l_{dh} = \left(\frac{f_y \psi_e \psi_c \psi_r}{50 \lambda \sqrt{F'_c}} \right) d_b = \left[\frac{(60000 \text{ psi})(1.0)(0.70)(1.0)}{50 (1.0) \sqrt{3000 \text{ psi}}} \right] (0.5 \text{ in}) = 7.67 \text{ in}$$

Factoring l_{dh} by the excess reinforcement ratio (0.1326) per 25.4.10: l_{dh} = 1.02 in

8 d_b = 8 (0.5 in) = 4.0 (minimum limit, does not control)

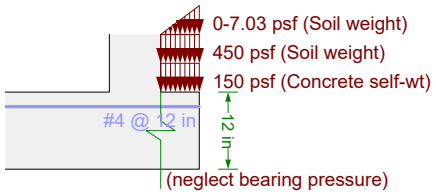
6 inch minimum controls

l_{dh_prov} = 6 in ≥ l_{dh} = 6 in ✓

Heel Checks [1.2D + 1.6L + 1.6H]

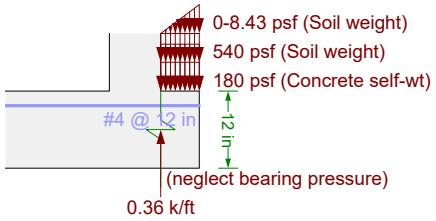
Heel Unfactored Loads

Unfactored Loads



Heel Factored Loads

1.2D + 1.6L + 1.6H



Controlling Moment

Design moment M_u for heel need not exceed moment at stem base:

M_{heel} = 0.09 ft·k / ft < M_{stem} = 1.02 ft·k / ft

M_u = 0.09 ft·k / ft (stem moment does not control)

Flexure Check (ACI 318-14 13.3.2.1, 7.5.2.1, »22.3, »22.2, 7.5.1.1a)

$$a = \frac{A_s f_y}{0.85 f'_c} = \frac{(0.02 \text{ in}^2 / \text{in})(60000 \text{ psi})}{0.85 (3000 \text{ psi})} = 0.39 \text{ in}$$

$$\phi M_n = \phi A_s f_y (d - a / 2) = (0.90)(0.02 \text{ in}^2 / \text{in})(60000 \text{ psi})[(9.75 \text{ in}) - (0.39 \text{ in}) / 2] = 8.6 \text{ ft·k / ft}$$

ϕM_n = 8.6 ft·k / ft ≥ M_u = 0.09 ft·k / ft ✓

Shear Check (ACI 318-14 13.3.2.1, 7.5.3.1, »22.5.1, »22.5.5, 7.5.1.1b)

λ = 1.0 (normal weight concrete)

$$V_c = 2 \lambda \sqrt{f'_c}\, d = 2 (1.0) \sqrt{3000 \text{ psi}}\, (9.75 \text{ in}) = 12.82 \text{ k / ft}$$

$$\phi V_n = \phi V_c = (0.750)(12.82 \text{ k / ft}) = 9.61 \text{ k / ft}$$

ϕV_n = 9.61 k / ft ≥ V_u = 0.36 k / ft ✓

Minimum Strain Check (ACI 318-14 13.3.2.1, 7.3.3.1)

β_1 = 0.850 ($f'_c \le 4000$ psi)

$$a = \frac{A_s f_y}{0.85 f'_c} = \frac{(0.02 \text{ in}^2 / \text{in})(60000 \text{ psi})}{0.85 (3000 \text{ psi})} = 0.39 \text{ in}$$

$$\varepsilon_t = 0.003 \left(\frac{d}{a / \beta_1} - 1 \right) = 0.003 \left[\frac{(9.75 \text{ in})}{(0.39 \text{ in}) / (0.850)} - 1 \right] = 0.0604$$

ε_t = 0.0604 ≥ 0.004 ✓

Minimum Steel Check (ACI 318-14 13.3.2.1, 9.6.1)

ϕM_n = 8.6 ft·k / ft ≥ (4 / 3) M_u = [4 / 3] (0.09 ft·k / ft) = 0.12 ft·k / ft

Check is waived per ACI 9.6.1.3 ✓

Shrinkage and Temperature Steel (ACI 318-14 13.2.8.1, 7.6.4.1, 24.4.3.2, 24.4.3.3)

$$\rho_{ST_prov} = \frac{A_{ST}}{t\, s_{ST}} = \frac{(0.4 \text{ in}^2 / \text{in})}{(12 \text{ in})(12 \text{ in})} = 0.0028$$

$$\rho_{ST_prov} = \frac{A_{ST}}{t\, s_{ST}} = \frac{(0.4 \text{ in}^2 / \text{in})}{(12 \text{ in})(12 \text{ in})} = 0.0028$$

$$\frac{0.0018 (60000)}{f_y} = \frac{0.0018 (60000)}{(60000 \text{ psi})} = 0.0018$$

ρ_{ST_min} = 0.0018

ρ_{ST_prov} = 0.0028 ≥ ρ_{ST_min} = 0.0018 ✓

18 inch limit governs

s_{ST_max} = 18 in

s_{ST} = 12 in ≤ s_{ST_max} = 18 in ✓

Development Check (ACI 318-14 13.2.8.1, 25.4.2.3, 25.4.10)

$$\frac{M_u}{\phi M_n} = \frac{(0.09 \text{ ft·k / ft})}{(8.6 \text{ ft·k / ft})} = 0.0105$$ (ratio to represent excess reinforcement)

ψ_t = 1.0 (12 inches or less cast below - 9.50 inches)

ψ_e = 1.0 (bar not epoxy coated)

ψ_s = 0.80 (bars are #6 or smaller)

λ = 1.0 (normal weight concrete)

s / 2 = (12 in) / 2 = 6 in

cover + d_b / 2 = (2 in) + (0.5 in) / 2 = 2.25 in

c_b = 2.25 in (lesser of half spacing, ctr to surface)

K_{tr} = 0.0 (no transverse reinforcement)

$$\frac{c_b + K_{tr}}{d_b} = \frac{(2.25 \text{ in}) + (0.0)}{(0.5 \text{ in})} = 4.50$$

$$l_d = \left(\frac{3.}{40} \frac{f_y}{\lambda \sqrt{f'_c}} \frac{\psi_t \psi_e \psi_s}{2.5} \right) d_b = \left[\frac{3.}{40} \frac{(60000 \text{ psi})}{(1.0) \sqrt{3000 \text{ psi}}} \frac{(1.0)(1.0)(0.80)}{2.5} \right] (0.5 \text{ in}) = 13.15 \text{ in}$$

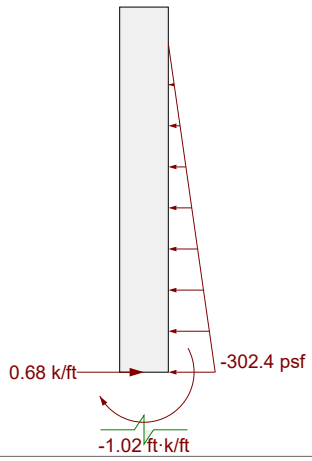
Factoring l_d by the excess reinforcement ratio (0.0105) per 25.4.10: l_d = 0.14 in

12 inch minimum controls

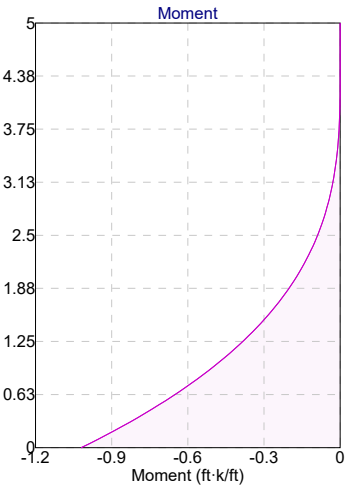
l_{d_prov} = 24.24 in ≥ l_d = 12 in ✓

Stem Forces [1.2D + 1.6L + 1.6H]

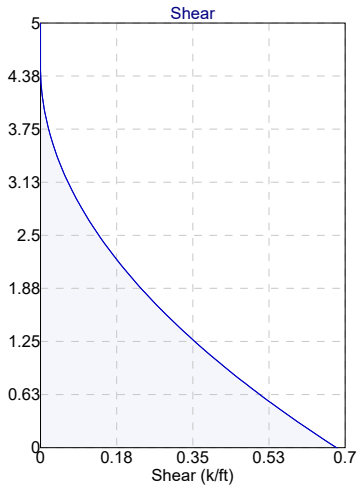
Stem Internal Forces



Stem Internal Forces



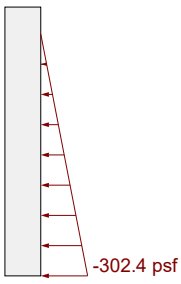
Stem Internal Forces



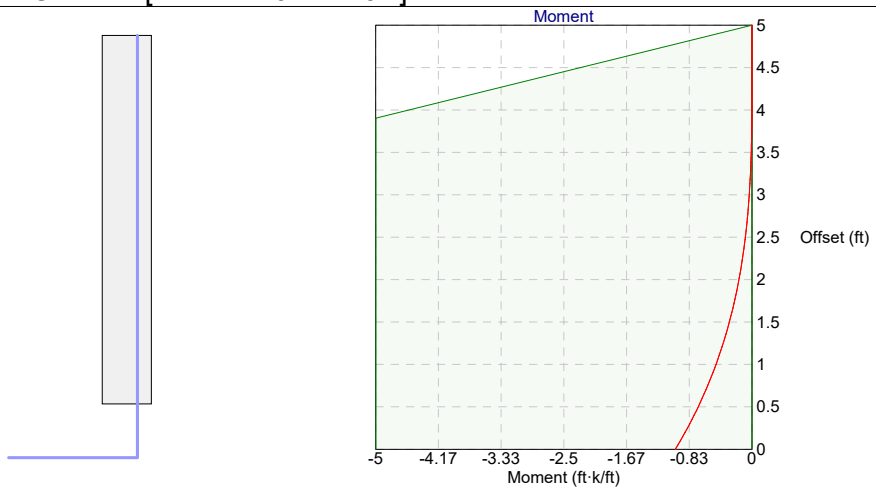
Stem Joint Force Transfer

Location	Force
@ stem base	0.68 k/ft

Stem Internal Forces



Stem Moment Checks [1.2D + 1.6L + 1.6H]



Check (ACI 318-14 11.5.5.1b) @ 0 ft from base

ϕM_n = 5 ft·k / ft ≥ M_u = 1.02 ft·k / ft ✓

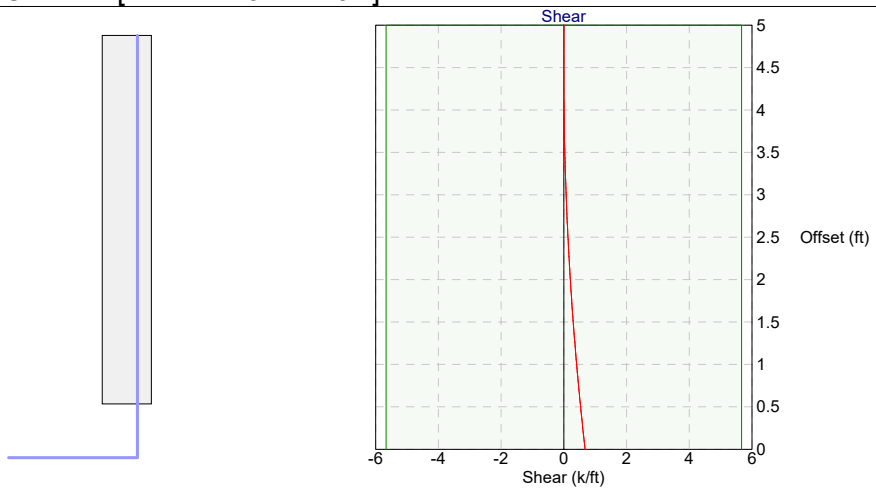
Check (ACI 318-14 11.5.5.1b) @ 3.9 ft from base

ϕM_n = 5 ft·k / ft ≥ M_u = 0 ft·k / ft ✓

Check (ACI 318-14 11.5.5.1b) @ 3.94 ft from base

ϕM_n = 4.84 ft·k / ft ≥ M_u = 0 ft·k / ft ✓

Stem Shear Checks [1.2D + 1.6L + 1.6H]



Shear Check (ACI 318-14 11.5.5.1c) @ 0 ft from base

ϕV_n = 5.67 k / ft ≥ V_u = 0.68 k / ft ✓

Stem Miscellaneous Checks [1.2D + 1.6L + 1.6H]

Minimum Steel Check (ACI 318-14 9.6.1) @ 0 ft from base [Stem in negative flexure]

$\phi M_n = 5$ ft·k / ft $\geq (4 / 3) M_u = [4 / 3] (1.02$ ft·k / ft$) = 1.36$ ft·k / ft

Check is waived per ACI 9.6.1.3 ✓

Minimum Steel Check (ACI 318-14 9.6.1) @ 5 ft from base [Stem in negative flexure]

$\phi M_n = 0$ ft·k / ft $\geq (4 / 3) M_u = [4 / 3] (0$ ft·k / ft$) = 0$ ft·k / ft

Check is waived per ACI 9.6.1.3 ✓

Maximum Steel Check (ACI 318-14 9.3.3.1) @ 0 ft from base [Stem in negative flexure]

$\beta_1 = 0.850$ ($F'_c \leq 4000$ psi)

$$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0.02 \text{ in}^2 / \text{in}) (60000 \text{ psi})}{0.85 (3000 \text{ psi})} = 0.39 \text{ in}$$

$$\varepsilon_t = 0.003 \left(\frac{d}{a / \beta_1} - 1 \right) = 0.003 \left[\frac{(5.75 \text{ in})}{(0.39 \text{ in}) / (0.850)} - 1 \right] = 0.0344$$

$\varepsilon_t = 0.0344 \geq 0.004$ ✓

Maximum Steel Check (ACI 318-14 9.3.3.1) @ 5 ft from base [Stem in negative flexure]

$\beta_1 = 0.850$ ($F'_c \leq 4000$ psi)

$$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0.02 \text{ in}^2 / \text{in}) (60000 \text{ psi})}{0.85 (3000 \text{ psi})} = 0.39 \text{ in}$$

$$\varepsilon_t = 0.003 \left(\frac{d}{a / \beta_1} - 1 \right) = 0.003 \left[\frac{(5.75 \text{ in})}{(0.39 \text{ in}) / (0.850)} - 1 \right] = 0.0344$$

$\varepsilon_t = 0.0344 \geq 0.004$ ✓

Wall Horizontal Steel (ACI 318-14 11.6.1, 11.7.3)

$$\rho_t = \frac{A_{s_horz} / s_{horz}}{t} = \frac{(0.2 \text{ in}^2) / (12 \text{ in})}{(8 \text{ in})} = 0.0021$$

$\rho_{t_min} = 0.0020$ (bars No. 5 or less, not less than 60 ksi)

$\rho_t = 0.0021 \geq \rho_{t_min} = 0.0020$ ✓

$3 h = 3 (8 \text{ in}) = 24$ in

18 inch limit governs

$s_{horz} = 12$ in $\leq s_{horz_max} = 18$ in ✓

Development Check (ACI 318-14 11.7.1.2, 25.4.2.3, 25.4.10)

$\frac{M_u}{\phi M_n} = \frac{(1.02 \text{ ft·k / ft})}{(5 \text{ ft·k / ft})} = 0.2042$ (ratio to represent excess reinforcement)

$\psi_e = 1.0$ (uncoated hooked bars)

$\psi_c = 0.70$ (based on side cover and extension cover)

$\psi_r = 1.0$ (no confining reinforcement)

$\lambda = 1.0$ (normal weight concrete)

$$l_{dh} = \left(\frac{f_y \psi_e \psi_c \psi_r}{50 \lambda \sqrt{F'_c}} \right) d_b = \left[\frac{(60000 \text{ psi}) (1.0) (0.70) (1.0)}{50 (1.0) \sqrt{3000 \text{ psi}}} \right] (0.5 \text{ in}) = 7.67 \text{ in}$$

Factoring l_{dh} by the excess reinforcement ratio (0.2042) per 25.4.10: $l_{dh} = 1.57$ in

$8 d_b = 8 (0.5 \text{ in}) = 4.0$ (minimum limit, does not control)

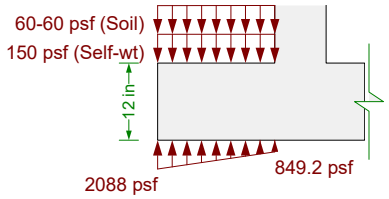
6 inch minimum controls

$l_{dh_prov} = 9$ in $\geq l_{dh} = 6$ in ✓

Toe Checks [1.2D + 0.5L + 1.6H + 1.0E]

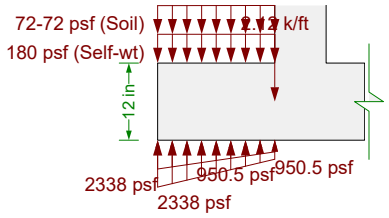
Toe Unfactored Loads

Unfactored Loads



Toe Factored Loads

1.2D + 0.5L + 1.6H + 1.0E



Controlling Moment

Design moment M_u for toe need not exceed moment at stem base:

M_{toe} = 1.88 ft·k / ft ≥ M_{stem} = 1.79 ft·k / ft

M_u = 1.79 ft·k / ft (stem base moment controls)

Flexure Check (ACI 318-14 13.3.2.1, 7.5.2.1, »22.3, »22.2, 7.5.1.1a)

$$a = \frac{A_s f_y}{0.85 f'_c} = \frac{(0.02 \text{ in}^2 / \text{in})(60000 \text{ psi})}{0.85 (3000 \text{ psi})} = 0.39 \text{ in}$$

$\phi M_n = \phi A_s f_y (d - a / 2)$ = (0.90) (0.02 in² / in) (60000 psi) [(8.75 in) - (0.39 in) / 2] = 7.7 ft·k / ft

ϕM_n = 7.7 ft·k / ft ≥ M_u = 1.79 ft·k / ft ✓

Shear Check (ACI 318-14 13.3.2.1, 7.5.3.1, »22.5.1, »22.5.5, 7.5.1.1b)

λ = 1.0 (normal weight concrete)

$V_c = 2 \lambda \sqrt{f'_c}\, d$ = 2 (1.0) $\sqrt{3000 \text{ psi}}$ (8.75 in) = 11.5 k / ft

$\phi V_n = \phi V_c$ = (0.750) (11.5 k / ft) = 8.63 k / ft

ϕV_n = 8.63 k / ft ≥ V_u = 1.37 k / ft ✓

Minimum Strain Check (ACI 318-14 13.3.2.1, 7.3.3.1)

β_1 = 0.850 ($f'_c \le 4000$ psi)

$$a = \frac{A_s f_y}{0.85 f'_c} = \frac{(0.02 \text{ in}^2 / \text{in})(60000 \text{ psi})}{0.85 (3000 \text{ psi})} = 0.39 \text{ in}$$

$$\varepsilon_t = 0.003 \left(\frac{d}{a / \beta_1} - 1 \right) = 0.003 \left[\frac{(8.75 \text{ in})}{(0.39 \text{ in}) / (0.850)} - 1 \right] = 0.0539$$

ε_t = 0.0539 ≥ 0.004 ✓

Minimum Steel Check (ACI 318-14 13.3.2.1, 9.6.1)

ϕM_n = 7.7 ft·k / ft ≥ (4 / 3) M_u = [4 / 3] (1.79 ft·k / ft) = 2.39 ft·k / ft

Check is waived per ACI 9.6.1.3 ✓

Shrinkage and Temperature Steel (ACI 318-14 13.2.8.1, 7.6.4.1, 24.4.3.2, 24.4.3.3)

$$\rho_{ST_prov} = \frac{A_{ST}}{t\, s_{ST}} = \frac{(0.4 \text{ in}^2 / \text{in})}{(12 \text{ in})(12 \text{ in})} = 0.0028$$

$$\rho_{ST_prov} = \frac{A_{ST}}{t\, s_{ST}} = \frac{(0.4 \text{ in}^2 / \text{in})}{(12 \text{ in})(12 \text{ in})} = 0.0028$$

$$\frac{0.0018 (60000)}{f_y} = \frac{0.0018 (60000)}{(60000 \text{ psi})} = 0.0018$$

ρ_{ST_min} = 0.0018

ρ_{ST_prov} = 0.0028 ≥ ρ_{ST_min} = 0.0018 ✓

18 inch limit governs

s_{ST_max} = 18 in

s_{ST} = 12 in ≤ s_{ST_max} = 18 in ✓

Development Check (ACI 318-14 13.2.8.1, 25.4.2.3, 25.4.10)

$$\frac{M_u}{\phi M_n} = \frac{(1.79 \text{ ft·k / ft})}{(7.7 \text{ ft·k / ft})} = 0.2326$$ (ratio to represent excess reinforcement)

ψ_e = 1.0 (uncoated hooked bars)

ψ_c = 0.70 (based on side cover and extension cover)

ψ_r = 1.0 (no confining reinforcement)

λ = 1.0 (normal weight concrete)

$$l_{dh} = \left(\frac{f_y \psi_e \psi_c \psi_r}{50 \lambda \sqrt{f'_c}} \right) d_b = \left[\frac{(60000 \text{ psi})(1.0)(0.70)(1.0)}{50 (1.0) \sqrt{3000 \text{ psi}}} \right] (0.5 \text{ in}) = 7.67 \text{ in}$$

Factoring l_{dh} by the excess reinforcement ratio (0.2326) per 25.4.10: l_{dh} = 1.78 in

8 d_b = 8 (0.5 in) = 4.0 (minimum limit, does not control)

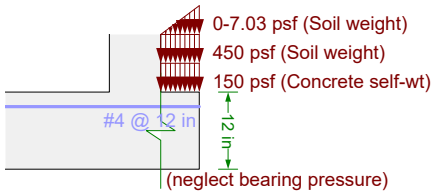
6 inch minimum controls

l_{dh_prov} = 6 in ≥ l_{dh} = 6 in ✓

Heel Checks [1.2D + 0.5L + 1.6H + 1.0E]

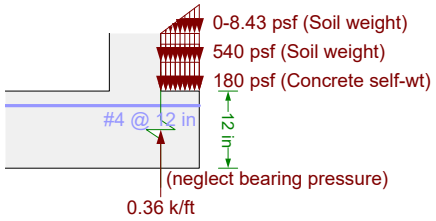
Heel Unfactored Loads

Unfactored Loads



Heel Factored Loads

1.2D + 0.5L + 1.6H + 1.0E



Controlling Moment

Design moment M_u for heel need not exceed moment at stem base:

M_{heel} = 0.09 ft·k / ft < M_{stem} = 1.79 ft·k / ft

M_u = 0.09 ft·k / ft (stem moment does not control)

Flexure Check (ACI 318-14 13.3.2.1, 7.5.2.1, »22.3, »22.2, 7.5.1.1a)

$$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0.02 \text{ in}^2 / \text{in})(60000 \text{ psi})}{0.85 (3000 \text{ psi})} = 0.39 \text{ in}$$

$$\phi M_n = \phi A_s f_y (d - a / 2) = (0.90)(0.02 \text{ in}^2 / \text{in})(60000 \text{ psi})[(9.75 \text{ in}) - (0.39 \text{ in}) / 2] = 8.6 \text{ ft·k / ft}$$

ϕM_n = 8.6 ft·k / ft ≥ M_u = 0.09 ft·k / ft ✓

Shear Check (ACI 318-14 13.3.2.1, 7.5.3.1, »22.5.1, »22.5.5, 7.5.1.1b)

λ = 1.0 (normal weight concrete)

$$V_c = 2 \lambda \sqrt{F'_c}\, d = 2 (1.0) \sqrt{3000 \text{ psi}}\, (9.75 \text{ in}) = 12.82 \text{ k / ft}$$

$$\phi V_n = \phi V_c = (0.750)(12.82 \text{ k / ft}) = 9.61 \text{ k / ft}$$

ϕV_n = 9.61 k / ft ≥ V_u = 0.36 k / ft ✓

Minimum Strain Check (ACI 318-14 13.3.2.1, 7.3.3.1)

β_1 = 0.850 ($F'_c \le 4000$ psi)

$$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0.02 \text{ in}^2 / \text{in})(60000 \text{ psi})}{0.85 (3000 \text{ psi})} = 0.39 \text{ in}$$

$$\varepsilon_t = 0.003 \left(\frac{d}{a / \beta_1} - 1 \right) = 0.003 \left[\frac{(9.75 \text{ in})}{(0.39 \text{ in}) / (0.850)} - 1 \right] = 0.0604$$

ε_t = 0.0604 ≥ 0.004 ✓

Minimum Steel Check (ACI 318-14 13.3.2.1, 9.6.1)

ϕM_n = 8.6 ft·k / ft ≥ (4 / 3) M_u = [4 / 3] (0.09 ft·k / ft) = 0.12 ft·k / ft

Check is waived per ACI 9.6.1.3 ✓

Shrinkage and Temperature Steel (ACI 318-14 13.2.8.1, 7.6.4.1, 24.4.3.2, 24.4.3.3)

$$\rho_{ST_prov} = \frac{A_{ST}}{t\, s_{ST}} = \frac{(0.4 \text{ in}^2 / \text{in})}{(12 \text{ in})(12 \text{ in})} = 0.0028$$

$$\rho_{ST_prov} = \frac{A_{ST}}{t\, s_{ST}} = \frac{(0.4 \text{ in}^2 / \text{in})}{(12 \text{ in})(12 \text{ in})} = 0.0028$$

$$\frac{0.0018 (60000)}{f_y} = \frac{0.0018 (60000)}{(60000 \text{ psi})} = 0.0018$$

ρ_{ST_min} = 0.0018

ρ_{ST_prov} = 0.0028 ≥ ρ_{ST_min} = 0.0018 ✓

18 inch limit governs

s_{ST_max} = 18 in

s_{ST} = 12 in ≤ s_{ST_max} = 18 in ✓

Development Check (ACI 318-14 13.2.8.1, 25.4.2.3, 25.4.10)

$$\frac{M_u}{\phi M_n} = \frac{(0.09 \text{ ft·k / ft})}{(8.6 \text{ ft·k / ft})} = 0.0105$$ (ratio to represent excess reinforcement)

ψ_t = 1.0 (12 inches or less cast below - 9.50 inches)

ψ_e = 1.0 (bar not epoxy coated)

ψ_s = 0.80 (bars are #6 or smaller)

λ = 1.0 (normal weight concrete)

s / 2 = (12 in) / 2 = 6 in

cover + d_b / 2 = (2 in) + (0.5 in) / 2 = 2.25 in

c_b = 2.25 in (lesser of half spacing, ctr to surface)

K_{tr} = 0.0 (no transverse reinforcement)

$$\frac{c_b + K_{tr}}{d_b} = \frac{(2.25 \text{ in}) + (0.0)}{(0.5 \text{ in})} = 4.50$$

$$l_d = \left(\frac{3.}{40} \frac{f_y}{\lambda \sqrt{F'_c}} \frac{\psi_t \psi_e \psi_s}{2.5} \right) d_b = \left[\frac{3.}{40} \frac{(60000 \text{ psi})}{(1.0) \sqrt{3000 \text{ psi}}} \frac{(1.0)(1.0)(0.80)}{2.5} \right] (0.5 \text{ in}) = 13.15 \text{ in}$$

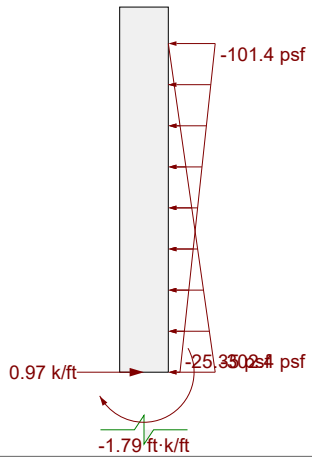
Factoring l_d by the excess reinforcement ratio (0.0105) per 25.4.10: l_d = 0.14 in

12 inch minimum controls

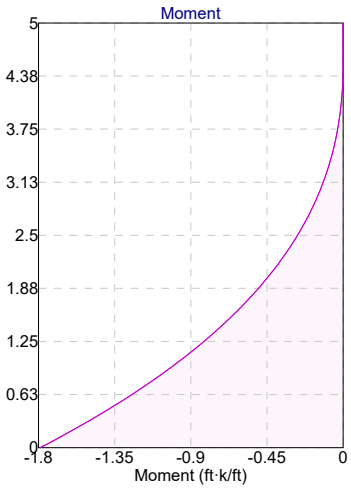
l_{d_prov} = 24.24 in ≥ l_d = 12 in ✓

Stem Forces [1.2D + 0.5L + 1.6H + 1.0E]

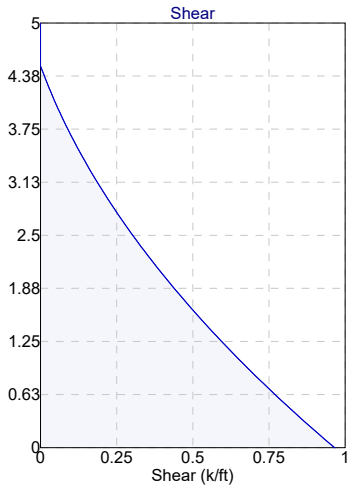
Stem Internal Forces



Stem Internal Forces



Stem Internal Forces



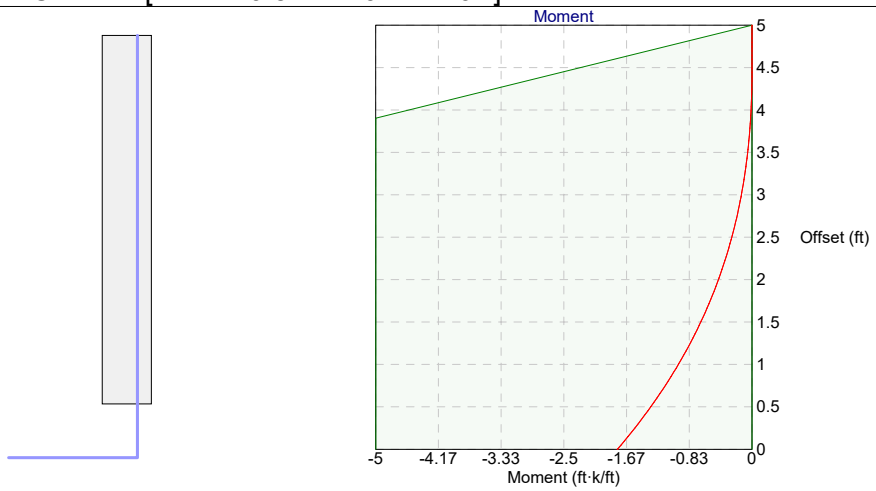
Stem Joint Force Transfer

Location	Force
@ stem base	0.97 k/ft

Stem Internal Forces



Stem Moment Checks [1.2D + 0.5L + 1.6H + 1.0E]



Check (ACI 318-14 11.5.5.1b) @ 0 ft from base

ϕM_n = 5 ft·k / ft ≥ M_u = 1.79 ft·k / ft ✓

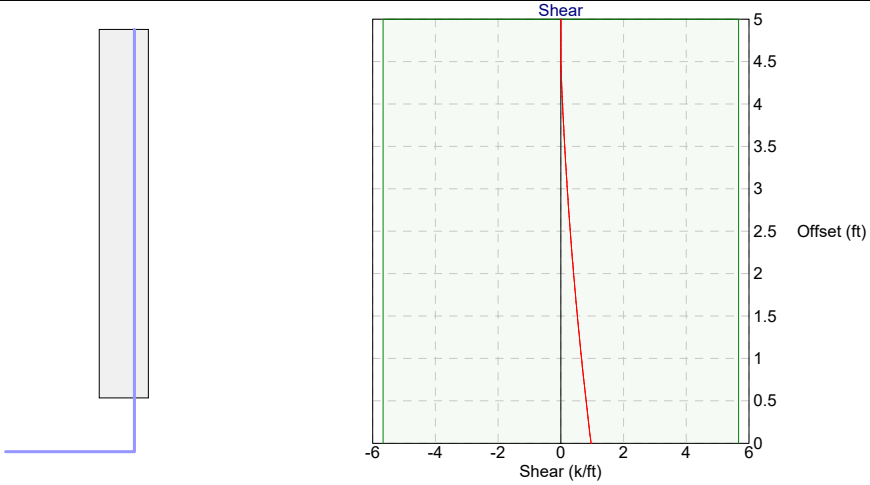
Check (ACI 318-14 11.5.5.1b) @ 3.9 ft from base

ϕM_n = 5 ft·k / ft ≥ M_u = 0.02 ft·k / ft ✓

Check (ACI 318-14 11.5.5.1b) @ 3.94 ft from base

ϕM_n = 4.84 ft·k / ft ≥ M_u = 0.02 ft·k / ft ✓

Stem Shear Checks [1.2D + 0.5L + 1.6H + 1.0E]



Shear Check (ACI 318-14 11.5.5.1c) @ 0 ft from base

ϕV_n = 5.67 k / ft ≥ V_u = 0.97 k / ft ✓

Stem Miscellaneous Checks [1.2D + 0.5L + 1.6H + 1.0E]

Minimum Steel Check (ACI 318-14 9.6.1) @ 0 ft from base [Stem in negative flexure]

ϕM_n = 5 ft·k / ft ≥ (4 / 3) M_u = [4 / 3] (1.79 ft·k / ft) = 2.39 ft·k / ft

Check is waived per ACI 9.6.1.3 ✓

Minimum Steel Check (ACI 318-14 9.6.1) @ 5 ft from base [Stem in negative flexure]

ϕM_n = 0 ft·k / ft ≥ (4 / 3) M_u = [4 / 3] (0 ft·k / ft) = 0 ft·k / ft

Check is waived per ACI 9.6.1.3 ✓

Maximum Steel Check (ACI 318-14 9.3.3.1) @ 0 ft from base [Stem in negative flexure]

β_1 = 0.850 ($F'_c \le 4000$ psi)

$$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0.02 \text{ in}^2 / \text{in})(60000 \text{ psi})}{0.85 (3000 \text{ psi})} = 0.39 \text{ in}$$

$$\varepsilon_t = 0.003 \left(\frac{d}{a / \beta_1} - 1 \right) = 0.003 \left[\frac{(5.75 \text{ in})}{(0.39 \text{ in}) / (0.850)} - 1 \right] = 0.0344$$

ε_t = 0.0344 ≥ 0.004 ✓

Maximum Steel Check (ACI 318-14 9.3.3.1) @ 5 ft from base [Stem in negative flexure]

β_1 = 0.850 ($F'_c \le 4000$ psi)

$$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0.02 \text{ in}^2 / \text{in})(60000 \text{ psi})}{0.85 (3000 \text{ psi})} = 0.39 \text{ in}$$

$$\varepsilon_t = 0.003 \left(\frac{d}{a / \beta_1} - 1 \right) = 0.003 \left[\frac{(5.75 \text{ in})}{(0.39 \text{ in}) / (0.850)} - 1 \right] = 0.0344$$

ε_t = 0.0344 ≥ 0.004 ✓

Wall Horizontal Steel (ACI 318-14 11.6.1, 11.7.3)

$$\rho_t = \frac{A_{s_horz} / s_{horz}}{t} = \frac{(0.2 \text{ in}^2) / (12 \text{ in})}{(8 \text{ in})} = 0.0021$$

ρ_{t_min} = 0.0020 (bars No. 5 or less, not less than 60 ksi)

ρ_t = 0.0021 ≥ ρ_{t_min} = 0.0020 ✓

3 h = 3 (8 in) = 24 in

18 inch limit governs

s_{horz} = 12 in ≤ s_{horz_max} = 18 in ✓

Development Check (ACI 318-14 11.7.1.2, 25.4.2.3, 25.4.10)

$$\frac{M_u}{\phi M_n} = \frac{(1.79 \text{ ft·k / ft})}{(5 \text{ ft·k / ft})} = 0.3582$$ (ratio to represent excess reinforcement)

ψ_e = 1.0 (uncoated hooked bars)

ψ_c = 0.70 (based on side cover and extension cover)

ψ_r = 1.0 (no confining reinforcement)

λ = 1.0 (normal weight concrete)

$$l_{dh} = \left(\frac{f_y \psi_e \psi_c \psi_r}{50 \lambda \sqrt{F'_c}} \right) d_b = \left[\frac{(60000 \text{ psi})(1.0)(0.70)(1.0)}{50 (1.0) \sqrt{3000 \text{ psi}}} \right] (0.5 \text{ in}) = 7.67 \text{ in}$$

Factoring l_{dh} by the excess reinforcement ratio (0.3582) per 25.4.10: l_{dh} = 2.75 in

8 d_b = 8 (0.5 in) = 4.0 (minimum limit, does not control)

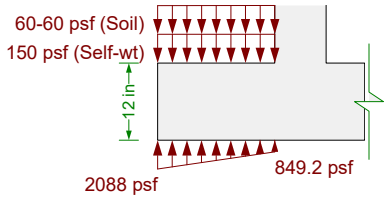
6 inch minimum controls

l_{dh_prov} = 9 in ≥ l_{dh} = 6 in ✓

Toe Checks [1.4D]

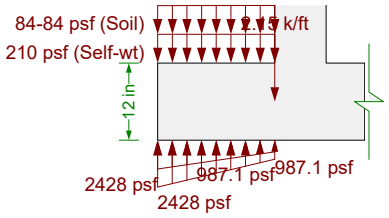
Toe Unfactored Loads

Unfactored Loads



Toe Factored Loads

1.4D



Controlling Moment

Design moment M_u for toe need not exceed moment at stem base:

M_{toe} = 1.91 ft·k / ft ≥ M_{stem} = - 0 ft·k / ft

M_u = - 0 ft·k / ft (stem base moment controls)

Flexure Check (ACI 318-14 13.3.2.1, 7.5.2.1, »22.3, »22.2, 7.5.1.1a)

$$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0.02 \text{ in}^2 / \text{in})(60000 \text{ psi})}{0.85 (3000 \text{ psi})} = 0.39 \text{ in}$$

$$\phi M_n = \phi A_s f_y (d - a / 2) = (0.90)(0.02 \text{ in}^2 / \text{in})(60000 \text{ psi})[(8.75 \text{ in}) - (0.39 \text{ in}) / 2] = 7.7 \text{ ft·k / ft}$$

ϕM_n = 7.7 ft·k / ft ≥ M_u = - 0 ft·k / ft ✓

Shear Check (ACI 318-14 13.3.2.1, 7.5.3.1, »22.5.1, »22.5.5, 7.5.1.1b)

λ = 1.0 (normal weight concrete)

$$V_c = 2 \lambda \sqrt{F'_c} d = 2 (1.0) \sqrt{3000 \text{ psi}} (8.75 \text{ in}) = 11.5 \text{ k / ft}$$

$\phi V_n = \phi V_c$ = (0.750)(11.5 k / ft) = 8.63 k / ft

ϕV_n = 8.63 k / ft ≥ V_u = 1.4 k / ft ✓

Minimum Strain Check (ACI 318-14 13.3.2.1, 7.3.3.1)

β_1 = 0.850 ($F'_c \le 4000$ psi)

$$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0.02 \text{ in}^2 / \text{in})(60000 \text{ psi})}{0.85 (3000 \text{ psi})} = 0.39 \text{ in}$$

$$\varepsilon_t = 0.003 \left(\frac{d}{a / \beta_1} - 1 \right) = 0.003 \left[\frac{(8.75 \text{ in})}{(0.39 \text{ in}) / (0.850)} - 1 \right] = 0.0539$$

ε_t = 0.0539 ≥ 0.004 ✓

Minimum Steel Check (ACI 318-14 13.3.2.1, 9.6.1)

ϕM_n = 7.7 ft·k / ft ≥ (4 / 3) M_u = [4 / 3](- 0 ft·k / ft) = - 0 ft·k / ft

Check is waived per ACI 9.6.1.3 ✓

Shrinkage and Temperature Steel (ACI 318-14 13.2.8.1, 7.6.4.1, 24.4.3.2, 24.4.3.3)

$$\rho_{ST_prov} = \frac{A_{ST}}{t \, s_{ST}} = \frac{(0.4 \text{ in}^2 / \text{in})}{(12 \text{ in})(12 \text{ in})} = 0.0028$$

$$\rho_{ST_prov} = \frac{A_{ST}}{t \, s_{ST}} = \frac{(0.4 \text{ in}^2 / \text{in})}{(12 \text{ in})(12 \text{ in})} = 0.0028$$

$$\frac{0.0018 (60000)}{f_y} = \frac{0.0018 (60000)}{(60000 \text{ psi})} = 0.0018$$

ρ_{ST_min} = 0.0018

ρ_{ST_prov} = 0.0028 ≥ ρ_{ST_min} = 0.0018 ✓

18 inch limit governs

s_{ST_max} = 18 in

s_{ST} = 12 in ≤ s_{ST_max} = 18 in ✓

Development Check (ACI 318-14 13.2.8.1, 25.4.2.3, 25.4.10)

$$\frac{M_u}{\phi M_n} = \frac{(- 0 \text{ ft·k / ft})}{(7.7 \text{ ft·k / ft})} = - 0.0$$ (ratio to represent excess reinforcement)

ψ_e = 1.0 (uncoated hooked bars)

ψ_c = 0.70 (based on side cover and extension cover)

ψ_r = 1.0 (no confining reinforcement)

λ = 1.0 (normal weight concrete)

$$l_{dh} = \left(\frac{f_y \psi_e \psi_c \psi_r}{50 \lambda \sqrt{F'_c}} \right) d_b = \left[\frac{(60000 \text{ psi})(1.0)(0.70)(1.0)}{50 (1.0) \sqrt{3000 \text{ psi}}} \right] (0.5 \text{ in}) = 7.67 \text{ in}$$

Factoring l_{dh} by the excess reinforcement ratio (- 0.0000) per 25.4.10: l_{dh} = - 0 in

8 d_b = 8 (0.5 in) = 4.0 (minimum limit, does not control)

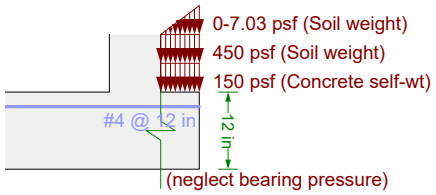
6 inch minimum controls

l_{dh_prov} = 6 in ≥ l_{dh} = 6 in ✓

Heel Checks [1.4D]

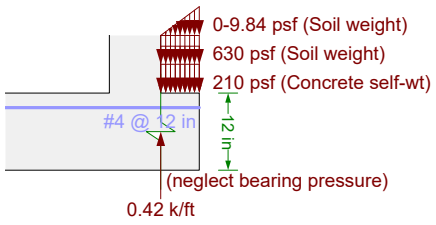
Heel Unfactored Loads

Unfactored Loads



Heel Factored Loads

1.4D



Controlling Moment

Design moment M_u for heel need not exceed moment at stem base:

M_{heel} = 0.11 ft·k / ft ≥ M_{stem} = - 0 ft·k / ft

M_u = - 0 ft·k / ft (stem base moment controls)

Flexure Check (ACI 318-14 13.3.2.1, 7.5.2.1, »22.3, »22.2, 7.5.1.1a)

$$a = \frac{A_s f_y}{0.85 f'_c} = \frac{(0.02 \text{ in}^2 / \text{in})(60000 \text{ psi})}{0.85 (3000 \text{ psi})} = 0.39 \text{ in}$$

$$\phi M_n = \phi A_s f_y (d - a / 2) = (0.90)(0.02 \text{ in}^2 / \text{in})(60000 \text{ psi})[(9.75 \text{ in}) - (0.39 \text{ in}) / 2] = 8.6 \text{ ft·k / ft}$$

ϕM_n = 8.6 ft·k / ft ≥ M_u = - 0 ft·k / ft ✓

Shear Check (ACI 318-14 13.3.2.1, 7.5.3.1, »22.5.1, »22.5.5, 7.5.1.1b)

λ = 1.0 (normal weight concrete)

$$V_c = 2 \lambda \sqrt{f'_c}\, d = 2 (1.0) \sqrt{3000 \text{ psi}}\, (9.75 \text{ in}) = 12.82 \text{ k / ft}$$

$$\phi V_n = \phi V_c = (0.750)(12.82 \text{ k / ft}) = 9.61 \text{ k / ft}$$

ϕV_n = 9.61 k / ft ≥ V_u = 0.42 k / ft ✓

Minimum Strain Check (ACI 318-14 13.3.2.1, 7.3.3.1)

β_1 = 0.850 ($f'_c \le 4000$ psi)

$$a = \frac{A_s f_y}{0.85 f'_c} = \frac{(0.02 \text{ in}^2 / \text{in})(60000 \text{ psi})}{0.85 (3000 \text{ psi})} = 0.39 \text{ in}$$

$$\varepsilon_t = 0.003 \left(\frac{d}{a / \beta_1} - 1 \right) = 0.003 \left[\frac{(9.75 \text{ in})}{(0.39 \text{ in}) / (0.850)} - 1 \right] = 0.0604$$

ε_t = 0.0604 ≥ 0.004 ✓

Minimum Steel Check (ACI 318-14 13.3.2.1, 9.6.1)

ϕM_n = 8.6 ft·k / ft ≥ (4 / 3) M_u = [4 / 3] (- 0 ft·k / ft) = - 0 ft·k / ft

Check is waived per ACI 9.6.1.3 ✓

Shrinkage and Temperature Steel (ACI 318-14 13.2.8.1, 7.6.4.1, 24.4.3.2, 24.4.3.3)

$$\rho_{ST_prov} = \frac{A_{ST}}{t\, s_{ST}} = \frac{(0.4 \text{ in}^2 / \text{in})}{(12 \text{ in})(12 \text{ in})} = 0.0028$$

$$\rho_{ST_prov} = \frac{A_{ST}}{t\, s_{ST}} = \frac{(0.4 \text{ in}^2 / \text{in})}{(12 \text{ in})(12 \text{ in})} = 0.0028$$

$$\frac{0.0018 (60000)}{f_y} = \frac{0.0018 (60000)}{(60000 \text{ psi})} = 0.0018$$

ρ_{ST_min} = 0.0018

ρ_{ST_prov} = 0.0028 ≥ ρ_{ST_min} = 0.0018 ✓

18 inch limit governs

s_{ST_max} = 18 in

s_{ST} = 12 in ≤ s_{ST_max} = 18 in ✓

Development Check (ACI 318-14 13.2.8.1, 25.4.2.3, 25.4.10)

$$\frac{M_u}{\phi M_n} = \frac{(- 0 \text{ ft·k / ft})}{(8.6 \text{ ft·k / ft})} = - 0.0$$ (ratio to represent excess reinforcement)

ψ_t = 1.0 (12 inches or less cast below - 9.50 inches)

ψ_e = 1.0 (bar not epoxy coated)

ψ_s = 0.80 (bars are #6 or smaller)

λ = 1.0 (normal weight concrete)

s / 2 = (12 in) / 2 = 6 in

cover + d_b / 2 = (2 in) + (0.5 in) / 2 = 2.25 in

c_b = 2.25 in (lesser of half spacing, ctr to surface)

K_{tr} = 0.0 (no transverse reinforcement)

$$\frac{c_b + K_{tr}}{d_b} = \frac{(2.25 \text{ in}) + (0.0)}{(0.5 \text{ in})} = 4.50$$

$$l_d = \left(\frac{3.}{40} \frac{f_y}{\lambda \sqrt{f'_c}} \frac{\psi_t \psi_e \psi_s}{2.5} \right) d_b = \left[\frac{3.}{40} \frac{(60000 \text{ psi})}{(1.0) \sqrt{3000 \text{ psi}}} \frac{(1.0)(1.0)(0.80)}{2.5} \right] (0.5 \text{ in}) = 13.15 \text{ in}$$

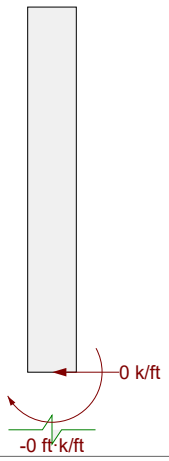
Factoring l_d by the excess reinforcement ratio (- 0.0000) per 25.4.10: l_d = - 0 in

12 inch minimum controls

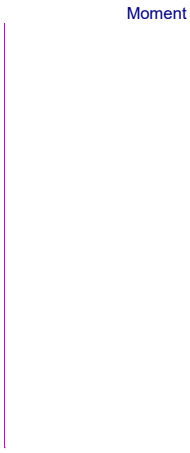
l_{d_prov} = 24.24 in ≥ l_d = 12 in ✓

Stem Forces [1.4D]

Stem Internal Forces



Stem Internal Forces



Stem Internal Forces

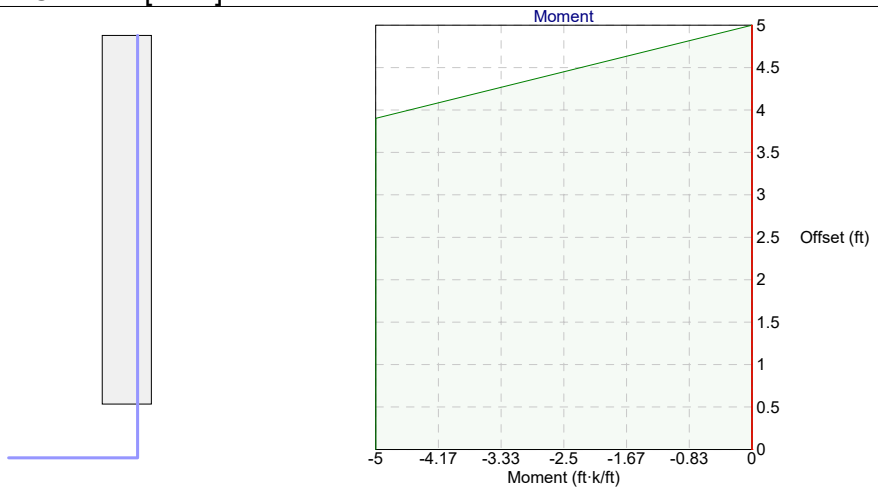


Stem Joint Force Transfer

Location	Force
@ stem base	0 k/ft

Stem Internal Forces

Stem Moment Checks [1.4D]



Stem Shear Checks [1.4D]



Stem Miscellaneous Checks [1.4D]

Minimum Steel Check (ACI 318-14 9.6.1) @ 0 ft from base [Stem in negative flexure]

$\phi M_n = 5$ ft·k / ft $\geq (4 / 3) M_u = [4 / 3] (0$ ft·k / ft$) = 0$ ft·k / ft

Check is waived per ACI 9.6.1.3 ✓

Minimum Steel Check (ACI 318-14 9.6.1) @ 5 ft from base [Stem in negative flexure]

$\phi M_n = 0$ ft·k / ft $\geq (4 / 3) M_u = [4 / 3] (0$ ft·k / ft$) = 0$ ft·k / ft

Check is waived per ACI 9.6.1.3 ✓

Maximum Steel Check (ACI 318-14 9.3.3.1) @ 0 ft from base [Stem in negative flexure]

$\beta_1 = 0.850$ ($F'_c \leq 4000$ psi)

$$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0.02 \text{ in}^2 / \text{in}) (60000 \text{ psi})}{0.85 (3000 \text{ psi})} = 0.39 \text{ in}$$

$$\varepsilon_t = 0.003 \left(\frac{d}{a / \beta_1} - 1 \right) = 0.003 \left[\frac{(5.75 \text{ in})}{(0.39 \text{ in}) / (0.850)} - 1 \right] = 0.0344$$

$\varepsilon_t = 0.0344 \geq 0.004$ ✓

Maximum Steel Check (ACI 318-14 9.3.3.1) @ 5 ft from base [Stem in negative flexure]

$\beta_1 = 0.850$ ($F'_c \leq 4000$ psi)

$$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0.02 \text{ in}^2 / \text{in}) (60000 \text{ psi})}{0.85 (3000 \text{ psi})} = 0.39 \text{ in}$$

$$\varepsilon_t = 0.003 \left(\frac{d}{a / \beta_1} - 1 \right) = 0.003 \left[\frac{(5.75 \text{ in})}{(0.39 \text{ in}) / (0.850)} - 1 \right] = 0.0344$$

$\varepsilon_t = 0.0344 \geq 0.004$ ✓

Wall Horizontal Steel (ACI 318-14 11.6.1, 11.7.3)

$$\rho_t = \frac{A_{s_horz} / s_{horz}}{t} = \frac{(0.2 \text{ in}^2) / (12 \text{ in})}{(8 \text{ in})} = 0.0021$$

$\rho_{t_min} = 0.0020$ (bars No. 5 or less, not less than 60 ksi)

$\rho_t = 0.0021 \geq \rho_{t_min} = 0.0020$ ✓

$3 h = 3 (8 \text{ in}) = 24$ in

18 inch limit governs

$s_{horz} = 12$ in $\leq s_{horz_max} = 18$ in ✓

Development Check (ACI 318-14 11.7.1.2, 25.4.2.3, 25.4.10)

$\frac{M_u}{\phi M_n} = \frac{(0 \text{ ft·k / ft})}{(5 \text{ ft·k / ft})} = 0.0$ (ratio to represent excess reinforcement)

$\psi_e = 1.0$ (uncoated hooked bars)

$\psi_c = 0.70$ (based on side cover and extension cover)

$\psi_r = 1.0$ (no confining reinforcement)

$\lambda = 1.0$ (normal weight concrete)

$$l_{dh} = \left(\frac{f_y \psi_e \psi_c \psi_r}{50 \lambda \sqrt{F'_c}} \right) d_b = \left[\frac{(60000 \text{ psi}) (1.0) (0.70) (1.0)}{50 (1.0) \sqrt{3000 \text{ psi}}} \right] (0.5 \text{ in}) = 7.67 \text{ in}$$

Factoring l_{dh} by the excess reinforcement ratio (0.0000) per 25.4.10: $l_{dh} = 0$ in

$8 d_b = 8 (0.5 \text{ in}) = 4.0$ (minimum limit, does not control)

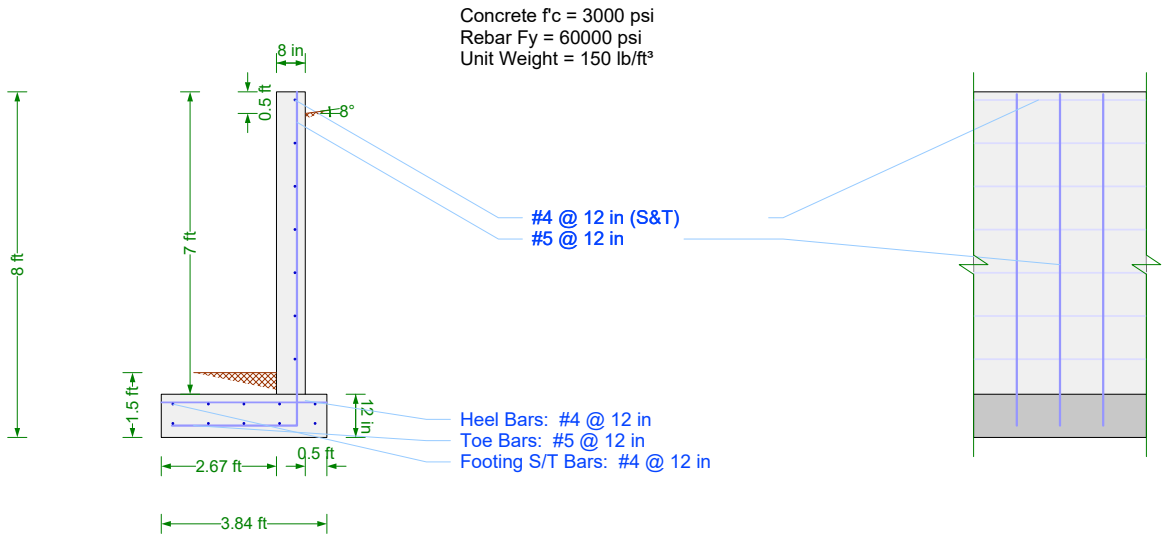
6 inch minimum controls

$l_{dh_prov} = 9$ in $\geq l_{dh} = 6$ in ✓

JOSEPH LUCIA
LUCIA ENGINEERING, INC.

Cantilever Wall 6' Tall

Design Detail



Check Summary

Ratio	Check	Provided	Required	Combination
----- Stability Checks -----				
✓ 0.882	Overturning	1.36	1.20	1.0D + 1.0L + 1.0H + 0.7E
✓ 0.959	Bearing Pressure	2500 psf	2397 psf	1.0D + 1.0L + 1.0H + 0.7E
✓ 0.525	Bearing Eccentricity	12.09 in	23.02 in	1.0D + 1.0L + 1.0H + 0.7E
----- Toe Checks -----				
✓ 0.351	Shear	8.57 k/ft	3.01 k/ft	1.2D + 0.5L + 1.6H + 1.0E
✓ 0.461	Moment	11.7 ft·k/ft	5.4 ft·k/ft	1.2D + 0.5L + 1.6H + 1.0E
✓ 0.120	Min Strain	0.0334	0.0040	1.2D + 1.6L + 1.6H
✓ 0.000	Min Steel	0.03 in²	0 in²	1.2D + 1.6L + 1.6H
✓ 1.000	Development	6 in	6 in	1.2D + 1.6L + 1.6H
✓ 0.667	S&T Max Spacing	12 in	18 in	1.2D + 1.6L + 1.6H
✓ 0.648	S&T Min Rho	0.0028	0.0018	1.2D + 1.6L + 1.6H
----- Heel Checks -----				
✓ 0.059	Shear	9.61 k/ft	0.56 k/ft	1.4D
✓ 0.014	Moment	8.6 ft·k/ft	0.12 ft·k/ft	1.2D + 1.6L + 1.6H
✓ 0.066	Min Strain	0.0604	0.0040	1.2D + 1.6L + 1.6H
✓ 0.000	Min Steel	0.02 in²	0 in²	1.2D + 1.6L + 1.6H
✓ 0.315	Development	38.04 in	12 in	1.2D + 1.6L + 1.6H
✓ 0.667	S&T Max Spacing	12 in	18 in	1.2D + 1.6L + 1.6H
✓ 0.648	S&T Min Rho	0.0028	0.0018	1.2D + 1.6L + 1.6H
----- Stem Checks -----				
✓ 0.719	Moment	7.51 ft·k/ft	5.4 ft·k/ft	1.2D + 0.5L + 1.6H + 1.0E
✓ 0.359	Shear	5.61 k/ft	2.01 k/ft	1.2D + 0.5L + 1.6H + 1.0E
✓ 0.192	Max Steel	0.0209	0.0040	1.2D + 1.6L + 1.6H
✓ 0.000	Min Steel	0.03 in²/in	0 in²/in	1.2D + 1.6L + 1.6H
✓ 0.765	Base Development	9 in	6.89 in	1.2D + 0.5L + 1.6H + 1.0E
✓ 0.000	Horz Bar Rho	0.0000	0.0000	1.2D + 1.6L + 1.6H
✓ 0.667	Horz Bar Spacing	12 in	18 in	1.2D + 1.6L + 1.6H

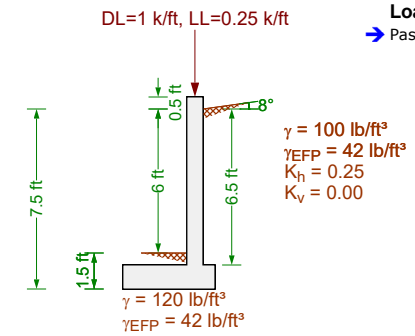
Criteria

Use basic criteria from common proje...	Yes
Building Code	IBC 2021
Concrete Load Combs	IBC 2021 (Strength)
Masonry Load Combs	ASCE 7-16 (ASD)
Stability Load Combs	IBC Retaining Wall St...
Apply Sds Factor to Seismic Combin...	No
Restrained Against Sliding	Yes
Neglect Bearing At Heel	Yes
Use Vert. Comp. for OT	No
Use Vert. Comp. for Sliding	No
Use Vert. Comp. for Bearing	Yes
Use Surcharge for Sliding & OT	Yes
Use Surcharge for Bearing	Yes
Neglect Soil Over Toe	No
Neglect Backfill Wt. for Coulomb	No
Factor Soil Weight As Dead	Yes
Use Passive Force for OT	Yes
Assume Pressure To Top	Yes
Extend Backfill Pressure To Key Bott...	No
Use Toe Passive Pressure for Bearing	No
Required F.S. for OT	1.50
Required F.S. for Sliding	1.50
Has Different Safety Factors for Seis...	Yes
Seismic F.S. for OT	1.20
Seismic F.S. for Sliding	1.20
Allowable Bearing Pressure	2500 psf
Req'd Bearing Location	Over footing
Wall Friction Angle	25°
Friction Coefficent	0.35
Soil Reaction Modulus	172800 lb/ft³

JOSEPH LUCIA
LUCIA ENGINEERING, INC.

Cantilever Wall 6' Tall

Loads



Loading Options/Assumptions

➔ Passive pressure neglects top 0 ft of soil.

Load Combinations

IBC 2018 (Strength)

1.2D + 1.6L + 1.6H
1.2D + 1.6L + 0.9H
1.2D + 0.5L + 1.6H + 1.0E
1.2D + 0.5L + 1.6H
1.2D + 0.5L + 0.9H + 1.0E
1.2D + 0.5L + 0.9H
1.2D + 1.6H + 1.0E
1.2D + 1.6H
1.2D + 0.9H + 1.0E
1.2D + 0.9H
0.9D + 1.6H + 1.0E
0.9D + 1.6H
0.9D + 0.9H + 1.0E
0.9D + 0.9H
1.4D

Strength Check Results Summary

Load Combination	Stem M-applied (ft·k/ft)	Stem M-allow (ft·k/ft)	Stem V-applied (k/ft)	Stem V-allow (k/ft)	Stem Min. ld (in)	Stem Actual ld (in)	Stem Min. strain	Stem Actual strain	Stem Min. steel (in²/in)
1.2D + 1.6L + 1.6H	3.08	7.51	1.42	5.61	6	9	0.0040	0.0209	0
1.2D + 1.6L + 0.9H	1.73	7.51	0.8	5.61	6	9	0.0040	0.0209	0
1.2D + 0.5L + 1.6H + 1.0E	5.4	7.51	2.01	5.61	6.89	9	0.0040	0.0209	0
1.2D + 0.5L + 1.6H	3.08	7.51	1.42	5.61	6	9	0.0040	0.0209	0
1.2D + 0.5L + 0.9H + 1.0E	4.05	7.51	1.39	5.61	6	9	0.0040	0.0209	0
1.2D + 0.5L + 0.9H	1.73	7.51	0.8	5.61	6	9	0.0040	0.0209	0
1.2D + 1.6H + 1.0E	5.4	7.51	2.01	5.61	6.89	9	0.0040	0.0209	0
1.2D + 1.6H	3.08	7.51	1.42	5.61	6	9	0.0040	0.0209	0
1.2D + 0.9H + 1.0E	4.05	7.51	1.39	5.61	6	9	0.0040	0.0209	0
1.2D + 0.9H	1.73	7.51	0.8	5.61	6	9	0.0040	0.0209	0
0.9D + 1.6H + 1.0E	5.4	7.51	2.01	5.61	6.89	9	0.0040	0.0209	0
0.9D + 1.6H	3.08	7.51	1.42	5.61	6	9	0.0040	0.0209	0
0.9D + 0.9H + 1.0E	4.05	7.51	1.39	5.61	6	9	0.0040	0.0209	0
0.9D + 0.9H	1.73	7.51	0.8	5.61	6	9	0.0040	0.0209	0
1.4D	0	0	0	0	6	9	0.0040	0.0209	0

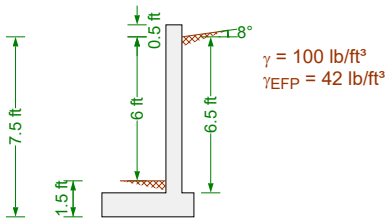
Load Combination	Stem Actual steel (in²/in)	Heel M-applied (ft·k/ft)	Heel M-allow (ft·k/ft)	Heel V-applied (k/ft)	Heel V-allow (k/ft)	Toe M-applied (ft·k/ft)	Toe M-allow (ft·k/ft)	Toe V-applied (k/ft)	Toe V-allow (k/ft)
1.2D + 1.6L + 1.6H	0.03	0.12	8.6	0.48	9.61	5.64	11.7	2.91	8.57
1.2D + 1.6L + 0.9H	0.03	0.12	8.6	0.48	9.61	5.64	11.7	2.91	8.57
1.2D + 0.5L + 1.6H + 1.0E	0.03	0.12	8.6	0.48	9.61	5.81	11.7	3.01	8.57
1.2D + 0.5L + 1.6H	0.03	0.12	8.6	0.48	9.61	5.15	11.7	2.66	8.57
1.2D + 0.5L + 0.9H + 1.0E	0.03	0.12	8.6	0.48	9.61	5.81	11.7	3.01	8.57
1.2D + 0.5L + 0.9H	0.03	0.12	8.6	0.48	9.61	5.15	11.7	2.66	8.57
1.2D + 1.6H + 1.0E	0.03	0.12	8.6	0.48	9.61	5.59	11.7	2.89	8.57
1.2D + 1.6H	0.03	0.12	8.6	0.48	9.61	4.93	11.7	2.55	8.57
1.2D + 0.9H + 1.0E	0.03	0.12	8.6	0.48	9.61	5.59	11.7	2.89	8.57
1.2D + 0.9H	0.03	0.12	8.6	0.48	9.61	4.93	11.7	2.55	8.57
0.9D + 1.6H + 1.0E	0.03	0.09	8.6	0.36	9.61	4.36	11.7	2.26	8.57
0.9D + 1.6H	0.03	0.09	8.6	0.36	9.61	3.7	11.7	1.91	8.57
0.9D + 0.9H + 1.0E	0.03	0.09	8.6	0.36	9.61	4.36	11.7	2.26	8.57
0.9D + 0.9H	0.03	0.09	8.6	0.36	9.61	3.7	11.7	1.91	8.57
1.4D	0.03	0.14	8.6	0.56	9.61	5.75	11.7	2.97	8.57

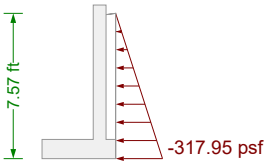
Stability Check Results Summary

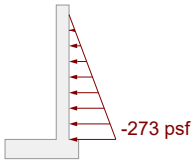
Load Combination	Overturning Moment (ft·k/ft)	Resisting Moment (ft·k/ft)	Overturning F.S.	Overturning F.S. Req'd	Overturning F.S. Req'd Seismic	Sliding Force (lb/in)	Resisting Force (lb/in)	Sliding F.S.
1.0D + 1.0L + 1.0H + 0.7E	3.04	7.62	2.509	1.500	1.200	100.3	99.48	0.992
1.0D + 1.0L + 1.0H	3.04	7.62	2.509	1.500	1.200	100.3	91.8	0.915
1.0D + 1.0H + 0.7E	3.04	7.62	2.509	1.500	1.200	100.3	92.19	0.919
1.0D + 1.0H	3.04	7.62	2.509	1.500	1.200	100.3	84.51	0.843

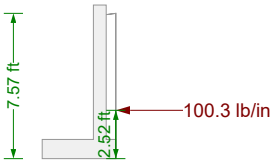
Load Combination	Sliding F.S. Req'd	Sliding F.S. Req'd Seismic	Bearing Pressure Actual (psf)	Bearing Pressure Allowable (psf)	Bearing Eccentricity Actual (in)	Bearing Eccentricity Allowable (in)	Wall Top Actual Deflection (in)
1.0D + 1.0L + 1.0H + 0.7E	1.500	1.200	2397	2500	12.09	23.02	0.36
1.0D + 1.0L + 1.0H	1.500	1.200	2204	2500	12.09	23.02	0.36
1.0D + 1.0H + 0.7E	1.500	1.200	2214	2500	12.09	23.02	0.36
1.0D + 1.0H	1.500	1.200	2021	2500	12.09	23.02	0.36

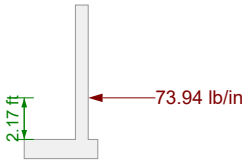
Backfill Pressure











Lateral Earth Pressure

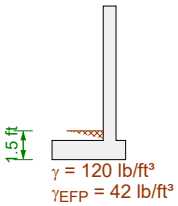
Equivalent Fluid Pressure

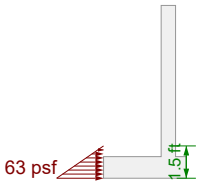
$\sigma_h = H\,\gamma_{fluid}$ = (7.57 ft)(42 lb / ft³) = 318 psf

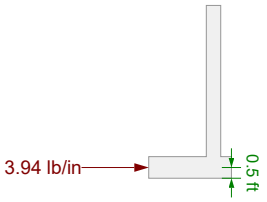
Lateral Earth Pressure (stem only)

$\sigma_h = H\,\gamma_{fluid}$ = (6.5 ft)(42 lb / ft³) = 273 psf

Passive Pressure





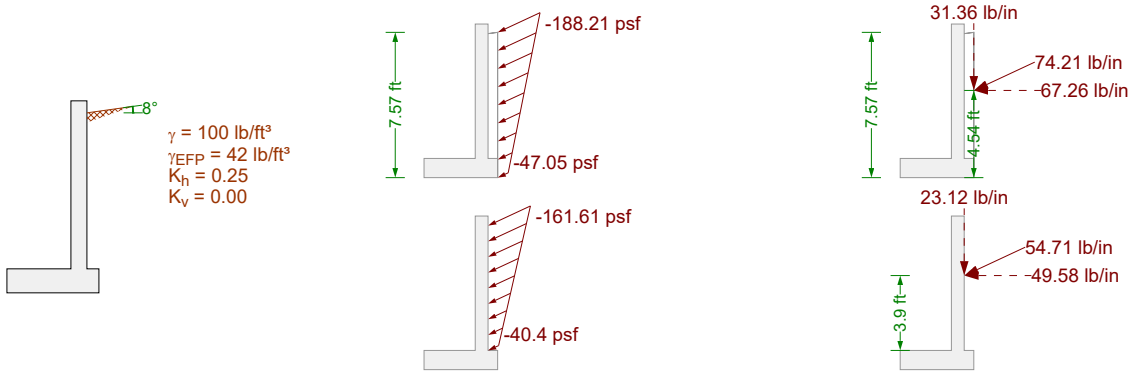


Lateral Earth Pressure

Equivalent Fluid Pressure

$\sigma_h = H\,\gamma_{fluid} = (1.5\text{ ft})(42\text{ lb / ft}^3) = 63\text{ psf}$

Seismic Pressure



Seismic Pressure

Dynamic + static force (Mononobe - Okabe equation)

$$\theta' = \text{atan}\left(\frac{k_h}{1 - k_v}\right) = \arctan\left[\frac{(0.250)}{1 - (0.0)}\right] = 14.04°$$

$$K_{ae} = \frac{\sin^2(\beta + \phi - \theta')}{\cos(\theta')\sin^2(\beta)\sin(\beta - \theta' - \delta)\left[1 + \sqrt{\frac{\sin(\phi + \delta)\sin(\phi - \theta' - \alpha)}{\sin(\beta - \delta - \theta')\sin(\alpha + \beta)}}\right]^{\wedge} 2}$$

$$= \frac{\sin^2[(90°) + (30°) - (14.04°)]}{\cos((14.04°))\sin^2((90°))\sin[(90°) - (14.04°) - (25°)]\left[1 + \sqrt{\frac{\sin[(30°) + (25°)]\sin[(30°) - (14.04°) - (8°)]}{\sin[(90°) - (25°) - (14.04°)]\sin[(8°) + (90°)]}}\right]^{\wedge} 2}$$

$$= 0.6403$$

$$P_{ae} = \frac{1}{2} K_{ae}\,\gamma\, H^{\wedge} 2\,(1 - k_v) = \frac{1}{2}(0.6403)(100\text{ lb / ft}^3)(7.57\text{ ft})^{\wedge} 2\,[1 - (0.0)] = 152.9\text{ lb / in}$$

Static - only force (Coulomb equation)

$$K_a = \frac{\sin^2(\beta + \phi)}{\sin^2(\beta)\sin(\beta - \delta)\left[1 + \sqrt{\frac{\sin(\phi + \delta)\sin(\phi - \alpha)}{\sin(\beta - \delta)\sin(\alpha + \beta)}}\right]^{\wedge} 2}$$

$$= \frac{\sin^2[(90°) + (30°)]}{\sin^2((90°))\sin[(90°) - (25°)]\left[1 + \sqrt{\frac{\sin[(30°) + (25°)]\sin[(30°) - (8°)]}{\sin[(90°) - (25°)]\sin[(8°) + (90°)]}}\right]^{\wedge} 2}$$

$$= 0.3295$$

$$P_a = \frac{1}{2} K_a\,\gamma\, H^{\wedge} 2 = \frac{1}{2}(0.3295)(100\text{ lb / ft}^3)(7.57\text{ ft})^{\wedge} 2 = 78.68\text{ lb / in}$$

Net dynamic force

$\Delta P_{ae} = P_{ae} - P_a = (152.9\text{ lb / in}) - (78.68\text{ lb / in}) = 74.21\text{ lb / in}$

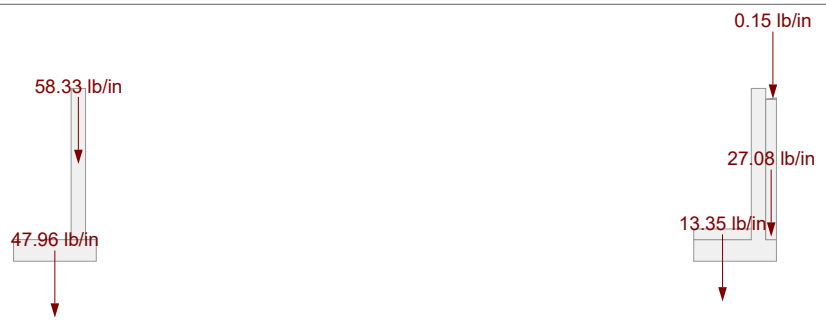
$\alpha_P = 90° - \beta + \delta = 90° - (90°) + (25°) = 25°$ (resultant force angle with horizontal)

To arrive at the pressure distribution illustrated above (used to determine stem moments), apply inverted triangular pressure plus a uniform portion to bring resultant to 0.6H

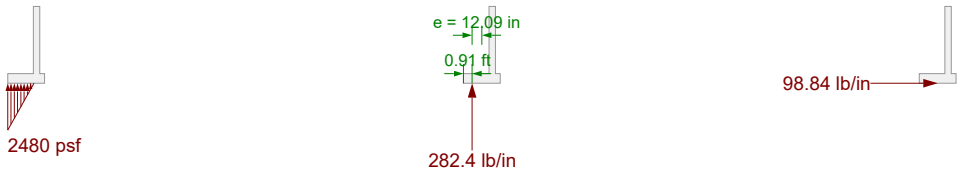
$$\sigma_{e_top} = \frac{8}{5}\frac{\Delta P_{ae}}{H} = \frac{8}{5}\frac{(74.21\text{ lb / in})}{(7.57\text{ ft})} = 188.2\text{ psf}$$

$$\sigma_{e_bot} = \frac{2}{5}\frac{\Delta P_{ae}}{H} = \frac{2}{5}\frac{(74.21\text{ lb / in})}{(7.57\text{ ft})} = 47.05\text{ psf}$$

Wall/Soil Weights



Bearing Pressure



Friction

$F = \mu R = (0.350)(282.4\ lb/in) = 98.84\ lb/in$

Bearing Pressure Calculation

Contributing Forces

	Vert Force	...offset	Horz Force	...offset	OT Moment
Backfill Pressure	-0 lb/in	-	-100.29 lb/in	2.52 ft	36443 in·lb/ft
Axial Dead Load	-83.33 lb/in	3 ft	0 lb/in	-	-36040 in·lb/ft
Axial Live Load	-20.83 lb/in	3 ft	0 lb/in	-	-9010 in·lb/ft
Seismic Force	-31.36 lb/in	3.84 ft	-67.26 lb/in	4.54 ft	26664 in·lb/ft
Footing Weight	-47.96 lb/in	1.92 ft	0 lb/in	-	-13248.01 in·lb/ft
Stem Weight	-58.33 lb/in	3 ft	0 lb/in	-	-25228 in·lb/ft
Backfill Weight	-27.08 lb/in	3.59 ft	0 lb/in	-	-13988 in·lb/ft
Backfill Weight	-0.15 lb/in	3.67 ft	0 lb/in	-	-77.37 in·lb/ft
Soil over toe Weight	-13.35 lb/in	1.34 ft	0 lb/in	-	-2566.4 in·lb/ft
	-282.4 lb/in				-37050.99 in·lb/ft

$$\frac{-37050.99\ in\cdot lb/ft}{-282.4\ lb/in} = 0.91\ ft$$

Stability Checks [1.0D + 1.0L + 1.0H + 0.7E]

Overturning Check

Overturning Moments

	Force	Distance	Moment
Backfill pressure (horz)	100.3 lb/in	2.52 ft	36443 in·lb/ft
Seismic force	47.08 lb/in	4.54 ft	30794 in·lb/ft
		Total:	67237 in·lb/ft

Resisting Moments

	Force	Distance	Moment
Passive pressure @ toe	3.94 lb/in	0.5 ft	283.5 in·lb/ft
Axial dead load	-83.33 lb/in	3 ft	36040 in·lb/ft
Footing Weight	-47.96 lb/in	1.92 ft	13248 in·lb/ft
Stem Weight	-58.33 lb/in	3 ft	25228 in·lb/ft
Backfill Weight	-27.08 lb/in	3.59 ft	13988 in·lb/ft
Backfill Weight	-0.15 lb/in	3.67 ft	77.37 in·lb/ft
Soil over toe Weight	-13.35 lb/in	1.34 ft	2566 in·lb/ft
		Total:	91431 in·lb/ft

Without seismic loads:

$$F.S. = \frac{RM}{OTM} = \frac{91431 \text{ in·lb / ft}}{36443 \text{ in·lb / ft}} = 2.509 > 1.50 \text{ (OK)}$$

Including seismic loads:

$$F.S. = \frac{RM}{OTM} = \frac{91431 \text{ in·lb / ft}}{67237 \text{ in·lb / ft}} = 1.360 > 1.20 \text{ (OK)}$$

Sliding Check

Check not performed; restrained against sliding.

Bearing Capacity Check

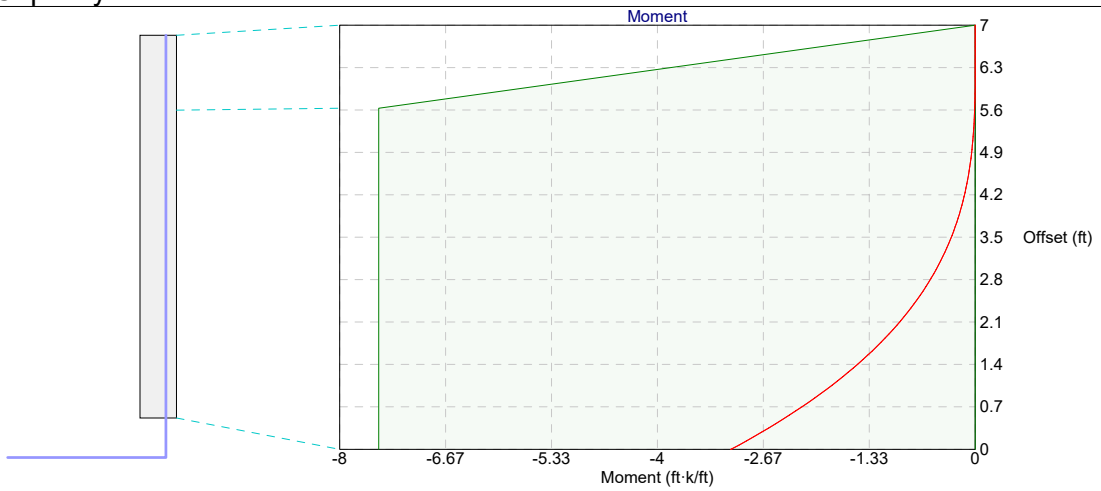
Bearing pressure < allowable (2397 psf < 2500 psf) - OK
Bearing resultant eccentricity < allowable (12.09 in < 23.02 in) - OK

Wall Top Displacement

(based on unfactored service loads)

Deflection due to stem flexural displacement	0.05 in
Deflection due to rotation from settlement	0.314 in
Total deflection at top of wall (positive towards toe)	0.365 in

Stem Flexural Capacity



Capacity (ACI 318-14 11.5.2.2, »22.3, »22.2) @ 0 ft from base

$$a = \frac{A_s f_y}{0.85 f'_c} = \frac{(0.03 \text{ in}^2 / \text{in})(60000 \text{ psi})}{0.85 (3000 \text{ psi})} = 0.61 \text{ in}$$

$$\phi M_n = \phi A_s f_y (d - a / 2) = (0.90)(0.03 \text{ in}^2 / \text{in})(60000 \text{ psi})[(5.69 \text{ in}) - (0.61 \text{ in}) / 2] = 7.51 \text{ ft·k / ft}$$

Capacity (ACI 318-14 11.5.2.2, »22.3, »22.2) @ 5.63 ft from base

$$a = \frac{A_s f_y}{0.85 f'_c} = \frac{(0.03 \text{ in}^2 / \text{in})(60000 \text{ psi})}{0.85 (3000 \text{ psi})} = 0.61 \text{ in}$$

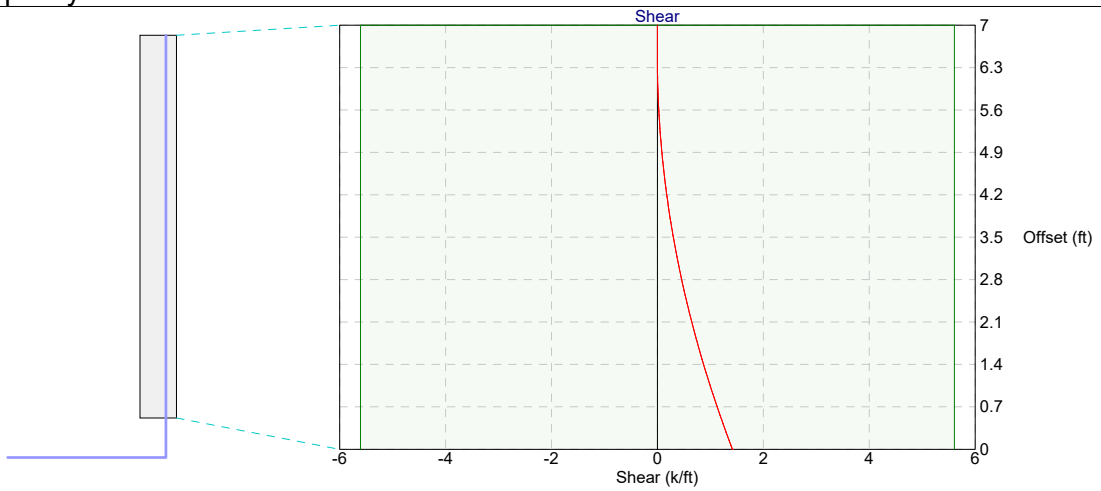
$$\phi M_n = \phi A_s f_y (d - a / 2) = (0.90)(0.03 \text{ in}^2 / \text{in})(60000 \text{ psi})[(5.69 \text{ in}) - (0.61 \text{ in}) / 2] = 7.51 \text{ ft·k / ft}$$

Capacity (ACI 318-14 11.5.2.2, »22.3, »22.2) @ 7 ft from base

$$a = \frac{A_s f_y}{0.85 f'_c} = \frac{(0 \text{ in}^2 / \text{in})(60000 \text{ psi})}{0.85 (3000 \text{ psi})} = 0 \text{ in}$$

$$\phi M_n = \phi A_s f_y (d - a / 2) = (0.90)(0 \text{ in}^2 / \text{in})(60000 \text{ psi})[(5.69 \text{ in}) - (0 \text{ in}) / 2] = 0 \text{ ft·k / ft}$$

Stem Shear Capacity



Shear Capacity (ACI 318-14 11.5.5.1, 22.5.1.1, 22.5.5.1) @ 0 ft from base

λ = 1.0 (normal weight concrete)

$V_c = 2 \lambda \sqrt{F'_c}\, d$ = 2 (1.0) $\sqrt{3000 \text{ psi}}$ (5.69 in) = 7.48 k / ft

$\phi V_n = \phi V_c$ = (0.750) (7.48 k / ft) = 5.61 k / ft

Shear Capacity (ACI 318-14 11.5.5.1, 22.5.1.1, 22.5.5.1) @ 7 ft from base

λ = 1.0 (normal weight concrete)

$V_c = 2 \lambda \sqrt{F'_c}\, d$ = 2 (1.0) $\sqrt{3000 \text{ psi}}$ (5.69 in) = 7.48 k / ft

$\phi V_n = \phi V_c$ = (0.750) (7.48 k / ft) = 5.61 k / ft

Stem Development/Lap Length Calculations

Main vertical stem bars (bottom end) - Development Length Calculation (ACI 318-14 11.7.1.2, 25.4.2.3, 25.4.3)

ψ_e = 1.0 (uncoated hooked bars)

ψ_c = 0.70 (based on side cover and extension cover)

ψ_r = 1.0 (no confining reinforcement)

λ = 1.0 (normal weight concrete)

$$l_{dh} = \left(\frac{f_y \, \psi_e \, \psi_c \, \psi_r}{50 \, \lambda \sqrt{F'_c}} \right) d_b = \left[\frac{(60000 \text{ psi}) \, (1.0) \, (0.70) \, (1.0)}{50 \, (1.0) \sqrt{3000 \text{ psi}}} \right] (0.63 \text{ in}) = 9.59 \text{ in}$$

$8 \, d_b = 8 \, (0.63 \text{ in}) = 5.0$ (minimum limit, does not control)

Main vertical stem bars (top end) - Development Length Calculation (ACI 318-14 11.7.1.2, 25.4.2.3, 25.4.3)

ψ_t = 1.0 (bars are not horizontal)

ψ_e = 1.0 (bar not epoxy coated)

ψ_s = 0.80 (bars are #6 or smaller)

λ = 1.0 (normal weight concrete)

$s / 2 = (12 \text{ in}) / 2 = 6 \text{ in}$

$\text{cover} + d_b / 2 = (2 \text{ in}) + (0.63 \text{ in}) / 2 = 2.31 \text{ in}$

c_b = 2.31 in (lesser of half spacing, ctr to surface)

K_{tr} = 0.0 (no transverse reinforcement)

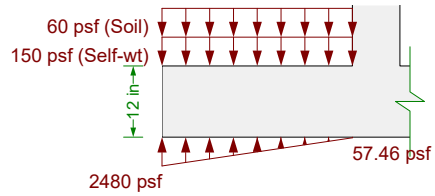
$$\frac{c_b + K_{tr}}{d_b} = \frac{(2.31 \text{ in}) + (0.0)}{(0.63 \text{ in})} = 3.70$$

$$l_d = \left(\frac{3.}{40} \frac{f_y}{\lambda \sqrt{F'_c}} \frac{\psi_t \, \psi_e \, \psi_s}{2.5} \right) d_b = \left[\frac{3.}{40} \frac{(60000 \text{ psi})}{(1.0) \sqrt{3000 \text{ psi}}} \frac{(1.0) \, (1.0) \, (0.80)}{2.5} \right] (0.63 \text{ in}) = 16.43 \text{ in}$$

Toe Checks [1.2D + 1.6L + 1.6H]

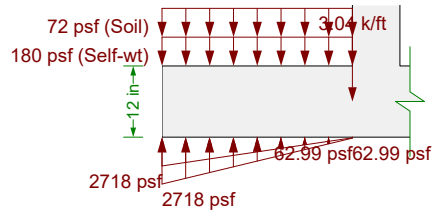
Toe Unfactored Loads

Unfactored Loads



Toe Factored Loads

1.2D + 1.6L + 1.6H



Controlling Moment

Design moment M_u for toe need not exceed moment at stem base:

M_{toe} = 5.64 ft·k / ft ≥ M_{stem} = 3.08 ft·k / ft

M_u = 3.08 ft·k / ft (stem base moment controls)

Flexure Check (ACI 318-14 13.3.2.1, 7.5.2.1, »22.3, »22.2, 7.5.1.1a)

$$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0.03 \text{ in}^2 / \text{in})(60000 \text{ psi})}{0.85 (3000 \text{ psi})} = 0.61 \text{ in}$$

$$\phi M_n = \phi A_s f_y (d - a / 2) = (0.90)(0.03 \text{ in}^2 / \text{in})(60000 \text{ psi})[(8.69 \text{ in}) - (0.61 \text{ in}) / 2] = 11.7 \text{ ft·k / ft}$$

ϕM_n = 11.7 ft·k / ft ≥ M_u = 3.08 ft·k / ft ✓

Shear Check (ACI 318-14 13.3.2.1, 7.5.3.1, »22.5.1, »22.5.5, 7.5.1.1b)

λ = 1.0 (normal weight concrete)

$$V_c = 2 \lambda \sqrt{F'_c} d = 2 (1.0) \sqrt{3000 \text{ psi}} (8.69 \text{ in}) = 11.42 \text{ k / ft}$$

$$\phi V_n = \phi V_c = (0.750)(11.42 \text{ k / ft}) = 8.57 \text{ k / ft}$$

ϕV_n = 8.57 k / ft ≥ V_u = 2.91 k / ft ✓

Minimum Strain Check (ACI 318-14 13.3.2.1, 7.3.3.1)

β_1 = 0.850 ($F'_c \le 4000$ psi)

$$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0.03 \text{ in}^2 / \text{in})(60000 \text{ psi})}{0.85 (3000 \text{ psi})} = 0.61 \text{ in}$$

$$\varepsilon_t = 0.003 \left(\frac{d}{a / \beta_1} - 1 \right) = 0.003 \left[\frac{(8.69 \text{ in})}{(0.61 \text{ in}) / (0.850)} - 1 \right] = 0.0334$$

ε_t = 0.0334 ≥ 0.004 ✓

Minimum Steel Check (ACI 318-14 13.3.2.1, 9.6.1)

ϕM_n = 11.7 ft·k / ft ≥ (4 / 3) M_u = [4 / 3](3.08 ft·k / ft) = 4.1 ft·k / ft

Check is waived per ACI 9.6.1.3 ✓

Shrinkage and Temperature Steel (ACI 318-14 13.2.8.1, 7.6.4.1, 24.4.3.2, 24.4.3.3)

$$\rho_{ST_prov} = \frac{A_{ST}}{t\, s_{ST}} = \frac{(0.4 \text{ in}^2 / \text{in})}{(12 \text{ in})(12 \text{ in})} = 0.0028$$

$$\rho_{ST_prov} = \frac{A_{ST}}{t\, s_{ST}} = \frac{(0.4 \text{ in}^2 / \text{in})}{(12 \text{ in})(12 \text{ in})} = 0.0028$$

$$\frac{0.0018 (60000)}{f_y} = \frac{0.0018 (60000)}{(60000 \text{ psi})} = 0.0018$$

ρ_{ST_min} = 0.0018

ρ_{ST_prov} = 0.0028 ≥ ρ_{ST_min} = 0.0018 ✓

18 inch limit governs

s_{ST_max} = 18 in

s_{ST} = 12 in ≤ s_{ST_max} = 18 in ✓

Development Check (ACI 318-14 13.2.8.1, 25.4.2.3, 25.4.10)

$$\frac{M_u}{\phi M_n} = \frac{(3.08 \text{ ft·k / ft})}{(11.7 \text{ ft·k / ft})} = 0.2630$$ (ratio to represent excess reinforcement)

ψ_e = 1.0 (uncoated hooked bars)

ψ_c = 0.70 (based on side cover and extension cover)

ψ_r = 1.0 (no confining reinforcement)

λ = 1.0 (normal weight concrete)

$$l_{dh} = \left(\frac{f_y \psi_e \psi_c \psi_r}{50 \lambda \sqrt{F'_c}} \right) d_b = \left[\frac{(60000 \text{ psi})(1.0)(0.70)(1.0)}{50 (1.0) \sqrt{3000 \text{ psi}}} \right] (0.63 \text{ in}) = 9.59 \text{ in}$$

Factoring l_{dh} by the excess reinforcement ratio (0.2630) per 25.4.10: l_{dh} = 2.52 in

8 d_b = 8 (0.63 in) = 5.0 (minimum limit, does not control)

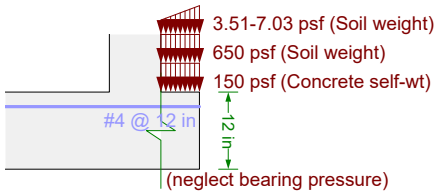
6 inch minimum controls

l_{dh_prov} = 6 in ≥ l_{dh} = 6 in ✓

Heel Checks [1.2D + 1.6L + 1.6H]

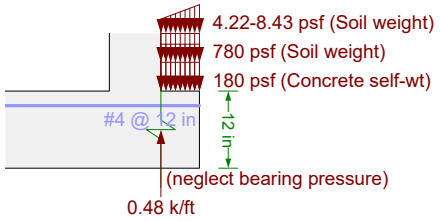
Heel Unfactored Loads

Unfactored Loads



Heel Factored Loads

1.2D + 1.6L + 1.6H



Controlling Moment

Design moment M_u for heel need not exceed moment at stem base:

M_{heel} = 0.12 ft·k / ft < M_{stem} = 3.08 ft·k / ft

M_u = 0.12 ft·k / ft (stem moment does not control)

Flexure Check (ACI 318-14 13.3.2.1, 7.5.2.1, »22.3, »22.2, 7.5.1.1a)

$$a = \frac{A_s f_y}{0.85 f'_c} = \frac{(0.02 \text{ in}^2 / \text{in})(60000 \text{ psi})}{0.85 (3000 \text{ psi})} = 0.39 \text{ in}$$

$$\phi M_n = \phi A_s f_y (d - a / 2) = (0.90)(0.02 \text{ in}^2 / \text{in})(60000 \text{ psi})[(9.75 \text{ in}) - (0.39 \text{ in}) / 2] = 8.6 \text{ ft·k / ft}$$

ϕM_n = 8.6 ft·k / ft ≥ M_u = 0.12 ft·k / ft ✓

Shear Check (ACI 318-14 13.3.2.1, 7.5.3.1, »22.5.1, »22.5.5, 7.5.1.1b)

λ = 1.0 (normal weight concrete)

$$V_c = 2 \lambda \sqrt{f'_c}\, d = 2 (1.0) \sqrt{3000 \text{ psi}}\, (9.75 \text{ in}) = 12.82 \text{ k / ft}$$

$$\phi V_n = \phi V_c = (0.750)(12.82 \text{ k / ft}) = 9.61 \text{ k / ft}$$

ϕV_n = 9.61 k / ft ≥ V_u = 0.48 k / ft ✓

Minimum Strain Check (ACI 318-14 13.3.2.1, 7.3.3.1)

β_1 = 0.850 ($f'_c \le 4000$ psi)

$$a = \frac{A_s f_y}{0.85 f'_c} = \frac{(0.02 \text{ in}^2 / \text{in})(60000 \text{ psi})}{0.85 (3000 \text{ psi})} = 0.39 \text{ in}$$

$$\varepsilon_t = 0.003 \left(\frac{d}{a / \beta_1} - 1 \right) = 0.003 \left[\frac{(9.75 \text{ in})}{(0.39 \text{ in}) / (0.850)} - 1 \right] = 0.0604$$

ε_t = 0.0604 ≥ 0.004 ✓

Minimum Steel Check (ACI 318-14 13.3.2.1, 9.6.1)

ϕM_n = 8.6 ft·k / ft ≥ (4 / 3) M_u = [4 / 3] (0.12 ft·k / ft) = 0.16 ft·k / ft

Check is waived per ACI 9.6.1.3 ✓

Shrinkage and Temperature Steel (ACI 318-14 13.2.8.1, 7.6.4.1, 24.4.3.2, 24.4.3.3)

$$\rho_{ST_prov} = \frac{A_{ST}}{t\, s_{ST}} = \frac{(0.4 \text{ in}^2 / \text{in})}{(12 \text{ in})(12 \text{ in})} = 0.0028$$

$$\rho_{ST_prov} = \frac{A_{ST}}{t\, s_{ST}} = \frac{(0.4 \text{ in}^2 / \text{in})}{(12 \text{ in})(12 \text{ in})} = 0.0028$$

$$\frac{0.0018 (60000)}{f_y} = \frac{0.0018 (60000)}{(60000 \text{ psi})} = 0.0018$$

ρ_{ST_min} = 0.0018

ρ_{ST_prov} = 0.0028 ≥ ρ_{ST_min} = 0.0018 ✓

18 inch limit governs

s_{ST_max} = 18 in

s_{ST} = 12 in ≤ s_{ST_max} = 18 in ✓

Development Check (ACI 318-14 13.2.8.1, 25.4.2.3, 25.4.10)

$$\frac{M_u}{\phi M_n} = \frac{(0.12 \text{ ft·k / ft})}{(8.6 \text{ ft·k / ft})} = 0.0141$$ (ratio to represent excess reinforcement)

ψ_t = 1.0 (12 inches or less cast below - 9.50 inches)

ψ_e = 1.0 (bar not epoxy coated)

ψ_s = 0.80 (bars are #6 or smaller)

λ = 1.0 (normal weight concrete)

s / 2 = (12 in) / 2 = 6 in

cover + d_b / 2 = (2 in) + (0.5 in) / 2 = 2.25 in

c_b = 2.25 in (lesser of half spacing, ctr to surface)

K_{tr} = 0.0 (no transverse reinforcement)

$$\frac{c_b + K_{tr}}{d_b} = \frac{(2.25 \text{ in}) + (0.0)}{(0.5 \text{ in})} = 4.50$$

$$l_d = \left(\frac{3.}{40} \frac{f_y}{\lambda \sqrt{f'_c}} \frac{\psi_t \psi_e \psi_s}{2.5} \right) d_b = \left[\frac{3.}{40} \frac{(60000 \text{ psi})}{(1.0) \sqrt{3000 \text{ psi}}} \frac{(1.0)(1.0)(0.80)}{2.5} \right] (0.5 \text{ in}) = 13.15 \text{ in}$$

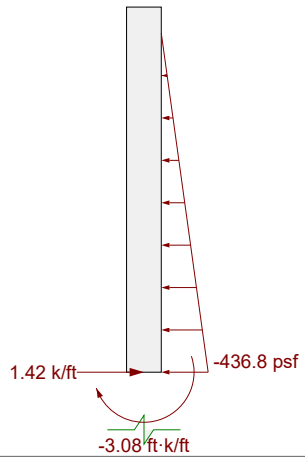
Factoring l_d by the excess reinforcement ratio (0.0141) per 25.4.10: l_d = 0.18 in

12 inch minimum controls

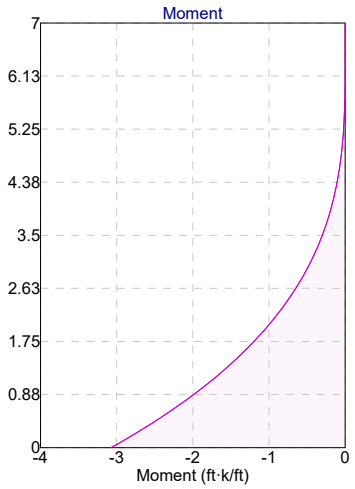
l_{d_prov} = 38.04 in ≥ l_d = 12 in ✓

Stem Forces [1.2D + 1.6L + 1.6H]

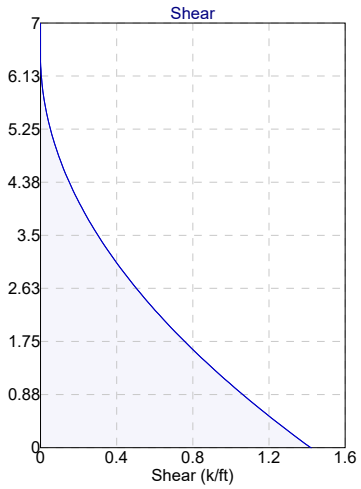
Stem Internal Forces



Stem Internal Forces



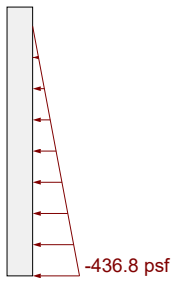
Stem Internal Forces



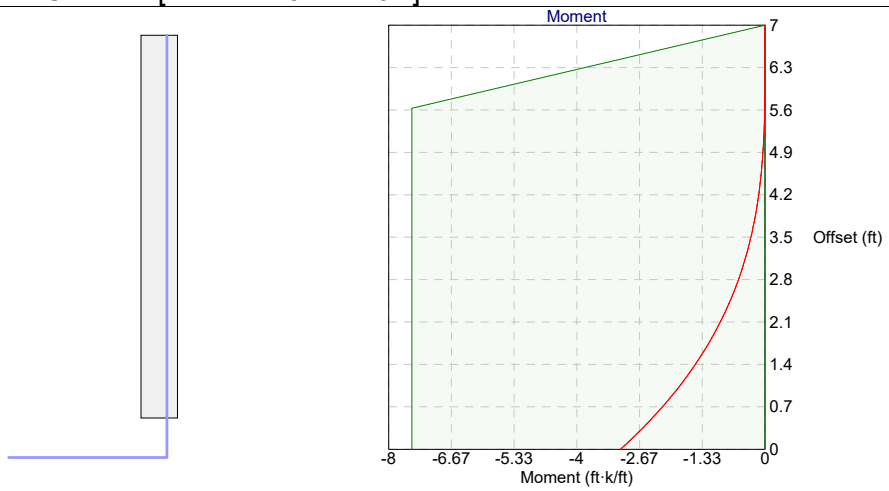
Stem Joint Force Transfer

Location	Force
@ stem base	1.42 k/ft

Stem Internal Forces



Stem Moment Checks [1.2D + 1.6L + 1.6H]



Check (ACI 318-14 11.5.5.1b) @ 0 ft from base

ϕM_n = 7.51 ft·k / ft ≥ M_u = 3.08 ft·k / ft ✓

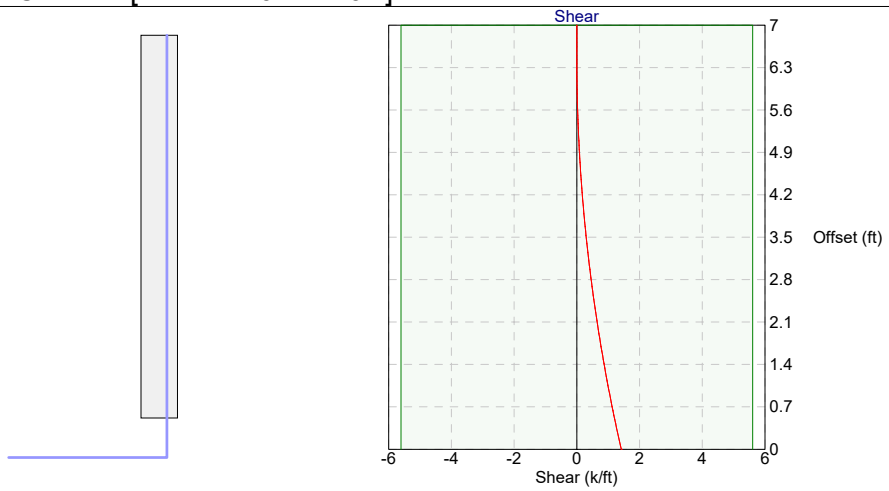
Check (ACI 318-14 11.5.5.1b) @ 5.63 ft from base

ϕM_n = 7.51 ft·k / ft ≥ M_u = 0.01 ft·k / ft ✓

Check (ACI 318-14 11.5.5.1b) @ 5.66 ft from base

ϕM_n = 7.37 ft·k / ft ≥ M_u = 0.01 ft·k / ft ✓

Stem Shear Checks [1.2D + 1.6L + 1.6H]



Shear Check (ACI 318-14 11.5.5.1c) @ 0 ft from base

ϕV_n = 5.61 k / ft ≥ V_u = 1.42 k / ft ✓

Stem Miscellaneous Checks [1.2D + 1.6L + 1.6H]

Minimum Steel Check (ACI 318-14 9.6.1) @ 0 ft from base [Stem in negative flexure]

ϕM_n = 7.51 ft·k / ft ≥ (4 / 3) M_u = [4 / 3] (3.08 ft·k / ft) = 4.1 ft·k / ft

Check is waived per ACI 9.6.1.3 ✓

Minimum Steel Check (ACI 318-14 9.6.1) @ 7 ft from base [Stem in negative flexure]

ϕM_n = 0 ft·k / ft ≥ (4 / 3) M_u = [4 / 3] (0 ft·k / ft) = 0 ft·k / ft

Check is waived per ACI 9.6.1.3 ✓

Maximum Steel Check (ACI 318-14 9.3.3.1) @ 0 ft from base [Stem in negative flexure]

β_1 = 0.850 ($F'_c \le 4000$ psi)

$$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0.03 \text{ in}^2 / \text{in})(60000 \text{ psi})}{0.85 (3000 \text{ psi})} = 0.61 \text{ in}$$

$$\varepsilon_t = 0.003 \left(\frac{d}{a / \beta_1} - 1 \right) = 0.003 \left[\frac{(5.69 \text{ in})}{(0.61 \text{ in}) / (0.850)} - 1 \right] = 0.0209$$

ε_t = 0.0209 ≥ 0.004 ✓

Maximum Steel Check (ACI 318-14 9.3.3.1) @ 7 ft from base [Stem in negative flexure]

β_1 = 0.850 ($F'_c \le 4000$ psi)

$$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0.03 \text{ in}^2 / \text{in})(60000 \text{ psi})}{0.85 (3000 \text{ psi})} = 0.61 \text{ in}$$

$$\varepsilon_t = 0.003 \left(\frac{d}{a / \beta_1} - 1 \right) = 0.003 \left[\frac{(5.69 \text{ in})}{(0.61 \text{ in}) / (0.850)} - 1 \right] = 0.0209$$

ε_t = 0.0209 ≥ 0.004 ✓

Wall Horizontal Steel (ACI 318-14 11.6.1, 11.7.3)

$$\rho_t = \frac{A_{s_horz} / s_{horz}}{t} = \frac{(0.2 \text{ in}^2) / (12 \text{ in})}{(8 \text{ in})} = 0.0021$$

ρ_{t_min} = 0.0020 (bars No. 5 or less, not less than 60 ksi)

ρ_t = 0.0021 ≥ ρ_{t_min} = 0.0020 ✓

3 h = 3 (8 in) = 24 in

18 inch limit governs

s_{horz} = 12 in ≤ s_{horz_max} = 18 in ✓

Development Check (ACI 318-14 11.7.1.2, 25.4.2.3, 25.4.10)

$$\frac{M_u}{\phi M_n} = \frac{(3.08 \text{ ft·k / ft})}{(7.51 \text{ ft·k / ft})} = 0.4096$$ (ratio to represent excess reinforcement)

ψ_e = 1.0 (uncoated hooked bars)

ψ_c = 0.70 (based on side cover and extension cover)

ψ_r = 1.0 (no confining reinforcement)

λ = 1.0 (normal weight concrete)

$$l_{dh} = \left(\frac{f_y \psi_e \psi_c \psi_r}{50 \lambda \sqrt{F'_c}} \right) d_b = \left[\frac{(60000 \text{ psi})(1.0)(0.70)(1.0)}{50 (1.0) \sqrt{3000 \text{ psi}}} \right] (0.63 \text{ in}) = 9.59 \text{ in}$$

Factoring l_{dh} by the excess reinforcement ratio (0.4096) per 25.4.10: l_{dh} = 3.93 in

8 d_b = 8 (0.63 in) = 5.0 (minimum limit, does not control)

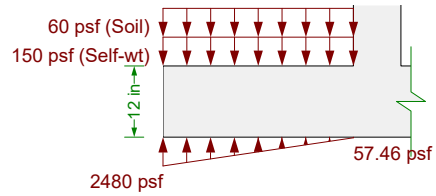
6 inch minimum controls

l_{dh_prov} = 9 in ≥ l_{dh} = 6 in ✓

Toe Checks [1.2D + 0.5L + 1.6H + 1.0E]

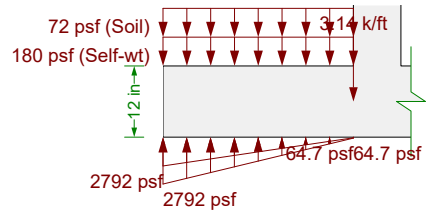
Toe Unfactored Loads

Unfactored Loads



Toe Factored Loads

1.2D + 0.5L + 1.6H + 1.0E



Controlling Moment

Design moment M_u for toe need not exceed moment at stem base:

M_{toe} = 5.81 ft·k / ft ≥ M_{stem} = 5.4 ft·k / ft

M_u = 5.4 ft·k / ft (stem base moment controls)

Flexure Check (ACI 318-14 13.3.2.1, 7.5.2.1, »22.3, »22.2, 7.5.1.1a)

$$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0.03 \text{ in}^2 / \text{in})(60000 \text{ psi})}{0.85 (3000 \text{ psi})} = 0.61 \text{ in}$$

$$\phi M_n = \phi A_s f_y (d - a / 2) = (0.90)(0.03 \text{ in}^2 / \text{in})(60000 \text{ psi})[(8.69 \text{ in}) - (0.61 \text{ in}) / 2] = 11.7 \text{ ft·k / ft}$$

ϕM_n = 11.7 ft·k / ft ≥ M_u = 5.4 ft·k / ft ✓

Shear Check (ACI 318-14 13.3.2.1, 7.5.3.1, »22.5.1, »22.5.5, 7.5.1.1b)

λ = 1.0 (normal weight concrete)

$$V_c = 2 \lambda \sqrt{F'_c} d = 2 (1.0) \sqrt{3000 \text{ psi}} (8.69 \text{ in}) = 11.42 \text{ k / ft}$$

$$\phi V_n = \phi V_c = (0.750)(11.42 \text{ k / ft}) = 8.57 \text{ k / ft}$$

ϕV_n = 8.57 k / ft ≥ V_u = 3.01 k / ft ✓

Minimum Strain Check (ACI 318-14 13.3.2.1, 7.3.3.1)

β_1 = 0.850 ($F'_c \le 4000$ psi)

$$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0.03 \text{ in}^2 / \text{in})(60000 \text{ psi})}{0.85 (3000 \text{ psi})} = 0.61 \text{ in}$$

$$\varepsilon_t = 0.003 \left(\frac{d}{a / \beta_1} - 1 \right) = 0.003 \left[\frac{(8.69 \text{ in})}{(0.61 \text{ in}) / (0.850)} - 1 \right] = 0.0334$$

ε_t = 0.0334 ≥ 0.004 ✓

Minimum Steel Check (ACI 318-14 13.3.2.1, 9.6.1)

ϕM_n = 11.7 ft·k / ft ≥ (4 / 3) M_u = [4 / 3](5.4 ft·k / ft) = 7.2 ft·k / ft

Check is waived per ACI 9.6.1.3 ✓

Shrinkage and Temperature Steel (ACI 318-14 13.2.8.1, 7.6.4.1, 24.4.3.2, 24.4.3.3)

$$\rho_{ST_prov} = \frac{A_{ST}}{t\, s_{ST}} = \frac{(0.4 \text{ in}^2 / \text{in})}{(12 \text{ in})(12 \text{ in})} = 0.0028$$

$$\rho_{ST_prov} = \frac{A_{ST}}{t\, s_{ST}} = \frac{(0.4 \text{ in}^2 / \text{in})}{(12 \text{ in})(12 \text{ in})} = 0.0028$$

$$\frac{0.0018 (60000)}{f_y} = \frac{0.0018 (60000)}{(60000 \text{ psi})} = 0.0018$$

ρ_{ST_min} = 0.0018

ρ_{ST_prov} = 0.0028 ≥ ρ_{ST_min} = 0.0018 ✓

18 inch limit governs

s_{ST_max} = 18 in

s_{ST} = 12 in ≤ s_{ST_max} = 18 in ✓

Development Check (ACI 318-14 13.2.8.1, 25.4.2.3, 25.4.10)

$$\frac{M_u}{\phi M_n} = \frac{(5.4 \text{ ft·k / ft})}{(11.7 \text{ ft·k / ft})} = 0.4614$$ (ratio to represent excess reinforcement)

ψ_e = 1.0 (uncoated hooked bars)

ψ_c = 0.70 (based on side cover and extension cover)

ψ_r = 1.0 (no confining reinforcement)

λ = 1.0 (normal weight concrete)

$$l_{dh} = \left(\frac{f_y \psi_e \psi_c \psi_r}{50 \lambda \sqrt{F'_c}} \right) d_b = \left[\frac{(60000 \text{ psi})(1.0)(0.70)(1.0)}{50 (1.0) \sqrt{3000 \text{ psi}}} \right] (0.63 \text{ in}) = 9.59 \text{ in}$$

Factoring l_{dh} by the excess reinforcement ratio (0.4614) per 25.4.10: l_{dh} = 4.42 in

8 d_b = 8 (0.63 in) = 5.0 (minimum limit, does not control)

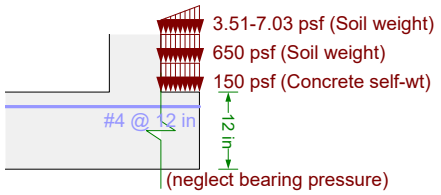
6 inch minimum controls

l_{dh_prov} = 6 in ≥ l_{dh} = 6 in ✓

Heel Checks [1.2D + 0.5L + 1.6H + 1.0E]

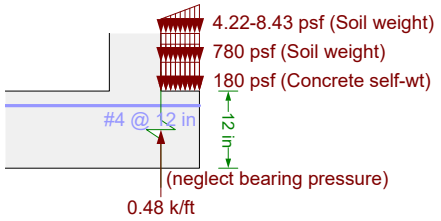
Heel Unfactored Loads

Unfactored Loads



Heel Factored Loads

1.2D + 0.5L + 1.6H + 1.0E



Controlling Moment

Design moment M_u for heel need not exceed moment at stem base:

M_{heel} = 0.12 ft·k / ft < M_{stem} = 5.4 ft·k / ft

M_u = 0.12 ft·k / ft (stem moment does not control)

Flexure Check (ACI 318-14 13.3.2.1, 7.5.2.1, »22.3, »22.2, 7.5.1.1a)

$$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0.02 \text{ in}^2 / \text{in})(60000 \text{ psi})}{0.85 (3000 \text{ psi})} = 0.39 \text{ in}$$

$\phi M_n = \phi A_s f_y (d - a / 2)$ = (0.90)(0.02 in² / in)(60000 psi)[(9.75 in) - (0.39 in) / 2] = 8.6 ft·k / ft

ϕM_n = 8.6 ft·k / ft ≥ M_u = 0.12 ft·k / ft ✓

Shear Check (ACI 318-14 13.3.2.1, 7.5.3.1, »22.5.1, »22.5.5, 7.5.1.1b)

λ = 1.0 (normal weight concrete)

$V_c = 2 \lambda \sqrt{F'_c}\, d = 2 (1.0) \sqrt{3000 \text{ psi}} (9.75 \text{ in})$ = 12.82 k / ft

$\phi V_n = \phi V_c$ = (0.750)(12.82 k / ft) = 9.61 k / ft

ϕV_n = 9.61 k / ft ≥ V_u = 0.48 k / ft ✓

Minimum Strain Check (ACI 318-14 13.3.2.1, 7.3.3.1)

β_1 = 0.850 ($F'_c \le 4000$ psi)

$$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0.02 \text{ in}^2 / \text{in})(60000 \text{ psi})}{0.85 (3000 \text{ psi})} = 0.39 \text{ in}$$

$$\varepsilon_t = 0.003 \left(\frac{d}{a / \beta_1} - 1 \right) = 0.003 \left[\frac{(9.75 \text{ in})}{(0.39 \text{ in}) / (0.850)} - 1 \right] = 0.0604$$

ε_t = 0.0604 ≥ 0.004 ✓

Minimum Steel Check (ACI 318-14 13.3.2.1, 9.6.1)

ϕM_n = 8.6 ft·k / ft ≥ (4 / 3) M_u = [4 / 3](0.12 ft·k / ft) = 0.16 ft·k / ft

Check is waived per ACI 9.6.1.3 ✓

Shrinkage and Temperature Steel (ACI 318-14 13.2.8.1, 7.6.4.1, 24.4.3.2, 24.4.3.3)

$$\rho_{ST_prov} = \frac{A_{ST}}{t\, s_{ST}} = \frac{(0.4 \text{ in}^2 / \text{in})}{(12 \text{ in})(12 \text{ in})} = 0.0028$$

$$\rho_{ST_prov} = \frac{A_{ST}}{t\, s_{ST}} = \frac{(0.4 \text{ in}^2 / \text{in})}{(12 \text{ in})(12 \text{ in})} = 0.0028$$

$$\frac{0.0018 (60000)}{f_y} = \frac{0.0018 (60000)}{(60000 \text{ psi})} = 0.0018$$

ρ_{ST_min} = 0.0018

ρ_{ST_prov} = 0.0028 ≥ ρ_{ST_min} = 0.0018 ✓

18 inch limit governs

s_{ST_max} = 18 in

s_{ST} = 12 in ≤ s_{ST_max} = 18 in ✓

Development Check (ACI 318-14 13.2.8.1, 25.4.2.3, 25.4.10)

$$\frac{M_u}{\phi M_n} = \frac{(0.12 \text{ ft·k / ft})}{(8.6 \text{ ft·k / ft})} = 0.0141$$ (ratio to represent excess reinforcement)

ψ_t = 1.0 (12 inches or less cast below - 9.50 inches)

ψ_e = 1.0 (bar not epoxy coated)

ψ_s = 0.80 (bars are #6 or smaller)

λ = 1.0 (normal weight concrete)

s / 2 = (12 in) / 2 = 6 in

cover + d_b / 2 = (2 in) + (0.5 in) / 2 = 2.25 in

c_b = 2.25 in (lesser of half spacing, ctr to surface)

K_{tr} = 0.0 (no transverse reinforcement)

$$\frac{c_b + K_{tr}}{d_b} = \frac{(2.25 \text{ in}) + (0.0)}{(0.5 \text{ in})} = 4.50$$

$$l_d = \left(\frac{3.}{40} \frac{f_y}{\lambda \sqrt{F'_c}} \frac{\psi_t \psi_e \psi_s}{2.5} \right) d_b = \left[\frac{3.}{40} \frac{(60000 \text{ psi})}{(1.0) \sqrt{3000 \text{ psi}}} \frac{(1.0)(1.0)(0.80)}{2.5} \right] (0.5 \text{ in}) = 13.15 \text{ in}$$

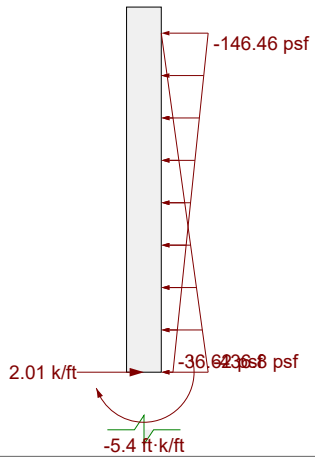
Factoring l_d by the excess reinforcement ratio (0.0141) per 25.4.10: l_d = 0.18 in

12 inch minimum controls

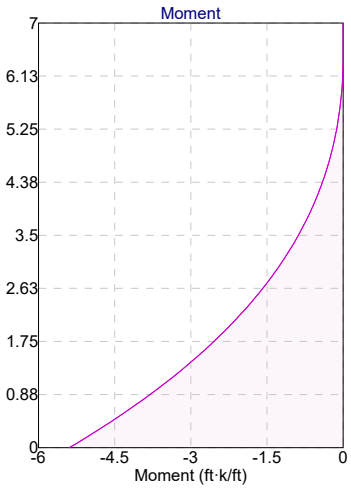
l_{d_prov} = 38.04 in ≥ l_d = 12 in ✓

Stem Forces [1.2D + 0.5L + 1.6H + 1.0E]

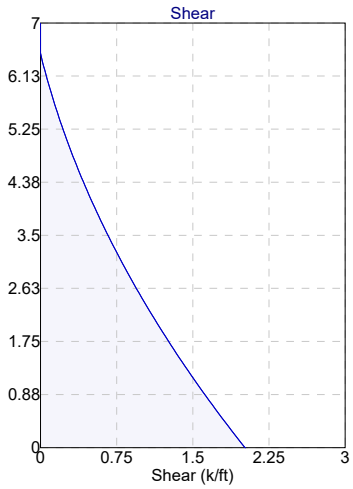
Stem Internal Forces



Stem Internal Forces



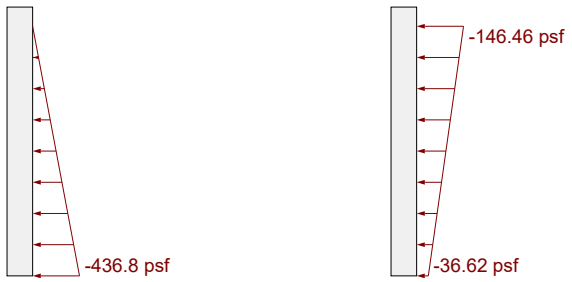
Stem Internal Forces



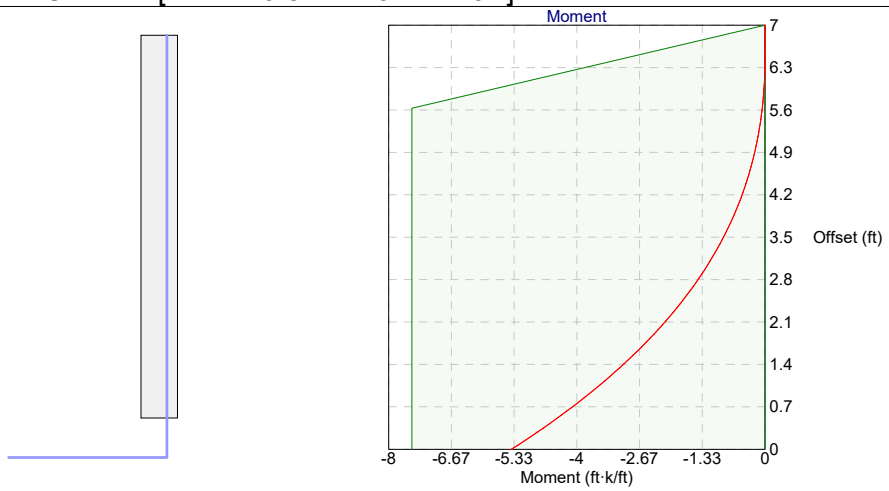
Stem Joint Force Transfer

Location	Force
@ stem base	2.01 k/ft

Stem Internal Forces



Stem Moment Checks [1.2D + 0.5L + 1.6H + 1.0E]



Check (ACI 318-14 11.5.5.1b) @ 0 ft from base

ϕM_n = 7.51 ft·k / ft ≥ M_u = 5.4 ft·k / ft ✓

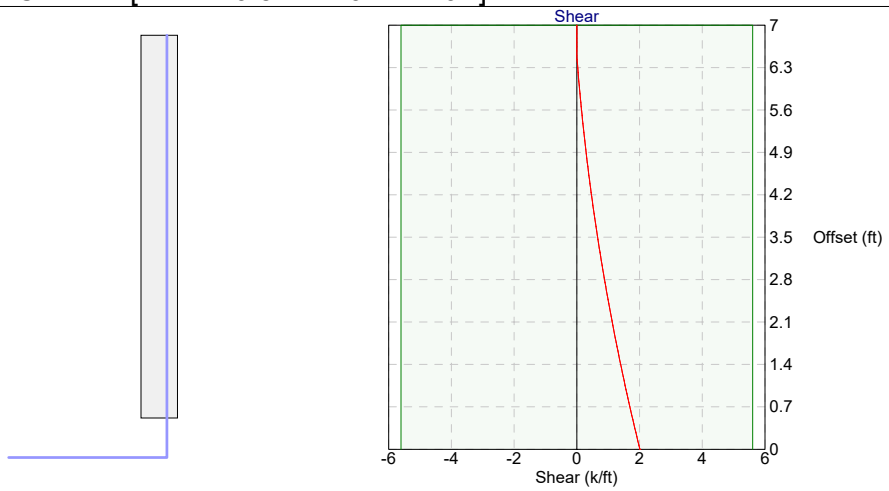
Check (ACI 318-14 11.5.5.1b) @ 5.63 ft from base

ϕM_n = 7.51 ft·k / ft ≥ M_u = 0.06 ft·k / ft ✓

Check (ACI 318-14 11.5.5.1b) @ 5.66 ft from base

ϕM_n = 7.37 ft·k / ft ≥ M_u = 0.06 ft·k / ft ✓

Stem Shear Checks [1.2D + 0.5L + 1.6H + 1.0E]



Shear Check (ACI 318-14 11.5.5.1c) @ 0 ft from base

ϕV_n = 5.61 k / ft ≥ V_u = 2.01 k / ft ✓

Stem Miscellaneous Checks [1.2D + 0.5L + 1.6H + 1.0E]

Minimum Steel Check (ACI 318-14 9.6.1) @ 0 ft from base [Stem in negative flexure]

$\phi M_n = 7.51$ ft·k / ft $\geq (4 / 3) M_u = [4 / 3] (5.4$ ft·k / ft$) = 7.2$ ft·k / ft

Check is waived per ACI 9.6.1.3 ✓

Minimum Steel Check (ACI 318-14 9.6.1) @ 7 ft from base [Stem in negative flexure]

$\phi M_n = 0$ ft·k / ft $\geq (4 / 3) M_u = [4 / 3] (0$ ft·k / ft$) = 0$ ft·k / ft

Check is waived per ACI 9.6.1.3 ✓

Maximum Steel Check (ACI 318-14 9.3.3.1) @ 0 ft from base [Stem in negative flexure]

$\beta_1 = 0.850$ ($F'_c \leq 4000$ psi)

$$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0.03 \text{ in}^2 / \text{in}) (60000 \text{ psi})}{0.85 (3000 \text{ psi})} = 0.61 \text{ in}$$

$$\varepsilon_t = 0.003 \left(\frac{d}{a / \beta_1} - 1 \right) = 0.003 \left[\frac{(5.69 \text{ in})}{(0.61 \text{ in}) / (0.850)} - 1 \right] = 0.0209$$

$\varepsilon_t = 0.0209 \geq 0.004$ ✓

Maximum Steel Check (ACI 318-14 9.3.3.1) @ 7 ft from base [Stem in negative flexure]

$\beta_1 = 0.850$ ($F'_c \leq 4000$ psi)

$$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0.03 \text{ in}^2 / \text{in}) (60000 \text{ psi})}{0.85 (3000 \text{ psi})} = 0.61 \text{ in}$$

$$\varepsilon_t = 0.003 \left(\frac{d}{a / \beta_1} - 1 \right) = 0.003 \left[\frac{(5.69 \text{ in})}{(0.61 \text{ in}) / (0.850)} - 1 \right] = 0.0209$$

$\varepsilon_t = 0.0209 \geq 0.004$ ✓

Wall Horizontal Steel (ACI 318-14 11.6.1, 11.7.3)

$$\rho_t = \frac{A_{s_horz} / s_{horz}}{t} = \frac{(0.2 \text{ in}^2) / (12 \text{ in})}{(8 \text{ in})} = 0.0021$$

$\rho_{t_min} = 0.0020$ (bars No. 5 or less, not less than 60 ksi)

$\rho_t = 0.0021 \geq \rho_{t_min} = 0.0020$ ✓

$3 h = 3 (8 \text{ in}) = 24$ in

18 inch limit governs

$s_{horz} = 12$ in $\leq s_{horz_max} = 18$ in ✓

Development Check (ACI 318-14 11.7.1.2, 25.4.2.3, 25.4.10)

$\frac{M_u}{\phi M_n} = \frac{(5.4 \text{ ft·k / ft})}{(7.51 \text{ ft·k / ft})} = 0.7185$ (ratio to represent excess reinforcement)

$\psi_e = 1.0$ (uncoated hooked bars)

$\psi_c = 0.70$ (based on side cover and extension cover)

$\psi_r = 1.0$ (no confining reinforcement)

$\lambda = 1.0$ (normal weight concrete)

$$l_{dh} = \left(\frac{f_y \psi_e \psi_c \psi_r}{50 \lambda \sqrt{F'_c}} \right) d_b = \left[\frac{(60000 \text{ psi}) (1.0) (0.70) (1.0)}{50 (1.0) \sqrt{3000 \text{ psi}}} \right] (0.63 \text{ in}) = 9.59 \text{ in}$$

Factoring l_{dh} by the excess reinforcement ratio (0.7185) per 25.4.10: $l_{dh} = 6.89$ in

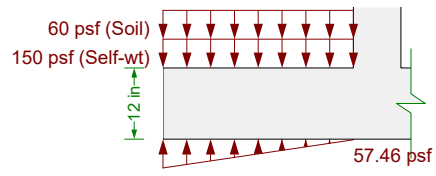
$8 d_b = 8 (0.63 \text{ in}) = 5.0$ (minimum limit, does not control)

$l_{dh_prov} = 9$ in $\geq l_{dh} = 6.89$ in ✓

Toe Checks [1.4D]

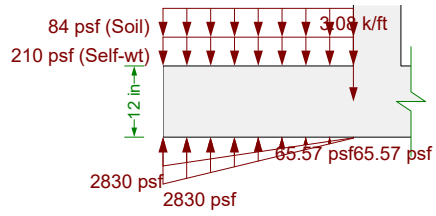
Toe Unfactored Loads

Unfactored Loads



Toe Factored Loads

1.4D



Controlling Moment

Design moment M_u for toe need not exceed moment at stem base:

M_{toe} = 5.75 ft·k / ft ≥ M_{stem} = - 0 ft·k / ft

M_u = - 0 ft·k / ft (stem base moment controls)

Flexure Check (ACI 318-14 13.3.2.1, 7.5.2.1, »22.3, »22.2, 7.5.1.1a)

$$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0.03 \text{ in}^2 / \text{in})(60000 \text{ psi})}{0.85 (3000 \text{ psi})} = 0.61 \text{ in}$$

$$\phi M_n = \phi A_s f_y (d - a / 2) = (0.90)(0.03 \text{ in}^2 / \text{in})(60000 \text{ psi})[(8.69 \text{ in}) - (0.61 \text{ in}) / 2] = 11.7 \text{ ft·k / ft}$$

ϕM_n = 11.7 ft·k / ft ≥ M_u = - 0 ft·k / ft ✓

Shear Check (ACI 318-14 13.3.2.1, 7.5.3.1, »22.5.1, »22.5.5, 7.5.1.1b)

λ = 1.0 (normal weight concrete)

$$V_c = 2 \lambda \sqrt{F'_c} d = 2 (1.0) \sqrt{3000 \text{ psi}} (8.69 \text{ in}) = 11.42 \text{ k / ft}$$

$$\phi V_n = \phi V_c = (0.750)(11.42 \text{ k / ft}) = 8.57 \text{ k / ft}$$

ϕV_n = 8.57 k / ft ≥ V_u = 2.97 k / ft ✓

Minimum Strain Check (ACI 318-14 13.3.2.1, 7.3.3.1)

β_1 = 0.850 ($F'_c \le 4000$ psi)

$$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0.03 \text{ in}^2 / \text{in})(60000 \text{ psi})}{0.85 (3000 \text{ psi})} = 0.61 \text{ in}$$

$$\varepsilon_t = 0.003 \left(\frac{d}{a / \beta_1} - 1 \right) = 0.003 \left[\frac{(8.69 \text{ in})}{(0.61 \text{ in}) / (0.850)} - 1 \right] = 0.0334$$

ε_t = 0.0334 ≥ 0.004 ✓

Minimum Steel Check (ACI 318-14 13.3.2.1, 9.6.1)

ϕM_n = 11.7 ft·k / ft ≥ (4 / 3) M_u = [4 / 3] (- 0 ft·k / ft) = - 0 ft·k / ft

Check is waived per ACI 9.6.1.3 ✓

Shrinkage and Temperature Steel (ACI 318-14 13.2.8.1, 7.6.4.1, 24.4.3.2, 24.4.3.3)

$$\rho_{ST_prov} = \frac{A_{ST}}{t\, s_{ST}} = \frac{(0.4 \text{ in}^2 / \text{in})}{(12 \text{ in})(12 \text{ in})} = 0.0028$$

$$\rho_{ST_prov} = \frac{A_{ST}}{t\, s_{ST}} = \frac{(0.4 \text{ in}^2 / \text{in})}{(12 \text{ in})(12 \text{ in})} = 0.0028$$

$$\frac{0.0018 (60000)}{f_y} = \frac{0.0018 (60000)}{(60000 \text{ psi})} = 0.0018$$

ρ_{ST_min} = 0.0018

ρ_{ST_prov} = 0.0028 ≥ ρ_{ST_min} = 0.0018 ✓

18 inch limit governs

s_{ST_max} = 18 in

s_{ST} = 12 in ≤ s_{ST_max} = 18 in ✓

Development Check (ACI 318-14 13.2.8.1, 25.4.2.3, 25.4.10)

$$\frac{M_u}{\phi M_n} = \frac{(- 0 \text{ ft·k / ft})}{(11.7 \text{ ft·k / ft})} = - 0.0$$ (ratio to represent excess reinforcement)

ψ_e = 1.0 (uncoated hooked bars)

ψ_c = 0.70 (based on side cover and extension cover)

ψ_r = 1.0 (no confining reinforcement)

λ = 1.0 (normal weight concrete)

$$l_{dh} = \left(\frac{f_y \psi_e \psi_c \psi_r}{50 \lambda \sqrt{F'_c}} \right) d_b = \left[\frac{(60000 \text{ psi})(1.0)(0.70)(1.0)}{50 (1.0) \sqrt{3000 \text{ psi}}} \right] (0.63 \text{ in}) = 9.59 \text{ in}$$

Factoring l_{dh} by the excess reinforcement ratio (- 0.0000) per 25.4.10: l_{dh} = - 0 in

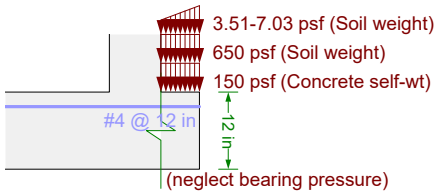
8 d_b = 8 (0.63 in) = 5.0 (minimum limit, does not control)

6 inch minimum controls

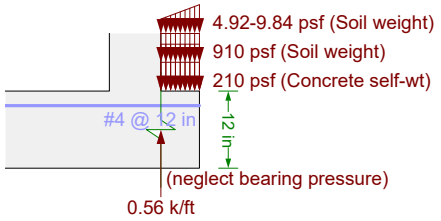
l_{dh_prov} = 6 in ≥ l_{dh} = 6 in ✓

Heel Checks [1.4D]

Heel Unfactored Loads



Heel Factored Loads



Controlling Moment

Design moment M_u for heel need not exceed moment at stem base:

M_{heel} = 0.14 ft·k / ft ≥ M_{stem} = - 0 ft·k / ft

M_u = - 0 ft·k / ft (stem base moment controls)

Flexure Check (ACI 318-14 13.3.2.1, 7.5.2.1, »22.3, »22.2, 7.5.1.1a)

$$a = \frac{A_s f_y}{0.85 f'_c} = \frac{(0.02 \text{ in}^2 / \text{in})(60000 \text{ psi})}{0.85 (3000 \text{ psi})} = 0.39 \text{ in}$$

$\phi M_n = \phi A_s f_y (d - a / 2)$ = (0.90) (0.02 in² / in) (60000 psi) [(9.75 in) - (0.39 in) / 2] = 8.6 ft·k / ft

ϕM_n = 8.6 ft·k / ft ≥ M_u = - 0 ft·k / ft ✓

Shear Check (ACI 318-14 13.3.2.1, 7.5.3.1, »22.5.1, »22.5.5, 7.5.1.1b)

λ = 1.0 (normal weight concrete)

$V_c = 2 \lambda \sqrt{f'_c}\, d$ = 2 (1.0) $\sqrt{3000 \text{ psi}}$ (9.75 in) = 12.82 k / ft

$\phi V_n = \phi V_c$ = (0.750) (12.82 k / ft) = 9.61 k / ft

ϕV_n = 9.61 k / ft ≥ V_u = 0.56 k / ft ✓

Minimum Strain Check (ACI 318-14 13.3.2.1, 7.3.3.1)

β_1 = 0.850 ($f'_c \le 4000$ psi)

$$a = \frac{A_s f_y}{0.85 f'_c} = \frac{(0.02 \text{ in}^2 / \text{in})(60000 \text{ psi})}{0.85 (3000 \text{ psi})} = 0.39 \text{ in}$$

$$\varepsilon_t = 0.003 \left(\frac{d}{a / \beta_1} - 1 \right) = 0.003 \left[\frac{(9.75 \text{ in})}{(0.39 \text{ in}) / (0.850)} - 1 \right] = 0.0604$$

ε_t = 0.0604 ≥ 0.004 ✓

Minimum Steel Check (ACI 318-14 13.3.2.1, 9.6.1)

ϕM_n = 8.6 ft·k / ft ≥ (4 / 3) M_u = [4 / 3] (- 0 ft·k / ft) = - 0 ft·k / ft

Check is waived per ACI 9.6.1.3 ✓

Shrinkage and Temperature Steel (ACI 318-14 13.2.8.1, 7.6.4.1, 24.4.3.2, 24.4.3.3)

$$\rho_{ST_prov} = \frac{A_{ST}}{t\, s_{ST}} = \frac{(0.4 \text{ in}^2 / \text{in})}{(12 \text{ in})(12 \text{ in})} = 0.0028$$

$$\rho_{ST_prov} = \frac{A_{ST}}{t\, s_{ST}} = \frac{(0.4 \text{ in}^2 / \text{in})}{(12 \text{ in})(12 \text{ in})} = 0.0028$$

$$\frac{0.0018 (60000)}{f_y} = \frac{0.0018 (60000)}{(60000 \text{ psi})} = 0.0018$$

ρ_{ST_min} = 0.0018

ρ_{ST_prov} = 0.0028 ≥ ρ_{ST_min} = 0.0018 ✓

18 inch limit governs

s_{ST_max} = 18 in

s_{ST} = 12 in ≤ s_{ST_max} = 18 in ✓

Development Check (ACI 318-14 13.2.8.1, 25.4.2.3, 25.4.10)

$\frac{M_u}{\phi M_n} = \frac{(- 0 \text{ ft·k / ft})}{(8.6 \text{ ft·k / ft})}$ = - 0.0 (ratio to represent excess reinforcement)

ψ_t = 1.0 (12 inches or less cast below - 9.50 inches)

ψ_e = 1.0 (bar not epoxy coated)

ψ_s = 0.80 (bars are #6 or smaller)

λ = 1.0 (normal weight concrete)

s / 2 = (12 in) / 2 = 6 in

cover + d_b / 2 = (2 in) + (0.5 in) / 2 = 2.25 in

c_b = 2.25 in (lesser of half spacing, ctr to surface)

K_{tr} = 0.0 (no transverse reinforcement)

$$\frac{c_b + K_{tr}}{d_b} = \frac{(2.25 \text{ in}) + (0.0)}{(0.5 \text{ in})} = 4.50$$

$$l_d = \left(\frac{3.}{40} \frac{f_y}{\lambda \sqrt{f'_c}} \frac{\psi_t \psi_e \psi_s}{2.5} \right) d_b = \left[\frac{3.}{40} \frac{(60000 \text{ psi})}{(1.0) \sqrt{3000 \text{ psi}}} \frac{(1.0)(1.0)(0.80)}{2.5} \right] (0.5 \text{ in}) = 13.15 \text{ in}$$

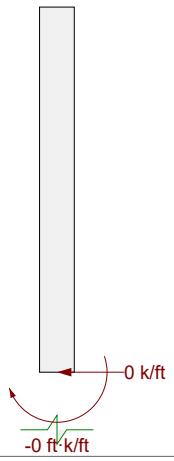
Factoring l_d by the excess reinforcement ratio (- 0.0000) per 25.4.10: l_d = - 0 in

12 inch minimum controls

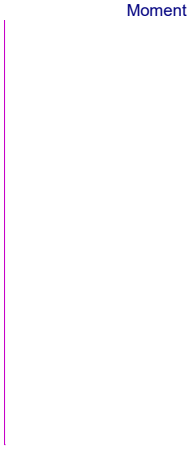
l_{d_prov} = 38.04 in ≥ l_d = 12 in ✓

Stem Forces [1.4D]

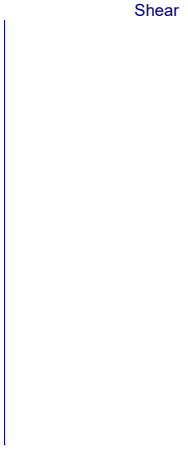
Stem Internal Forces



Stem Internal Forces



Stem Internal Forces

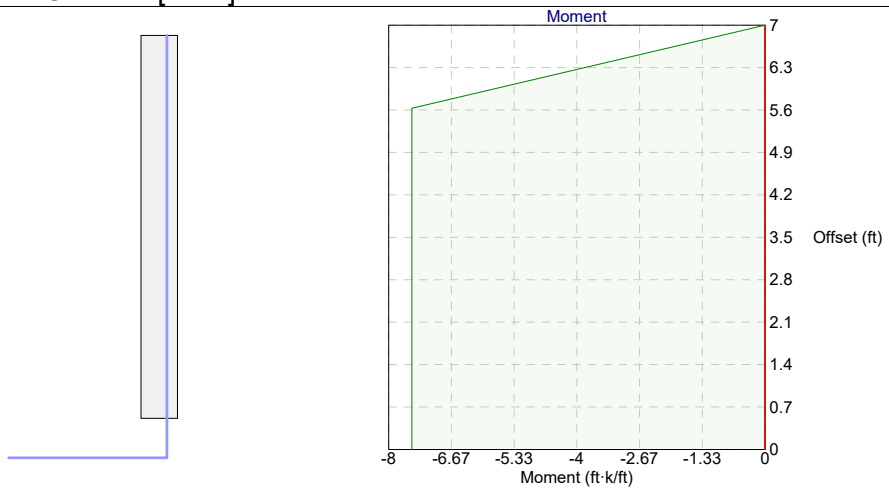


Stem Joint Force Transfer

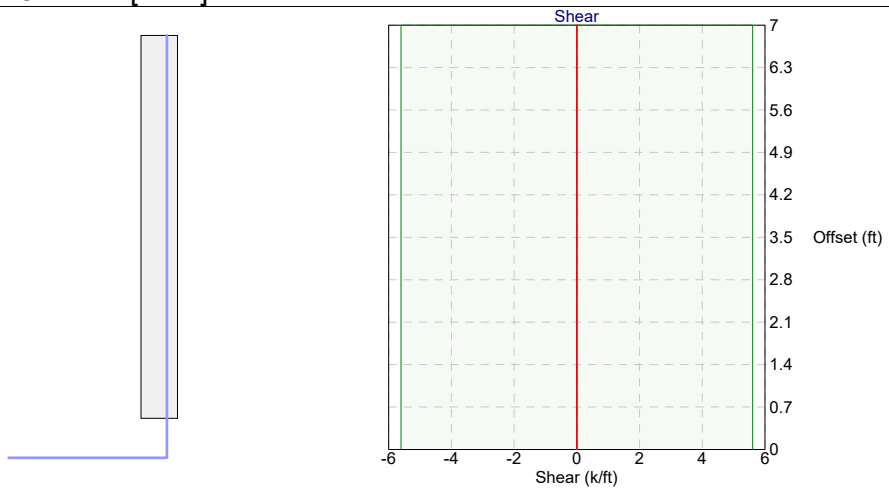
Location	Force
@ stem base	0 k/ft

Stem Internal Forces

Stem Moment Checks [1.4D]



Stem Shear Checks [1.4D]



Stem Miscellaneous Checks [1.4D]

Minimum Steel Check (ACI 318-14 9.6.1) @ 0 ft from base [Stem in negative flexure]

$\phi M_n = 7.51$ ft·k / ft $\geq (4 / 3) M_u = [4 / 3] (0$ ft·k / ft$) = 0$ ft·k / ft

Check is waived per ACI 9.6.1.3 ✓

Minimum Steel Check (ACI 318-14 9.6.1) @ 7 ft from base [Stem in negative flexure]

$\phi M_n = 0$ ft·k / ft $\geq (4 / 3) M_u = [4 / 3] (0$ ft·k / ft$) = 0$ ft·k / ft

Check is waived per ACI 9.6.1.3 ✓

Maximum Steel Check (ACI 318-14 9.3.3.1) @ 0 ft from base [Stem in negative flexure]

$\beta_1 = 0.850$ ($F'_c \leq 4000$ psi)

$$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0.03 \text{ in}^2 / \text{in}) (60000 \text{ psi})}{0.85 (3000 \text{ psi})} = 0.61 \text{ in}$$

$$\varepsilon_t = 0.003 \left(\frac{d}{a / \beta_1} - 1 \right) = 0.003 \left[\frac{(5.69 \text{ in})}{(0.61 \text{ in}) / (0.850)} - 1 \right] = 0.0209$$

$\varepsilon_t = 0.0209 \geq 0.004$ ✓

Maximum Steel Check (ACI 318-14 9.3.3.1) @ 7 ft from base [Stem in negative flexure]

$\beta_1 = 0.850$ ($F'_c \leq 4000$ psi)

$$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0.03 \text{ in}^2 / \text{in}) (60000 \text{ psi})}{0.85 (3000 \text{ psi})} = 0.61 \text{ in}$$

$$\varepsilon_t = 0.003 \left(\frac{d}{a / \beta_1} - 1 \right) = 0.003 \left[\frac{(5.69 \text{ in})}{(0.61 \text{ in}) / (0.850)} - 1 \right] = 0.0209$$

$\varepsilon_t = 0.0209 \geq 0.004$ ✓

Wall Horizontal Steel (ACI 318-14 11.6.1, 11.7.3)

$$\rho_t = \frac{A_{s_horz} / s_{horz}}{t} = \frac{(0.2 \text{ in}^2) / (12 \text{ in})}{(8 \text{ in})} = 0.0021$$

$\rho_{t_min} = 0.0020$ (bars No. 5 or less, not less than 60 ksi)

$\rho_t = 0.0021 \geq \rho_{t_min} = 0.0020$ ✓

$3 h = 3 (8 \text{ in}) = 24$ in

18 inch limit governs

$s_{horz} = 12$ in $\leq s_{horz_max} = 18$ in ✓

Development Check (ACI 318-14 11.7.1.2, 25.4.2.3, 25.4.10)

$$\frac{M_u}{\phi M_n} = \frac{(0 \text{ ft·k / ft})}{(7.51 \text{ ft·k / ft})} = 0.0$$ (ratio to represent excess reinforcement)

$\psi_e = 1.0$ (uncoated hooked bars)

$\psi_c = 0.70$ (based on side cover and extension cover)

$\psi_r = 1.0$ (no confining reinforcement)

$\lambda = 1.0$ (normal weight concrete)

$$l_{dh} = \left(\frac{f_y \psi_e \psi_c \psi_r}{50 \lambda \sqrt{F'_c}} \right) d_b = \left[\frac{(60000 \text{ psi}) (1.0) (0.70) (1.0)}{50 (1.0) \sqrt{3000 \text{ psi}}} \right] (0.63 \text{ in}) = 9.59 \text{ in}$$

Factoring l_{dh} by the excess reinforcement ratio (0.0000) per 25.4.10: $l_{dh} = 0$ in

$8 d_b = 8 (0.63 \text{ in}) = 5.0$ (minimum limit, does not control)

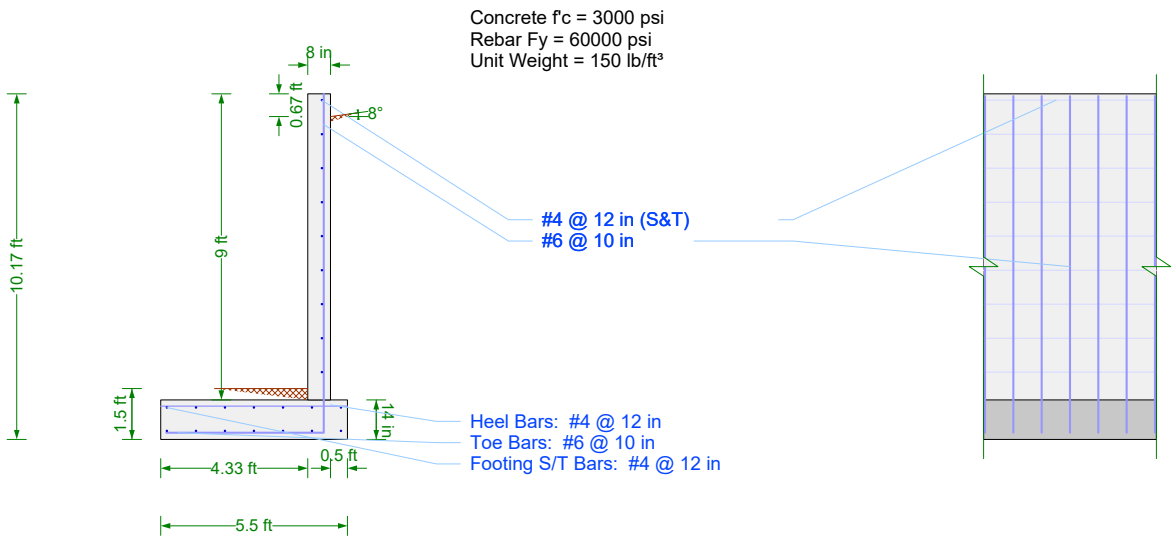
6 inch minimum controls

$l_{dh_prov} = 9$ in $\geq l_{dh} = 6$ in ✓

JOSEPH LUCIA
LUCIA ENGINEERING, INC.

Cantilever Wall 8' Tall

Design Detail



Check Summary

Ratio	Check	Provided	Required	Combination
----- Stability Checks -----				
✓ 0.964	Overturning	1.25	1.20	1.0D + 1.0L + 1.0H + 0.7E
✓ 0.946	Bearing Pressure	2500 psf	2366 psf	1.0D + 1.0L + 1.0H + 0.7E
✓ 0.577	Bearing Eccentricity	19.03 in	32.98 in	1.0D + 1.0L + 1.0H + 0.7E
----- Toe Checks -----				
✓ 0.349	Shear	11.46 k/ft	4 k/ft	1.2D + 0.5L + 1.6H + 1.0E
✓ 0.431	Moment	26.39 ft·k/ft	11.37 ft·k/ft	1.2D + 0.5L + 1.6H + 1.0E
✓ 0.156	Min Strain	0.0256	0.0040	1.2D + 1.6L + 1.6H
✓ 0.000	Min Steel	0.04 in²	0 in²	1.2D + 1.6L + 1.6H
✓ 1.000	Development	6 in	6 in	1.2D + 1.6L + 1.6H
✓ 0.667	S&T Max Spacing	12 in	18 in	1.2D + 1.6L + 1.6H
✓ 0.756	S&T Min Rho	0.0024	0.0018	1.2D + 1.6L + 1.6H
----- Heel Checks -----				
✓ 0.061	Shear	11.58 k/ft	0.71 k/ft	1.4D
✓ 0.015	Moment	10.4 ft·k/ft	0.15 ft·k/ft	1.2D + 1.6L + 1.6H
✓ 0.054	Min Strain	0.0734	0.0040	1.2D + 1.6L + 1.6H
✓ 0.000	Min Steel	0.02 in²	0 in²	1.2D + 1.6L + 1.6H
✓ 0.207	Development	57.96 in	12 in	1.2D + 1.6L + 1.6H
✓ 0.667	S&T Max Spacing	12 in	18 in	1.2D + 1.6L + 1.6H
✓ 0.756	S&T Min Rho	0.0024	0.0018	1.2D + 1.6L + 1.6H
----- Stem Checks -----				
✓ 0.937	Moment	12.14 ft·k/ft	11.37 ft·k/ft	1.2D + 0.5L + 1.6H + 1.0E
✓ 0.597	Shear	5.55 k/ft	3.31 k/ft	1.2D + 0.5L + 1.6H + 1.0E
✓ 0.369	Max Steel	0.0109	0.0040	1.2D + 1.6L + 1.6H
✓ 0.426	Min Steel	0.04 in²/in	0.02 in²/in	1.2D + 0.5L + 1.6H + 1.0E
✓ 0.898	Base Development	12 in	10.78 in	1.2D + 0.5L + 1.6H + 1.0E
✓ 0.000	Horz Bar Rho	0.0000	0.0000	1.2D + 1.6L + 1.6H
✓ 0.667	Horz Bar Spacing	12 in	18 in	1.2D + 1.6L + 1.6H

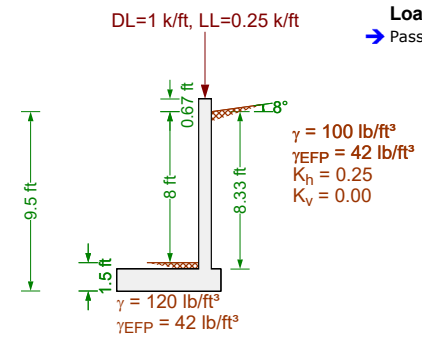
Criteria

Use basic criteria from common proje...	Yes
Building Code	IBC 2021
Concrete Load Combs	IBC 2021 (Strength)
Masonry Load Combs	ASCE 7-16 (ASD)
Stability Load Combs	IBC Retaining Wall St...
Apply Sds Factor to Seismic Combin...	No
Restrained Against Sliding	Yes
Neglect Bearing At Heel	Yes
Use Vert. Comp. for OT	No
Use Vert. Comp. for Sliding	No
Use Vert. Comp. for Bearing	Yes
Use Surcharge for Sliding & OT	Yes
Use Surcharge for Bearing	Yes
Neglect Soil Over Toe	No
Neglect Backfill Wt. for Coulomb	No
Factor Soil Weight As Dead	Yes
Use Passive Force for OT	Yes
Assume Pressure To Top	Yes
Extend Backfill Pressure To Key Bott...	No
Use Toe Passive Pressure for Bearing	No
Required F.S. for OT	1.50
Required F.S. for Sliding	1.50
Has Different Safety Factors for Seis...	Yes
Seismic F.S. for OT	1.20
Seismic F.S. for Sliding	1.20
Allowable Bearing Pressure	2500 psf
Req'd Bearing Location	Over footing
Wall Friction Angle	25°
Friction Coefficent	0.35
Soil Reaction Modulus	172800 lb/ft³

JOSEPH LUCIA
LUCIA ENGINEERING, INC.

Cantilever Wall 4' Tall

Loads



Loading Options/Assumptions

➔ Passive pressure neglects top 0 ft of soil.

Load Combinations

IBC 2018 (Strength)

1.2D + 1.6L + 1.6H
1.2D + 1.6L + 0.9H
1.2D + 0.5L + 1.6H + 1.0E
1.2D + 0.5L + 1.6H
1.2D + 0.5L + 0.9H + 1.0E
1.2D + 0.5L + 0.9H
1.2D + 1.6H + 1.0E
1.2D + 1.6H
1.2D + 0.9H + 1.0E
1.2D + 0.9H
0.9D + 1.6H + 1.0E
0.9D + 1.6H
0.9D + 0.9H + 1.0E
0.9D + 0.9H
1.4D

Strength Check Results Summary

Load Combination	Stem M-applied (ft·k/ft)	Stem M-allow (ft·k/ft)	Stem V-applied (k/ft)	Stem V-allow (k/ft)	Stem Min. ld (in)	Stem Actual ld (in)	Stem Min. strain	Stem Actual strain	Stem Min. steel (in²/in)
1.2D + 1.6L + 1.6H	6.48	12.14	2.33	5.55	6.14	12	0.0040	0.0109	0
1.2D + 1.6L + 0.9H	3.65	12.14	1.31	5.55	6	12	0.0040	0.0109	0
1.2D + 0.5L + 1.6H + 1.0E	11.37	12.14	3.31	5.55	10.78	12	0.0040	0.0109	0.02
1.2D + 0.5L + 1.6H	6.48	12.14	2.33	5.55	6.14	12	0.0040	0.0109	0
1.2D + 0.5L + 0.9H + 1.0E	8.54	12.14	2.29	5.55	8.09	12	0.0040	0.0109	0
1.2D + 0.5L + 0.9H	3.65	12.14	1.31	5.55	6	12	0.0040	0.0109	0
1.2D + 1.6H + 1.0E	11.37	12.14	3.31	5.55	10.78	12	0.0040	0.0109	0.02
1.2D + 1.6H	6.48	12.14	2.33	5.55	6.14	12	0.0040	0.0109	0
1.2D + 0.9H + 1.0E	8.54	12.14	2.29	5.55	8.09	12	0.0040	0.0109	0
1.2D + 0.9H	3.65	12.14	1.31	5.55	6	12	0.0040	0.0109	0
0.9D + 1.6H + 1.0E	11.37	12.14	3.31	5.55	10.78	12	0.0040	0.0109	0.02
0.9D + 1.6H	6.48	12.14	2.33	5.55	6.14	12	0.0040	0.0109	0
0.9D + 0.9H + 1.0E	8.54	12.14	2.29	5.55	8.09	12	0.0040	0.0109	0
0.9D + 0.9H	3.65	12.14	1.31	5.55	6	12	0.0040	0.0109	0
1.4D	0	0	0	0	6	12	0.0040	0.0109	0

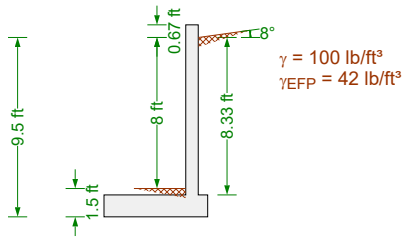
Load Combination	Stem Actual steel (in²/in)	Heel M-applied (ft·k/ft)	Heel M-allow (ft·k/ft)	Heel V-applied (k/ft)	Heel V-allow (k/ft)	Toe M-applied (ft·k/ft)	Toe M-allow (ft·k/ft)	Toe V-applied (k/ft)	Toe V-allow (k/ft)
1.2D + 1.6L + 1.6H	0.04	0.15	10.4	0.61	11.58	11.98	26.39	3.67	11.46
1.2D + 1.6L + 0.9H	0.04	0.15	10.4	0.61	11.58	11.98	26.39	3.67	11.46
1.2D + 0.5L + 1.6H + 1.0E	0.04	0.15	10.4	0.61	11.58	13.01	26.39	4	11.46
1.2D + 0.5L + 1.6H	0.04	0.15	10.4	0.61	11.58	11.1	26.39	3.4	11.46
1.2D + 0.5L + 0.9H + 1.0E	0.04	0.15	10.4	0.61	11.58	13.01	26.39	4	11.46
1.2D + 0.5L + 0.9H	0.04	0.15	10.4	0.61	11.58	11.1	26.39	3.4	11.46
1.2D + 1.6H + 1.0E	0.04	0.15	10.4	0.61	11.58	12.61	26.39	3.87	11.46
1.2D + 1.6H	0.04	0.15	10.4	0.61	11.58	10.71	26.39	3.27	11.46
1.2D + 0.9H + 1.0E	0.04	0.15	10.4	0.61	11.58	12.61	26.39	3.87	11.46
1.2D + 0.9H	0.04	0.15	10.4	0.61	11.58	10.71	26.39	3.27	11.46
0.9D + 1.6H + 1.0E	0.04	0.11	10.4	0.46	11.58	9.94	26.39	3.05	11.46
0.9D + 1.6H	0.04	0.11	10.4	0.46	11.58	8.03	26.39	2.45	11.46
0.9D + 0.9H + 1.0E	0.04	0.11	10.4	0.46	11.58	9.94	26.39	3.05	11.46
0.9D + 0.9H	0.04	0.11	10.4	0.46	11.58	8.03	26.39	2.45	11.46
1.4D	0.04	0.18	10.4	0.71	11.58	12.49	26.39	3.82	11.46

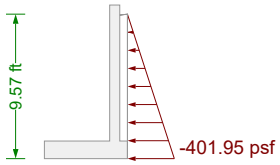
Stability Check Results Summary

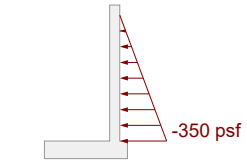
Load Combination	Overturning Moment (ft·k/ft)	Resisting Moment (ft·k/ft)	Overturning F.S.	Overturning F.S. Req'd	Overturning F.S. Req'd Seismic	Sliding Force (lb/in)	Resisting Force (lb/in)	Sliding F.S.
1.0D + 1.0L + 1.0H + 0.7E	6.14	14.1	2.298	1.500	1.200	160.3	124.2	0.775
1.0D + 1.0L + 1.0H	6.14	14.1	2.298	1.500	1.200	160.3	112	0.699
1.0D + 1.0H + 0.7E	6.14	14.1	2.298	1.500	1.200	160.3	116.9	0.730
1.0D + 1.0H	6.14	14.1	2.298	1.500	1.200	160.3	104.7	0.653

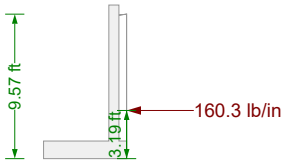
Load Combination	Sliding F.S. Req'd	Sliding F.S. Req'd Seismic	Bearing Pressure Actual (psf)	Bearing Pressure Allowable (psf)	Bearing Eccentricity Actual (in)	Bearing Eccentricity Allowable (in)	Wall Top Actual Deflection (in)
1.0D + 1.0L + 1.0H + 0.7E	1.500	1.200	2366	2500	19.03	32.98	0.46
1.0D + 1.0L + 1.0H	1.500	1.200	2124	2500	19.03	32.98	0.46
1.0D + 1.0H + 0.7E	1.500	1.200	2222	2500	19.03	32.98	0.46
1.0D + 1.0H	1.500	1.200	1981	2500	19.03	32.98	0.46

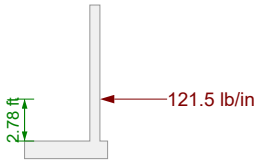
Backfill Pressure











Lateral Earth Pressure

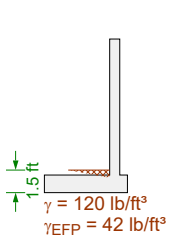
Equivalent Fluid Pressure

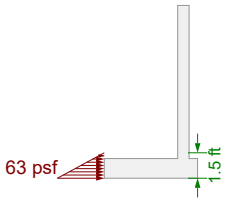
$\sigma_h = H\,\gamma_{fluid}$ = (9.57 ft)(42 lb / ft³) = 402 psf

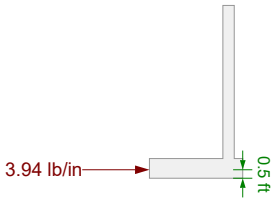
Lateral Earth Pressure (stem only)

$\sigma_h = H\,\gamma_{fluid}$ = (8.33 ft)(42 lb / ft³) = 350 psf

Passive Pressure





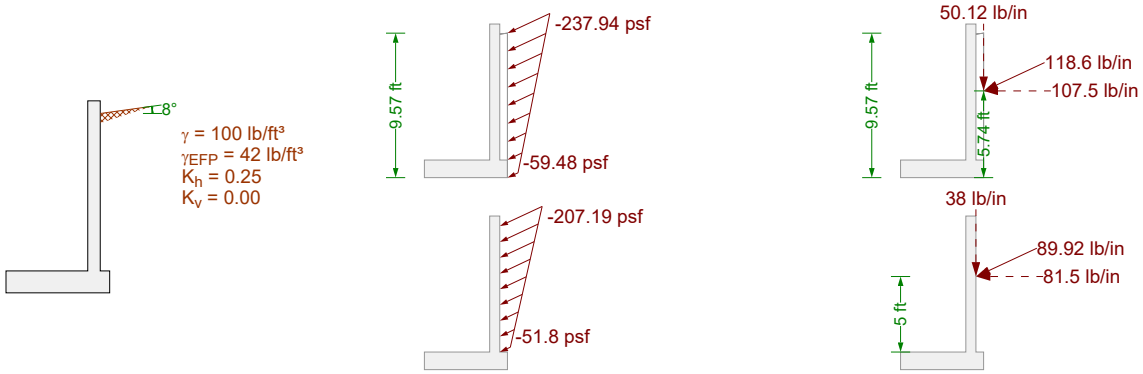


Lateral Earth Pressure

Equivalent Fluid Pressure

$\sigma_h = H\,\gamma_{fluid} = (1.5\text{ ft})(42\text{ lb / ft}^3) = 63\text{ psf}$

Seismic Pressure



Seismic Pressure

Dynamic + static force (Mononobe - Okabe equation)

$$\theta' = \text{atan}\left(\frac{k_h}{1 - k_v}\right) = \arctan\left[\frac{(0.250)}{1 - (0.0)}\right] = 14.04°$$

$$K_{ae} = \frac{\sin^2(\beta + \phi - \theta')}{\cos(\theta')\sin^2(\beta)\sin(\beta - \theta' - \delta)\left[1 + \sqrt{\frac{\sin(\phi + \delta)\sin(\phi - \theta' - \alpha)}{\sin(\beta - \delta - \theta')\sin(\alpha + \beta)}}\right]^{\wedge} 2}$$

$$= \frac{\sin^2[(90°) + (30°) - (14.04°)]}{\cos((14.04°))\sin^2((90°))\sin[(90°) - (14.04°) - (25°)]\left[1 + \sqrt{\frac{\sin[(30°) + (25°)]\sin[(30°) - (14.04°) - (8°)]}{\sin[(90°) - (25°) - (14.04°)]\sin[(8°) + (90°)]}}\right]^{\wedge} 2}$$

$$= 0.6403$$

$$P_{ae} = \frac{1}{2} K_{ae}\,\gamma\, H^{\wedge} 2\,(1 - k_v) = \frac{1}{2}(0.6403)(100\text{ lb / ft}^3)(9.57\text{ ft})^{\wedge} 2\,[1 - (0.0)] = 244.4\text{ lb / in}$$

Static - only force (Coulomb equation)

$$K_a = \frac{\sin^2(\beta + \phi)}{\sin^2(\beta)\sin(\beta - \delta)\left[1 + \sqrt{\frac{\sin(\phi + \delta)\sin(\phi - \alpha)}{\sin(\beta - \delta)\sin(\alpha + \beta)}}\right]^{\wedge} 2}$$

$$= \frac{\sin^2[(90°) + (30°)]}{\sin^2((90°))\sin[(90°) - (25°)]\left[1 + \sqrt{\frac{\sin[(30°) + (25°)]\sin[(30°) - (8°)]}{\sin[(90°) - (25°)]\sin[(8°) + (90°)]}}\right]^{\wedge} 2}$$

$$= 0.3295$$

$$P_a = \frac{1}{2} K_a\,\gamma\, H^{\wedge} 2 = \frac{1}{2}(0.3295)(100\text{ lb / ft}^3)(9.57\text{ ft})^{\wedge} 2 = 125.8\text{ lb / in}$$

Net dynamic force

$$\Delta P_{ae} = P_{ae} - P_a = (244.4\text{ lb / in}) - (125.8\text{ lb / in}) = 118.6\text{ lb / in}$$

$$\alpha_P = 90° - \beta + \delta = 90° - (90°) + (25°) = 25°$$ (resultant force angle with horizontal)

To arrive at the pressure distribution illustrated above (used to determine stem moments), apply inverted triangular pressure plus a uniform portion to bring resultant to 0.6H

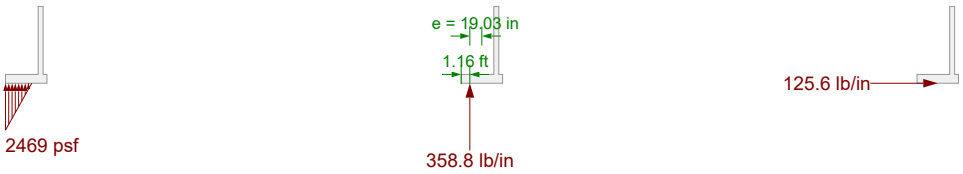
$$\sigma_{e_top} = \frac{8}{5}\frac{\Delta P_{ae}}{H} = \frac{8}{5}\frac{(118.6\text{ lb / in})}{(9.57\text{ ft})} = 237.9\text{ psf}$$

$$\sigma_{e_bot} = \frac{2}{5}\frac{\Delta P_{ae}}{H} = \frac{2}{5}\frac{(118.6\text{ lb / in})}{(9.57\text{ ft})} = 59.48\text{ psf}$$

Wall/Soil Weights



Bearing Pressure



Friction

$F = \mu R = (0.350)(358.8\ lb/in) = 125.6\ lb/in$

Bearing Pressure Calculation

Contributing Forces

	Vert Force	...offset	Horz Force	...offset	OT Moment
Backfill Pressure	-0 lb/in	-	-160.28 lb/in	3.19 ft	73630 in·lb/ft
Axial Dead Load	-83.33 lb/in	4.66 ft	0 lb/in	-	-55960 in·lb/ft
Axial Live Load	-20.83 lb/in	4.66 ft	0 lb/in	-	-13990 in·lb/ft
Seismic Force	-50.12 lb/in	5.5 ft	-107.49 lb/in	5.74 ft	49206 in·lb/ft
Footing Weight	-80.16 lb/in	2.75 ft	0 lb/in	-	-31724.01 in·lb/ft
Stem Weight	-75 lb/in	4.66 ft	0 lb/in	-	-50364 in·lb/ft
Backfill Weight	-34.72 lb/in	5.25 ft	0 lb/in	-	-26233.33 in·lb/ft
Backfill Weight	-0.15 lb/in	5.33 ft	0 lb/in	-	-112.36 in·lb/ft
Soil over toe Weight	-14.43 lb/in	2.17 ft	0 lb/in	-	-4499.74 in·lb/ft
	-358.75 lb/in				-60047.59 in·lb/ft

$$\frac{-60047.59\ in{\cdot}lb/ft}{-358.75\ lb/in} = 1.16\ ft$$

Stability Checks [1.0D + 1.0L + 1.0H + 0.7E]

Overturning Check

Overturning Moments

	Force	Distance	Moment
Backfill pressure (horz)	160.3 lb/in	3.19 ft	73630 in·lb/ft
Seismic force	75.24 lb/in	5.74 ft	62216 in·lb/ft
		Total:	135845 in·lb/ft

Resisting Moments

	Force	Distance	Moment
Passive pressure @ toe	3.94 lb/in	0.5 ft	283.5 in·lb/ft
Axial dead load	-83.33 lb/in	4.66 ft	55960 in·lb/ft
Footing Weight	-80.16 lb/in	2.75 ft	31724 in·lb/ft
Stem Weight	-75 lb/in	4.66 ft	50364 in·lb/ft
Backfill Weight	-34.72 lb/in	5.25 ft	26233 in·lb/ft
Backfill Weight	-0.15 lb/in	5.33 ft	112.4 in·lb/ft
Soil over toe Weight	-14.43 lb/in	2.17 ft	4500 in·lb/ft
		Total:	169177 in·lb/ft

Without seismic loads:

$$F.S. = \frac{RM}{OTM} = \frac{169177 \text{ in·lb / ft}}{73630 \text{ in·lb / ft}} = 2.298 > 1.50 \text{ (OK)}$$

Including seismic loads:

$$F.S. = \frac{RM}{OTM} = \frac{169177 \text{ in·lb / ft}}{135845 \text{ in·lb / ft}} = 1.245 > 1.20 \text{ (OK)}$$

Sliding Check

Check not performed; restrained against sliding.

Bearing Capacity Check

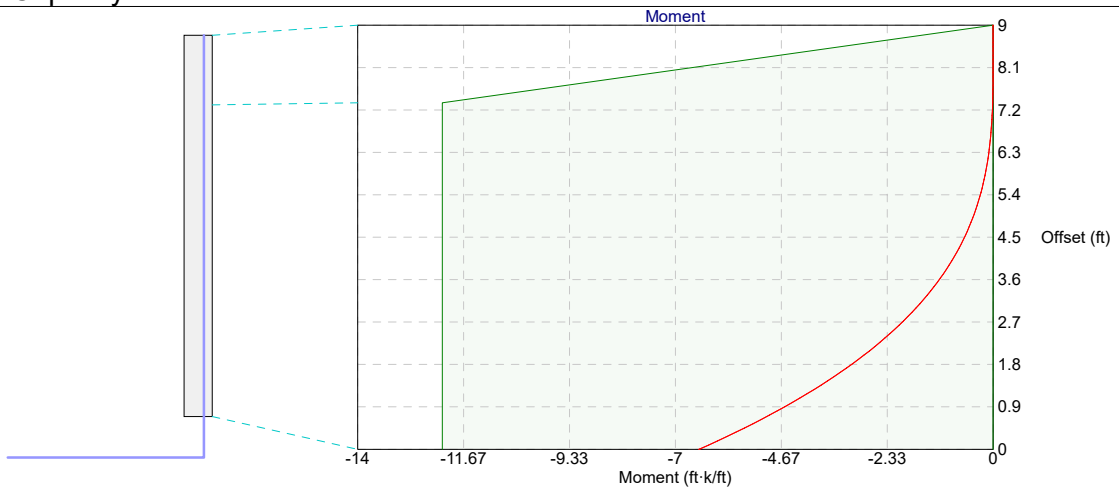
Bearing pressure < allowable (2366 psf < 2500 psf) - OK
Bearing resultant eccentricity < allowable (19.03 in < 32.98 in) - OK

Wall Top Displacement

(based on unfactored service loads)

Deflection due to stem flexural displacement	0.175 in
Deflection due to rotation from settlement	0.281 in
Total deflection at top of wall (positive towards toe)	0.456 in

Stem Flexural Capacity



Capacity (ACI 318-14 11.5.2.2, »22.3, »22.2) @ 0 ft from base

$$a = \frac{A_s f_y}{0.85 f'_c} = \frac{(0.04 \text{ in}^2 / \text{in})(60000 \text{ psi})}{0.85 (3000 \text{ psi})} = 1.04 \text{ in}$$

$$\phi M_n = \phi A_s f_y (d - a / 2) = (0.90)(0.04 \text{ in}^2 / \text{in})(60000 \text{ psi})[(5.63 \text{ in}) - (1.04 \text{ in}) / 2] = 12.14 \text{ ft·k / ft}$$

Capacity (ACI 318-14 11.5.2.2, »22.3, »22.2) @ 7.36 ft from base

$$a = \frac{A_s f_y}{0.85 f'_c} = \frac{(0.04 \text{ in}^2 / \text{in})(60000 \text{ psi})}{0.85 (3000 \text{ psi})} = 1.04 \text{ in}$$

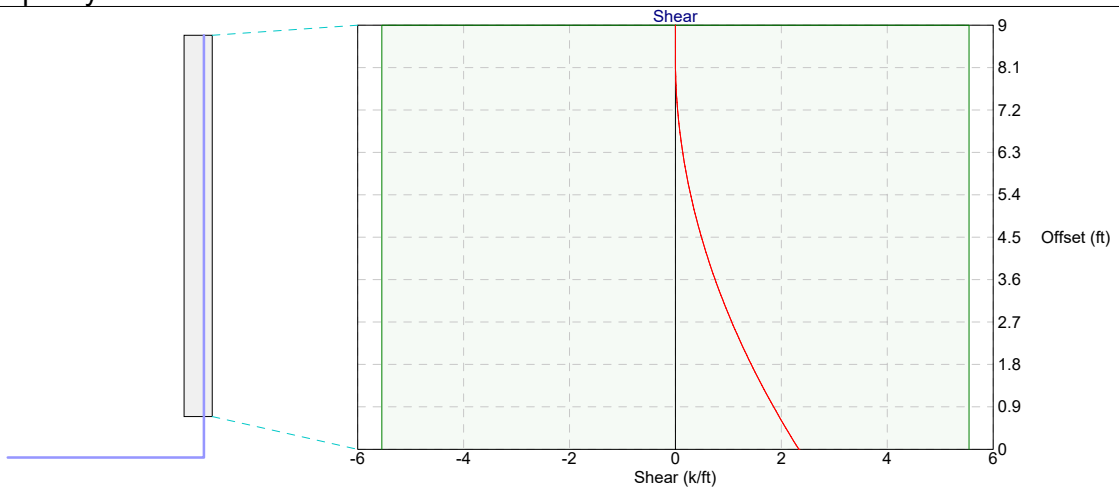
$$\phi M_n = \phi A_s f_y (d - a / 2) = (0.90)(0.04 \text{ in}^2 / \text{in})(60000 \text{ psi})[(5.63 \text{ in}) - (1.04 \text{ in}) / 2] = 12.14 \text{ ft·k / ft}$$

Capacity (ACI 318-14 11.5.2.2, »22.3, »22.2) @ 9 ft from base

$$a = \frac{A_s f_y}{0.85 f'_c} = \frac{(0 \text{ in}^2 / \text{in})(60000 \text{ psi})}{0.85 (3000 \text{ psi})} = 0 \text{ in}$$

$$\phi M_n = \phi A_s f_y (d - a / 2) = (0.90)(0 \text{ in}^2 / \text{in})(60000 \text{ psi})[(5.63 \text{ in}) - (0 \text{ in}) / 2] = 0 \text{ ft·k / ft}$$

Stem Shear Capacity



Shear Capacity (ACI 318-14 11.5.5.1, 22.5.1.1, 22.5.5.1) @ 0 ft from base

λ = 1.0 (normal weight concrete)

$V_c = 2 \lambda \sqrt{F'_c}\, d$ = 2 (1.0) $\sqrt{3000 \text{ psi}}$ (5.63 in) = 7.39 k / ft

$\phi V_n = \phi V_c$ = (0.750) (7.39 k / ft) = 5.55 k / ft

Shear Capacity (ACI 318-14 11.5.5.1, 22.5.1.1, 22.5.5.1) @ 9 ft from base

λ = 1.0 (normal weight concrete)

$V_c = 2 \lambda \sqrt{F'_c}\, d$ = 2 (1.0) $\sqrt{3000 \text{ psi}}$ (5.63 in) = 7.39 k / ft

$\phi V_n = \phi V_c$ = (0.750) (7.39 k / ft) = 5.55 k / ft

Stem Development/Lap Length Calculations

Main vertical stem bars (bottom end) - Development Length Calculation (ACI 318-14 11.7.1.2, 25.4.2.3, 25.4.3)

ψ_e = 1.0 (uncoated hooked bars)

ψ_c = 0.70 (based on side cover and extension cover)

ψ_r = 1.0 (no confining reinforcement)

λ = 1.0 (normal weight concrete)

$$l_{dh} = \left(\frac{f_y \, \psi_e \, \psi_c \, \psi_r}{50 \, \lambda \sqrt{F'_c}} \right) d_b = \left[\frac{(60000 \text{ psi}) \, (1.0) \, (0.70) \, (1.0)}{50 \, (1.0) \sqrt{3000 \text{ psi}}} \right] (0.75 \text{ in}) = 11.5 \text{ in}$$

$8 \, d_b = 8 \, (0.75 \text{ in}) = 6.0$ (minimum limit, does not control)

Main vertical stem bars (top end) - Development Length Calculation (ACI 318-14 11.7.1.2, 25.4.2.3, 25.4.3)

ψ_t = 1.0 (bars are not horizontal)

ψ_e = 1.0 (bar not epoxy coated)

ψ_s = 0.80 (bars are #6 or smaller)

λ = 1.0 (normal weight concrete)

$s / 2 = (10 \text{ in}) / 2 = 5 \text{ in}$

$\text{cover} + d_b / 2 = (2 \text{ in}) + (0.75 \text{ in}) / 2 = 2.38 \text{ in}$

c_b = 2.38 in (lesser of half spacing, ctr to surface)

K_{tr} = 0.0 (no transverse reinforcement)

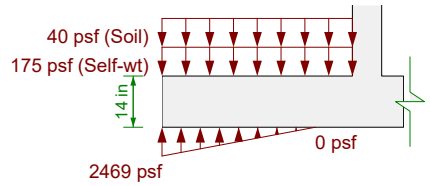
$$\frac{c_b + K_{tr}}{d_b} = \frac{(2.38 \text{ in}) + (0.0)}{(0.75 \text{ in})} = 3.1667$$

$$l_d = \left(\frac{3.}{40} \frac{f_y}{\lambda \sqrt{F'_c}} \frac{\psi_t \, \psi_e \, \psi_s}{2.5} \right) d_b = \left[\frac{3.}{40} \frac{(60000 \text{ psi})}{(1.0) \sqrt{3000 \text{ psi}}} \frac{(1.0) \, (1.0) \, (0.80)}{2.5} \right] (0.75 \text{ in}) = 19.72 \text{ in}$$

Toe Checks [1.2D + 1.6L + 1.6H]

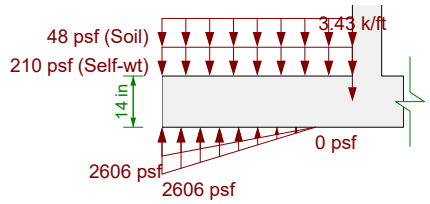
Toe Unfactored Loads

Unfactored Loads



Toe Factored Loads

1.2D + 1.6L + 1.6H



Controlling Moment

Design moment M_u for toe need not exceed moment at stem base:

M_{toe} = 11.98 ft·k / ft ≥ M_{stem} = 6.48 ft·k / ft

M_u = 6.48 ft·k / ft (stem base moment controls)

Flexure Check (ACI 318-14 13.3.2.1, 7.5.2.1, »22.3, »22.2, 7.5.1.1a)

$$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0.04 \text{ in}^2 / \text{in})(60000 \text{ psi})}{0.85 (3000 \text{ psi})} = 1.04 \text{ in}$$

$\phi M_n = \phi A_s f_y (d - a / 2)$ = (0.90) (0.04 in² / in) (60000 psi) [(11.63 in) - (1.04 in) / 2] = 26.39 ft·k / ft

ϕM_n = 26.39 ft·k / ft ≥ M_u = 6.48 ft·k / ft ✓

Shear Check (ACI 318-14 13.3.2.1, 7.5.3.1, »22.5.1, »22.5.5, 7.5.1.1b)

λ = 1.0 (normal weight concrete)

$V_c = 2 \lambda \sqrt{F'_c} d = 2 (1.0) \sqrt{3000 \text{ psi}} (11.63 \text{ in})$ = 15.28 k / ft

$\phi V_n = \phi V_c$ = (0.750) (15.28 k / ft) = 11.46 k / ft

ϕV_n = 11.46 k / ft ≥ V_u = 3.67 k / ft ✓

Minimum Strain Check (ACI 318-14 13.3.2.1, 7.3.3.1)

β_1 = 0.850 ($F'_c \le 4000$ psi)

$$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0.04 \text{ in}^2 / \text{in})(60000 \text{ psi})}{0.85 (3000 \text{ psi})} = 1.04 \text{ in}$$

$$\varepsilon_t = 0.003 \left(\frac{d}{a / \beta_1} - 1 \right) = 0.003 \left[\frac{(11.63 \text{ in})}{(1.04 \text{ in}) / (0.850)} - 1 \right] = 0.0256$$

ε_t = 0.0256 ≥ 0.004 ✓

Minimum Steel Check (ACI 318-14 13.3.2.1, 9.6.1)

ϕM_n = 26.39 ft·k / ft ≥ (4 / 3) M_u = [4 / 3] (6.48 ft·k / ft) = 8.64 ft·k / ft

Check is waived per ACI 9.6.1.3 ✓

Shrinkage and Temperature Steel (ACI 318-14 13.2.8.1, 7.6.4.1, 24.4.3.2, 24.4.3.3)

$$\rho_{ST_prov} = \frac{A_{ST}}{t\, s_{ST}} = \frac{(0.4 \text{ in}^2 / \text{in})}{(14 \text{ in})(12 \text{ in})} = 0.0024$$

$$\rho_{ST_prov} = \frac{A_{ST}}{t\, s_{ST}} = \frac{(0.4 \text{ in}^2 / \text{in})}{(14 \text{ in})(12 \text{ in})} = 0.0024$$

$$\frac{0.0018 (60000)}{f_y} = \frac{0.0018 (60000)}{(60000 \text{ psi})} = 0.0018$$

ρ_{ST_min} = 0.0018

ρ_{ST_prov} = 0.0024 ≥ ρ_{ST_min} = 0.0018 ✓

18 inch limit governs

s_{ST_max} = 18 in

s_{ST} = 12 in ≤ s_{ST_max} = 18 in ✓

Development Check (ACI 318-14 13.2.8.1, 25.4.2.3, 25.4.10)

$$\frac{M_u}{\phi M_n} = \frac{(6.48 \text{ ft·k / ft})}{(26.39 \text{ ft·k / ft})} = 0.2456$$ (ratio to represent excess reinforcement)

ψ_e = 1.0 (uncoated hooked bars)

ψ_c = 0.70 (based on side cover and extension cover)

ψ_r = 1.0 (no confining reinforcement)

λ = 1.0 (normal weight concrete)

$$l_{dh} = \left(\frac{f_y \psi_e \psi_c \psi_r}{50 \lambda \sqrt{F'_c}} \right) d_b = \left[\frac{(60000 \text{ psi})(1.0)(0.70)(1.0)}{50 (1.0) \sqrt{3000 \text{ psi}}} \right] (0.75 \text{ in}) = 11.5 \text{ in}$$

Factoring l_{dh} by the excess reinforcement ratio (0.2456) per 25.4.10: l_{dh} = 2.82 in

8 d_b = 8 (0.75 in) = 6.0 (minimum limit, does not control)

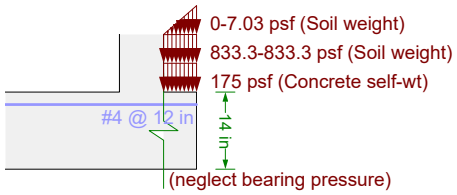
6 inch minimum controls

l_{dh_prov} = 6 in ≥ l_{dh} = 6 in ✓

Heel Checks [1.2D + 1.6L + 1.6H]

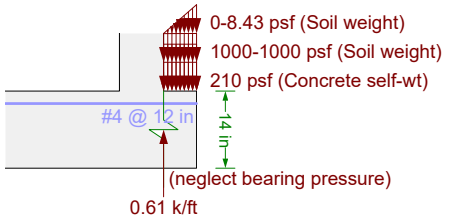
Heel Unfactored Loads

Unfactored Loads



Heel Factored Loads

1.2D + 1.6L + 1.6H



Controlling Moment

Design moment M_u for heel need not exceed moment at stem base:

M_{heel} = 0.15 ft·k / ft < M_{stem} = 6.48 ft·k / ft

M_u = 0.15 ft·k / ft (stem moment does not control)

Flexure Check (ACI 318-14 13.3.2.1, 7.5.2.1, »22.3, »22.2, 7.5.1.1a)

$$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0.02 \text{ in}^2 / \text{in})(60000 \text{ psi})}{0.85 (3000 \text{ psi})} = 0.39 \text{ in}$$

$$\phi M_n = \phi A_s f_y (d - a / 2) = (0.90)(0.02 \text{ in}^2 / \text{in})(60000 \text{ psi})[(11.75 \text{ in}) - (0.39 \text{ in}) / 2] = 10.4 \text{ ft·k / ft}$$

ϕM_n = 10.4 ft·k / ft ≥ M_u = 0.15 ft·k / ft ✓

Shear Check (ACI 318-14 13.3.2.1, 7.5.3.1, »22.5.1, »22.5.5, 7.5.1.1b)

λ = 1.0 (normal weight concrete)

$$V_c = 2 \lambda \sqrt{F'_c} d = 2 (1.0) \sqrt{3000 \text{ psi}} (11.75 \text{ in}) = 15.45 \text{ k / ft}$$

$$\phi V_n = \phi V_c = (0.750)(15.45 \text{ k / ft}) = 11.58 \text{ k / ft}$$

ϕV_n = 11.58 k / ft ≥ V_u = 0.61 k / ft ✓

Minimum Strain Check (ACI 318-14 13.3.2.1, 7.3.3.1)

β_1 = 0.850 ($F'_c \le 4000$ psi)

$$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0.02 \text{ in}^2 / \text{in})(60000 \text{ psi})}{0.85 (3000 \text{ psi})} = 0.39 \text{ in}$$

$$\varepsilon_t = 0.003 \left(\frac{d}{a / \beta_1} - 1 \right) = 0.003 \left[\frac{(11.75 \text{ in})}{(0.39 \text{ in}) / (0.850)} - 1 \right] = 0.0734$$

ε_t = 0.0734 ≥ 0.004 ✓

Minimum Steel Check (ACI 318-14 13.3.2.1, 9.6.1)

ϕM_n = 10.4 ft·k / ft ≥ (4 / 3) M_u = [4 / 3] (0.15 ft·k / ft) = 0.2 ft·k / ft

Check is waived per ACI 9.6.1.3 ✓

Shrinkage and Temperature Steel (ACI 318-14 13.2.8.1, 7.6.4.1, 24.4.3.2, 24.4.3.3)

$$\rho_{ST_prov} = \frac{A_{ST}}{t\, s_{ST}} = \frac{(0.4 \text{ in}^2 / \text{in})}{(14 \text{ in})(12 \text{ in})} = 0.0024$$

$$\rho_{ST_prov} = \frac{A_{ST}}{t\, s_{ST}} = \frac{(0.4 \text{ in}^2 / \text{in})}{(14 \text{ in})(12 \text{ in})} = 0.0024$$

$$\frac{0.0018 (60000)}{f_y} = \frac{0.0018 (60000)}{(60000 \text{ psi})} = 0.0018$$

ρ_{ST_min} = 0.0018

ρ_{ST_prov} = 0.0024 ≥ ρ_{ST_min} = 0.0018 ✓

18 inch limit governs

s_{ST_max} = 18 in

s_{ST} = 12 in ≤ s_{ST_max} = 18 in ✓

Development Check (ACI 318-14 13.2.8.1, 25.4.2.3, 25.4.10)

$$\frac{M_u}{\phi M_n} = \frac{(0.15 \text{ ft·k / ft})}{(10.4 \text{ ft·k / ft})} = 0.0146$$ (ratio to represent excess reinforcement)

ψ_t = 1.0 (12 inches or less cast below - 11.50 inches)

ψ_e = 1.0 (bar not epoxy coated)

ψ_s = 0.80 (bars are #6 or smaller)

λ = 1.0 (normal weight concrete)

s / 2 = (12 in) / 2 = 6 in

cover + d_b / 2 = (2 in) + (0.5 in) / 2 = 2.25 in

c_b = 2.25 in (lesser of half spacing, ctr to surface)

K_{tr} = 0.0 (no transverse reinforcement)

$$\frac{c_b + K_{tr}}{d_b} = \frac{(2.25 \text{ in}) + (0.0)}{(0.5 \text{ in})} = 4.50$$

$$l_d = \left(\frac{3.}{40} \frac{f_y}{\lambda \sqrt{F'_c}} \frac{\psi_t \psi_e \psi_s}{2.5} \right) d_b = \left[\frac{3.}{40} \frac{(60000 \text{ psi})}{(1.0) \sqrt{3000 \text{ psi}}} \frac{(1.0)(1.0)(0.80)}{2.5} \right] (0.5 \text{ in}) = 13.15 \text{ in}$$

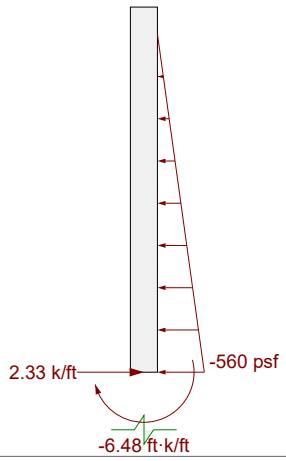
Factoring l_d by the excess reinforcement ratio (0.0146) per 25.4.10: l_d = 0.19 in

12 inch minimum controls

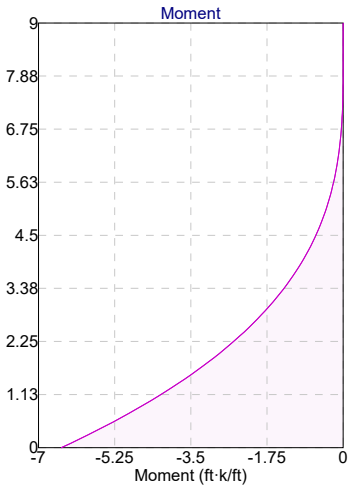
l_{d_prov} = 57.96 in ≥ l_d = 12 in ✓

Stem Forces [1.2D + 1.6L + 1.6H]

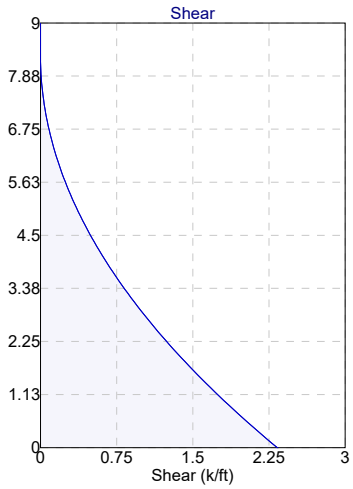
Stem Internal Forces



Stem Internal Forces



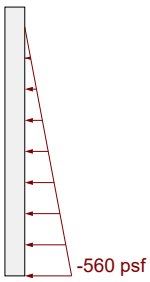
Stem Internal Forces



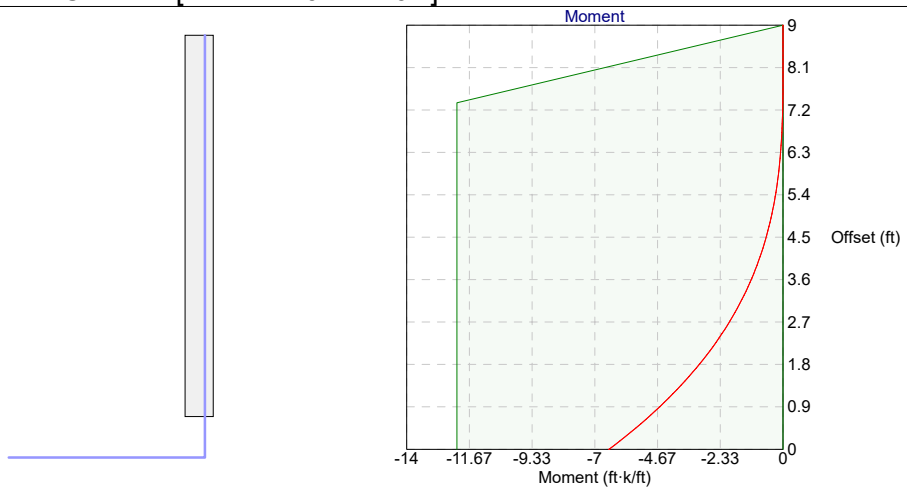
Stem Joint Force Transfer

Location	Force
@ stem base	2.33 k/ft

Stem Internal Forces



Stem Moment Checks [1.2D + 1.6L + 1.6H]



Check (ACI 318-14 11.5.5.1b) @ 0 ft from base

ϕM_n = 12.14 ft·k / ft ≥ M_u = 6.48 ft·k / ft ✓

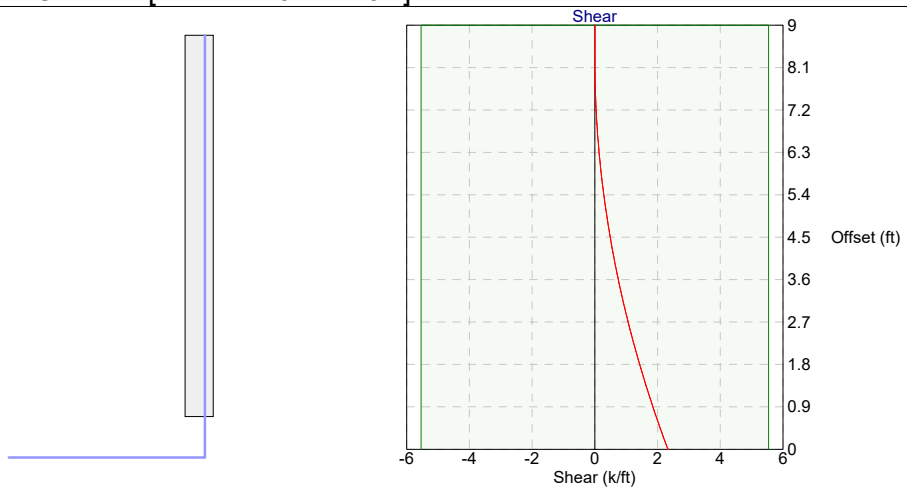
Check (ACI 318-14 11.5.5.1b) @ 7.36 ft from base

ϕM_n = 12.14 ft·k / ft ≥ M_u = 0.01 ft·k / ft ✓

Check (ACI 318-14 11.5.5.1b) @ 7.36 ft from base

ϕM_n = 12.08 ft·k / ft ≥ M_u = 0.01 ft·k / ft ✓

Stem Shear Checks [1.2D + 1.6L + 1.6H]



Shear Check (ACI 318-14 11.5.5.1c) @ 0 ft from base

ϕV_n = 5.55 k / ft ≥ V_u = 2.33 k / ft ✓

Stem Miscellaneous Checks [1.2D + 1.6L + 1.6H]

Minimum Steel Check (ACI 318-14 9.6.1) @ 0 ft from base [Stem in negative flexure]

ϕM_n = 12.14 ft·k / ft ≥ (4 / 3) M_u = [4 / 3] (6.48 ft·k / ft) = 8.64 ft·k / ft

Check is waived per ACI 9.6.1.3 ✓

Minimum Steel Check (ACI 318-14 9.6.1) @ 9 ft from base [Stem in negative flexure]

ϕM_n = 0 ft·k / ft ≥ (4 / 3) M_u = [4 / 3] (0 ft·k / ft) = 0 ft·k / ft

Check is waived per ACI 9.6.1.3 ✓

Maximum Steel Check (ACI 318-14 9.3.3.1) @ 0 ft from base [Stem in negative flexure]

β_1 = 0.850 ($F'_c \le 4000$ psi)

$$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0.04 \text{ in}^2 / \text{in})(60000 \text{ psi})}{0.85 (3000 \text{ psi})} = 1.04 \text{ in}$$

$$\varepsilon_t = 0.003 \left(\frac{d}{a / \beta_1} - 1 \right) = 0.003 \left[\frac{(5.63 \text{ in})}{(1.04 \text{ in}) / (0.850)} - 1 \right] = 0.0109$$

ε_t = 0.0109 ≥ 0.004 ✓

Maximum Steel Check (ACI 318-14 9.3.3.1) @ 9 ft from base [Stem in negative flexure]

β_1 = 0.850 ($F'_c \le 4000$ psi)

$$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0.04 \text{ in}^2 / \text{in})(60000 \text{ psi})}{0.85 (3000 \text{ psi})} = 1.04 \text{ in}$$

$$\varepsilon_t = 0.003 \left(\frac{d}{a / \beta_1} - 1 \right) = 0.003 \left[\frac{(5.63 \text{ in})}{(1.04 \text{ in}) / (0.850)} - 1 \right] = 0.0109$$

ε_t = 0.0109 ≥ 0.004 ✓

Wall Horizontal Steel (ACI 318-14 11.6.1, 11.7.3)

$$\rho_t = \frac{A_{s_horz} / s_{horz}}{t} = \frac{(0.2 \text{ in}^2) / (12 \text{ in})}{(8 \text{ in})} = 0.0021$$

ρ_{t_min} = 0.0020 (bars No. 5 or less, not less than 60 ksi)

ρ_t = 0.0021 ≥ ρ_{t_min} = 0.0020 ✓

3 h = 3 (8 in) = 24 in

18 inch limit governs

s_{horz} = 12 in ≤ s_{horz_max} = 18 in ✓

Development Check (ACI 318-14 11.7.1.2, 25.4.2.3, 25.4.10)

$\frac{M_u}{\phi M_n}$ = $\frac{(6.48 \text{ ft·k / ft})}{(12.14 \text{ ft·k / ft})}$ = 0.5341 (ratio to represent excess reinforcement)

ψ_e = 1.0 (uncoated hooked bars)

ψ_c = 0.70 (based on side cover and extension cover)

ψ_r = 1.0 (no confining reinforcement)

λ = 1.0 (normal weight concrete)

$$l_{dh} = \left(\frac{f_y \psi_e \psi_c \psi_r}{50 \lambda \sqrt{F'_c}} \right) d_b = \left[\frac{(60000 \text{ psi})(1.0)(0.70)(1.0)}{50 (1.0) \sqrt{3000 \text{ psi}}} \right] (0.75 \text{ in}) = 11.5 \text{ in}$$

Factoring l_{dh} by the excess reinforcement ratio (0.5341) per 25.4.10: l_{dh} = 6.14 in

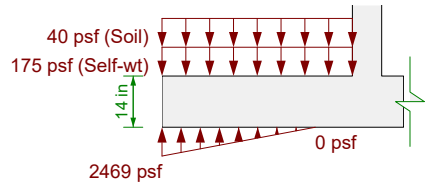
8 d_b = 8 (0.75 in) = 6.0 (minimum limit, does not control)

l_{dh_prov} = 12 in ≥ l_{dh} = 6.14 in ✓

Toe Checks [1.2D + 0.5L + 1.6H + 1.0E]

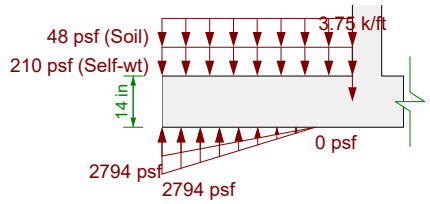
Toe Unfactored Loads

Unfactored Loads



Toe Factored Loads

1.2D + 0.5L + 1.6H + 1.0E



Controlling Moment

Design moment M_u for toe need not exceed moment at stem base:

M_{toe} = 13.01 ft·k / ft ≥ M_{stem} = 11.37 ft·k / ft

M_u = 11.37 ft·k / ft (stem base moment controls)

Flexure Check (ACI 318-14 13.3.2.1, 7.5.2.1, »22.3, »22.2, 7.5.1.1a)

$$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0.04 \text{ in}^2 / \text{in})(60000 \text{ psi})}{0.85 (3000 \text{ psi})} = 1.04 \text{ in}$$

$\phi M_n = \phi A_s f_y (d - a / 2)$ = (0.90) (0.04 in² / in) (60000 psi) [(11.63 in) - (1.04 in) / 2] = 26.39 ft·k / ft

ϕM_n = 26.39 ft·k / ft ≥ M_u = 11.37 ft·k / ft ✓

Shear Check (ACI 318-14 13.3.2.1, 7.5.3.1, »22.5.1, »22.5.5, 7.5.1.1b)

λ = 1.0 (normal weight concrete)

$V_c = 2 \lambda \sqrt{F'_c} d = 2 (1.0) \sqrt{3000 \text{ psi}} (11.63 \text{ in}) = 15.28$ k / ft

$\phi V_n = \phi V_c$ = (0.750) (15.28 k / ft) = 11.46 k / ft

ϕV_n = 11.46 k / ft ≥ V_u = 4 k / ft ✓

Minimum Strain Check (ACI 318-14 13.3.2.1, 7.3.3.1)

β_1 = 0.850 ($F'_c \le 4000$ psi)

$$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0.04 \text{ in}^2 / \text{in})(60000 \text{ psi})}{0.85 (3000 \text{ psi})} = 1.04 \text{ in}$$

$$\varepsilon_t = 0.003 \left(\frac{d}{a / \beta_1} - 1 \right) = 0.003 \left[\frac{(11.63 \text{ in})}{(1.04 \text{ in}) / (0.850)} - 1 \right] = 0.0256$$

ε_t = 0.0256 ≥ 0.004 ✓

Minimum Steel Check (ACI 318-14 13.3.2.1, 9.6.1)

ϕM_n = 26.39 ft·k / ft ≥ (4 / 3) M_u = [4 / 3] (11.37 ft·k / ft) = 15.16 ft·k / ft

Check is waived per ACI 9.6.1.3 ✓

Shrinkage and Temperature Steel (ACI 318-14 13.2.8.1, 7.6.4.1, 24.4.3.2, 24.4.3.3)

$$\rho_{ST_prov} = \frac{A_{ST}}{t\, s_{ST}} = \frac{(0.4 \text{ in}^2 / \text{in})}{(14 \text{ in})(12 \text{ in})} = 0.0024$$

$$\rho_{ST_prov} = \frac{A_{ST}}{t\, s_{ST}} = \frac{(0.4 \text{ in}^2 / \text{in})}{(14 \text{ in})(12 \text{ in})} = 0.0024$$

$$\frac{0.0018 (60000)}{f_y} = \frac{0.0018 (60000)}{(60000 \text{ psi})} = 0.0018$$

ρ_{ST_min} = 0.0018

ρ_{ST_prov} = 0.0024 ≥ ρ_{ST_min} = 0.0018 ✓

18 inch limit governs

s_{ST_max} = 18 in

s_{ST} = 12 in ≤ s_{ST_max} = 18 in ✓

Development Check (ACI 318-14 13.2.8.1, 25.4.2.3, 25.4.10)

$$\frac{M_u}{\phi M_n} = \frac{(11.37 \text{ ft·k / ft})}{(26.39 \text{ ft·k / ft})} = 0.4309$$ (ratio to represent excess reinforcement)

ψ_e = 1.0 (uncoated hooked bars)

ψ_c = 0.70 (based on side cover and extension cover)

ψ_r = 1.0 (no confining reinforcement)

λ = 1.0 (normal weight concrete)

$$l_{dh} = \left(\frac{f_y \psi_e \psi_c \psi_r}{50 \lambda \sqrt{F'_c}} \right) d_b = \left[\frac{(60000 \text{ psi})(1.0)(0.70)(1.0)}{50 (1.0) \sqrt{3000 \text{ psi}}} \right] (0.75 \text{ in}) = 11.5 \text{ in}$$

Factoring l_{dh} by the excess reinforcement ratio (0.4309) per 25.4.10: l_{dh} = 4.96 in

8 d_b = 8 (0.75 in) = 6.0 (minimum limit, does not control)

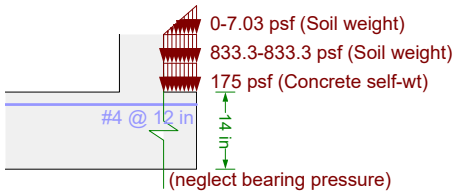
6 inch minimum controls

l_{dh_prov} = 6 in ≥ l_{dh} = 6 in ✓

Heel Checks [1.2D + 0.5L + 1.6H + 1.0E]

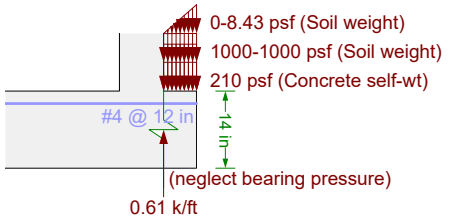
Heel Unfactored Loads

Unfactored Loads



Heel Factored Loads

1.2D + 0.5L + 1.6H + 1.0E



Controlling Moment

Design moment M_u for heel need not exceed moment at stem base:

M_{heel} = 0.15 ft·k / ft < M_{stem} = 11.37 ft·k / ft

M_u = 0.15 ft·k / ft (stem moment does not control)

Flexure Check (ACI 318-14 13.3.2.1, 7.5.2.1, »22.3, »22.2, 7.5.1.1a)

$$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0.02 \text{ in}^2 / \text{in})(60000 \text{ psi})}{0.85 (3000 \text{ psi})} = 0.39 \text{ in}$$

$$\phi M_n = \phi A_s f_y (d - a / 2) = (0.90)(0.02 \text{ in}^2 / \text{in})(60000 \text{ psi})[(11.75 \text{ in}) - (0.39 \text{ in}) / 2] = 10.4 \text{ ft·k / ft}$$

ϕM_n = 10.4 ft·k / ft ≥ M_u = 0.15 ft·k / ft ✓

Shear Check (ACI 318-14 13.3.2.1, 7.5.3.1, »22.5.1, »22.5.5, 7.5.1.1b)

λ = 1.0 (normal weight concrete)

$$V_c = 2 \lambda \sqrt{F'_c}\, d = 2 (1.0) \sqrt{3000 \text{ psi}}\, (11.75 \text{ in}) = 15.45 \text{ k / ft}$$

$$\phi V_n = \phi V_c = (0.750)(15.45 \text{ k / ft}) = 11.58 \text{ k / ft}$$

ϕV_n = 11.58 k / ft ≥ V_u = 0.61 k / ft ✓

Minimum Strain Check (ACI 318-14 13.3.2.1, 7.3.3.1)

β_1 = 0.850 ($F'_c \le 4000$ psi)

$$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0.02 \text{ in}^2 / \text{in})(60000 \text{ psi})}{0.85 (3000 \text{ psi})} = 0.39 \text{ in}$$

$$\varepsilon_t = 0.003 \left(\frac{d}{a / \beta_1} - 1 \right) = 0.003 \left[\frac{(11.75 \text{ in})}{(0.39 \text{ in}) / (0.850)} - 1 \right] = 0.0734$$

ε_t = 0.0734 ≥ 0.004 ✓

Minimum Steel Check (ACI 318-14 13.3.2.1, 9.6.1)

ϕM_n = 10.4 ft·k / ft ≥ (4 / 3) M_u = [4 / 3] (0.15 ft·k / ft) = 0.2 ft·k / ft

Check is waived per ACI 9.6.1.3 ✓

Shrinkage and Temperature Steel (ACI 318-14 13.2.8.1, 7.6.4.1, 24.4.3.2, 24.4.3.3)

$$\rho_{ST_prov} = \frac{A_{ST}}{t\, s_{ST}} = \frac{(0.4 \text{ in}^2 / \text{in})}{(14 \text{ in})(12 \text{ in})} = 0.0024$$

$$\rho_{ST_prov} = \frac{A_{ST}}{t\, s_{ST}} = \frac{(0.4 \text{ in}^2 / \text{in})}{(14 \text{ in})(12 \text{ in})} = 0.0024$$

$$\frac{0.0018 (60000)}{f_y} = \frac{0.0018 (60000)}{(60000 \text{ psi})} = 0.0018$$

ρ_{ST_min} = 0.0018

ρ_{ST_prov} = 0.0024 ≥ ρ_{ST_min} = 0.0018 ✓

18 inch limit governs

s_{ST_max} = 18 in

s_{ST} = 12 in ≤ s_{ST_max} = 18 in ✓

Development Check (ACI 318-14 13.2.8.1, 25.4.2.3, 25.4.10)

$$\frac{M_u}{\phi M_n} = \frac{(0.15 \text{ ft·k / ft})}{(10.4 \text{ ft·k / ft})} = 0.0146$$ (ratio to represent excess reinforcement)

ψ_t = 1.0 (12 inches or less cast below - 11.50 inches)

ψ_e = 1.0 (bar not epoxy coated)

ψ_s = 0.80 (bars are #6 or smaller)

λ = 1.0 (normal weight concrete)

s / 2 = (12 in) / 2 = 6 in

cover + d_b / 2 = (2 in) + (0.5 in) / 2 = 2.25 in

c_b = 2.25 in (lesser of half spacing, ctr to surface)

K_{tr} = 0.0 (no transverse reinforcement)

$$\frac{c_b + K_{tr}}{d_b} = \frac{(2.25 \text{ in}) + (0.0)}{(0.5 \text{ in})} = 4.50$$

$$l_d = \left(\frac{3.}{40} \frac{f_y}{\lambda \sqrt{F'_c}} \frac{\psi_t \psi_e \psi_s}{2.5} \right) d_b = \left[\frac{3.}{40} \frac{(60000 \text{ psi})}{(1.0) \sqrt{3000 \text{ psi}}} \frac{(1.0)(1.0)(0.80)}{2.5} \right] (0.5 \text{ in}) = 13.15 \text{ in}$$

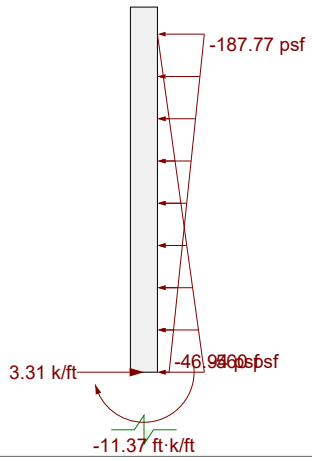
Factoring l_d by the excess reinforcement ratio (0.0146) per 25.4.10: l_d = 0.19 in

12 inch minimum controls

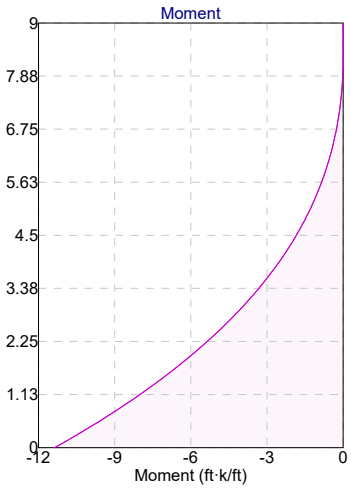
l_{d_prov} = 57.96 in ≥ l_d = 12 in ✓

Stem Forces [1.2D + 0.5L + 1.6H + 1.0E]

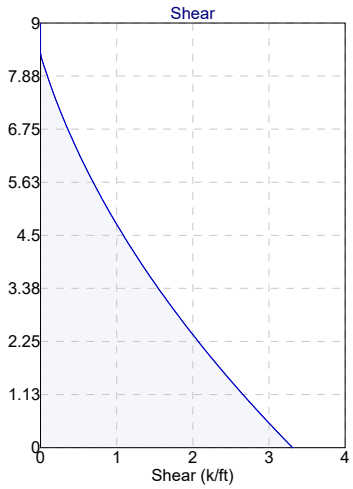
Stem Internal Forces



Stem Internal Forces



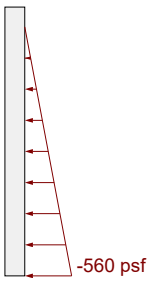
Stem Internal Forces

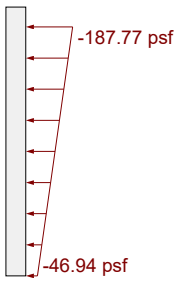


Stem Joint Force Transfer

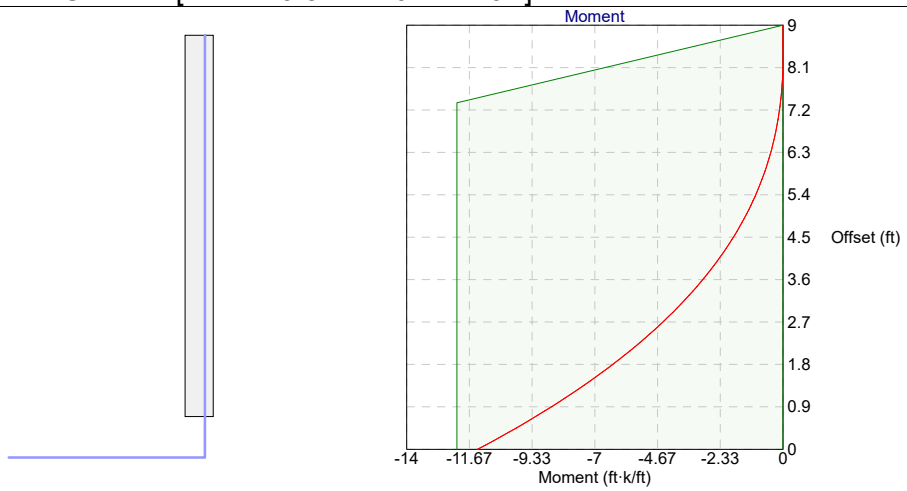
Location	Force
@ stem base	3.31 k/ft

Stem Internal Forces





Stem Moment Checks [1.2D + 0.5L + 1.6H + 1.0E]



Check (ACI 318-14 11.5.5.1b) @ 0 ft from base

ϕM_n = 12.14 ft·k / ft ≥ M_u = 11.37 ft·k / ft ✓

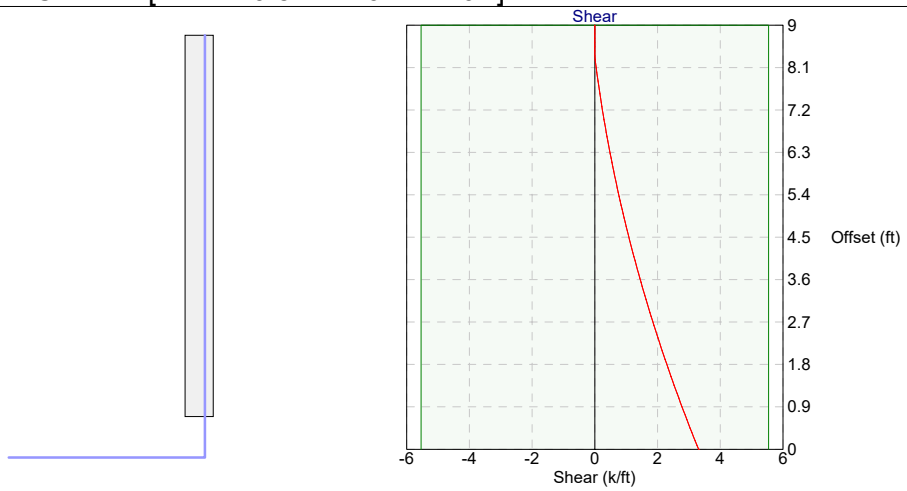
Check (ACI 318-14 11.5.5.1b) @ 7.36 ft from base

ϕM_n = 12.14 ft·k / ft ≥ M_u = 0.1 ft·k / ft ✓

Check (ACI 318-14 11.5.5.1b) @ 7.36 ft from base

ϕM_n = 12.08 ft·k / ft ≥ M_u = 0.1 ft·k / ft ✓

Stem Shear Checks [1.2D + 0.5L + 1.6H + 1.0E]



Shear Check (ACI 318-14 11.5.5.1c) @ 0 ft from base

ϕV_n = 5.55 k / ft ≥ V_u = 3.31 k / ft ✓

Stem Miscellaneous Checks [1.2D + 0.5L + 1.6H + 1.0E]

Minimum Steel Check (ACI 318-14 9.6.1) @ 0 ft from base [Stem in negative flexure]

$\phi M_n = 12.14$ ft·k / ft $<$ $(4 / 3) M_u = [4 / 3] (11.37$ ft·k / ft$) = 15.16$ ft·k / ft

$A_{s_min} = \frac{3 \sqrt{F'_c}}{f_y} d = \frac{3 \sqrt{3000 \text{ psi}}}{(60000 \text{ psi})} (5.63 \text{ in}) = 0.02$ in² / in

$200\ d / f_y = 200\ (5.63 \text{ in}) / (60000 \text{ psi}) = 0.02$ in² / in

$A_{s_min} = 0.02$ in² / in

$A_s = 0.04$ in² / in $\geq$ $A_{s_min} = 0.02$ in² / in ✓

Minimum Steel Check (ACI 318-14 9.6.1) @ 9 ft from base [Stem in negative flexure]

$\phi M_n = 0$ ft·k / ft $\geq$ $(4 / 3) M_u = [4 / 3] (0$ ft·k / ft$) = 0$ ft·k / ft

Check is waived per ACI 9.6.1.3 ✓

Maximum Steel Check (ACI 318-14 9.3.3.1) @ 0 ft from base [Stem in negative flexure]

$\beta_1 = 0.850$ $(F'_c \leq 4000 \text{ psi})$

$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0.04 \text{ in² / in}) (60000 \text{ psi})}{0.85 (3000 \text{ psi})} = 1.04$ in

$\varepsilon_t = 0.003 \left(\frac{d}{a / \beta_1} - 1 \right) = 0.003 \left[\frac{(5.63 \text{ in})}{(1.04 \text{ in}) / (0.850)} - 1 \right] = 0.0109$

$\varepsilon_t = 0.0109 \geq 0.004$ ✓

Maximum Steel Check (ACI 318-14 9.3.3.1) @ 9 ft from base [Stem in negative flexure]

$\beta_1 = 0.850$ $(F'_c \leq 4000 \text{ psi})$

$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0.04 \text{ in² / in}) (60000 \text{ psi})}{0.85 (3000 \text{ psi})} = 1.04$ in

$\varepsilon_t = 0.003 \left(\frac{d}{a / \beta_1} - 1 \right) = 0.003 \left[\frac{(5.63 \text{ in})}{(1.04 \text{ in}) / (0.850)} - 1 \right] = 0.0109$

$\varepsilon_t = 0.0109 \geq 0.004$ ✓

Wall Horizontal Steel (ACI 318-14 11.6.1, 11.7.3)

$\rho_t = \frac{A_{s_horz} / s_{horz}}{t} = \frac{(0.2 \text{ in²}) / (12 \text{ in})}{(8 \text{ in})} = 0.0021$

$\rho_{t_min} = 0.0020$ (bars No. 5 or less, not less than 60 ksi)

$\rho_t = 0.0021 \geq \rho_{t_min} = 0.0020$ ✓

$3\ h = 3\ (8 \text{ in}) = 24$ in

18 inch limit governs

$s_{horz} = 12 \text{ in} \leq s_{horz_max} = 18 \text{ in}$ ✓

Development Check (ACI 318-14 11.7.1.2, 25.4.2.3, 25.4.10)

$\frac{M_u}{\phi M_n} = \frac{(11.37 \text{ ft·k / ft})}{(12.14 \text{ ft·k / ft})} = 0.9371$ (ratio to represent excess reinforcement)

$\psi_e = 1.0$ (uncoated hooked bars)

$\psi_c = 0.70$ (based on side cover and extension cover)

$\psi_r = 1.0$ (no confining reinforcement)

$\lambda = 1.0$ (normal weight concrete)

$l_{dh} = \left(\frac{f_y \psi_e \psi_c \psi_r}{50 \lambda \sqrt{F'_c}} \right) d_b = \left[\frac{(60000 \text{ psi}) (1.0) (0.70) (1.0)}{50 (1.0) \sqrt{3000 \text{ psi}}} \right] (0.75 \text{ in}) = 11.5$ in

Factoring l_{dh} by the excess reinforcement ratio (0.9371) per 25.4.10: $l_{dh} = 10.78$ in

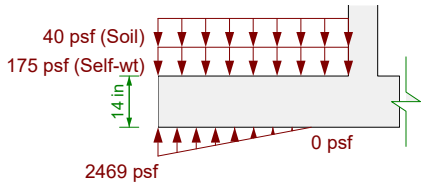
$8\ d_b = 8\ (0.75 \text{ in}) = 6.0$ (minimum limit, does not control)

$l_{dh_prov} = 12 \text{ in} \geq l_{dh} = 10.78 \text{ in}$ ✓

Toe Checks [1.4D]

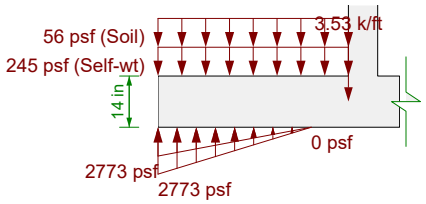
Toe Unfactored Loads

Unfactored Loads



Toe Factored Loads

1.4D



Controlling Moment

Design moment M_u for toe need not exceed moment at stem base:

M_{toe} = 12.49 ft·k / ft ≥ M_{stem} = - 0 ft·k / ft

M_u = - 0 ft·k / ft (stem base moment controls)

Flexure Check (ACI 318-14 13.3.2.1, 7.5.2.1, »22.3, »22.2, 7.5.1.1a)

$$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0.04 \text{ in}^2 / \text{in})(60000 \text{ psi})}{0.85 (3000 \text{ psi})} = 1.04 \text{ in}$$

$\phi M_n = \phi A_s f_y (d - a / 2)$ = (0.90) (0.04 in² / in) (60000 psi) [(11.63 in) - (1.04 in) / 2] = 26.39 ft·k / ft

ϕM_n = 26.39 ft·k / ft ≥ M_u = - 0 ft·k / ft ✓

Shear Check (ACI 318-14 13.3.2.1, 7.5.3.1, »22.5.1, »22.5.5, 7.5.1.1b)

λ = 1.0 (normal weight concrete)

$V_c = 2 \lambda \sqrt{F'_c} d = 2 (1.0) \sqrt{3000 \text{ psi}} (11.63 \text{ in})$ = 15.28 k / ft

$\phi V_n = \phi V_c$ = (0.750) (15.28 k / ft) = 11.46 k / ft

ϕV_n = 11.46 k / ft ≥ V_u = 3.82 k / ft ✓

Minimum Strain Check (ACI 318-14 13.3.2.1, 7.3.3.1)

β_1 = 0.850 ($F'_c \le 4000$ psi)

$$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0.04 \text{ in}^2 / \text{in})(60000 \text{ psi})}{0.85 (3000 \text{ psi})} = 1.04 \text{ in}$$

$$\varepsilon_t = 0.003 \left(\frac{d}{a / \beta_1} - 1 \right) = 0.003 \left[\frac{(11.63 \text{ in})}{(1.04 \text{ in}) / (0.850)} - 1 \right] = 0.0256$$

ε_t = 0.0256 ≥ 0.004 ✓

Minimum Steel Check (ACI 318-14 13.3.2.1, 9.6.1)

ϕM_n = 26.39 ft·k / ft ≥ (4 / 3) M_u = [4 / 3] (- 0 ft·k / ft) = - 0 ft·k / ft

Check is waived per ACI 9.6.1.3 ✓

Shrinkage and Temperature Steel (ACI 318-14 13.2.8.1, 7.6.4.1, 24.4.3.2, 24.4.3.3)

$$\rho_{ST_prov} = \frac{A_{ST}}{t\, s_{ST}} = \frac{(0.4 \text{ in}^2 / \text{in})}{(14 \text{ in})(12 \text{ in})} = 0.0024$$

$$\rho_{ST_prov} = \frac{A_{ST}}{t\, s_{ST}} = \frac{(0.4 \text{ in}^2 / \text{in})}{(14 \text{ in})(12 \text{ in})} = 0.0024$$

$$\frac{0.0018 (60000)}{f_y} = \frac{0.0018 (60000)}{(60000 \text{ psi})} = 0.0018$$

ρ_{ST_min} = 0.0018

ρ_{ST_prov} = 0.0024 ≥ ρ_{ST_min} = 0.0018 ✓

18 inch limit governs

s_{ST_max} = 18 in

s_{ST} = 12 in ≤ s_{ST_max} = 18 in ✓

Development Check (ACI 318-14 13.2.8.1, 25.4.2.3, 25.4.10)

$$\frac{M_u}{\phi M_n} = \frac{(- 0 \text{ ft·k / ft})}{(26.39 \text{ ft·k / ft})} = - 0.0$$ (ratio to represent excess reinforcement)

ψ_e = 1.0 (uncoated hooked bars)

ψ_c = 0.70 (based on side cover and extension cover)

ψ_r = 1.0 (no confining reinforcement)

λ = 1.0 (normal weight concrete)

$$l_{dh} = \left(\frac{f_y \psi_e \psi_c \psi_r}{50 \lambda \sqrt{F'_c}} \right) d_b = \left[\frac{(60000 \text{ psi})(1.0)(0.70)(1.0)}{50 (1.0) \sqrt{3000 \text{ psi}}} \right] (0.75 \text{ in}) = 11.5 \text{ in}$$

Factoring l_{dh} by the excess reinforcement ratio (- 0.0000) per 25.4.10: l_{dh} = - 0 in

8 d_b = 8 (0.75 in) = 6.0 (minimum limit, does not control)

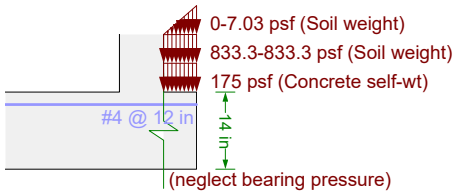
6 inch minimum controls

l_{dh_prov} = 6 in ≥ l_{dh} = 6 in ✓

Heel Checks [1.4D]

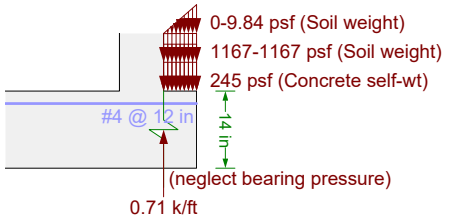
Heel Unfactored Loads

Unfactored Loads



Heel Factored Loads

1.4D



Controlling Moment

Design moment M_u for heel need not exceed moment at stem base:

M_{heel} = 0.18 ft·k / ft ≥ M_{stem} = - 0 ft·k / ft

M_u = - 0 ft·k / ft (stem base moment controls)

Flexure Check (ACI 318-14 13.3.2.1, 7.5.2.1, »22.3, »22.2, 7.5.1.1a)

$$a = \frac{A_s f_y}{0.85 f'_c} = \frac{(0.02 \text{ in}^2 / \text{in})(60000 \text{ psi})}{0.85 (3000 \text{ psi})} = 0.39 \text{ in}$$

$$\phi M_n = \phi A_s f_y (d - a / 2) = (0.90)(0.02 \text{ in}^2 / \text{in})(60000 \text{ psi})[(11.75 \text{ in}) - (0.39 \text{ in}) / 2] = 10.4 \text{ ft·k / ft}$$

ϕM_n = 10.4 ft·k / ft ≥ M_u = - 0 ft·k / ft ✓

Shear Check (ACI 318-14 13.3.2.1, 7.5.3.1, »22.5.1, »22.5.5, 7.5.1.1b)

λ = 1.0 (normal weight concrete)

$$V_c = 2 \lambda \sqrt{f'_c}\, d = 2 (1.0) \sqrt{3000 \text{ psi}}\, (11.75 \text{ in}) = 15.45 \text{ k / ft}$$

$$\phi V_n = \phi V_c = (0.750)(15.45 \text{ k / ft}) = 11.58 \text{ k / ft}$$

ϕV_n = 11.58 k / ft ≥ V_u = 0.71 k / ft ✓

Minimum Strain Check (ACI 318-14 13.3.2.1, 7.3.3.1)

β_1 = 0.850 ($f'_c \le 4000$ psi)

$$a = \frac{A_s f_y}{0.85 f'_c} = \frac{(0.02 \text{ in}^2 / \text{in})(60000 \text{ psi})}{0.85 (3000 \text{ psi})} = 0.39 \text{ in}$$

$$\varepsilon_t = 0.003 \left(\frac{d}{a / \beta_1} - 1 \right) = 0.003 \left[\frac{(11.75 \text{ in})}{(0.39 \text{ in}) / (0.850)} - 1 \right] = 0.0734$$

ε_t = 0.0734 ≥ 0.004 ✓

Minimum Steel Check (ACI 318-14 13.3.2.1, 9.6.1)

ϕM_n = 10.4 ft·k / ft ≥ (4 / 3) M_u = [4 / 3] (- 0 ft·k / ft) = - 0 ft·k / ft

Check is waived per ACI 9.6.1.3 ✓

Shrinkage and Temperature Steel (ACI 318-14 13.2.8.1, 7.6.4.1, 24.4.3.2, 24.4.3.3)

$$\rho_{ST_prov} = \frac{A_{ST}}{t\, s_{ST}} = \frac{(0.4 \text{ in}^2 / \text{in})}{(14 \text{ in})(12 \text{ in})} = 0.0024$$

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$$\frac{0.0018 (60000)}{f_y} = \frac{0.0018 (60000)}{(60000 \text{ psi})} = 0.0018$$

ρ_{ST_min} = 0.0018

ρ_{ST_prov} = 0.0024 ≥ ρ_{ST_min} = 0.0018 ✓

18 inch limit governs

s_{ST_max} = 18 in

s_{ST} = 12 in ≤ s_{ST_max} = 18 in ✓

Development Check (ACI 318-14 13.2.8.1, 25.4.2.3, 25.4.10)

$$\frac{M_u}{\phi M_n} = \frac{(- 0 \text{ ft·k / ft})}{(10.4 \text{ ft·k / ft})} = - 0.0$$ (ratio to represent excess reinforcement)

ψ_t = 1.0 (12 inches or less cast below - 11.50 inches)

ψ_e = 1.0 (bar not epoxy coated)

ψ_s = 0.80 (bars are #6 or smaller)

λ = 1.0 (normal weight concrete)

s / 2 = (12 in) / 2 = 6 in

cover + d_b / 2 = (2 in) + (0.5 in) / 2 = 2.25 in

c_b = 2.25 in (lesser of half spacing, ctr to surface)

K_{tr} = 0.0 (no transverse reinforcement)

$$\frac{c_b + K_{tr}}{d_b} = \frac{(2.25 \text{ in}) + (0.0)}{(0.5 \text{ in})} = 4.50$$

$$l_d = \left(\frac{3.}{40} \frac{f_y}{\lambda \sqrt{f'_c}} \frac{\psi_t \psi_e \psi_s}{2.5} \right) d_b = \left[\frac{3.}{40} \frac{(60000 \text{ psi})}{(1.0) \sqrt{3000 \text{ psi}}} \frac{(1.0)(1.0)(0.80)}{2.5} \right] (0.5 \text{ in}) = 13.15 \text{ in}$$

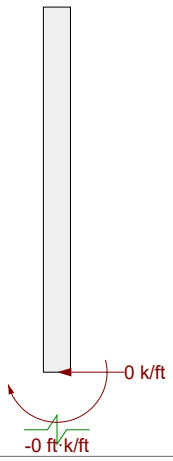
Factoring l_d by the excess reinforcement ratio (- 0.0000) per 25.4.10: l_d = - 0 in

12 inch minimum controls

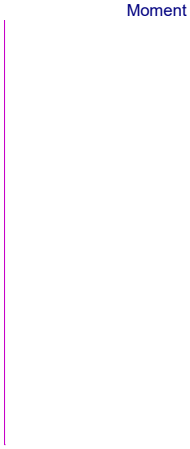
l_{d_prov} = 57.96 in ≥ l_d = 12 in ✓

Stem Forces [1.4D]

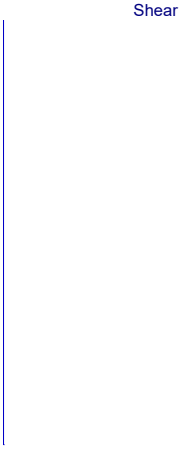
Stem Internal Forces



Stem Internal Forces



Stem Internal Forces

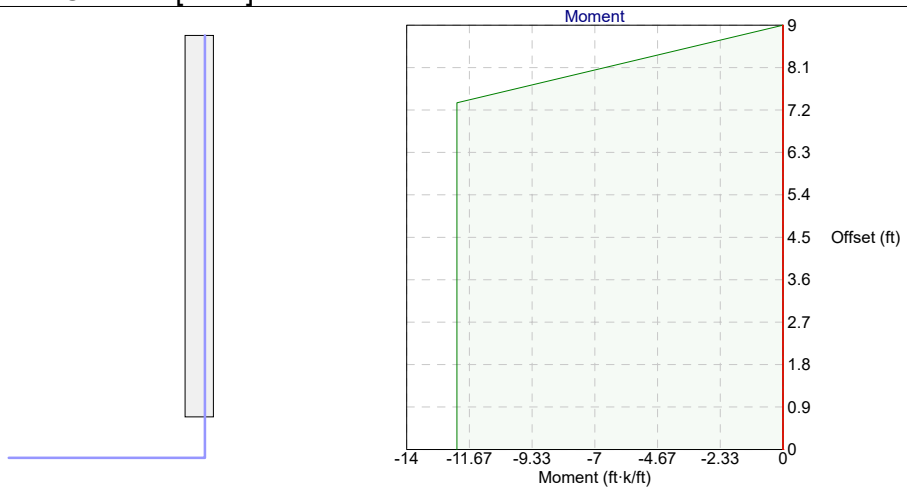


Stem Joint Force Transfer

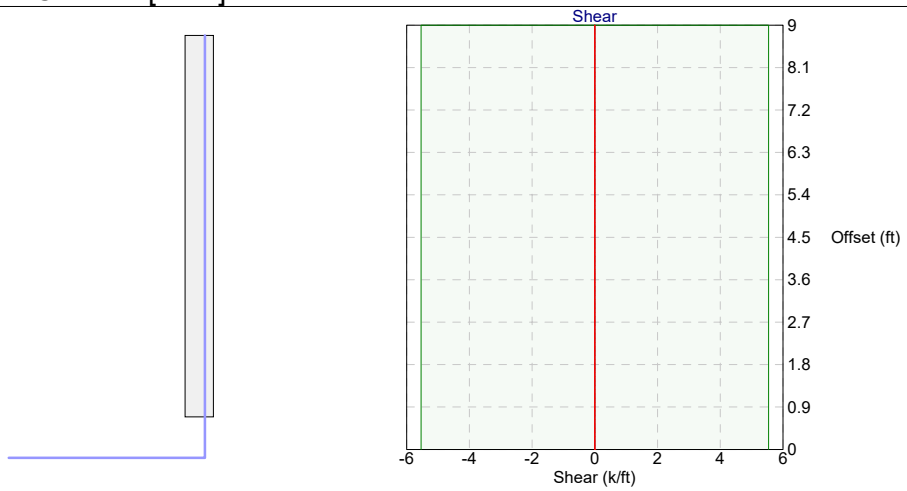
Location	Force
@ stem base	0 k/ft

Stem Internal Forces

Stem Moment Checks [1.4D]



Stem Shear Checks [1.4D]



Stem Miscellaneous Checks [1.4D]

Minimum Steel Check (ACI 318-14 9.6.1) @ 0 ft from base [Stem in negative flexure]

$\phi M_n = 12.14$ ft·k / ft $\geq (4 / 3) M_u = [4 / 3] (0$ ft·k / ft$) = 0$ ft·k / ft

Check is waived per ACI 9.6.1.3 ✓

Minimum Steel Check (ACI 318-14 9.6.1) @ 9 ft from base [Stem in negative flexure]

$\phi M_n = 0$ ft·k / ft $\geq (4 / 3) M_u = [4 / 3] (0$ ft·k / ft$) = 0$ ft·k / ft

Check is waived per ACI 9.6.1.3 ✓

Maximum Steel Check (ACI 318-14 9.3.3.1) @ 0 ft from base [Stem in negative flexure]

$\beta_1 = 0.850$ ($F'_c \leq 4000$ psi)

$$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0.04 \text{ in}^2 / \text{in}) (60000 \text{ psi})}{0.85 (3000 \text{ psi})} = 1.04 \text{ in}$$

$$\varepsilon_t = 0.003 \left(\frac{d}{a / \beta_1} - 1 \right) = 0.003 \left[\frac{(5.63 \text{ in})}{(1.04 \text{ in}) / (0.850)} - 1 \right] = 0.0109$$

$\varepsilon_t = 0.0109 \geq 0.004$ ✓

Maximum Steel Check (ACI 318-14 9.3.3.1) @ 9 ft from base [Stem in negative flexure]

$\beta_1 = 0.850$ ($F'_c \leq 4000$ psi)

$$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0.04 \text{ in}^2 / \text{in}) (60000 \text{ psi})}{0.85 (3000 \text{ psi})} = 1.04 \text{ in}$$

$$\varepsilon_t = 0.003 \left(\frac{d}{a / \beta_1} - 1 \right) = 0.003 \left[\frac{(5.63 \text{ in})}{(1.04 \text{ in}) / (0.850)} - 1 \right] = 0.0109$$

$\varepsilon_t = 0.0109 \geq 0.004$ ✓

Wall Horizontal Steel (ACI 318-14 11.6.1, 11.7.3)

$$\rho_t = \frac{A_{s_horz} / s_{horz}}{t} = \frac{(0.2 \text{ in}^2) / (12 \text{ in})}{(8 \text{ in})} = 0.0021$$

$\rho_{t_min} = 0.0020$ (bars No. 5 or less, not less than 60 ksi)

$\rho_t = 0.0021 \geq \rho_{t_min} = 0.0020$ ✓

$3 h = 3 (8 \text{ in}) = 24$ in

18 inch limit governs

$s_{horz} = 12$ in $\leq s_{horz_max} = 18$ in ✓

Development Check (ACI 318-14 11.7.1.2, 25.4.2.3, 25.4.10)

$\frac{M_u}{\phi M_n} = \frac{(0 \text{ ft·k / ft})}{(12.14 \text{ ft·k / ft})} = 0.0$ (ratio to represent excess reinforcement)

$\psi_e = 1.0$ (uncoated hooked bars)

$\psi_c = 0.70$ (based on side cover and extension cover)

$\psi_r = 1.0$ (no confining reinforcement)

$\lambda = 1.0$ (normal weight concrete)

$$l_{dh} = \left(\frac{f_y \psi_e \psi_c \psi_r}{50 \lambda \sqrt{F'_c}} \right) d_b = \left[\frac{(60000 \text{ psi}) (1.0) (0.70) (1.0)}{50 (1.0) \sqrt{3000 \text{ psi}}} \right] (0.75 \text{ in}) = 11.5 \text{ in}$$

Factoring l_{dh} by the excess reinforcement ratio (0.0000) per 25.4.10: $l_{dh} = 0$ in

$8 d_b = 8 (0.75 \text{ in}) = 6.0$ (minimum limit, does not control)

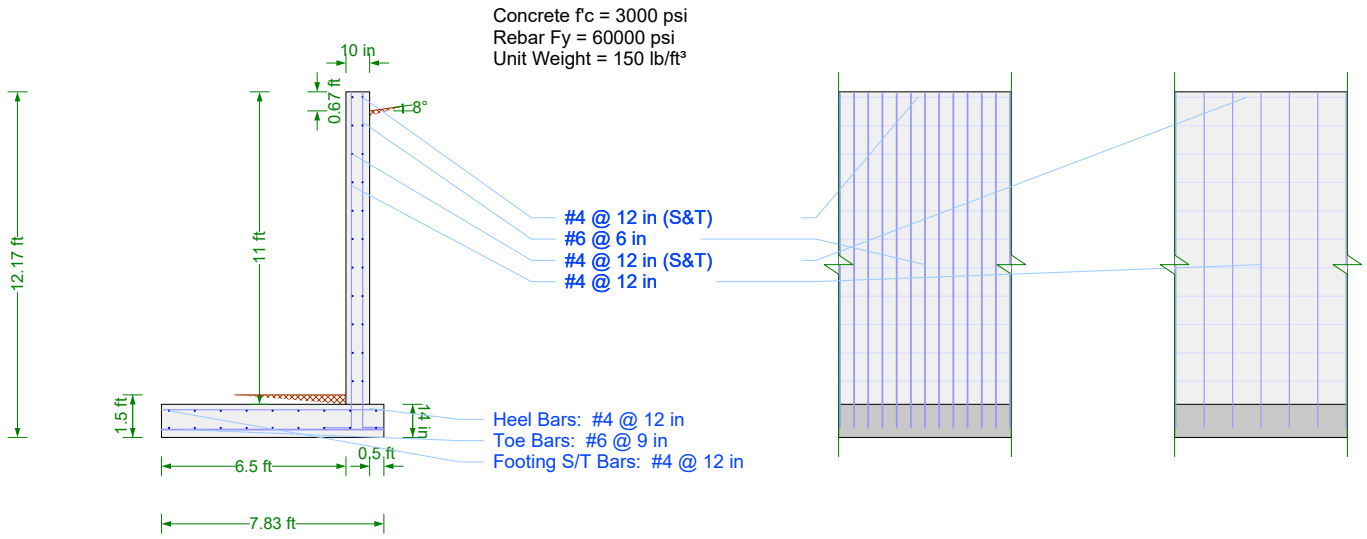
6 inch minimum controls

$l_{dh_prov} = 12$ in $\geq l_{dh} = 6$ in ✓

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Cantilever Wall 10' Tall

Design Detail



Check Summary

Ratio	Check	Provided	Required	Combination
----- Stability Checks -----				
✓ 0.903	Overturning	1.33	1.20	1.0D + 1.0L + 1.0H + 0.7E
✓ 0.722	Bearing Pressure	2500 psf	1804 psf	1.0D + 1.0L + 1.0H + 0.7E
✓ 0.492	Bearing Eccentricity	23.1 in	47 in	1.0D + 1.0L + 1.0H + 0.7E
----- Toe Checks -----				
✓ 0.473	Shear	10.48 k/ft	4.96 k/ft	1.2D + 0.5L + 1.6H + 1.0E
✓ 0.817	Moment	26.53 ft·k/ft	21.68 ft·k/ft	1.2D + 0.5L + 1.6H + 1.0E
✓ 0.195	Min Strain	0.0206	0.0040	1.2D + 1.6L + 1.6H
✓ 0.724	Min Steel	0.05 in²	0.04 in²	1.2D + 0.5L + 1.6H + 1.0E
✓ 1.000	Development	13 in	16.11 in	1.2D + 0.5L + 1.6H + 1.0E
✓ 0.667	S&T Max Spacing	12 in	18 in	1.2D + 1.6L + 1.6H
✓ 0.756	S&T Min Rho	0.0024	0.0018	1.2D + 1.6L + 1.6H
----- Heel Checks -----				
✓ 0.073	Shear	11.58 k/ft	0.85 k/ft	1.4D
✓ 0.018	Moment	10.4 ft·k/ft	0.18 ft·k/ft	1.2D + 1.6L + 1.6H
✓ 0.054	Min Strain	0.0734	0.0040	1.2D + 1.6L + 1.6H
✓ 0.000	Min Steel	0.02 in²	0 in²	1.2D + 1.6L + 1.6H
✓ 0.140	Development	86 in	12 in	1.2D + 1.6L + 1.6H
✓ 0.667	S&T Max Spacing	12 in	18 in	1.2D + 1.6L + 1.6H
✓ 0.756	S&T Min Rho	0.0024	0.0018	1.2D + 1.6L + 1.6H
----- Stem Checks -----				
✓ 0.810	Moment	26.78 ft·k/ft	21.68 ft·k/ft	1.2D + 0.5L + 1.6H + 1.0E
✓ 0.677	Shear	7.52 k/ft	5.09 k/ft	1.2D + 0.5L + 1.6H + 1.0E
✓ 0.484	Max Steel	0.0083	0.0040	1.2D + 1.6L + 1.6H
✓ 0.347	Min Steel	0.07 in²/in	0.03 in²/in	1.2D + 0.5L + 1.6H + 1.0E
✓ 0.847	Base Development	11 in	9.31 in	1.2D + 0.5L + 1.6H + 1.0E
✓ 0.000	Horz Bar Rho	0.0000	0.0000	1.2D + 1.6L + 1.6H
✓ 0.667	Horz Bar Spacing	12 in	18 in	1.2D + 1.6L + 1.6H

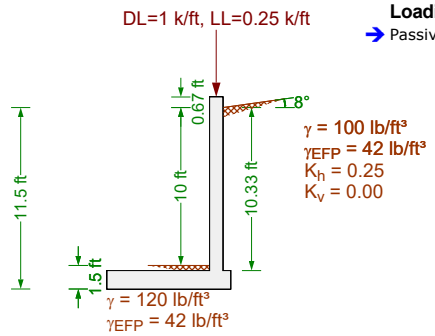
Criteria

Use basic criteria from common proje...	Yes
Building Code	IBC 2021
Concrete Load Combs	IBC 2021 (Strength)
Masonry Load Combs	ASCE 7-16 (ASD)
Stability Load Combs	IBC Retaining Wall St...
Apply Sds Factor to Seismic Combin...	No
Restrained Against Sliding	Yes
Neglect Bearing At Heel	Yes
Use Vert. Comp. for OT	No
Use Vert. Comp. for Sliding	No
Use Vert. Comp. for Bearing	Yes
Use Surcharge for Sliding & OT	Yes
Use Surcharge for Bearing	Yes
Neglect Soil Over Toe	No
Neglect Backfill Wt. for Coulomb	No
Factor Soil Weight As Dead	Yes
Use Passive Force for OT	Yes
Assume Pressure To Top	Yes
Extend Backfill Pressure To Key Bott...	No
Use Toe Passive Pressure for Bearing	No
Required F.S. for OT	1.50
Required F.S. for Sliding	1.50
Has Different Safety Factors for Seis...	Yes
Seismic F.S. for OT	1.20
Seismic F.S. for Sliding	1.20
Allowable Bearing Pressure	2500 psf
Req'd Bearing Location	Over footing
Wall Friction Angle	25°
Friction Coefficent	0.35
Soil Reaction Modulus	172800 lb/ft³

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Cantilever Wall 10' Tall

Loads



Loading Options/Assumptions

➔ Passive pressure neglects top 0 ft of soil.

Load Combinations

IBC 2018 (Strength)

1.2D + 1.6L + 1.6H
1.2D + 1.6L + 0.9H
1.2D + 0.5L + 1.6H + 1.0E
1.2D + 0.5L + 1.6H
1.2D + 0.5L + 0.9H + 1.0E
1.2D + 0.5L + 0.9H
1.2D + 1.6H + 1.0E
1.2D + 1.6H
1.2D + 0.9H + 1.0E
1.2D + 0.9H
0.9D + 1.6H + 1.0E
0.9D + 1.6H
0.9D + 0.9H + 1.0E
0.9D + 0.9H
1.4D

Strength Check Results Summary

Load Combination	Stem M-applied (ft·k/ft)	Stem M-allow (ft·k/ft)	Stem V-applied (k/ft)	Stem V-allow (k/ft)	Stem Min. ld (in)	Stem Actual ld (in)	Stem Min. strain	Stem Actual strain	Stem Min. steel (in²/in)
1.2D + 1.6L + 1.6H	12.36	26.78	3.59	7.52	6	11	0.0040	0.0083	0
1.2D + 1.6L + 0.9H	6.95	26.78	2.02	7.52	6	11	0.0040	0.0083	0
1.2D + 0.5L + 1.6H + 1.0E	21.68	26.78	5.09	7.52	9.31	11	0.0040	0.0083	0.03
1.2D + 0.5L + 1.6H	12.36	26.78	3.59	7.52	6	11	0.0040	0.0083	0
1.2D + 0.5L + 0.9H + 1.0E	16.27	26.78	3.52	7.52	6.99	11	0.0040	0.0083	0
1.2D + 0.5L + 0.9H	6.95	26.78	2.02	7.52	6	11	0.0040	0.0083	0
1.2D + 1.6H + 1.0E	21.68	26.78	5.09	7.52	9.31	11	0.0040	0.0083	0.03
1.2D + 1.6H	12.36	26.78	3.59	7.52	6	11	0.0040	0.0083	0
1.2D + 0.9H + 1.0E	16.27	26.78	3.52	7.52	6.99	11	0.0040	0.0083	0
1.2D + 0.9H	6.95	26.78	2.02	7.52	6	11	0.0040	0.0083	0
0.9D + 1.6H + 1.0E	21.68	26.78	5.09	7.52	9.31	11	0.0040	0.0083	0.03
0.9D + 1.6H	12.36	26.78	3.59	7.52	6	11	0.0040	0.0083	0
0.9D + 0.9H + 1.0E	16.27	26.78	3.52	7.52	6.99	11	0.0040	0.0083	0
0.9D + 0.9H	6.95	26.78	2.02	7.52	6	11	0.0040	0.0083	0
1.4D	0	0	0	0	6	11	0.0040	0.0083	0

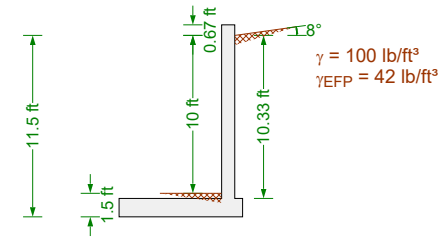
Load Combination	Stem Actual steel (in²/in)	Heel M-applied (ft·k/ft)	Heel M-allow (ft·k/ft)	Heel V-applied (k/ft)	Heel V-allow (k/ft)	Toe M-applied (ft·k/ft)	Toe M-allow (ft·k/ft)	Toe V-applied (k/ft)	Toe V-allow (k/ft)
1.2D + 1.6L + 1.6H	0.07	0.18	10.4	0.73	11.58	20.83	26.53	4.35	10.48
1.2D + 1.6L + 0.9H	0.07	0.18	10.4	0.73	11.58	20.83	26.53	4.35	10.48
1.2D + 0.5L + 1.6H + 1.0E	0.07	0.18	10.4	0.73	11.58	23.56	26.53	4.96	10.48
1.2D + 0.5L + 1.6H	0.07	0.18	10.4	0.73	11.58	19.59	26.53	4.08	10.48
1.2D + 0.5L + 0.9H + 1.0E	0.07	0.18	10.4	0.73	11.58	23.56	26.53	4.96	10.48
1.2D + 0.5L + 0.9H	0.07	0.18	10.4	0.73	11.58	19.59	26.53	4.08	10.48
1.2D + 1.6H + 1.0E	0.07	0.18	10.4	0.73	11.58	22.99	26.53	4.83	10.48
1.2D + 1.6H	0.07	0.18	10.4	0.73	11.58	19.03	26.53	3.96	10.48
1.2D + 0.9H + 1.0E	0.07	0.18	10.4	0.73	11.58	22.99	26.53	4.83	10.48
1.2D + 0.9H	0.07	0.18	10.4	0.73	11.58	19.03	26.53	3.96	10.48
0.9D + 1.6H + 1.0E	0.07	0.14	10.4	0.55	11.58	18.23	26.53	3.84	10.48
0.9D + 1.6H	0.07	0.14	10.4	0.55	11.58	14.27	26.53	2.97	10.48
0.9D + 0.9H + 1.0E	0.07	0.14	10.4	0.55	11.58	18.23	26.53	3.84	10.48
0.9D + 0.9H	0.07	0.14	10.4	0.55	11.58	14.27	26.53	2.97	10.48
1.4D	0.07	0.21	10.4	0.85	11.58	22.2	26.53	4.62	10.48

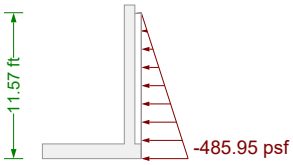
Stability Check Results Summary

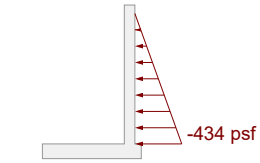
Load Combination	Overturning Moment (ft·k/ft)	Resisting Moment (ft·k/ft)	Overturning F.S.	Overturning F.S. Req'd	Overturning F.S. Req'd Seismic	Sliding Force (lb/in)	Resisting Force (lb/in)	Sliding F.S.
1.0D + 1.0L + 1.0H + 0.7E	10.84	26.6	2.453	1.500	1.200	234.3	161.1	0.688
1.0D + 1.0L + 1.0H	10.84	26.6	2.453	1.500	1.200	234.3	143.2	0.611
1.0D + 1.0H + 0.7E	10.84	26.6	2.453	1.500	1.200	234.3	153.8	0.657
1.0D + 1.0H	10.84	26.6	2.453	1.500	1.200	234.3	135.9	0.580

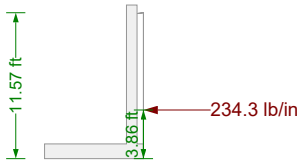
Load Combination	Sliding F.S. Req'd	Sliding F.S. Req'd Seismic	Bearing Pressure Actual (psf)	Bearing Pressure Allowable (psf)	Bearing Eccentricity Actual (in)	Bearing Eccentricity Allowable (in)	Wall Top Actual Deflection (in)
1.0D + 1.0L + 1.0H + 0.7E	1.500	1.200	1804	2500	23.1	47	0.44
1.0D + 1.0L + 1.0H	1.500	1.200	1598	2500	23.1	47	0.44
1.0D + 1.0H + 0.7E	1.500	1.200	1721	2500	23.1	47	0.44
1.0D + 1.0H	1.500	1.200	1515	2500	23.1	47	0.44

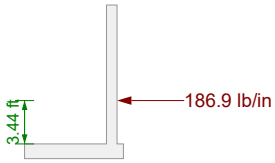
Backfill Pressure











Lateral Earth Pressure

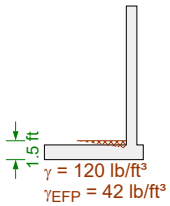
Equivalent Fluid Pressure

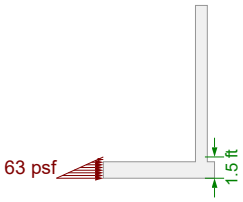
$\sigma_h = H\,\gamma_{fluid}$ = (11.57 ft)(42 lb / ft³) = 486 psf

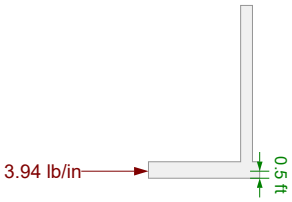
Lateral Earth Pressure (stem only)

$\sigma_h = H\,\gamma_{fluid}$ = (10.33 ft)(42 lb / ft³) = 434 psf

Passive Pressure





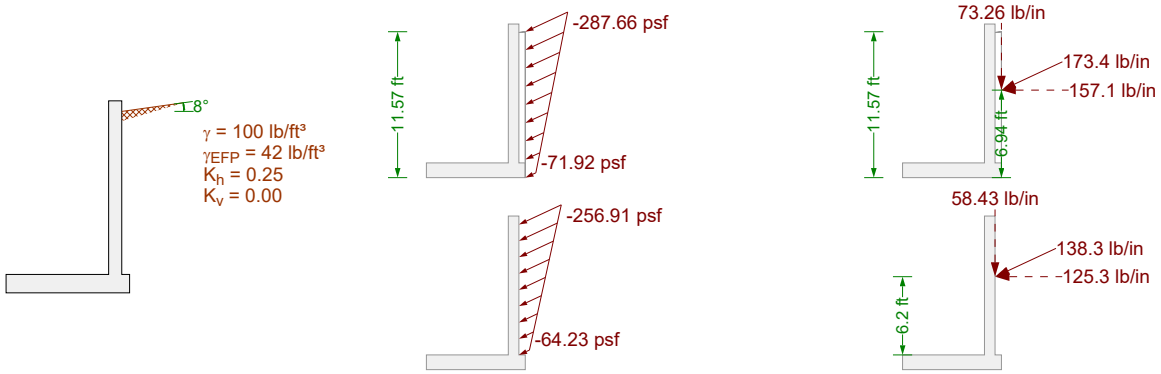


Lateral Earth Pressure

Equivalent Fluid Pressure

$\sigma_h = H\,\gamma_{fluid} = (1.5\text{ ft})(42\text{ lb / ft}^3) = 63\text{ psf}$

Seismic Pressure



Seismic Pressure

Dynamic + static force (Mononobe - Okabe equation)

$$\theta' = \text{atan}\left(\frac{k_h}{1 - k_v}\right) = \arctan\left[\frac{(0.250)}{1 - (0.0)}\right] = 14.04°$$

$$K_{ae} = \frac{\sin^2(\beta + \phi - \theta')}{\cos(\theta')\sin^2(\beta)\sin(\beta - \theta' - \delta)\left[1 + \sqrt{\frac{\sin(\phi + \delta)\sin(\phi - \theta' - \alpha)}{\sin(\beta - \delta - \theta')\sin(\alpha + \beta)}}\right]^{\wedge} 2}$$

$$= \frac{\sin^2[(90°) + (30°) - (14.04°)]}{\cos((14.04°))\sin^2((90°))\sin[(90°) - (14.04°) - (25°)]\left[1 + \sqrt{\frac{\sin[(30°) + (25°)]\sin[(30°) - (14.04°) - (8°)]}{\sin[(90°) - (25°) - (14.04°)]\sin[(8°) + (90°)]}}\right]^{\wedge} 2}$$

$$= 0.6403$$

$$P_{ae} = \frac{1}{2} K_{ae}\,\gamma\, H^{\wedge} 2\,(1 - k_v) = \frac{1}{2}(0.6403)(100\text{ lb / ft}^3)(11.57\text{ ft})^{\wedge} 2\,[1 - (0.0)] = 357.2\text{ lb / in}$$

Static - only force (Coulomb equation)

$$K_a = \frac{\sin^2(\beta + \phi)}{\sin^2(\beta)\sin(\beta - \delta)\left[1 + \sqrt{\frac{\sin(\phi + \delta)\sin(\phi - \alpha)}{\sin(\beta - \delta)\sin(\alpha + \beta)}}\right]^{\wedge} 2}$$

$$= \frac{\sin^2[(90°) + (30°)]}{\sin^2((90°))\sin[(90°) - (25°)]\left[1 + \sqrt{\frac{\sin[(30°) + (25°)]\sin[(30°) - (8°)]}{\sin[(90°) - (25°)]\sin[(8°) + (90°)]}}\right]^{\wedge} 2}$$

$$= 0.3295$$

$$P_a = \frac{1}{2} K_a\,\gamma\, H^{\wedge} 2 = \frac{1}{2}(0.3295)(100\text{ lb / ft}^3)(11.57\text{ ft})^{\wedge} 2 = 183.8\text{ lb / in}$$

Net dynamic force

$$\Delta P_{ae} = P_{ae} - P_a = (357.2\text{ lb / in}) - (183.8\text{ lb / in}) = 173.4\text{ lb / in}$$

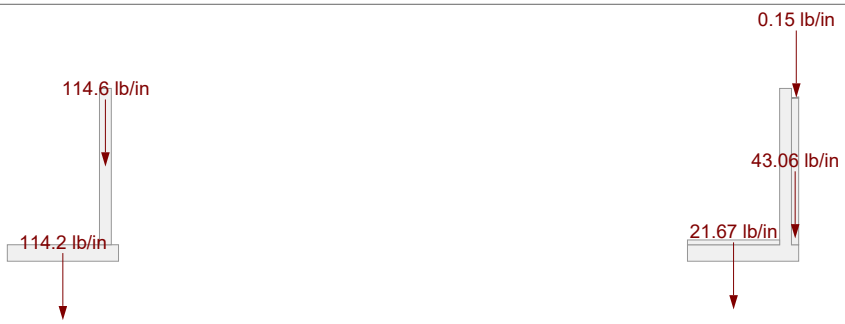
$$\alpha_P = 90° - \beta + \delta = 90° - (90°) + (25°) = 25°$$ (resultant force angle with horizontal)

To arrive at the pressure distribution illustrated above (used to determine stem moments), apply inverted triangular pressure plus a uniform portion to bring resultant to 0.6H

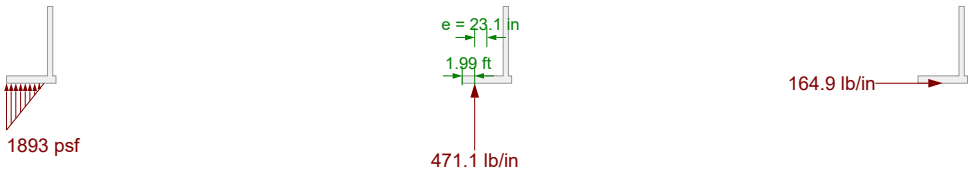
$$\sigma_{e_top} = \frac{8}{5}\frac{\Delta P_{ae}}{H} = \frac{8}{5}\frac{(173.4\text{ lb / in})}{(11.57\text{ ft})} = 287.7\text{ psf}$$

$$\sigma_{e_bot} = \frac{2}{5}\frac{\Delta P_{ae}}{H} = \frac{2}{5}\frac{(173.4\text{ lb / in})}{(11.57\text{ ft})} = 71.92\text{ psf}$$

Wall/Soil Weights



Bearing Pressure



Friction

$F = \mu R = (0.350)(471.1 \text{ lb / in}) = 164.9 \text{ lb / in}$

Bearing Pressure Calculation

Contributing Forces

	Vert Force	...offset	Horz Force	...offset	OT Moment
Backfill Pressure	-0 lb/in	-	-234.27 lb/in	3.86 ft	130110 in·lb/ft
Axial Dead Load	-83.33 lb/in	6.92 ft	0 lb/in	-	-83000 in·lb/ft
Axial Live Load	-20.83 lb/in	6.92 ft	0 lb/in	-	-20750 in·lb/ft
Seismic Force	-73.26 lb/in	7.83 ft	-157.11 lb/in	6.94 ft	74419 in·lb/ft
Footing Weight	-114.24 lb/in	3.92 ft	0 lb/in	-	-64429.17 in·lb/ft
Stem Weight	-114.58 lb/in	6.92 ft	0 lb/in	-	-114125 in·lb/ft
Backfill Weight	-43.06 lb/in	7.58 ft	0 lb/in	-	-47016.67 in·lb/ft
Backfill Weight	-0.15 lb/in	7.67 ft	0 lb/in	-	-161.62 in·lb/ft
Soil over toe Weight	-21.67 lb/in	3.25 ft	0 lb/in	-	-10140 in·lb/ft
	-471.12 lb/in				-135093.51 in·lb/ft

$$\frac{-135093.51 \text{ in·lb / ft}}{-471.12 \text{ lb / in}} = 1.99 \text{ ft}$$

Stability Checks [1.0D + 1.0L + 1.0H + 0.7E]

Overturning Check

Overturning Moments

	Force	Distance	Moment
Backfill pressure (horz)	234.3 lb/in	3.86 ft	130110 in·lb/ft
Seismic force	110 lb/in	6.94 ft	109941 in·lb/ft
		Total:	240050 in·lb/ft

Resisting Moments

	Force	Distance	Moment
Passive pressure @ toe	3.94 lb/in	0.5 ft	283.5 in·lb/ft
Axial dead load	-83.33 lb/in	6.92 ft	83000 in·lb/ft
Footing Weight	-114.24 lb/in	3.92 ft	64429 in·lb/ft
Stem Weight	-114.58 lb/in	6.92 ft	114125 in·lb/ft
Backfill Weight	-43.06 lb/in	7.58 ft	47017 in·lb/ft
Backfill Weight	-0.15 lb/in	7.67 ft	161.6 in·lb/ft
Soil over toe Weight	-21.67 lb/in	3.25 ft	10140 in·lb/ft
		Total:	319156 in·lb/ft

Without seismic loads:

$$F.S. = \frac{RM}{OTM} = \frac{319156 \text{ in·lb / ft}}{130110 \text{ in·lb / ft}} = 2.453 > 1.50 \text{ (OK)}$$

Including seismic loads:

$$F.S. = \frac{RM}{OTM} = \frac{319156 \text{ in·lb / ft}}{240050 \text{ in·lb / ft}} = 1.330 > 1.20 \text{ (OK)}$$

Sliding Check

Check not performed; restrained against sliding.

Bearing Capacity Check

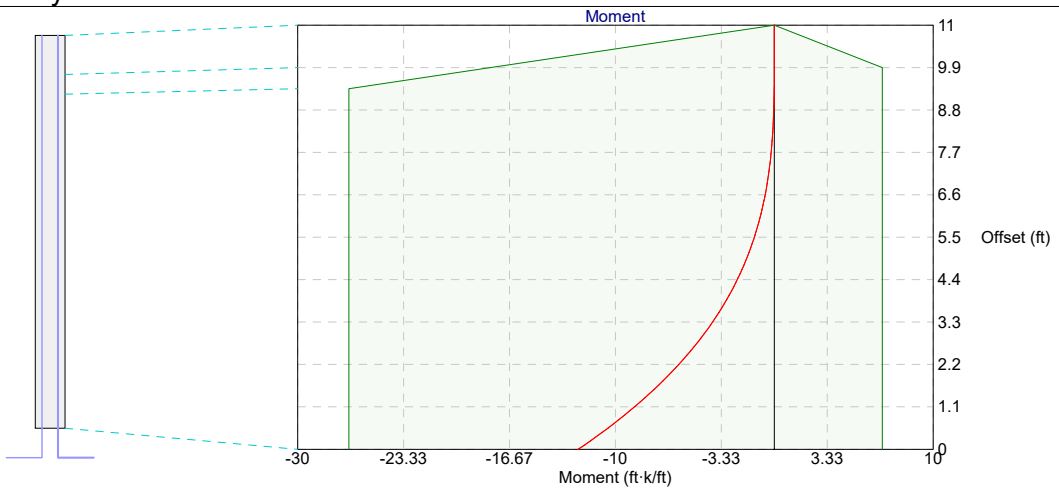
Bearing pressure < allowable (1804 psf < 2500 psf) - OK
Bearing resultant eccentricity < allowable (23.1 in < 47 in) - OK

Wall Top Displacement

(based on unfactored service loads)

Deflection due to stem flexural displacement	0.258 in
Deflection due to rotation from settlement	0.185 in
Total deflection at top of wall (positive towards toe)	0.443 in

Stem Flexural Capacity



Capacity (ACI 318-14 11.5.2.2, »22.3, »22.2) @ 0 ft from base [Negative bending]

$$a = \frac{A_s f_y}{0.85 f'_c} = \frac{(0.07 \text{ in}^2 / \text{in})(60000 \text{ psi})}{0.85 (3000 \text{ psi})} = 1.73 \text{ in}$$

$$\phi M_n = \phi A_s f_y (d - a / 2) = (0.90)(0.07 \text{ in}^2 / \text{in})(60000 \text{ psi})[(7.63 \text{ in}) - (1.73 \text{ in}) / 2] = 26.78 \text{ ft·k / ft}$$

Capacity (ACI 318-14 11.5.2.2, »22.3, »22.2) @ 0 ft from base [Positive bending]

$$a = \frac{A_s f_y}{0.85 f'_c} = \frac{(0.02 \text{ in}^2 / \text{in})(60000 \text{ psi})}{0.85 (3000 \text{ psi})} = 0.39 \text{ in}$$

$$\phi M_n = \phi A_s f_y (d - a / 2) = (0.90)(0.02 \text{ in}^2 / \text{in})(60000 \text{ psi})[(7.75 \text{ in}) - (0.39 \text{ in}) / 2] = 6.8 \text{ ft·k / ft}$$

Capacity (ACI 318-14 11.5.2.2, »22.3, »22.2) @ 9.36 ft from base [Negative bending]

$$a = \frac{A_s f_y}{0.85 f'_c} = \frac{(0.07 \text{ in}^2 / \text{in})(60000 \text{ psi})}{0.85 (3000 \text{ psi})} = 1.73 \text{ in}$$

$$\phi M_n = \phi A_s f_y (d - a / 2) = (0.90)(0.07 \text{ in}^2 / \text{in})(60000 \text{ psi})[(7.63 \text{ in}) - (1.73 \text{ in}) / 2] = 26.78 \text{ ft·k / ft}$$

Capacity (ACI 318-14 11.5.2.2, »22.3, »22.2) @ 9.9 ft from base [Positive bending]

$$a = \frac{A_s f_y}{0.85 f'_c} = \frac{(0.02 \text{ in}^2 / \text{in})(60000 \text{ psi})}{0.85 (3000 \text{ psi})} = 0.39 \text{ in}$$

$$\phi M_n = \phi A_s f_y (d - a / 2) = (0.90)(0.02 \text{ in}^2 / \text{in})(60000 \text{ psi})[(7.75 \text{ in}) - (0.39 \text{ in}) / 2] = 6.8 \text{ ft·k / ft}$$

Capacity (ACI 318-14 11.5.2.2, »22.3, »22.2) @ 11 ft from base [Negative bending]

$$a = \frac{A_s f_y}{0.85 f'_c} = \frac{(0 \text{ in}^2 / \text{in})(60000 \text{ psi})}{0.85 (3000 \text{ psi})} = 0 \text{ in}$$

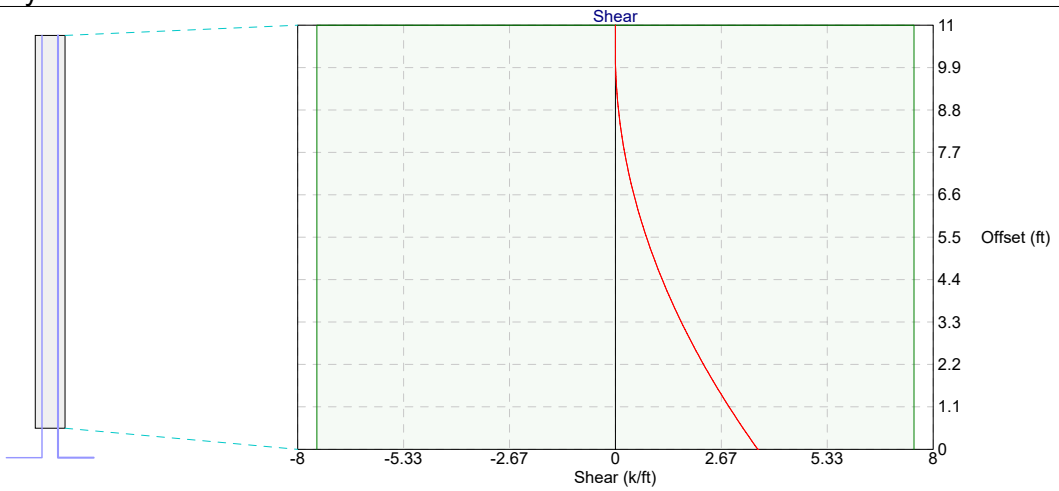
$$\phi M_n = \phi A_s f_y (d - a / 2) = (0.90)(0 \text{ in}^2 / \text{in})(60000 \text{ psi})[(7.63 \text{ in}) - (0 \text{ in}) / 2] = 0 \text{ ft·k / ft}$$

Capacity (ACI 318-14 11.5.2.2, »22.3, »22.2) @ 11 ft from base [Positive bending]

$$a = \frac{A_s f_y}{0.85 f'_c} = \frac{(0 \text{ in}^2 / \text{in})(60000 \text{ psi})}{0.85 (3000 \text{ psi})} = 0 \text{ in}$$

$$\phi M_n = \phi A_s f_y (d - a / 2) = (0.90)(0 \text{ in}^2 / \text{in})(60000 \text{ psi})[(7.75 \text{ in}) - (0 \text{ in}) / 2] = 0 \text{ ft·k / ft}$$

Stem Shear Capacity



Shear Capacity (ACI 318-14 11.5.5.1, 22.5.1.1, 22.5.5.1) @ 0 ft from base [Positive shear]

λ = 1.0 (normal weight concrete)

$V_c = 2\,\lambda\,\sqrt{F'_c}\,d$ = 2 (1.0) $\sqrt{3000\ psi}$ (7.63 in) = 10.02 k / ft

$\phi V_n = \phi\,V_c$ = (0.750) (10.02 k / ft) = 7.52 k / ft

Shear Capacity (ACI 318-14 11.5.5.1, 22.5.1.1, 22.5.5.1) @ 0 ft from base [Negative shear]

λ = 1.0 (normal weight concrete)

$V_c = 2\,\lambda\,\sqrt{F'_c}\,d$ = 2 (1.0) $\sqrt{3000\ psi}$ (7.63 in) = 10.02 k / ft

$\phi V_n = \phi\,V_c$ = (0.750) (10.02 k / ft) = 7.52 k / ft

Shear Capacity (ACI 318-14 11.5.5.1, 22.5.1.1, 22.5.5.1) @ 11 ft from base [Positive shear]

λ = 1.0 (normal weight concrete)

$V_c = 2\,\lambda\,\sqrt{F'_c}\,d$ = 2 (1.0) $\sqrt{3000\ psi}$ (7.63 in) = 10.02 k / ft

$\phi V_n = \phi\,V_c$ = (0.750) (10.02 k / ft) = 7.52 k / ft

Shear Capacity (ACI 318-14 11.5.5.1, 22.5.1.1, 22.5.5.1) @ 11 ft from base [Negative shear]

λ = 1.0 (normal weight concrete)

$V_c = 2\,\lambda\,\sqrt{F'_c}\,d$ = 2 (1.0) $\sqrt{3000\ psi}$ (7.63 in) = 10.02 k / ft

$\phi V_n = \phi\,V_c$ = (0.750) (10.02 k / ft) = 7.52 k / ft

Stem Development/Lap Length Calculations

Main vertical stem bars (bottom end) - Development Length Calculation (ACI 318-14 11.7.1.2, 25.4.2.3, 25.4.3)

ψ_e = 1.0 (uncoated hooked bars)

ψ_c = 0.70 (based on side cover and extension cover)

ψ_r = 1.0 (no confining reinforcement)

λ = 1.0 (normal weight concrete)

$$l_{dh} = \left(\frac{f_y \, \psi_e \, \psi_c \, \psi_r}{50 \, \lambda \sqrt{F'_c}} \right) d_b = \left[\frac{(60000 \text{ psi}) \, (1.0) \, (0.70) \, (1.0)}{50 \, (1.0) \sqrt{3000 \text{ psi}}} \right] (0.75 \text{ in}) = 11.5 \text{ in}$$

$8 \, d_b = 8 \, (0.75 \text{ in}) = 6.0$ (minimum limit, does not control)

Main vertical stem bars (top end) - Development Length Calculation (ACI 318-14 11.7.1.2, 25.4.2.3, 25.4.3)

ψ_t = 1.0 (bars are not horizontal)

ψ_e = 1.0 (bar not epoxy coated)

ψ_s = 0.80 (bars are #6 or smaller)

λ = 1.0 (normal weight concrete)

$s / 2 = (6 \text{ in}) / 2 = 3 \text{ in}$

$\text{cover} + d_b / 2 = (2 \text{ in}) + (0.75 \text{ in}) / 2 = 2.38 \text{ in}$

c_b = 2.38 in (lesser of half spacing, ctr to surface)

K_{tr} = 0.0 (no transverse reinforcement)

$$\frac{c_b + K_{tr}}{d_b} = \frac{(2.38 \text{ in}) + (0.0)}{(0.75 \text{ in})} = 3.1667$$

$$l_d = \left(\frac{3.}{40} \frac{f_y}{\lambda \sqrt{F'_c}} \frac{\psi_t \, \psi_e \, \psi_s}{2.5} \right) d_b = \left[\frac{3.}{40} \frac{(60000 \text{ psi})}{(1.0) \sqrt{3000 \text{ psi}}} \frac{(1.0) \, (1.0) \, (0.80)}{2.5} \right] (0.75 \text{ in}) = 19.72 \text{ in}$$

2nd curtain vertical bars (top end) - Development Length Calculation (ACI 318-14 11.7.1.2, 25.4.2.3, 25.4.3)

ψ_t = 1.0 (bars are not horizontal)

ψ_e = 1.0 (bar not epoxy coated)

ψ_s = 0.80 (bars are #6 or smaller)

λ = 1.0 (normal weight concrete)

$s / 2 = (12 \text{ in}) / 2 = 6 \text{ in}$

$\text{cover} + d_b / 2 = (2 \text{ in}) + (0.5 \text{ in}) / 2 = 2.25 \text{ in}$

c_b = 2.25 in (lesser of half spacing, ctr to surface)

K_{tr} = 0.0 (no transverse reinforcement)

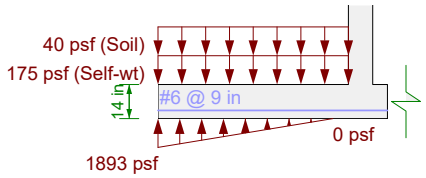
$$\frac{c_b + K_{tr}}{d_b} = \frac{(2.25 \text{ in}) + (0.0)}{(0.5 \text{ in})} = 4.50$$

$$l_d = \left(\frac{3.}{40} \frac{f_y}{\lambda \sqrt{F'_c}} \frac{\psi_t \, \psi_e \, \psi_s}{2.5} \right) d_b = \left[\frac{3.}{40} \frac{(60000 \text{ psi})}{(1.0) \sqrt{3000 \text{ psi}}} \frac{(1.0) \, (1.0) \, (0.80)}{2.5} \right] (0.5 \text{ in}) = 13.15 \text{ in}$$

Toe Checks [1.2D + 1.6L + 1.6H]

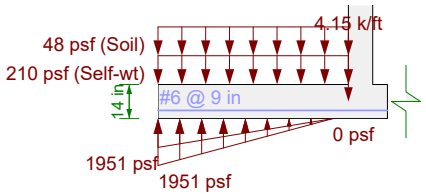
Toe Unfactored Loads

Unfactored Loads



Toe Factored Loads

1.2D + 1.6L + 1.6H



Controlling Moment

Design moment M_u for toe need not exceed moment at stem base:

M_{toe} = 20.83 ft·k / ft ≥ M_{stem} = 12.36 ft·k / ft

M_u = 12.36 ft·k / ft (stem base moment controls)

Flexure Check (ACI 318-14 13.3.2.1, 7.5.2.1, »22.3, »22.2, 7.5.1.1a)

$$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0.05 \text{ in}^2 / \text{in})(60000 \text{ psi})}{0.85 (3000 \text{ psi})} = 1.15 \text{ in}$$

$\phi M_n = \phi A_s f_y (d - a / 2)$ = (0.90) (0.05 in² / in) (60000 psi) [(10.63 in) - (1.15 in) / 2] = 26.53 ft·k / ft

ϕM_n = 26.53 ft·k / ft ≥ M_u = 12.36 ft·k / ft ✓

Shear Check (ACI 318-14 13.3.2.1, 7.5.3.1, »22.5.1, »22.5.5, 7.5.1.1b)

λ = 1.0 (normal weight concrete)

$V_c = 2 \lambda \sqrt{F'_c}\, d = 2 (1.0) \sqrt{3000 \text{ psi}} (10.63 \text{ in})$ = 13.97 k / ft

$\phi V_n = \phi V_c$ = (0.750) (13.97 k / ft) = 10.48 k / ft

ϕV_n = 10.48 k / ft ≥ V_u = 4.35 k / ft ✓

Minimum Strain Check (ACI 318-14 13.3.2.1, 7.3.3.1)

β_1 = 0.850 ($F'_c \le 4000$ psi)

$$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0.05 \text{ in}^2 / \text{in})(60000 \text{ psi})}{0.85 (3000 \text{ psi})} = 1.15 \text{ in}$$

$$\varepsilon_t = 0.003 \left(\frac{d}{a / \beta_1} - 1 \right) = 0.003 \left[\frac{(10.63 \text{ in})}{(1.15 \text{ in}) / (0.850)} - 1 \right] = 0.0206$$

ε_t = 0.0206 ≥ 0.004 ✓

Minimum Steel Check (ACI 318-14 13.3.2.1, 9.6.1)

ϕM_n = 26.53 ft·k / ft ≥ (4 / 3) M_u = [4 / 3] (12.36 ft·k / ft) = 16.48 ft·k / ft

Check is waived per ACI 9.6.1.3 ✓

Shrinkage and Temperature Steel (ACI 318-14 13.2.8.1, 7.6.4.1, 24.4.3.2, 24.4.3.3)

$$\rho_{ST_prov} = \frac{A_{ST}}{t\, s_{ST}} = \frac{(0.4 \text{ in}^2 / \text{in})}{(14 \text{ in})(12 \text{ in})} = 0.0024$$

$$\rho_{ST_prov} = \frac{A_{ST}}{t\, s_{ST}} = \frac{(0.4 \text{ in}^2 / \text{in})}{(14 \text{ in})(12 \text{ in})} = 0.0024$$

$$\frac{0.0018 (60000)}{f_y} = \frac{0.0018 (60000)}{(60000 \text{ psi})} = 0.0018$$

ρ_{ST_min} = 0.0018

ρ_{ST_prov} = 0.0024 ≥ ρ_{ST_min} = 0.0018 ✓

18 inch limit governs

s_{ST_max} = 18 in

s_{ST} = 12 in ≤ s_{ST_max} = 18 in ✓

Development Check (ACI 318-14 13.2.8.1, 25.4.2.3, 25.4.10)

$$\frac{M_u}{\phi M_n} = \frac{(12.36 \text{ ft·k / ft})}{(26.53 \text{ ft·k / ft})} = 0.4658$$ (ratio to represent excess reinforcement)

ψ_t = 1.0 (12 inches or less cast below - 3.00 inches)

ψ_e = 1.0 (bar not epoxy coated)

ψ_s = 0.80 (bars are #6 or smaller)

λ = 1.0 (normal weight concrete)

s / 2 = (9 in) / 2 = 4.5 in

cover + d_b / 2 = (3 in) + (0.75 in) / 2 = 3.38 in

c_b = 3.38 in (lesser of half spacing, ctr to surface)

K_{tr} = 0.0 (no transverse reinforcement)

$$\frac{c_b + K_{tr}}{d_b} = \frac{(3.38 \text{ in}) + (0.0)}{(0.75 \text{ in})} = 4.50$$

$$l_d = \left(\frac{3.}{40} \frac{f_y}{\lambda \sqrt{F'_c}} \frac{\psi_t \psi_e \psi_s}{2.5} \right) d_b = \left[\frac{3.}{40} \frac{(60000 \text{ psi})}{(1.0) \sqrt{3000 \text{ psi}}} \frac{(1.0)(1.0)(0.80)}{2.5} \right] (0.75 \text{ in}) = 19.72 \text{ in}$$

Factoring l_d by the excess reinforcement ratio (0.4658) per 25.4.10: l_d = 9.18 in

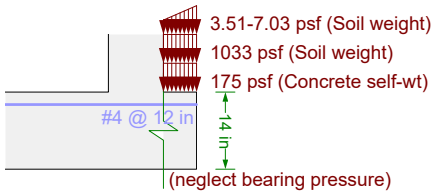
12 inch minimum controls

l_{d_prov} = 13 in ≥ l_d = 12 in ✓

Heel Checks [1.2D + 1.6L + 1.6H]

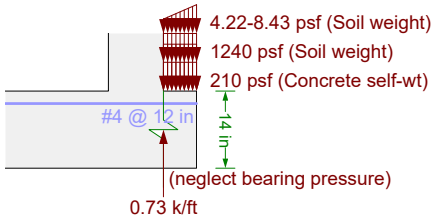
Heel Unfactored Loads

Unfactored Loads



Heel Factored Loads

1.2D + 1.6L + 1.6H



Controlling Moment

Design moment M_u for heel need not exceed moment at stem base:

M_{heel} = 0.18 ft·k / ft < M_{stem} = 12.36 ft·k / ft

M_u = 0.18 ft·k / ft (stem moment does not control)

Flexure Check (ACI 318-14 13.3.2.1, 7.5.2.1, »22.3, »22.2, 7.5.1.1a)

$$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0.02 \text{ in}^2 / \text{in})(60000 \text{ psi})}{0.85 (3000 \text{ psi})} = 0.39 \text{ in}$$

$\phi M_n = \phi A_s f_y (d - a / 2)$ = (0.90) (0.02 in² / in) (60000 psi) [(11.75 in) - (0.39 in) / 2] = 10.4 ft·k / ft

ϕM_n = 10.4 ft·k / ft ≥ M_u = 0.18 ft·k / ft ✓

Shear Check (ACI 318-14 13.3.2.1, 7.5.3.1, »22.5.1, »22.5.5, 7.5.1.1b)

λ = 1.0 (normal weight concrete)

$V_c = 2 \lambda \sqrt{F'_c}\, d = 2 (1.0) \sqrt{3000 \text{ psi}} (11.75 \text{ in})$ = 15.45 k / ft

$\phi V_n = \phi V_c$ = (0.750) (15.45 k / ft) = 11.58 k / ft

ϕV_n = 11.58 k / ft ≥ V_u = 0.73 k / ft ✓

Minimum Strain Check (ACI 318-14 13.3.2.1, 7.3.3.1)

β_1 = 0.850 ($F'_c \le 4000$ psi)

$$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0.02 \text{ in}^2 / \text{in})(60000 \text{ psi})}{0.85 (3000 \text{ psi})} = 0.39 \text{ in}$$

$$\varepsilon_t = 0.003 \left(\frac{d}{a / \beta_1} - 1 \right) = 0.003 \left[\frac{(11.75 \text{ in})}{(0.39 \text{ in}) / (0.850)} - 1 \right] = 0.0734$$

ε_t = 0.0734 ≥ 0.004 ✓

Minimum Steel Check (ACI 318-14 13.3.2.1, 9.6.1)

ϕM_n = 10.4 ft·k / ft ≥ (4 / 3) M_u = [4 / 3] (0.18 ft·k / ft) = 0.24 ft·k / ft

Check is waived per ACI 9.6.1.3 ✓

Shrinkage and Temperature Steel (ACI 318-14 13.2.8.1, 7.6.4.1, 24.4.3.2, 24.4.3.3)

$$\rho_{ST_prov} = \frac{A_{ST}}{t\, s_{ST}} = \frac{(0.4 \text{ in}^2 / \text{in})}{(14 \text{ in})(12 \text{ in})} = 0.0024$$

$$\rho_{ST_prov} = \frac{A_{ST}}{t\, s_{ST}} = \frac{(0.4 \text{ in}^2 / \text{in})}{(14 \text{ in})(12 \text{ in})} = 0.0024$$

$$\frac{0.0018 (60000)}{f_y} = \frac{0.0018 (60000)}{(60000 \text{ psi})} = 0.0018$$

ρ_{ST_min} = 0.0018

ρ_{ST_prov} = 0.0024 ≥ ρ_{ST_min} = 0.0018 ✓

18 inch limit governs

s_{ST_max} = 18 in

s_{ST} = 12 in ≤ s_{ST_max} = 18 in ✓

Development Check (ACI 318-14 13.2.8.1, 25.4.2.3, 25.4.10)

$$\frac{M_u}{\phi M_n} = \frac{(0.18 \text{ ft·k / ft})}{(10.4 \text{ ft·k / ft})} = 0.0175$$ (ratio to represent excess reinforcement)

ψ_t = 1.0 (12 inches or less cast below - 11.50 inches)

ψ_e = 1.0 (bar not epoxy coated)

ψ_s = 0.80 (bars are #6 or smaller)

λ = 1.0 (normal weight concrete)

s / 2 = (12 in) / 2 = 6 in

cover + d_b / 2 = (2 in) + (0.5 in) / 2 = 2.25 in

c_b = 2.25 in (lesser of half spacing, ctr to surface)

K_{tr} = 0.0 (no transverse reinforcement)

$$\frac{c_b + K_{tr}}{d_b} = \frac{(2.25 \text{ in}) + (0.0)}{(0.5 \text{ in})} = 4.50$$

$$l_d = \left(\frac{3.}{40} \frac{f_y}{\lambda \sqrt{F'_c}} \frac{\psi_t \psi_e \psi_s}{2.5} \right) d_b = \left[\frac{3.}{40} \frac{(60000 \text{ psi})}{(1.0) \sqrt{3000 \text{ psi}}} \frac{(1.0)(1.0)(0.80)}{2.5} \right] (0.5 \text{ in}) = 13.15 \text{ in}$$

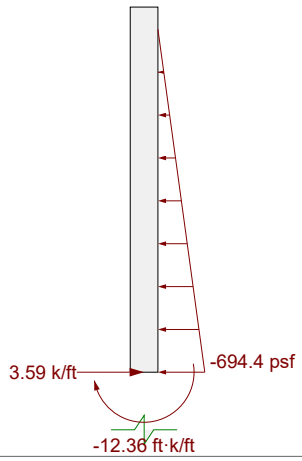
Factoring l_d by the excess reinforcement ratio (0.0175) per 25.4.10: l_d = 0.23 in

12 inch minimum controls

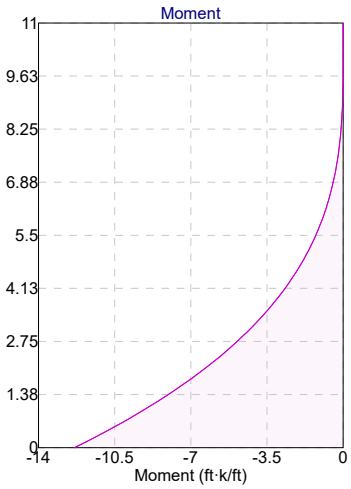
l_{d_prov} = 86 in ≥ l_d = 12 in ✓

Stem Forces [1.2D + 1.6L + 1.6H]

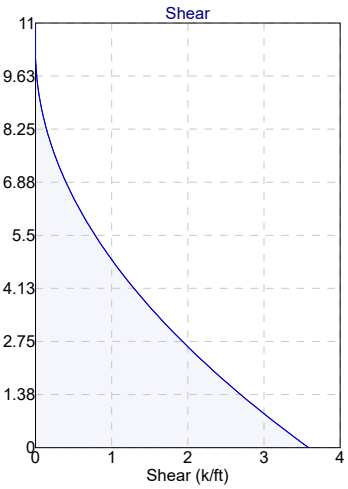
Stem Internal Forces



Stem Internal Forces



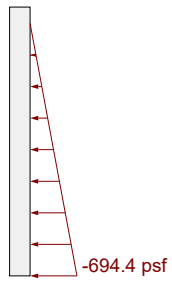
Stem Internal Forces



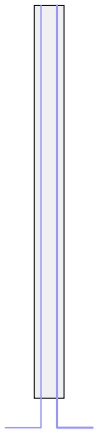
Stem Joint Force Transfer

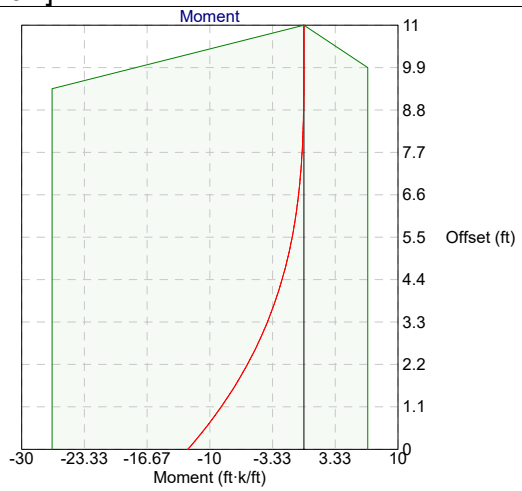
Location	Force
@ stem base	3.59 k/ft

Stem Internal Forces



Stem Moment Checks [1.2D + 1.6L + 1.6H]





Check (ACI 318-14 11.5.5.1b) @ 0 ft from base

ϕM_n = 26.78 ft·k / ft ≥ M_u = 12.36 ft·k / ft ✓

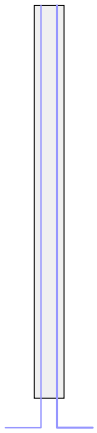
Check (ACI 318-14 11.5.5.1b) @ 9.36 ft from base

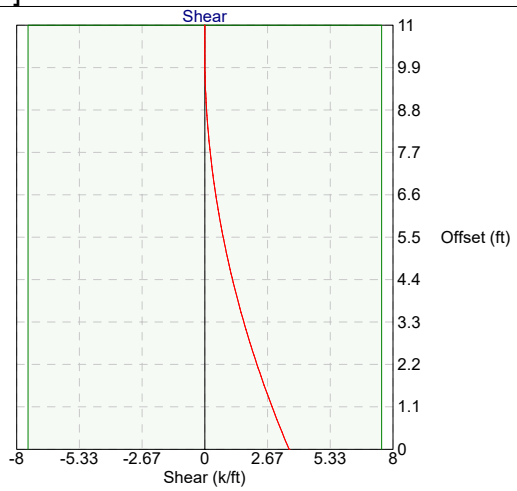
ϕM_n = 26.78 ft·k / ft ≥ M_u = 0.01 ft·k / ft ✓

Check (ACI 318-14 11.5.5.1b) @ 9.44 ft from base

ϕM_n = 25.35 ft·k / ft ≥ M_u = 0.01 ft·k / ft ✓

Stem Shear Checks [1.2D + 1.6L + 1.6H]





Shear Check (ACI 318-14 11.5.5.1c) @ 0 ft from base

ϕV_n = 7.52 k / ft ≥ V_u = 3.59 k / ft ✓

Stem Miscellaneous Checks [1.2D + 1.6L + 1.6H]

Minimum Steel Check (ACI 318-14 9.6.1) @ 0 ft from base [Stem in negative flexure]

$\phi M_n = 26.78$ ft·k / ft $\geq (4 / 3) M_u = [4 / 3] (12.36$ ft·k / ft$) = 16.48$ ft·k / ft

Check is waived per ACI 9.6.1.3 ✓

Minimum Steel Check (ACI 318-14 9.6.1) @ 11 ft from base [Stem in negative flexure]

$\phi M_n = 0$ ft·k / ft $\geq (4 / 3) M_u = [4 / 3] (0$ ft·k / ft$) = 0$ ft·k / ft

Check is waived per ACI 9.6.1.3 ✓

Maximum Steel Check (ACI 318-14 9.3.3.1) @ 0 ft from base [Stem in negative flexure]

$\beta_1 = 0.850$ ($F'_c \leq 4000$ psi)

$$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0.07 \text{ in}^2 / \text{in}) (60000 \text{ psi})}{0.85 (3000 \text{ psi})} = 1.73 \text{ in}$$

$$\varepsilon_t = 0.003 \left(\frac{d}{a / \beta_1} - 1 \right) = 0.003 \left[\frac{(7.63 \text{ in})}{(1.73 \text{ in}) / (0.850)} - 1 \right] = 0.0083$$

$\varepsilon_t = 0.0083 \geq 0.004$ ✓

Maximum Steel Check (ACI 318-14 9.3.3.1) @ 11 ft from base [Stem in negative flexure]

$\beta_1 = 0.850$ ($F'_c \leq 4000$ psi)

$$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0.07 \text{ in}^2 / \text{in}) (60000 \text{ psi})}{0.85 (3000 \text{ psi})} = 1.73 \text{ in}$$

$$\varepsilon_t = 0.003 \left(\frac{d}{a / \beta_1} - 1 \right) = 0.003 \left[\frac{(7.63 \text{ in})}{(1.73 \text{ in}) / (0.850)} - 1 \right] = 0.0083$$

$\varepsilon_t = 0.0083 \geq 0.004$ ✓

Wall Horizontal Steel (ACI 318-14 11.6.1, 11.7.3)

$$\rho_t = \frac{A_{s_horz} / s_{horz}}{t} = \frac{(0.4 \text{ in}^2) / (12 \text{ in})}{(10 \text{ in})} = 0.0033$$

$\rho_{t_min} = 0.0020$ (bars No. 5 or less, not less than 60 ksi)

$\rho_t = 0.0033 \geq \rho_{t_min} = 0.0020$ ✓

$3 h = 3 (10 \text{ in}) = 30$ in

18 inch limit governs

$s_{horz} = 12$ in $\leq s_{horz_max} = 18$ in ✓

Development Check (ACI 318-14 11.7.1.2, 25.4.2.3, 25.4.10)

$\frac{M_u}{\phi M_n} = \frac{(12.36 \text{ ft·k / ft})}{(26.78 \text{ ft·k / ft})} = 0.4615$ (ratio to represent excess reinforcement)

$\psi_e = 1.0$ (uncoated hooked bars)

$\psi_c = 0.70$ (based on side cover and extension cover)

$\psi_r = 1.0$ (no confining reinforcement)

$\lambda = 1.0$ (normal weight concrete)

$$l_{dh} = \left(\frac{f_y \psi_e \psi_c \psi_r}{50 \lambda \sqrt{F'_c}} \right) d_b = \left[\frac{(60000 \text{ psi}) (1.0) (0.70) (1.0)}{50 (1.0) \sqrt{3000 \text{ psi}}} \right] (0.75 \text{ in}) = 11.5 \text{ in}$$

Factoring l_{dh} by the excess reinforcement ratio (0.4615) per 25.4.10: $l_{dh} = 5.31$ in

$8 d_b = 8 (0.75 \text{ in}) = 6.0$ (minimum limit, does not control)

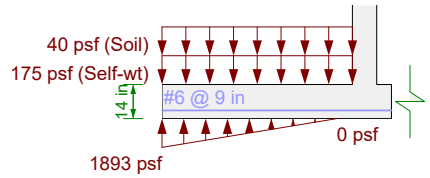
6 inch minimum controls

$l_{dh_prov} = 11$ in $\geq l_{dh} = 6$ in ✓

Toe Checks [1.2D + 0.5L + 1.6H + 1.0E]

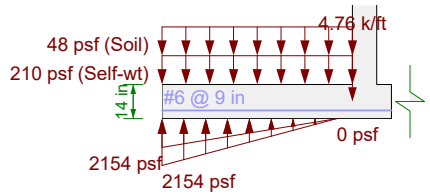
Toe Unfactored Loads

Unfactored Loads



Toe Factored Loads

1.2D + 0.5L + 1.6H + 1.0E



Controlling Moment

Design moment M_u for toe need not exceed moment at stem base:

M_{toe} = 23.56 ft·k / ft ≥ M_{stem} = 21.68 ft·k / ft

M_u = 21.68 ft·k / ft (stem base moment controls)

Flexure Check (ACI 318-14 13.3.2.1, 7.5.2.1, »22.3, »22.2, 7.5.1.1a)

$$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0.05 \text{ in}^2 / \text{in})(60000 \text{ psi})}{0.85 (3000 \text{ psi})} = 1.15 \text{ in}$$

$\phi M_n = \phi A_s f_y (d - a / 2)$ = (0.90) (0.05 in² / in) (60000 psi) [(10.63 in) - (1.15 in) / 2] = 26.53 ft·k / ft

ϕM_n = 26.53 ft·k / ft ≥ M_u = 21.68 ft·k / ft ✓

Shear Check (ACI 318-14 13.3.2.1, 7.5.3.1, »22.5.1, »22.5.5, 7.5.1.1b)

λ = 1.0 (normal weight concrete)

$V_c = 2 \lambda \sqrt{F'_c} d$ = 2 (1.0) $\sqrt{3000 \text{ psi}}$ (10.63 in) = 13.97 k / ft

$\phi V_n = \phi V_c$ = (0.750) (13.97 k / ft) = 10.48 k / ft

ϕV_n = 10.48 k / ft ≥ V_u = 4.96 k / ft ✓

Minimum Strain Check (ACI 318-14 13.3.2.1, 7.3.3.1)

β_1 = 0.850 ($F'_c \le 4000$ psi)

$$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0.05 \text{ in}^2 / \text{in})(60000 \text{ psi})}{0.85 (3000 \text{ psi})} = 1.15 \text{ in}$$

$$\varepsilon_t = 0.003 \left(\frac{d}{a / \beta_1} - 1 \right) = 0.003 \left[\frac{(10.63 \text{ in})}{(1.15 \text{ in}) / (0.850)} - 1 \right] = 0.0206$$

ε_t = 0.0206 ≥ 0.004 ✓

Minimum Steel Check (ACI 318-14 13.3.2.1, 9.6.1)

ϕM_n = 26.53 ft·k / ft < (4 / 3) M_u = [4 / 3] (21.68 ft·k / ft) = 28.91 ft·k / ft

$$A_{s_min} = \frac{3 \sqrt{F'_c}}{f_y} d = \frac{3 \sqrt{3000 \text{ psi}}}{(60000 \text{ psi})} (10.63 \text{ in}) = 0.03 \text{ in}^2 / \text{in}$$

200 d / f_y = 200 (10.63 in) / (60000 psi) = 0.04 in² / in

A_{s_min} = 0.04 in² / in

A_s = 0.05 in² / in ≥ A_{s_min} = 0.04 in² / in ✓

Shrinkage and Temperature Steel (ACI 318-14 13.2.8.1, 7.6.4.1, 24.4.3.2, 24.4.3.3)

$$\rho_{ST_prov} = \frac{A_{ST}}{t \, s_{ST}} = \frac{(0.4 \text{ in}^2 / \text{in})}{(14 \text{ in})(12 \text{ in})} = 0.0024$$

$$\rho_{ST_prov} = \frac{A_{ST}}{t \, s_{ST}} = \frac{(0.4 \text{ in}^2 / \text{in})}{(14 \text{ in})(12 \text{ in})} = 0.0024$$

$$\frac{0.0018 (60000)}{f_y} = \frac{0.0018 (60000)}{(60000 \text{ psi})} = 0.0018$$

ρ_{ST_min} = 0.0018

ρ_{ST_prov} = 0.0024 ≥ ρ_{ST_min} = 0.0018 ✓

18 inch limit governs

s_{ST_max} = 18 in

s_{ST} = 12 in ≤ s_{ST_max} = 18 in ✓

Development Check (ACI 318-14 13.2.8.1, 25.4.2.3, 25.4.10)

$$\frac{M_u}{\phi M_n} = \frac{(21.68 \text{ ft·k / ft})}{(26.53 \text{ ft·k / ft})} = 0.8172$$ (ratio to represent excess reinforcement)

ψ_t = 1.0 (12 inches or less cast below - 3.00 inches)

ψ_e = 1.0 (bar not epoxy coated)

ψ_s = 0.80 (bars are #6 or smaller)

λ = 1.0 (normal weight concrete)

s / 2 = (9 in) / 2 = 4.5 in

cover + d_b / 2 = (3 in) + (0.75 in) / 2 = 3.38 in

c_b = 3.38 in (lesser of half spacing, ctr to surface)

K_{tr} = 0.0 (no transverse reinforcement)

$$\frac{c_b + K_{tr}}{d_b} = \frac{(3.38 \text{ in}) + (0.0)}{(0.75 \text{ in})} = 4.50$$

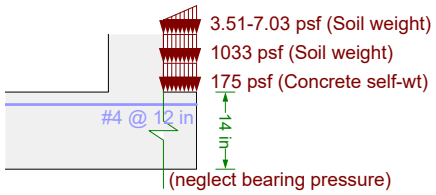
$$l_d = \left(\frac{3.}{40} \frac{f_y}{\lambda \sqrt{F'_c}} \frac{\psi_t \psi_e \psi_s}{2.5} \right) d_b = \left[\frac{3.}{40} \frac{(60000 \text{ psi})}{(1.0) \sqrt{3000 \text{ psi}}} \frac{(1.0)(1.0)(0.80)}{2.5} \right] (0.75 \text{ in}) = 19.72 \text{ in}$$

Factoring l_d by the excess reinforcement ratio (0.8172) per 25.4.10: l_d = 16.11 in

Heel Checks [1.2D + 0.5L + 1.6H + 1.0E]

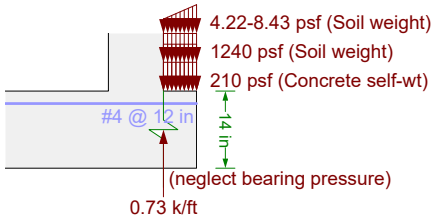
Heel Unfactored Loads

Unfactored Loads



Heel Factored Loads

1.2D + 0.5L + 1.6H + 1.0E



Controlling Moment

Design moment M_u for heel need not exceed moment at stem base:

M_{heel} = 0.18 ft·k / ft < M_{stem} = 21.68 ft·k / ft

M_u = 0.18 ft·k / ft (stem moment does not control)

Flexure Check (ACI 318-14 13.3.2.1, 7.5.2.1, »22.3, »22.2, 7.5.1.1a)

$$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0.02 \text{ in}^2 / \text{in})(60000 \text{ psi})}{0.85 (3000 \text{ psi})} = 0.39 \text{ in}$$

$\phi M_n = \phi A_s f_y (d - a / 2)$ = (0.90)(0.02 in² / in)(60000 psi)[(11.75 in) - (0.39 in) / 2] = 10.4 ft·k / ft

ϕM_n = 10.4 ft·k / ft ≥ M_u = 0.18 ft·k / ft ✓

Shear Check (ACI 318-14 13.3.2.1, 7.5.3.1, »22.5.1, »22.5.5, 7.5.1.1b)

λ = 1.0 (normal weight concrete)

$V_c = 2 \lambda \sqrt{F'_c}\, d = 2 (1.0) \sqrt{3000 \text{ psi}} (11.75 \text{ in})$ = 15.45 k / ft

$\phi V_n = \phi V_c$ = (0.750)(15.45 k / ft) = 11.58 k / ft

ϕV_n = 11.58 k / ft ≥ V_u = 0.73 k / ft ✓

Minimum Strain Check (ACI 318-14 13.3.2.1, 7.3.3.1)

β_1 = 0.850 ($F'_c \le 4000$ psi)

$$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0.02 \text{ in}^2 / \text{in})(60000 \text{ psi})}{0.85 (3000 \text{ psi})} = 0.39 \text{ in}$$

$$\varepsilon_t = 0.003 \left(\frac{d}{a / \beta_1} - 1 \right) = 0.003 \left[\frac{(11.75 \text{ in})}{(0.39 \text{ in}) / (0.850)} - 1 \right] = 0.0734$$

ε_t = 0.0734 ≥ 0.004 ✓

Minimum Steel Check (ACI 318-14 13.3.2.1, 9.6.1)

ϕM_n = 10.4 ft·k / ft ≥ (4 / 3) M_u = [4 / 3](0.18 ft·k / ft) = 0.24 ft·k / ft

Check is waived per ACI 9.6.1.3 ✓

Shrinkage and Temperature Steel (ACI 318-14 13.2.8.1, 7.6.4.1, 24.4.3.2, 24.4.3.3)

$$\rho_{ST_prov} = \frac{A_{ST}}{t\, s_{ST}} = \frac{(0.4 \text{ in}^2 / \text{in})}{(14 \text{ in})(12 \text{ in})} = 0.0024$$

$$\rho_{ST_prov} = \frac{A_{ST}}{t\, s_{ST}} = \frac{(0.4 \text{ in}^2 / \text{in})}{(14 \text{ in})(12 \text{ in})} = 0.0024$$

$$\frac{0.0018 (60000)}{f_y} = \frac{0.0018 (60000)}{(60000 \text{ psi})} = 0.0018$$

ρ_{ST_min} = 0.0018

ρ_{ST_prov} = 0.0024 ≥ ρ_{ST_min} = 0.0018 ✓

18 inch limit governs

s_{ST_max} = 18 in

s_{ST} = 12 in ≤ s_{ST_max} = 18 in ✓

Development Check (ACI 318-14 13.2.8.1, 25.4.2.3, 25.4.10)

$$\frac{M_u}{\phi M_n} = \frac{(0.18 \text{ ft·k / ft})}{(10.4 \text{ ft·k / ft})} = 0.0175$$ (ratio to represent excess reinforcement)

ψ_t = 1.0 (12 inches or less cast below - 11.50 inches)

ψ_e = 1.0 (bar not epoxy coated)

ψ_s = 0.80 (bars are #6 or smaller)

λ = 1.0 (normal weight concrete)

s / 2 = (12 in) / 2 = 6 in

cover + d_b / 2 = (2 in) + (0.5 in) / 2 = 2.25 in

c_b = 2.25 in (lesser of half spacing, ctr to surface)

K_{tr} = 0.0 (no transverse reinforcement)

$$\frac{c_b + K_{tr}}{d_b} = \frac{(2.25 \text{ in}) + (0.0)}{(0.5 \text{ in})} = 4.50$$

$$l_d = \left(\frac{3.}{40} \frac{f_y}{\lambda \sqrt{F'_c}} \frac{\psi_t \psi_e \psi_s}{2.5} \right) d_b = \left[\frac{3.}{40} \frac{(60000 \text{ psi})}{(1.0) \sqrt{3000 \text{ psi}}} \frac{(1.0)(1.0)(0.80)}{2.5} \right] (0.5 \text{ in}) = 13.15 \text{ in}$$

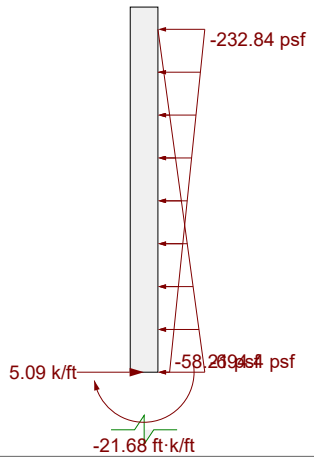
Factoring l_d by the excess reinforcement ratio (0.0175) per 25.4.10: l_d = 0.23 in

12 inch minimum controls

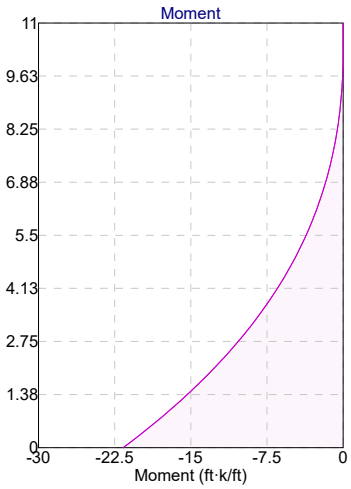
l_{d_prov} = 86 in ≥ l_d = 12 in ✓

Stem Forces [1.2D + 0.5L + 1.6H + 1.0E]

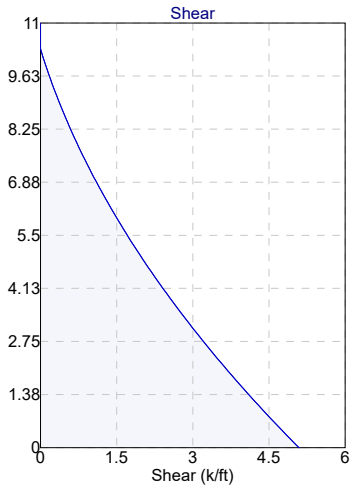
Stem Internal Forces



Stem Internal Forces



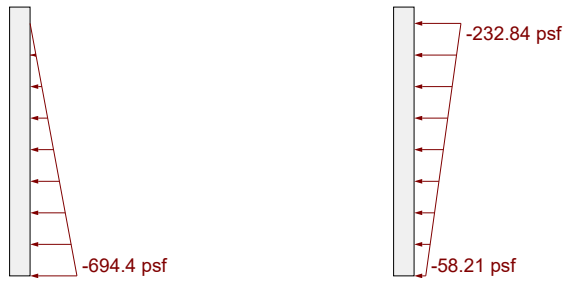
Stem Internal Forces



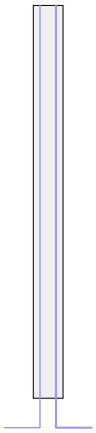
Stem Joint Force Transfer

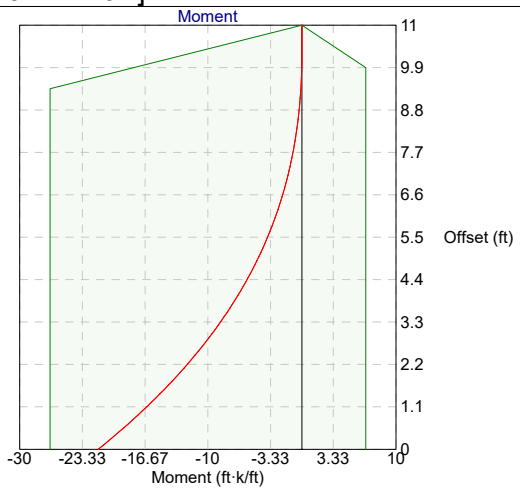
Location	Force
@ stem base	5.09 k/ft

Stem Internal Forces



Stem Moment Checks [1.2D + 0.5L + 1.6H + 1.0E]





Check (ACI 318-14 11.5.5.1b) @ 0 ft from base

ϕM_n = 26.78 ft·k / ft ≥ M_u = 21.68 ft·k / ft ✓

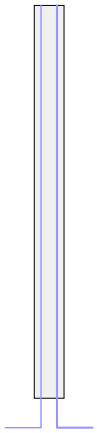
Check (ACI 318-14 11.5.5.1b) @ 9.36 ft from base

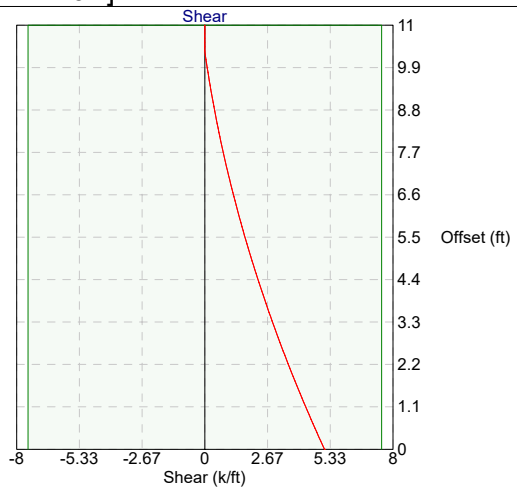
ϕM_n = 26.78 ft·k / ft ≥ M_u = 0.12 ft·k / ft ✓

Check (ACI 318-14 11.5.5.1b) @ 9.44 ft from base

ϕM_n = 25.35 ft·k / ft ≥ M_u = 0.1 ft·k / ft ✓

Stem Shear Checks [1.2D + 0.5L + 1.6H + 1.0E]





Shear Check (ACI 318-14 11.5.5.1c) @ 0 ft from base

ϕV_n = 7.52 k / ft ≥ V_u = 5.09 k / ft ✓

Stem Miscellaneous Checks [1.2D + 0.5L + 1.6H + 1.0E]

Minimum Steel Check (ACI 318-14 9.6.1) @ 0 ft from base [Stem in negative flexure]

$\phi M_n = 26.78$ ft·k / ft $<$ $(4 / 3) M_u = [4 / 3] (21.68$ ft·k / ft$) = 28.91$ ft·k / ft

$A_{s_min} = \frac{3 \sqrt{F'_c}}{f_y} d = \frac{3 \sqrt{3000 \text{ psi}}}{(60000 \text{ psi})} (7.63 \text{ in}) = 0.02$ in² / in

$200\ d / f_y = 200\ (7.63 \text{ in}) / (60000 \text{ psi}) = 0.03$ in² / in

$A_{s_min} = 0.03$ in² / in

$A_s = 0.07$ in² / in $\geq$ $A_{s_min} = 0.03$ in² / in ✓

Minimum Steel Check (ACI 318-14 9.6.1) @ 11 ft from base [Stem in negative flexure]

$\phi M_n = 0$ ft·k / ft $\geq$ $(4 / 3) M_u = [4 / 3] (0$ ft·k / ft$) = 0$ ft·k / ft

Check is waived per ACI 9.6.1.3 ✓

Maximum Steel Check (ACI 318-14 9.3.3.1) @ 0 ft from base [Stem in negative flexure]

$\beta_1 = 0.850$ $(F'_c \leq 4000 \text{ psi})$

$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0.07 \text{ in}^2 / \text{in}) (60000 \text{ psi})}{0.85 (3000 \text{ psi})} = 1.73$ in

$\varepsilon_t = 0.003 \left(\frac{d}{a / \beta_1} - 1 \right) = 0.003 \left[\frac{(7.63 \text{ in})}{(1.73 \text{ in}) / (0.850)} - 1 \right] = 0.0083$

$\varepsilon_t = 0.0083 \geq 0.004$ ✓

Maximum Steel Check (ACI 318-14 9.3.3.1) @ 11 ft from base [Stem in negative flexure]

$\beta_1 = 0.850$ $(F'_c \leq 4000 \text{ psi})$

$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0.07 \text{ in}^2 / \text{in}) (60000 \text{ psi})}{0.85 (3000 \text{ psi})} = 1.73$ in

$\varepsilon_t = 0.003 \left(\frac{d}{a / \beta_1} - 1 \right) = 0.003 \left[\frac{(7.63 \text{ in})}{(1.73 \text{ in}) / (0.850)} - 1 \right] = 0.0083$

$\varepsilon_t = 0.0083 \geq 0.004$ ✓

Wall Horizontal Steel (ACI 318-14 11.6.1, 11.7.3)

$\rho_t = \frac{A_{s_horz} / s_{horz}}{t} = \frac{(0.4 \text{ in}^2) / (12 \text{ in})}{(10 \text{ in})} = 0.0033$

$\rho_{t_min} = 0.0020$ (bars No. 5 or less, not less than 60 ksi)

$\rho_t = 0.0033 \geq \rho_{t_min} = 0.0020$ ✓

$3\ h = 3\ (10 \text{ in}) = 30$ in

18 inch limit governs

$s_{horz} = 12 \text{ in} \leq s_{horz_max} = 18 \text{ in}$ ✓

Development Check (ACI 318-14 11.7.1.2, 25.4.2.3, 25.4.10)

$\frac{M_u}{\phi M_n} = \frac{(21.68 \text{ ft·k / ft})}{(26.78 \text{ ft·k / ft})} = 0.8096$ (ratio to represent excess reinforcement)

$\psi_e = 1.0$ (uncoated hooked bars)

$\psi_c = 0.70$ (based on side cover and extension cover)

$\psi_r = 1.0$ (no confining reinforcement)

$\lambda = 1.0$ (normal weight concrete)

$l_{dh} = \left(\frac{f_y \psi_e \psi_c \psi_r}{50 \lambda \sqrt{F'_c}} \right) d_b = \left[\frac{(60000 \text{ psi}) (1.0) (0.70) (1.0)}{50 (1.0) \sqrt{3000 \text{ psi}}} \right] (0.75 \text{ in}) = 11.5$ in

Factoring l_{dh} by the excess reinforcement ratio (0.8096) per 25.4.10: $l_{dh} = 9.31$ in

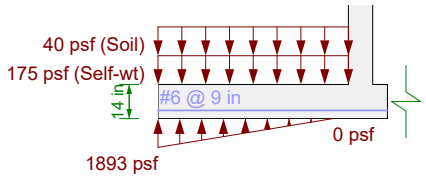
$8\ d_b = 8\ (0.75 \text{ in}) = 6.0$ (minimum limit, does not control)

$l_{dh_prov} = 11 \text{ in} \geq l_{dh} = 9.31 \text{ in}$ ✓

Toe Checks [1.4D]

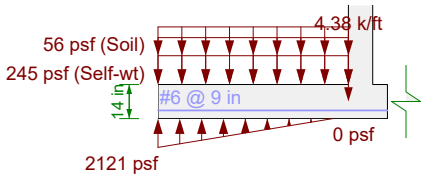
Toe Unfactored Loads

Unfactored Loads



Toe Factored Loads

1.4D



Controlling Moment

Design moment M_u for toe need not exceed moment at stem base:

M_{toe} = 22.2 ft·k / ft ≥ M_{stem} = - 0 ft·k / ft

M_u = - 0 ft·k / ft (stem base moment controls)

Flexure Check (ACI 318-14 13.3.2.1, 7.5.2.1, »22.3, »22.2, 7.5.1.1a)

$$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0.05 \text{ in}^2 / \text{in})(60000 \text{ psi})}{0.85 (3000 \text{ psi})} = 1.15 \text{ in}$$

$\phi M_n = \phi A_s f_y (d - a / 2)$ = (0.90) (0.05 in² / in) (60000 psi) [(10.63 in) - (1.15 in) / 2] = 26.53 ft·k / ft

ϕM_n = 26.53 ft·k / ft ≥ M_u = - 0 ft·k / ft ✓

Shear Check (ACI 318-14 13.3.2.1, 7.5.3.1, »22.5.1, »22.5.5, 7.5.1.1b)

λ = 1.0 (normal weight concrete)

$V_c = 2 \lambda \sqrt{F'_c} d$ = 2 (1.0) $\sqrt{3000 \text{ psi}}$ (10.63 in) = 13.97 k / ft

$\phi V_n = \phi V_c$ = (0.750) (13.97 k / ft) = 10.48 k / ft

ϕV_n = 10.48 k / ft ≥ V_u = 4.62 k / ft ✓

Minimum Strain Check (ACI 318-14 13.3.2.1, 7.3.3.1)

β_1 = 0.850 ($F'_c \le 4000$ psi)

$$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0.05 \text{ in}^2 / \text{in})(60000 \text{ psi})}{0.85 (3000 \text{ psi})} = 1.15 \text{ in}$$

$$\varepsilon_t = 0.003 \left(\frac{d}{a / \beta_1} - 1 \right) = 0.003 \left[\frac{(10.63 \text{ in})}{(1.15 \text{ in}) / (0.850)} - 1 \right] = 0.0206$$

ε_t = 0.0206 ≥ 0.004 ✓

Minimum Steel Check (ACI 318-14 13.3.2.1, 9.6.1)

ϕM_n = 26.53 ft·k / ft ≥ (4 / 3) M_u = [4 / 3] (- 0 ft·k / ft) = - 0 ft·k / ft

Check is waived per ACI 9.6.1.3 ✓

Shrinkage and Temperature Steel (ACI 318-14 13.2.8.1, 7.6.4.1, 24.4.3.2, 24.4.3.3)

$$\rho_{ST_prov} = \frac{A_{ST}}{t\, s_{ST}} = \frac{(0.4 \text{ in}^2 / \text{in})}{(14 \text{ in})(12 \text{ in})} = 0.0024$$

$$\rho_{ST_prov} = \frac{A_{ST}}{t\, s_{ST}} = \frac{(0.4 \text{ in}^2 / \text{in})}{(14 \text{ in})(12 \text{ in})} = 0.0024$$

$$\frac{0.0018 (60000)}{f_y} = \frac{0.0018 (60000)}{(60000 \text{ psi})} = 0.0018$$

ρ_{ST_min} = 0.0018

ρ_{ST_prov} = 0.0024 ≥ ρ_{ST_min} = 0.0018 ✓

18 inch limit governs

s_{ST_max} = 18 in

s_{ST} = 12 in ≤ s_{ST_max} = 18 in ✓

Development Check (ACI 318-14 13.2.8.1, 25.4.2.3, 25.4.10)

$$\frac{M_u}{\phi M_n} = \frac{(- 0 \text{ ft·k / ft})}{(26.53 \text{ ft·k / ft})} = - 0.0$$ (ratio to represent excess reinforcement)

ψ_t = 1.0 (12 inches or less cast below - 3.00 inches)

ψ_e = 1.0 (bar not epoxy coated)

ψ_s = 0.80 (bars are #6 or smaller)

λ = 1.0 (normal weight concrete)

s / 2 = (9 in) / 2 = 4.5 in

cover + d_b / 2 = (3 in) + (0.75 in) / 2 = 3.38 in

c_b = 3.38 in (lesser of half spacing, ctr to surface)

K_{tr} = 0.0 (no transverse reinforcement)

$$\frac{c_b + K_{tr}}{d_b} = \frac{(3.38 \text{ in}) + (0.0)}{(0.75 \text{ in})} = 4.50$$

$$l_d = \left(\frac{3.}{40} \frac{f_y}{\lambda \sqrt{F'_c}} \frac{\psi_t \psi_e \psi_s}{2.5} \right) d_b = \left[\frac{3.}{40} \frac{(60000 \text{ psi})}{(1.0) \sqrt{3000 \text{ psi}}} \frac{(1.0)(1.0)(0.80)}{2.5} \right] (0.75 \text{ in}) = 19.72 \text{ in}$$

Factoring l_d by the excess reinforcement ratio (- 0.0000) per 25.4.10: l_d = - 0 in

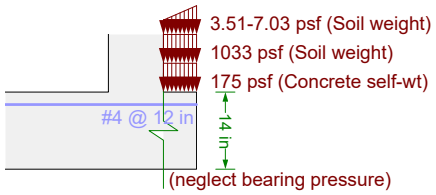
12 inch minimum controls

l_{d_prov} = 13 in ≥ l_d = 12 in ✓

Heel Checks [1.4D]

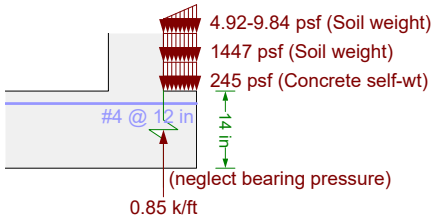
Heel Unfactored Loads

Unfactored Loads



Heel Factored Loads

1.4D



Controlling Moment

Design moment M_u for heel need not exceed moment at stem base:

M_{heel} = 0.21 ft·k / ft ≥ M_{stem} = - 0 ft·k / ft

M_u = - 0 ft·k / ft (stem base moment controls)

Flexure Check (ACI 318-14 13.3.2.1, 7.5.2.1, »22.3, »22.2, 7.5.1.1a)

$$a = \frac{A_s f_y}{0.85 f'_c} = \frac{(0.02 \text{ in}^2 / \text{in})(60000 \text{ psi})}{0.85 (3000 \text{ psi})} = 0.39 \text{ in}$$

$$\phi M_n = \phi A_s f_y (d - a / 2) = (0.90)(0.02 \text{ in}^2 / \text{in})(60000 \text{ psi})[(11.75 \text{ in}) - (0.39 \text{ in}) / 2] = 10.4 \text{ ft·k / ft}$$

ϕM_n = 10.4 ft·k / ft ≥ M_u = - 0 ft·k / ft ✓

Shear Check (ACI 318-14 13.3.2.1, 7.5.3.1, »22.5.1, »22.5.5, 7.5.1.1b)

λ = 1.0 (normal weight concrete)

$$V_c = 2 \lambda \sqrt{f'_c}\, d = 2 (1.0) \sqrt{3000 \text{ psi}}\, (11.75 \text{ in}) = 15.45 \text{ k / ft}$$

$$\phi V_n = \phi V_c = (0.750)(15.45 \text{ k / ft}) = 11.58 \text{ k / ft}$$

ϕV_n = 11.58 k / ft ≥ V_u = 0.85 k / ft ✓

Minimum Strain Check (ACI 318-14 13.3.2.1, 7.3.3.1)

β_1 = 0.850 ($f'_c \le 4000$ psi)

$$a = \frac{A_s f_y}{0.85 f'_c} = \frac{(0.02 \text{ in}^2 / \text{in})(60000 \text{ psi})}{0.85 (3000 \text{ psi})} = 0.39 \text{ in}$$

$$\varepsilon_t = 0.003 \left(\frac{d}{a / \beta_1} - 1 \right) = 0.003 \left[\frac{(11.75 \text{ in})}{(0.39 \text{ in}) / (0.850)} - 1 \right] = 0.0734$$

56 psf

ε_t = 0.0734 ≥ 0.004 ✓

Minimum Steel Check (ACI 318-14 13.3.2.1, 9.6.1)

ϕM_n = 10.4 ft·k / ft ≥ (4 / 3) M_u = [4 / 3] (- 0 ft·k / ft) = - 0 ft·k / ft

Check is waived per ACI 9.6.1.3 ✓

Shrinkage and Temperature Steel (ACI 318-14 13.2.8.1, 7.6.4.1, 24.4.3.2, 24.4.3.3)

$$\rho_{ST_prov} = \frac{A_{ST}}{t\, s_{ST}} = \frac{(0.4 \text{ in}^2 / \text{in})}{(14 \text{ in})(12 \text{ in})} = 0.0024$$

$$\rho_{ST_prov} = \frac{A_{ST}}{t\, s_{ST}} = \frac{(0.4 \text{ in}^2 / \text{in})}{(14 \text{ in})(12 \text{ in})} = 0.0024$$

$$\frac{0.0018 (60000)}{f_y} = \frac{0.0018 (60000)}{(60000 \text{ psi})} = 0.0018$$

ρ_{ST_min} = 0.0018

ρ_{ST_prov} = 0.0024 ≥ ρ_{ST_min} = 0.0018 ✓

18 inch limit governs

s_{ST_max} = 18 in

s_{ST} = 12 in ≤ s_{ST_max} = 18 in ✓

Development Check (ACI 318-14 13.2.8.1, 25.4.2.3, 25.4.10)

$$\frac{M_u}{\phi M_n} = \frac{(- 0 \text{ ft·k / ft})}{(10.4 \text{ ft·k / ft})} = - 0.0$$ (ratio to represent excess reinforcement)

ψ_t = 1.0 (12 inches or less cast below - 11.50 inches)

ψ_e = 1.0 (bar not epoxy coated)

ψ_s = 0.80 (bars are #6 or smaller)

λ = 1.0 (normal weight concrete)

s / 2 = (12 in) / 2 = 6 in

cover + d_b / 2 = (2 in) + (0.5 in) / 2 = 2.25 in

c_b = 2.25 in (lesser of half spacing, ctr to surface)

K_{tr} = 0.0 (no transverse reinforcement)

$$\frac{c_b + K_{tr}}{d_b} = \frac{(2.25 \text{ in}) + (0.0)}{(0.5 \text{ in})} = 4.50$$

$$l_d = \left(\frac{3.}{40} \frac{f_y}{\lambda \sqrt{f'_c}} \frac{\psi_t \psi_e \psi_s}{2.5} \right) d_b = \left[\frac{3.}{40} \frac{(60000 \text{ psi})}{(1.0) \sqrt{3000 \text{ psi}}} \frac{(1.0)(1.0)(0.80)}{2.5} \right] (0.5 \text{ in}) = 13.15 \text{ in}$$

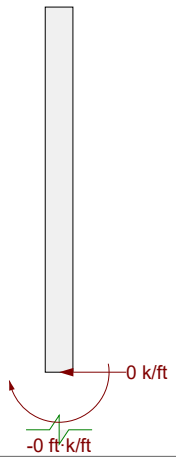
Factoring l_d by the excess reinforcement ratio (- 0.0000) per 25.4.10: l_d = - 0 in

12 inch minimum controls

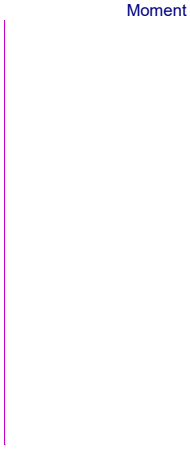
l_{d_prov} = 86 in ≥ l_d = 12 in ✓

Stem Forces [1.4D]

Stem Internal Forces



Stem Internal Forces



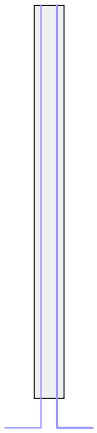
Stem Internal Forces

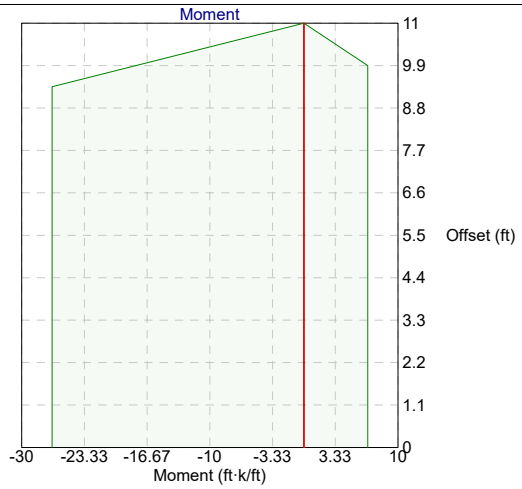
Stem Joint Force Transfer

Location	Force
@ stem base	0 k/ft

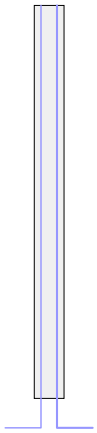
Stem Internal Forces

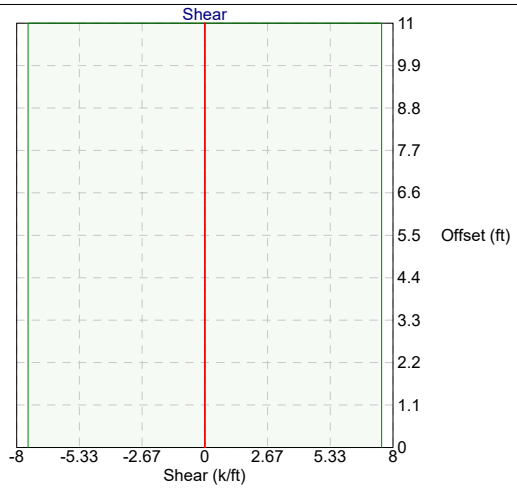
Stem Moment Checks [1.4D]





Stem Shear Checks [1.4D]





Stem Miscellaneous Checks [1.4D]

Minimum Steel Check (ACI 318-14 9.6.1) @ 0 ft from base [Stem in negative flexure]

$\phi M_n = 26.78$ ft·k / ft $\geq (4 / 3) M_u = [4 / 3] (0$ ft·k / ft$) = 0$ ft·k / ft

Check is waived per ACI 9.6.1.3 ✓

Minimum Steel Check (ACI 318-14 9.6.1) @ 11 ft from base [Stem in negative flexure]

$\phi M_n = 0$ ft·k / ft $\geq (4 / 3) M_u = [4 / 3] (0$ ft·k / ft$) = 0$ ft·k / ft

Check is waived per ACI 9.6.1.3 ✓

Maximum Steel Check (ACI 318-14 9.3.3.1) @ 0 ft from base [Stem in negative flexure]

$\beta_1 = 0.850$ ($F'_c \leq 4000$ psi)

$$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0.07 \text{ in}^2 / \text{in}) (60000 \text{ psi})}{0.85 (3000 \text{ psi})} = 1.73 \text{ in}$$

$$\varepsilon_t = 0.003 \left(\frac{d}{a / \beta_1} - 1 \right) = 0.003 \left[\frac{(7.63 \text{ in})}{(1.73 \text{ in}) / (0.850)} - 1 \right] = 0.0083$$

$\varepsilon_t = 0.0083 \geq 0.004$ ✓

Maximum Steel Check (ACI 318-14 9.3.3.1) @ 11 ft from base [Stem in negative flexure]

$\beta_1 = 0.850$ ($F'_c \leq 4000$ psi)

$$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0.07 \text{ in}^2 / \text{in}) (60000 \text{ psi})}{0.85 (3000 \text{ psi})} = 1.73 \text{ in}$$

$$\varepsilon_t = 0.003 \left(\frac{d}{a / \beta_1} - 1 \right) = 0.003 \left[\frac{(7.63 \text{ in})}{(1.73 \text{ in}) / (0.850)} - 1 \right] = 0.0083$$

$\varepsilon_t = 0.0083 \geq 0.004$ ✓

Wall Horizontal Steel (ACI 318-14 11.6.1, 11.7.3)

$$\rho_t = \frac{A_{s_horz} / s_{horz}}{t} = \frac{(0.4 \text{ in}^2) / (12 \text{ in})}{(10 \text{ in})} = 0.0033$$

$\rho_{t_min} = 0.0020$ (bars No. 5 or less, not less than 60 ksi)

$\rho_t = 0.0033 \geq \rho_{t_min} = 0.0020$ ✓

$3 h = 3 (10 \text{ in}) = 30$ in

18 inch limit governs

$s_{horz} = 12$ in $\leq s_{horz_max} = 18$ in ✓

Development Check (ACI 318-14 11.7.1.2, 25.4.2.3, 25.4.10)

$\frac{M_u}{\phi M_n} = \frac{(0 \text{ ft·k / ft})}{(26.78 \text{ ft·k / ft})} = 0.0$ (ratio to represent excess reinforcement)

$\psi_e = 1.0$ (uncoated hooked bars)

$\psi_c = 0.70$ (based on side cover and extension cover)

$\psi_r = 1.0$ (no confining reinforcement)

$\lambda = 1.0$ (normal weight concrete)

$$l_{dh} = \left(\frac{f_y \psi_e \psi_c \psi_r}{50 \lambda \sqrt{F'_c}} \right) d_b = \left[\frac{(60000 \text{ psi}) (1.0) (0.70) (1.0)}{50 (1.0) \sqrt{3000 \text{ psi}}} \right] (0.75 \text{ in}) = 11.5 \text{ in}$$

Factoring l_{dh} by the excess reinforcement ratio (0.0000) per 25.4.10: $l_{dh} = 0$ in

$8 d_b = 8 (0.75 \text{ in}) = 6.0$ (minimum limit, does not control)

6 inch minimum controls

$l_{dh_prov} = 11$ in $\geq l_{dh} = 6$ in ✓



FOUNDATION PLAN

LANZ RESIDENCE
8020 SE 57th Street
Mercer Island, WA 98040

Foundation Plan

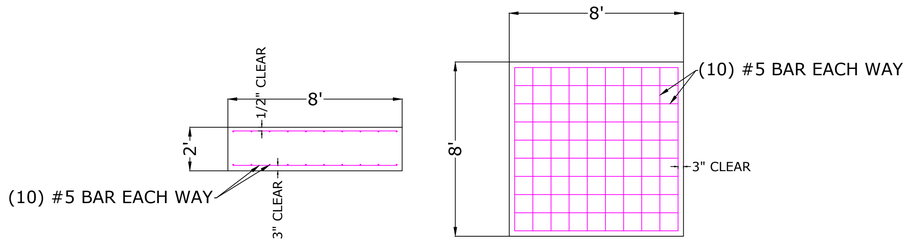
LUCIA ENGINEERING, INC.
12527 Huckleberry Lane
Arlington, Washington 98223
PHONE: (206) 790-8039
E-MAIL: joe@luciaeng.com

JOSEPH M. LUCIA
STATE OF WASHINGTON
23314
REGISTERED
PROFESSIONAL ENGINEER

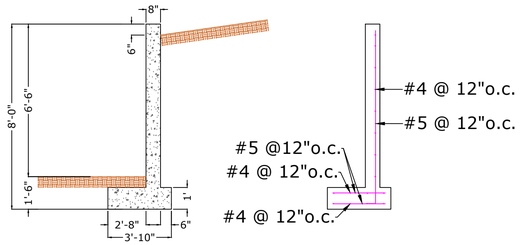
08-15-24

Number	Date	By	Description
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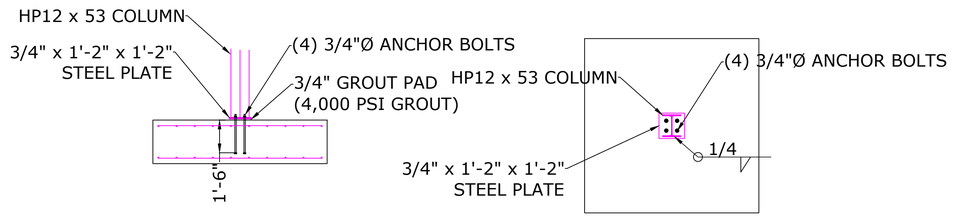
SHEET
S-13.0



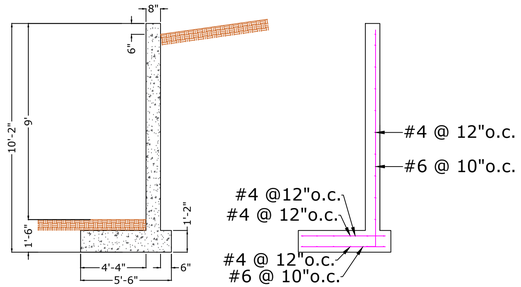
COLUMN FOOTING DETAIL



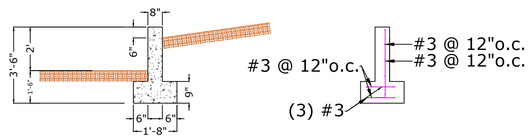
8' WALL HEIGHT DETAIL - SECTION C-C



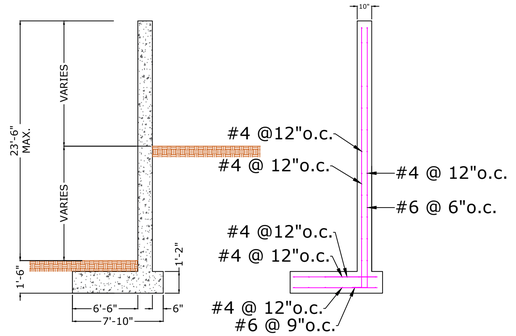
COLUMN TO FOOTING DETAIL



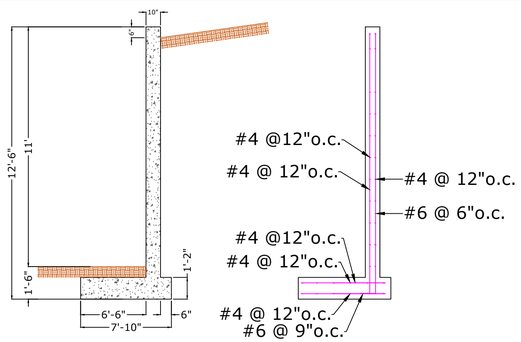
10.17' WALL HEIGHT DETAIL - SECTION D-D



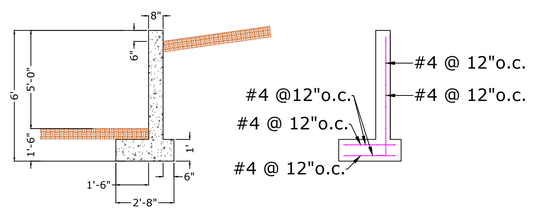
STANDARD FOOTING DETAIL - SECTION A-A



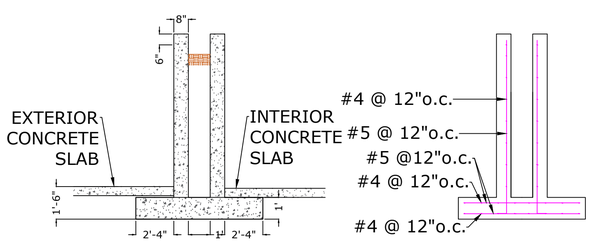
12.5' WALL HEIGHT DETAIL - SECTION G-G



12.5' WALL HEIGHT DETAIL - SECTION E-E



6' WALL HEIGHT DETAIL - SECTION B-B



CANTILEVERED WALLS WALL HEIGHT DETAIL - SECTION F-F

LANZ RESIDENCE
8020 SE 57th Street
Mercer Island, WA 98040

Foundation Wall Sections
Retaining Wall Sections

LUCIA ENGINEERING, INC.
12527 Huckleberry Lane
Arlington, Washington 98223
PHONE: (206) 790-8039
E-MAIL: joe@luciaeng.com

JOSEPH M. LUCIA
STATE OF WASHINGTON
23314
REGISTERED
PROFESSIONAL ENGINEER

08-15-24

Number	Date	By	Description
5	08-15-24	JML	

SHEET
S-13.1

FRAMING MEMBER DESIGN

First Level Framing
- Steel Member Design
- Timber Header-Beam Design
- Floor Joist Design
- Simpson Holdowns & Straps

Second Level Framing
- Steel Member Design
- Timber Header-Beam Design
- Floor Joist Design
- Simpson Holdowns & Straps

Third Level Framing
- Steel Member Design
- Timber Header-Beam Design
- Floor Joist Design
- Simpson Holdowns & Straps

Framing Plans

LUCIA ENGINEERING, INC.
12527 Huckleberry Lane
Arlington, Washington 98223
Phone: 206.790-8039

LOADS TO STEEL BEAMS

2ND LEVEL

ROOF D.L. = 12.3 PSF × 1.2 = 14.76 PLF
ROOF L.L. = 25 PSF × 1.6 = 40 PLF > 55 PLF
WALL D.L. = 10 PSF × 8.0' = 80 PLF × 1.2 = 96 PLF

MAX ROOF RAFTER SPAN = 20'

⇒ LOAD TO STEEL BEAM = (55 PLF × 20'/2) + 96 PLF
≅ 650 PLF

1ST LEVEL

FLOOR D.L. = 12.3 PSF × 1.2 = 14.76 PLF
FLOOR L.L. = 40 PSF × 1.6 = 64 PLF > 79 PLF
WALL D.L. = 10 PSF × 8.0' = 80 PLF × 1.2 = 96 PLF

MAX FLOOR JOIST SPAN = 31'

⇒ LOAD TO STEEL BEAM = (79 PLF × 31'/2) + 96 PLF
≅ 1,325 PLF

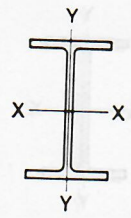
MAX SPAN BETWEEN STEEL BEAMS = 27'

⇒ LOAD TO STEEL BEAM = 79 PLF × 27'/2 ≅ 1,100 PLF

F_y = 36 ksi

F_y = 50 ksi

COLUMNS
W shapes
Allowable axial loads in kips



Designation		W12									
Wt./ft		58		53		50		45		40	
F_y		36	50	36	50	36	50	36	50	36	50‡
Effective length in ft KL with respect to least radius of gyration r_y	0	367	510	337	468	318	441	285	396	255	354
	6	341	464	312	425	286	386	256	346	229	309
	7	335	454	307	416	279	374	250	335	223	299
	8	329	443	301	406	271	360	243	322	217	288
	9	322	432	295	395	263	346	235	309	210	276
	10	315	420	288	384	254	331	228	296	203	264
	11	308	407	282	372	246	315	220	281	196	251
	12	301	394	275	360	236	298	211	266	188	237
	13	293	380	268	347	226	281	202	250	180	222
	14	285	365	260	333	216	262	193	233	172	207
	15	276	351	252	319	206	243	183	216	163	191
	16	268	335	244	305	195	223	173	197	154	175
	18	249	302	227	274	171	181	152	159	135	141
	20	230	267	209	241	146	146	129	129	114	114
	22	209	229	189	206	121	121	106	106	94	94
	24	187	193	169	173	102	102	89	89	79	79
	26	164	164	147	147	87	87	76	76	67	67
	28	142	142	127	127	75	75	66	66	58	58
	30	123	123	111	111	65	65	57	57	51	51
	32	108	108	97	97	57	57	50	50	45	45
	34	96	96	86	86						
	38	77	77	69	69						
	41	66	66	59	59						
Properties											
U		3.21	3.21	3.24	2.94	4.10	4.10	4.12	3.75	3.77	3.77
P_{wo} (kips)		89	124	78	108	92	127	75	105	66	92
P_{wi} (kips/in.)		13	18	12	17	13	19	12	17	11	15
P_{wb} (kips)		121	142	106	125	131	155	97	115	66	78
P_{fb} (kips)		92	128	74	103	92	128	74	103	60	83
L_c (ft)		10.6	9.0	10.6	9.0	8.5	7.2	8.5	7.2	8.4	7.2
L_u (ft)		24.4	17.5	22.0	15.9	19.6	14.1	17.7	12.8	16.0	11.5
A (in.^{2})		17.0		15.6		14.7		13.2		11.8	
I_x (in.^{4})		475		425		394		350		310	
I_y (in.^{4})		107		95.8		56.3		50.0		44.1	
r_y (in.)		2.51		2.48		1.96		1.94		1.93	
Ratio r_x/r_y		2.10		2.11		2.64		2.65		2.66	
B_x } Bending		0.218		0.221		0.227		0.227		0.227	
B_y } factors		0.794		0.813		1.058		1.065		1.073	
$a_x/10^6$		70.6		63.6		58.8		52.2		46.3	
$a_y/10^6$		16.0		14.3		8.4		7.4		6.5	
F'_{ex} $(K_x L_x)^2/10^2$ (kips)		289		284		278		275		273	
F'_{ey} $(K_y L_y)^2/10^2$ (kips)		65.3		63.8		39.8		39.0		38.6	

‡Web may be noncompact for combined axial and bending stress; see AISC ASD Specification Sect. B5.1.

Note: Heavy line indicates Kl/r of 200.



Designation		
Wt./ft		1
F_y		36
Effective length in ft KL with respect to least radius of gyration r_y	0	711
	6	663
	7	653
	8	642
	9	631
	10	619
	11	606
	12	593
	13	579
	14	565
	15	550
	16	535
	17	519
	18	503
	19	486
	20	469
	22	433
	24	395
	26	355
	28	313
	30	272
	32	239
	34	212
	36	189
	38	170
	40	153
U		2.45
P_{wo} (kips)		255
P_{wi} (kips/in.)		27
P_{wb} (kips)		1388
P_{fb} (kips)		352
L_c (ft)		11.0
L_u (ft)		53.2
A (in.^{2})		32
I_x (in.^{4})		7
I_y (in.^{4})		2
r_y (in.)		2.
Ratio r_x/r_y		1.
B_x } Bending		0.2
B_y } factors		0.7
$a_x/10^6$		106
$a_y/10^6$		35
F'_{ex} $(K_x L_x)^2/10^2$ (kips)		2
F'_{ey} $(K_y L_y)^2/10^2$ (kips)		74

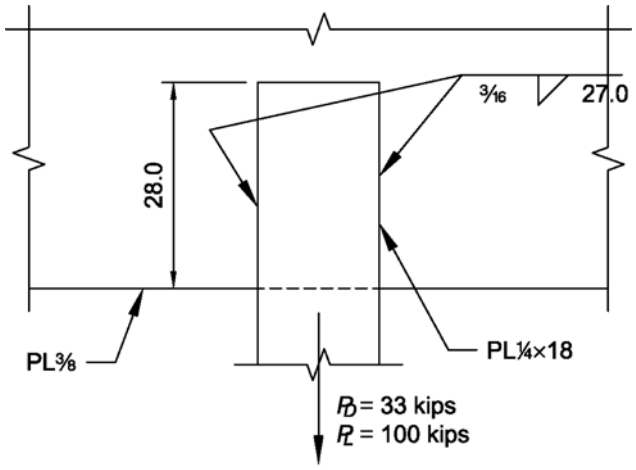
A

Example J.1 Fillet Weld in Longitudinal Shear

Given:

An ¼ in.×18-in. wide plate is fillet welded to a ⅜-in. plate. Assume that the plates are ASTM A572 grade 50 and have been properly sized. Assume F_{EXX} = 70 ksi. Note that plates would normally be specified as ASTM A36, but F_y = 50 ksi plate has been used here to demonstrate requirements for long welds.

Size the welds for the loads shown.



Solution:

Determine the maximum weld size

Because the overlapping plate is ¼ in., the maximum fillet weld size that can be used without special notation (built out to obtain full-throat thickness as required in AISC Specification Section J2.2b) is a ⅜-in. fillet weld. A ⅜-in. fillet weld can be deposited in the flat or horizontal position in a single pass (true up to ⅝-in).

Determine the required strength

LRFD	ASD
P_u = 1.2(33 kips) + 1.6(100 kips) = 200 kips	P_a = 33 kips + 100 kips = 133 kips

Determine the length of weld required

LRFD	ASD
The design strength per inch of a ⅜-in. fillet weld is	The allowable strength per inch of a ⅜-in. fillet weld is
ϕR_n = 1.392 (3) = 4.17 kips/ in.	R_n/Ω= 0.928 (3) = 2.78 kips/ in.
$\frac{P_u}{\phi R_n} = \frac{200 \text{ kips}}{4.17 \text{ kips/in.}} = 48$ in.	$\frac{P_a \Omega}{R_n} = \frac{133 \text{ kips}}{2.78 \text{ kips/in.}} = 48$ in.
or 24 in. of weld on each side	or 24 in. of weld on each side.

Manual Part 8

LUCIA ENGINEERING, INC.
12527 Huckleberry Lane
Arlington, Washington 98223
Phone: 206.790-8039

CHECK WELDS:

$P_{MAX} = 20,538^{LBS}$ (SEE STEEL BEAM DESIGN)

P_{DESIGN} $20,538 \times 1.6 = 33^{KIPS}$

ASSUME $3/8"$ FILLET WELD

$L_w = (3)(1.392) = 4.17^{K/IN}$

$\Rightarrow 33^{K} / 4.17^{K/IN} \doteq 8"$ OF WELD REQ'D.

CHECK BOLTS

$7/8"\emptyset$ A325 BOLTS

$V_{ALLOW} = 21.6^{KIPS}$

$33^{K} / 21.6^{K/BOLT} \doteq 2$ BOLT REQ'D.

FLOOR JOIST DESIGN: 9' SPAN

Assumptions:

1. 1.5" x 9.25" LVL Joist
2. Maximum span: **9.00** feet **(MAX)** 108 inches
   Therefore: **L** = 108 inches
3. Joist Spacing: **1.50** feet o.c. = 18 inches
4. Minimum Bearing Area: (1.5" x 2.5") Simpson Hanger = **3.75** $inches^2$

1.5" x 9.25" Joist Properties:

ANSI Design Valu Hem-Fir (North)

A	=	**13.875**	$inches^2$
E	=	**2,000,000**	PSI
I	=	**98.9**	$inches^4$
S	=	**21.4**	$inches^3$
$Fv_{Allowable}$	=	**2,755**	LBS
$Fp_{Allowable}$	=	**2755**	PSI
M_{max}	=	**5791**	FT-LBS
Fb	=	**3100**	PSI

Reactions:

W = Floor Load x Span Supported 493.50 PLF

Framing Deck Dea	=		15	PSF
Green Roof Soil D	=	115^{PCF} x 2^{FT}	230	PSF
Live Load	=		25	PSF
			270	PSF

Design Load = DL x 1.2 + LL x 1.4 = 329 PSF

V_{max}	=	0.5**WL**	=	**2,221**	lbs
P_{max}	=	0.5**WL**	=	**2,221**	lbs
M_{max}	=	$(WL^2)/8$	=	**4,997**	ft-lbs
D_{max}	=	$0.013WL^4 / EI$	=	**0.17**	inches

Check Shear:

Fv	=	=	**2,220.75**	PSI
$Fv_{Allowable}$	=		**2,755**	PSI
			Check - O.K.	

Check Bearing:

Fp	=	P_{max} / **A** =	**2220.75**	PSI
$Fp_{Allowable}$	=		**2755**	PSI
			Check - O.K.	

Check Bending:

$M_{Req'd}$	=	=	**4996.69**	ft-lbs
$M_{Furnished}$	=		**5791.0**	ft-lbs
			Check - O.K.	

Check Deflection:

$D_{max.}$	=		**0.17**	inches
$D_{Allowable}$	=	**L**/350 =	**0.31**	inches
			Check - O.K.	

FLOOR JOIST DESIGN: 12.33 SPAN

Assumptions:

1. .75" x119.25" LVL Joist
2. Maximum span: **12.33** feet **(MAX)** 147.96 inches
   Therefore: **L** = 147.96 inches
3. Joist Spacing: **1.50** feet o.c. = 18 inches
4. Minimum Bearing Area: (1.5" x 2.5") Simpson Hanger = **3.75** $inches^2$

1.5" x 11.25" Joist Properties:

ANSI Design Valu Hem-Fir (North)

A	=	**19.6875**	$inches^2$
E	=	**2,000,000**	PSI
I	=	**207.6**	$inches^4$
S	=	**36.9**	$inches^3$
$Fv_{Allowable}$	=	**3,806**	LBS
$Fp_{Allowable}$	=	**3806**	PSI
M_{max}	=	**9648**	FT-LBS
Fb	=	**3100**	PSI

Reactions:

W = Floor Load x Span Supported 493.50 PLF

Framing Deck Dea	=	15	PSF
Green Roof Soil D	= 115^{PCF} x 2^{F}	230	PSF
Live Load	=	25	PSF
		270	PSF

Design Load = DL x 1.2 + LL x 1.4 = 329 PSF

V_{max}	=	0.5**WL**	=	**3,042**	lbs
P_{max}	=	0.5**WL**	=	**3,042**	lbs
M_{max}	=	$(WL^2)/8$	=	**9,378**	ft-lbs
D_{max}	=	$0.013WL^4 / EI$	=	**0.29**	inches

Check Shear:

F_V	=	=	**3,042.43**	PSI
$Fv_{Allowable}$	=		**3,806**	PSI
			Check - O.K.	

Check Bearing:

F_P	=	P_{max} / **A**	=	**3042.43** PSI
$Fp_{Allowable}$	=		**3806**	PSI
			Check - O.K.	

Check Bending:

$M_{Req'd}$	=	=	**9378.28**	ft-lbs
$M_{Furnished}$	=		**9648.0**	ft-lbs
			Check - O.K.	

Check Deflection:

$D_{max.}$	=		**0.29**	inches
$D_{Allowable}$	=	**L**/350	= **0.42**	inches
			Check - O.K.	

Steel Floor Beam Design

First Level Beam 22

Max Span - First Floor Level — Beam 22 is the worst case for beams 18, 23, 27, 28, 31 & 32

Assumptions:

1. W12 x 96 Beam — **26.00** Feet = 312.00 Inches
   Supports are: **3.00** inches from each end of the beam
   Therefore: **L** = 306.00 Inches
2. Beam designed as a simple span.
   Therefore: L_d = 306 Inches
3. Bearing area: (Assume: 6"x6" Plate) **N** = **6** Inches

W12 x 96 Beam Properties:

A	=	**28.20** inches^2		
A_{web}	=	**6.99** inches2	(Web area from top of beam to bottom of beam x web width.)	
E	=	**29,000,000** PSI		
I	=	**833** inches^4		
S	=	**131** inches^3		
$Fv_{Allowable}$	=	**9,500** PSI		SDOT 6-02.3(17)B For Salvaged Steel
		30,000 PSI	0.6F_y	AISC (Chapter G 16.1-64)
$Fp_{Allowable}$	=	**22,500** PSI		SDOT 6-02.3(17)B For Salvaged Steel
Fb	=	**33,000.00** PSI	0.66F_y	AISC (F3-1)
t_w	=	**0.55** inches		
d	=	**12.71** inches		
t_f	=	**0.9** inches		
F_y	=	**50,000** PSI		
k	=	**0.88** inches		
b_f	=	**12.16** inches		
W_t	=	**96** lbs/lf		

Reactions:

W_{design}	=	Load from deck joist plus DL from Beam	=	**3,138** plf	See Deck Jist Design
V_{max}	=		=	**40,794** lbs	
P_{max}	=		=	**40,794** lbs	
M_{max}	=		=	**265,161** ft-lbs	
D_{max}	=		=	**0.111** inches	

Check Shear:

Fv	=	V_{max} / A_{web}		=	**5,846** PSI
$Fv_{Allowable}$	=	0.6F_y		=	**30,000** PSI
					Check - O.K.
			V_{max}	=	**40,794** LBS
$Fv_{Allowable}$	=	$0.66F_wt_w(N + 2.5k)$		=	**148,603** LBS
		ASD Section K1.3			**Check - O.K.**

Check Bearing-Web Crippling:

Fp	=	P_{max} / A_{web}		=	**5,836** PSI
$Fp_{Allowable}$	=	0.6F_y		=	**30,000** PSI
					Check - O.K.
			V_{max}	=	**40,794** LBS
$Fp_{Allowable}$	=	$0.66F_wt_w(N + 2.5k)$		=	**148,603** LBS
		ASD Section K1.3			**Check - O.K.**

Check Bending:

$S_{Req'd}$	=	$M_{max}(12)$ / **Fb**	=	**96.4** inches^3
$S_{Furnished}$	=			**131** inches^3
				Check - O.K.

Check Deflection:

$D_{max.}$	=	D_{max}	=	**0.111** inches
$D_{Allowable}$	=	L/350	=	**0.891** inches
				Check - O.K.

FIRST FLOOR - FLOOR BEAM DESIGN:

First Floor - Beam 31

Baam 31 is a wosrt case to Beams 32, 33, 34, 35, 36 & 37

Assumptions:

1. Spans:		**ℓ =**	**30.25**	feet span
		=	363	inches
		a =	**5.42**	feet span
		=	65.04	inches
2. Floor Joist Span Supported			**18.00**	feet
Floor Joist Span Supported			**4.00**	feet
Floor Joist Span Supported			**0.00**	feet
Floor Joist Span Supported			**0.00**	feet
4. Assume bearing area =	Exterior Wall: (5.5" x 5.5")		**5.50**	inches

Steel Beam - W16 x 26

AISC Design Values

A	=	**7.68**	inches^2		
A_{web}	=	**7.55**	inches^2		
E	=	**29,000,000**	PSI		
S	=	**38**	inches3		
I	=	**301**	inches4		
$Fv_{Allowable}$	=	**9,500**	PSI		SDOT 6-02.3(17)B For Salvaged Steel
		30,000	PSI	0.6**F_y**	AISC (Chapter G 16.1-64)
$Fp_{Allowable}$	=	**22,500**	PSI		SDOT 6-02.3(17)B For Salvaged Steel
Fb	=	**33,000**	PSI		AISC (F3-1)
t_w	=	**0.25**	inches		
d	=	**15.69**	inches		
t_f	=	**0.345**	inches		
F_y	=	**50,000**	PSI		
k	=	**0.75**	inches		
b_f	=	**5.5**	inches		
W_t	=	**26**	lbs/lf		

Reactions:

W	=	Floor Framing DL	12.3	PSF
		Steel Beam DL	0.7	PSF
		Floor LL	40	PSF
		Design Load = DL x 1.2 + LL x 1.4 =	**71.63**	PSF

Reactions: **See Visual Analysis Resilts**

W	=	**645**	plf	18.00 ft span
W	=	**143**	plf	4.00 ft span
W	=		plf	0.00 ft span
W	=	**-**	plf	0.00 ft span
V_{max}	=	**5,959**	lbs	
P_{max}	=	**5,959**	lbs	
M_{max}	=	**45,066**	ft-lbs	
D_{max}	=	**0.07**	inches	

Check Shear:

Fvmax	=	**V_{max} / A_{web}**	=	**789**	lbs
$Fv_{Allowable}$	=			**9,500.0**	lbs
				Check - O.K.	
$Fv_{Allowable}$	=	**0.66$F_w t_w$(N + 2.5k)**	=	**60,844**	LBS
		ASD Section K1.3		**Check - O.K.**	

Check Bearing-Web Crippling:

F_p	=	P_{max} / A_{web}	=	**789** PSI
$F_{p_{Allowable}}$	=			**22,500.0** PSI **Check - O.K.**
$Fv_{Allowable}$	=	$0.66F_w t_w(N + 2.5k)$ ASD Section K1.3	=	**60,844** LBS **Check - O.K.**

Check Bending:

$S_{Req'd}$	=	$M_{max}(12) / F_b$	=	**16.4** IN^2
$S_{Furnished}$	=			**38.4** IN^2 **Check - O.K.**

Check Deflection:

$D_{max.}$	=			**0.07** inches
$D_{Allowable}$	=	L/350	=	**1.04** inches **Check - O.K.**

FLOOR JOIST DESIGN: 9' SPAN

Assumptions:

1. .5" x 7.25 Joist
2. Maximum span: **9.00** feet **(MAX)** 108 inches
   Therefore: **L** = 108 inches
3. Joist Spacing: **1.50** feet o.c. = 18 inches
4. Minimum Bearing Area: 1.5" x 1.5" = **2.25** $inches^2$

2" x 8" Joist Properties:

ANSI Design Value Hem-Fir (North)

A	=	**11.25**	$inches^2$
E	=	**1,700,000**	PSI
I	=	**52.7**	$inches^4$
S	=	**14.1**	$inches^3$
$Fv_{Allowable}$	=	**150**	LBS
$Fp_{Allowable}$	=	**405**	PSI
Fb	=	**1700**	PSI

Reactions:

W = Floor Load x Span Supported 192.90 PLF

Deck Dead Load	=	10.5	PSF
Green Roof DL	=	50	PSF
Deck Live Load	=	40	PSF
Design Load = DL x 1.2 + LL x 1.4 =		129	PSF

V_{max}	=	0.5**WL**	=	**868**	lbs
P_{max}	=	0.5**WL**	=	**868**	lbs
M_{max}	=	$(WL^2)/8$	=	**1,953**	ft-lbs
D_{max}	=	$0.013WL^4 / EI$	=	**0.23**	inches

Check Shear:

Fv	=		=	**77.16**	PSI
$Fv_{Allowable}$	=			**150**	PSI
				Check - O.K.	

Check Bearing:

Fp	=	P_{max} / **A**	=	**385.80**	PSI
$Fp_{Allowable}$	=			**405**	PSI
				Check - O.K.	

Check Bending:

$S_{Req'd}$	=	$M_{max}(12)$ / **Fb**	=	**13.79**	$inches^3$
$S_{Furnished}$	=			**14.1**	$inches^3$
				Check - O.K.	

Check Deflection:

$D_{max.}$	=			**0.2251**	inches
$D_{Allowable}$	=	**L**/350	=	**0.3086**	inches
				Check - O.K.	

HHDQ Heavy Holdown, Quick-Install



On This Page

Product Details

The HHDQ is designed for extra-high loads and provides ease of installation. This heavy holdown comes with Strong-Drive® SDS Heavy-Duty Connector screws, which install easily, reduce fastener slip and provide a greater net section than bolts. The HHDQ is installed directly on the sill plate and is suitable for back-to-back applications where eccentricity is a concern.

Key Features

- Strong-Drive SDS Heavy-Duty Connector screws are supplied with the holdowns to ensure proper fasteners are used
- Screws may be installed either flush or raised off the mudsill without a reduction in load value
- No stud bolts to countersink at openings

Material

- Body: 7 gauge, washer: 1/2" plate

Finish

- Simpson Strong-Tie gray paint; HHDQ11 — available in stainless steel

Installation

- See General Notes for Holdowns and Tension Ties
- Strong-Drive SDS Heavy-Duty Connector screws install best with a low-speed, high-torque drill equipped with a 3/8" hex-head driver
- No additional washer is required
- HHDQ14 requires a heavy-hex anchor nut (supplied with holdown)

Related Links

- Wood Construction Connectors Technical and Installation Notes
- General Notes for Holdowns and Tension Ties
- Holdown Selector Software

Catalog Pages

- C-C-2021 (Wood Construction Connectors), page 51



Wood Construction Connectors 2021-2023

C-C-2021 — *Catalog*

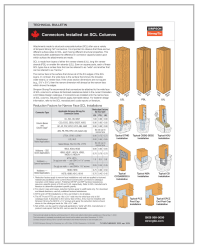
A catalog including new product information as well as any applicable specification and installation instructions for wood-to-wood and wood-to-concrete structural connectors.



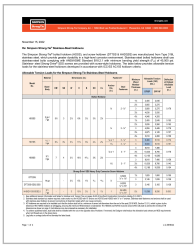
Conectores para la construcción con madera 2021–2023

C-C-2021SP — *Catalog*

El catálogo incluye información sobre nuevos productos, así como especificaciones e instrucciones de instalación aplicables para conectores estructurales de madera a madera y de madera a concreto.



Connectors Installed on SCL Columns



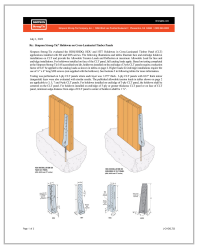
Simpson Strong-Tie® Stainless-Steel Holdowns

LMCAN23 — *Technical Bulletin*

A Canadian Limit States technical bulletin addressing typical installations of Simpson Strong-Tie® connectors on SCL columns as well as reduction factors for narrow-face SCL installations.

L-C-SSHD22 — *Engineering Letter*

The Simpson Strong-Tie bolted holdown (HDSS) and screw holdown (DTTSS & HHDQSS) are manufactured from Type 316L stainless steel, which provides superior durability in a highly corrosive environment.



Simpson Strong-Tie® Holdowns on Cross-Laminated Timber Panels

L-C-HDCLT23 — *Engineering Letter*

Simpson Strong-Tie evaluated the HDQ/HHDQ, HDU and HTT holdowns in cross-laminated timber (CLT) panel applications installed with SD and SDS screws.

Load Tables

These products are available with additional corrosion protection. Additional products on this page may also be available with this option, check with Simpson Strong-Tie for details.

SS For stainless-steel fasteners, see Fastener Types and Sizes Specified for Simpson Strong-Tie Connectors.

Model No.	Ga.	Dimensions (in.)					Fasteners		Minimum Wood Member Size (in.)	Allowable Tension Loads (160)		
		W	H	B	CL	SO	Anchor Bolt Dia. (in.)	SDS Screws		DF/SP	SPF/HF	Deflection at Allowable Load (in.)
HDQ8-SDS3	7	2⅞	14	2½	1¼	2⅜	⅞	(20) ¼" x 3"	3 x 3½	5,715	4,915	0.073
								(20) ¼" x 3"	3½ x 3½	7,630	6,560	0.091
								(20) ¼" x 3"	3½ x 4½	9,230	7,020	0.095
SS HHDQ11-SDS2.5	7	3	15⅛	3½	1½	⅞	1	(24) ¼" x 2½"	3½ x 5½	11,810	8,425	0.131
HHDQ14-SDS2.5	7	3	18¾	3½	1½	⅞	1	(30) ¼" x 2½"	3½ x 7¼	13,015	10,530	0.107
									5½ x 5½	13,710	10,530	0.107

1. HHDQ14 requires a heavy-hex anchor nut (supplied with holdown).
2. HDQ and HHDQ installed horizontally achieve compression loads with the addition of a standard nut on the underside of the load transfer plate. Refer to ICC-ES ESR-2330 for design values. HDQ8 requires a standard nut and BP 7/8-2 load washer (sold separately) on the underside of the holdown for compression load. Design of anchorage rods for compression force shall be per the Designer.

Code Reports & Compliance

Product Series	Compliance/Certification	Report/Approval
HHDQ	International Building Code International Residential Code City of Los Angeles Building Code City of Los Angeles Residential Code	ESR-2330
	Florida Building Code	FL10441

Footnotes

1. Please review compliance documents for information about specific product models. In some cases, Compliance documents cover most but not all models associated with a product series.
2. For additional information regarding Florida's Statewide Product Approval System and Miami-Dade County Notice of Acceptance (NOA), click here.

Related Products

SECOND FLOOR - FLOOR BEAM DESIGN: Second Floor - Beam 13

Assumptions:

1. Spans:		**ℓ =**	**22.17**	feet span
		=	266.04	inches
		a =	**0**	feet span
		=	0	inches
2. Floor Joist Span Supported			**15.25**	feet
Floor Joist Span Supported			**0.00**	feet
Floor Joist Span Supported			**0.00**	feet
Floor Joist Span Supported			**0.00**	feet
4. Assume bearing area =	Exterior Wall: (5.5" x 5.5")		**5.50**	inches

Steel Beam - W10 x 22

AISC Design Values

A	=	**6.49**	inches^2		
A_{web}	=	**7.55**	inches^2		
E	=	**29,000,000**	PSI		
S	=	**23**	inches3		
I	=	**118**	inches4		
$Fv_{Allowable}$	=	**9,500**	PSI		SDOT 6-02.3(17)B For Salvaged Steel
		30,000	PSI	$0.6F_y$	AISC (Chapter G 16.1-64)
$Fp_{Allowable}$	=	**22,500**	PSI		SDOT 6-02.3(17)B For Salvaged Steel
Fb	=	**33,000**	PSI		AISC (F3-1)
t_w	=	**0.24**	inches		
d	=	**10.17**	inches		
t_f	=	**0.36**	inches		
F_y	=	**50,000**	PSI		
k	=	**0.50**	inches		
b_f	=	**5.57**	inches		
W_t	=	**22**	lbs/lf		

Reactions:

W	=	Floor Framing DL	12.3	PSF
		Steel Beam DL	1.0	PSF
		Floor LL	40	PSF
		Design Load = DL x 1.2 + LL x 1.4 =	**71.95**	PSF

Reactions: **See Visual Analysis Resilts**

W	=	**550**	plf	15.25 ft span
W	=		plf	0.00 ft span
W	=		plf	0.00 ft span
W	=	**-**	plf	0.00 ft span
V_{max}	=	**5,812**	lbs	
P_{max}	=	**5,812**	lbs	
M_{max}	=	**32,214**	ft-lbs	
D_{max}	=	**0.39**	inches	

Check Shear:

Fvmax	=	**V_{max} / A_{web}**	=	**770**	lbs
$Fv_{Allowable}$	=			**9,500.0**	lbs
				Check - O.K.	
$Fv_{Allowable}$	=	**0.66$F_w t_w$(N + 2.5k)**	=	**53,460**	LBS
		ASD Section K1.3		**Check - O.K.**	

Check Bearing-Web Crippling:

F_p	=	P_{max} / A_{web}	=	**770** PSI
$F_{p_{Allowable}}$	=			**22,500.0** PSI **Check - O.K.**
$Fv_{Allowable}$	=	$0.66F_w t_w(N + 2.5k)$ ASD Section K1.3	=	**53,460** LBS **Check - O.K.**

Check Bending:

$S_{Req'd}$	=	$M_{max}(12) / F_b$	=	**11.7** IN^2
$S_{Furnished}$	=			**23.2** IN^2 **Check - O.K.**

Check Deflection:

$D_{max.}$	=			**0.39** inches
$D_{Allowable}$	=	L/350	=	**0.76** inches **Check - O.K.**

SECOND FLOOR - FLOOR BEAM DESIGN: Second Floor - Beam 5

Assumptions:

1. Spans:		**ℓ =**	**16.33**	feet span
		=	195.96	inches
		a =	**5.6**	feet span
		=	66.96	inches
2. Floor Joist Span Supported			**20.50**	feet
Floor Joist Span Supported			**0.00**	feet
Floor Joist Span Supported			**0.00**	feet
Floor Joist Span Supported			**0.00**	feet
4. Assume bearing area =	Exterior Wall: (5.5" x 5.5")		**5.50**	inches

Steel Beam - W10 x 30

AISC Design Values

A	=	**8.84**	inches^2		
A_{web}	=	**7.55**	inches^2		
E	=	**29,000,000**	PSI		
S	=	**32**	inches3		
I	=	**170**	inches4		
$Fv_{Allowable}$	=	**9,500**	PSI		SDOT 6-02.3(17)B For Salvaged Steel
		30,000	PSI	$0.6F_y$	AISC (Chapter G 16.1-64)
$Fp_{Allowable}$	=	**22,500**	PSI		SDOT 6-02.3(17)B For Salvaged Steel
Fb	=	**33,000**	PSI		AISC (F3-1)
t_w	=	**3**	inches		
d	=	**10.47**	inches		
t_f	=	**0.51**	inches		
F_y	=	**50,000**	PSI		
k	=	**0.50**	inches		
b_f	=	**5.81**	inches		
W_t	=	**30**	lbs/lf		

Reactions:

W	=	Floor Framing DL	12.3	PSF
		Steel Beam DL	1.4	PSF
		Floor LL	40	PSF
		Design Load = DL x 1.2 + LL x 1.4 =	**72.40**	PSF

Reactions: **See Visual Analysis Resilts**

W	=	**772**	plf	20.50 ft span
W	=	**-**	plf	0.00 ft span
W	=		plf	0.00 ft span
W	=	**-**	plf	0.00 ft span
V_{max}	=	**7,417**	lbs	
P_{max}	=	**7,417**	lbs	
M_{max}	=	**17,091**	ft-lbs	
D_{max}	=	**0.05**	inches	

Check Shear:

Fvmax	=	**V_{max} / A_{web}**	=	**982**	lbs
$Fv_{Allowable}$	=			**9,500.0**	lbs
				Check - O.K.	
$Fv_{Allowable}$	=	**0.66$F_w t_w$(N + 2.5k)**	=	**668,250**	LBS
		ASD Section K1.3		**Check - O.K.**	

Check Bearing-Web Crippling:

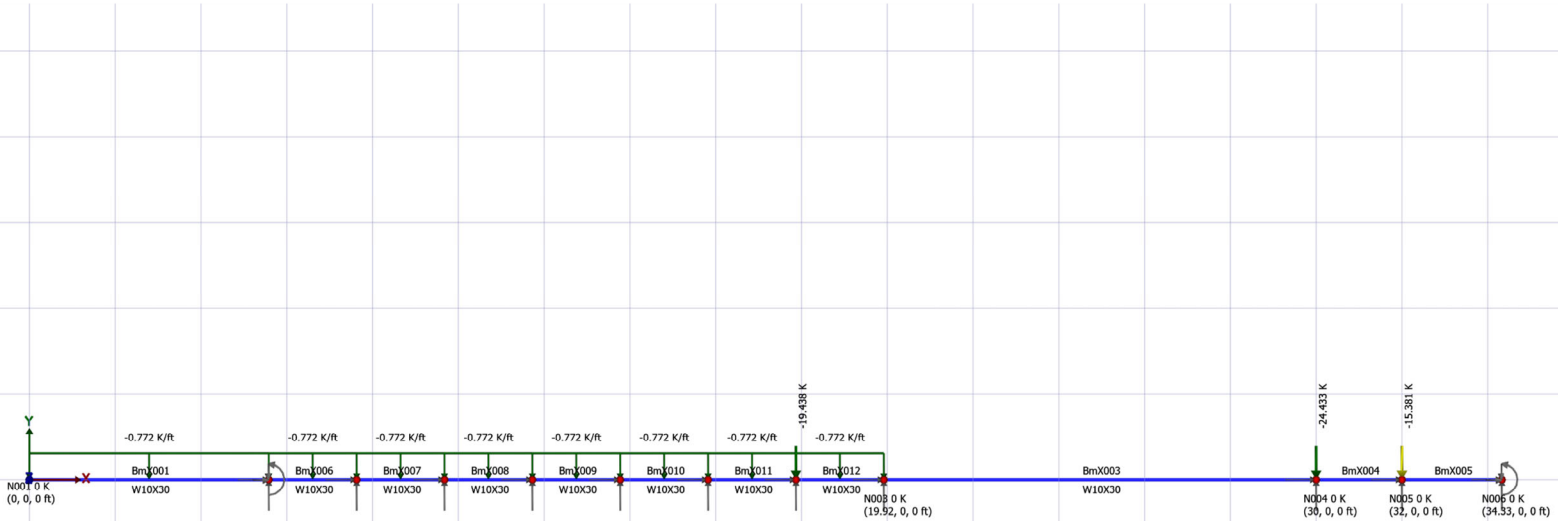
F_p	=	P_{max} / A_{web}	=	**982** PSI
$F_{p_{Allowable}}$	=			**22,500.0** PSI **Check - O.K.**
$Fv_{Allowable}$	=	$0.66F_w t_w(N + 2.5k)$ ASD Section K1.3	=	**668,250** LBS **Check - O.K.**

Check Bending:

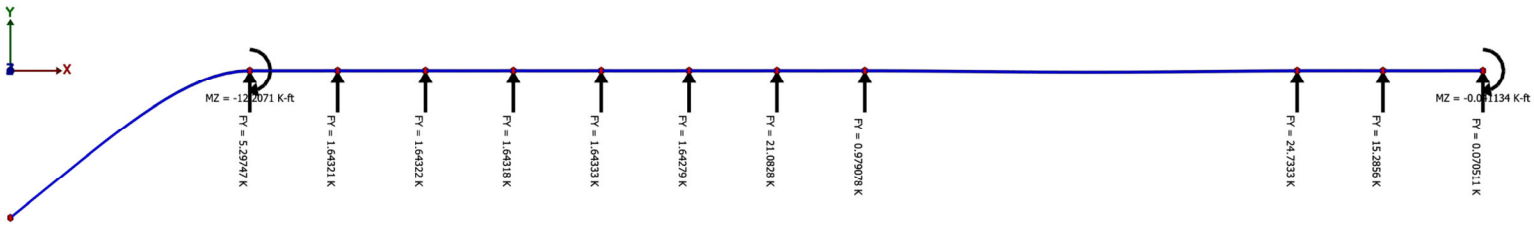
$S_{Req'd}$	=	$M_{max}(12) / F_b$	=	**6.2** IN^2
$S_{Furnished}$	=			**32.4** IN^2 **Check - O.K.**

Check Deflection:

$D_{max.}$	=			**0.05** inches
$D_{Allowable}$	=	L/350	=	**0.56** inches **Check - O.K.**


-0.772 K/ft
-0.772 K/ft
-0.772 K/ft
-0.772 K/ft
-0.772 K/ft
-0.772 K/ft
-0.772 K/ft
-19.438 K
-0.772 K/ft
-24.433 K
-15.381 K
BmX001
BmX006
BmX007
BmX008
BmX009
BmX010
BmX011
BmX012
BmX003
BmX004
BmX005
W10X30
N001 0 K
(0, 0, 0 ft)
N003 0 K
(19.92, 0, 0 ft)
N004 0 K
(30, 0, 0 ft)
N005 0 K
(32, 0, 0 ft)
N006 0 K
(34.33, 0, 0 ft)

Front


Y
X
MZ = -12.2071 K-ft
FY = 5.29747 K
FY = 1.64321 K
FY = 1.64322 K
FY = 1.64318 K
FY = 1.64333 K
FY = 1.64279 K
FY = 21.0828 K
FY = 0.979078 K
FY = 24.7333 K
FY = 15.2856 K
FY = 0.0705:1 K
MZ = -0.0

Front

Factored Load Combinations

Name	Code	Effective Equation	Design	Deflection
1. 1.4D	ASCE 7-16 LRFD	1.4D	Strength	Other
2. 1.2D+1.6L+0.5Lr	ASCE 7-16 LRFD	1.2D	Strength	Other
5. 0.9D+W	ASCE 7-16 LRFD	0.9D	Strength	Other

Member Analysis Properties

Name	Member Section	X1 ft	Y1 ft	Z1 ft	X2 ft	Y2 ft	Z2 ft	Elasticity, E Ksi	A in^2	Iz in^4	Iy in^4	Releases	Action
BmX001	W10X30	0.00000	0.00000	0.00000	5.58000	0.00000	0.00000	29000.00000	8.84000	170.00000	16.70000	(none)	Normal
BmX003	W10X30	19.92000	0.00000	0.00000	30.00000	0.00000	0.00000	29000.00000	8.84000	170.00000	16.70000	(none)	Normal
BmX004	W10X30	30.00000	0.00000	0.00000	32.00000	0.00000	0.00000	29000.00000	8.84000	170.00000	16.70000	(none)	Normal
BmX005	W10X30	32.00000	0.00000	0.00000	34.33000	0.00000	0.00000	29000.00000	8.84000	170.00000	16.70000	(none)	Normal
BmX006	W10X30	5.58000	0.00000	0.00000	7.62857	0.00000	0.00000	29000.00000	8.84000	170.00000	16.70000	(none)	Normal
BmX007	W10X30	7.62857	0.00000	0.00000	9.67714	0.00000	0.00000	29000.00000	8.84000	170.00000	16.70000	(none)	Normal
BmX008	W10X30	9.67714	0.00000	0.00000	11.72571	0.00000	0.00000	29000.00000	8.84000	170.00000	16.70000	(none)	Normal
BmX009	W10X30	11.72571	0.00000	0.00000	13.77429	0.00000	0.00000	29000.00000	8.84000	170.00000	16.70000	(none)	Normal
BmX010	W10X30	13.77429	0.00000	0.00000	15.82286	0.00000	0.00000	29000.00000	8.84000	170.00000	16.70000	(none)	Normal
BmX011	W10X30	15.82286	0.00000	0.00000	17.87143	0.00000	0.00000	29000.00000	8.84000	170.00000	16.70000	(none)	Normal
BmX012	W10X30	17.87143	0.00000	0.00000	19.92000	0.00000	0.00000	29000.00000	8.84000	170.00000	16.70000	(none)	Normal

Section Properties

Section	Source	Area in^2	Iz in^4	Iy in^4	J in^4	Alpha, α deg
W10X30	Database Shape	8.84000	170.00000	16.70000	0.62200	0.00000

Member Stresses

(extreme rows: max and min)

Member	fa Min Ksi	fa Max Ksi	fby Min Ksi	fby Max Ksi	fbz Min Ksi	fbz Max Ksi	fc comb Min Ksi	fc comb Max Ksi
BmX001	0.00000 (4)	0.00000 (4)	0.00000 (4)	0.00000 (4)	**-6.47890 (2)**	**6.47890 (2)**	**-6.47890 (2)**	**6.47890 (2)**
BmX005	0.00000 (4)	0.00000 (4)	0.00000 (4)	0.00000 (4)	**-0.02147 (2)**	**0.02147 (2)**	**-0.02147 (2)**	**0.02147 (2)**

Node Results

(extreme rows: max and min)

Node	Result Case	DX in	DY in	DZ in	FX K	FY K	FZ K	MX K-ft	MY K-ft	MZ K-ft
N001	1. 1.4D	0.00000	**-0.04770**	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
N001	5. 0.9D+W	0.00000	**-0.03066**	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
N002	1. 1.4D	0.00000	0.00000	0.00000	0.00000	7.41646	0.00000	0.00000	0.00000	**-17.09001**
N004	1. 1.4D	0.00000	0.00000	0.00000	0.00000	**34.62658**	0.00000	0.00000	0.00000	0.00000
N006	5. 0.9D+W	0.00000	0.00000	0.00000	0.00000	**0.06346**	0.00000	0.00000	0.00000	**-0.03702**

Node Reactions

(extreme rows: max and min)

Node	Result Case	FX K	FY K	FZ K	MX K-ft	MY K-ft	MZ K-ft
N002	1. 1.4D	0.00000	7.41646	0.00000	0.00000	0.00000	**-17.09001**
N004	1. 1.4D	0.00000	**34.62658**	0.00000	0.00000	0.00000	0.00000
N006	5. 0.9D+W	0.00000	**0.06346**	0.00000	0.00000	0.00000	**-0.03702**

Steel_Beam X_G 1: Results

Deflections Strong (dy): None Weak (dz): None	Axial Manual Kz: False Kz Sidesway?: False Manual Ky: False Ky Sidesway?: False	Size Constraints Limit Depth?: False Limit Width?: False
Overrides Override Fy?: False Override Cb?: False Override HSS t_des?: False Advanced Torsion: False		
Steel Material: ASTM A992 Grade 50 Specification: AISC 360-16 LRFD Composite Beam?: False Seismic Compactness: Not Ductile Check Constrained Axis FTB?: False Overstrength?: False Live Load Reduction: None Disable Checks?: False Check Level: Each Limit State	Bracing Lateral Top (+y): Unbraced Lateral Bottom (-y): Unbraced Strong (z): Unbraced	Torsional Bracing Lateral Top (+y): True Lateral Bottom (-y): True Strong (z): True

Steel_Beam X_G 1: Strong Flexure Check (extreme rows: max)

Member	Section	Offset ft	Result Case	Demand Mz K-ft	Capacity Mz K-ft	Code Reference	Unity Check	Details
BmX001	W10X30	5.58000	1. 1.4D	-17.48274	137.25000	F2-1	**0.12738**	Lb = 5.58 ft, Cb = 2.32532

Steel_Beam X_G 1: Strong Shear Check (extreme rows: max)

Member	Section	Offset ft	Result Case	Demand Vy K	Capacity Vy K	Code Reference	Unity Check	Details
BmX001	W10X30	5.58000	1. 1.4D	-6.26621	94.50000	G2-1	**0.06631**	Shear Area = 3.15 in^2, Cv = 1, h/tw = 29.6

Member Displacements (extreme rows: max and min)

Member	Dy Min in	Dy Max in	Dz Min in	Dz Max in
BmX001	**-0.04770 (2)**	0.00000 (4)	0.00000 (4)	0.00000 (4)
BmX004	0.00000 (2)	**0.00002 (2)**	0.00000 (4)	0.00000 (4)
BmX005	**-0.00001 (2)**	0.00000 (4)	0.00000 (4)	0.00000 (4)

SECOND FLOOR - FLOOR BEAM DESIGN: Second Floor - Beam 10

Assumptions:

1. Spans:		**ℓ =**	**20.50**	feet span
		=	246	inches
		a =	**18.0**	feet span
		=	216	inches
2. Floor Joist Span Supported			**21.17**	feet
Floor Joist Span Supported			**10.50**	feet
Floor Joist Span Supported			**0.00**	feet
Floor Joist Span Supported			**0.00**	feet
4. Assume bearing area =	Exterior Wall: (5.5" x 5.5")		**5.50**	inches

Steel Beam - W10 x 68

AISC Design Values

A	=	**20.00**	$inches^2$		
$\mathbf{A_{web}}$	=	**7.55**	$inches^2$		
E	=	**29,000,000**	PSI		
S	=	**76**	inches3		
I	=	**394**	inches4		
$\mathbf{Fv_{Allowable}}$	=	**9,500**	PSI		SDOT 6-02.3(17)B For Salvaged Steel
		30,000	PSI	$0.6\mathbf{F_y}$	AISC (Chapter G 16.1-64)
$\mathbf{Fp_{Allowable}}$	=	**22,500**	PSI		SDOT 6-02.3(17)B For Salvaged Steel
Fb	=	**33,000**	PSI		AISC (F3-1)
$\mathbf{t_w}$	=	**0.47**	inches		
d	=	**10.4**	inches		
$\mathbf{t_f}$	=	**0.77**	inches		
$\mathbf{F_y}$	=	**50,000**	PSI		
k	=	**0.75**	inches		
$\mathbf{b_f}$	=	**10.13**	inches		
$\mathbf{W_t}$	=	**68**	lbs/lf		

Reactions:

W	=	Floor Framing DL	12.3	PSF
		Steel Beam DL	1.8	PSF
		Floor LL	40	PSF
		Design Load = DL x 1.2 + LL x 1.4 =	**72.88**	PSF

Reactions: **See Visual Analysis Resilts**

W	=	**772**	plf	21.17 ft span
W	=	**383**	plf	10.50 ft span
W	=		plf	0.00 ft span
W	=	**-**	plf	0.00 ft span
$\mathbf{V_{max}}$	=	**41,180**	lbs	
$\mathbf{P_{max}}$	=	**41,180**	lbs	
$\mathbf{M_{max}}$	=	**78,302**	ft-lbs	
$\mathbf{D_{max}}$	=	**0.14**	inches	

Check Shear:

Fvmax	=	$\mathbf{V_{max}} / \mathbf{A_{web}}$	=	**5,454**	lbs
$\mathbf{Fv_{Allowable}}$	=			**9,500.0**	lbs
				Check - O.K.	
$\mathbf{Fv_{Allowable}}$	=	$\mathbf{0.66F_w t_w}(N + 2.5k)$	=	**114,386**	LBS
		ASD Section K1.3		**Check - O.K.**	

Check Bearing-Web Crippling:

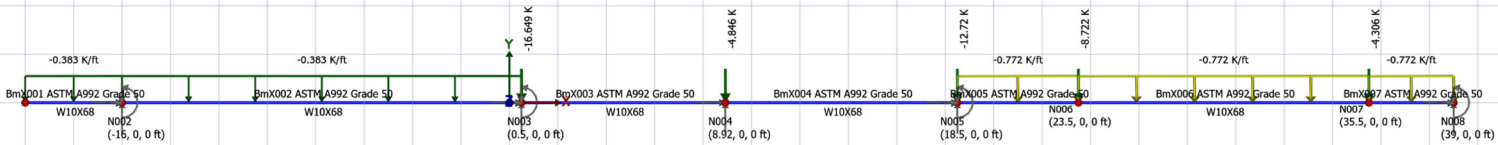
F_p	=	P_{max} / A_{web}	=	**5,454** PSI
$F_{p_{Allowable}}$	=			**22,500.0** PSI **Check - O.K.**
$Fv_{Allowable}$	=	$0.66F_w t_w(N + 2.5k)$ ASD Section K1.3	=	**114,386** LBS **Check - O.K.**

Check Bending:

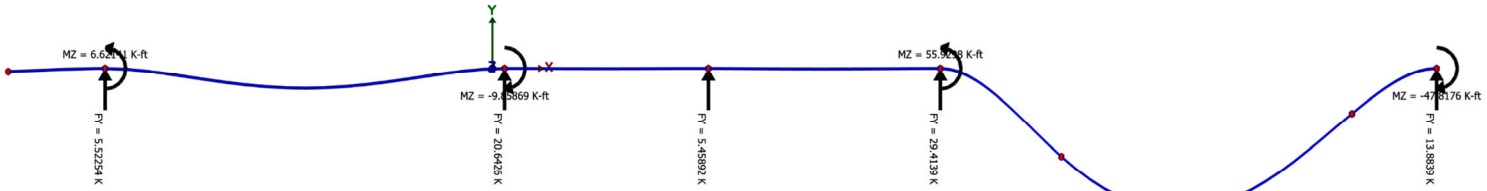
$S_{Req'd}$	=	$M_{max}(12) / F_b$	=	**28.5** IN^2
$S_{Furnished}$	=			**75.8** IN^2 **Check - O.K.**

Check Deflection:

$D_{max.}$	=			**0.14** inches
$D_{Allowable}$	=	L/350	=	**0.70** inches **Check - O.K.**


-0.383 K/ft
-0.383 K/ft
-16.649 K
-4.846 K
-12.72 K
-8.722 K
-4.306 K
-0.772 K/ft
-0.772 K/ft
-0.772 K/ft
Y
BmX001 ASTM A992 Grade 50
W10X68
N002
(-16, 0, 0 ft)
BmX002 ASTM A992 Grade 50
W10X68
N003
(0.5, 0, 0 ft)
BmX003 ASTM A992 Grade 50
W10X68
N004
(8.92, 0, 0 ft)
BmX004 ASTM A992 Grade 50
W10X68
N005
(18.5, 0, 0 ft)
BmX005 ASTM A992 Grade 50
N006
(23.5, 0, 0 ft)
BmX006 ASTM A992 Grade 50
W10X68
N007
(35.5, 0, 0 ft)
BmX007 ASTM A992 Grade 50
N008
(39, 0, 0 ft)

Front


Y
MZ = 6.62141 K-ft
MZ = -9.45869 K-ft
MZ = 55.9238 K-ft
MZ = -47.8176 K-ft
FY = 5.52254 K
FY = 20.6425 K
FY = 5.45892 K
FY = 29.4139 K
FY = 13.8839 K

Front

Factored Load Combinations

Name	Code	Effective Equation	Design	Deflection
1. 1.4D	ASCE 7-16 LRFD	1.4D	Strength	Other
2. 1.2D+1.6L+0.5Lr	ASCE 7-16 LRFD	1.2D	Strength	Other
5. 0.9D+W	ASCE 7-16 LRFD	0.9D	Strength	Other

Member Analysis Properties

Name	Member Section	X1 ft	Y1 ft	Z1 ft	X2 ft	Y2 ft	Z2 ft	Elasticity, E Ksi	A in^2	Iz in^4	Iy in^4	Releases	Action
BmX001	W10X68	-20.00000	0.00000	0.00000	-16.00000	0.00000	0.00000	29000.00000	19.90000	394.00000	134.00000	(none)	Normal
BmX002	W10X68	-16.00000	0.00000	0.00000	0.50000	0.00000	0.00000	29000.00000	19.90000	394.00000	134.00000	(none)	Normal
BmX003	W10X68	0.50000	0.00000	0.00000	8.92000	0.00000	0.00000	29000.00000	19.90000	394.00000	134.00000	(none)	Normal
BmX004	W10X68	8.92000	0.00000	0.00000	18.50000	0.00000	0.00000	29000.00000	19.90000	394.00000	134.00000	(none)	Normal
BmX005	W10X68	18.50000	0.00000	0.00000	23.50000	0.00000	0.00000	29000.00000	19.90000	394.00000	134.00000	(none)	Normal
BmX006	W10X68	23.50000	0.00000	0.00000	35.50000	0.00000	0.00000	29000.00000	19.90000	394.00000	134.00000	(none)	Normal
BmX007	W10X68	35.50000	0.00000	0.00000	39.00000	0.00000	0.00000	29000.00000	19.90000	394.00000	134.00000	(none)	Normal

Section Properties

Section	Source	Area in^2	Iz in^4	Iy in^4	J in^4	Alpha, α deg
W10X68	Database Shape	19.90000	394.00000	134.00000	3.56000	0.00000

Member Stresses

(extreme rows: max and min)

Member	fa Min Ksi	fa Max Ksi	fby Min Ksi	fby Max Ksi	fbz Min Ksi	fbz Max Ksi	fc comb Min Ksi	fc comb Max Ksi
BmX003	0.00000 (4)	0.00000 (4)	0.00000 (4)	0.00000 (4)	**-0.10277 (2)**	**0.10277 (2)**	**-0.10277 (2)**	**0.10277 (2)**
BmX005	0.00000 (4)	0.00000 (4)	0.00000 (4)	0.00000 (4)	**-12.52221 (2)**	**12.52221 (2)**	**-12.52221 (2)**	**12.52221 (2)**

Node Results

(extreme rows: max and min)

Node	Result Case	DX in	DY in	DZ in	FX K	FY K	FZ K	MX K-ft	MY K-ft	MZ K-ft
N001	5. 0.9D+W	0.00000	**-0.00196**	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000

Node Results (continued) (extreme rows: max and min)

Node	Result Case	DX in	DY in	DZ in	FX K	FY K	FZ K	MX K-ft	MY K-ft	MZ K-ft
N004	5. 0.9D+W	0.00000	0.00000	0.00000	0.00000	**4.91303**	0.00000	0.00000	0.00000	0.00000
N005	1. 1.4D	0.00000	0.00000	0.00000	0.00000	**41.17941**	0.00000	0.00000	0.00000	**78.30171**
N006	1. 1.4D	0.00000	**-0.08193**	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
N008	1. 1.4D	0.00000	0.00000	0.00000	0.00000	19.43750	0.00000	0.00000	0.00000	**-66.94461**

Node Reactions (extreme rows: max and min)

Node	Result Case	FX K	FY K	FZ K	MX K-ft	MY K-ft	MZ K-ft
N004	5. 0.9D+W	0.00000	**4.91303**	0.00000	0.00000	0.00000	0.00000
N005	1. 1.4D	0.00000	**41.17941**	0.00000	0.00000	0.00000	**78.30171**
N008	1. 1.4D	0.00000	19.43750	0.00000	0.00000	0.00000	**-66.94461**

Steel_Beam X_G 1: Results

Deflections Strong (dy): None Weak (dz): None	Axial Manual Kz: False Kz Sidesway?: False Manual Ky: False Ky Sidesway?: False	Size Constraints Limit Depth?: False Limit Width?: False
Overrides Override Fy?: False Override Cb?: False Override HSS t_des?: False Advanced Torsion: False		
Steel Material: ASTM A992 Grade 50 Specification: AISC 360-16 LRFD Composite Beam?: False Seismic Compactness: Not Ductile Check Constrained Axis FTB?: False Overstrength?: False Live Load Reduction: None Disable Checks?: False Check Level: Each Limit State	Bracing Lateral Top (+y): Unbraced Lateral Bottom (-y): Unbraced Strong (z): Unbraced	Torsional Bracing Lateral Top (+y): True Lateral Bottom (-y): True Strong (z): True

Steel_Beam X_G 1: Strong Flexure Check (extreme rows: max)

Member	Section	Offset ft	Result Case	Demand Mz K-ft	Capacity Mz K-ft	Code Reference	Unity Check	Details
BmX005	W10X68	0.00000	1. 1.4D	-79.06650	319.87500	F2-1	**0.24718**	Lb = 5 ft, Cb = 2.15802

Steel_Beam X_G 1: Strong Shear Check (extreme rows: max)

Member	Section	Offset ft	Result Case	Demand Vy K	Capacity Vy K	Code Reference	Unity Check	Details
BmX005	W10X68	0.00000	1. 1.4D	22.90451	146.64000	G2-1	**0.15620**	Shear Area = 4.888 in^2, Cv = 1, h/tw = 16.7234

Member Displacements (extreme rows: max and min)

Member	Dy Min in	Dy Max in	Dz Min in	Dz Max in
BmX003	**-0.00016 (2)**	**0.00000 (2)**	0.00000 (4)	0.00000 (4)
BmX006	**-0.13320 (2)**	**-0.02708 (4)**	0.00000 (4)	0.00000 (4)

Node Results 2 (extreme rows: max and min)

Node	Result Case	DX in	DY in	DZ in	FX K	FY K	FZ K	MX K-ft	MY K-ft	MZ K-ft
N001	5. 0.9D+W	0.00000	**-0.00196**	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
N004	5. 0.9D+W	0.00000	0.00000	0.00000	0.00000	**4.91303**	0.00000	0.00000	0.00000	0.00000
N005	1. 1.4D	0.00000	0.00000	0.00000	0.00000	**41.17941**	0.00000	0.00000	0.00000	**78.30171**
N006	1. 1.4D	0.00000	**-0.08193**	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
N008	1. 1.4D	0.00000	0.00000	0.00000	0.00000	19.43750	0.00000	0.00000	0.00000	**-66.94461**

FLOOR JOIST DESIGN: 14.25' SPAN

Assumptions:

1. 2" x 10 Joist Beam
2. Maximum span: **14.25** feet **(MAX)** = 171 inches
   Therefore: **L** = 171 inches
3. Joist Spacing: **1.50** feet o.c. = 18 inches
4. Minimum Bearing Area: (1.5" x 2.5") Simpson Hanger = **3.75** $inches^2$

2" x 10" Joist Properties:

ANSI Design Valu Hem-Fir (North)

A	=	**13.875**	$inches^2$
E	=	**1,700,000**	PSI
I	=	**98.9**	$inches^4$
S	=	**21.4**	$inches^3$
$Fv_{Allowable}$	=	**150**	LBS
$Fp_{Allowable}$	=	**405**	PSI
Fb	=	**1700**	PSI

Reactions:

W = Floor Load x Span Supported 102.90 PLF

Deck Dead Load	=	10.5	PSF
Deck Live Load	=	40	PSF
Design Load = DL x 1.2 + LL x 1.4 =		69	PSF

V_{max}	=	0.5**WL**	=	**733**	lbs
P_{max}	=	0.5**WL**	=	**733**	lbs
M_{max}	=	**(WL**^{2}**)**/8	=	**2,612**	ft-lbs
D_{max}	=	0.013**WL**^{4} / **EI**	=	**0.26**	inches

Check Shear:

Fv = = **52.84** PSI

$Fv_{Allowable}$ = **150** PSI

Check - O.K.

Check Bearing:

Fp = **P_{max} / A** = **195.51** PSI

$Fp_{Allowable}$ = **405** PSI

Check - O.K.

Check Bending:

$S_{Req'd}$ = **M_{max}**(12) / **Fb** = **18.44** $inches^3$

$S_{Furnished}$ = **21.4** $inches^3$

Check - O.K.

Check Deflection:

$D_{max.}$ = **0.2583** inches

$D_{Allowable}$ = **L**/350 = **0.4886** inches

Check - O.K.

FLOOR JOIST DESIGN: 15.25' SPAN

Assumptions:

1. 1.75" x 7.25" LVL Joist
2. Maximum span: **15.25** feet **(MAX)** = 183 inches
   Therefore: **L** = 183 inches
3. Joist Spacing: **1.00** feet o.c. = 12 inches
4. Minimum Bearing Area: (1.5" x 2.5") Simpson Hanger = **3.75** $inches^2$

1.75" x 7.25" Joist Properties:

ANSI Design Valu Hem-Fir (North)

A	=	**12.6875**	$inches^2$
E	=	**2,000,000**	PSI
I	=	**55.6**	$inches^4$
S	=	**15.3**	$inches^3$
$\mathbf{Fv}_{Allowable}$	=	**2,453**	LBS
$\mathbf{Fp}_{Allowable}$	=	**750**	PSI
$\mathbf{M}_{max}$	=	**4336**	FT-LBS
Fb	=	**3100**	PSI

Reactions:

W = Floor Load x Span Supported = 128.60 PLF

Deck Dead Load	=	10.5	PSF
Green Roof DL	=	50	PSF
Deck Live Load	=	40	PSF
Design Load = DL x 1.2 + LL x 1.4 =		129	PSF

$\mathbf{V}_{max}$	=	0.5**WL**	=	**981**	lbs
$\mathbf{P}_{max}$	=	0.5**WL**	=	**981**	lbs
$\mathbf{M}_{max}$	=	$(\mathbf{WL}^2)/8$	=	**3,738**	ft-lbs
$\mathbf{D}_{max}$	=	$0.013\mathbf{WL}^4 / \mathbf{EI}$	=	**0.25**	inches

Check Shear:

F_v	=		=	**77.29**	PSI
$Fv_{Allowable}$	=			**2,453**	PSI
				Check - O.K.	

Check Bearing:

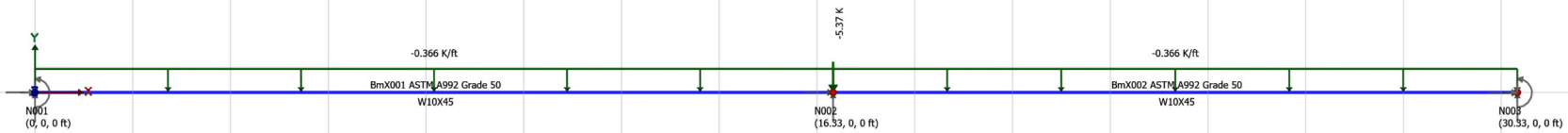
F_p	=	$\mathbf{P}_{max} / \mathbf{A}$	=	**261.49**	PSI
$Fp_{Allowable}$	=			**750**	PSI
				Check - O.K.	

Check Bending:

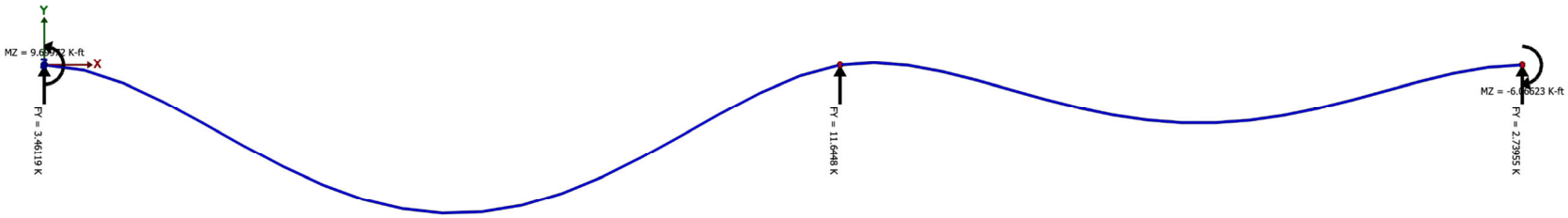
$M_{Req'd}$	=		=	**3738.44**	ft-lbs
$M_{Furnished}$	=			**4336.0**	ft-lbs
				Check - O.K.	

Check Deflection:

$D_{max.}$	=			**0.25**	inches
$D_{Allowable}$	=	**L**/350	=	**0.52**	inches
				Check - O.K.	


Y
X
-0.366 K/ft
-5.37 K
-0.366 K/ft
BmX001 ASTM A992 Grade 50
W10X45
BmX002 ASTM A992 Grade 50
W10X45
N001
(0, 0, 0 ft)
N002
(16.33, 0, 0 ft)
N003
(30.33, 0, 0 ft)

Front


Y
X
MZ = 9.69372 K-ft
FY = 3.46119 K
FY = 11.6448 K
MZ = -6.06523 K-ft
FY = 2.73955 K

Front

Factored Load Combinations

Name	Code	Effective Equation	Design	Deflection
1. 1.4D	ASCE 7-16 LRFD	1.4D	Strength	Other
2. 1.2D+1.6L+0.5Lr	ASCE 7-16 LRFD	1.2D	Strength	Other
5. 0.9D+W	ASCE 7-16 LRFD	0.9D	Strength	Other

Member Analysis Properties

Name	Member Section	X1 ft	Y1 ft	Z1 ft	X2 ft	Y2 ft	Z2 ft	Elasticity, E Ksi	A in^2	Iz in^4	Iy in^4	Releases	Action
BmX001	W10X45	0.00000	0.00000	0.00000	16.33000	0.00000	0.00000	29000.00000	13.30000	248.00000	53.40000	(none)	Normal
BmX002	W10X45	16.33000	0.00000	0.00000	30.33000	0.00000	0.00000	29000.00000	13.30000	248.00000	53.40000	(none)	Normal

Section Properties

Section	Source	Area in^2	Iz in^4	Iy in^4	J in^4	Alpha, α deg
W10X45	Database Shape	13.30000	248.00000	53.40000	1.51000	0.00000

Member Stresses (extreme rows: max and min)

Member	fa Min Ksi	fa Max Ksi	fby Min Ksi	fby Max Ksi	fbz Min Ksi	fbz Max Ksi	fc comb Min Ksi	fc comb Max Ksi
BmX001	0.00000 (4)	0.00000 (4)	0.00000 (4)	0.00000 (4)	**-3.31824 (2)**	**3.31824 (2)**	**-3.31824 (2)**	**3.31824 (2)**
BmX002	0.00000 (4)	0.00000 (4)	0.00000 (4)	0.00000 (4)	**-2.74449 (2)**	**2.74449 (2)**	**-2.74449 (2)**	**2.74449 (2)**

Node Reactions (extreme rows: max and min)

Node	Result Case	FX K	FY K	FZ K	MX K-ft	MY K-ft	MZ K-ft
N001	1. 1.4D	0.00000	4.84566	0.00000	0.00000	0.00000	**13.57961**
N002	1. 1.4D	0.00000	**16.30271**	0.00000	0.00000	0.00000	0.00000
N003	1. 1.4D	0.00000	3.83537	0.00000	0.00000	0.00000	**-8.49272**
N003	5. 0.9D+W	0.00000	**2.46559**	0.00000	0.00000	0.00000	-5.45960

Steel_Beam X_G 1: Results

Deflections Strong (dy): None Weak (dz): None	Axial Manual Kz: False Kz Sidesway?: False Manual Ky: False Ky Sidesway?: False	Size Constraints Limit Depth?: False Limit Width?: False
Overrides Override Fy?: False Override Cb?: False Override HSS t_des?: False Advanced Torsion: False		
Steel Material: ASTM A992 Grade 50 Specification: AISC 360-16 LRFD Composite Beam?: False Seismic Compactness: Not Ductile Check Constrained Axis FTB?: False Overstrength?: False Live Load Reduction: None Disable Checks?: False Check Level: Each Limit State	Bracing Lateral Top (+y): Unbraced Lateral Bottom (-y): Unbraced Strong (z): Unbraced	Torsional Bracing Lateral Top (+y): True Lateral Bottom (-y): True Strong (z): True

Steel_Beam X_G 1: Strong Flexure Check (extreme rows: max)

Member	Section	Offset ft	Result Case	Demand Mz K-ft	Capacity Mz K-ft	Code Reference	Unity Check	Details
BmX001	W10X45	0.00000	1. 1.4D	-13.57961	205.87500	F2-1	**0.06596**	Lb = 16.33 ft, Cb = 2.32502

Steel_Beam X_G 1: Strong Shear Check (extreme rows: max)

Member	Section	Offset ft	Result Case	Demand Vy K	Capacity Vy K	Code Reference	Unity Check	Details
BmX001	W10X45	0.00000	1. 1.4D	4.84566	106.05000	G2-1	**0.04569**	Shear Area = 3.535 in^2, Cv = 1, h/tw = 22.4571

Member Displacements (extreme rows: max and min)

Member	Dy Min in	Dy Max in	Dz Min in	Dz Max in
BmX001	**-0.02880 (2)**	0.00000 (4)	0.00000 (4)	0.00000 (4)
BmX002	**-0.01122 (2)**	**0.00047 (2)**	0.00000 (4)	0.00000 (4)

SECOND FLOOR - FLOOR BEAM DESIGN: Second Floor - Beam 11

Assumptions:

1. Spans:		**ℓ =**	**16.33**	feet span
		=	195.96	inches
		a =	**14**	feet span
		=	168	inches
2. Floor Joist Span Supported			**9.58**	feet
Floor Joist Span Supported			**9.83**	feet
Floor Joist Span Supported			**0.00**	feet
Floor Joist Span Supported			**0.00**	feet
4. Assume bearing area =	Exterior Wall: (5.5" x 5.5")		**5.50**	inches

Steel Beam - W10 x 112

		AISC Design Values			
A	=	**32.90**	$inches^2$		
$\mathbf{A_{web}}$	=	**7.55**	$inches^2$		
E	=	**29,000,000**	PSI		
S	=	**126**	inches3		
I	=	**716**	inches4		
$\mathbf{Fv_{Allowable}}$	=	**9,500**	PSI		SDOT 6-02.3(17)B For Salvaged Steel
		30,000	PSI	$0.6\mathbf{F_y}$	AISC (Chapter G 16.1-64)
$\mathbf{Fp_{Allowable}}$	=	**22,500**	PSI		SDOT 6-02.3(17)B For Salvaged Steel
Fb	=	**33,000**	PSI		AISC (F3-1)
$\mathbf{t_w}$	=	**0.755**	inches		
d	=	**11.36**	inches		
$\mathbf{t_f}$	=	**1.25**	inches		
$\mathbf{F_y}$	=	**50,000**	PSI		
k	=	**0.94**	inches		
$\mathbf{b_f}$	=	**10.415**	inches		
$\mathbf{W_t}$	=	**120**	lbs/lf		

Reactions:

W	=	Floor Framing DL	12.3	PSF
		Steel Beam DL	4.0	PSF
		Floor LL	40	PSF
		Design Load = DL x 1.2 + LL x 1.4 =	**75.51**	PSF

Reactions: **See Visual Analysis Resilts**

W	=	**366**	plf	9.58 ft span
W	=		plf	9.83 ft span
W	=		plf	0.00 ft span
W	=	**-**	plf	0.00 ft span
$\mathbf{V_{max}}$	=	**15,381**	lbs	
$\mathbf{P_{max}}$	=	**15,381**	lbs	
$\mathbf{M_{max}}$	=	**102,784**	ft-lbs	
$\mathbf{D_{max}}$	=	**0.39**	inches	

Check Shear:

Fvmax	=	$\mathbf{V_{max}} / \mathbf{A_{web}}$	=	**2,037**	lbs
$\mathbf{Fv_{Allowable}}$	=			**9,500.0**	lbs
				Check - O.K.	
$\mathbf{Fv_{Allowable}}$	=	$\mathbf{0.66F_w t_w}(N + 2.5k)$	=	**195,427**	LBS
		ASD Section K1.3		**Check - O.K.**	

Check Bearing-Web Crippling:

F_p	=	P_{max} / A_{web}	=	**2,037** PSI
$F_{p_{Allowable}}$	=			**22,500.0** PSI **Check - O.K.**
$Fv_{Allowable}$	=	$0.66F_w t_w(N + 2.5k)$ ASD Section K1.3	=	**195,427** LBS **Check - O.K.**

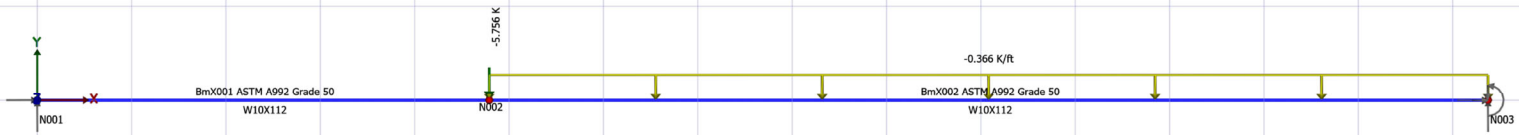
Check Bending:

$S_{Req'd}$	=	$M_{max}(12) / F_b$	=	**37.4** IN^2
$S_{Furnished}$	=			**126.0** IN^2 **Check - O.K.**

Check Deflection:

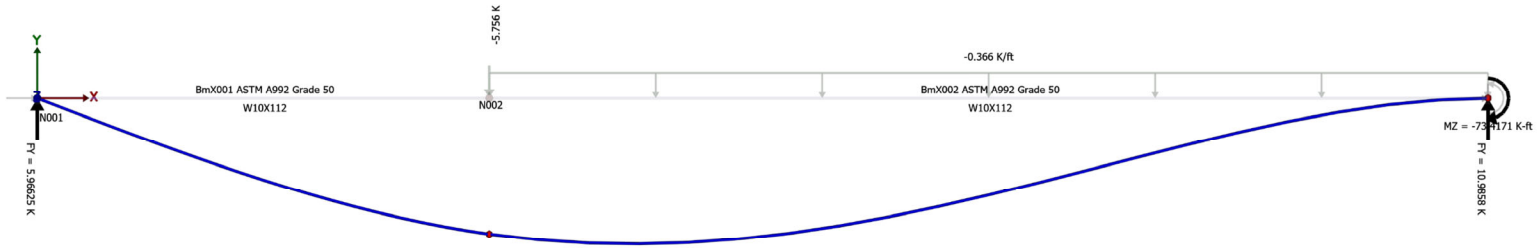
$D_{max.}$	=			**0.39** inches
$D_{Allowable}$	=	L/350	=	**0.56** inches **Check - O.K.**

Second Floor Beam 6.vap
Service Case: D
IES VisualAnalysis 20.00.0001
Saturday, May 4, 2024



Front

Second Floor Beam 6.vap
Result Case: D
IES VisualAnalysis 20.00.0001
Saturday, May 4, 2024



Front

Factored Load Combinations

Name	Code	Effective Equation	Design	Deflection
1. 1.4D	ASCE 7-16 LRFD	1.4D	Strength	Other
2. 1.2D+1.6L+0.5Lr	ASCE 7-16 LRFD	1.2D	Strength	Other
5. 0.9D+W	ASCE 7-16 LRFD	0.9D	Strength	Other

Member Analysis Properties

Name	Member Section	X1 ft	Y1 ft	Z1 ft	X2 ft	Y2 ft	Z2 ft	Elasticity, E Ksi	A in^2	Iz in^4	Iy in^4	Releases	Action
BmX001	W10X112	0.00000	0.00000	0.00000	9.58000	0.00000	0.00000	29000.00000	32.90000	716.00000	236.00000	(none)	Normal
BmX002	W10X112	9.58000	0.00000	0.00000	30.75000	0.00000	0.00000	29000.00000	32.90000	716.00000	236.00000	(none)	Normal

Section Properties

Section	Source	Area in^2	Iz in^4	Iy in^4	J in^4	Alpha, α deg
W10X112	Database Shape	32.90000	716.00000	236.00000	15.10000	0.00000

Member Stresses (extreme rows: max and min)

Member	fa Min Ksi	fa Max Ksi	fby Min Ksi	fby Max Ksi	fbz Min Ksi	fbz Max Ksi	fc comb Min Ksi	fc comb Max Ksi
BmX001	0.00000 (4)	0.00000 (4)	0.00000 (4)	0.00000 (4)	**-6.95618 (2)**	**6.95618 (2)**	**-6.95618 (2)**	**6.95618 (2)**
BmX002	0.00000 (4)	0.00000 (4)	0.00000 (4)	0.00000 (4)	**-9.81902 (2)**	**9.81902 (2)**	**-9.81902 (2)**	**9.81902 (2)**

Node Reactions (extreme rows: max and min)

Node	Result Case	FX K	FY K	FZ K	MX K-ft	MY K-ft	MZ K-ft
N001	5. 0.9D+W	0.00000	**5.36962**	0.00000	0.00000	0.00000	0.00000
N003	1. 1.4D	0.00000	**15.38007**	0.00000	0.00000	0.00000	**-102.78392**
N003	5. 0.9D+W	0.00000	9.88719	0.00000	0.00000	0.00000	**-66.07538**

Steel_Beam X_G 1: Results

Deflections Strong (dy): None Weak (dz): None	Axial Manual Kz: False Kz Sidesway?: False Manual Ky: False Ky Sidesway?: False	Size Constraints Limit Depth?: False Limit Width?: False
Overrides Override Fy?: False Override Cb?: False Override HSS t_des?: False Advanced Torsion: False		
Steel Material: ASTM A992 Grade 50 Specification: AISC 360-16 LRFD Composite Beam?: False Seismic Compactness: Not Ductile Check Constrained Axis FTB?: False Overstrength?: False Live Load Reduction: None Disable Checks?: False Check Level: Each Limit State	Bracing Lateral Top (+y): Unbraced Lateral Bottom (-y): Unbraced Strong (z): Unbraced	Torsional Bracing Lateral Top (+y): True Lateral Bottom (-y): True Strong (z): True

Steel_Beam X_G 1: Strong Flexure Check (extreme rows: max)

Member	Section	Offset ft	Result Case	Demand Mz K-ft	Capacity Mz K-ft	Code Reference	Unity Check	Details
BmX002	W10X112	21.17000	1. 1.4D	-102.78392	551.25000	F2-1	**0.18646**	Lb = 21.17 ft, Cb = 2.10478

Steel_Beam X_G 1: Strong Shear Check (extreme rows: max)

Member	Section	Offset ft	Result Case	Demand Vy K	Capacity Vy K	Code Reference	Unity Check	Details
BmX002	W10X112	21.17000	1. 1.4D	-15.38007	258.21000	G2-1	**0.05956**	Shear Area = 8.607 in^2, Cv = 1, h/tw = 10.4636

Member Displacements (extreme rows: max and min)

Member	Dy Min in	Dy Max in	Dz Min in	Dz Max in
BmX001	**-0.35901 (2)**	0.00000 (4)	0.00000 (4)	0.00000 (4)
BmX002	**-0.38530 (2)**	0.00000 (4)	0.00000 (4)	0.00000 (4)

SECOND FLOOR - FLOOR BEAM DESIGN: Second Floor - Beam 6

Assumptions:

1. Spans:		**ℓ =**	**21.17**	feet span
		=	254.04	inches
		a =	**9.58**	feet span
		=	114.96	inches
2. Floor Joist Span Supported			**21.17**	feet
Floor Joist Span Supported			**6.33**	feet
Floor Joist Span Supported			**14.00**	feet
Floor Joist Span Supported			**16.33**	feet
4. Assume bearing area =	Exterior Wall: (5.5" x 5.5")		**5.50**	inches

Steel Beam - W10 x 112

AISC Design Values

A	=	**32.90**	inches^2		
A_{web}	=	**7.55**	inches^2		
E	=	**29,000,000**	PSI		
S	=	**126**	inches3		
I	=	**716**	inches4		
$Fv_{Allowable}$	=	**9,500**	PSI		SDOT 6-02.3(17)B For Salvaged Steel
		30,000	PSI	$0.6F_y$	AISC (Chapter G 16.1-64)
$Fp_{Allowable}$	=	**22,500**	PSI		SDOT 6-02.3(17)B For Salvaged Steel
Fb	=	**33,000**	PSI		AISC (F3-1)
t_w	=	**0.755**	inches		
d	=	**11.36**	inches		
t_f	=	**1.25**	inches		
F_y	=	**50,000**	PSI		
k	=	**0.94**	inches		
b_f	=	**10.415**	inches		
W_t	=	**120**	lbs/lf		

Reactions:

W	=	Floor Framing DL	12.3	PSF
		Steel Beam DL	3.9	PSF
		Floor LL	40	PSF
		Design Load = DL x 1.2 + LL x 1.4 =	**75.44**	PSF

Reactions: **See Visual Analysis Resilts**

W	=	**366**	plf	21.17 ft span
W	=		plf	6.33 ft span
W	=		plf	14.00 ft span
W	=	**-**	plf	16.33 ft span
V_{max}	=	**15,381**	lbs	
P_{max}	=	**15,381**	lbs	
M_{max}	=	**102,784**	ft-lbs	
D_{max}	=	**0.39**	inches	

Check Shear:

Fvmax	=	**V_{max} / A_{web}**	=	**2,037**	lbs
$Fv_{Allowable}$	=			**9,500.0**	lbs
				Check - O.K.	
$Fv_{Allowable}$	=	**$0.66F_wt_w$(N + 2.5k)**	=	**195,427**	LBS
		ASD Section K1.3		**Check - O.K.**	

Check Bearing-Web Crippling:

F_p	=	P_{max} / A_{web}	=	**2,037** PSI
$F_{p_{Allowable}}$	=			**22,500.0** PSI **Check - O.K.**
$Fv_{Allowable}$	=	$0.66F_w t_w(N + 2.5k)$ ASD Section K1.3	=	**195,427** LBS **Check - O.K.**

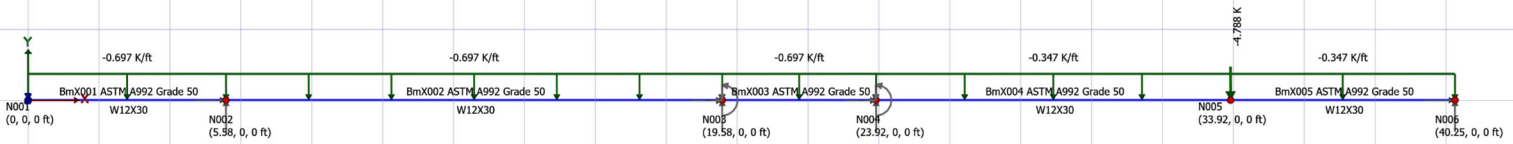
Check Bending:

$S_{Req'd}$	=	$M_{max}(12) / F_b$	=	**37.4** IN^2
$S_{Furnished}$	=			**126.0** IN^2 **Check - O.K.**

Check Deflection:

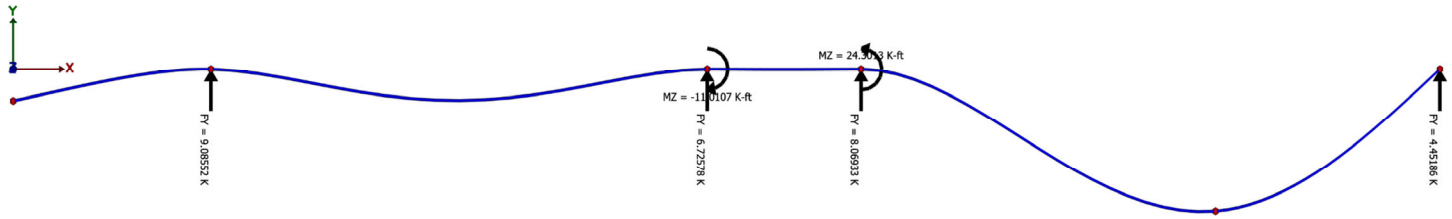
$D_{max.}$	=			**0.39** inches
$D_{Allowable}$	=	L/350	=	**0.73** inches **Check - O.K.**

Second Floor Beam 9.vap
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Front

Second Floor Beam 9.vap
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Front

Section Properties

Section	Source	Area in^2	Iz in^4	Iy in^4	J in^4	Alpha, α deg
W12X30	Database Shape	8.79000	238.00000	20.30000	0.45700	0.00000

Member Stresses (extreme rows: max and min)

Member	fa Min Ksi	fa Max Ksi	fby Min Ksi	fby Max Ksi	fbz Min Ksi	fbz Max Ksi	fc comb Min Ksi	fc comb Max Ksi
BmX003	0.00000 (4)	0.00000 (4)	0.00000 (4)	0.00000 (4)	**-0.49535 (2)**	**0.49535 (2)**	**-0.49535 (2)**	**0.49535 (2)**
BmX004	0.00000 (4)	0.00000 (4)	0.00000 (4)	0.00000 (4)	**-11.04497 (2)**	**11.04497 (2)**	**-11.04497 (2)**	**11.04497 (2)**

Node Reactions (extreme rows: max and min)

Node	Result Case	FX K	FY K	FZ K	MX K-ft	MY K-ft	MZ K-ft
N002	1. 1.4D	0.00000	**12.71972**	0.00000	0.00000	0.00000	0.00000
N003	1. 1.4D	0.00000	9.41610	0.00000	0.00000	0.00000	**-15.41494**
N004	1. 1.4D	0.00000	11.29706	0.00000	0.00000	0.00000	**34.02179**
N006	5. 0.9D+W	0.00000	**4.00668**	0.00000	0.00000	0.00000	0.00000

Steel_Beam X_G 1: Results

Deflections Strong (dy): None Weak (dz): None	Axial Manual Kz: False Kz Sidesway?: False Manual Ky: False Ky Sidesway?: False	Size Constraints Limit Depth?: False Limit Width?: False
Overrides Override Fy?: False Override Cb?: False Override HSS t_des?: False Advanced Torsion: False		

Result Check Information

INFORMATION:
Analysis results could be inaccurate. This report might assist your validation efforts.
The following are heuristic and may or may not indicate problems.
Please check your results carefully.

INFORMATION:
Displacements appear to be within reason.

INFORMATION:
Stresses are less than yield or crushing strength.

Member Displacements

(extreme rows: max and min)

Member	Dy Min in	Dy Max in	Dz Min in	Dz Max in
BmX003	**-0.00024 (2)**	0.00000 (4)	0.00000 (4)	0.00000 (4)
BmX001	-0.02707 (2)	**0.00006 (2)**	0.00000 (4)	0.00000 (4)
BmX004	**-0.12216 (2)**	0.00000 (4)	0.00000 (4)	0.00000 (4)

Factored Load Combinations

Name	Code	Effective Equation	Design	Deflection
1. 1.4D	ASCE 7-16 LRFD	1.4D	Strength	Other
2. 1.2D+1.6L+0.5Lr	ASCE 7-16 LRFD	1.2D	Strength	Other
5. 0.9D+W	ASCE 7-16 LRFD	0.9D	Strength	Other

Member Analysis Properties

Name	Member Section	X1 ft	Y1 ft	Z1 ft	X2 ft	Y2 ft	Z2 ft	Elasticity, E Ksi	A in^2	Iz in^4	Iy in^4	Releases	Action
BmX001	W12X30	0.00000	0.00000	0.00000	5.58000	0.00000	0.00000	29000.00000	8.79000	238.00000	20.30000	(none)	Normal
BmX002	W12X30	5.58000	0.00000	0.00000	19.58000	0.00000	0.00000	29000.00000	8.79000	238.00000	20.30000	(none)	Normal
BmX003	W12X30	19.58000	0.00000	0.00000	23.92000	0.00000	0.00000	29000.00000	8.79000	238.00000	20.30000	(none)	Normal
BmX004	W12X30	23.92000	0.00000	0.00000	33.92000	0.00000	0.00000	29000.00000	8.79000	238.00000	20.30000	(none)	Normal
BmX005	W12X30	33.92000	0.00000	0.00000	40.25000	0.00000	0.00000	29000.00000	8.79000	238.00000	20.30000	(none)	Normal

Steel_Beam X_G 1: Results (continued)

Steel Material: ASTM A992 Grade 50 Specification: AISC 360-16 LRFD Composite Beam?: False Seismic Compactness: Not Ductile Check Constrained Axis FTB?: False Overstrength?: False Live Load Reduction: None Disable Checks?: False Check Level: Each Limit State	Bracing Lateral Top (+y): Unbraced Lateral Bottom (-y): Unbraced Strong (z): Unbraced	Torsional Bracing Lateral Top (+y): True Lateral Bottom (-y): True Strong (z): True

Steel_Beam X_G 1: Strong Flexure Check (extreme rows: max)

Member	Section	Offset ft	Result Case	Demand Mz K-ft	Capacity Mz K-ft	Code Reference	Unity Check	Details
BmX004	W12X30	0.00000	1. 1.4D	-35.61927	161.62500	F2-1	**0.22038**	Lb = 10 ft, Cb = 2.24073

Steel_Beam X_G 1: Strong Shear Check (extreme rows: max)

Member	Section	Offset ft	Result Case	Demand Vy K	Capacity Vy K	Code Reference	Unity Check	Details
BmX004	W12X30	0.00000	1. 1.4D	9.08857	95.94000	G2-1	**0.09473**	Shear Area = 3.198 in^2, Cv = 1, h/tw = 41.6154

SECOND FLOOR - FLOOR BEAM DESIGN: Second Floor - Beam 9

Assumptions:

1. Spans:		**ℓ =**	**16.33**	feet span
		=	195.96	inches
		a =	**5.58**	feet span
		=	66.96	inches
2. Floor Joist Span Supported			**18.00**	feet
Floor Joist Span Supported			**20.50**	feet
Floor Joist Span Supported			**21.17**	feet
Floor Joist Span Supported			**9.58**	feet
4. Assume bearing area =	Exterior Wall: (5.5" x 5.5")		**5.50**	inches

Steel Beam - W10 x 30

		AISC Design Values			
A	=	**8.84**	$inches^2$		
$\mathbf{A_{web}}$	=	**7.55**	$inches^2$		
E	=	**29,000,000**	PSI		
S	=	**32**	inches3		
I	=	**170**	inches4		
$\mathbf{Fv_{Allowable}}$	=	**9,500**	PSI		SDOT 6-02.3(17)B For Salvaged Steel
		30,000	PSI	$0.6\mathbf{F_y}$	AISC (Chapter G 16.1-64)
$\mathbf{Fp_{Allowable}}$	=	**22,500**	PSI		SDOT 6-02.3(17)B For Salvaged Steel
Fb	=	**33,000**	PSI		AISC (F3-1)
$\mathbf{t_w}$	=	**0.3**	inches		
d	=	**10.47**	inches		
$\mathbf{t_f}$	=	**0.51**	inches		
$\mathbf{F_y}$	=	**50,000**	PSI		
k	=	**0.50**	inches		
$\mathbf{b_f}$	=	**5.81**	inches		
$\mathbf{W_t}$	=	**30**	lbs/lf		

Reactions:

W	=	Floor Framing DL	12.3	PSF
		Steel Beam DL	1.4	PSF
		Floor LL	40	PSF
		Design Load = DL x 1.2 + LL x 1.4 =	**72.40**	PSF

Reactions: **See Visual Analysis Resilts**

W	=	**652**	plf	18.00 ft span
W	=	**659**	plf	20.50 ft span
W	=	**351**	plf	21.17 ft span
W	=	**-**	plf	9.58 ft span
$\mathbf{V_{max}}$	=	**12,720**	lbs	
$\mathbf{P_{max}}$	=	**12,720**	lbs	
$\mathbf{M_{max}}$	=	**34,022**	ft-lbs	
$\mathbf{D_{max}}$	=	**0.13**	inches	

Check Shear:

Fvmax	=	$\mathbf{V_{max}} / \mathbf{A_{web}}$	=	**1,685**	lbs
$\mathbf{Fv_{Allowable}}$	=			**9,500.0**	lbs
				Check - O.K.	
$\mathbf{Fv_{Allowable}}$	=	$\mathbf{0.66F_w t_w}(N + 2.5k)$	=	**66,825**	LBS
		ASD Section K1.3		**Check - O.K.**	

Check Bearing-Web Crippling:

F_p	=	P_{max} / A_{web}	=	**1,685** PSI
$F_{p_{Allowable}}$	=			**22,500.0** PSI **Check - O.K.**
$Fv_{Allowable}$	=	$0.66F_w t_w(N + 2.5k)$ ASD Section K1.3	=	**66,825** LBS **Check - O.K.**

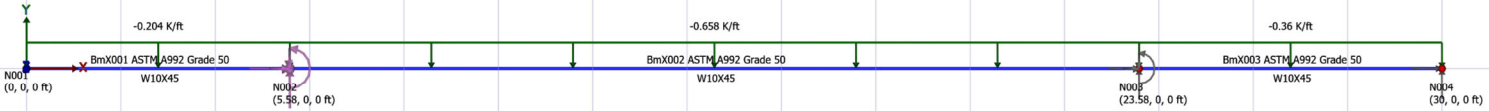
Check Bending:

$S_{Req'd}$	=	$M_{max}(12) / F_b$	=	**12.4** IN^2
$S_{Furnished}$	=			**32.4** IN^2 **Check - O.K.**

Check Deflection:

$D_{max.}$	=			**0.13** inches
$D_{Allowable}$	=	L/350	=	**0.56** inches **Check - O.K.**

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Front

Second Floor Beam 12.vap
Result Case: D
IES VisualAnalysis 20.00.0001
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Front

Factored Load Combinations

Name	Code	Effective Equation	Design	Deflection
1. 1.4D	ASCE 7-16 LRFD	1.4D	Strength	Other
2. 1.2D+1.6L+0.5Lr	ASCE 7-16 LRFD	1.2D	Strength	Other
5. 0.9D+W	ASCE 7-16 LRFD	0.9D	Strength	Other

Member Analysis Properties

Name	Member Section	X1 ft	Y1 ft	Z1 ft	X2 ft	Y2 ft	Z2 ft	Elasticity, E Ksi	A in^2	Iz in^4	Iy in^4	Releases	Action
BmX001	W10X45	0.00000	0.00000	0.00000	5.58000	0.00000	0.00000	29000.00000	13.30000	248.00000	53.40000	(none)	Normal
BmX002	W10X45	5.58000	0.00000	0.00000	23.58000	0.00000	0.00000	29000.00000	13.30000	248.00000	53.40000	(none)	Normal
BmX003	W10X45	23.58000	0.00000	0.00000	30.00000	0.00000	0.00000	29000.00000	13.30000	248.00000	53.40000	(none)	Normal

Section Properties

Section	Source	Area in^2	Iz in^4	Iy in^4	J in^4	Alpha, α deg
W10X45	Database Shape	13.30000	248.00000	53.40000	1.51000	0.00000

Member Stresses (extreme rows: max and min)

Member	fa Min Ksi	fa Max Ksi	fby Min Ksi	fby Max Ksi	fbz Min Ksi	fbz Max Ksi	fc comb Min Ksi	fc comb Max Ksi
BmX002	0.00000 (4)	0.00000 (4)	0.00000 (4)	0.00000 (4)	**-6.49635 (2)**	**6.49635 (2)**	**-6.49635 (2)**	**6.49635 (2)**
BmX003	0.00000 (4)	0.00000 (4)	0.00000 (4)	0.00000 (4)	**-0.71439 (2)**	**0.71439 (2)**	**-0.71439 (2)**	**0.71439 (2)**

Node Reactions (extreme rows: max and min)

Node	Result Case	FX K	FY K	FZ K	MX K-ft	MY K-ft	MZ K-ft
N002	1. 1.4D	0.00000	10.80965	0.00000	0.00000	0.00000	**21.15155**
N003	1. 1.4D	0.00000	**11.13883**	0.00000	0.00000	0.00000	**-23.66217**
N004	5. 0.9D+W	0.00000	**0.87824**	0.00000	0.00000	0.00000	0.00000

Steel_Beam X_G 1: Results

Deflections Strong (dy): None Weak (dz): None	Axial Manual Kz: False Kz Sidesway?: False Manual Ky: False Ky Sidesway?: False	Size Constraints Limit Depth?: False Limit Width?: False
Overrides Override Fy?: False Override Cb?: False Override HSS t_des?: False Advanced Torsion: False		
Steel Material: ASTM A992 Grade 50 Specification: AISC 360-16 LRFD Composite Beam?: False Seismic Compactness: Not Ductile Check Constrained Axis FTB?: False Overstrength?: False Live Load Reduction: None Disable Checks?: False Check Level: Each Limit State	Bracing Lateral Top (+y): Unbraced Lateral Bottom (-y): Unbraced Strong (z): Unbraced	Torsional Bracing Lateral Top (+y): True Lateral Bottom (-y): True Strong (z): True

Steel_Beam X_G 1: Strong Flexure Check (extreme rows: max)

Member	Section	Offset ft	Result Case	Demand Mz K-ft	Capacity Mz K-ft	Code Reference	Unity Check	Details
BmX002	W10X45	0.00000	1. 1.4D	-26.58574	205.87500	F2-1	**0.12914**	Lb = 18 ft, Cb = 2.38259

Steel_Beam X_G 1: Strong Shear Check (extreme rows: max)

Member	Section	Offset ft	Result Case	Demand Vy K	Capacity Vy K	Code Reference	Unity Check	Details
BmX002	W10X45	0.00000	1. 1.4D	8.86191	106.05000	G2-1	**0.08356**	Shear Area = 3.535 in^2, Cv = 1, h/tw = 22.4571

Member Displacements (extreme rows: max and min)

Member	Dy Min in	Dy Max in	Dz Min in	Dz Max in
BmX002	**-0.06468 (2)**	0.00000 (4)	0.00000 (4)	0.00000 (4)
BmX003	**-0.00125 (2)**	0.00000 (4)	0.00000 (4)	0.00000 (4)

Node Results

(extreme rows: max and min)

Node	Result Case	DX in	DY in	DZ in	FX K	FY K	FZ K	MX K-ft	MY K-ft	MZ K-ft
N001	1. 1.4D	0.00000	**-0.01016**	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
N001	5. 0.9D+W	0.00000	**-0.00653**	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
N002	1. 1.4D	0.00000	0.00000	0.00000	0.00000	10.80965	0.00000	0.00000	0.00000	**21.15155**
N003	1. 1.4D	0.00000	0.00000	0.00000	0.00000	**11.13883**	0.00000	0.00000	0.00000	**-23.66217**
N004	5. 0.9D+W	0.00000	0.00000	0.00000	0.00000	**0.87824**	0.00000	0.00000	0.00000	0.00000

SECOND FLOOR - FLOOR BEAM DESIGN: Second Floor - Beam 12

Assumptions:

1. Spans:		**ℓ =**	**18.00**	feet span
		=	216	inches
		a =	**5.58**	feet span
		=	66.96	inches
2. Floor Joist Span Supported			**5.58**	feet
Floor Joist Span Supported			**18.00**	feet
Floor Joist Span Supported			**9.83**	feet
Floor Joist Span Supported			**0.00**	feet
4. Assume bearing area =	Exterior Wall: (5.5" x 5.5")		**5.50**	inches

Steel Beam - W10 x 45

AISC Design Values

A	=	**13.30**	inches^2		
A_{web}	=	**7.55**	inches^2		
E	=	**29,000,000**	PSI		
S	=	**49**	inches3		
I	=	**248**	inches4		
$Fv_{Allowable}$	=	**9,500**	PSI		SDOT 6-02.3(17)B For Salvaged Steel
		30,000	PSI	$0.6F_y$	AISC (Chapter G 16.1-64)
$Fp_{Allowable}$	=	**22,500**	PSI		SDOT 6-02.3(17)B For Salvaged Steel
Fb	=	**33,000**	PSI		AISC (F3-1)
t_w	=	**0.35**	inches		
d	=	**10.1**	inches		
t_f	=	**0.6**	inches		
F_y	=	**50,000**	PSI		
k	=	**0.69**	inches		
b_f	=	**8.02**	inches		
W_t	=	**45**	lbs/lf		

Reactions:

W	=	Floor Framing DL	12.3	PSF
		Steel Beam DL	1.9	PSF
		Floor LL	40	PSF
		Design Load = DL x 1.2 + LL x 1.4 =	**73.05**	PSF

Reactions: **See Visual Analysis Resilts**

W	=	**204**	plf	5.58 ft span
W	=	**658**	plf	18.00 ft span
W	=	**366**	plf	9.83 ft span
W	=	**-**	plf	0.00 ft span
V_{max}	=	**12,911**	lbs	
P_{max}	=	**12,911**	lbs	
M_{max}	=	**34,238**	ft-lbs	
D_{max}	=	**0.12**	inches	

Check Shear:

Fvmax	=	**V_{max} / A_{web}**	=	**1,710**	lbs
$Fv_{Allowable}$	=			**9,500.0**	lbs
				Check - O.K.	
$Fv_{Allowable}$	=	**0.66$F_w t_w$(N + 2.5k)**	=	**83,377**	LBS
		ASD Section K1.3		**Check - O.K.**	

Check Bearing-Web Crippling:

F_p	=	P_{max} / A_{web}	=	**1,710** PSI
$F_{p_{Allowable}}$	=			**22,500.0** PSI **Check - O.K.**
$Fv_{Allowable}$	=	$0.66F_w t_w(N + 2.5k)$ ASD Section K1.3	=	**83,377** LBS **Check - O.K.**

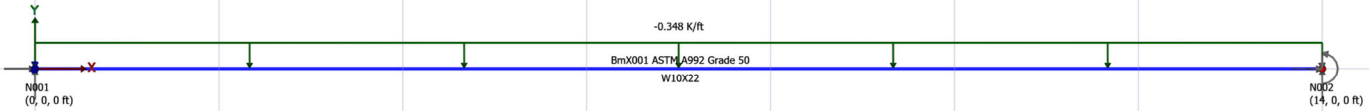
Check Bending:

$S_{Req'd}$	=	$M_{max}(12) / F_b$	=	**12.5** IN^2
$S_{Furnished}$	=			**49.1** IN^2 **Check - O.K.**

Check Deflection:

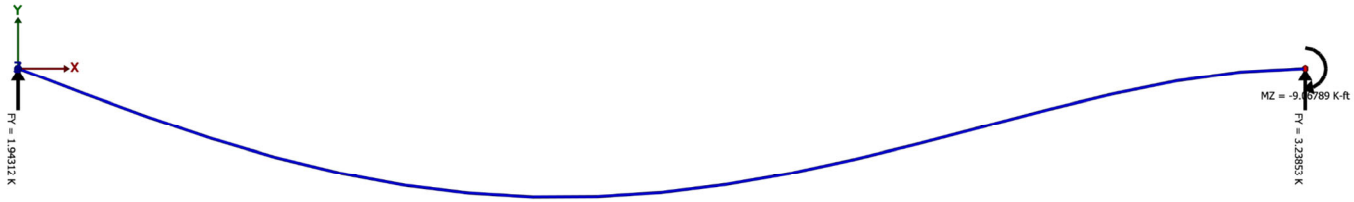
$D_{max.}$	=			**0.12** inches
$D_{Allowable}$	=	L/350	=	**0.62** inches **Check - O.K.**

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Result Case: D
IES VisualAnalysis 20.00.0001
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Front

Factored Load Combinations

Name	Code	Effective Equation	Design	Deflection
1. 1.4D	ASCE 7-16 LRFD	1.4D	Strength	Other
2. 1.2D+1.6L+0.5Lr	ASCE 7-16 LRFD	1.2D	Strength	Other
5. 0.9D+W	ASCE 7-16 LRFD	0.9D	Strength	Other

Member Analysis Properties

Name	Member Section	X1 ft	Y1 ft	Z1 ft	X2 ft	Y2 ft	Z2 ft	Elasticity, E Ksi	A in^2	Iz in^4	Iy in^4	Releases	Action
BmX001	W10X22	0.00000	0.00000	0.00000	14.00000	0.00000	0.00000	29000.00000	6.49000	118.00000	11.40000	(none)	Normal

Section Properties

Section	Source	Area in^2	Iz in^4	Iy in^4	J in^4	Alpha, α deg
W10X22	Database Shape	6.49000	118.00000	11.40000	0.23900	0.00000

Member Stresses (extreme rows: max and min)

Member	fa Min Ksi	fa Max Ksi	fby Min Ksi	fby Max Ksi	fbz Min Ksi	fbz Max Ksi	fc comb Min Ksi	fc comb Max Ksi
BmX001	0.00000 (4)	0.00000 (4)	0.00000 (4)	0.00000 (4)	**-6.58421 (2)**	**6.58421 (2)**	**-6.58421 (2)**	**6.58421 (2)**

Node Reactions (extreme rows: max and min)

Node	Result Case	FX K	FY K	FZ K	MX K-ft	MY K-ft	MZ K-ft
N001	5. 0.9D+W	0.00000	**1.74881**	0.00000	0.00000	0.00000	0.00000
N002	1. 1.4D	0.00000	**4.53394**	0.00000	0.00000	0.00000	**-12.69504**
N002	5. 0.9D+W	0.00000	2.91468	0.00000	0.00000	0.00000	**-8.16110**

Steel_Beam X_G 1: Results

Deflections Strong (dy): None Weak (dz): None	Axial Manual Kz: False Kz Sidesway?: False Manual Ky: False Ky Sidesway?: False	Size Constraints Limit Depth?: False Limit Width?: False

Steel_Beam X_G 1: Results (continued)

Overrides Override Fy?: False Override Cb?: False Override HSS t_des?: False Advanced Torsion: False		
Steel Material: ASTM A992 Grade 50 Specification: AISC 360-16 LRFD Composite Beam?: False Seismic Compactness: Not Ductile Check Constrained Axis FTB?: False Overstrength?: False Live Load Reduction: None Disable Checks?: False Check Level: Each Limit State	Bracing Lateral Top (+y): Unbraced Lateral Bottom (-y): Unbraced Strong (z): Unbraced	Torsional Bracing Lateral Top (+y): True Lateral Bottom (-y): True Strong (z): True

Steel_Beam X_G 1: Strong Flexure Check (extreme rows: max)

Member	Section	Offset ft	Result Case	Demand Mz K-ft	Capacity Mz K-ft	Code Reference	Unity Check	Details
BmX001	W10X22	14.00000	1. 1.4D	-12.69504	97.50000	F2-1	**0.13021**	Lb = 14 ft, Cb = 2.08333

Steel_Beam X_G 1: Strong Shear Check (extreme rows: max)

Member	Section	Offset ft	Result Case	Demand Vy K	Capacity Vy K	Code Reference	Unity Check	Details
BmX001	W10X22	14.00000	1. 1.4D	-4.53394	73.44000	G2-1	**0.06174**	Shear Area = 2.448 in^2, Cv = 1, h/tw = 37

Member Displacements (extreme rows: max and min)

Member	Dy Min in	Dy Max in	Dz Min in	Dz Max in
BmX001	**-0.05444 (2)**	0.00000 (4)	0.00000 (4)	0.00000 (4)

SECOND FLOOR - FLOOR BEAM DESIGN: Second Floor - Beam 8

Assumptions:

1. Spans:		**ℓ =**	**14.00**	feet span
		=	168	inches
		a =	**0**	feet span
		=	0	inches
2. Floor Joist Span Supported			**9.58**	feet
Floor Joist Span Supported			**0.00**	feet
Floor Joist Span Supported			**0.00**	feet
Floor Joist Span Supported			**0.00**	feet
4. Assume bearing area =	Exterior Wall: (5.5" x 5.5")		**5.50**	inches

Steel Beam - W10 x 22

		AISC Design Values			
A	=	**6.49**	$inches^2$		
$\mathbf{A_{web}}$	=	**7.55**	$inches^2$		
E	=	**29,000,000**	PSI		
S	=	**23**	inches3		
I	=	**118**	inches4		
$\mathbf{Fv_{Allowable}}$	=	**9,500**	PSI		SDOT 6-02.3(17)B For Salvaged Steel
		30,000	PSI	$0.6\mathbf{F_y}$	AISC (Chapter G 16.1-64)
$\mathbf{Fp_{Allowable}}$	=	**22,500**	PSI		SDOT 6-02.3(17)B For Salvaged Steel
Fb	=	**33,000**	PSI		AISC (F3-1)
$\mathbf{t_w}$	=	**0.24**	inches		
d	=	**10.17**	inches		
$\mathbf{t_f}$	=	**0.36**	inches		
$\mathbf{F_y}$	=	**50,000**	PSI		
k	=	**0.50**	inches		
$\mathbf{b_f}$	=	**5.57**	inches		
$\mathbf{W_t}$	=	**22**	lbs/lf		

Reactions:

W	=	Floor Framing DL	12.3	PSF
		Steel Beam DL	1.6	PSF
		Floor LL	40	PSF
		Design Load = DL x 1.2 + LL x 1.4 =	**72.65**	PSF

Reactions: **See Visual Analysis Resilts**

W	=	**348**	plf	9.58 ft span
W	=	**-**	plf	0.00 ft span
W	=	**-**	plf	10.00 ft span
W	=	**-**	plf	0.00 ft span
$\mathbf{V_{max}}$	=	**4,534**	lbs	
$\mathbf{P_{max}}$	=	**4,534**	lbs	
$\mathbf{M_{max}}$	=	**12,696**	ft-lbs	
$\mathbf{D_{max}}$	=	**0.13**	inches	

Check Shear:

Fvmax	=	$\mathbf{V_{max}} / \mathbf{A_{web}}$	=	**601**	lbs
$\mathbf{Fv_{Allowable}}$	=			**9,500.0**	lbs
				Check - O.K.	
$\mathbf{Fv_{Allowable}}$	=	$\mathbf{0.66F_wt_w}(N + 2.5k)$	=	**53,460**	LBS
		ASD Section K1.3		**Check - O.K.**	

Check Bearing-Web Crippling:

F_p	=	P_{max} / A_{web}	=	**601** PSI
$F_{p_{Allowable}}$	=			**22,500.0** PSI **Check - O.K.**
$Fv_{Allowable}$	=	$0.66F_w t_w(N + 2.5k)$ ASD Section K1.3	=	**53,460** LBS **Check - O.K.**

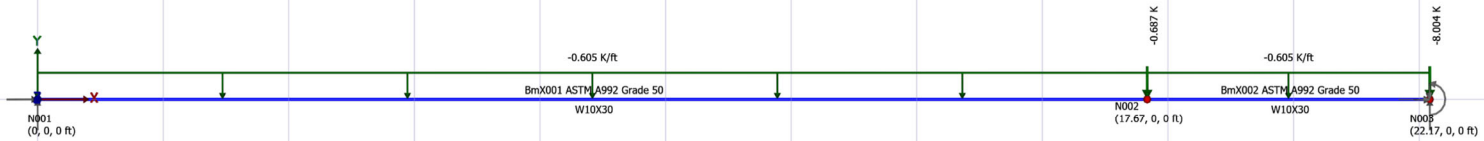
Check Bending:

$S_{Req'd}$	=	$M_{max}(12) / F_b$	=	**4.6** IN^2
$S_{Furnished}$	=			**23.2** IN^2 **Check - O.K.**

Check Deflection:

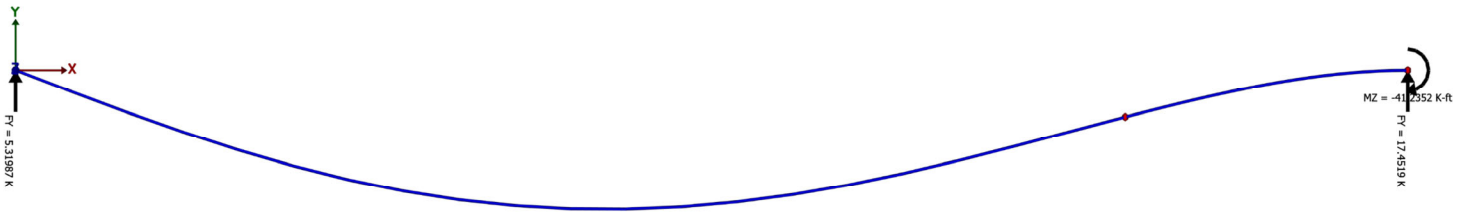
$D_{max.}$	=			**0.13** inches
$D_{Allowable}$	=	L/350	=	**0.48** inches **Check - O.K.**

Second Floor Beam 7.vap
Service Case: D
IES VisualAnalysis 20.00.0001
Saturday, May 4, 2024



Front

Second Floor Beam 7.vap
Result Case: D
IES VisualAnalysis 20.00.0001
Saturday, May 4, 2024



Front

Model Check Information

No errors were found in your model.

Factored Load Combinations

Name	Code	Effective Equation	Design	Deflection
1. 1.4D	ASCE 7-16 LRFD	1.4D	Strength	Other
2. 1.2D+1.6L+0.5Lr	ASCE 7-16 LRFD	1.2D	Strength	Other
5. 0.9D+W	ASCE 7-16 LRFD	0.9D	Strength	Other

Member Analysis Properties

Name	Member Section	X1 ft	Y1 ft	Z1 ft	X2 ft	Y2 ft	Z2 ft	Elasticity, E Ksi	A in^2	Iz in^4	Iy in^4	Releases	Action
BmX001	W10X30	0.00000	0.00000	0.00000	17.67000	0.00000	0.00000	29000.00000	8.84000	170.00000	16.70000	(none)	Normal
BmX002	W10X30	17.67000	0.00000	0.00000	22.17000	0.00000	0.00000	29000.00000	8.84000	170.00000	16.70000	(none)	Normal

Section Properties

Section	Source	Area in^2	Iz in^4	Iy in^4	J in^4	Alpha, α deg
W10X30	Database Shape	8.84000	170.00000	16.70000	0.62200	0.00000

Member Stresses (extreme rows: max and min)

Member	fa Min Ksi	fa Max Ksi	fby Min Ksi	fby Max Ksi	fbz Min Ksi	fbz Max Ksi	fc comb Min Ksi	fc comb Max Ksi
BmX001	0.00000 (4)	0.00000 (4)	0.00000 (4)	0.00000 (4)	**-11.55745 (2)**	**11.55745 (2)**	**-11.55745 (2)**	**11.55745 (2)**
BmX002	0.00000 (4)	0.00000 (4)	0.00000 (4)	0.00000 (4)	**-21.39381 (2)**	**21.39381 (2)**	**-21.39381 (2)**	**21.39381 (2)**

Node Reactions (extreme rows: max and min)

Node	Result Case	FX K	FY K	FZ K	MX K-ft	MY K-ft	MZ K-ft
N001	5. 0.9D+W	0.00000	**4.78788**	0.00000	0.00000	0.00000	0.00000
N003	1. 1.4D	0.00000	**24.43265**	0.00000	0.00000	0.00000	**-57.72933**
N003	5. 0.9D+W	0.00000	15.70670	0.00000	0.00000	0.00000	**-37.11171**

Steel_Beam X_G 1: Results

Deflections Strong (dy): None Weak (dz): None	Axial Manual Kz: False Kz Sidesway?: False Manual Ky: False Ky Sidesway?: False	Size Constraints Limit Depth?: False Limit Width?: False
Overrides Override Fy?: False Override Cb?: False Override HSS t_des?: False Advanced Torsion: False		
Steel Material: ASTM A992 Grade 50 Specification: AISC 360-16 LRFD Composite Beam?: False Seismic Compactness: Not Ductile Check Constrained Axis FTB?: False Overstrength?: False Live Load Reduction: None Disable Checks?: False Check Level: Each Limit State	Bracing Lateral Top (+y): Unbraced Lateral Bottom (-y): Unbraced Strong (z): Unbraced	Torsional Bracing Lateral Top (+y): True Lateral Bottom (-y): True Strong (z): True

Steel_Beam X_G 1: Strong Flexure Check (extreme rows: max)

Member	Section	Offset ft	Result Case	Demand Mz K-ft	Capacity Mz K-ft	Code Reference	Unity Check	Details
BmX002	W10X30	4.50000	1. 1.4D	-57.72933	137.25000	F2-1	**0.42061**	Lb = 4.5 ft, Cb = 1.60396

Steel_Beam X_G 1: Strong Shear Check (extreme rows: max)

Member	Section	Offset ft	Result Case	Demand Vy K	Capacity Vy K	Code Reference	Unity Check	Details
BmX002	W10X30	4.50000	1. 1.4D	-13.22705	94.50000	G2-1	**0.13997**	Shear Area = 3.15 in^2, Cv = 1, h/tw = 29.6

Member Displacements (extreme rows: max and min)

Member	Dy Min in	Dy Max in	Dz Min in	Dz Max in
BmX001	**-0.41775 (2)**	0.00000 (4)	0.00000 (4)	0.00000 (4)
BmX002	**-0.13979 (2)**	0.00000 (4)	0.00000 (4)	0.00000 (4)

SECOND FLOOR - FLOOR BEAM DESIGN: Second Floor - Beam 7

Assumptions:

1. Spans:		**ℓ =**	**17.67**	feet span
		=	212.04	inches
		a =	**3.5**	feet span
		=	42	inches
2. Floor Joist Span Supported			**3.50**	feet
Floor Joist Span Supported			**0.00**	feet
Floor Joist Span Supported			**0.00**	feet
Floor Joist Span Supported			**0.00**	feet
4. Assume bearing area =	Exterior Wall: (5.5" x 5.5")		**5.50**	inches

Steel Beam - W10 x 30

AISC Design Values

A	=	**8.84**	inches^2		
A_{web}	=	**7.55**	inches^2		
E	=	**29,000,000**	PSI		
S	=	**32**	inches3		
I	=	**170**	inches4		
$Fv_{Allowable}$	=	**9,500**	PSI		SDOT 6-02.3(17)B For Salvaged Steel
		30,000	PSI	$0.6F_y$	AISC (Chapter G 16.1-64)
$Fp_{Allowable}$	=	**22,500**	PSI		SDOT 6-02.3(17)B For Salvaged Steel
Fb	=	**33,000**	PSI		AISC (F3-1)
t_w	=	**0.3**	inches		
d	=	**10.47**	inches		
t_f	=	**0.51**	inches		
F_y	=	**50,000**	PSI		
k	=	**0.50**	inches		
b_f	=	**5.81**	inches		
W_t	=	**30**	lbs/lf		

Reactions:

W	=	Floor Framing DL	12.3	PSF
		Steel Beam DL	1.4	PSF
		Floor LL	40	PSF
		Design Load = DL x 1.2 + LL x 1.4 =	**72.46**	PSF

Reactions: **See Visual Analysis Resilts**

W	=	**127**	plf	3.50 ft span
W	=	**605**	plf	0.00 ft span
W	=	**362**	plf	10.00 ft span
W	=	**-**	plf	0.00 ft span
V_{max}	=	**24,433**	lbs	
P_{max}	=	**24,433**	lbs	
M_{max}	=	**57,730**	ft-lbs	
D_{max}	=	**0.14**	inches	

Check Shear:

Fvmax	=	**V_{max} / A_{web}**	=	**3,236**	lbs
$Fv_{Allowable}$	=			**9,500.0**	lbs
				Check - O.K.	
$Fv_{Allowable}$	=	**$0.66F_w t_w$(N + 2.5k)**	=	**66,825**	LBS
		ASD Section K1.3		**Check - O.K.**	

Check Bearing-Web Crippling:

F_p	=	P_{max} / A_{web}	=	**3,236** PSI
$F_{p_{Allowable}}$	=			**22,500.0** PSI **Check - O.K.**
$Fv_{Allowable}$	=	$0.66F_w t_w(N + 2.5k)$ ASD Section K1.3	=	**66,825** LBS **Check - O.K.**

Check Bending:

$S_{Req'd}$	=	$M_{max}(12) / F_b$	=	**21.0** IN^2
$S_{Furnished}$	=			**32.4** IN^2 **Check - O.K.**

Check Deflection:

$D_{max.}$	=			**0.14** inches
$D_{Allowable}$	=	L/350	=	**0.61** inches **Check - O.K.**

TIMBER FLOOR JOIST DESIGN: Second Level 10'-0" Span

Assumptions:

1. 2" x 8 Timber Floor Joists H.F. No. 2 or Better
2. Maximum span: **10** feet **(MAX)** = 120 inches
3. Support a simple span.
   Therefore: **L** = 120 inches
4. Floor Joists Spaced @ **1.00** feet o.c. = 12 inches
5. Minimum Bearing Area: 1 5" x 5.5" = **8.25** $inches^2$

2" x 8" Timber Properties:

Reference: ANSI National Design Specifications For Wood Construction

		ANSI Design Values
A	=	**10.875** $inches^2$
E	=	**1,700,000** PSI
I	=	**47.6** $inches^4$
S	=	**13.1** $inches^3$
$Fv_{Allowable}$	=	**145** PSI
$Fp_{Allowable}$	=	**405** PSI
Fb	=	**1300** PSI

Reactions:

W = Floor Load x Joist Spacing = 97.00 PLF

Where **W** =

Floor D.L.	=	10.5 PSF
Floor L.L.	=	60 PSF
Design Load = DL x 1.2 + LL x 1.4 =		96.6
Used in Calculations:		97 PSF

V_{max}	=	0.5**WL**	=	**485** lbs
P_{max}	=	0.5**WL**	=	**485** lbs
M_{max}	=	$(WL^2)/8$	=	**1,213** ft-lbs
D_{max}	=	$0.013WL^4 / EI$	=	**0.27** inches

Check Shear:

Fv	=	$V_{max}(1.5) / A$	=	**66.90** PSI
$Fv_{Allowable}$	=			**145** PSI

Check - O.K.

Check Bearing:

Fp	=	P_{max} / A	=	**58.79** PSI
$Fp_{Allowable}$	=			**405** PSI

Check - O.K.

Check Bending:

$S_{Req'd}$	=	$M_{max}(12) / Fb$	=	**11.19** $inches^3$
$S_{Furnished}$	=			**13.1** $inches^3$

Check - O.K.

Check Deflection:

$D_{max.}$	=			**0.2691** inches
$D_{Allowable}$	=	L/350	=	**0.3429** inches

Check - O.K.

LTT/HTT

Tension Ties

Tension ties offer a solution for resisting tension loads that are fastened with nails or Strong-Drive® SD Connector screws. The new LTTP2 light tension tie, designed for wood joist attachments to concrete or masonry walls, features two separate nailing patterns: obround holes spaced 3" apart for I-joist purlins and square holes spaced to accommodate the narrow face of 2x solid-sawn purlins. LTTP2 may also be installed vertically on the wide face of a minimum 2x4 stud for holdown application. It features an extruded anchor bolt hole to accommodate ¾", ⅝" and ½" bolt diameters.

The LTTI31 is designed for wood chord open-web truss attachments to concrete or masonry walls.

The HTT4 and HTT5 tension ties feature an optimized nailing pattern which results in better performance with less deflection. HTT5KT is sold as a kit with the holdown, bearing plate washer and Strong-Drive SD Connector screws.

The HTT5-¾ is designed to use a ¾"-diameter anchor bolt.

When using LTT or HTT tension ties with unreinforced concrete masonry, ¾" post-installed anchor bolts are commonly used.

Material: See table

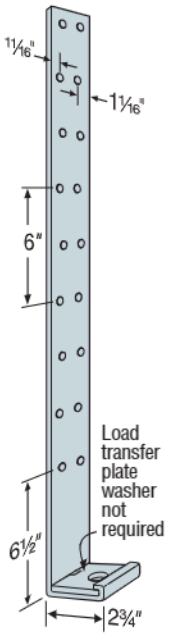
Finish: Galvanized. May be ordered HDG.

Installation:

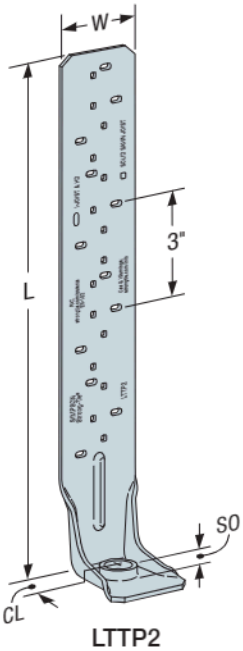
- See Holdown and Tension Tie General Notes on pp. 51–52.
- LTTP2 — one standard cut-washer is required when using ½" and ⅝" anchor bolts; and no additional washer is required for ¾" anchor bolts.
- LTTP2 — For installations on narrow edge of solid sawn (2x, 3x) joists use (15) square holes; for all other installations use (12) obround holes.
- For tension ties installed over wood structural panel sheathing, use a 2½"-long fastener minimum.
- For information about marriage strap at panelized roof applications, see **strongtie.com**.
- HTT5-KT requires BP 5/8-2 bearing plate and #10 x 2½" SD Strong-Drive screws (included in kit).

Codes: Codes: See p. 13 for Code Reference Key Chart

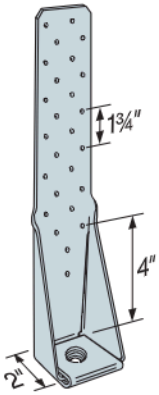
Web Applications: Visit **app.strongtie.com/pfd** to access our Post-to-Foundation Designer web application.



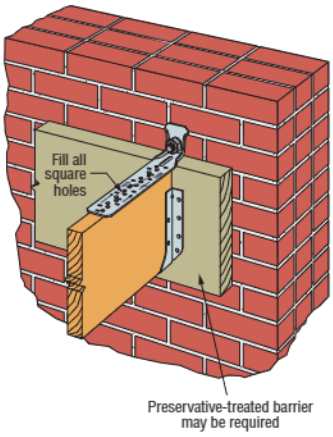
LTTI31



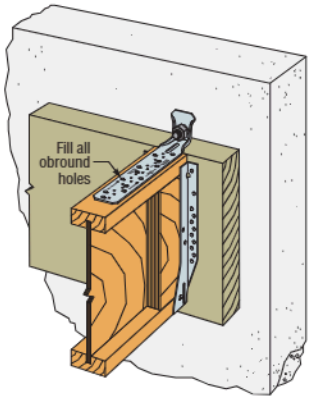
LTTP2
US Patent Pending



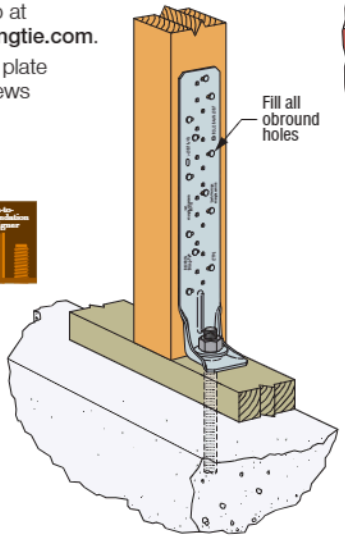
HTT5
(HTT4 similar)



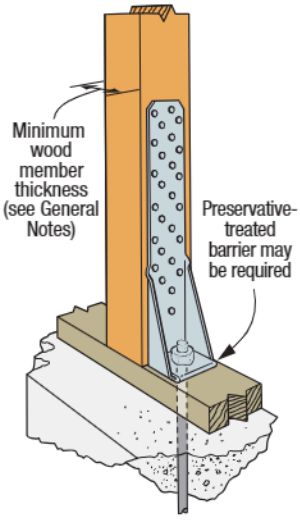
Typical LTTP2 Installation for Solid Sawn Joist



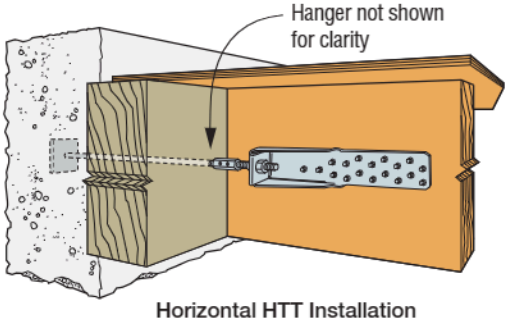
Typical LTTP2 Installation for I-joist



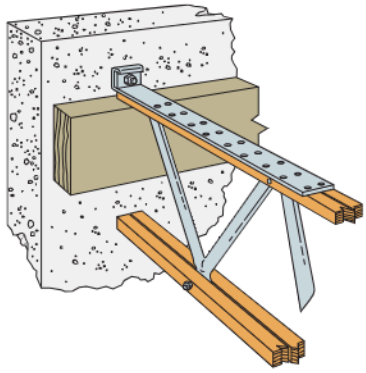
Typical LTTP2 Installation for Holdown Application



Vertical HTT5 Installation
(HTT4 similar)



Horizontal HTT Installation



Horizontal LTTI31 Installation

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Tension Ties (cont.)

These products are available with additional corrosion protection. For more information, see p. 16.

SD Many of these products are approved for installation with Strong-Drive® SD Connector screws. See pp. 362–366 for more information.

Model No.	Ga.	Dimensions (in.)			SO (in.)	Fasteners (in.)		Minimum Wood Member Size (in.)	Allowable Tension Loads (160)		Deflection at Highest Allowable Load (in.)	Code Ref.
		W	L	CL		Anchor Rod Diameter	Wood Fasteners		DF/SP	SPF/HF		
LTTP2	10	2⁹⁄₁₆	14 15⁄16	1⅛	7⁄16	½, ⅝, ¾	(15) 0.148 x 2½	1½ x 3½ (narrow edge)[4,5]	1,845	1,695	0.104	IBC®, FL, LA
						½	(12) 0.148 x 1½	1½ x 3½	1,680[6]	1,545[6]	0.138	
						⅝, ¾			2,135	1,965	0.112	
						½	(12) #9 x 1½" SD	1½ x 3½	2,320	1,970	0.112	
						⅝, ¾			2,570	2,045	0.136	
						½, ⅝, ¾	(12) 0.148 x 2½	3 x 3½	2,275	2,230	0.128	
LTTI31	18	3¾	31	1⅜	¼	⅝	(18) 0.148 x 1½	3 x 3½	1,350	1,160	0.193	
HTT4	11	2½	12⅜	1 5⁄16	7⁄16	⅝	(18) 0.148 x 1½	1½ x 3½	3,000	2,580	0.090	—
							(18) 0.148 x 1½	3 x 3½	3,610	3,105	0.086	IBC, FL, LA
							(18) 0.162 x 2½	3 x 3½	4,235	3,640	0.123	
							(18) #10 x 1½" SD	1½ x 5½	4,455	3,830	0.112	—
							(18) #10 x 1½" SD	3 x 3½	4,455	3,830	0.112	
HTT5	11	2½	16	1⅜	7⁄16	⅝	(26) 0.148 x 1½	3 x 3½	4,350	3,740	0.120	IBC, FL, LA
							(26) 0.148 x 3	3 x 3½	4,670	4,015	0.116	
							(26) 0.162 x 2½	3 x 3½	5,090	4,375	0.135	
							(26) #10 x 1½" SD	1½ x 5½	4,555	3,915	0.114	—
HTT5KT	11	2½	16	1⅜	7⁄16	⅝	(26) #10 x 2½" SD	3 x 3½	5,445	5,360	0.103	
HTT5-3/4	11	2½	16	1⅜	7⁄16	¾	(26) 0.148 x 1½	1½ x 5½	4,065	3,495	0.103	IBC, FL
							(26) 0.162 x 2½	3 x 3½	5,090	4,375	0.121	
							(26) #10 x 1½" SD	1½ x 7¼	4,830	4,155	0.100	

1. LTTI31 installed flush with concrete or masonry has an allowable load of 2,285 lb.
2. Allowable load for HTT5 with a BP5/8-2 bearing-plate washer installed in the seat of the holdown is 5,295 lb. for DF/SP and 4,555 lb. for SPF/HF.
3. For LTTP2, standard cut washer is required when using ½" and ⅝" anchor rods.
4. For (15) nail installations on narrow edge of 2x4 (minimum) joist, LTTP2 installed flush with concrete or masonry has an allowable load of 2,560 lb. for DF/SP and 2,355 lb. for SPF/HF.
5. LTTP2 installed with (15) #9 x 1½" SD screws on narrow edge of 2x joist has an allowable load of 2,105 lb. for DF/SP and 1,935 lb. for SPF/HF.
6. For (12) nail installations on I-joist or wide face of 2x member, LTTP2 installed flush with concrete or masonry has an allowable load of 1,950 lb. for DF/SP and 1,795 lb. for SPF/HF.
7. **Fasteners:** Nail dimensions are listed diameter by length. SD screws are Simpson Strong-Tie Strong-Drive SD Connector screws. See pp. 23–24 for fastener information.

Table 1 — Anchorage Selection Guide for Holdowns Attached to DF/SP Lumber

Holdown on DF/SP Lumber	Stemwall				Slab on Grade			
	Stemwall Width (in.)	Wind and Seismic Design Category A&B		Seismic Design Category C–F		Wind and Seismic Design Category A&B		Seismic Design Category C–F	
		Midwall/Corner	End Wall	Midwall/Corner	End Wall	Midwall/Corner	Garage Curb	Midwall/Corner	Garage Curb
HDU2	6	SSTB16		SSTB24		SSTB16		SSTB16	SSTB20* (2,960)
HDU4	6	SB5/8X24		SB5/8X24		SSTB16	SB5/8X24	SSTB20	SB5/8X24
HDU5	6	SB5/8X24		SB5/8X24		SSTB20	SB5/8X24	SSTB24	SB5/8X24
HDU8									
HDQ8									
HDU11									
HHDQ11									
HDU14									
HHDQ14									
LTTP2									
LTTI31									
HTT4									
HTT5									
HD3B									
HD5B									
HD7B									
HD9B									
HD12									

See footnotes be

Table 2 — Anchorage Selection Guide for Holdowns Attached to SPF/HF Lumber

Holdown on SPF/HF Lumber	Stemwall					Slab on Grade			
	Stemwall Width (in.)	Wind and Seismic Design Category A&B		Seismic Design Category C–F		Wind and Seismic Design Category A&B		Seismic Design Category C–F	
		Midwall/Corner	End Wall	Midwall/Corner	End Wall	Midwall/Corner	Garage Curb	Midwall/Corner	Garage Curb
HDU2	6	SSTB16		SSTB16		SSTB16		SSTB16	
HDU4	6	SSTB16		SSTB24		SSTB16		SSTB16	SSTB24
HDU5	6	SSTB24* (4,295)		SB5/8X24		SSTB16	SSTB24* (4,295)	SSTB20	SB5/8X24
HDU8	8	SSTB28		SSTB28	SSTB28* (6,395)	SSTB28		SSTB28	SSTB28
HDQ8	8	SSTB28		SSTB28	SSTB28* (6,395)	SSTB28		SSTB28	SSTB28
HDU11	8	SB1X30* (9,505)	PAB8	PAB8	PAB8	SB1x30		SB1x30	
HHDQ11	8	SB1X30	PAB8	PAB8					
HDU14	—	PAB8		PAB8		SB1x30		SB1x30	
HHDQ14	—								
LTTP2	6	SSTB16		SSTB16		SSTB16		SSTB16	
LTTI31	6								
HTT4	6	SSTB20		SB5/8X24		SSTB16	SSTB20	SSTB16* (3,780)	SB5/8X24
HTT5	6	SB5/8X24		SB5/8X24		SSTB20	SB5/8X24	SSTB24	SB5/8X24
HD3B	6	SSTB16		SSTB24		SSTB16		SSTB16	SSTB20* (2,960)

We've made selecting the right anchor bolt for the holdown easier; check out our Holdown Anchorage Solutions table on p. 46 or visit **app.strongtie.com/pfd** to access the Post-to-Foundation Designer web application.

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ROOF HEADER BEAM DESIGN:

Roof Beam - Beam -1

Assumptions:

1. Spans:		**ℓ =**	**17.42**	feet span
		=	209.04	inches
		a =	**5.25**	feet span
		=	63	inches
		Cant. =	**15.08**	feet
2. Roof Rafter Span Supported			**14.83**	feet
Roof Rafter Span Supported			**11.17**	feet
Roof Rafter Span Supported			**3.67**	feet
4. Assume bearing area =	Exterior Wall: (5.5" x 5.5")		**5.50**	inches

Steel Beam - W10 x 45

AISC Design Values

A	=	**13.30**	inches^2		
A_{web}	=	**3.54**	inches^2		
E	=	**29,000,000**	PSI		
S	=	**49**	inches3		
I	=	**248**	inches4		
$Fv_{Allowable}$	=	**9,500**	PSI		SDOT 6-02.3(17)B For Salvaged Steel
		30,000	PSI	$0.6F_y$	AISC (Chapter G 16.1-64)
$Fp_{Allowable}$	=	**22,500**	PSI		SDOT 6-02.3(17)B For Salvaged Steel
Fb	=	**33,000**	PSI		AISC (F3-1)
t_w	=	**0.35**	inches		
d	=	**10.1**	inches		
t_f	=	**0.62**	inches		
F_y	=	**50,000**	PSI		
k	=	**0.69**	inches		
b_f	=	**8.02**	inches		
W_t	=	**45**	lbs/lf		

Reactions:

W	=	Roof Framing DL	12.3	PSF
		Steel Beam DL	2.0	PLF
		Roof LL (Snow)	25	PSF
		Design Load = DL x 1.2 + LL x 1.4 =	**52.14**	PSF

Reactions: **See Visual Analysis Resilts**

W	=	**387**	plf	14.83 ft span
W	=	**291**	plf	11.17 ft span
W	=	**96**	plf	3.67 ft span
V_{max}	=	**8,001**	lbs	
P_{max}	=	**8,001**	lbs	
M_{max}	=	**17,305**	ft-lbs	
D_{max}	=	**0.32**	inches	

Check Shear:

Fvmax	=	**V_{max} / A_{web}**	=	**2,263**	lbs
$Fv_{Allowable}$	=			**9,500.0**	lbs
				Check - O.K.	
$Fv_{Allowable}$	=	**$0.66F_w t_w$(N + 2.5k)**	=	**83,377**	LBS
		ASD Section K1.3		**Check - O.K.**	

Check Bearing-Web Crippling:

F_p	=	P_{max} / A_{web}	=	**2,263** PSI
$Fp_{Allowable}$	=			**22,500.0** PSI **Check - O.K.**
$Fv_{Allowable}$	=	**0.66**$F_w t_w$(N + 2.5k) ASD Section K1.3	=	**83,377** LBS **Check - O.K.**

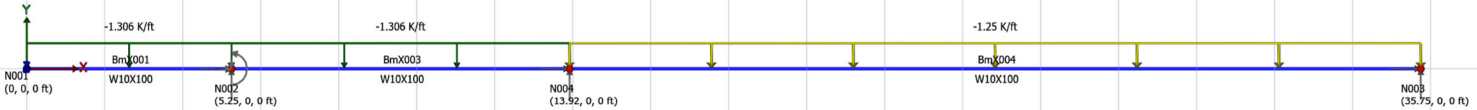
Check Bending:

$S_{Req'd}$	=	M_{max}(12) / **Fb**	=	**6.3** IN^2
$S_{Furnished}$	=			**49.1** IN^2 **Check - O.K.**

Check Deflection:

$D_{max.}$	=			**0.32** inches
$D_{Allowable}$	=	L/350	=	**0.52** inches **Check - O.K.**

Roof Beam - 4 - Steel Beam.vap
Service Case: D
IES VisualAnalysis 20.00.0001
Friday, May 3, 2024





BEAM 1 - COLUMN LOADING @ NODE 4 (Worst Case)

See Visual Analysis Results

$P_{max\ (factored)}$ = 14,767 lbs

Allowable Bearing for the timber bottom plate: 450 psi

Bearing area required = 14,767 / 450 = 32.82 sq-in

2 x 6 Bottom Plate = 5.5 inches in width

Length of bottom plate required = 32.82 / 5.5 = 5.97 in

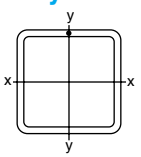
Therefore the steel plate attached to the steel HSS column section needs to be: 5.5" x 6"

Use a 5/8" x 5.5" x 8.5" plate

LRFD Columns
Square HSS

BEAM 1 - POST @ NODE 4

Fy=50



ERW

Design Axial Strength in kips (ϕ=0.85)

Nominal Size	5 x 5						4 1/2 x 4 1/2					
Wall Thickness	1/2	3/8	5/16	1/4	3/16	1/8	1/2	3/8	5/16	1/4	3/16	1/8
Weight Per Foot	28.43	22.37	19.08	15.62	11.97	8.16	25.03	19.82	16.96	13.91	10.70	7.31
Design Wall Thickness	0.465	0.349	0.291	0.233	0.174	0.116*	0.465	0.349	0.291	0.233	0.174	0.116*
	F_y = 50 ksi											
Effective length KL in feet												
0	335	263	224	183	139	90	295	233	199	163	125	85
2	331	260	221	181	138	90	291	229	196	161	123	84
3	325	256	218	178	136	89	285	225	192	158	121	82
4	318	250	213	175	133	88	277	219	188	154	118	81
5	309	244	208	170	130	86	267	212	182	149	114	78
6	299	236	201	165	126	84	255	203	174	144	110	75
7	287	227	194	159	122	82	242	194	166	137	105	72
8	273	217	185	153	117	79	228	183	158	130	100	69
9	259	206	177	145	112	76	213	172	148	123	94	65
10	244	194	167	138	106	73	197	160	138	115	88	61
11	228	182	157	130	100	69	181	147	128	107	82	57
12	212	170	147	122	94	65	165	135	118	98	76	53
13	196	158	137	113	88	60	149	123	107	90	70	48
14	180	146	126	105	81	56	133	111	97	82	63	44
15	164	133	116	97	75	52	118	100	88	74	57	40
16	148	122	106	89	69	48	104	88	78	66	52	36
17	134	110	96	81	63	44	92	78	69	59	46	32
18	119	99	87	73	57	40	82	70	62	53	41	29
19	107	89	78	66	52	36	74	63	56	47	37	26
20	97	80	70	59	47	33	67	57	50	43	33	23
21	88	73	64	54	42	30	61	51	45	39	30	21
22	80	66	58	49	39	27	55	47	41	35	27	19
23	73	61	53	45	35	25	50	43	38	32	25	18
24	67	56	49	41	32	23	46	39	35	30	23	16
25	62	51	45	38	30	21	43	36	32	27	21	15
26	57	47	42	35	28	19	39	33	30	25	20	14
27	53	44	39	33	26	18		31	27	23	18	13
28	49	41	36	30	24	17			26	22	17	12
29	46	38	33	28	22	16					16	11
30	43	36	31	26	21	15						
31		33	29	25	19	14						
32				23	18	13						
33						12						
34												
35												
36												
37												
38												
39												
40												
PROPERTIES												
Area, In.^{2}	7.88	6.18	5.26	4.30	3.28	2.23	6.95	5.48	4.68	3.84	2.93	2.00
I, In.^{4}	26.0	21.7	19.0	16.0	12.6	8.80	18.0	15.3	13.5	11.4	9.02	6.35
r, In.	1.82	1.87	1.90	1.93	1.96	1.99	1.61	1.67	1.70	1.73	1.75	1.78

*Slender- element cross-section. Width-Thickness and/or Depth-Thickness ratio, λ_r, exceeds AISC "Specification for the Design of Steel Hollow Structural Sections" Section 2.2.1 limiting value of 1.40 $\sqrt{E/F_y}$.

Note: Double Horizontal Line indicates Kℓ/r limit of 200.

TIMBER POST DESIGN: BEAM 1 POST AT NODE 2

Assumptions:

1. 6 x 6 Timber Post
2. Maximum unbraced length is: **12.00** feet
   Therefore L_u = **12.00** feet
3. Bearing area: (5.5x3.5) = 19.25 inches^2

Support Reactions:

P_{max} = **7,065** lbs See Visual Analysis Results

6 x 6 Timber Post Properties:

A = **19.25** inches2
d = **5.5** inches

6 x 6 Timber Post Reactions:

$P_{allowable}$ = $480{,}000/(L/d)^2$ = **700** psi

P_c = P_{max} / A = **367** psi
Check - O.K.

Check Crushing:

$P_{allowable}$ = = **450** psi

P_c = P_{max} / A = **367** psi
Check - O.K.

Required Bearing Pad Area:

7,065 LBS / **1,500** PSF = 4.71 SF
5.5" x 5.5" bearing area: 1'x1' pad = 19.3 SF
Check - O.K.

TIMBER POST DESIGN: BEAM 1 POST AT NODE 3

Assumptions:

1. 6 x 6 Timber Post
2. Maximum unbraced length is: **12.00** feet
   Therefore L_u = **12.00** feet
3. Bearing area: (5.5x3.5) = 19.25 $inches^2$

Support Reactions:

P_{max} = **3,624** lbs See Visual Analysis Results

6 x 6 Timber Post Properties:

A = **19.25** inches2
d = **5.5** inches

6 x 6 Timber Post Reactions:

$P_{allowable}$ = $480{,}000/(L/d)^2$ = **700** psi

P_c = P_{max} / A = **188** psi
Check - O.K.

Check Crushing:

$P_{allowable}$ = = **450** psi

P_c = P_{max} / A = **188** psi
Check - O.K.

Required Bearing Pad Area:

3,624 LBS / **1,500** PSF = 2.42 SF
5.5" x 5.5" bearing area: 1'x1' pad = 19.3 SF
Check - O.K.

TIMBER POST DESIGN: BEAM 2 POST AT NODE 2

Assumptions:

1. 6 x 6 Timber Post
2. Maximum unbraced length is: **12.00** feet
   Therefore L_u = **12.00** feet
3. Bearing area: (5.5x3.5) = 19.25 inches^2

Support Reactions:

P_{max} = **5,510** lbs See Visual Analysis Results

6 x 6 Timber Post Properties:

A = **19.25** inches2
d = **5.5** inches

6 x 6 Timber Post Reactions:

$P_{allowable}$ = $480,000/(L/d)^2$ = **700** psi

P_c = P_{max} / A = **286** psi
Check - O.K.

Check Crushing:

$P_{allowable}$ = = **450** psi

P_c = P_{max} / A = **286** psi
Check - O.K.

Required Bearing Pad Area:

5,510 LBS / **1,500** PSF = 3.67 SF
5.5" x 5.5" bearing area: 1'x1' pad = 19.3 SF
Check - O.K.

TIMBER POST DESIGN: BEAM 4 POST AT NODE 2

Assumptions:

1. 6 x 6 Timber Post
2. Maximum unbraced length is: **12.00** feet
   Therefore L_u = **12.00** feet
3. Bearing area: (5.5x3.5) = 19.25 $inches^2$

Support Reactions:

P_{max} = **5,503** lbs See Visual Analysis Results

6 x 6 Timber Post Properties:

A = **19.25** inches2
d = **5.5** inches

6 x 6 Timber Post Reactions:

$P_{allowable}$ = $480,000/(L/d)^2$ = **700** psi

P_c = P_{max} / A = **286** psi
Check - O.K.

Check Crushing:

$P_{allowable}$ = = **450** psi

P_c = P_{max} / A = **286** psi
Check - O.K.

Required Bearing Pad Area:

5,503 LBS / **1,500** PSF = 3.67 SF
5.5" x 5.5" bearing area: 1'x1' pad = 19.3 SF
Check - O.K.

ROOF SPAN TABLE

Maximum Horizontal Clear Spans—Roof

14'-10" SPAN

O.C. Spacing	Depth	TJI®	Design Live Load (LL) and Dead Load (DL) in PSF											
			Non-Snow (125%)				Snow Load Area (115%)							
			20LL + 15DL		20LL + 20DL		25LL + 15DL		30LL + 15DL		40LL + 15DL		50LL + 15DL	
			Low	High	Low	High	Low	High	Low	High	Low	High	Low	High
16"	9½"	110	20'-0"	17'-10"	19'-1"	16'-11"	19'-2"	17'-2"	18'-5"	16'-7"	17'-2"	15'-7"	15'-11"	14'-9"
		210	21'-2"	18'-10"	20'-2"	17'-10"	20'-3"	18'-2"	19'-6"	17'-6"	18'-2"	16'-6"	17'-2"	15'-7"
		230	21'-11"	19'-6"	20'-10"	18'-6"	20'-11"	18'-9"	20'-2"	18'-1"	18'-10"	17'-0"	17'-9"	16'-2"
	11⅞"	110	23'-11"	21'-4"	22'-9"	20'-2"	22'-8"	20'-6"	21'-5"	19'-10"	19'-5"	18'-7"	17'-11"	17'-4"
		210	25'-3"	22'-6"	24'-1"	21'-4"	24'-2"	21'-8"	23'-3"	20'-11"	21'-4"	19'-8"	19'-8"	18'-8"
		230	26'-1"	23'-3"	24'-10"	22'-0"	24'-11"	22'-4"	24'-0"	21'-7"	22'-5"	20'-4"	20'-9"	19'-3"
		360	27'-9"	24'-9"	26'-5"	23'-5"	26'-7"	23'-10"	25'-6"	23'- 0"	23'-11"	21'-7"	22'-7"	20'-6"
		560	31'-11"	28'-6"	30'-5"	27'- 0"	30'-7"	27'-5"	29'-5"	26'-5"	27'-6"	24'-10"	26'- 0"	23'-7"
	14"	110	27'-2"	24'-3"	25'-7"	23'-0"	24'-9"	23'-4"	23'-4"	22'-4"	21'-2"	20'-5"	19'-6"	18'-11"
		210	28'-9"	25'-7"	27'-4"	24'-3"	27'-1"	24'-8"	25'-7"	23'-9"	23'-3"	22'-4"	21'-5"	20'-9"
		230	29'-8"	26'-6"	28'-3"	25'-1"	28'-5"	25'-5"	27'-0"	24'-7"	24'-6"	23'-1"	22'-7"	21'-10"
		360	31'-6"	28'-2"	30'-0"	26'-8"	30'-2"	27'-1"	29'-0"	26'-1"	27'-2"	24'-7"	25'-8"	23'-4"
		560	36'-3"	32'-4"	34'-6"	30'-7"	34'-8"	31'-1"	33'-4"	30'-0"	31'-2"	28'-3"	29'-6"	26'-9"
	16"	110	29'-5"	26'-11"	27'-5"	25'-6"	26'-5"	25'-2"	25'-0"	23'-10"	22'-8"	21'-10"	20'-5"	20'-3"
		210	31'-10"	28'-5"	30'-0"	26'-11"	29'-0"	27'-4"	27'-5"	26'-2"	24'-10"	23'-11"	22'-8"	22'-2"
		230	32'-10"	29'-4"	31'-4"	27'-9"	30'-7"	28'-2"	28'-11"	27'-3"	26'-2"	25'-3"	24'-2"	23'-5"
		360	34'-11"	31'-2"	33'-3"	29'-6"	33'-5"	30'- 0"	32'-2"	28'-11"	30'-1"	27'-2"	26'- 0"	25'-10"
		560	40'-1"	35'-9"	38'-2"	33'-11"	38'-4"	34'-5"	36'-11"	33'-2"	34'-6"	31'-3"	31'-8"	29'-8"
19.2"	9½"	110	18'-9"	16'-9"	17'-11"	15'-10"	18'-0"	16'-1"	17'-3"	15'-7"	15'-9"	14'-7"	14'-6"	13'-10"
		210	19'-10"	17'-9"	18'-11"	16'-9"	19'-0"	17'-0"	18'-3"	16'-5"	17'-1"	15'-5"	15'-11"	14'-8"
		230	20'-7"	18'-4"	19'-7"	17'-4"	19'-8"	17'-7"	18'-11"	17'-0"	17'-8"	16'-0"	16'-8"	15'-2"
	11⅞"	110	22'-5"	20'-0"	21'-5"	19'-0"	20'-9"	19'-3"	19'-7"	18'-7"	17'-9"	17'-1"	16'-4"	15'-10"
		210	23'-9"	21'-2"	22'-7"	20'-0"	22'-8"	20'-4"	21'-5"	19'-8"	19'-6"	18'-6"	17'-11"	17'-4"
		230	24'-6"	21'-10"	23'-4"	20'-8"	23'-5"	21'-0"	22'-6"	20'-3"	20'-6"	19'-1"	18'-11"	18'-1"
		360	26'-1"	23'-3"	24'-10"	22'-0"	24'-11"	22'-4"	24'- 0"	21'-7"	22'-5"	20'-3"	21'-2"	19'-3"
		560	30'- 0"	26'-9"	28'-7"	25'-4"	28'-8"	25'-9"	27'-7"	24'-10"	25'-9"	23'-4"	24'-4"	22'-2"
	14"	110	25'-1"	22'-10"	23'-4"	21'-7"	22'-7"	21'-5"	21'-4"	20'-4"	19'-4"	18'-7"	17'-0"	17'-3"
		210	27'-0"	24'-1"	25'-7"	22'-10"	24'-9"	23'-2"	23'-4"	22'-4"	21'-2"	20'-5"	18'-10"	18'-11"
		230	27'-10"	24'-10"	26'-6"	23'-7"	26'-1"	23'-11"	24'-7"	23'-1"	22'-4"	21'-6"	20'-7"	19'-11"
		360	29'-7"	26'-5"	28'-2"	25'-0"	28'-4"	25'-5"	27'-3"	24'-6"	25'-6"	23'-1"	21'-7"	21'-8"
		560	34'-0"	30'-4"	32'-5"	28'-9"	32'-7"	29'-2"	31'-4"	28'-2"	29'-3"	26'-6"	26'-5"	25'-2"
	16"	110	26'-10"	25'-4"	25'-0"	23'-5"	24'-2"	22'-11"	22'-10"	21'-9"	20'-1"	19'-11"	17'-0"	18'-3"
		210	29'-5"	26'-8"	27'-5"	25'-4"	26'-5"	25'-2"	25'-0"	23'-11"	22'-3"	21'-10"	18'-10"	20'-2"
		230	30'-11"	27'-7"	28'-11"	26'-1"	27'-11"	26'-6"	26'-4"	25'-2"	23'-11"	23'-0"	21'-2"	21'-3"
		360	32'-10"	29'-3"	31'-3"	27'-9"	31'-5"	28'-2"	30'-2"	27'-2"	25'-7"	25'-3"	21'-7"	21'-8"
		560	37'-8"	33'-7"	35'-10"	31'-10"	36'-0"	32'-4"	34'-8"	31'-2"	31'-3"	29'-4"	26'-5"	25'-5"
24"	9½"	110	17'-5"	15'-6"	16'-7"	14'-8"	16'-5"	14'-11"	15'-6"	14'-5"	14'-1"	13'-6"	13'-0"	12'-7"
		210	18'-5"	16'-5"	17'-6"	15'-6"	17'-7"	15'-9"	16'-11"	15'-3"	15'-5"	14'-4"	14'-3"	13'-7"
		230	19'-0"	17'-0"	18'-1"	16'-1"	18'-2"	16'-4"	17'-6"	15'-9"	16'-3"	14'-10"	15'-0"	14'-0"
	11⅞"	110	20'-7"	18'-7"	19'-2"	17'-7"	18'-6"	17'-7"	17'-6"	16'-8"	15'-10"	15'-3"	13'-7"	14'-2"
		210	21'-11"	19'-7"	20'-11"	18'-7"	20'-4"	18'-10"	19'-2"	18'-2"	17'-5"	16'-9"	15'-0"	15'-6"
		230	22'-8"	20'-3"	21'-7"	19'-2"	21'-5"	19'-5"	20'-3"	18'-9"	18'-4"	17'-8"	16'-11"	16'-4"
		360	24'-1"	21'-6"	23'- 0"	20'-5"	23'-1"	20'-8"	22'-2"	20'-0"	20'-5"	18'-9"	17'-3"	17'-4"
		560	27'-9"	24'-9"	26'-5"	23'-6"	26'-7"	23'-10"	25'-6"	23'-0"	23'-10"	21'-7"	21'-1"	20'-3"
	14"	110	22'-5"	21'-1"	20'-10"	19'-6"	20'-2"	19'-2"	19'-0"	18'-2"	16'-0"	16'-7"	13'-7"	14'-7"
		210	24'-7"	22'-4"	22'-11"	21'-1"	22'-1"	21'-0"	20'-10"	19'-11"	17'-10"	18'-3"	15'-0"	16'-1"
		230	25'-9"	23'-0"	24'-1"	21'-10"	23'-4"	22'-2"	22'-0"	21'-0"	20'-0"	19'-3"	16'-11"	17'-0"
		360	27'-5"	24'-6"	26'-1"	23'-2"	26'-3"	23'-6"	25'-0"	22'-8"	20'-5"	20'-2"	17'-3"	17'-4"
		560	31'-6"	28'-1"	30'-0"	26'-8"	30'-2"	27'-0"	29'-0"	26'-1"	24'-11"	23'-7"	21'-1"	20'-3"
	16"	110	24'-0"	22'-8"	22'-4"	20'-11"	21'-7"	20'-6"	19'-8"	19'-6"	16'-0"	16'-11"	13'-7"	14'-7"
		210	26'-3"	24'-9"	24'-6"	22'-11"	23'-8"	22'-6"	21'-9"	21'-4"	17'-10"	18'-9"	15'-0"	16'-1"
		230	27'-9"	25'-6"	25'-10"	24'-2"	24'-11"	23'-8"	23'-7"	22'-6"	20'-0"	19'-9"	16'-11"	17'-0"
		360	30'-4"	27'-1"	28'-11"	25'-8"	28'-2"	26'-1"	25'-0"	24'-1"	20'-5"	20'-2"	17'-3"	17'-4"
		560	34'-10"	31'-2"	33'-2"	29'-6"	33'-4"	29'-11"	30'-6"	28'-3"	24'-11"	23'-7"	21'-1"	20'-3"

How to Use This Table

1. Determine appropriate live and dead load, and the load duration factor.
2. If your slope is 6:12 or less, use the **Low** slope column. If it is between 6:12 and 12:12, use the **High** column.
3. Scan down the column until you find a span that meets or exceeds the span of your application.
4. Select TJI® joist and on-center spacing.

General Notes

- Table is based on:
  - Minimum bearing length of 1¾" end and 3½" intermediate, without web stiffeners.
  - Uniform loads.
  - More restrictive of simple or continuous span.
  - Minimum roof slope of ¼:12.
- Total load values are limited to deflection of L/180 and live load is based on joist deflection of L/240.
- For continuous spans, ratio of short span to long span should be 0.4 or greater to prevent uplift.
- A support beam or wall at the high end is required. Ridge board applications do not provide adequate support.
- For flat roofs or other loading conditions not shown, refer to Weyerhaeuser software.

ROOF SPAN TABLE

Maximum Horizontal Clear Spans—Roof

23'-0" SPAN

O.C. Spacing	Depth	TJI®	Design Live Load (LL) and Dead Load (DL) in PSF											
			Non-Snow (125%)				Snow Load Area (115%)							
			20LL + 15DL		20LL + 20DL		25LL + 15DL		30LL + 15DL		40LL + 15DL		50LL + 15DL	
			Low	High	Low	High	Low	High	Low	High	Low	High	Low	High
16"	9½"	110	20'-0"	17'-10"	19'-1"	16'-11"	19'-2"	17'-2"	18'-5"	16'-7"	17'-2"	15'-7"	15'-11"	14'-9"
		210	21'-2"	18'-10"	20'-2"	17'-10"	20'-3"	18'-2"	19'-6"	17'-6"	18'-2"	16'-6"	17'-2"	15'-7"
		230	21'-11"	19'-6"	20'-10"	18'-6"	20'-11"	18'-9"	20'-2"	18'-1"	18'-10"	17'-0"	17'-9"	16'-2"
	11⅞"	110	23'-11"	21'-4"	22'-9"	20'-2"	22'-8"	20'-6"	21'-5"	19'-10"	19'-5"	18'-7"	17'-11"	17'-4"
		210	25'-3"	22'-6"	24'-1"	21'-4"	24'-2"	21'-8"	23'-3"	20'-11"	21'-4"	19'-8"	19'-8"	18'-8"
		230	26'-1"	23'-3"	24'-10"	22'-0"	24'-11"	22'-4"	24'-0"	21'-7"	22'-5"	20'-4"	20'-9"	19'-3"
		360	27'-9"	24'-9"	26'-5"	23'-5"	26'-7"	23'-10"	25'-6"	23'- 0"	23'-11"	21'-7"	22'-7"	20'-6"
		560	31'-11"	28'-6"	30'-5"	27'- 0"	30'-7"	27'-5"	29'-5"	26'-5"	27'-6"	24'-10"	26'- 0"	23'-7"
	14"	110	27'-2"	24'-3"	25'-7"	23'-0"	24'-9"	23'-4"	23'-4"	22'-4"	21'-2"	20'-5"	19'-6"	18'-11"
		210	28'-9"	25'-7"	27'-4"	24'-3"	27'-1"	24'-8"	25'-7"	23'-9"	23'-3"	22'-4"	21'-5"	20'-9"
		230	29'-8"	26'-6"	28'-3"	25'-1"	28'-5"	25'-5"	27'-0"	24'-7"	24'-6"	23'-1"	22'-7"	21'-10"
		360	31'-6"	28'-2"	30'-0"	26'-8"	30'-2"	27'-1"	29'-0"	26'-1"	27'-2"	24'-7"	25'-8"	23'-4"
		560	36'-3"	32'-4"	34'-6"	30'-7"	34'-8"	31'-1"	33'-4"	30'-0"	31'-2"	28'-3"	29'-6"	26'-9"
	16"	110	29'-5"	26'-11"	27'-5"	25'-6"	26'-5"	25'-2"	25'-0"	23'-10"	22'-8"	21'-10"	20'-5"	20'-3"
		210	31'-10"	28'-5"	30'-0"	26'-11"	29'-0"	27'-4"	27'-5"	26'-2"	24'-10"	23'-11"	22'-8"	22'-2"
		230	32'-10"	29'-4"	31'-4"	27'-9"	30'-7"	28'-2"	28'-11"	27'-3"	26'-2"	25'-3"	24'-2"	23'-5"
		360	34'-11"	31'-2"	33'-3"	29'-6"	33'-5"	30'- 0"	32'-2"	28'-11"	30'-1"	27'-2"	26'- 0"	25'-10"
		560	40'-1"	35'-9"	38'-2"	33'-11"	38'-4"	34'-5"	36'-11"	33'-2"	34'-6"	31'-3"	31'-8"	29'-8"
19.2"	9½"	110	18'-9"	16'-9"	17'-11"	15'-10"	18'-0"	16'-1"	17'-3"	15'-7"	15'-9"	14'-7"	14'-6"	13'-10"
		210	19'-10"	17'-9"	18'-11"	16'-9"	19'-0"	17'-0"	18'-3"	16'-5"	17'-1"	15'-5"	15'-11"	14'-8"
		230	20'-7"	18'-4"	19'-7"	17'-4"	19'-8"	17'-7"	18'-11"	17'-0"	17'-8"	16'-0"	16'-8"	15'-2"
	11⅞"	110	22'-5"	20'-0"	21'-5"	19'-0"	20'-9"	19'-3"	19'-7"	18'-7"	17'-9"	17'-1"	16'-4"	15'-10"
		210	23'-9"	21'-2"	22'-7"	20'-0"	22'-8"	20'-4"	21'-5"	19'-8"	19'-6"	18'-6"	17'-11"	17'-4"
		230	24'-6"	21'-10"	23'-4"	20'-8"	23'-5"	21'-0"	22'-6"	20'-3"	20'-6"	19'-1"	18'-11"	18'-1"
		360	26'-1"	23'-3"	24'-10"	22'-0"	24'-11"	22'-4"	24'- 0"	21'-7"	22'-5"	20'-3"	21'-2"	19'-3"
		560	30'- 0"	26'-9"	28'-7"	25'-4"	28'-8"	25'-9"	27'-7"	24'-10"	25'-9"	23'-4"	24'-4"	22'-2"
	14"	110	25'-1"	22'-10"	23'-4"	21'-7"	22'-7"	21'-5"	21'-4"	20'-4"	19'-4"	18'-7"	17'-0"	17'-3"
		210	27'-0"	24'-1"	25'-7"	22'-10"	24'-9"	23'-2"	23'-4"	22'-4"	21'-2"	20'-5"	18'-10"	18'-11"
		230	27'-10"	24'-10"	26'-6"	23'-7"	26'-1"	23'-11"	24'-7"	23'-1"	22'-4"	21'-6"	20'-7"	19'-11"
		360	29'-7"	26'-5"	28'-2"	25'-0"	28'-4"	25'-5"	27'-3"	24'-6"	25'-6"	23'-1"	21'-7"	21'-8"
		560	34'-0"	30'-4"	32'-5"	28'-9"	32'-7"	29'-2"	31'-4"	28'-2"	29'-3"	26'-6"	26'-5"	25'-2"
	16"	110	26'-10"	25'-4"	25'-0"	23'-5"	24'-2"	22'-11"	22'-10"	21'-9"	20'-1"	19'-11"	17'-0"	18'-3"
		210	29'-5"	26'-8"	27'-5"	25'-4"	26'-5"	25'-2"	25'-0"	23'-11"	22'-3"	21'-10"	18'-10"	20'-2"
		230	30'-11"	27'-7"	28'-11"	26'-1"	27'-11"	26'-6"	26'-4"	25'-2"	23'-11"	23'-0"	21'-2"	21'-3"
		360	32'-10"	29'-3"	31'-3"	27'-9"	31'-5"	28'-2"	30'-2"	27'-2"	25'-7"	25'-3"	21'-7"	21'-8"
		560	37'-8"	33'-7"	35'-10"	31'-10"	36'-0"	32'-4"	34'-8"	31'-2"	31'-3"	29'-4"	26'-5"	25'-5"
24"	9½"	110	17'-5"	15'-6"	16'-7"	14'-8"	16'-5"	14'-11"	15'-6"	14'-5"	14'-1"	13'-6"	13'-0"	12'-7"
		210	18'-5"	16'-5"	17'-6"	15'-6"	17'-7"	15'-9"	16'-11"	15'-3"	15'-5"	14'-4"	14'-3"	13'-7"
		230	19'-0"	17'-0"	18'-1"	16'-1"	18'-2"	16'-4"	17'-6"	15'-9"	16'-3"	14'-10"	15'-0"	14'-0"
	11⅞"	110	20'-7"	18'-7"	19'-2"	17'-7"	18'-6"	17'-7"	17'-6"	16'-8"	15'-10"	15'-3"	13'-7"	14'-2"
		210	21'-11"	19'-7"	20'-11"	18'-7"	20'-4"	18'-10"	19'-2"	18'-2"	17'-5"	16'-9"	15'-0"	15'-6"
		230	22'-8"	20'-3"	21'-7"	19'-2"	21'-5"	19'-5"	20'-3"	18'-9"	18'-4"	17'-8"	16'-11"	16'-4"
		360	24'-1"	21'-6"	23'- 0"	20'-5"	23'-1"	20'-8"	22'-2"	20'-0"	20'-5"	18'-9"	17'-3"	17'-4"
		560	27'-9"	24'-9"	26'-5"	23'-6"	26'-7"	23'-10"	25'-6"	23'-0"	23'-10"	21'-7"	21'-1"	20'-3"
	14"	110	22'-5"	21'-1"	20'-10"	19'-6"	20'-2"	19'-2"	19'-0"	18'-2"	16'-0"	16'-7"	13'-7"	14'-7"
		210	24'-7"	22'-4"	22'-11"	21'-1"	22'-1"	21'-0"	20'-10"	19'-11"	17'-10"	18'-3"	15'-0"	16'-1"
		230	25'-9"	23'-0"	24'-1"	21'-10"	23'-4"	22'-2"	22'-0"	21'-0"	20'-0"	19'-3"	16'-11"	17'-0"
		360	27'-5"	24'-6"	26'-1"	23'-2"	26'-3"	23'-6"	25'-0"	22'-8"	20'-5"	20'-2"	17'-3"	17'-4"
		560	31'-6"	28'-1"	30'-0"	26'-8"	30'-2"	27'-0"	29'-0"	26'-1"	24'-11"	23'-7"	21'-1"	20'-3"
	16"	110	24'-0"	22'-8"	22'-4"	20'-11"	21'-7"	20'-6"	19'-8"	19'-6"	16'-0"	16'-11"	13'-7"	14'-7"
		210	26'-3"	24'-9"	24'-6"	22'-11"	23'-8"	22'-6"	21'-9"	21'-4"	17'-10"	18'-9"	15'-0"	16'-1"
		230	27'-9"	25'-6"	25'-10"	24'-2"	24'-11"	23'-8"	23'-7"	22'-6"	20'-0"	19'-9"	16'-11"	17'-0"
		360	30'-4"	27'-1"	28'-11"	25'-8"	28'-2"	26'-1"	25'-0"	24'-1"	20'-5"	20'-2"	17'-3"	17'-4"
		560	34'-10"	31'-2"	33'-2"	29'-6"	33'-4"	29'-11"	30'-6"	28'-3"	24'-11"	23'-7"	21'-1"	20'-3"

How to Use This Table

1. Determine appropriate live and dead load, and the load duration factor.
2. If your slope is 6:12 or less, use the **Low** slope column. If it is between 6:12 and 12:12, use the **High** column.
3. Scan down the column until you find a span that meets or exceeds the span of your application.
4. Select TJI® joist and on-center spacing.

General Notes

- Table is based on:
  - Minimum bearing length of 1¾" end and 3½" intermediate, without web stiffeners.
  - Uniform loads.
  - More restrictive of simple or continuous span.
  - Minimum roof slope of ¼:12.
- Total load values are limited to deflection of L/180 and live load is based on joist deflection of L/240.
- For continuous spans, ratio of short span to long span should be 0.4 or greater to prevent uplift.
- A support beam or wall at the high end is required. Ridge board applications do not provide adequate support.
- For flat roofs or other loading conditions not shown, refer to Weyerhaeuser software.

ROOF SPAN TABLE

Maximum Horizontal Clear Spans—Roof

25'-2" SPAN

O.C. Spacing	Depth	TJI®	Design Live Load (LL) and Dead Load (DL) in PSF											
			Non-Snow (125%)				Snow Load Area (115%)							
			20LL + 15DL		20LL + 20DL		25LL + 15DL		30LL + 15DL		40LL + 15DL		50LL + 15DL	
			Low	High	Low	High	Low	High	Low	High	Low	High	Low	High
16"	9½"	110	20'-0"	17'-10"	19'-1"	16'-11"	19'-2"	17'-2"	18'-5"	16'-7"	17'-2"	15'-7"	15'-11"	14'-9"
		210	21'-2"	18'-10"	20'-2"	17'-10"	20'-3"	18'-2"	19'-6"	17'-6"	18'-2"	16'-6"	17'-2"	15'-7"
		230	21'-11"	19'-6"	20'-10"	18'-6"	20'-11"	18'-9"	20'-2"	18'-1"	18'-10"	17'-0"	17'-9"	16'-2"
	11⅞"	110	23'-11"	21'-4"	22'-9"	20'-2"	22'-8"	20'-6"	21'-5"	19'-10"	19'-5"	18'-7"	17'-11"	17'-4"
		210	25'-3"	22'-6"	24'-1"	21'-4"	24'-2"	21'-8"	23'-3"	20'-11"	21'-4"	19'-8"	19'-8"	18'-8"
		230	26'-1"	23'-3"	24'-10"	22'-0"	24'-11"	22'-4"	24'-0"	21'-7"	22'-5"	20'-4"	20'-9"	19'-3"
		360	27'-9"	24'-9"	26'-5"	23'-5"	26'-7"	23'-10"	25'-6"	23'- 0"	23'-11"	21'-7"	22'-7"	20'-6"
		560	31'-11"	28'-6"	30'-5"	27'- 0"	30'-7"	27'-5"	29'-5"	26'-5"	27'-6"	24'-10"	26'- 0"	23'-7"
	14"	110	27'-2"	24'-3"	25'-7"	23'-0"	24'-9"	23'-4"	23'-4"	22'-4"	21'-2"	20'-5"	19'-6"	18'-11"
		210	28'-9"	25'-7"	27'-4"	24'-3"	27'-1"	24'-8"	25'-7"	23'-9"	23'-3"	22'-4"	21'-5"	20'-9"
		230	29'-8"	26'-6"	28'-3"	25'-1"	28'-5"	25'-5"	27'-0"	24'-7"	24'-6"	23'-1"	22'-7"	21'-10"
		360	31'-6"	28'-2"	30'-0"	26'-8"	30'-2"	27'-1"	29'-0"	26'-1"	27'-2"	24'-7"	25'-8"	23'-4"
		560	36'-3"	32'-4"	34'-6"	30'-7"	34'-8"	31'-1"	33'-4"	30'-0"	31'-2"	28'-3"	29'-6"	26'-9"
	16"	110	29'-5"	26'-11"	27'-5"	25'-6"	26'-5"	25'-2"	25'-0"	23'-10"	22'-8"	21'-10"	20'-5"	20'-3"
		210	31'-10"	28'-5"	30'-0"	26'-11"	29'-0"	27'-4"	27'-5"	26'-2"	24'-10"	23'-11"	22'-8"	22'-2"
		230	32'-10"	29'-4"	31'-4"	27'-9"	30'-7"	28'-2"	28'-11"	27'-3"	26'-2"	25'-3"	24'-2"	23'-5"
		360	34'-11"	31'-2"	33'-3"	29'-6"	33'-5"	30'- 0"	32'-2"	28'-11"	30'-1"	27'-2"	26'- 0"	25'-10"
		560	40'-1"	35'-9"	38'-2"	33'-11"	38'-4"	34'-5"	36'-11"	33'-2"	34'-6"	31'-3"	31'-8"	29'-8"
19.2"	9½"	110	18'-9"	16'-9"	17'-11"	15'-10"	18'-0"	16'-1"	17'-3"	15'-7"	15'-9"	14'-7"	14'-6"	13'-10"
		210	19'-10"	17'-9"	18'-11"	16'-9"	19'-0"	17'-0"	18'-3"	16'-5"	17'-1"	15'-5"	15'-11"	14'-8"
		230	20'-7"	18'-4"	19'-7"	17'-4"	19'-8"	17'-7"	18'-11"	17'-0"	17'-8"	16'-0"	16'-8"	15'-2"
	11⅞"	110	22'-5"	20'-0"	21'-5"	19'-0"	20'-9"	19'-3"	19'-7"	18'-7"	17'-9"	17'-1"	16'-4"	15'-10"
		210	23'-9"	21'-2"	22'-7"	20'-0"	22'-8"	20'-4"	21'-5"	19'-8"	19'-6"	18'-6"	17'-11"	17'-4"
		230	24'-6"	21'-10"	23'-4"	20'-8"	23'-5"	21'-0"	22'-6"	20'-3"	20'-6"	19'-1"	18'-11"	18'-1"
		360	26'-1"	23'-3"	24'-10"	22'-0"	24'-11"	22'-4"	24'- 0"	21'-7"	22'-5"	20'-3"	21'-2"	19'-3"
		560	30'- 0"	26'-9"	28'-7"	25'-4"	28'-8"	25'-9"	27'-7"	24'-10"	25'-9"	23'-4"	24'-4"	22'-2"
	14"	110	25'-1"	22'-10"	23'-4"	21'-7"	22'-7"	21'-5"	21'-4"	20'-4"	19'-4"	18'-7"	17'-0"	17'-3"
		210	27'-0"	24'-1"	25'-7"	22'-10"	24'-9"	23'-2"	23'-4"	22'-4"	21'-2"	20'-5"	18'-10"	18'-11"
		230	27'-10"	24'-10"	26'-6"	23'-7"	26'-1"	23'-11"	24'-7"	23'-1"	22'-4"	21'-6"	20'-7"	19'-11"
		360	29'-7"	26'-5"	28'-2"	25'-0"	28'-4"	25'-5"	27'-3"	24'-6"	25'-6"	23'-1"	21'-7"	21'-8"
		560	34'-0"	30'-4"	32'-5"	28'-9"	32'-7"	29'-2"	31'-4"	28'-2"	29'-3"	26'-6"	26'-5"	25'-2"
	16"	110	26'-10"	25'-4"	25'-0"	23'-5"	24'-2"	22'-11"	22'-10"	21'-9"	20'-1"	19'-11"	17'-0"	18'-3"
		210	29'-5"	26'-8"	27'-5"	25'-4"	26'-5"	25'-2"	25'-0"	23'-11"	22'-3"	21'-10"	18'-10"	20'-2"
		230	30'-11"	27'-7"	28'-11"	26'-1"	27'-11"	26'-6"	26'-4"	25'-2"	23'-11"	23'-0"	21'-2"	21'-3"
		360	32'-10"	29'-3"	31'-3"	27'-9"	31'-5"	28'-2"	30'-2"	27'-2"	25'-7"	25'-3"	21'-7"	21'-8"
		560	37'-8"	33'-7"	35'-10"	31'-10"	36'-0"	32'-4"	34'-8"	31'-2"	31'-3"	29'-4"	26'-5"	25'-5"
24"	9½"	110	17'-5"	15'-6"	16'-7"	14'-8"	16'-5"	14'-11"	15'-6"	14'-5"	14'-1"	13'-6"	13'-0"	12'-7"
		210	18'-5"	16'-5"	17'-6"	15'-6"	17'-7"	15'-9"	16'-11"	15'-3"	15'-5"	14'-4"	14'-3"	13'-7"
		230	19'-0"	17'-0"	18'-1"	16'-1"	18'-2"	16'-4"	17'-6"	15'-9"	16'-3"	14'-10"	15'-0"	14'-0"
	11⅞"	110	20'-7"	18'-7"	19'-2"	17'-7"	18'-6"	17'-7"	17'-6"	16'-8"	15'-10"	15'-3"	13'-7"	14'-2"
		210	21'-11"	19'-7"	20'-11"	18'-7"	20'-4"	18'-10"	19'-2"	18'-2"	17'-5"	16'-9"	15'-0"	15'-6"
		230	22'-8"	20'-3"	21'-7"	19'-2"	21'-5"	19'-5"	20'-3"	18'-9"	18'-4"	17'-8"	16'-11"	16'-4"
		360	24'-1"	21'-6"	23'- 0"	20'-5"	23'-1"	20'-8"	22'-2"	20'-0"	20'-5"	18'-9"	17'-3"	17'-4"
		560	27'-9"	24'-9"	26'-5"	23'-6"	26'-7"	23'-10"	25'-6"	23'-0"	23'-10"	21'-7"	21'-1"	20'-3"
	14"	110	22'-5"	21'-1"	20'-10"	19'-6"	20'-2"	19'-2"	19'-0"	18'-2"	16'-0"	16'-7"	13'-7"	14'-7"
		210	24'-7"	22'-4"	22'-11"	21'-1"	22'-1"	21'-0"	20'-10"	19'-11"	17'-10"	18'-3"	15'-0"	16'-1"
		230	25'-9"	23'-0"	24'-1"	21'-10"	23'-4"	22'-2"	22'-0"	21'-0"	20'-0"	19'-3"	16'-11"	17'-0"
		360	27'-5"	24'-6"	26'-1"	23'-2"	26'-3"	23'-6"	25'-0"	22'-8"	20'-5"	20'-2"	17'-3"	17'-4"
		560	31'-6"	28'-1"	30'-0"	26'-8"	30'-2"	27'-0"	29'-0"	26'-1"	24'-11"	23'-7"	21'-1"	20'-3"
	16"	110	24'-0"	22'-8"	22'-4"	20'-11"	21'-7"	20'-6"	19'-8"	19'-6"	16'-0"	16'-11"	13'-7"	14'-7"
		210	26'-3"	24'-9"	24'-6"	22'-11"	23'-8"	22'-6"	21'-9"	21'-4"	17'-10"	18'-9"	15'-0"	16'-1"
		230	27'-9"	25'-6"	25'-10"	24'-2"	24'-11"	23'-8"	23'-7"	22'-6"	20'-0"	19'-9"	16'-11"	17'-0"
		360	30'-4"	27'-1"	28'-11"	25'-8"	28'-2"	26'-1"	25'-0"	24'-1"	20'-5"	20'-2"	17'-3"	17'-4"
		560	34'-10"	31'-2"	33'-2"	29'-6"	33'-4"	29'-11"	30'-6"	28'-3"	24'-11"	23'-7"	21'-1"	20'-3"

How to Use This Table

1. Determine appropriate live and dead load, and the load duration factor.
2. If your slope is 6:12 or less, use the **Low** slope column. If it is between 6:12 and 12:12, use the **High** column.
3. Scan down the column until you find a span that meets or exceeds the span of your application.
4. Select TJI® joist and on-center spacing.

General Notes

- Table is based on:
  - Minimum bearing length of 1¾" end and 3½" intermediate, without web stiffeners.
  - Uniform loads.
  - More restrictive of simple or continuous span.
  - Minimum roof slope of ¼:12.
- Total load values are limited to deflection of L/180 and live load is based on joist deflection of L/240.
- For continuous spans, ratio of short span to long span should be 0.4 or greater to prevent uplift.
- A support beam or wall at the high end is required. Ridge board applications do not provide adequate support.
- For flat roofs or other loading conditions not shown, refer to Weyerhaeuser software.

ROOF SPAN TABLE

Maximum Horizontal Clear Spans—Roof

O.C. Spacing	Depth	TJI®	Design Live Load (LL) and Dead Load (DL) in PSF											
			Non-Snow (125%)				Snow Load Area (115%)							
			20LL + 15DL		20LL + 20DL		25LL + 15DL		30LL + 15DL		40LL + 15DL		50LL + 15DL	
			Low	High	Low	High	Low	High	Low	High	Low	High	Low	High
16"	9½"	110	20'-0"	17'-10"	19'-1"	16'-11"	19'-2"	17'-2"	18'-5"	16'-7"	17'-2"	15'-7"	15'-11"	14'-9"
		210	21'-2"	18'-10"	20'-2"	17'-10"	20'-3"	18'-2"	19'-6"	17'-6"	18'-2"	16'-6"	17'-2"	15'-7"
		230	21'-11"	19'-6"	20'-10"	18'-6"	20'-11"	18'-9"	20'-2"	18'-1"	18'-10"	17'-0"	17'-9"	16'-2"
	11⅞"	110	23'-11"	21'-4"	22'-9"	20'-2"	22'-8"	20'-6"	21'-5"	19'-10"	19'-5"	18'-7"	17'-11"	17'-4"
		210	25'-3"	22'-6"	24'-1"	21'-4"	24'-2"	21'-8"	23'-3"	20'-11"	21'-4"	19'-8"	19'-8"	18'-8"
		230	26'-1"	23'-3"	24'-10"	22'-0"	24'-11"	22'-4"	24'-0"	21'-7"	22'-5"	20'-4"	20'-9"	19'-3"
		360	27'-9"	24'-9"	26'-5"	23'-5"	26'-7"	23'-10"	25'-6"	23'- 0"	23'-11"	21'-7"	22'-7"	20'-6"
		560	31'-11"	28'-6"	30'-5"	27'- 0"	30'-7"	27'-5"	29'-5"	26'-5"	27'-6"	24'-10"	26'- 0"	23'-7"
	14"	110	27'-2"	24'-3"	25'-7"	23'-0"	24'-9"	23'-4"	23'-4"	22'-4"	21'-2"	20'-5"	19'-6"	18'-11"
		210	28'-9"	25'-7"	27'-4"	24'-3"	27'-1"	24'-8"	25'-7"	23'-9"	23'-3"	22'-4"	21'-5"	20'-9"
		230	29'-8"	26'-6"	28'-3"	25'-1"	28'-5"	25'-5"	27'-0"	24'-7"	24'-6"	23'-1"	22'-7"	21'-10"
		360	31'-6"	28'-2"	30'-0"	26'-8"	30'-2"	27'-1"	29'-0"	26'-1"	27'-2"	24'-7"	25'-8"	23'-4"
		560	36'-3"	32'-4"	34'-6"	30'-7"	34'-8"	31'-1"	33'-4"	30'-0"	31'-2"	28'-3"	29'-6"	26'-9"
	16"	110	29'-5"	26'-11"	27'-5"	25'-6"	26'-5"	25'-2"	25'-0"	23'-10"	22'-8"	21'-10"	20'-5"	20'-3"
		210	31'-10"	28'-5"	30'-0"	26'-11"	29'-0"	27'-4"	27'-5"	26'-2"	24'-10"	23'-11"	22'-8"	22'-2"
		230	32'-10"	29'-4"	31'-4"	27'-9"	30'-7"	28'-2"	28'-11"	27'-3"	26'-2"	25'-3"	24'-2"	23'-5"
		360	34'-11"	31'-2"	33'-3"	29'-6"	33'-5"	30'- 0"	32'-2"	28'-11"	30'-1"	27'-2"	26'- 0"	25'-10"
		560	40'-1"	35'-9"	38'-2"	33'-11"	38'-4"	34'-5"	36'-11"	33'-2"	34'-6"	31'-3"	31'-8"	29'-8"
19.2"	9½"	110	18'-9"	16'-9"	17'-11"	15'-10"	18'-0"	16'-1"	17'-3"	15'-7"	15'-9"	14'-7"	14'-6"	13'-10"
		210	19'-10"	17'-9"	18'-11"	16'-9"	19'-0"	17'-0"	18'-3"	16'-5"	17'-1"	15'-5"	15'-11"	14'-8"
		230	20'-7"	18'-4"	19'-7"	17'-4"	19'-8"	17'-7"	18'-11"	17'-0"	17'-8"	16'-0"	16'-8"	15'-2"
	11⅞"	110	22'-5"	20'-0"	21'-5"	19'-0"	20'-9"	19'-3"	19'-7"	18'-7"	17'-9"	17'-1"	16'-4"	15'-10"
		210	23'-9"	21'-2"	22'-7"	20'-0"	22'-8"	20'-4"	21'-5"	19'-8"	19'-6"	18'-6"	17'-11"	17'-4"
		230	24'-6"	21'-10"	23'-4"	20'-8"	23'-5"	21'-0"	22'-6"	20'-3"	20'-6"	19'-1"	18'-11"	18'-1"
		360	26'-1"	23'-3"	24'-10"	22'-0"	24'-11"	22'-4"	24'- 0"	21'-7"	22'-5"	20'-3"	21'-2"	19'-3"
		560	30'- 0"	26'-9"	28'-7"	25'-4"	28'-8"	25'-9"	27'-7"	24'-10"	25'-9"	23'-4"	24'-4"	22'-2"
	14"	110	25'-1"	22'-10"	23'-4"	21'-7"	22'-7"	21'-5"	21'-4"	20'-4"	19'-4"	18'-7"	17'-0"	17'-3"
		210	27'-0"	24'-1"	25'-7"	22'-10"	24'-9"	23'-2"	23'-4"	22'-4"	21'-2"	20'-5"	18'-10"	18'-11"
		230	27'-10"	24'-10"	26'-6"	23'-7"	26'-1"	23'-11"	24'-7"	23'-1"	22'-4"	21'-6"	20'-7"	19'-11"
		360	29'-7"	26'-5"	28'-2"	25'-0"	28'-4"	25'-5"	27'-3"	24'-6"	25'-6"	23'-1"	21'-7"	21'-8"
		560	34'-0"	30'-4"	32'-5"	28'-9"	32'-7"	29'-2"	31'-4"	28'-2"	29'-3"	26'-6"	26'-5"	25'-2"
	16"	110	26'-10"	25'-4"	25'-0"	23'-5"	24'-2"	22'-11"	22'-10"	21'-9"	20'-1"	19'-11"	17'-0"	18'-3"
		210	29'-5"	26'-8"	27'-5"	25'-4"	26'-5"	25'-2"	25'-0"	23'-11"	22'-3"	21'-10"	18'-10"	20'-2"
		230	30'-11"	27'-7"	28'-11"	26'-1"	27'-11"	26'-6"	26'-4"	25'-2"	23'-11"	23'-0"	21'-2"	21'-3"
		360	32'-10"	29'-3"	31'-3"	27'-9"	31'-5"	28'-2"	30'-2"	27'-2"	25'-7"	25'-3"	21'-7"	21'-8"
		560	37'-8"	33'-7"	35'-10"	31'-10"	36'-0"	32'-4"	34'-8"	31'-2"	31'-3"	29'-4"	26'-5"	25'-5"
24"	9½"	110	17'-5"	15'-6"	16'-7"	14'-8"	16'-5"	14'-11"	15'-6"	14'-5"	14'-1"	13'-6"	13'-0"	12'-7"
		210	18'-5"	16'-5"	17'-6"	15'-6"	17'-7"	15'-9"	16'-11"	15'-3"	15'-5"	14'-4"	14'-3"	13'-7"
		230	19'-0"	17'-0"	18'-1"	16'-1"	18'-2"	16'-4"	17'-6"	15'-9"	16'-3"	14'-10"	15'-0"	14'-0"
	11⅞"	110	20'-7"	18'-7"	19'-2"	17'-7"	18'-6"	17'-7"	17'-6"	16'-8"	15'-10"	15'-3"	13'-7"	14'-2"
		210	21'-11"	19'-7"	20'-11"	18'-7"	20'-4"	18'-10"	19'-2"	18'-2"	17'-5"	16'-9"	15'-0"	15'-6"
		230	22'-8"	20'-3"	21'-7"	19'-2"	21'-5"	19'-5"	20'-3"	18'-9"	18'-4"	17'-8"	16'-11"	16'-4"
		360	24'-1"	21'-6"	23'- 0"	20'-5"	23'-1"	20'-8"	22'-2"	20'-0"	20'-5"	18'-9"	17'-3"	17'-4"
		560	27'-9"	24'-9"	26'-5"	23'-6"	26'-7"	23'-10"	25'-6"	23'-0"	23'-10"	21'-7"	21'-1"	20'-3"
	14"	110	22'-5"	21'-1"	20'-10"	19'-6"	20'-2"	19'-2"	19'-0"	18'-2"	16'-0"	16'-7"	13'-7"	14'-7"
		210	24'-7"	22'-4"	22'-11"	21'-1"	22'-1"	21'-0"	20'-10"	19'-11"	17'-10"	18'-3"	15'-0"	16'-1"
		230	25'-9"	23'-0"	24'-1"	21'-10"	23'-4"	22'-2"	22'-0"	21'-0"	20'-0"	19'-3"	16'-11"	17'-0"
		360	27'-5"	24'-6"	26'-1"	23'-2"	26'-3"	23'-6"	25'-0"	22'-8"	20'-5"	20'-2"	17'-3"	17'-4"
		560	31'-6"	28'-1"	30'-0"	26'-8"	30'-2"	27'-0"	29'-0"	26'-1"	24'-11"	23'-7"	21'-1"	20'-3"
	16"	110	24'-0"	22'-8"	22'-4"	20'-11"	21'-7"	20'-6"	19'-8"	19'-6"	16'-0"	16'-11"	13'-7"	14'-7"
		210	26'-3"	24'-9"	24'-6"	22'-11"	23'-8"	22'-6"	21'-9"	21'-4"	17'-10"	18'-9"	15'-0"	16'-1"
		230	27'-9"	25'-6"	25'-10"	24'-2"	24'-11"	23'-8"	23'-7"	22'-6"	20'-0"	19'-9"	16'-11"	17'-0"
		360	30'-4"	27'-1"	28'-11"	25'-8"	28'-2"	26'-1"	25'-0"	24'-1"	20'-5"	20'-2"	17'-3"	17'-4"
		560	34'-10"	31'-2"	33'-2"	29'-6"	33'-4"	29'-11"	30'-6"	28'-3"	24'-11"	23'-7"	21'-1"	20'-3"

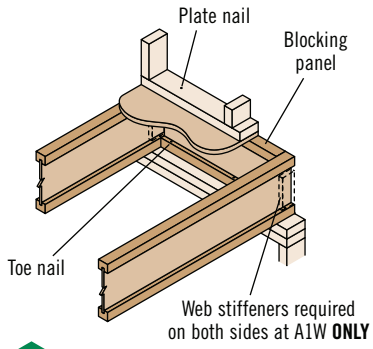
How to Use This Table

1. Determine appropriate live and dead load, and the load duration factor.
2. If your slope is 6:12 or less, use the **Low** slope column. If it is between 6:12 and 12:12, use the **High** column.
3. Scan down the column until you find a span that meets or exceeds the span of your application.
4. Select TJI® joist and on-center spacing.

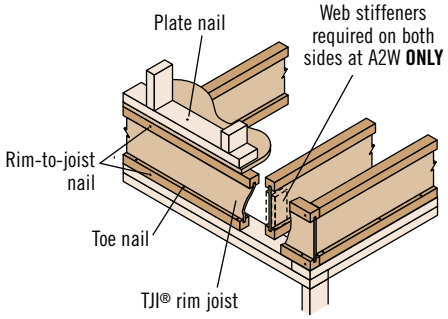
General Notes

- Table is based on:
  - Minimum bearing length of 1¾" end and 3½" intermediate, without web stiffeners.
  - Uniform loads.
  - More restrictive of simple or continuous span.
  - Minimum roof slope of ¼:12.
- Total load values are limited to deflection of L/180 and live load is based on joist deflection of L/240.
- For continuous spans, ratio of short span to long span should be 0.4 or greater to prevent uplift.
- A support beam or wall at the high end is required. Ridge board applications do not provide adequate support.
- For flat roofs or other loading conditions not shown, refer to Weyerhaeuser software.

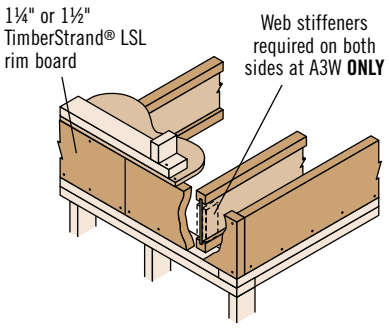
FLOOR DETAILS



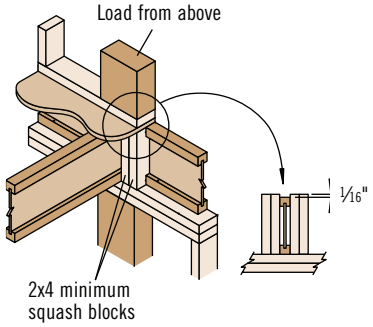
A1 A1W



A2 A2W *Must have 1¾" minimum bearing at ends*



A3 A3W *Attach TimberStrand LSL rim board to bearing plate with connections equivalent to decking schedule*



CS *Use 2x4 minimum squash blocks to transfer load around TJI® joist*

Vertical Load Transfer at Bearing

Rim Joist, Rim Board, or Blocking	Depth	Allowable Uniform Vertical Loads (PLF)
TJI® 360	18"–20"	1,550
TJI® 560		1,550
TJI® 560D		2,250
TJI® 560D	22"–24"	1,700(1)
1¼" TimberStrand® LSL	18"–20"	3,450
1½" TimberStrand® LSL	22"–24"	4,140

(1) Capacity is based on calculation

- Loads may not be increased for duration of load.

Fastening Floor Panels to 1¼" or 1½" TimberStrand® LSL Rim Board

Nail Size	Closest O.C. Spacing per Row	
	Rim Board Size	
	1¼"	1½"
8d (0.113" or 0.131" x 2½"), 10d (0.128" x 3"), 12d (0.128" x 3¼")	4"	3"
10d (0.148" x 3"), 12d (0.148" x 3¼")	4"	4"
16d (0.162" x 3½")	6"(1)	6"(1)

(1) Can be reduced to 4" on-center if nail penetration into the narrow edge is no more than 1⅜" (to avoid splitting).

- Nailing rows must be offset at least ½" and staggered
- Maximum spacing of nails is 24" on-center.
- 14 gauge staples may be substituted for 8d (0.113" x 2½") nails if minimum penetration of 1" is achieved.

FLOOR LOAD TABLES

Floor—100% (PLF) For 6'–18' Spans

Depth	TJI®	Joist Clear Span													
		6'		8'		10'		12'		14'		16'		18'	
		Live Load L/480	Total Load	Live Load L/480	Total Load	Live Load L/480	Total Load	Live Load L/480	Total Load	Live Load L/480	Total Load	Live Load L/480	Total Load	Live Load L/480	Total Load
18"	360	*	320	*	241	*	193	*	162	*	139	*	121	*	108
	560, 560D	*	390	*	294	*	236	*	197	*	169	*	148	*	132
20"	360	*	320	*	241	*	193	*	162	*	139	*	121	*	108
	560, 560D	*	390	*	294	*	236	*	197	*	169	*	148	*	132
22"	560D	*	617	*	465	*	373	*	312	*	268	*	234	*	208
24"	560D	*	658	*	496	*	398	*	332	*	285	*	250	*	222

Floor—100% (PLF) For 20'–30' Spans

Depth	TJI®	Joist Clear Span											
		20'		22'		24'		26'		28'		30'	
		Live Load L/480	Total Load	Live Load L/480	Total Load	Live Load L/480	Total Load	Live Load L/480	Total Load	Live Load L/480	Total Load	Live Load L/480	Total Load
18"	360	*	97	*	88	76	81	61	75				
	560, 560D	*	119	*	108	*	99	89	91	72	85	60	79
20"	360	*	97	*	88	*	81	*	75				
	560, 560D	*	119	*	108	*	99	*	91	*	85	75	79
22"	560D	*	188	*	171	*	157	137	145	112	134	93	125
24"	560D	*	200	*	182	*	167	*	154	134	143	111	134

* Indicates that **Total Load** value controls.

How to Use These Tables

1. Calculate actual total and live load in pounds per linear foot (plf).
2. Select appropriate **Joist Clear Span**.
3. Scan down the column to find a TJI® joist that meets or exceeds actual total and live loads.

General Notes

- Tables are based on:
  - Uniform loads.
  - No composite action provided by sheathing.
  - More restrictive of simple or continuous span.
  - **18" and 20" TJI® joists:** Minimum bearing length of 1¾" end and 3½" intermediate (no web stiffeners).
  - **22" and 24" TJI® joists:** Minimum bearing length of 1¾" end and 3½" intermediate; web stiffeners required at all bearings.
- **Total Load** values are limited to deflection of L/240.
- **Live Load** is based on joist deflection of L/480.
- If a live load deflection limit of L/360 is desired, multiply value in **Live Load** column by 1.33. The resulting live load must not exceed the **Total Load** shown.
- **Table does not account for safe loading.** Use Weyerhaeuser software when this condition applies.

See page 13 for PSF to PLF Conversion Table

ROOF HEADER BEAM DESIGN:

Roof Beam 2

Assumptions:

1. Spans:		**ℓ =**	**9.83**	feet span
		=	117.96	inches
		a =	**5.25**	feet span
		=	63	inches
		Cant. =	**5.25**	feet
2. Roof Rafter Span Supported			**0.00**	feet
Roof Rafter Span Supported			**11.17**	feet
Roof Rafter Span Supported			**3.67**	feet
4. Assume bearing area =	Exterior Wall: (5.5" x 5.5")		**5.50**	inches

Steel Beam - W10 x 22

AISC Design Values

A	=	**6.49**	$inches^2$		
$\mathbf{A_{web}}$	=	**2.44**	$inches^2$		
E	=	**29,000,000**	PSI		
S	=	**23**	inches3		
I	=	**118**	inches4		
$\mathbf{Fv_{Allowable}}$	=	**9,500**	PSI		SDOT 6-02.3(17)B For Salvaged Steel
		30,000	PSI	$0.6\mathbf{F_y}$	AISC (Chapter G 16.1-64)
$\mathbf{Fp_{Allowable}}$	=	**22,500**	PSI		SDOT 6-02.3(17)B For Salvaged Steel
Fb	=	**33,000**	PSI		AISC (F3-1)
$\mathbf{t_w}$	=	**0.24**	inches		
d	=	**10.17**	inches		
$\mathbf{t_f}$	=	**0.36**	inches		
$\mathbf{F_y}$	=	**50,000**	PSI		
k	=	**0.50**	inches		
$\mathbf{b_f}$	=	**5.75**	inches		
$\mathbf{W_t}$	=	**22**	lbs/lf		

Reactions:

W	=	Roof Framing DL	12.3	PSF
		Steel Beam DL	1.5	PSF
		Roof LL (Snow)	25	PSF
		Design Load = DL x 1.2 + LL x 1.4 =	**51.51**	PSF

Reactions: **See Visual Analysis Resilts**

W	=	**382**	plf	0.00 ft span
W	=	**425**	plf	11.17 ft span
W	=	**565**	plf	14.83 ft span
$\mathbf{V_{max}}$	=	**4,306**	lbs	
$\mathbf{P_{max}}$	=	**4,306**	lbs	
$\mathbf{M_{max}}$	=	**6,751**	ft-lbs	
$\mathbf{D_{max}}$	=	**0.03**	inches	

Check Shear:

Fvmax	=	$\mathbf{V_{max}} / \mathbf{A_{web}}$	=	**1,764**	lbs
$\mathbf{Fv_{Allowable}}$	=			**9,500.0**	lbs
				Check - O.K.	
$\mathbf{Fv_{Allowable}}$	=	$\mathbf{0.66F_w t_w}(N + 2.5k)$	=	**53,460**	LBS
		ASD Section K1.3		**Check - O.K.**	

Check Bearing-Web Crippling:

F_p	=	P_{max} / A_{web}	=	**1,764** PSI
$Fp_{Allowable}$	=			**22,500.0** PSI **Check - O.K.**
$Fv_{Allowable}$	=	**0.66**$F_w t_w$(N + 2.5k) ASD Section K1.3	=	**53,460** LBS **Check - O.K.**

Check Bending:

$S_{Req'd}$	=	M_{max}(12) / **Fb**	=	**2.5** IN^2
$S_{Furnished}$	=			**23.2** IN^2 **Check - O.K.**

Check Deflection:

$D_{max.}$	=			**0.03** inches
$D_{Allowable}$	=	L/350	=	**0.18** inches **Check - O.K.**

ROOF HEADER BEAM DESIGN:

Roof Beam 3

Assumptions:

1. Spans:		**ℓ =**	**35.75**	feet span
		=	429	inches
		a =	**5.25**	feet span
		=	63	inches
2. Roof Rafter Span Supported			**20.42**	feet
Roof Rafter Span Supported			**11.17**	feet
Roof Rafter Span Supported			**14.83**	feet
4. Assume bearing area =	Exterior Wall: (5.5" x 5.5")		**5.50**	inches

Steel Beam - W10 x 68

		AISC Design Values			
A	=	**20.00**	$inches^2$		
$\mathbf{A_{web}}$	=	**4.89**	$inches^2$		
E	=	**29,000,000**	PSI		
S	=	**76**	inches3		
I	=	**394**	inches4		
$\mathbf{Fv_{Allowable}}$	=	**9,500**	PSI		SDOT 6-02.3(17)B For Salvaged Steel
		30,000	PSI	$0.6\mathbf{F_y}$	AISC (Chapter G 16.1-64)
$\mathbf{Fp_{Allowable}}$	=	**22,500**	PSI		SDOT 6-02.3(17)B For Salvaged Steel
Fb	=	**33,000**	PSI		AISC (F3-1)
$\mathbf{t_w}$	=	**0.47**	inches		
d	=	**10.4**	inches		
$\mathbf{t_f}$	=	**0.77**	inches		
$\mathbf{F_y}$	=	**50,000**	PSI		
k	=	**0.75**	inches		
$\mathbf{b_f}$	=	**10.13**	inches		
$\mathbf{W_t}$	=	**68**	lbs/lf		

Reactions:

W	=	Roof Framing DL	12.3	PSF
		Steel Beam DL	1.9	PSF
		Roof LL (Snow)	25	PSF
		Design Load = DL x 1.2 + LL x 1.4 =	**52.04**	PSF

Reactions: **See Visual Analysis Resilts**

W	=	**531**	plf	20.42 ft span
W	=	**291**	plf	11.17 ft span
W	=	**386**	plf	14.83 ft span
$\mathbf{V_{max}}$	=	**36,043**	lbs	
$\mathbf{P_{max}}$	=	**36,043**	lbs	
$\mathbf{M_{max}}$	=	**46,100**	ft-lbs	
$\mathbf{D_{max}}$	=	**0.02**	inches	

Check Shear:

Fvmax	=	$\mathbf{V_{max}} / \mathbf{A_{web}}$	=	**7,374**	lbs
$\mathbf{Fv_{Allowable}}$	=			**9,500.0**	lbs
				Check - O.K.	
$\mathbf{Fv_{Allowable}}$	=	$\mathbf{0.66F_w t_w}(N + 2.5k)$	=	**114,386**	LBS
		ASD Section K1.3		**Check - O.K.**	

Check Bearing-Web Crippling:

F_p	=	P_{max} / A_{web}	=	**7,374** PSI
$Fp_{Allowable}$	=			**22,500.0** PSI **Check - O.K.**
$Fv_{Allowable}$	=	$0.66F_w t_w(N + 2.5k)$ ASD Section K1.3	=	**114,386** LBS **Check - O.K.**

Check Bending:

$S_{Req'd}$	=	$M_{max}(12) / Fb$	=	**16.8** IN^2
$S_{Furnished}$	=			**75.7** IN^2 **Check - O.K.**

Check Deflection:

$D_{max.}$	=			**0.02** inches
$D_{Allowable}$	=	L/350	=	**1.23** inches **Check - O.K.**

ROOF HEADER BEAM DESIGN:

Roof Beam 4

Assumptions:

1. Spans:			**ℓ =**	**35.75**	feet span
			=	429	inches
			a =	**5.25**	feet span
			=	63	inches
2. Roof Rafter Span Supported				**20.42**	feet
Roof Rafter Span Supported				**11.17**	feet
Roof Rafter Span Supported				**14.83**	feet
4. Assume bearing area	=	Exterior Wall: (5.5" x 5.5")		**5.50**	inches

Steel Beam - W10 x 54

		AISC Design Values			
A	=	**15.80**	inches^2		
A_{web}	=	**3.73**	inches^2		
E	=	**29,000,000**	PSI		
S	=	**60**	inches3		
I	=	**303**	inches4		
$Fv_{Allowable}$	=	**9,500**	PSI		SDOT 6-02.3(17)B For Salvaged Steel
		30,000	PSI	0.6**F_y**	AISC (Chapter G 16.1-64)
$Fp_{Allowable}$	=	**22,500**	PSI		SDOT 6-02.3(17)B For Salvaged Steel
Fb	=	**33,000**	PSI		AISC (F3-1)
t_w	=	**0.37**	inches		
d	=	**10.09**	inches		
t_f	=	**0.615**	inches		
F_y	=	**50,000**	PSI		
k	=	**0.69**	inches		
b_f	=	**10.03**	inches		
W_t	=	**54**	lbs/lf		

Reactions:

W	=	Roof Framing DL	12.3	PSF
		Steel Beam DL	1.5	PSF
		Roof LL (Snow)	25	PSF
		Design Load = DL x 1.2 + LL x 1.4 =	**51.57**	PSF

Reactions: **See Visual Analysis Resilts**

W	=	**649**	plf	25.17 ft span
W	=	**649**	plf	23.00 ft span
W	=	**622**	plf	ft span
V_{max}	=	**23,345**	lbs	
P_{max}	=	**23,345**	lbs	
M_{max}	=	**26,743**	ft-lbs	
D_{max}	=	**0.02**	inches	

Check Shear:

Fvmax	=	**V_{max} / A_{web}**	=	**6,253**	lbs
$Fv_{Allowable}$	=			**9,500.0**	lbs
				Check - O.K.	
$Fv_{Allowable}$	=	**0.66$F_w t_w$(N + 2.5k)**	=	**88,141**	LBS
		ASD Section K1.3		**Check - O.K.**	

Check Bearing-Web Crippling:

F_p	=	P_{max} / A_{web}	=	**6,253** PSI
$Fp_{Allowable}$	=			**22,500.0** PSI **Check - O.K.**
$Fv_{Allowable}$	=	$0.66F_w t_w(N + 2.5k)$ ASD Section K1.3	=	**88,141** LBS **Check - O.K.**

Check Bending:

$S_{Req'd}$	=	$M_{max}(12) / Fb$	=	**9.7** IN^2
$S_{Furnished}$	=			**60.0** IN^2 **Check - O.K.**

Check Deflection:

$D_{max.}$	=			**0.02** inches
$D_{Allowable}$	=	L/350	=	**1.23** inches **Check - O.K.**

Factored Load Combinations

Name	Code	Effective Equation	Design	Deflection
1. 1.4D	ASCE 7-16 LRFD	1.4D	Strength	Other
2. 1.2D+1.6L+0.5Lr	ASCE 7-16 LRFD	1.2D	Strength	Other
5. 0.9D+W	ASCE 7-16 LRFD	0.9D	Strength	Other

Member Analysis Properties

Name	Member Section	X1 ft	Y1 ft	Z1 ft	X2 ft	Y2 ft	Z2 ft	Elasticity, E Ksi	A in^2	Iz in^4	Iy in^4	Releases	Action
BmX001	W10X45	0.00000	0.00000	0.00000	15.08000	0.00000	0.00000	29000.00000	13.30000	248.00000	53.40000	(none)	Normal
BmX003	W10X45	15.08000	0.00000	0.00000	26.41500	0.00000	0.00000	29000.00000	13.30000	248.00000	53.40000	(none)	Normal
BmX004	W10X45	26.41500	0.00000	0.00000	37.75000	0.00000	0.00000	29000.00000	13.30000	248.00000	53.40000	(none)	Normal

Section Properties

Section	Source	Area in^2	Iz in^4	Iy in^4	J in^4	Alpha, α deg
W10X45	Database Shape	13.30000	248.00000	53.40000	1.51000	0.00000

Member Stresses (extreme rows: max and min)

Member	fa Min Ksi	fa Max Ksi	fby Min Ksi	fby Max Ksi	fbz Min Ksi	fbz Max Ksi	fc comb Min Ksi	fc comb Max Ksi
BmX001	0.00000 (4)	0.00000 (4)	0.00000 (4)	0.00000 (4)	**-5.61394 (2)**	**5.61394 (2)**	**-5.61394 (2)**	**5.61394 (2)**
BmX003	0.00000 (4)	0.00000 (4)	0.00000 (4)	0.00000 (4)	**-2.07833 (2)**	**2.07833 (2)**	**-2.07833 (2)**	**2.07833 (2)**
BmX004	0.00000 (4)	0.00000 (4)	0.00000 (4)	0.00000 (4)	**-2.07833 (2)**	**2.07833 (2)**	**-2.07833 (2)**	**2.07833 (2)**

Node Reactions (extreme rows: max and min)

Node	Result Case	FX K	FY K	FZ K	MX K-ft	MY K-ft	MZ K-ft
N002	1. 1.4D	0.00000	6.29860	0.00000	0.00000	0.00000	**-17.30426**
N002	5. 0.9D+W	0.00000	4.04910	0.00000	0.00000	0.00000	**-11.12417**
N003	5. 0.9D+W	0.00000	**1.76872**	0.00000	0.00000	0.00000	0.00000
N004	1. 1.4D	0.00000	**8.00390**	0.00000	0.00000	0.00000	0.00000

Steel_Beam X_G 1: Results

Deflections Strong (dy): None Weak (dz): None	Axial Manual Kz: False Kz Sidesway?: False Manual Ky: False Ky Sidesway?: False	Size Constraints Limit Depth?: False Limit Width?: False
Overrides Override Fy?: False Override Cb?: False Override HSS t_des?: False Advanced Torsion: False		
Steel Material: ASTM A992 Grade 50 Specification: AISC 360-16 LRFD Composite Beam?: False Seismic Compactness: Not Ductile Check Constrained Axis FTB?: False Overstrength?: False Live Load Reduction: None Disable Checks?: False Check Level: Each Limit State	Bracing Lateral Top (+y): Unbraced Lateral Bottom (-y): Unbraced Strong (z): Unbraced	Torsional Bracing Lateral Top (+y): True Lateral Bottom (-y): True Strong (z): True

Steel_Beam X_G 1: Strong Flexure Check (extreme rows: max)

Member	Section	Offset ft	Result Case	Demand Mz K-ft	Capacity Mz K-ft	Code Reference	Unity Check	Details
BmX001	W10X45	15.08000	1. 1.4D	-22.97452	205.87500	F2-1	**0.11159**	Lb = 15.08 ft, Cb = 2.32532

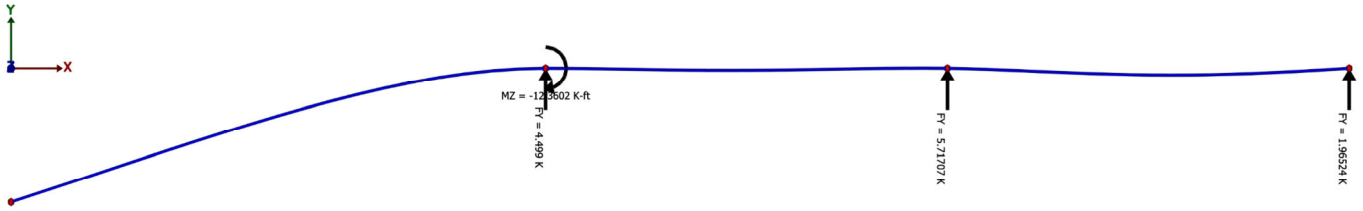
Steel_Beam X_G 1: Strong Shear Check (extreme rows: max)

Member	Section	Offset ft	Result Case	Demand Vy K	Capacity Vy K	Code Reference	Unity Check	Details
BmX004	W10X45	0.00000	1. 1.4D	4.25207	106.05000	G2-1	**0.04009**	Shear Area = 3.535 in^2, Cv = 1, h/tw = 22.4571

Member Displacements (extreme rows: max and min)

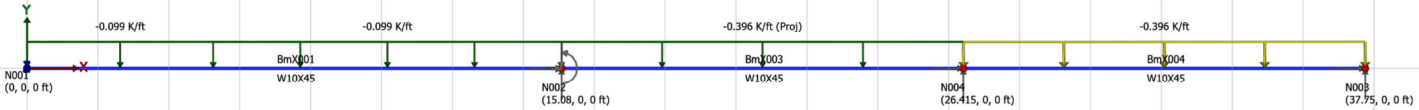
Member	Dy Min in	Dy Max in	Dz Min in	Dz Max in
BmX001	**-0.31382 (2)**	0.00000 (4)	0.00000 (4)	0.00000 (4)
BmX003	**-0.00463 (2)**	**0.00045 (2)**	0.00000 (4)	0.00000 (4)

Roof Beam - 1 - Steel Beam.vap
Result Case: D
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Friday, May 3, 2024





Roof Beam - 1 - Steel Beam.vap
Service Case: D
IES VisualAnalysis 20.00.0001
Friday, May 3, 2024





Factored Load Combinations

Name	Code	Effective Equation	Design	Deflection
1. 1.4D	ASCE 7-16 LRFD	1.4D	Strength	Other
2. 1.2D+1.6L+0.5Lr	ASCE 7-16 LRFD	1.2D	Strength	Other
5. 0.9D+W	ASCE 7-16 LRFD	0.9D	Strength	Other

Member Analysis Properties

Name	Member Section	X1 ft	Y1 ft	Z1 ft	X2 ft	Y2 ft	Z2 ft	Elasticity, E Ksi	A in^2	Iz in^4	Iy in^4	Releases	Action
BmX001	W10X22	0.00000	0.00000	0.00000	5.25000	0.00000	0.00000	29000.00000	6.49000	118.00000	11.40000	(none)	Normal
BmX003	W10X22	5.25000	0.00000	0.00000	10.25000	0.00000	0.00000	29000.00000	6.49000	118.00000	11.40000	(none)	Normal
BmX004	W10X22	10.25000	0.00000	0.00000	15.08000	0.00000	0.00000	29000.00000	6.49000	118.00000	11.40000	(none)	Normal

Section Properties

Section	Source	Area in^2	Iz in^4	Iy in^4	J in^4	Alpha, α deg
W10X22	Database Shape	6.49000	118.00000	11.40000	0.23900	0.00000

Member Stresses (extreme rows: max and min)

Member	fa Min Ksi	fa Max Ksi	fby Min Ksi	fby Max Ksi	fbz Min Ksi	fbz Max Ksi	fc comb Min Ksi	fc comb Max Ksi
BmX001	0.00000 (4)	0.00000 (4)	0.00000 (4)	0.00000 (4)	**-4.04384 (2)**	**4.04384 (2)**	**-4.04384 (2)**	**4.04384 (2)**
BmX003	0.00000 (4)	0.00000 (4)	0.00000 (4)	0.00000 (4)	**-0.74887 (2)**	**0.74887 (2)**	**-0.74887 (2)**	**0.74887 (2)**
BmX004	0.00000 (4)	0.00000 (4)	0.00000 (4)	0.00000 (4)	**-0.74887 (2)**	**0.74887 (2)**	**-0.74887 (2)**	**0.74887 (2)**

Node Reactions (extreme rows: max and min)

Node	Result Case	FX K	FY K	FZ K	MX K-ft	MY K-ft	MZ K-ft
N002	1. 1.4D	0.00000	**4.30511**	0.00000	0.00000	0.00000	**-6.75089**
N002	5. 0.9D+W	0.00000	2.76757	0.00000	0.00000	0.00000	**-4.33986**
N003	5. 0.9D+W	0.00000	**0.68617**	0.00000	0.00000	0.00000	0.00000

Steel_Beam X_G 1: Results

Deflections Strong (dy): None Weak (dz): None	Axial Manual Kz: False Kz Sidesway?: False Manual Ky: False Ky Sidesway?: False	Size Constraints Limit Depth?: False Limit Width?: False
Overrides Override Fy?: False Override Cb?: False Override HSS t_des?: False Advanced Torsion: False		
Steel Material: ASTM A992 Grade 50 Specification: AISC 360-16 LRFD Composite Beam?: False Seismic Compactness: Not Ductile Check Constrained Axis FTB?: False Overstrength?: False Live Load Reduction: None Disable Checks?: False Check Level: Each Limit State	Bracing Lateral Top (+y): Unbraced Lateral Bottom (-y): Unbraced Strong (z): Unbraced	Torsional Bracing Lateral Top (+y): True Lateral Bottom (-y): True Strong (z): True

Steel_Beam X_G 1: Strong Flexure Check (extreme rows: max)

Member	Section	Offset ft	Result Case	Demand Mz K-ft	Capacity Mz K-ft	Code Reference	Unity Check	Details
BmX001	W10X22	5.25000	1. 1.4D	-7.79695	97.50000	F2-1	**0.07997**	Lb = 5.25 ft, Cb = 2.32532

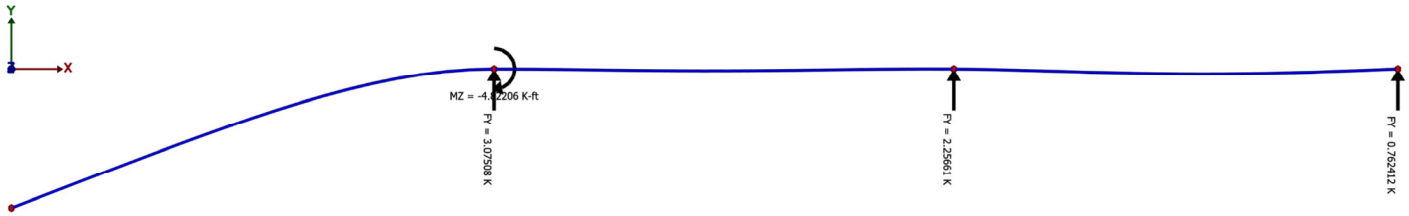
Steel_Beam X_G 1: Strong Shear Check (extreme rows: max)

Member	Section	Offset ft	Result Case	Demand Vy K	Capacity Vy K	Code Reference	Unity Check	Details
BmX001	W10X22	5.25000	1. 1.4D	-2.97027	73.44000	G2-1	**0.04044**	Shear Area = 2.448 in^2, Cv = 1, h/tw = 37

Member Displacements (extreme rows: max and min)

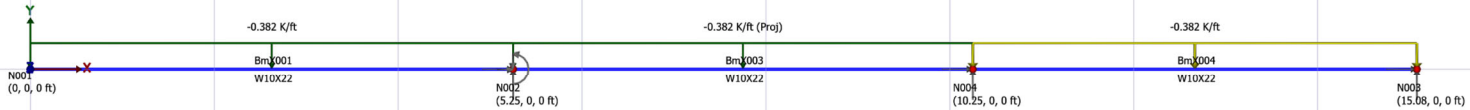
Member	Dy Min in	Dy Max in	Dz Min in	Dz Max in
BmX001	**-0.02713 (2)**	0.00000 (4)	0.00000 (4)	0.00000 (4)
BmX003	**-0.00036 (2)**	**0.00002 (2)**	0.00000 (4)	0.00000 (4)

Roof Beam - 2 - Steel Beam.vap
Result Case: D
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Roof Beam - 2 - Steel Beam.vap
Service Case: D
IES VisualAnalysis 20.00.0001
Friday, May 3, 2024





Factored Load Combinations

Name	Code	Effective Equation	Design	Deflection
1. 1.4D	ASCE 7-16 LRFD	1.4D	Strength	Other
2. 1.2D+1.6L+0.5Lr	ASCE 7-16 LRFD	1.2D	Strength	Other
5. 0.9D+W	ASCE 7-16 LRFD	0.9D	Strength	Other

Member Analysis Properties

Name	Member Section	X1 ft	Y1 ft	Z1 ft	X2 ft	Y2 ft	Z2 ft	Elasticity, E Ksi	A in^2	Iz in^4	Iy in^4	Releases	Action
BmX001	W10X45	0.00000	0.00000	0.00000	5.25000	0.00000	0.00000	29000.00000	13.30000	248.00000	53.40000	(none)	Normal
BmX003	W10X45	5.25000	0.00000	0.00000	35.75000	0.00000	0.00000	29000.00000	13.30000	248.00000	53.40000	(none)	Normal
BmX004	W10X45	35.75000	0.00000	0.00000	37.75000	0.00000	0.00000	29000.00000	13.30000	248.00000	53.40000	(none)	Normal

Section Properties

Section	Source	Area in^2	Iz in^4	Iy in^4	J in^4	Alpha, α deg
W10X45	Database Shape	13.30000	248.00000	53.40000	1.51000	0.00000

Member Stresses (extreme rows: max and min)

Member	fa Min Ksi	fa Max Ksi	fby Min Ksi	fby Max Ksi	fbz Min Ksi	fbz Max Ksi	fc comb Min Ksi	fc comb Max Ksi
BmX001	0.00000 (4)	0.00000 (4)	0.00000 (4)	0.00000 (4)	**-2.12779 (2)**	**2.12779 (2)**	**-2.12779 (2)**	**2.12779 (2)**
BmX003	0.00000 (4)	0.00000 (4)	0.00000 (4)	0.00000 (4)	**-13.23814 (2)**	**13.23814 (2)**	**-13.23814 (2)**	**13.23814 (2)**

Node Reactions (extreme rows: max and min)

Node	Result Case	FX K	FY K	FZ K	MX K-ft	MY K-ft	MZ K-ft
N002	1. 1.4D	0.00000	13.56752	0.00000	0.00000	0.00000	**45.46811**
N002	5. 0.9D+W	0.00000	8.72198	0.00000	0.00000	0.00000	**29.22950**
N003	1. 1.4D	0.00000	**-23.99943**	0.00000	0.00000	0.00000	0.00000
N004	1. 1.4D	0.00000	**35.54567**	0.00000	0.00000	0.00000	0.00000

Steel_Beam X_G 1: Results

Deflections Strong (dy): None Weak (dz): None	Axial Manual Kz: False Kz Sidesway?: False Manual Ky: False Ky Sidesway?: False	Size Constraints Limit Depth?: False Limit Width?: False
Overrides Override Fy?: False Override Cb?: False Override HSS t_des?: False Advanced Torsion: False		
Steel Material: ASTM A992 Grade 50 Specification: AISC 360-16 LRFD Composite Beam?: False Seismic Compactness: Not Ductile Check Constrained Axis FTB?: False Overstrength?: False Live Load Reduction: None Disable Checks?: False Check Level: Each Limit State	Bracing Lateral Top (+y): Unbraced Lateral Bottom (-y): Unbraced Strong (z): Unbraced	Torsional Bracing Lateral Top (+y): True Lateral Bottom (-y): True Strong (z): True

Steel_Beam X_G 1: Strong Flexure Check (extreme rows: max)

Member	Section	Offset ft	Result Case	Demand Mz K-ft	Capacity Mz K-ft	Code Reference	Unity Check	Details
BmX003	W10X45	0.00000	1. 1.4D	-54.17589	205.87500	F2-1	**0.26315**	Lb = 30.5 ft, Cb = 2.32317

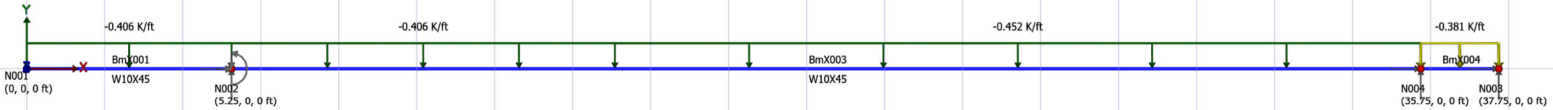
Steel_Beam X_G 1: Strong Shear Check (extreme rows: max)

Member	Section	Offset ft	Result Case	Demand Vy K	Capacity Vy K	Code Reference	Unity Check	Details
BmX004	W10X45	0.00000	1. 1.4D	25.19315	106.05000	G2-1	**0.23756**	Shear Area = 3.535 in^2, Cv = 1, h/tw = 22.4571

Member Displacements (extreme rows: max and min)

Member	Dy Min in	Dy Max in	Dz Min in	Dz Max in
BmX001	**-0.01442 (2)**	0.00000 (4)	0.00000 (4)	0.00000 (4)
BmX003	**-0.40067 (2)**	0.00000 (4)	0.00000 (4)	0.00000 (4)
BmX004	0.00000 (2)	**0.00300 (2)**	0.00000 (4)	0.00000 (4)

Roof Beam - 3 - Steel Beam.vap
Service Case: D
IES VisualAnalysis 20.00.0001
Friday, May 3, 2024





Factored Load Combinations

Name	Code	Effective Equation	Design	Deflection
1. 1.4D	ASCE 7-16 LRFD	1.4D	Strength	Other
2. 1.2D+1.6L+0.5Lr	ASCE 7-16 LRFD	1.2D	Strength	Other
5. 0.9D+W	ASCE 7-16 LRFD	0.9D	Strength	Other

Member Analysis Properties

Name	Member Section	X1 ft	Y1 ft	Z1 ft	X2 ft	Y2 ft	Z2 ft	Elasticity, E Ksi	A in^2	Iz in^4	Iy in^4	Releases	Action
BmX001	W10X45	0.00000	0.00000	0.00000	5.25000	0.00000	0.00000	29000.00000	13.30000	248.00000	53.40000	(none)	Normal
BmX003	W10X45	5.25000	0.00000	0.00000	35.75000	0.00000	0.00000	29000.00000	13.30000	248.00000	53.40000	(none)	Normal
BmX004	W10X45	35.75000	0.00000	0.00000	37.75000	0.00000	0.00000	29000.00000	13.30000	248.00000	53.40000	(none)	Normal

Section Properties

Section	Source	Area in^2	Iz in^4	Iy in^4	J in^4	Alpha, α deg
W10X45	Database Shape	13.30000	248.00000	53.40000	1.51000	0.00000

Member Stresses (extreme rows: max and min)

Member	fa Min Ksi	fa Max Ksi	fby Min Ksi	fby Max Ksi	fbz Min Ksi	fbz Max Ksi	fc comb Min Ksi	fc comb Max Ksi
BmX001	0.00000 (4)	0.00000 (4)	0.00000 (4)	0.00000 (4)	**-2.15607 (2)**	**2.15607 (2)**	**-2.15607 (2)**	**2.15607 (2)**
BmX003	0.00000 (4)	0.00000 (4)	0.00000 (4)	0.00000 (4)	**-13.42082 (2)**	**13.42082 (2)**	**-13.42082 (2)**	**13.42082 (2)**

Node Reactions (extreme rows: max and min)

Node	Result Case	FX K	FY K	FZ K	MX K-ft	MY K-ft	MZ K-ft
N002	1. 1.4D	0.00000	13.75145	0.00000	0.00000	0.00000	**46.09993**
N002	5. 0.9D+W	0.00000	8.84022	0.00000	0.00000	0.00000	**29.63567**
N003	1. 1.4D	0.00000	**-24.33682**	0.00000	0.00000	0.00000	0.00000
N004	1. 1.4D	0.00000	**36.04235**	0.00000	0.00000	0.00000	0.00000

Steel_Beam X_G 1: Results

Deflections Strong (dy): None Weak (dz): None	Axial Manual Kz: False Kz Sidesway?: False Manual Ky: False Ky Sidesway?: False	Size Constraints Limit Depth?: False Limit Width?: False
Overrides Override Fy?: False Override Cb?: False Override HSS t_des?: False Advanced Torsion: False		
Steel Material: ASTM A992 Grade 50 Specification: AISC 360-16 LRFD Composite Beam?: False Seismic Compactness: Not Ductile Check Constrained Axis FTB?: False Overstrength?: False Live Load Reduction: None Disable Checks?: False Check Level: Each Limit State	Bracing Lateral Top (+y): Unbraced Lateral Bottom (-y): Unbraced Strong (z): Unbraced	Torsional Bracing Lateral Top (+y): True Lateral Bottom (-y): True Strong (z): True

Steel_Beam X_G 1: Strong Flexure Check (extreme rows: max)

Member	Section	Offset ft	Result Case	Demand Mz K-ft	Capacity Mz K-ft	Code Reference	Unity Check	Details
BmX003	W10X45	0.00000	1. 1.4D	-54.92347	205.87500	F2-1	**0.26678**	Lb = 30.5 ft, Cb = 2.323

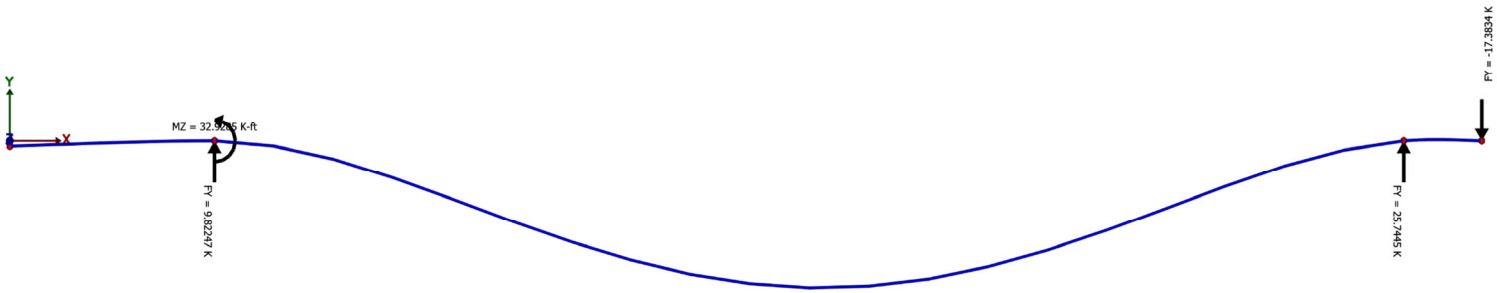
Steel_Beam X_G 1: Strong Shear Check (extreme rows: max)

Member	Section	Offset ft	Result Case	Demand Vy K	Capacity Vy K	Code Reference	Unity Check	Details
BmX004	W10X45	0.00000	1. 1.4D	25.54453	106.05000	G2-1	**0.24087**	Shear Area = 3.535 in^2, Cv = 1, h/tw = 22.4571

Member Displacements (extreme rows: max and min)

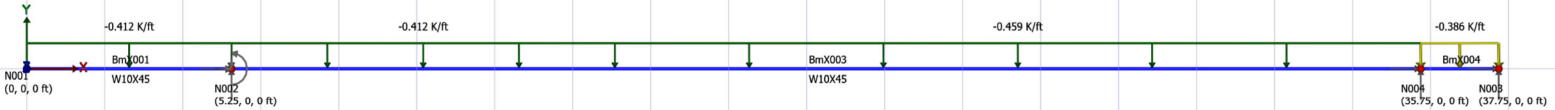
Member	Dy Min in	Dy Max in	Dz Min in	Dz Max in
BmX001	**-0.01461 (2)**	0.00000 (4)	0.00000 (4)	0.00000 (4)
BmX003	**-0.40626 (2)**	0.00000 (4)	0.00000 (4)	0.00000 (4)
BmX004	0.00000 (2)	**0.00305 (2)**	0.00000 (4)	0.00000 (4)

Roof Beam - 3 - Steel Beam.vap
Result Case: D
IES VisualAnalysis 20.00.0001
Friday, May 3, 2024





Roof Beam - 3 - Steel Beam.vap
Service Case: D
IES VisualAnalysis 20.00.0001
Friday, May 3, 2024





Factored Load Combinations

Name	Code	Effective Equation	Design	Deflection
1. 1.4D	ASCE 7-16 LRFD	1.4D	Strength	Other
2. 1.2D+1.6L+0.5Lr	ASCE 7-16 LRFD	1.2D	Strength	Other
5. 0.9D+W	ASCE 7-16 LRFD	0.9D	Strength	Other

Member Analysis Properties

Name	Member Section	X1 ft	Y1 ft	Z1 ft	X2 ft	Y2 ft	Z2 ft	Elasticity, E Ksi	A in^2	Iz in^4	Iy in^4	Releases	Action
BmX001	W10X54	0.00000	0.00000	0.00000	5.25000	0.00000	0.00000	29000.00000	15.80000	303.00000	103.00000	(none)	Normal
BmX003	W10X54	5.25000	0.00000	0.00000	13.92000	0.00000	0.00000	29000.00000	15.80000	303.00000	103.00000	(none)	Normal
BmX004	W10X54	13.92000	0.00000	0.00000	35.75000	0.00000	0.00000	29000.00000	15.80000	303.00000	103.00000	(none)	Normal

Section Properties

Section	Source	Area in^2	Iz in^4	Iy in^4	J in^4	Alpha, α deg
W10X54	Database Shape	15.80000	303.00000	103.00000	1.82000	0.00000

Member Stresses (extreme rows: max and min)

Member	fa Min Ksi	fa Max Ksi	fby Min Ksi	fby Max Ksi	fbz Min Ksi	fbz Max Ksi	fc comb Min Ksi	fc comb Max Ksi
BmX001	0.00000 (4)	0.00000 (4)	0.00000 (4)	0.00000 (4)	**-2.71597 (2)**	**2.71597 (2)**	**-2.71597 (2)**	**2.71597 (2)**
BmX003	0.00000 (4)	0.00000 (4)	0.00000 (4)	0.00000 (4)	**-8.96878 (2)**	**8.96878 (2)**	**-8.96878 (2)**	**8.96878 (2)**
BmX004	0.00000 (4)	0.00000 (4)	0.00000 (4)	0.00000 (4)	**-8.96878 (2)**	**8.96878 (2)**	**-8.96878 (2)**	**8.96878 (2)**

Node Reactions (extreme rows: max and min)

Node	Result Case	FX K	FY K	FZ K	MX K-ft	MY K-ft	MZ K-ft
N002	1. 1.4D	0.00000	2.75437	0.00000	0.00000	0.00000	**-26.74299**
N002	5. 0.9D+W	0.00000	**1.77066**	0.00000	0.00000	0.00000	**-17.19192**
N004	1. 1.4D	0.00000	**23.34403**	0.00000	0.00000	0.00000	0.00000

Steel_Beam X_G 1: Results

Deflections Strong (dy): None Weak (dz): None	Axial Manual Kz: False Kz Sidesway?: False Manual Ky: False Ky Sidesway?: False	Size Constraints Limit Depth?: False Limit Width?: False
Overrides Override Fy?: False Override Cb?: False Override HSS t_des?: False Advanced Torsion: False		
Steel Material: ASTM A992 Grade 50 Specification: AISC 360-16 LRFD Composite Beam?: False Seismic Compactness: Not Ductile Check Constrained Axis FTB?: False Overstrength?: False Live Load Reduction: None Disable Checks?: False Check Level: Each Limit State	Bracing Lateral Top (+y): Unbraced Lateral Bottom (-y): Unbraced Strong (z): Unbraced	Torsional Bracing Lateral Top (+y): True Lateral Bottom (-y): True Strong (z): True

Steel_Beam X_G 1: Strong Flexure Check

(extreme rows: max)

Member	Section	Offset ft	Result Case	Demand Mz K-ft	Capacity Mz K-ft	Code Reference	Unity Check	Details
BmX003	W10X54	8.67000	1. 1.4D	-44.84389	249.75000	F2-1	**0.17956**	Lb = 8.67 ft, Cb = 2.48639
BmX004	W10X54	0.00000	1. 1.4D	-44.84389	249.75000	F2-1	**0.17956**	Lb = 21.83 ft, Cb = 1.52805

Steel_Beam X_G 1: Strong Shear Check

(extreme rows: max)

Member	Section	Offset ft	Result Case	Demand Vy K	Capacity Vy K	Code Reference	Unity Check	Details
BmX004	W10X54	0.00000	1. 1.4D	12.38184	112.11000	G2-1	**0.11044**	Shear Area = 3.737 in^2, Cv = 1, h/tw = 21.2432

Member Displacements

(extreme rows: max and min)

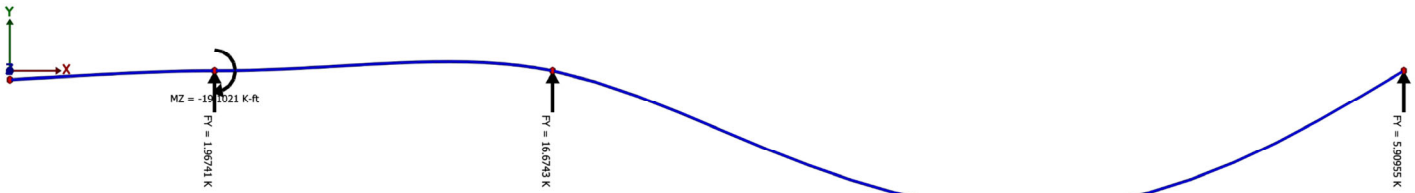
Member	Dy Min in	Dy Max in	Dz Min in	Dz Max in
BmX001	**-0.01840 (2)**	0.00000 (4)	0.00000 (4)	0.00000 (4)
BmX003	0.00000 (2)	**0.01900 (2)**	0.00000 (4)	0.00000 (4)

Member Displacements (continued)

(extreme rows: max and min)

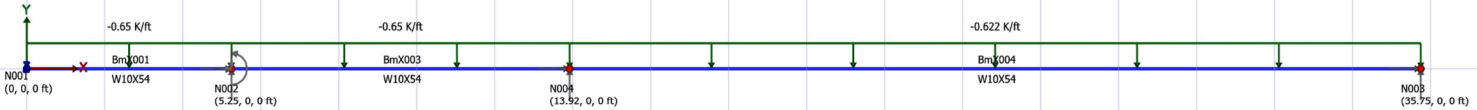
Member	Dy Min in	Dy Max in	Dz Min in	Dz Max in
BmX004	**-0.29221 (2)**	0.00000 (4)	0.00000 (4)	0.00000 (4)

Roof Beam - 4 - Steel Beam.vap
Result Case: D
IES VisualAnalysis 20.00.0001
Friday, May 3, 2024





Roof Beam - 4 - Steel Beam.vap
Service Case: D
IES VisualAnalysis 20.00.0001
Friday, May 3, 2024





ALTERNATIVE STEEL MEMBER SECTIONS:

W 8 x 67

- W10 x 54
- W12 x 50
- W14 X 43
- W16 X 40

W10 X 88

- W12 x 79
- W14 x 68
- W16 x 67
- W18 x55

W10 x 100

- W12 x 79
- W14 x 74
- W16 x 67
- W18 x 65

W16 x 26

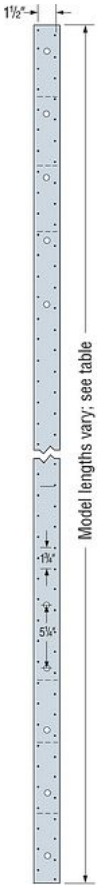
- W18 x 35

W16 x 89

- W18 x 86

MST Medium Strap Tie

Select Media



MST

Straight Straps

This product's information may differ depending on the category of use. You are currently viewing details related to **Straight Straps**. You can also view product information related to the category: Lateral Connectors, Ties and Straps for Cold-Formed Steel Construction

Product Details

Load Tables

The MST is designed to resist tension loads in floor-to-floor and other applications. This high-capacity strap features a dense nailing pattern and can be installed with either nails or bolts. The MST is a load-rated, precut strap designed for quick installation and optimal performance. It is suitable for connecting double 2x members or greater.

Key Features

- Can be used in conjunction with embedded purlin anchor (PA) straps for additional length
- Designed to resist wind and seismic forces

Material

- See table

Finish

- Galvanized. Some products are available in stainless steel, ZMAX® coating or black powder coat (add PC to SKU); see Corrosion Information.

Installation

- Use all specified fasteners; see General Notes

Related Links

- Wood Construction Connectors Technical and Installation Notes
- Cold-Formed Steel Connectors Technical and Installation Notes
- General Notes for Wood Construction Connectors
- General Notes for Cold-Formed Steel
- Corrosion Information

Catalog Pages

- C-C-2024 (Wood Construction Connectors), pages 278–279 and page 280
- C-CF-2023 (Connectors for Cold-Formed Steel Construction), pages 231–233

No Matches for Your Current Keyword Search

The Keyword Filter searches literature titles, descriptions and form numbers for the exact term you entered. Please double-check the spelling, or try a less specific term.

Product Information Table

Model No.	Thickness (mil)	Ga.	W (in.)	L (in.)	Coating/Material	Packaging Qty.
MST27	97	12	2 1/16	27	Zinc Galvanized, G90	1
MST27HDG	97	12	2 1/16	27	Hot-Dip Galvanized	1
MST37	97	12	2 1/16	37	Zinc Galvanized, G90	1
MST37HDG	97	12	2 1/16	37	Hot-Dip Galvanized	1
MST48	97	12	2 1/16	48	Zinc Galvanized, G90	1
MST48HDG	97	12	2 1/16	48	Hot-Dip Galvanized	1
MST60	118	10	2 1/16	60	Zinc Galvanized, G90	1
MST60HDG	118	10	2 1/16	60	Hot-Dip Galvanized	1
MST72	118	10	2 1/16	72	Zinc Galvanized, G90	1

Load Tables

Floor-to-Floor Span Table

These products are available with additional corrosion protection. Additional products on this page may also be available with this option, check with Simpson Strong-Tie for details.

SIMPSON Strong-Tie Many of these products are approved for installation with Strong-Drive® SD Connector screws.

Load Tables

Model No.	Clear Span (in.)	Fasteners (Total) (in.)	DF/SP Allowable Tension Loads (160)	SPF/HF Allowable Tension Loads (160)
MST37	24	(14) 0.162 x 2½	1,720	1,500
	18	(20) 0.162 x 2½	2,460	2,140
	16	(22) 0.162 x 2½	2,705	2,355
MST48	24	(26) 0.162 x 2½	3,210	2,780
	18	(32) 0.162 x 2½	3,950	3,425
	16	(34) 0.162 x 2½	4,200	3,640
MST60	30	(34) 0.162 x 2½	4,605	3,995
	24	(40) 0.162 x 2½	5,240	4,700
	18	(46) 0.162 x 2½	6,235	5,405
MST72	30	(48) 0.162 x 2½	6,505	5,640
	24	(54) 0.162 x 2½	6,730	6,345
	18	(62) 0.162 x 2½	6,730	6,475

See footnotes below.

Model No.	Ga.	Dimensions (in.)		Fasteners (Total)			DF/SP Allowable Tension Loads		SPF/HF Allowable Tension Loads	
		W	L	Nails (in.)	Bolts		Nails	Bolts	Nails	Bolts
					Qty.	Dia.	(160)	(160)	(160)	(160)
MST27	12	2⅛	27	(30) 0.162 x 2½	4	½	3,700	2,165	3,210	2,000
MST37		2⅛	37½	(42) 0.162 x 2½	6	½	5,070	3,030	4,495	2,800
MST48		2⅛	48	(50) 0.162 x 2½	8	½	5,310	3,675	5,190	3,395
MST60	10	2⅛	60	(68) 0.162 x 2½	10	½	6,730	4,490	6,475	4,150
MST72		2⅛	72	(68) 0.162 x 2½	10	½	6,730	4,490	6,475	4,150

1. See General Notes for Straps and Ties.
2. Install bolts or nails as specified by designer. Bolt and nail values may not be combined.
3. Allowable bolt loads are based on parallel-to-grain loading and minimum member thickness: MST — 2 1/2".
4. Splitting may be a problem with installations on lumber smaller than 3 1/2"; either fill every nail hole with 0.148" x 1 1/2" nails or fill every other hole with 0.162" x 2 1/2" nails. Reduce the allowable load based on the size and quantity of fasteners used.
5. Fasteners: Nail dimensions in the table are listed diameter by length. For additional information, see Fastener Types and Sizes Specified for Simpson Strong-Tie Connectors.

Load Values with Strong-Drive® SD Connector Screws

Model No.	Fasteners (Total)	(DF/SP) Allowable Tension Loads (160)	(SPF/HF) Allowable Tension Loads (160)
MST27	30-SD10112	4150	3310
MST37	40-SD10112	5070	4415
MST48	52-SD10112	5310	5035
MST60	68-SD10112	6765	6375
MST72	70-SD10112	6765	6375

These products are available with additional corrosion protection. Additional products on this page may also be available with this option, check with Simpson Strong-Tie for details.

Code Reports & Compliance

Product Series	Compliance/Certification	Report/Approval
MST	International Building Code International Residential Code City of Los Angeles Building Code City of Los Angeles Residential Code	ESR-3096
	International Building Code International Residential Code City of Los Angeles Building Code City of Los Angeles Residential Code	ESR-2105
	Florida Building Code	FL10456

Footnotes

1. Please review compliance documents for information about specific product models. In some cases, Compliance documents cover most but not all models associated with a product series.
2. For additional information regarding Florida's Statewide Product Approval System and Miami-Dade County Notice of Acceptance (NOA), click here.

Related Products



HRS
Strap Tie



MSTA
Medium Strap Tie



MSTC
Medium Strap Tie with Countersunk Nail Slots

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TIMBER FLOOR JOIST DESIGN: Second Level 10'-0" Span

Assumptions:

1. 2" x 8 Timber Floor Joists H.F. No. 2 or Better
2. Maximum span: **10** feet **(MAX)** = 120 inches
3. Support a simple span.
   Therefore: **L** = 120 inches
4. Floor Joists Spaced @ **1.50** feet o.c. = 18 inches
5. Minimum Bearing Area: 1 5" x 5.5" = **8.25** $inches^2$

2" x 8" Timber Properties:

Reference: ANSI National Design Specifications For Wood Construction

		ANSI Design Values
A	=	**10.875** $inches^2$
E	=	**1,700,000** PSI
I	=	**47.6** $inches^4$
S	=	**13.1** $inches^3$
$Fv_{Allowable}$	=	**145** PSI
$Fp_{Allowable}$	=	**405** PSI
Fb	=	**1300** PSI

Reactions:

W = Floor Load x Joist Spacing = 106.50 PLF

Where **W** =

Floor D.L.	=	12 PSF
Floor L.L.	=	40 PSF
Design Load = DL x 1.2 + LL x 1.4 =		70.4
Used in Calculations:		71 PSF

V_{max}	=	0.5**WL**	=	**533** lbs
P_{max}	=	0.5**WL**	=	**533** lbs
M_{max}	=	$(WL^2)/8$	=	**1,331** ft-lbs
D_{max}	=	$0.013WL^4 / EI$	=	**0.30** inches

Check Shear:

Fv	=	$V_{max}(1.5) / A$	=	**73.45** PSI
$Fv_{Allowable}$	=			**145** PSI

Check - O.K.

Check Bearing:

Fp	=	P_{max} / A	=	**64.55** PSI
$Fp_{Allowable}$	=			**405** PSI

Check - O.K.

Check Bending:

$S_{Req'd}$	=	$M_{max}(12) / Fb$	=	**12.29** $inches^3$
$S_{Furnished}$	=			**13.1** $inches^3$

Check - O.K.

Check Deflection:

$D_{max.}$	=			**0.2954** inches
$D_{Allowable}$	=	L/350	=	**0.3429** inches

Check - O.K.

DESIGN PROPERTIES

Design Properties (100% Load Duration)

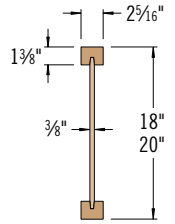
20'-6" SPAN

Depth	TJI	Basic Properties				Reaction Properties					
		Joist Weight (lbs/ft)	Maximum Resistive Moment(1) (ft-lbs)	Joist Only EI x 10^6 (lbs-in.^{2})	Maximum Vertical Shear (lbs)	1¾" End Reaction (lbs)		3½" Intermediate Reaction (lbs)		5¼" Intermediate Reaction (lbs)	
						No Web Stiffeners	With Web Stiffeners(2)	No Web Stiffeners	With Web Stiffeners(2)	No Web Stiffeners	With Web Stiffeners(2)
18"	360	3.7	9,465	1,085	2,425	1,080	1,440	2,460	2,815	3,000	3,360
	560	4.8	14,550	1,631	3,030	1,265	1,740	3,000	3,475	3,455	3,930
	560D	5.0	14,785	1,661	3,030	1,265	1,895	3,000	3,630	3,455	4,085
20"	360	4.0	10,515	1,376	2,660	1,080	1,440	2,460	2,815	3,000	3,360
	560	5.1	16,165	2,064	3,345	1,265	1,740	3,000	3,475	3,455	3,930
	560D	5.3	16,435	2,105	3,345	1,265	2,055	3,000	3,790	3,455	4,245
22"	560D	5.6	18,075	2,606	3,605	NA(3)	2,215	NA(3)	4,740	NA(3)	5,195
24"	560D	5.8	19,700	3,165	3,200	NA(3)	2,215	NA(3)	5,055	NA(3)	5,510

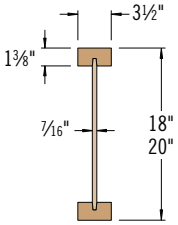
(1) **Caution: Do not** increase joist moment design properties by a repetitive-member-use factor.

(2) See detail W on page 5 for web stiffener requirements and nailing information.

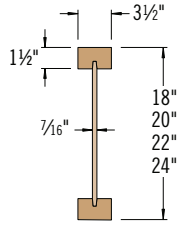
(3) Web stiffeners are required at all bearing locations for 22" and 24" TJI® 560D joists.



TJI® 360 Joist



TJI® 560 Joist



TJI® 560D Joist

General Notes

- Design reaction includes all loads on the joist. Design shear is computed at the inside face of supports and includes all loads on the span(s). Allowable shear may sometimes be increased at interior supports in accordance with ICC ES ESR-1153, and these increases are reflected in span tables.
- The following formulas approximate the uniform load deflection of Δ (inches):

For TJI® 360 Joists

$$\Delta = \frac{22.5\ wL^4}{EI} + \frac{2.67\ wL^2}{d \times 10^5}$$

For TJI® 560 and 560D Joists

$$\Delta = \frac{22.5\ wL^4}{EI} + \frac{2.29\ wL^2}{d \times 10^5}$$

w = uniform load in pounds per linear foot
L = span in feet
d = out-to-out depth of the joist in inches
EI = value from table above

TJI® joists are intended for dry-use applications

Some TJI® joist series may not be available in your region. Contact your Weyerhaeuser representative for information.

FLOOR SPAN TABLES

L/480 Live Load Deflection

Depth	TJI®	40 PSF Live Load / 10 PSF Dead Load			40 PSF Live Load / 25 PSF Dead Load		
		16" o.c.	19.2" o.c.	24" o.c.	16" o.c.	19.2" o.c.	24" o.c.
18"	360	28'-8"	26'-10"(1)	21'-5"(1)	24'-9"(1)	20'-7"(1)	16'-6"(1)
	560, 560D	32'-5"	30'-7"(1)	25'-2"(1)	***29'-1"(1)***	24'-2"(1)	19'-4"(1)
20"	360	31'-0"(1)	26'-10"(1)	21'-5"(1)	24'-9"(1)	20'-7"(1)	16'-6"(1)
	560, 560D	35'-1"	31'-6"(1)	25'-2"(1)	29'-1"(1)	24'-2"(1)	19'-4"(1)
22"	560D	37'-11"	35'-9"	33'-2"	***37'-11"***	***35'-9"***	29'-0"
24"	560D	40'-6"	38'-1"	35'-5"	***40'-6"***	***38'-1"***	***31'-0"***

L/360 Live Load Deflection (Minimum Criteria per Code)

Depth	TJI®	40 PSF Live Load / 10 PSF Dead Load			40 PSF Live Load / 25 PSF Dead Load		
		16" o.c.	19.2" o.c.	24" o.c.	16" o.c.	19.2" o.c.	24" o.c.
18"	360	31'-9"(1)	26'-10"(1)	21'-5"(1)	24'-9"(1)	20'-7"(1)	16'-6"(1)
	560, 560D	35'-11"(1)	31'-6"(1)	25'-2"(1)	***29'-1"(1)***	24'-2"(1)	19'-4"(1)
20"	360	32'-3"(1)	26'-10"(1)	21'-5"(1)	24'-9"(1)	20'-7"(1)	16'-6 "(1)
	560, 560D	37'-10"(1)	31'-6"(1)	25'-2"(1)	29'-1"(1)	24'-2"(1)	19'-4"(1)
22"	560D	***42'-0"***	***39'-7"***	36'-10"	***40'-8"***	***36'-4"***	29'-0"
24"	560D	***44'-9"***	***42'-3"***	39'-3"	***42'-6"***	***38'-9"***	***31'-0"***

(1) Web stiffeners are required at intermediate supports of continuous-span joists when the intermediate bearing length is **less** than 5¼" and the span on either side of the intermediate bearing is greater than the following spans:

Depth	TJI®	40 PSF Live Load / 10 PSF Dead Load			40 PSF Live Load / 25 PSF Dead Load		
		16" o.c.	19.2" o.c.	24" o.c.	16" o.c.	19.2" o.c.	24" o.c.
18" and 20"	360	29'-4"	24'-5"	19'-6"	22'-7"	18'-9"	15'-0"
	560, 560D	35'-10"	29'-10"	23'-10"	27'-7"	22'-11"	18'-4"

- Long term deflection under dead load, which includes the effect of creep, has not been considered. ***Bold italic*** spans reflect initial dead load deflection exceeding 0.33".

Live load deflection is not the only factor that affects how a floor will perform. To more accurately predict floor performance, use our TJ-Pro™ Ratings.

How to Use These Tables

1. Determine the appropriate live load deflection criteria.
2. Identify the live and dead load condition.
3. Select on-center spacing.
4. Scan down the column until you meet or exceed the span of your application.
5. Select TJI® joist and depth.

General Notes

- Tables are based on:
  - Uniform loads.
  - More restrictive of simple or continuous span.
  - Clear distance between supports
  - **18" and 20" TJI® joists:** Minimum bearing length of 1¾" end (no web stiffeners) and 3½" intermediate.
  - **22" and 24" TJI® joists:** Minimum bearing length of 1¾" end and 3½" intermediate; web stiffeners required at all bearings.
- Assumed composite action with a single layer of 24" on-center span-rated, glue-nailed floor panels for deflection only. **Spans shall be reduced 6" when floor panels are nailed only.**
- Spans generated from Weyerhaeuser software may exceed the spans shown in these tables because software reflects actual design conditions.
- For multi-family applications and other loading conditions not shown, refer to Weyerhaeuser software or to the load tables on page 7.

DESIGN PROPERTIES

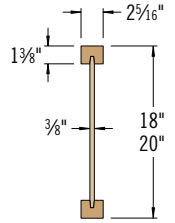
Design Properties (100% Load Duration)

Depth	TJI	Basic Properties				Reaction Properties					
		Joist Weight (lbs/ft)	Maximum Resistive Moment(1) (ft-lbs)	Joist Only EI x 10^6 (lbs-in.^{2})	Maximum Vertical Shear (lbs)	1¾" End Reaction (lbs)		3½" Intermediate Reaction (lbs)		5¼" Intermediate Reaction (lbs)	
						No Web Stiffeners	With Web Stiffeners(2)	No Web Stiffeners	With Web Stiffeners(2)	No Web Stiffeners	With Web Stiffeners(2)
18"	360	3.7	9,465	1,085	2,425	1,080	1,440	2,460	2,815	3,000	3,360
	560	4.8	14,550	1,631	3,030	1,265	1,740	3,000	3,475	3,455	3,930
	560D	5.0	14,785	1,661	3,030	1,265	1,895	3,000	3,630	3,455	4,085
20"	360	4.0	10,515	1,376	2,660	1,080	1,440	2,460	2,815	3,000	3,360
	560	5.1	16,165	2,064	3,345	1,265	1,740	3,000	3,475	3,455	3,930
	560D	5.3	16,435	2,105	3,345	1,265	2,055	3,000	3,790	3,455	4,245
22"	560D	5.6	18,075	2,606	3,605	NA(3)	2,215	NA(3)	4,740	NA(3)	5,195
24"	560D	5.8	19,700	3,165	3,200	NA(3)	2,215	NA(3)	5,055	NA(3)	5,510

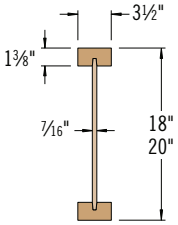
(1) **Caution: Do not** increase joist moment design properties by a repetitive-member-use factor.

(2) See detail W on page 5 for web stiffener requirements and nailing information.

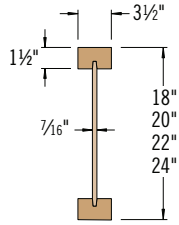
(3) Web stiffeners are required at all bearing locations for 22" and 24" TJI® 560D joists.



TJI® 360 Joist



TJI® 560 Joist



TJI® 560D Joist

General Notes

- Design reaction includes all loads on the joist. Design shear is computed at the inside face of supports and includes all loads on the span(s). Allowable shear may sometimes be increased at interior supports in accordance with ICC ES ESR-1153, and these increases are reflected in span tables.
- The following formulas approximate the uniform load deflection of Δ (inches):

For TJI® 360 Joists

$$\Delta = \frac{22.5\ wL^4}{EI} + \frac{2.67\ wL^2}{d \times 10^5}$$

For TJI® 560 and 560D Joists

$$\Delta = \frac{22.5\ wL^4}{EI} + \frac{2.29\ wL^2}{d \times 10^5}$$

w = uniform load in pounds per linear foot
L = span in feet
d = out-to-out depth of the joist in inches
EI = value from table above

TJI® joists are intended for dry-use applications

Some TJI® joist series may not be available in your region. Contact your Weyerhaeuser representative for information.

FLOOR SPAN TABLES

L/480 Live Load Deflection

Depth	TJI®	40 PSF Live Load / 10 PSF Dead Load			40 PSF Live Load / 25 PSF Dead Load		
		16" o.c.	19.2" o.c.	24" o.c.	16" o.c.	19.2" o.c.	24" o.c.
18"	360	28'-8"	26'-10"(1)	21'-5"(1)	24'-9"(1)	20'-7"(1)	16'-6"(1)
	560, 560D	32'-5"	30'-7"(1)	25'-2"(1)	***29'-1"(1)***	24'-2"(1)	19'-4"(1)
20"	360	31'-0"(1)	26'-10"(1)	21'-5"(1)	24'-9"(1)	20'-7"(1)	16'-6"(1)
	560, 560D	35'-1"	31'-6"(1)	25'-2"(1)	29'-1"(1)	24'-2"(1)	19'-4"(1)
22"	560D	37'-11"	35'-9"	33'-2"	***37'-11"***	***35'-9"***	29'-0"
24"	560D	40'-6"	38'-1"	35'-5"	***40'-6"***	***38'-1"***	***31'-0"***

L/360 Live Load Deflection (Minimum Criteria per Code)

Depth	TJI®	40 PSF Live Load / 10 PSF Dead Load			40 PSF Live Load / 25 PSF Dead Load		
		16" o.c.	19.2" o.c.	24" o.c.	16" o.c.	19.2" o.c.	24" o.c.
18"	360	31'-9"(1)	26'-10"(1)	21'-5"(1)	24'-9"(1)	20'-7"(1)	16'-6"(1)
	560, 560D	35'-11"(1)	31'-6"(1)	25'-2"(1)	***29'-1"(1)***	24'-2"(1)	19'-4"(1)
20"	360	32'-3"(1)	26'-10"(1)	21'-5"(1)	24'-9"(1)	20'-7"(1)	16'-6 "(1)
	560, 560D	37'-10"(1)	31'-6"(1)	25'-2"(1)	29'-1"(1)	24'-2"(1)	19'-4"(1)
22"	560D	***42'-0"***	***39'-7"***	36'-10"	***40'-8"***	***36'-4"***	29'-0"
24"	560D	***44'-9"***	***42'-3"***	39'-3"	***42'-6"***	***38'-9"***	***31'-0"***

(1) Web stiffeners are required at intermediate supports of continuous-span joists when the intermediate bearing length is **less** than 5¼" and the span on either side of the intermediate bearing is greater than the following spans:

Depth	TJI®	40 PSF Live Load / 10 PSF Dead Load			40 PSF Live Load / 25 PSF Dead Load		
		16" o.c.	19.2" o.c.	24" o.c.	16" o.c.	19.2" o.c.	24" o.c.
18" and 20"	360	29'-4"	24'-5"	19'-6"	22'-7"	18'-9"	15'-0"
	560, 560D	35'-10"	29'-10"	23'-10"	27'-7"	22'-11"	18'-4"

- Long term deflection under dead load, which includes the effect of creep, has not been considered. ***Bold italic*** spans reflect initial dead load deflection exceeding 0.33".

Live load deflection is not the only factor that affects how a floor will perform. To more accurately predict floor performance, use our TJ-Pro™ Ratings.

How to Use These Tables

1. Determine the appropriate live load deflection criteria.
2. Identify the live and dead load condition.
3. Select on-center spacing.
4. Scan down the column until you meet or exceed the span of your application.
5. Select TJI® joist and depth.

General Notes

- Tables are based on:
  - Uniform loads.
  - More restrictive of simple or continuous span.
  - Clear distance between supports
  - **18" and 20" TJI® joists:** Minimum bearing length of 1¾" end (no web stiffeners) and 3½" intermediate.
  - **22" and 24" TJI® joists:** Minimum bearing length of 1¾" end and 3½" intermediate; web stiffeners required at all bearings.
- Assumed composite action with a single layer of 24" on-center span-rated, glue-nailed floor panels for deflection only. **Spans shall be reduced 6" when floor panels are nailed only.**
- Spans generated from Weyerhaeuser software may exceed the spans shown in these tables because software reflects actual design conditions.
- For multi-family applications and other loading conditions not shown, refer to Weyerhaeuser software or to the load tables on page 7.

L/480 Live Load Deflection

Depth	TJI®	40 PSF Live Load / 10 PSF Dead Load				40 PSF Live Load / 20 PSF Dead Load			
		12" o.c.	16" o.c.	19.2" o.c.	24" o.c.	12" o.c.	16" o.c.	19.2" o.c.	24" o.c.
9½"	110	16'-11"	15'-6"	14'-7"	13'-7"	16'-11"	15'-6"	14'-3"	12'-9"
	210	17'-9"	16'-3"	15'-4"	14'-3"	17'-9"	16'-3"	15'-4"	14'-0"
	230	18'-3"	16'-8"	15'-9"	14'-8"	18'-3"	16'-8"	15'-9"	14'-8"
11⅞"	110	20'-2"	18'-5"	17'-4"	15'-9"(1)	20'-2"	17'-8"	16'-1"(1)	14'-4"(1)
	210	21'-1"	19'-3"	18'-2"	16'-11"	21'-1"	19'-3"	17'-8"	15'-9"(1)
	230	21'-8"	19'-10"	18'-8"	17'-5"	21'-8"	19'-10"	18'-7"	16'-7"(1)
	360	22'-11"	20'-11"	19'-8"	18'-4"	22'-11"	20'-11"	19'-8"	17'-10"(1)
	560	26'-1"	23'-8"	22'-4"	20'-9"	26'-1"	23'-8"	22'-4"	20'-9"(1)
14"	110	22'-10"	20'-11"	19'-2"	17'-2"(1)	22'-2"	19'-2"	17'-6"(1)	15'-0"(1)
	210	23'-11"	21'-10"	20'-8"	18'-10"(1)	23'-11"	21'-1"	19'-2"(1)	16'-7"(1)
	230	24'-8"	22'-6"	21'-2"	19'-9"(1)	24'-8"	22'-2"	20'-3"(1)	17'-6"(1)
	360	26'-0"	23'-8"	22'-4"	20'-9"(1)	26'-0"	23'-8"	22'-4"(1)	17'-10"(1)
	560	29'-6"	26'-10"	25'-4"	23'-6"	***29'-6"***	***26'-10"***	25'-4"(1)	20'-11"(1)
16"	110	25'-4"	22'-6"	20'-7"(1)	18'-1"(1)	23'-9"	20'-7"(1)	18'-9"(1)	15'-0"(1)
	210	26'-6"	24'-3"	22'-6"(1)	19'-11"(1)	26'-0"	22'-6"(1)	20'-7"(1)	16'-7"(1)
	230	27'-3"	24'-10"	23'-6"	21'-1"(1)	***27'-3"***	23'-9"	21'-8"(1)	17'-6"(1)
	360	28'-9"	26'-3"	24'-8"(1)	21'-5"(1)	***28'-9"***	26'-3"(1)	22'-4"(1)	17'-10"(1)
	560	32'-8"	29'-8"	28'-0"	25'-2"(1)	***32'-8"***	***29'-8"***	26'-3"(1)	20'-11"(1)

L/360 Live Load Deflection (Minimum Criteria per Code)

Depth	TJI®	40 PSF Live Load / 10 PSF Dead Load				40 PSF Live Load / 20 PSF Dead Load			
		12" o.c.	16" o.c.	19.2" o.c.	24" o.c.	12" o.c.	16" o.c.	19.2" o.c.	24" o.c.
9½"	110	18'-9"	17'-2"	15'-8"	14'-0"	18'-1"	15'-8"	14'-3"	12'-9"
	210	19'-8"	18'-0"	17'-0"	15'-4"	19'-8"	17'-2"	15'-8"	14'-0"
	230	20'-3"	18'-6"	17'-5"	16'-2"	***20'-3"***	18'-1"	16'-6"	14'-9"
11⅞"	110	22'-3"	19'-4"	17'-8"	15'-9"(1)	20'-5"	17'-8"	16'-1"(1)	14'-4"(1)
	210	23'-4"	21'-2"	19'-4"	17'-3"(1)	***22'-4"***	19'-4"	17'-8"	15'-9"(1)
	230	24'-0"	21'-11"	20'-5"	18'-3"	***23'-7"***	20'-5"	18'-7"	16'-7"(1)
	360	25'-4"	23'-2"	21'-10"	20'-4"(1)	***25'-4"***	***23'-2"***	***21'-10"***(1)	17'-10"(1)
	560	28'-10"	26'-3"	24'-9"	23'-0"	***28'-10"***	***26'-3"***	***24'-9"***	20'-11"(1)
14"	110	24'-4"	21'-0"	19'-2"	17'-2"(1)	22'-2"	19'-2"	17'-6"(1)	15'-0"(1)
	210	26'-6"	23'-1"	21'-1"	18'-10"(1)	24'-4"	21'-1"	19'-2"(1)	16'-7"(1)
	230	27'-3"	24'-4"	22'-2"	19'-10"(1)	***25'-8"***	22'-2"	20'-3"(1)	17'-6"(1)
	360	28'-9"	26'-3"	24'-9"(1)	21'-5"(1)	***28'-9"***	***26'-3"***(1)	22'-4"(1)	17'-10"(1)
	560	32'-8"	29'-9"	28'-0"	25'-2"(1)	***32'-8"***	***29'-9"***	***26'-3"***(1)	20'-11"(1)
16"	110	26'-0"	22'-6"	20'-7"(1)	18'-1"(1)	23'-9"	20'-7"(1)	18'-9"(1)	15'-0"(1)
	210	28'-6"	24'-8"	22'-6"(1)	19'-11"(1)	26'-0"	22'-6"(1)	20'-7"(1)	16'-7"(1)
	230	30'-1"	26'-0"	23'-9"	21'-1"(1)	***27'-5"***	23'-9"	21'-8"(1)	17'-6"(1)
	360	31'-10"	29'-0"	26'-10"(1)	21'-5"(1)	***31'-10"***	***26'-10"***(1)	22'-4"(1)	17'-10"(1)
	560	36'-1"	32'-11"	31'-0"(1)	25'-2"(1)	***36'-1"***	***31'-6"***(1)	26'-3"(1)	20'-11"(1)

(1) Web stiffeners are required at intermediate supports of continuous-span joists when the intermediate bearing length is ***less*** than 5¼" and the span on either side of the intermediate bearing is greater than the following spans:

TJI®	40 PSF Live Load / 10 PSF Dead Load				40 PSF Live Load / 20 PSF Dead Load			
	12" o.c.	16" o.c.	19.2" o.c.	24" o.c.	12" o.c.	16" o.c.	19.2" o.c.	24" o.c.
110	Not Req.	Not Req.	19'-2"	15'-4"	Not Req.	19'-2"	16'-0"	12'-9"
210			21'-4"	17'-0"		21'-4"	17'-9"	14'-2"
230			Not Req.	19'-2"		Not Req.	19'-11"	15'-11"
360			24'-5"	19'-6"		24'-5"	20'-4"	16'-3"
560			29'-10"	23'-10"		29'-10"	24'-10"	19'-10"

- Long-term deflection under dead load, which includes the effect of creep, has not been considered. Bold italic spans reflect initial dead load deflection exceeding 0.33".

How to Use These Tables

1. Determine the appropriate live load deflection criteria.
2. Identify the live and dead load condition.
3. Select on-center spacing.
4. Scan down the column until you meet or exceed the span of your application.
5. Select TJI® joist and depth.

General Notes

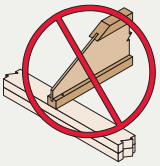
- Tables are based on:
  - Uniform loads.
  - More restrictive of simple or continuous span.
  - Clear distance between supports
  - Minimum bearing length of 1¾" end (no web stiffeners) and 3½" intermediate.
- Assumed composite action with a single layer of 24" on-center span-rated, glue-nailed floor panels for deflection only. **When subfloor adhesive is not applied, spans shall be reduced 6" for nails and 12" for proprietary fasteners.**
- For continuous spans, ratio of short span to long span should be 0.4 or greater to prevent uplift.
- Spans generated from Weyerhaeuser software may exceed the spans shown in these tables because software reflects actual design conditions.
- For multi-family applications and other loading conditions not shown, refer to Weyerhaeuser software or to the load table on page 8.

Live load deflection is not the only factor that affects how a floor will perform. To more accurately predict floor performance, use our TJ-Pro™ Ratings.

These Conditions Are NOT Permitted:



DO NOT use sawn lumber for rim board or blocking as it may shrink after installation. Use only engineered lumber



DO NOT bevel cut joist beyond inside face of wall.



DO NOT install hanger overhanging face of plate or beam. Flush bearing plate with inside face of wall or beam.

L/480 Live Load Deflection

Depth	TJI®	40 PSF Live Load / 10 PSF Dead Load				40 PSF Live Load / 20 PSF Dead Load			
		12" o.c.	16" o.c.	19.2" o.c.	24" o.c.	12" o.c.	16" o.c.	19.2" o.c.	24" o.c.
9½"	110	16'-11"	15'-6"	14'-7"	13'-7"	16'-11"	15'-6"	14'-3"	12'-9"
	210	17'-9"	16'-3"	15'-4"	14'-3"	17'-9"	16'-3"	15'-4"	14'-0"
	230	18'-3"	16'-8"	15'-9"	14'-8"	18'-3"	16'-8"	15'-9"	14'-8"
11⅞"	110	20'-2"	18'-5"	17'-4"	15'-9"(1)	20'-2"	17'-8"	16'-1"(1)	14'-4"(1)
	210	21'-1"	19'-3"	18'-2"	16'-11"	21'-1"	19'-3"	17'-8"	15'-9"(1)
	230	21'-8"	19'-10"	18'-8"	17'-5"	21'-8"	19'-10"	18'-7"	16'-7"(1)
	360	22'-11"	20'-11"	19'-8"	18'-4"	22'-11"	20'-11"	19'-8"	17'-10"(1)
	560	26'-1"	23'-8"	22'-4"	20'-9"	26'-1"	23'-8"	22'-4"	20'-9"(1)
14"	110	22'-10"	20'-11"	19'-2"	17'-2"(1)	22'-2"	19'-2"	17'-6"(1)	15'-0"(1)
	210	23'-11"	21'-10"	20'-8"	18'-10"(1)	23'-11"	21'-1"	19'-2"(1)	16'-7"(1)
	230	24'-8"	22'-6"	21'-2"	19'-9"(1)	24'-8"	22'-2"	20'-3"(1)	17'-6"(1)
	360	26'-0"	23'-8"	22'-4"	20'-9"(1)	26'-0"	23'-8"	22'-4"(1)	17'-10"(1)
	560	29'-6"	26'-10"	25'-4"	23'-6"	***29'-6"***	***26'-10"***	25'-4"(1)	20'-11"(1)
16"	110	25'-4"	22'-6"	20'-7"(1)	18'-1"(1)	23'-9"	20'-7"(1)	18'-9"(1)	15'-0"(1)
	210	26'-6"	24'-3"	22'-6"(1)	19'-11"(1)	26'-0"	22'-6"(1)	20'-7"(1)	16'-7"(1)
	230	27'-3"	24'-10"	23'-6"	21'-1"(1)	***27'-3"***	23'-9"	21'-8"(1)	17'-6"(1)
	360	28'-9"	26'-3"	24'-8"(1)	21'-5"(1)	***28'-9"***	26'-3"(1)	22'-4"(1)	17'-10"(1)
	560	32'-8"	29'-8"	28'-0"	25'-2"(1)	***32'-8"***	***29'-8"***	26'-3"(1)	20'-11"(1)

L/360 Live Load Deflection (Minimum Criteria per Code)

Depth	TJI®	40 PSF Live Load / 10 PSF Dead Load				40 PSF Live Load / 20 PSF Dead Load			
		12" o.c.	16" o.c.	19.2" o.c.	24" o.c.	12" o.c.	16" o.c.	19.2" o.c.	24" o.c.
9½"	110	18'-9"	17'-2"	15'-8"	14'-0"	18'-1"	15'-8"	14'-3"	12'-9"
	210	19'-8"	18'-0"	17'-0"	15'-4"	19'-8"	17'-2"	15'-8"	14'-0"
	230	20'-3"	18'-6"	17'-5"	16'-2"	***20'-3"***	18'-1"	16'-6"	14'-9"
11⅞"	110	22'-3"	19'-4"	17'-8"	15'-9"(1)	20'-5"	17'-8"	16'-1"(1)	14'-4"(1)
	210	23'-4"	21'-2"	19'-4"	17'-3"(1)	***22'-4"***	19'-4"	17'-8"	15'-9"(1)
	230	24'-0"	21'-11"	20'-5"	18'-3"	***23'-7"***	20'-5"	18'-7"	16'-7"(1)
	360	25'-4"	23'-2"	21'-10"	20'-4"(1)	***25'-4"***	***23'-2"***	***21'-10"***(1)	17'-10"(1)
	560	28'-10"	26'-3"	24'-9"	23'-0"	***28'-10"***	***26'-3"***	***24'-9"***	20'-11"(1)
14"	110	24'-4"	21'-0"	19'-2"	17'-2"(1)	22'-2"	19'-2"	17'-6"(1)	15'-0"(1)
	210	26'-6"	23'-1"	21'-1"	18'-10"(1)	24'-4"	21'-1"	19'-2"(1)	16'-7"(1)
	230	27'-3"	24'-4"	22'-2"	19'-10"(1)	***25'-8"***	22'-2"	20'-3"(1)	17'-6"(1)
	360	28'-9"	26'-3"	24'-9"(1)	21'-5"(1)	***28'-9"***	***26'-3"***(1)	22'-4"(1)	17'-10"(1)
	560	32'-8"	29'-9"	28'-0"	25'-2"(1)	***32'-8"***	***29'-9"***	***26'-3"***(1)	20'-11"(1)
16"	110	26'-0"	22'-6"	20'-7"(1)	18'-1"(1)	23'-9"	20'-7"(1)	18'-9"(1)	15'-0"(1)
	210	28'-6"	24'-8"	22'-6"(1)	19'-11"(1)	26'-0"	22'-6"(1)	20'-7"(1)	16'-7"(1)
	230	30'-1"	26'-0"	23'-9"	21'-1"(1)	***27'-5"***	23'-9"	21'-8"(1)	17'-6"(1)
	360	31'-10"	29'-0"	26'-10"(1)	21'-5"(1)	***31'-10"***	***26'-10"***(1)	22'-4"(1)	17'-10"(1)
	560	36'-1"	32'-11"	31'-0"(1)	25'-2"(1)	***36'-1"***	***31'-6"***(1)	26'-3"(1)	20'-11"(1)

(1) Web stiffeners are required at intermediate supports of continuous-span joists when the intermediate bearing length is ***less*** than 5¼" and the span on either side of the intermediate bearing is greater than the following spans:

TJI®	40 PSF Live Load / 10 PSF Dead Load				40 PSF Live Load / 20 PSF Dead Load			
	12" o.c.	16" o.c.	19.2" o.c.	24" o.c.	12" o.c.	16" o.c.	19.2" o.c.	24" o.c.
110	Not Req.	Not Req.	19'-2"	15'-4"	Not Req.	19'-2"	16'-0"	12'-9"
210			21'-4"	17'-0"		21'-4"	17'-9"	14'-2"
230			Not Req.	19'-2"		Not Req.	19'-11"	15'-11"
360			24'-5"	19'-6"		24'-5"	20'-4"	16'-3"
560			29'-10"	23'-10"		29'-10"	24'-10"	19'-10"

- Long-term deflection under dead load, which includes the effect of creep, has not been considered. Bold italic spans reflect initial dead load deflection exceeding 0.33".

How to Use These Tables

1. Determine the appropriate live load deflection criteria.
2. Identify the live and dead load condition.
3. Select on-center spacing.
4. Scan down the column until you meet or exceed the span of your application.
5. Select TJI® joist and depth.

General Notes

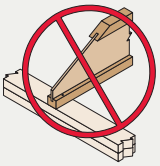
- Tables are based on:
  - Uniform loads.
  - More restrictive of simple or continuous span.
  - Clear distance between supports
  - Minimum bearing length of 1¾" end (no web stiffeners) and 3½" intermediate.
- Assumed composite action with a single layer of 24" on-center span-rated, glue-nailed floor panels for deflection only. **When subfloor adhesive is not applied, spans shall be reduced 6" for nails and 12" for proprietary fasteners.**
- For continuous spans, ratio of short span to long span should be 0.4 or greater to prevent uplift.
- Spans generated from Weyerhaeuser software may exceed the spans shown in these tables because software reflects actual design conditions.
- For multi-family applications and other loading conditions not shown, refer to Weyerhaeuser software or to the load table on page 8.

Live load deflection is not the only factor that affects how a floor will perform. To more accurately predict floor performance, use our TJ-Pro™ Ratings.

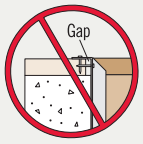
These Conditions Are NOT Permitted:



DO NOT use sawn lumber for rim board or blocking as it may shrink after installation. Use only engineered lumber



DO NOT bevel cut joist beyond inside face of wall.



DO NOT install hanger overhanging face of plate or beam. Flush bearing plate with inside face of wall or beam.

L/480 Live Load Deflection

Depth	TJI®	40 PSF Live Load / 10 PSF Dead Load				40 PSF Live Load / 20 PSF Dead Load			
		12" o.c.	16" o.c.	19.2" o.c.	24" o.c.	12" o.c.	16" o.c.	19.2" o.c.	24" o.c.
9½"	110	16'-11"	15'-6"	14'-7"	13'-7"	16'-11"	15'-6"	14'-3"	12'-9"
	210	17'-9"	16'-3"	15'-4"	14'-3"	17'-9"	16'-3"	15'-4"	14'-0"
	230	18'-3"	16'-8"	15'-9"	14'-8"	18'-3"	16'-8"	15'-9"	14'-8"
11⅞"	110	20'-2"	18'-5"	17'-4"	15'-9"(1)	20'-2"	17'-8"	16'-1"(1)	14'-4"(1)
	210	21'-1"	19'-3"	18'-2"	16'-11"	21'-1"	19'-3"	17'-8"	15'-9"(1)
	230	21'-8"	19'-10"	18'-8"	17'-5"	21'-8"	19'-10"	18'-7"	16'-7"(1)
	360	22'-11"	20'-11"	19'-8"	18'-4"	22'-11"	20'-11"	19'-8"	17'-10"(1)
	560	26'-1"	23'-8"	22'-4"	20'-9"	26'-1"	23'-8"	22'-4"	20'-9"(1)
14"	110	22'-10"	20'-11"	19'-2"	17'-2"(1)	22'-2"	19'-2"	17'-6"(1)	15'-0"(1)
	210	23'-11"	21'-10"	20'-8"	18'-10"(1)	23'-11"	21'-1"	19'-2"(1)	16'-7"(1)
	230	24'-8"	22'-6"	21'-2"	19'-9"(1)	24'-8"	22'-2"	20'-3"(1)	17'-6"(1)
	360	26'-0"	23'-8"	22'-4"	20'-9"(1)	26'-0"	23'-8"	22'-4"(1)	17'-10"(1)
	560	29'-6"	26'-10"	25'-4"	23'-6"	***29'-6"***	***26'-10"***	25'-4"(1)	20'-11"(1)
16"	110	25'-4"	22'-6"	20'-7"(1)	18'-1"(1)	23'-9"	20'-7"(1)	18'-9"(1)	15'-0"(1)
	210	26'-6"	24'-3"	22'-6"(1)	19'-11"(1)	26'-0"	22'-6"(1)	20'-7"(1)	16'-7"(1)
	230	27'-3"	24'-10"	23'-6"	21'-1"(1)	***27'-3"***	23'-9"	21'-8"(1)	17'-6"(1)
	360	28'-9"	26'-3"	24'-8"(1)	21'-5"(1)	***28'-9"***	26'-3"(1)	22'-4"(1)	17'-10"(1)
	560	32'-8"	29'-8"	28'-0"	25'-2"(1)	***32'-8"***	***29'-8"***	26'-3"(1)	20'-11"(1)

L/360 Live Load Deflection (Minimum Criteria per Code)

Depth	TJI®	40 PSF Live Load / 10 PSF Dead Load				40 PSF Live Load / 20 PSF Dead Load			
		12" o.c.	16" o.c.	19.2" o.c.	24" o.c.	12" o.c.	16" o.c.	19.2" o.c.	24" o.c.
9½"	110	18'-9"	17'-2"	15'-8"	14'-0"	18'-1"	15'-8"	14'-3"	12'-9"
	210	19'-8"	18'-0"	17'-0"	15'-4"	19'-8"	17'-2"	15'-8"	14'-0"
	230	20'-3"	18'-6"	17'-5"	16'-2"	***20'-3"***	18'-1"	16'-6"	14'-9"
11⅞"	110	22'-3"	19'-4"	17'-8"	15'-9"(1)	20'-5"	17'-8"	16'-1"(1)	14'-4"(1)
	210	23'-4"	21'-2"	19'-4"	17'-3"(1)	***22'-4"***	19'-4"	17'-8"	15'-9"(1)
	230	24'-0"	21'-11"	20'-5"	18'-3"	***23'-7"***	20'-5"	18'-7"	16'-7"(1)
	360	25'-4"	23'-2"	21'-10"	20'-4"(1)	***25'-4"***	***23'-2"***	***21'-10"***(1)	17'-10"(1)
	560	28'-10"	26'-3"	24'-9"	23'-0"	***28'-10"***	***26'-3"***	***24'-9"***	20'-11"(1)
14"	110	24'-4"	21'-0"	19'-2"	17'-2"(1)	22'-2"	19'-2"	17'-6"(1)	15'-0"(1)
	210	26'-6"	23'-1"	21'-1"	18'-10"(1)	24'-4"	21'-1"	19'-2"(1)	16'-7"(1)
	230	27'-3"	24'-4"	22'-2"	19'-10"(1)	***25'-8"***	22'-2"	20'-3"(1)	17'-6"(1)
	360	28'-9"	26'-3"	24'-9"(1)	21'-5"(1)	***28'-9"***	***26'-3"***(1)	22'-4"(1)	17'-10"(1)
	560	32'-8"	29'-9"	28'-0"	25'-2"(1)	***32'-8"***	***29'-9"***	***26'-3"***(1)	20'-11"(1)
16"	110	26'-0"	22'-6"	20'-7"(1)	18'-1"(1)	23'-9"	20'-7"(1)	18'-9"(1)	15'-0"(1)
	210	28'-6"	24'-8"	22'-6"(1)	19'-11"(1)	26'-0"	22'-6"(1)	20'-7"(1)	16'-7"(1)
	230	30'-1"	26'-0"	23'-9"	21'-1"(1)	***27'-5"***	23'-9"	21'-8"(1)	17'-6"(1)
	360	31'-10"	29'-0"	26'-10"(1)	21'-5"(1)	***31'-10"***	***26'-10"***(1)	22'-4"(1)	17'-10"(1)
	560	36'-1"	32'-11"	31'-0"(1)	25'-2"(1)	***36'-1"***	***31'-6"***(1)	26'-3"(1)	20'-11"(1)

(1) Web stiffeners are required at intermediate supports of continuous-span joists when the intermediate bearing length is ***less*** than 5¼" and the span on either side of the intermediate bearing is greater than the following spans:

TJI®	40 PSF Live Load / 10 PSF Dead Load				40 PSF Live Load / 20 PSF Dead Load			
	12" o.c.	16" o.c.	19.2" o.c.	24" o.c.	12" o.c.	16" o.c.	19.2" o.c.	24" o.c.
110	Not Req.	Not Req.	19'-2"	15'-4"	Not Req.	19'-2"	16'-0"	12'-9"
210			21'-4"	17'-0"		21'-4"	17'-9"	14'-2"
230			Not Req.	19'-2"		Not Req.	19'-11"	15'-11"
360			24'-5"	19'-6"		24'-5"	20'-4"	16'-3"
560			29'-10"	23'-10"		29'-10"	24'-10"	19'-10"

- Long-term deflection under dead load, which includes the effect of creep, has not been considered. Bold italic spans reflect initial dead load deflection exceeding 0.33".

How to Use These Tables

1. Determine the appropriate live load deflection criteria.
2. Identify the live and dead load condition.
3. Select on-center spacing.
4. Scan down the column until you meet or exceed the span of your application.
5. Select TJI® joist and depth.

General Notes

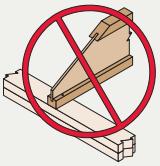
- Tables are based on:
  - Uniform loads.
  - More restrictive of simple or continuous span.
  - Clear distance between supports
  - Minimum bearing length of 1¾" end (no web stiffeners) and 3½" intermediate.
- Assumed composite action with a single layer of 24" on-center span-rated, glue-nailed floor panels for deflection only. **When subfloor adhesive is not applied, spans shall be reduced 6" for nails and 12" for proprietary fasteners.**
- For continuous spans, ratio of short span to long span should be 0.4 or greater to prevent uplift.
- Spans generated from Weyerhaeuser software may exceed the spans shown in these tables because software reflects actual design conditions.
- For multi-family applications and other loading conditions not shown, refer to Weyerhaeuser software or to the load table on page 8.

Live load deflection is not the only factor that affects how a floor will perform. To more accurately predict floor performance, use our TJ-Pro™ Ratings.

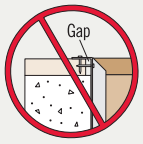
These Conditions Are NOT Permitted:



DO NOT use sawn lumber for rim board or blocking as it may shrink after installation. Use only engineered lumber



DO NOT bevel cut joist beyond inside face of wall.



DO NOT install hanger overhanging face of plate or beam. Flush bearing plate with inside face of wall or beam.

L/480 Live Load Deflection

Depth	TJI®	40 PSF Live Load / 10 PSF Dead Load				40 PSF Live Load / 20 PSF Dead Load			
		12" o.c.	16" o.c.	19.2" o.c.	24" o.c.	12" o.c.	16" o.c.	19.2" o.c.	24" o.c.
9½"	110	16'-11"	15'-6"	14'-7"	13'-7"	16'-11"	15'-6"	14'-3"	12'-9"
	210	17'-9"	16'-3"	15'-4"	14'-3"	17'-9"	16'-3"	15'-4"	14'-0"
	230	18'-3"	16'-8"	15'-9"	14'-8"	18'-3"	16'-8"	15'-9"	14'-8"
11⅞"	110	20'-2"	18'-5"	17'-4"	15'-9"(1)	20'-2"	17'-8"	16'-1"(1)	14'-4"(1)
	210	21'-1"	19'-3"	18'-2"	16'-11"	21'-1"	19'-3"	17'-8"	15'-9"(1)
	230	21'-8"	19'-10"	18'-8"	17'-5"	21'-8"	19'-10"	18'-7"	16'-7"(1)
	360	22'-11"	20'-11"	19'-8"	18'-4"	22'-11"	20'-11"	19'-8"	17'-10"(1)
	560	26'-1"	23'-8"	22'-4"	20'-9"	26'-1"	23'-8"	22'-4"	20'-9"(1)
14"	110	22'-10"	20'-11"	19'-2"	17'-2"(1)	22'-2"	19'-2"	17'-6"(1)	15'-0"(1)
	210	23'-11"	21'-10"	20'-8"	18'-10"(1)	23'-11"	21'-1"	19'-2"(1)	16'-7"(1)
	230	24'-8"	22'-6"	21'-2"	19'-9"(1)	24'-8"	22'-2"	20'-3"(1)	17'-6"(1)
	360	26'-0"	23'-8"	22'-4"	20'-9"(1)	26'-0"	23'-8"	22'-4"(1)	17'-10"(1)
	560	29'-6"	26'-10"	25'-4"	23'-6"	***29'-6"***	***26'-10"***	25'-4"(1)	20'-11"(1)
16"	110	25'-4"	22'-6"	20'-7"(1)	18'-1"(1)	23'-9"	20'-7"(1)	18'-9"(1)	15'-0"(1)
	210	26'-6"	24'-3"	22'-6"(1)	19'-11"(1)	26'-0"	22'-6"(1)	20'-7"(1)	16'-7"(1)
	230	27'-3"	24'-10"	23'-6"	21'-1"(1)	***27'-3"***	23'-9"	21'-8"(1)	17'-6"(1)
	360	28'-9"	26'-3"	24'-8"(1)	21'-5"(1)	***28'-9"***	26'-3"(1)	22'-4"(1)	17'-10"(1)
	560	32'-8"	29'-8"	28'-0"	25'-2"(1)	***32'-8"***	***29'-8"***	26'-3"(1)	20'-11"(1)

L/360 Live Load Deflection (Minimum Criteria per Code)

Depth	TJI®	40 PSF Live Load / 10 PSF Dead Load				40 PSF Live Load / 20 PSF Dead Load			
		12" o.c.	16" o.c.	19.2" o.c.	24" o.c.	12" o.c.	16" o.c.	19.2" o.c.	24" o.c.
9½"	110	18'-9"	17'-2"	15'-8"	14'-0"	18'-1"	15'-8"	14'-3"	12'-9"
	210	19'-8"	18'-0"	17'-0"	15'-4"	19'-8"	17'-2"	15'-8"	14'-0"
	230	20'-3"	18'-6"	17'-5"	16'-2"	***20'-3"***	18'-1"	16'-6"	14'-9"
11⅞"	110	22'-3"	19'-4"	17'-8"	15'-9"(1)	20'-5"	17'-8"	16'-1"(1)	14'-4"(1)
	210	23'-4"	21'-2"	19'-4"	17'-3"(1)	***22'-4"***	19'-4"	17'-8"	15'-9"(1)
	230	24'-0"	21'-11"	20'-5"	18'-3"	***23'-7"***	20'-5"	18'-7"	16'-7"(1)
	360	25'-4"	23'-2"	21'-10"	20'-4"(1)	***25'-4"***	***23'-2"***	***21'-10"***(1)	17'-10"(1)
	560	28'-10"	26'-3"	24'-9"	23'-0"	***28'-10"***	***26'-3"***	***24'-9"***	20'-11"(1)
14"	110	24'-4"	21'-0"	19'-2"	17'-2"(1)	22'-2"	19'-2"	17'-6"(1)	15'-0"(1)
	210	26'-6"	23'-1"	21'-1"	18'-10"(1)	24'-4"	21'-1"	19'-2"(1)	16'-7"(1)
	230	27'-3"	24'-4"	22'-2"	19'-10"(1)	***25'-8"***	22'-2"	20'-3"(1)	17'-6"(1)
	360	28'-9"	26'-3"	24'-9"(1)	21'-5"(1)	***28'-9"***	***26'-3"***(1)	22'-4"(1)	17'-10"(1)
	560	32'-8"	29'-9"	28'-0"	25'-2"(1)	***32'-8"***	***29'-9"***	***26'-3"***(1)	20'-11"(1)
16"	110	26'-0"	22'-6"	20'-7"(1)	18'-1"(1)	23'-9"	20'-7"(1)	18'-9"(1)	15'-0"(1)
	210	28'-6"	24'-8"	22'-6"(1)	19'-11"(1)	26'-0"	22'-6"(1)	20'-7"(1)	16'-7"(1)
	230	30'-1"	26'-0"	23'-9"	21'-1"(1)	***27'-5"***	23'-9"	21'-8"(1)	17'-6"(1)
	360	31'-10"	29'-0"	26'-10"(1)	21'-5"(1)	***31'-10"***	***26'-10"***(1)	22'-4"(1)	17'-10"(1)
	560	36'-1"	32'-11"	31'-0"(1)	25'-2"(1)	***36'-1"***	***31'-6"***(1)	26'-3"(1)	20'-11"(1)

(1) Web stiffeners are required at intermediate supports of continuous-span joists when the intermediate bearing length is ***less*** than 5¼" and the span on either side of the intermediate bearing is greater than the following spans:

TJI®	40 PSF Live Load / 10 PSF Dead Load				40 PSF Live Load / 20 PSF Dead Load			
	12" o.c.	16" o.c.	19.2" o.c.	24" o.c.	12" o.c.	16" o.c.	19.2" o.c.	24" o.c.
110	Not Req.	Not Req.	19'-2"	15'-4"	Not Req.	19'-2"	16'-0"	12'-9"
210			21'-4"	17'-0"		21'-4"	17'-9"	14'-2"
230			Not Req.	19'-2"		Not Req.	19'-11"	15'-11"
360			24'-5"	19'-6"		24'-5"	20'-4"	16'-3"
560			29'-10"	23'-10"		29'-10"	24'-10"	19'-10"

- Long-term deflection under dead load, which includes the effect of creep, has not been considered. Bold italic spans reflect initial dead load deflection exceeding 0.33".

How to Use These Tables

1. Determine the appropriate live load deflection criteria.
2. Identify the live and dead load condition.
3. Select on-center spacing.
4. Scan down the column until you meet or exceed the span of your application.
5. Select TJI® joist and depth.

General Notes

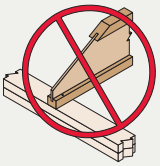
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  - Uniform loads.
  - More restrictive of simple or continuous span.
  - Clear distance between supports
  - Minimum bearing length of 1¾" end (no web stiffeners) and 3½" intermediate.
- Assumed composite action with a single layer of 24" on-center span-rated, glue-nailed floor panels for deflection only. **When subfloor adhesive is not applied, spans shall be reduced 6" for nails and 12" for proprietary fasteners.**
- For continuous spans, ratio of short span to long span should be 0.4 or greater to prevent uplift.
- Spans generated from Weyerhaeuser software may exceed the spans shown in these tables because software reflects actual design conditions.
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L/480 Live Load Deflection

Depth	TJI®	40 PSF Live Load / 10 PSF Dead Load				40 PSF Live Load / 20 PSF Dead Load			
		12" o.c.	16" o.c.	19.2" o.c.	24" o.c.	12" o.c.	16" o.c.	19.2" o.c.	24" o.c.
9½"	110	16'-11"	15'-6"	14'-7"	13'-7"	16'-11"	15'-6"	14'-3"	12'-9"
	210	17'-9"	16'-3"	15'-4"	14'-3"	17'-9"	16'-3"	15'-4"	14'-0"
	230	18'-3"	16'-8"	15'-9"	14'-8"	18'-3"	16'-8"	15'-9"	14'-8"
11⅞"	110	20'-2"	18'-5"	17'-4"	15'-9"(1)	20'-2"	17'-8"	16'-1"(1)	14'-4"(1)
	210	21'-1"	19'-3"	18'-2"	16'-11"	21'-1"	19'-3"	17'-8"	15'-9"(1)
	230	21'-8"	19'-10"	18'-8"	17'-5"	21'-8"	19'-10"	18'-7"	16'-7"(1)
	360	22'-11"	20'-11"	19'-8"	18'-4"	22'-11"	20'-11"	19'-8"	17'-10"(1)
	560	26'-1"	23'-8"	22'-4"	20'-9"	26'-1"	23'-8"	22'-4"	20'-9"(1)
14"	110	22'-10"	20'-11"	19'-2"	17'-2"(1)	22'-2"	19'-2"	17'-6"(1)	15'-0"(1)
	210	23'-11"	21'-10"	20'-8"	18'-10"(1)	23'-11"	21'-1"	19'-2"(1)	16'-7"(1)
	230	24'-8"	22'-6"	21'-2"	19'-9"(1)	24'-8"	22'-2"	20'-3"(1)	17'-6"(1)
	360	26'-0"	23'-8"	22'-4"	20'-9"(1)	26'-0"	23'-8"	22'-4"(1)	17'-10"(1)
	560	29'-6"	26'-10"	25'-4"	23'-6"	***29'-6"***	***26'-10"***	25'-4"(1)	20'-11"(1)
16"	110	25'-4"	22'-6"	20'-7"(1)	18'-1"(1)	23'-9"	20'-7"(1)	18'-9"(1)	15'-0"(1)
	210	26'-6"	24'-3"	22'-6"(1)	19'-11"(1)	26'-0"	22'-6"(1)	20'-7"(1)	16'-7"(1)
	230	27'-3"	24'-10"	23'-6"	21'-1"(1)	***27'-3"***	23'-9"	21'-8"(1)	17'-6"(1)
	360	28'-9"	26'-3"	24'-8"(1)	21'-5"(1)	***28'-9"***	26'-3"(1)	22'-4"(1)	17'-10"(1)
	560	32'-8"	29'-8"	28'-0"	25'-2"(1)	***32'-8"***	***29'-8"***	26'-3"(1)	20'-11"(1)

L/360 Live Load Deflection (Minimum Criteria per Code)

Depth	TJI®	40 PSF Live Load / 10 PSF Dead Load				40 PSF Live Load / 20 PSF Dead Load			
		12" o.c.	16" o.c.	19.2" o.c.	24" o.c.	12" o.c.	16" o.c.	19.2" o.c.	24" o.c.
9½"	110	18'-9"	17'-2"	15'-8"	14'-0"	18'-1"	15'-8"	14'-3"	12'-9"
	210	19'-8"	18'-0"	17'-0"	15'-4"	19'-8"	17'-2"	15'-8"	14'-0"
	230	20'-3"	18'-6"	17'-5"	16'-2"	***20'-3"***	18'-1"	16'-6"	14'-9"
11⅞"	110	22'-3"	19'-4"	17'-8"	15'-9"(1)	20'-5"	17'-8"	16'-1"(1)	14'-4"(1)
	210	23'-4"	21'-2"	19'-4"	17'-3"(1)	***22'-4"***	19'-4"	17'-8"	15'-9"(1)
	230	24'-0"	21'-11"	20'-5"	18'-3"	***23'-7"***	20'-5"	18'-7"	16'-7"(1)
	360	25'-4"	23'-2"	21'-10"	20'-4"(1)	***25'-4"***	***23'-2"***	***21'-10"***(1)	17'-10"(1)
	560	28'-10"	26'-3"	24'-9"	23'-0"	***28'-10"***	***26'-3"***	***24'-9"***	20'-11"(1)
14"	110	24'-4"	21'-0"	19'-2"	17'-2"(1)	22'-2"	19'-2"	17'-6"(1)	15'-0"(1)
	210	26'-6"	23'-1"	21'-1"	18'-10"(1)	24'-4"	21'-1"	19'-2"(1)	16'-7"(1)
	230	27'-3"	24'-4"	22'-2"	19'-10"(1)	***25'-8"***	22'-2"	20'-3"(1)	17'-6"(1)
	360	28'-9"	26'-3"	24'-9"(1)	21'-5"(1)	***28'-9"***	***26'-3"***(1)	22'-4"(1)	17'-10"(1)
	560	32'-8"	29'-9"	28'-0"	25'-2"(1)	***32'-8"***	***29'-9"***	***26'-3"***(1)	20'-11"(1)
16"	110	26'-0"	22'-6"	20'-7"(1)	18'-1"(1)	23'-9"	20'-7"(1)	18'-9"(1)	15'-0"(1)
	210	28'-6"	24'-8"	22'-6"(1)	19'-11"(1)	26'-0"	22'-6"(1)	20'-7"(1)	16'-7"(1)
	230	30'-1"	26'-0"	23'-9"	21'-1"(1)	***27'-5"***	23'-9"	21'-8"(1)	17'-6"(1)
	360	31'-10"	29'-0"	26'-10"(1)	21'-5"(1)	***31'-10"***	***26'-10"***(1)	22'-4"(1)	17'-10"(1)
	560	36'-1"	32'-11"	31'-0"(1)	25'-2"(1)	***36'-1"***	***31'-6"***(1)	26'-3"(1)	20'-11"(1)

(1) Web stiffeners are required at intermediate supports of continuous-span joists when the intermediate bearing length is ***less*** than 5¼" and the span on either side of the intermediate bearing is greater than the following spans:

TJI®	40 PSF Live Load / 10 PSF Dead Load				40 PSF Live Load / 20 PSF Dead Load			
	12" o.c.	16" o.c.	19.2" o.c.	24" o.c.	12" o.c.	16" o.c.	19.2" o.c.	24" o.c.
110	Not Req.	Not Req.	19'-2"	15'-4"	Not Req.	19'-2"	16'-0"	12'-9"
210			21'-4"	17'-0"		21'-4"	17'-9"	14'-2"
230			Not Req.	19'-2"		Not Req.	19'-11"	15'-11"
360			24'-5"	19'-6"		24'-5"	20'-4"	16'-3"
560			29'-10"	23'-10"		29'-10"	24'-10"	19'-10"

- Long-term deflection under dead load, which includes the effect of creep, has not been considered. Bold italic spans reflect initial dead load deflection exceeding 0.33".

How to Use These Tables

1. Determine the appropriate live load deflection criteria.
2. Identify the live and dead load condition.
3. Select on-center spacing.
4. Scan down the column until you meet or exceed the span of your application.
5. Select TJI® joist and depth.

General Notes

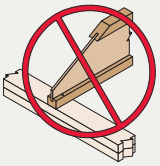
- Tables are based on:
  - Uniform loads.
  - More restrictive of simple or continuous span.
  - Clear distance between supports
  - Minimum bearing length of 1¾" end (no web stiffeners) and 3½" intermediate.
- Assumed composite action with a single layer of 24" on-center span-rated, glue-nailed floor panels for deflection only. **When subfloor adhesive is not applied, spans shall be reduced 6" for nails and 12" for proprietary fasteners.**
- For continuous spans, ratio of short span to long span should be 0.4 or greater to prevent uplift.
- Spans generated from Weyerhaeuser software may exceed the spans shown in these tables because software reflects actual design conditions.
- For multi-family applications and other loading conditions not shown, refer to Weyerhaeuser software or to the load table on page 8.

Live load deflection is not the only factor that affects how a floor will perform. To more accurately predict floor performance, use our TJ-Pro™ Ratings.

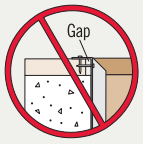
These Conditions Are NOT Permitted:



DO NOT use sawn lumber for rim board or blocking as it may shrink after installation. Use only engineered lumber



DO NOT bevel cut joist beyond inside face of wall.



DO NOT install hanger overhanging face of plate or beam. Flush bearing plate with inside face of wall or beam.

L/480 Live Load Deflection

Depth	TJI®	40 PSF Live Load / 10 PSF Dead Load				40 PSF Live Load / 20 PSF Dead Load			
		12" o.c.	16" o.c.	19.2" o.c.	24" o.c.	12" o.c.	16" o.c.	19.2" o.c.	24" o.c.
9½"	110	16'-11"	15'-6"	14'-7"	13'-7"	16'-11"	15'-6"	14'-3"	12'-9"
	210	17'-9"	16'-3"	15'-4"	14'-3"	17'-9"	16'-3"	15'-4"	14'-0"
	230	18'-3"	16'-8"	15'-9"	14'-8"	18'-3"	16'-8"	15'-9"	14'-8"
11⅞"	110	20'-2"	18'-5"	17'-4"	15'-9"(1)	20'-2"	17'-8"	16'-1"(1)	14'-4"(1)
	210	21'-1"	19'-3"	18'-2"	16'-11"	21'-1"	19'-3"	17'-8"	15'-9"(1)
	230	21'-8"	19'-10"	18'-8"	17'-5"	21'-8"	19'-10"	18'-7"	16'-7"(1)
	360	22'-11"	20'-11"	19'-8"	18'-4"	22'-11"	20'-11"	19'-8"	17'-10"(1)
	560	26'-1"	23'-8"	22'-4"	20'-9"	26'-1"	23'-8"	22'-4"	20'-9"(1)
14"	110	22'-10"	20'-11"	19'-2"	17'-2"(1)	22'-2"	19'-2"	17'-6"(1)	15'-0"(1)
	210	23'-11"	21'-10"	20'-8"	18'-10"(1)	23'-11"	21'-1"	19'-2"(1)	16'-7"(1)
	230	24'-8"	22'-6"	21'-2"	19'-9"(1)	24'-8"	22'-2"	20'-3"(1)	17'-6"(1)
	360	26'-0"	23'-8"	22'-4"	20'-9"(1)	26'-0"	23'-8"	22'-4"(1)	17'-10"(1)
	560	29'-6"	26'-10"	25'-4"	23'-6"	***29'-6"***	***26'-10"***	25'-4"(1)	20'-11"(1)
16"	110	25'-4"	22'-6"	20'-7"(1)	18'-1"(1)	23'-9"	20'-7"(1)	18'-9"(1)	15'-0"(1)
	210	26'-6"	24'-3"	22'-6"(1)	19'-11"(1)	26'-0"	22'-6"(1)	20'-7"(1)	16'-7"(1)
	230	27'-3"	24'-10"	23'-6"	21'-1"(1)	***27'-3"***	23'-9"	21'-8"(1)	17'-6"(1)
	360	28'-9"	26'-3"	24'-8"(1)	21'-5"(1)	***28'-9"***	26'-3"(1)	22'-4"(1)	17'-10"(1)
	560	32'-8"	29'-8"	28'-0"	25'-2"(1)	***32'-8"***	***29'-8"***	26'-3"(1)	20'-11"(1)

L/360 Live Load Deflection (Minimum Criteria per Code)

Depth	TJI®	40 PSF Live Load / 10 PSF Dead Load				40 PSF Live Load / 20 PSF Dead Load			
		12" o.c.	16" o.c.	19.2" o.c.	24" o.c.	12" o.c.	16" o.c.	19.2" o.c.	24" o.c.
9½"	110	18'-9"	17'-2"	15'-8"	14'-0"	18'-1"	15'-8"	14'-3"	12'-9"
	210	19'-8"	18'-0"	17'-0"	15'-4"	19'-8"	17'-2"	15'-8"	14'-0"
	230	20'-3"	18'-6"	17'-5"	16'-2"	***20'-3"***	18'-1"	16'-6"	14'-9"
11⅞"	110	22'-3"	19'-4"	17'-8"	15'-9"(1)	20'-5"	17'-8"	16'-1"(1)	14'-4"(1)
	210	23'-4"	21'-2"	19'-4"	17'-3"(1)	***22'-4"***	19'-4"	17'-8"	15'-9"(1)
	230	24'-0"	21'-11"	20'-5"	18'-3"	***23'-7"***	20'-5"	18'-7"	16'-7"(1)
	360	25'-4"	23'-2"	21'-10"	20'-4"(1)	***25'-4"***	***23'-2"***	***21'-10"***(1)	17'-10"(1)
	560	28'-10"	26'-3"	24'-9"	23'-0"	***28'-10"***	***26'-3"***	***24'-9"***	20'-11"(1)
14"	110	24'-4"	21'-0"	19'-2"	17'-2"(1)	22'-2"	19'-2"	17'-6"(1)	15'-0"(1)
	210	26'-6"	23'-1"	21'-1"	18'-10"(1)	24'-4"	21'-1"	19'-2"(1)	16'-7"(1)
	230	27'-3"	24'-4"	22'-2"	19'-10"(1)	***25'-8"***	22'-2"	20'-3"(1)	17'-6"(1)
	360	28'-9"	26'-3"	24'-9"(1)	21'-5"(1)	***28'-9"***	***26'-3"***(1)	22'-4"(1)	17'-10"(1)
	560	32'-8"	29'-9"	28'-0"	25'-2"(1)	***32'-8"***	***29'-9"***	***26'-3"***(1)	20'-11"(1)
16"	110	26'-0"	22'-6"	20'-7"(1)	18'-1"(1)	23'-9"	20'-7"(1)	18'-9"(1)	15'-0"(1)
	210	28'-6"	24'-8"	22'-6"(1)	19'-11"(1)	26'-0"	22'-6"(1)	20'-7"(1)	16'-7"(1)
	230	30'-1"	26'-0"	23'-9"	21'-1"(1)	***27'-5"***	23'-9"	21'-8"(1)	17'-6"(1)
	360	31'-10"	29'-0"	26'-10"(1)	21'-5"(1)	***31'-10"***	***26'-10"***(1)	22'-4"(1)	17'-10"(1)
	560	36'-1"	32'-11"	31'-0"(1)	25'-2"(1)	***36'-1"***	***31'-6"***(1)	26'-3"(1)	20'-11"(1)

(1) Web stiffeners are required at intermediate supports of continuous-span joists when the intermediate bearing length is ***less*** than 5¼" and the span on either side of the intermediate bearing is greater than the following spans:

TJI®	40 PSF Live Load / 10 PSF Dead Load				40 PSF Live Load / 20 PSF Dead Load			
	12" o.c.	16" o.c.	19.2" o.c.	24" o.c.	12" o.c.	16" o.c.	19.2" o.c.	24" o.c.
110	Not Req.	Not Req.	19'-2"	15'-4"	Not Req.	19'-2"	16'-0"	12'-9"
210			21'-4"	17'-0"		21'-4"	17'-9"	14'-2"
230			Not Req.	19'-2"		Not Req.	19'-11"	15'-11"
360			24'-5"	19'-6"		24'-5"	20'-4"	16'-3"
560			29'-10"	23'-10"		29'-10"	24'-10"	19'-10"

- Long-term deflection under dead load, which includes the effect of creep, has not been considered. Bold italic spans reflect initial dead load deflection exceeding 0.33".

How to Use These Tables

1. Determine the appropriate live load deflection criteria.
2. Identify the live and dead load condition.
3. Select on-center spacing.
4. Scan down the column until you meet or exceed the span of your application.
5. Select TJI® joist and depth.

General Notes

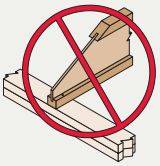
- Tables are based on:
  - Uniform loads.
  - More restrictive of simple or continuous span.
  - Clear distance between supports
  - Minimum bearing length of 1¾" end (no web stiffeners) and 3½" intermediate.
- Assumed composite action with a single layer of 24" on-center span-rated, glue-nailed floor panels for deflection only. **When subfloor adhesive is not applied, spans shall be reduced 6" for nails and 12" for proprietary fasteners.**
- For continuous spans, ratio of short span to long span should be 0.4 or greater to prevent uplift.
- Spans generated from Weyerhaeuser software may exceed the spans shown in these tables because software reflects actual design conditions.
- For multi-family applications and other loading conditions not shown, refer to Weyerhaeuser software or to the load table on page 8.

Live load deflection is not the only factor that affects how a floor will perform. To more accurately predict floor performance, use our TJ-Pro™ Ratings.

These Conditions Are NOT Permitted:



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DO NOT bevel cut joist beyond inside face of wall.



DO NOT install hanger overhanging face of plate or beam. Flush bearing plate with inside face of wall or beam.

L/480 Live Load Deflection

Depth	TJI®	40 PSF Live Load / 10 PSF Dead Load				40 PSF Live Load / 20 PSF Dead Load			
		12" o.c.	16" o.c.	19.2" o.c.	24" o.c.	12" o.c.	16" o.c.	19.2" o.c.	24" o.c.
9½"	110	16'-11"	15'-6"	14'-7"	13'-7"	16'-11"	15'-6"	14'-3"	12'-9"
	210	17'-9"	16'-3"	15'-4"	14'-3"	17'-9"	16'-3"	15'-4"	14'-0"
	230	18'-3"	16'-8"	15'-9"	14'-8"	18'-3"	16'-8"	15'-9"	14'-8"
11⅞"	110	20'-2"	18'-5"	17'-4"	15'-9"(1)	20'-2"	17'-8"	16'-1"(1)	14'-4"(1)
	210	21'-1"	19'-3"	18'-2"	16'-11"	21'-1"	19'-3"	17'-8"	15'-9"(1)
	230	21'-8"	19'-10"	18'-8"	17'-5"	21'-8"	19'-10"	18'-7"	16'-7"(1)
	360	22'-11"	20'-11"	19'-8"	18'-4"	22'-11"	20'-11"	19'-8"	17'-10"(1)
	560	26'-1"	23'-8"	22'-4"	20'-9"	26'-1"	23'-8"	22'-4"	20'-9"(1)
14"	110	22'-10"	20'-11"	19'-2"	17'-2"(1)	22'-2"	19'-2"	17'-6"(1)	15'-0"(1)
	210	23'-11"	21'-10"	20'-8"	18'-10"(1)	23'-11"	21'-1"	19'-2"(1)	16'-7"(1)
	230	24'-8"	22'-6"	21'-2"	19'-9"(1)	24'-8"	22'-2"	20'-3"(1)	17'-6"(1)
	360	26'-0"	23'-8"	22'-4"	20'-9"(1)	26'-0"	23'-8"	22'-4"(1)	17'-10"(1)
	560	29'-6"	26'-10"	25'-4"	23'-6"	***29'-6"***	***26'-10"***	25'-4"(1)	20'-11"(1)
16"	110	25'-4"	22'-6"	20'-7"(1)	18'-1"(1)	23'-9"	20'-7"(1)	18'-9"(1)	15'-0"(1)
	210	26'-6"	24'-3"	22'-6"(1)	19'-11"(1)	26'-0"	22'-6"(1)	20'-7"(1)	16'-7"(1)
	230	27'-3"	24'-10"	23'-6"	21'-1"(1)	***27'-3"***	23'-9"	21'-8"(1)	17'-6"(1)
	360	28'-9"	26'-3"	24'-8"(1)	21'-5"(1)	***28'-9"***	26'-3"(1)	22'-4"(1)	17'-10"(1)
	560	32'-8"	29'-8"	28'-0"	25'-2"(1)	***32'-8"***	***29'-8"***	26'-3"(1)	20'-11"(1)

L/360 Live Load Deflection (Minimum Criteria per Code)

Depth	TJI®	40 PSF Live Load / 10 PSF Dead Load				40 PSF Live Load / 20 PSF Dead Load			
		12" o.c.	16" o.c.	19.2" o.c.	24" o.c.	12" o.c.	16" o.c.	19.2" o.c.	24" o.c.
9½"	110	18'-9"	17'-2"	15'-8"	14'-0"	18'-1"	15'-8"	14'-3"	12'-9"
	210	19'-8"	18'-0"	17'-0"	15'-4"	19'-8"	17'-2"	15'-8"	14'-0"
	230	20'-3"	18'-6"	17'-5"	16'-2"	***20'-3"***	18'-1"	16'-6"	14'-9"
11⅞"	110	22'-3"	19'-4"	17'-8"	15'-9"(1)	20'-5"	17'-8"	16'-1"(1)	14'-4"(1)
	210	23'-4"	21'-2"	19'-4"	17'-3"(1)	***22'-4"***	19'-4"	17'-8"	15'-9"(1)
	230	24'-0"	21'-11"	20'-5"	18'-3"	***23'-7"***	20'-5"	18'-7"	16'-7"(1)
	360	25'-4"	23'-2"	21'-10"	20'-4"(1)	***25'-4"***	***23'-2"***	***21'-10"***(1)	17'-10"(1)
	560	28'-10"	26'-3"	24'-9"	23'-0"	***28'-10"***	***26'-3"***	***24'-9"***	20'-11"(1)
14"	110	24'-4"	21'-0"	19'-2"	17'-2"(1)	22'-2"	19'-2"	17'-6"(1)	15'-0"(1)
	210	26'-6"	23'-1"	21'-1"	18'-10"(1)	24'-4"	21'-1"	19'-2"(1)	16'-7"(1)
	230	27'-3"	24'-4"	22'-2"	19'-10"(1)	***25'-8"***	22'-2"	20'-3"(1)	17'-6"(1)
	360	28'-9"	26'-3"	24'-9"(1)	21'-5"(1)	***28'-9"***	***26'-3"***(1)	22'-4"(1)	17'-10"(1)
	560	32'-8"	29'-9"	28'-0"	25'-2"(1)	***32'-8"***	***29'-9"***	***26'-3"***(1)	20'-11"(1)
16"	110	26'-0"	22'-6"	20'-7"(1)	18'-1"(1)	23'-9"	20'-7"(1)	18'-9"(1)	15'-0"(1)
	210	28'-6"	24'-8"	22'-6"(1)	19'-11"(1)	26'-0"	22'-6"(1)	20'-7"(1)	16'-7"(1)
	230	30'-1"	26'-0"	23'-9"	21'-1"(1)	***27'-5"***	23'-9"	21'-8"(1)	17'-6"(1)
	360	31'-10"	29'-0"	26'-10"(1)	21'-5"(1)	***31'-10"***	***26'-10"***(1)	22'-4"(1)	17'-10"(1)
	560	36'-1"	32'-11"	31'-0"(1)	25'-2"(1)	***36'-1"***	***31'-6"***(1)	26'-3"(1)	20'-11"(1)

(1) Web stiffeners are required at intermediate supports of continuous-span joists when the intermediate bearing length is ***less*** than 5¼" and the span on either side of the intermediate bearing is greater than the following spans:

TJI®	40 PSF Live Load / 10 PSF Dead Load				40 PSF Live Load / 20 PSF Dead Load			
	12" o.c.	16" o.c.	19.2" o.c.	24" o.c.	12" o.c.	16" o.c.	19.2" o.c.	24" o.c.
110	Not Req.	Not Req.	19'-2"	15'-4"	Not Req.	19'-2"	16'-0"	12'-9"
210			21'-4"	17'-0"		21'-4"	17'-9"	14'-2"
230			Not Req.	19'-2"		Not Req.	19'-11"	15'-11"
360			24'-5"	19'-6"		24'-5"	20'-4"	16'-3"
560			29'-10"	23'-10"		29'-10"	24'-10"	19'-10"

- Long-term deflection under dead load, which includes the effect of creep, has not been considered. Bold italic spans reflect initial dead load deflection exceeding 0.33".

How to Use These Tables

1. Determine the appropriate live load deflection criteria.
2. Identify the live and dead load condition.
3. Select on-center spacing.
4. Scan down the column until you meet or exceed the span of your application.
5. Select TJI® joist and depth.

General Notes

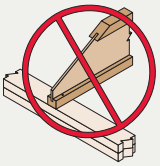
- Tables are based on:
  - Uniform loads.
  - More restrictive of simple or continuous span.
  - Clear distance between supports
  - Minimum bearing length of 1¾" end (no web stiffeners) and 3½" intermediate.
- Assumed composite action with a single layer of 24" on-center span-rated, glue-nailed floor panels for deflection only. **When subfloor adhesive is not applied, spans shall be reduced 6" for nails and 12" for proprietary fasteners.**
- For continuous spans, ratio of short span to long span should be 0.4 or greater to prevent uplift.
- Spans generated from Weyerhaeuser software may exceed the spans shown in these tables because software reflects actual design conditions.
- For multi-family applications and other loading conditions not shown, refer to Weyerhaeuser software or to the load table on page 8.

Live load deflection is not the only factor that affects how a floor will perform. To more accurately predict floor performance, use our TJ-Pro™ Ratings.

These Conditions Are NOT Permitted:



DO NOT use sawn lumber for rim board or blocking as it may shrink after installation. Use only engineered lumber



DO NOT bevel cut joist beyond inside face of wall.



DO NOT install hanger overhanging face of plate or beam. Flush bearing plate with inside face of wall or beam.

L/480 Live Load Deflection

Depth	TJI®	40 PSF Live Load / 10 PSF Dead Load				40 PSF Live Load / 20 PSF Dead Load			
		12" o.c.	16" o.c.	19.2" o.c.	24" o.c.	12" o.c.	16" o.c.	19.2" o.c.	24" o.c.
9½"	110	16'-11"	15'-6"	14'-7"	13'-7"	16'-11"	15'-6"	14'-3"	12'-9"
	210	17'-9"	16'-3"	15'-4"	14'-3"	17'-9"	16'-3"	15'-4"	14'-0"
	230	18'-3"	16'-8"	15'-9"	14'-8"	18'-3"	16'-8"	15'-9"	14'-8"
11⅞"	110	20'-2"	18'-5"	17'-4"	15'-9"(1)	20'-2"	17'-8"	16'-1"(1)	14'-4"(1)
	210	21'-1"	19'-3"	18'-2"	16'-11"	21'-1"	19'-3"	17'-8"	15'-9"(1)
	230	21'-8"	19'-10"	18'-8"	17'-5"	21'-8"	19'-10"	18'-7"	16'-7"(1)
	360	22'-11"	20'-11"	19'-8"	18'-4"	22'-11"	20'-11"	19'-8"	17'-10"(1)
	560	26'-1"	23'-8"	22'-4"	20'-9"	26'-1"	23'-8"	22'-4"	20'-9"(1)
14"	110	22'-10"	20'-11"	19'-2"	17'-2"(1)	22'-2"	19'-2"	17'-6"(1)	15'-0"(1)
	210	23'-11"	21'-10"	20'-8"	18'-10"(1)	23'-11"	21'-1"	19'-2"(1)	16'-7"(1)
	230	24'-8"	22'-6"	21'-2"	19'-9"(1)	24'-8"	22'-2"	20'-3"(1)	17'-6"(1)
	360	26'-0"	23'-8"	22'-4"	20'-9"(1)	26'-0"	23'-8"	22'-4"(1)	17'-10"(1)
	560	29'-6"	26'-10"	25'-4"	23'-6"	***29'-6"***	***26'-10"***	25'-4"(1)	20'-11"(1)
16"	110	25'-4"	22'-6"	20'-7"(1)	18'-1"(1)	23'-9"	20'-7"(1)	18'-9"(1)	15'-0"(1)
	210	26'-6"	24'-3"	22'-6"(1)	19'-11"(1)	26'-0"	22'-6"(1)	20'-7"(1)	16'-7"(1)
	230	27'-3"	24'-10"	23'-6"	21'-1"(1)	***27'-3"***	23'-9"	21'-8"(1)	17'-6"(1)
	360	28'-9"	26'-3"	24'-8"(1)	21'-5"(1)	***28'-9"***	26'-3"(1)	22'-4"(1)	17'-10"(1)
	560	32'-8"	29'-8"	28'-0"	25'-2"(1)	***32'-8"***	***29'-8"***	26'-3"(1)	20'-11"(1)

L/360 Live Load Deflection (Minimum Criteria per Code)

Depth	TJI®	40 PSF Live Load / 10 PSF Dead Load				40 PSF Live Load / 20 PSF Dead Load			
		12" o.c.	16" o.c.	19.2" o.c.	24" o.c.	12" o.c.	16" o.c.	19.2" o.c.	24" o.c.
9½"	110	18'-9"	17'-2"	15'-8"	14'-0"	18'-1"	15'-8"	14'-3"	12'-9"
	210	19'-8"	18'-0"	17'-0"	15'-4"	19'-8"	17'-2"	15'-8"	14'-0"
	230	20'-3"	18'-6"	17'-5"	16'-2"	***20'-3"***	18'-1"	16'-6"	14'-9"
11⅞"	110	22'-3"	19'-4"	17'-8"	15'-9"(1)	20'-5"	17'-8"	16'-1"(1)	14'-4"(1)
	210	23'-4"	21'-2"	19'-4"	17'-3"(1)	***22'-4"***	19'-4"	17'-8"	15'-9"(1)
	230	24'-0"	21'-11"	20'-5"	18'-3"	***23'-7"***	20'-5"	18'-7"	16'-7"(1)
	360	25'-4"	23'-2"	21'-10"	20'-4"(1)	***25'-4"***	***23'-2"***	***21'-10"***(1)	17'-10"(1)
	560	28'-10"	26'-3"	24'-9"	23'-0"	***28'-10"***	***26'-3"***	***24'-9"***	20'-11"(1)
14"	110	24'-4"	21'-0"	19'-2"	17'-2"(1)	22'-2"	19'-2"	17'-6"(1)	15'-0"(1)
	210	26'-6"	23'-1"	21'-1"	18'-10"(1)	24'-4"	21'-1"	19'-2"(1)	16'-7"(1)
	230	27'-3"	24'-4"	22'-2"	19'-10"(1)	***25'-8"***	22'-2"	20'-3"(1)	17'-6"(1)
	360	28'-9"	26'-3"	24'-9"(1)	21'-5"(1)	***28'-9"***	***26'-3"***(1)	22'-4"(1)	17'-10"(1)
	560	32'-8"	29'-9"	28'-0"	25'-2"(1)	***32'-8"***	***29'-9"***	***26'-3"***(1)	20'-11"(1)
16"	110	26'-0"	22'-6"	20'-7"(1)	18'-1"(1)	23'-9"	20'-7"(1)	18'-9"(1)	15'-0"(1)
	210	28'-6"	24'-8"	22'-6"(1)	19'-11"(1)	26'-0"	22'-6"(1)	20'-7"(1)	16'-7"(1)
	230	30'-1"	26'-0"	23'-9"	21'-1"(1)	***27'-5"***	23'-9"	21'-8"(1)	17'-6"(1)
	360	31'-10"	29'-0"	26'-10"(1)	21'-5"(1)	***31'-10"***	***26'-10"***(1)	22'-4"(1)	17'-10"(1)
	560	36'-1"	32'-11"	31'-0"(1)	25'-2"(1)	***36'-1"***	***31'-6"***(1)	26'-3"(1)	20'-11"(1)

(1) Web stiffeners are required at intermediate supports of continuous-span joists when the intermediate bearing length is ***less*** than 5¼" and the span on either side of the intermediate bearing is greater than the following spans:

TJI®	40 PSF Live Load / 10 PSF Dead Load				40 PSF Live Load / 20 PSF Dead Load			
	12" o.c.	16" o.c.	19.2" o.c.	24" o.c.	12" o.c.	16" o.c.	19.2" o.c.	24" o.c.
110	Not Req.	Not Req.	19'-2"	15'-4"	Not Req.	19'-2"	16'-0"	12'-9"
210			21'-4"	17'-0"		21'-4"	17'-9"	14'-2"
230			Not Req.	19'-2"		Not Req.	19'-11"	15'-11"
360			24'-5"	19'-6"		24'-5"	20'-4"	16'-3"
560			29'-10"	23'-10"		29'-10"	24'-10"	19'-10"

- Long-term deflection under dead load, which includes the effect of creep, has not been considered. Bold italic spans reflect initial dead load deflection exceeding 0.33".

How to Use These Tables

1. Determine the appropriate live load deflection criteria.
2. Identify the live and dead load condition.
3. Select on-center spacing.
4. Scan down the column until you meet or exceed the span of your application.
5. Select TJI® joist and depth.

General Notes

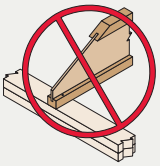
- Tables are based on:
  - Uniform loads.
  - More restrictive of simple or continuous span.
  - Clear distance between supports
  - Minimum bearing length of 1¾" end (no web stiffeners) and 3½" intermediate.
- Assumed composite action with a single layer of 24" on-center span-rated, glue-nailed floor panels for deflection only. **When subfloor adhesive is not applied, spans shall be reduced 6" for nails and 12" for proprietary fasteners.**
- For continuous spans, ratio of short span to long span should be 0.4 or greater to prevent uplift.
- Spans generated from Weyerhaeuser software may exceed the spans shown in these tables because software reflects actual design conditions.
- For multi-family applications and other loading conditions not shown, refer to Weyerhaeuser software or to the load table on page 8.

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DO NOT bevel cut joist beyond inside face of wall.



DO NOT install hanger overhanging face of plate or beam. Flush bearing plate with inside face of wall or beam.

L/480 Live Load Deflection

Depth	TJI®	40 PSF Live Load / 10 PSF Dead Load				40 PSF Live Load / 20 PSF Dead Load			
		12" o.c.	16" o.c.	19.2" o.c.	24" o.c.	12" o.c.	16" o.c.	19.2" o.c.	24" o.c.
9½"	110	16'-11"	15'-6"	14'-7"	13'-7"	16'-11"	15'-6"	14'-3"	12'-9"
	210	17'-9"	16'-3"	15'-4"	14'-3"	17'-9"	16'-3"	15'-4"	14'-0"
	230	18'-3"	16'-8"	15'-9"	14'-8"	18'-3"	16'-8"	15'-9"	14'-8"
11⅞"	110	20'-2"	18'-5"	17'-4"	15'-9"(1)	20'-2"	17'-8"	16'-1"(1)	14'-4"(1)
	210	21'-1"	19'-3"	18'-2"	16'-11"	21'-1"	19'-3"	17'-8"	15'-9"(1)
	230	21'-8"	19'-10"	18'-8"	17'-5"	21'-8"	19'-10"	18'-7"	16'-7"(1)
	360	22'-11"	20'-11"	19'-8"	18'-4"	22'-11"	20'-11"	19'-8"	17'-10"(1)
	560	26'-1"	23'-8"	22'-4"	20'-9"	26'-1"	23'-8"	22'-4"	20'-9"(1)
14"	110	22'-10"	20'-11"	19'-2"	17'-2"(1)	22'-2"	19'-2"	17'-6"(1)	15'-0"(1)
	210	23'-11"	21'-10"	20'-8"	18'-10"(1)	23'-11"	21'-1"	19'-2"(1)	16'-7"(1)
	230	24'-8"	22'-6"	21'-2"	19'-9"(1)	24'-8"	22'-2"	20'-3"(1)	17'-6"(1)
	360	26'-0"	23'-8"	22'-4"	20'-9"(1)	26'-0"	23'-8"	22'-4"(1)	17'-10"(1)
	560	29'-6"	26'-10"	25'-4"	23'-6"	***29'-6"***	***26'-10"***	25'-4"(1)	20'-11"(1)
16"	110	25'-4"	22'-6"	20'-7"(1)	18'-1"(1)	23'-9"	20'-7"(1)	18'-9"(1)	15'-0"(1)
	210	26'-6"	24'-3"	22'-6"(1)	19'-11"(1)	26'-0"	22'-6"(1)	20'-7"(1)	16'-7"(1)
	230	27'-3"	24'-10"	23'-6"	21'-1"(1)	***27'-3"***	23'-9"	21'-8"(1)	17'-6"(1)
	360	28'-9"	26'-3"	24'-8"(1)	21'-5"(1)	***28'-9"***	26'-3"(1)	22'-4"(1)	17'-10"(1)
	560	32'-8"	29'-8"	28'-0"	25'-2"(1)	***32'-8"***	***29'-8"***	26'-3"(1)	20'-11"(1)

L/360 Live Load Deflection (Minimum Criteria per Code)

Depth	TJI®	40 PSF Live Load / 10 PSF Dead Load				40 PSF Live Load / 20 PSF Dead Load			
		12" o.c.	16" o.c.	19.2" o.c.	24" o.c.	12" o.c.	16" o.c.	19.2" o.c.	24" o.c.
9½"	110	18'-9"	17'-2"	15'-8"	14'-0"	18'-1"	15'-8"	14'-3"	12'-9"
	210	19'-8"	18'-0"	17'-0"	15'-4"	19'-8"	17'-2"	15'-8"	14'-0"
	230	20'-3"	18'-6"	17'-5"	16'-2"	***20'-3"***	18'-1"	16'-6"	14'-9"
11⅞"	110	22'-3"	19'-4"	17'-8"	15'-9"(1)	20'-5"	17'-8"	16'-1"(1)	14'-4"(1)
	210	23'-4"	21'-2"	19'-4"	17'-3"(1)	***22'-4"***	19'-4"	17'-8"	15'-9"(1)
	230	24'-0"	21'-11"	20'-5"	18'-3"	***23'-7"***	20'-5"	18'-7"	16'-7"(1)
	360	25'-4"	23'-2"	21'-10"	20'-4"(1)	***25'-4"***	***23'-2"***	***21'-10"***(1)	17'-10"(1)
	560	28'-10"	26'-3"	24'-9"	23'-0"	***28'-10"***	***26'-3"***	***24'-9"***	20'-11"(1)
14"	110	24'-4"	21'-0"	19'-2"	17'-2"(1)	22'-2"	19'-2"	17'-6"(1)	15'-0"(1)
	210	26'-6"	23'-1"	21'-1"	18'-10"(1)	24'-4"	21'-1"	19'-2"(1)	16'-7"(1)
	230	27'-3"	24'-4"	22'-2"	19'-10"(1)	***25'-8"***	22'-2"	20'-3"(1)	17'-6"(1)
	360	28'-9"	26'-3"	24'-9"(1)	21'-5"(1)	***28'-9"***	***26'-3"***(1)	22'-4"(1)	17'-10"(1)
	560	32'-8"	29'-9"	28'-0"	25'-2"(1)	***32'-8"***	***29'-9"***	***26'-3"***(1)	20'-11"(1)
16"	110	26'-0"	22'-6"	20'-7"(1)	18'-1"(1)	23'-9"	20'-7"(1)	18'-9"(1)	15'-0"(1)
	210	28'-6"	24'-8"	22'-6"(1)	19'-11"(1)	26'-0"	22'-6"(1)	20'-7"(1)	16'-7"(1)
	230	30'-1"	26'-0"	23'-9"	21'-1"(1)	***27'-5"***	23'-9"	21'-8"(1)	17'-6"(1)
	360	31'-10"	29'-0"	26'-10"(1)	21'-5"(1)	***31'-10"***	***26'-10"***(1)	22'-4"(1)	17'-10"(1)
	560	36'-1"	32'-11"	31'-0"(1)	25'-2"(1)	***36'-1"***	***31'-6"***(1)	26'-3"(1)	20'-11"(1)

(1) Web stiffeners are required at intermediate supports of continuous-span joists when the intermediate bearing length is ***less*** than 5¼" and the span on either side of the intermediate bearing is greater than the following spans:

TJI®	40 PSF Live Load / 10 PSF Dead Load				40 PSF Live Load / 20 PSF Dead Load			
	12" o.c.	16" o.c.	19.2" o.c.	24" o.c.	12" o.c.	16" o.c.	19.2" o.c.	24" o.c.
110	Not Req.	Not Req.	19'-2"	15'-4"	Not Req.	19'-2"	16'-0"	12'-9"
210			21'-4"	17'-0"		21'-4"	17'-9"	14'-2"
230			Not Req.	19'-2"		Not Req.	19'-11"	15'-11"
360			24'-5"	19'-6"		24'-5"	20'-4"	16'-3"
560			29'-10"	23'-10"		29'-10"	24'-10"	19'-10"

- Long-term deflection under dead load, which includes the effect of creep, has not been considered. Bold italic spans reflect initial dead load deflection exceeding 0.33".

How to Use These Tables

1. Determine the appropriate live load deflection criteria.
2. Identify the live and dead load condition.
3. Select on-center spacing.
4. Scan down the column until you meet or exceed the span of your application.
5. Select TJI® joist and depth.

General Notes

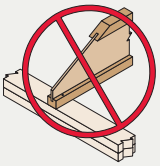
- Tables are based on:
  - Uniform loads.
  - More restrictive of simple or continuous span.
  - Clear distance between supports
  - Minimum bearing length of 1¾" end (no web stiffeners) and 3½" intermediate.
- Assumed composite action with a single layer of 24" on-center span-rated, glue-nailed floor panels for deflection only. **When subfloor adhesive is not applied, spans shall be reduced 6" for nails and 12" for proprietary fasteners.**
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DO NOT bevel cut joist beyond inside face of wall.



DO NOT install hanger overhanging face of plate or beam. Flush bearing plate with inside face of wall or beam.

L/480 Live Load Deflection

Depth	TJI®	40 PSF Live Load / 10 PSF Dead Load				40 PSF Live Load / 20 PSF Dead Load			
		12" o.c.	16" o.c.	19.2" o.c.	24" o.c.	12" o.c.	16" o.c.	19.2" o.c.	24" o.c.
9½"	110	16'-11"	15'-6"	14'-7"	13'-7"	16'-11"	15'-6"	14'-3"	12'-9"
	210	17'-9"	16'-3"	15'-4"	14'-3"	17'-9"	16'-3"	15'-4"	14'-0"
	230	18'-3"	16'-8"	15'-9"	14'-8"	18'-3"	16'-8"	15'-9"	14'-8"
11⅞"	110	20'-2"	18'-5"	17'-4"	15'-9"(1)	20'-2"	17'-8"	16'-1"(1)	14'-4"(1)
	210	21'-1"	19'-3"	18'-2"	16'-11"	21'-1"	19'-3"	17'-8"	15'-9"(1)
	230	21'-8"	19'-10"	18'-8"	17'-5"	21'-8"	19'-10"	18'-7"	16'-7"(1)
	360	22'-11"	20'-11"	19'-8"	18'-4"	22'-11"	20'-11"	19'-8"	17'-10"(1)
	560	26'-1"	23'-8"	22'-4"	20'-9"	26'-1"	23'-8"	22'-4"	20'-9"(1)
14"	110	22'-10"	20'-11"	19'-2"	17'-2"(1)	22'-2"	19'-2"	17'-6"(1)	15'-0"(1)
	210	23'-11"	21'-10"	20'-8"	18'-10"(1)	23'-11"	21'-1"	19'-2"(1)	16'-7"(1)
	230	24'-8"	22'-6"	21'-2"	19'-9"(1)	24'-8"	22'-2"	20'-3"(1)	17'-6"(1)
	360	26'-0"	23'-8"	22'-4"	20'-9"(1)	26'-0"	23'-8"	22'-4"(1)	17'-10"(1)
	560	29'-6"	26'-10"	25'-4"	23'-6"	***29'-6"***	***26'-10"***	25'-4"(1)	20'-11"(1)
16"	110	25'-4"	22'-6"	20'-7"(1)	18'-1"(1)	23'-9"	20'-7"(1)	18'-9"(1)	15'-0"(1)
	210	26'-6"	24'-3"	22'-6"(1)	19'-11"(1)	26'-0"	22'-6"(1)	20'-7"(1)	16'-7"(1)
	230	27'-3"	24'-10"	23'-6"	21'-1"(1)	***27'-3"***	23'-9"	21'-8"(1)	17'-6"(1)
	360	28'-9"	26'-3"	24'-8"(1)	21'-5"(1)	***28'-9"***	26'-3"(1)	22'-4"(1)	17'-10"(1)
	560	32'-8"	29'-8"	28'-0"	25'-2"(1)	***32'-8"***	***29'-8"***	26'-3"(1)	20'-11"(1)

L/360 Live Load Deflection (Minimum Criteria per Code)

Depth	TJI®	40 PSF Live Load / 10 PSF Dead Load				40 PSF Live Load / 20 PSF Dead Load			
		12" o.c.	16" o.c.	19.2" o.c.	24" o.c.	12" o.c.	16" o.c.	19.2" o.c.	24" o.c.
9½"	110	18'-9"	17'-2"	15'-8"	14'-0"	18'-1"	15'-8"	14'-3"	12'-9"
	210	19'-8"	18'-0"	17'-0"	15'-4"	19'-8"	17'-2"	15'-8"	14'-0"
	230	20'-3"	18'-6"	17'-5"	16'-2"	***20'-3"***	18'-1"	16'-6"	14'-9"
11⅞"	110	22'-3"	19'-4"	17'-8"	15'-9"(1)	20'-5"	17'-8"	16'-1"(1)	14'-4"(1)
	210	23'-4"	21'-2"	19'-4"	17'-3"(1)	***22'-4"***	19'-4"	17'-8"	15'-9"(1)
	230	24'-0"	21'-11"	20'-5"	18'-3"	***23'-7"***	20'-5"	18'-7"	16'-7"(1)
	360	25'-4"	23'-2"	21'-10"	20'-4"(1)	***25'-4"***	***23'-2"***	***21'-10"***(1)	17'-10"(1)
	560	28'-10"	26'-3"	24'-9"	23'-0"	***28'-10"***	***26'-3"***	***24'-9"***	20'-11"(1)
14"	110	24'-4"	21'-0"	19'-2"	17'-2"(1)	22'-2"	19'-2"	17'-6"(1)	15'-0"(1)
	210	26'-6"	23'-1"	21'-1"	18'-10"(1)	24'-4"	21'-1"	19'-2"(1)	16'-7"(1)
	230	27'-3"	24'-4"	22'-2"	19'-10"(1)	***25'-8"***	22'-2"	20'-3"(1)	17'-6"(1)
	360	28'-9"	26'-3"	24'-9"(1)	21'-5"(1)	***28'-9"***	***26'-3"***(1)	22'-4"(1)	17'-10"(1)
	560	32'-8"	29'-9"	28'-0"	25'-2"(1)	***32'-8"***	***29'-9"***	***26'-3"***(1)	20'-11"(1)
16"	110	26'-0"	22'-6"	20'-7"(1)	18'-1"(1)	23'-9"	20'-7"(1)	18'-9"(1)	15'-0"(1)
	210	28'-6"	24'-8"	22'-6"(1)	19'-11"(1)	26'-0"	22'-6"(1)	20'-7"(1)	16'-7"(1)
	230	30'-1"	26'-0"	23'-9"	21'-1"(1)	***27'-5"***	23'-9"	21'-8"(1)	17'-6"(1)
	360	31'-10"	29'-0"	26'-10"(1)	21'-5"(1)	***31'-10"***	***26'-10"***(1)	22'-4"(1)	17'-10"(1)
	560	36'-1"	32'-11"	31'-0"(1)	25'-2"(1)	***36'-1"***	***31'-6"***(1)	26'-3"(1)	20'-11"(1)

(1) Web stiffeners are required at intermediate supports of continuous-span joists when the intermediate bearing length is ***less*** than 5¼" and the span on either side of the intermediate bearing is greater than the following spans:

TJI®	40 PSF Live Load / 10 PSF Dead Load				40 PSF Live Load / 20 PSF Dead Load			
	12" o.c.	16" o.c.	19.2" o.c.	24" o.c.	12" o.c.	16" o.c.	19.2" o.c.	24" o.c.
110	Not Req.	Not Req.	19'-2"	15'-4"	Not Req.	19'-2"	16'-0"	12'-9"
210			21'-4"	17'-0"		21'-4"	17'-9"	14'-2"
230			Not Req.	19'-2"		Not Req.	19'-11"	15'-11"
360			24'-5"	19'-6"		24'-5"	20'-4"	16'-3"
560			29'-10"	23'-10"		29'-10"	24'-10"	19'-10"

- Long-term deflection under dead load, which includes the effect of creep, has not been considered. Bold italic spans reflect initial dead load deflection exceeding 0.33".

How to Use These Tables

1. Determine the appropriate live load deflection criteria.
2. Identify the live and dead load condition.
3. Select on-center spacing.
4. Scan down the column until you meet or exceed the span of your application.
5. Select TJI® joist and depth.

General Notes

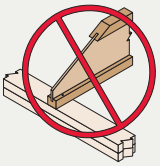
- Tables are based on:
  - Uniform loads.
  - More restrictive of simple or continuous span.
  - Clear distance between supports
  - Minimum bearing length of 1¾" end (no web stiffeners) and 3½" intermediate.
- Assumed composite action with a single layer of 24" on-center span-rated, glue-nailed floor panels for deflection only. **When subfloor adhesive is not applied, spans shall be reduced 6" for nails and 12" for proprietary fasteners.**
- For continuous spans, ratio of short span to long span should be 0.4 or greater to prevent uplift.
- Spans generated from Weyerhaeuser software may exceed the spans shown in these tables because software reflects actual design conditions.
- For multi-family applications and other loading conditions not shown, refer to Weyerhaeuser software or to the load table on page 8.

Live load deflection is not the only factor that affects how a floor will perform. To more accurately predict floor performance, use our TJ-Pro™ Ratings.

These Conditions Are NOT Permitted:



DO NOT use sawn lumber for rim board or blocking as it may shrink after installation. Use only engineered lumber



DO NOT bevel cut joist beyond inside face of wall.



DO NOT install hanger overhanging face of plate or beam. Flush bearing plate with inside face of wall or beam.

L/480 Live Load Deflection

Depth	TJI®	40 PSF Live Load / 10 PSF Dead Load				40 PSF Live Load / 20 PSF Dead Load			
		12" o.c.	16" o.c.	19.2" o.c.	24" o.c.	12" o.c.	16" o.c.	19.2" o.c.	24" o.c.
9½"	110	16'-11"	15'-6"	14'-7"	13'-7"	16'-11"	15'-6"	14'-3"	12'-9"
	210	17'-9"	16'-3"	15'-4"	14'-3"	17'-9"	16'-3"	15'-4"	14'-0"
	230	18'-3"	16'-8"	15'-9"	14'-8"	18'-3"	16'-8"	15'-9"	14'-8"
11⅞"	110	20'-2"	18'-5"	17'-4"	15'-9"(1)	20'-2"	17'-8"	16'-1"(1)	14'-4"(1)
	210	21'-1"	19'-3"	18'-2"	16'-11"	21'-1"	19'-3"	17'-8"	15'-9"(1)
	230	21'-8"	19'-10"	18'-8"	17'-5"	21'-8"	19'-10"	18'-7"	16'-7"(1)
	360	22'-11"	20'-11"	19'-8"	18'-4"	22'-11"	20'-11"	19'-8"	17'-10"(1)
	560	26'-1"	23'-8"	22'-4"	20'-9"	26'-1"	23'-8"	22'-4"	20'-9"(1)
14"	110	22'-10"	20'-11"	19'-2"	17'-2"(1)	22'-2"	19'-2"	17'-6"(1)	15'-0"(1)
	210	23'-11"	21'-10"	20'-8"	18'-10"(1)	23'-11"	21'-1"	19'-2"(1)	16'-7"(1)
	230	24'-8"	22'-6"	21'-2"	19'-9"(1)	24'-8"	22'-2"	20'-3"(1)	17'-6"(1)
	360	26'-0"	23'-8"	22'-4"	20'-9"(1)	26'-0"	23'-8"	22'-4"(1)	17'-10"(1)
	560	29'-6"	26'-10"	25'-4"	23'-6"	***29'-6"***	***26'-10"***	25'-4"(1)	20'-11"(1)
16"	110	25'-4"	22'-6"	20'-7"(1)	18'-1"(1)	23'-9"	20'-7"(1)	18'-9"(1)	15'-0"(1)
	210	26'-6"	24'-3"	22'-6"(1)	19'-11"(1)	26'-0"	22'-6"(1)	20'-7"(1)	16'-7"(1)
	230	27'-3"	24'-10"	23'-6"	21'-1"(1)	***27'-3"***	23'-9"	21'-8"(1)	17'-6"(1)
	360	28'-9"	26'-3"	24'-8"(1)	21'-5"(1)	***28'-9"***	26'-3"(1)	22'-4"(1)	17'-10"(1)
	560	32'-8"	29'-8"	28'-0"	25'-2"(1)	***32'-8"***	***29'-8"***	26'-3"(1)	20'-11"(1)

L/360 Live Load Deflection (Minimum Criteria per Code)

Depth	TJI®	40 PSF Live Load / 10 PSF Dead Load				40 PSF Live Load / 20 PSF Dead Load			
		12" o.c.	16" o.c.	19.2" o.c.	24" o.c.	12" o.c.	16" o.c.	19.2" o.c.	24" o.c.
9½"	110	18'-9"	17'-2"	15'-8"	14'-0"	18'-1"	15'-8"	14'-3"	12'-9"
	210	19'-8"	18'-0"	17'-0"	15'-4"	19'-8"	17'-2"	15'-8"	14'-0"
	230	20'-3"	18'-6"	17'-5"	16'-2"	***20'-3"***	18'-1"	16'-6"	14'-9"
11⅞"	110	22'-3"	19'-4"	17'-8"	15'-9"(1)	20'-5"	17'-8"	16'-1"(1)	14'-4"(1)
	210	23'-4"	21'-2"	19'-4"	17'-3"(1)	***22'-4"***	19'-4"	17'-8"	15'-9"(1)
	230	24'-0"	21'-11"	20'-5"	18'-3"	***23'-7"***	20'-5"	18'-7"	16'-7"(1)
	360	25'-4"	23'-2"	21'-10"	20'-4"(1)	***25'-4"***	***23'-2"***	***21'-10"***(1)	17'-10"(1)
	560	28'-10"	26'-3"	24'-9"	23'-0"	***28'-10"***	***26'-3"***	***24'-9"***	20'-11"(1)
14"	110	24'-4"	21'-0"	19'-2"	17'-2"(1)	22'-2"	19'-2"	17'-6"(1)	15'-0"(1)
	210	26'-6"	23'-1"	21'-1"	18'-10"(1)	24'-4"	21'-1"	19'-2"(1)	16'-7"(1)
	230	27'-3"	24'-4"	22'-2"	19'-10"(1)	***25'-8"***	22'-2"	20'-3"(1)	17'-6"(1)
	360	28'-9"	26'-3"	24'-9"(1)	21'-5"(1)	***28'-9"***	***26'-3"***(1)	22'-4"(1)	17'-10"(1)
	560	32'-8"	29'-9"	28'-0"	25'-2"(1)	***32'-8"***	***29'-9"***	***26'-3"***(1)	20'-11"(1)
16"	110	26'-0"	22'-6"	20'-7"(1)	18'-1"(1)	23'-9"	20'-7"(1)	18'-9"(1)	15'-0"(1)
	210	28'-6"	24'-8"	22'-6"(1)	19'-11"(1)	26'-0"	22'-6"(1)	20'-7"(1)	16'-7"(1)
	230	30'-1"	26'-0"	23'-9"	21'-1"(1)	***27'-5"***	23'-9"	21'-8"(1)	17'-6"(1)
	360	31'-10"	29'-0"	26'-10"(1)	21'-5"(1)	***31'-10"***	***26'-10"***(1)	22'-4"(1)	17'-10"(1)
	560	36'-1"	32'-11"	31'-0"(1)	25'-2"(1)	***36'-1"***	***31'-6"***(1)	26'-3"(1)	20'-11"(1)

(1) Web stiffeners are required at intermediate supports of continuous-span joists when the intermediate bearing length is ***less*** than 5¼" and the span on either side of the intermediate bearing is greater than the following spans:

TJI®	40 PSF Live Load / 10 PSF Dead Load				40 PSF Live Load / 20 PSF Dead Load			
	12" o.c.	16" o.c.	19.2" o.c.	24" o.c.	12" o.c.	16" o.c.	19.2" o.c.	24" o.c.
110	Not Req.	Not Req.	19'-2"	15'-4"	Not Req.	19'-2"	16'-0"	12'-9"
210			21'-4"	17'-0"		21'-4"	17'-9"	14'-2"
230			Not Req.	19'-2"		Not Req.	19'-11"	15'-11"
360			24'-5"	19'-6"		24'-5"	20'-4"	16'-3"
560			29'-10"	23'-10"		29'-10"	24'-10"	19'-10"

- Long-term deflection under dead load, which includes the effect of creep, has not been considered. Bold italic spans reflect initial dead load deflection exceeding 0.33".

How to Use These Tables

1. Determine the appropriate live load deflection criteria.
2. Identify the live and dead load condition.
3. Select on-center spacing.
4. Scan down the column until you meet or exceed the span of your application.
5. Select TJI® joist and depth.

General Notes

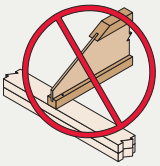
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  - Clear distance between supports
  - Minimum bearing length of 1¾" end (no web stiffeners) and 3½" intermediate.
- Assumed composite action with a single layer of 24" on-center span-rated, glue-nailed floor panels for deflection only. **When subfloor adhesive is not applied, spans shall be reduced 6" for nails and 12" for proprietary fasteners.**
- For continuous spans, ratio of short span to long span should be 0.4 or greater to prevent uplift.
- Spans generated from Weyerhaeuser software may exceed the spans shown in these tables because software reflects actual design conditions.
- For multi-family applications and other loading conditions not shown, refer to Weyerhaeuser software or to the load table on page 8.

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These Conditions Are NOT Permitted:



DO NOT use sawn lumber for rim board or blocking as it may shrink after installation. Use only engineered lumber



DO NOT bevel cut joist beyond inside face of wall.



DO NOT install hanger overhanging face of plate or beam. Flush bearing plate with inside face of wall or beam.

2.0E LVL Design Properties

Allowable Design Properties - 1½"

Depth	Max. Vertical Shear (lbs)			Max. Bending Moment (ft-lbs)			EI (x 10^6 lbs-in)	Weight (plf)
	100%	115%	125%	100%	115%	125%		
5½"	1595	1834	1994	2248	2585	2810	42	2.15
7¼"	2103	2418	2628	3717	4275	4646	95	2.83
9¼"	2683	3085	3353	5791	6660	7239	198	3.61
9½"	2755	3168	3444	6079	6991	7599	214	3.71
11¼"	3263	3752	4078	8269	9510	10337	356	4.39
11⅞"	3444	3960	4305	9124	10493	11406	419	4.64
14"	4060	4669	5075	12312	14159	15390	686	5.47
16"	4640	5336	5800	15699	18054	19624	1024	6.25
18"	5220	6003	6525	19452	22370	24315	1458	7.03
20"	5800	6670	7250	23564	27099	29455	2000	7.81
22"	6380	7337	7975	28027	32232	35034	2662	8.59
24"	6960	8004	8700	32837	37762	41046	3456	9.38

Allowable Design Properties - 1¾"

Depth	Max. Vertical Shear (lbs)			Max. Bending Moment (ft-lbs)			EI (x 10^6 lbs-in)	Weight (plf)
	100%	115%	125%	100%	115%	125%		
5½"	1861	2140	2326	2623	3016	3279	49	2.51
7¼"	2453	2821	3066	4336	4987	5421	111	3.3
9¼"	3130	3599	3912	6756	7770	8445	231	4.22
9½"	3214	3696	4018	7092	8156	8865	250	4.33
11¼"	3806	4377	4758	9648	11095	12059	415	5.13
11⅞"	4018	4620	5022	10645	12242	13306	488	5.41
14"	4737	5447	5921	14364	16519	17955	800	6.38
16"	5413	6225	6767	18315	21063	22894	1195	7.29
18"	6090	7004	7613	22694	26098	28368	1701	8.19
20"	6767	7782	8458	27491	31615	34364	2333	9.12
22"	7443	8560	9304	32699	37603	40873	3106	10.03
24"	8120	9338	10150	38309	44056	47877	4032	10.94

Allowable Design Properties - 3½"

Depth	Max. Vertical Shear (lbs)			Max. Bending Moment (ft-lbs)			EI (x 10^6 lbs-in)	Weight (plf)
	100%	115%	125%	100%	115%	125%		
5½"	3722	4280	4652	5246	6033	6557	97	5.01
7¼"	4906	5642	6132	8673	9974	10841	222	6.61
9¼"	6259	7198	7824	13512	15539	16890	462	8.43
9½"	6428	7393	8035	14184	16312	17730	500	8.66
11¼"	7613	8754	9516	19295	22189	24119	831	10.25
11⅞"	8035	9241	10044	21290	24484	26613	977	10.82
14"	9473	10894	11842	28728	33037	35910	1601	12.76
16"	10827	12451	13533	36631	42126	45789	2389	14.58
18"	12180	14007	15225	45389	52197	56736	3402	16.41
20"	13533	15563	16917	54983	63230	68728	4667	18.23
22"	14887	17120	18608	65397	75207	81747	6211	20.05
24"	16240	18676	20300	76619	88112	95773	8064	21.88

2.0E LVL Allowable Design Properties

Bending Fb = 3100 psi* (Adjust Fb value by a factor of $(12/d)^{0.18}$ where d = depth.)

Horizontal Shear Fv = 290 psi

Modulus of Elasticity E = 2,000,000 psi

Compression Perpendicular to Grain $Fc^{\perp}$ = 750 psi

Compression Parallel to Grain Fc∥ = 3,200 psi

TRI-STATE FOREST PRODUCTS

FLOOR BEAM DESIGN: At Stair Opening

Assumptions:

1. 3.5 x 15.0 GLULAM Beam Western Species				**16.50**	feet span
	Therefore:	L_1	=	198	inches
2. Span 1 Supposrted				**4.00**	feet
Span 2 Supported				**0.00**	feet
4. Assume bearing area = (3.5" x 5.5")				19.25	$inches^2$

3.50" x 15.0" GluLam Girder Properties:

Reference: 2020 APA Glued Laminated Beam Design Tables

Western Species GluLam Beam

ANSI Design Values

A	=	**52.50**	$inches^2$
E	=	**1,800,000**	PSI
S	=	**131**	inches3
I	=	**984**	inches4
$Fv_{Allowable}$	=	**8,400**	LBS
$Fp_{Allowable}$	=	**405**	PSI
Fb	=	**2,400**	PSI
$M_{Allowable}$	=	**26,250**	FT-LBS
W	=	**12.8**	LBS

Reactions:

W	=	Floor Framing DL	=	10.5	PSF
		Glulam DL	=	1.0	PLF
		Floor LL	=	40	PSF
		Design Load = DL x 1.2 + LL x 1.4	=	138.40	PLF
		W_{lf}	=	**275**	PLF

Reactions:

V_{max}	=	0.5**WL**	=	**2,269**	lbs
P_{max}	=	0.5**WL**	=	**2,269**	lbs
M_{max}	=	**$(WL^2)/8$**	=	**4,710**	ft-lbs
D_{max}	=	0.013**WL^4 / EI**	=	**0.13**	inches

Check Shear:

Fv	=	**V_{max}**(1.5) / **A**	=	**2,269**	lbs
$Fv_{Allowable}$	=			**8,400**	lbs
				Check - O.K.	

Check Bearing:

Fp	=	**P_{max} / A**	=	**117.86**	PSI

$Fp_{Allowable}$	=			**405**	PSI
				Check - O.K.	

Check Bending:

$\mathbf{M_{max}}$	=		=	**4,710**	FT-LBS
$\mathbf{Mm}_{Allowable}$	=			**26,250**	FT-LBS
				Check - O.K.	

Check Deflection:

$D_{max.}$	=			**0.1301**	inches
$D_{Allowable}$	=	**L**/350	=	**0.5657**	inches
				Check - O.K.	

Steel Floor Beam Design

First Level Beam 22

Max Span - First Floor Level — Beam 22 is the worst case for beams 18, 23, 27, 28, 31 & 32

Assumptions:

1. W8 x 67 Beam — **27.00** Feet = 324.00 Inches
   Supports are: **3.00** inches from each end of the beam
   Therefore: **L** = 318.00 Inches
2. Beam designed as a simple span.
   Therefore: L_d = 318 Inches
3. Bearing area: (Assume: 6"x6" Plate) **N** = **6** Inches

W8 x 67 Beam Properties:

A	=	**19.70** inches^2		
A_{web}	=	**5.13** inches2		(Web area from top of beam to bottom of beam x web width.)
E	=	**29,000,000** PSI		
I	=	**272** inches^4		
S	=	**60.4** inches^3		
$Fv_{Allowable}$	=	**9,500** PSI		SDOT 6-02.3(17)B For Salvaged Steel
		30,000 PSI	$0.6F_y$	AISC (Chapter G 16.1-64)
$Fp_{Allowable}$	=	**22,500** PSI		SDOT 6-02.3(17)B For Salvaged Steel
Fb	=	**33,000.00** PSI	$0.66F_y$	AISC (F3-1)
t_w	=	**0.57** inches		
d	=	**9** inches		
t_f	=	**0.935** inches		
F_y	=	**50,000** PSI		
k	=	**0.69** inches		
b_f	=	**8.28** inches		
W_t	=	**67** lbs/lf		

Reactions:

W_{design}	=	=	**1,325**	plf
V_{max}	=	=	**17,888**	lbs
P_{max}	=	=	**17,888**	lbs
M_{max}	=	=	**120,741**	ft-lbs
D_{max}	=	=	**0.167**	inches

Check Shear:

Fv	=	V_{max} / A_{web}	=	**3,497** PSI
$Fv_{Allowable}$	=	$0.6F_y$	=	**30,000** PSI
				Check - O.K.
		V_{max}	=	**17,888** LBS
$Fv_{Allowable}$	=	$0.66F_w t_w(N + 2.5k)$ ASD Section K1.3	=	**145,190** LBS
				Check - O.K.

Check Bearing-Web Crippling:

Fp	=	P_{max} / A_{web}	=	**3,487** PSI
$Fp_{Allowable}$	=	$0.6F_y$	=	**30,000** PSI
				Check - O.K.
		V_{max}	=	**17,888** LBS
$F_{pAllowable}$	=	$0.66F_w t_w(N + 2.5k)$ ASD Section K1.3	=	**145,190** LBS
				Check - O.K.

Check Bending:

$S_{Req'd}$	=	$M_{max}(12) / F_b$	=	**43.9** inches^3
$S_{Furnished}$	=			**60.4** inches^3
				Check - O.K.

Check Deflection:

$D_{max.}$	=	D_{max}	=	**0.167** inches
$D_{Allowable}$	=	L/350	=	**0.926** inches
				Check - O.K.

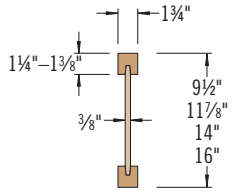
SECTION 1: 9½"–16" TJI® JOISTS

This section contains design information for 9½"–16" deep Trus Joist® TJI® joists.
These standard-size TJI® joists are readily available through your local Weyerhaeuser dealer or distributor.
Offered with the flange sizes shown below, they come in lengths up to 60' (in 1' increments).

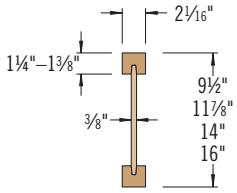
Design Properties (100% Load Duration)

Depth	TJI®	Basic Properties				Reaction Properties					
		Joist Weight (lbs/ft)	Maximum Resistive Moment(1) (ft-lbs)	Joist Only EI x 10^6 ($in.^2$-lbs)	Maximum Vertical Shear (lbs)	1¾" End Reaction (lbs)	3½" End Reaction (lbs)	3½" Intermediate Reaction (lbs)		5¼" Intermediate Reaction (lbs)	
								No Web Stiffeners	With Web Stiffeners(2)	No Web Stiffeners	With Web Stiffeners(2)
9½"	110	2.3	2,500	157	1,220	910	1,220	1,935	N.A.	2,350	N.A.
	210	2.6	3,000	186	1,330	1,005	1,330	2,145	N.A.	2,565	N.A.
	230	2.7	3,330	206	1,330	1,060	1,330	2,410	N.A.	2,790	N.A.
11⅞"	110	2.5	3,160	267	1,560	910	1,375	1,935	2,295	2,350	2,705
	210	2.8	3,795	315	1,655	1,005	1,460	2,145	2,505	2,565	2,925
	230	3.0	4,215	347	1,655	1,060	1,485	2,410	2,765	2,790	3,150
	360	3.0	6,180	419	1,705	1,080	1,505	2,460	2,815	3,000	3,360
	560	4.0	9,500	636	2,050	1,265	1,725	3,000	3,475	3,455	3,930
14"	110	2.8	3,740	392	1,860	910	1,375	1,935	2,295	2,350	2,705
	210	3.1	4,490	462	1,945	1,005	1,460	2,145	2,505	2,565	2,925
	230	3.3	4,990	509	1,945	1,060	1,485	2,410	2,765	2,790	3,150
	360	3.3	7,335	612	1,955	1,080	1,505	2,460	2,815	3,000	3,360
	560	4.2	11,275	926	2,390	1,265	1,725	3,000	3,475	3,455	3,930
16"	110	3.0	4,280	535	2,145	910	1,375	1,935	2,295	2,350	2,705
	210	3.3	5,140	629	2,190	1,005	1,460	2,145	2,505	2,565	2,925
	230	3.5	5,710	691	2,190	1,060	1,485	2,410	2,765	2,790	3,150
	360	3.5	8,405	830	2,190	1,080	1,505	2,460	2,815	3,000	3,360
	560	4.5	12,925	1,252	2,710	1,265	1,725	3,000	3,475	3,455	3,930

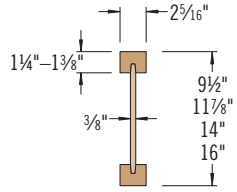
(1) **Caution: Do not** increase joist moment design properties by a repetitive member use factor.
(2) See detail W on page 27 for web stiffener requirements and nailing information.



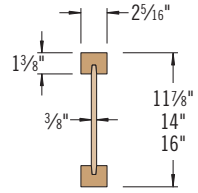
TJI® 110 joists



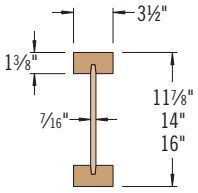
TJI® 210 joists



TJI® 230 joists



TJI® 360 joists



TJI® 560 joists

General Notes

- Design reaction includes all loads on the joist. Design shear is computed at the inside face of supports and includes all loads on the span(s). Allowable shear may sometimes be increased at interior supports in accordance with ICC ES ESR-1153, and these increases are reflected in span tables.
- The formulas at right approximate the uniform load deflection of Δ (inches).

TJI® joists are intended for dry-use applications

Some TJI® joist series may not be available in your region. Contact your Weyerhaeuser representative for information.

For TJI® 110, 210, and 360 Joists

$$\Delta = \frac{22.5\ wL^4}{EI} + \frac{2.67\ wL^2}{d \times 10^5}$$

For TJI® 560 Joists

$$\Delta = \frac{22.5\ wL^4}{EI} + \frac{2.29\ wL^2}{d \times 10^5}$$

w = uniform load in pounds per linear foot
L = span in feet
d = out-to-out depth of the joist in inches
EI = value from table above


DO NOT walk on joists until braced. INJURY MAY RESULT.


DO NOT stack building materials on unsheathed joists. Stack only over beams or walls.


DO NOT walk on joists that are lying flat.

WARNING

Joists are unstable until braced laterally

Bracing Includes:
- **Blocking**
- **Hangers**
- **Rim Board**
- **Sheathing**
- **Rim Joist**
- **Strut Lines**

WARNING NOTES:

Lack of proper bracing during construction can result in serious accidents. Observe the following guidelines:

1. All blocking, hangers, rim boards, and rim joists at the end supports of the TJI® joists must be completely installed and properly nailed.
2. Lateral strength, like a braced end wall or an existing deck, must be established at the ends of the bay. This can also be accomplished by a temporary or permanent deck (sheathing) fastened to the first 4 feet of joists at the end of the bay.
3. Safety bracing of 1x4 (minimum) must be nailed to a braced end wall or sheathed area (as in note 2) and to each joist. Without this bracing, buckling sideways or rollover is highly probable under light construction loads—such as a worker or one layer of unnailed sheathing.
4. Sheathing must be completely attached to each TJI® joist before additional loads can be placed on the system.
5. Ends of cantilevers require safety bracing on both the top and bottom flanges.
6. The flanges must remain straight within a tolerance of ½" from true alignment.

TJI - FLOOR JOIST DESIGN

First Floor Level — 10' Span With 5.5' Overhang

Assumptions:

1. 9.5" 210 Floor Joists
2. Supports spaced @: **10** feet simple span; **5.5** feet cantiliver
3. Floor Joist Spacing: **1.50** feet o.c.
4. Minimum Bearing Area: 2.31" x 3.5" = **8.09** $inches^2$

9.5" 110 Properties:

Weight	=	**2.30**	PLF
EI	=	**157,000,000**	PSI
$M_{Allowable}$	=	**2,500**	$inches^4$
$V_{Allowable}$	=	**1,220**	$inches^3$
$Pe_{Allowable}$	=	**1220**	PSI
$Ps_{Allowable}$	=	**1935**	PSI

Reactions: From Uniform Loading:

		Floor DL	=	**10.5**	PSF
		Deck Floor LL	=	**60**	PSF
W	=	Factored Floor DL x 1.2 +LL x 1.4	=	**96.6**	PSF
				98	PSF Used in design

W_{max}	=	W x Joist Spacing	=	**147**	PLF
V_{max}	=	**WA**	=	**539**	lbs
P_{max}	=	**WA**	=	**539**	lbs
M_1	=	$WA^2/2$	=	0	ft-lbs
M_2	=	$WA^2 / 2$	=	**1,482**	ft-lbs
D_{max}	=	**(WA/24EI)**	=	**0.00**	inches
			=		

Check Shear:

V_{max}	=	=	**539**	PSI
$V_{Allowable}$	=	=	**1,220**	PSI
			Check - O.K.	

Check Bearing:

P_{max}	=	=	**539**	PSI
$Ps_{Allowable}$	=		**1,935**	PSI
			Check - O.K.	

Check Bending:

M_{max}	=	=	**1,482**	$inches^3$
$M_{Allowable}$	=		**2,500**	$inches^3$
			Check - O.K.	

Check Deflection:

$D_{max.}$	=			**0.00**	inches
$D_{Allowable}$	=	**L**/350	=	**0.34**	inches
				Check - O.K.	

TJI - FLOOR JOIST DESIGN

First Floor Level

7' Span With 5.5' Overhang

Assumptions:

1. 9.5" 210 Floor Joists
2. Supports spaced @: **7** feet simple span
   5.5 feet cantiliver
3. Floor Joist Spacing: **1.50** feet O.C.
4. Minimum Bearing Area: 2.31" x 3.5" = **8.09** $inches^2$

9.5" 210 Properties:

Weight	=	**2.60**	PLF
EI	=	**186,000,000**	PSI
$M_{Allowable}$	=	**3,000**	$inches^4$
$V_{Allowable}$	=	**1,330**	$inches^3$
$Pe_{Allowable}$	=	**1330**	PSI
$Ps_{Allowable}$	=	**2656**	PSI

Reactions: From Uniform Loading:

		Floor DL	=	**10.5**	PSF
		Floor LL	=	**60**	PSF
W	=	Factored Floor DL x 1.2 +LL x 1.4	=	**96.6**	PSF
				98	PSF Used in design

W_{max}	=	W x Joist Spacing	=	**147**	PLF
V_{max}	=	$WL/2 + P_L$	=	**1,323**	lbs
P_{max}	=	$V_{max} + P_L$	=	**1,323**	lbs
M_1	=	$((W / (L^2 \times 8)) \times ((L+A)^2 * (L-A)^2)$	=	**132**	ft-lbs
M_2	=	$WA^2 / 2$	=	**2,223**	ft-lbs
D_{max}	=	$(5WL^4 / 384EI) + (PL^3 / 48EI)$	=	**0.00**	inches
			=		

Check Shear:

V_{max}	=	=	**1,323**	PSI
$V_{Allowable}$	=	=	**1,330**	PSI
			Check - O.K.	

Check Bearing:

P_{max}	=	=	**1,323**	PSI
$Ps_{Allowable}$	=		**2,656**	PSI
			Check - O.K.	

Check Bending:

M_{max}	=	=	**2,223**	$inches^3$
$M_{Allowable}$	=		**3,000**	$inches^3$
			Check - O.K.	

Check Deflection:

$D_{max.}$	=			**0.00**	inches
$D_{Allowable}$	=	**L**/350	=	**0.24**	inches
				Check - O.K.	

TJI - FLOOR JOIST DESIGN

First Floor Level

20' Span With 1.0' Overhang

Assumptions:

1. 16" 210 Floor Joists
2. Supports spaced @: **19.67** feet simple span
   1.0 feet cantiliver
3. Floor Joist Spacing: **1.50** feet o.c.
4. Minimum Bearing Area: 2.31" x 3.5" = **8.09** $inches^2$

16" 210 Properties:

Weight	=	**3.30**	PLF
EI	=	**629,000,000**	PSI
$M_{Allowable}$	=	**5,140**	$inches^4$
$V_{Allowable}$	=	**2,190**	$inches^3$
$Pe_{Allowable}$	=	**2190**	PSI
$Ps_{Allowable}$	=	**2145**	PSI

Reactions: From Uniform Loading:

		Floor DL	=	**10.5**	PSF
		Floor LL	=	**40**	PSF
W	=	Factored Floor DL x 1.2 +LL x 1.4	=	**68.6**	PSF
				70	PSF Used in design

W_{max}	=	W x Joist Spacing	=	**105**	PLF
V_{max}	=	$WL/2 + P_L$	=	**1,138**	lbs
P_{max}	=	$V_{max} + P_L$	=	**1,460**	lbs
M_1	=	$((W / (L^2 \times 8)) \times ((L+A)^2 * (L-A)^2)$	=	**5,052**	ft-lbs
M_2	=	$WA^2 / 2$	=	**53**	ft-lbs
D_{max}	=	$(5WL^4 / 384EI) + (PL^3 / 48EI)$	=	**0.05**	inches
			=		

Check Shear:

V_{max}	=	=	**1,138**	PSI
$V_{Allowable}$	=	=	**2,190**	PSI
			Check - O.K.	

Check Bearing:

P_{max}	=	=	**1,460**	PSI
$Ps_{Allowable}$	=		**2,145**	PSI
			Check - O.K.	

Check Bending:

M_{max}	=	=	**5,052**	$inches^3$
$M_{Allowable}$	=		**5,140**	$inches^3$
			Check - O.K.	

Check Deflection:

$D_{max.}$	=			**0.05**	inches
$D_{Allowable}$	=	**L**/350	=	**0.67**	inches
				Check - O.K.	

TJI - FLOOR JOIST DESIGN

First Floor Level — 20' Span With 5.5' Overhang

Assumptions:

1. 16" 210 Floor Joists
2. Supports spaced @: **20** feet simple span
   5.5 feet cantiliver
3. Floor Joist Spacing: **1.50** feet O.C.
4. Minimum Bearing Area: 2.31" x 3.5" = **8.09** $inches^2$

16" 210 Properties:

Weight	=	**3.30**	PLF
EI	=	**186,000,000**	PSI
$\mathbf{M}_{Allowable}$	=	**5,140**	$inches^4$
$\mathbf{V}_{Allowable}$	=	**2,190**	$inches^3$
$\mathbf{Pe}_{Allowable}$	=	**2190**	PSI
$\mathbf{Ps}_{Allowable}$	=	**2145**	PSI

Reactions: From Uniform Loading:

		Floor DL	=	**10.5**	PSF
		Floor LL	=	**40**	PSF
W	=	Factored Floor DL x 1.2 +LL x 1.4	=	**68.6**	PSF
				70	PSF Used in design

$\mathbf{W_{max}}$	=	W x Joist Spacing	=	**105**	PLF
$\mathbf{V_{max}}$	=	$\mathbf{WL/2 + P_L}$	=	**1,628**	lbs
$\mathbf{P_{max}}$	=	$\mathbf{V_{max} + P_L}$	=	**1,628**	lbs
$\mathbf{M_1}$	=	$\mathbf{((W / (L^2 \times 8)) \times ((L+A)^2 * (L-A)^2)}$	=	**4,486**	ft-lbs
$\mathbf{M_2}$	=	$\mathbf{WA^2 / 2}$	=	**1,588**	ft-lbs
$\mathbf{D_{max}}$	=	$(5\mathbf{WL}^4 / 384\mathbf{EI}) + (PL^3/48EI)$	=	**0.17**	inches
			=		

Check Shear:

$\mathbf{V_{max}}$	=		=	**1,628** PSI
$\mathbf{V}_{Allowable}$	=		=	**2,190** PSI
				Check - O.K.

Check Bearing:

$\mathbf{P_{max}}$	=		=	**1,628** PSI
$\mathbf{Ps}_{Allowable}$	=			**2,145** PSI
				Check - O.K.

Check Bending:

$\mathbf{M_{max}}$	=		=	**4,486** $inches^3$
$\mathbf{M}_{Allowable}$	=			**5,140** $inches^3$
				Check - O.K.

Check Deflection:

$D_{max.}$	=			**0.17**	inches
$D_{Allowable}$	=	**L**/350	=	**0.69**	inches
				Check - O.K.	

TJI - FLOOR JOIST DESIGN

First Floor Level

Assumptions:

1. 9.5" TJI 210 Floor Joists
2. Supports spaced @: **15.42** feet
3. Floor Joist Spacing: **2.00** feet O.C.
4. Minimum Bearing Area: 2.31" x 3.5" = **8.09** $inches^2$

9.5" TJI 210 Floor Joists

Weight	=	**2.60**	PLF
EI	=	**186,000,000**	PSI
$\mathbf{M}_{Allowable}$	=	**3,000**	$inches^4$
$\mathbf{V}_{Allowable}$	=	**1,330**	$inches^3$
$\mathbf{Pe}_{Allowable}$	=	**1330**	PSI
$\mathbf{Ps}_{Allowable}$	=	**2145**	PSI

Reactions: From Uniform Loading:

W	=	Floor Dead Laod	=	12.5	PSF
		Floor Live Load	=	25	PSF
		Factored Floor DL x 1.2 +LL x 1.4		**50**	PSF

$\mathbf{W}_{max}$	=	W x Joist Spacing	=	**100**	PLF
$\mathbf{V}_{max}$	=	**WL /2**	=	**771**	lbs
$\mathbf{P}_{max}$	=	$\mathbf{V}_{max}$	=	**771**	lbs
$\mathbf{M}_{max}$	=	$\mathbf{(WL^2)/8}$	=	**2,972**	ft-lbs
$\mathbf{D}_{max}$	=	$5\mathbf{WL}^4/\ 384\mathbf{EI}$	=	**0.06**	inches

Check Shear:

$\mathbf{V}_{max}$	=	=	**771**	PSI
$\mathbf{V}_{Allowable}$	=	=	**1,330**	PSI
			Check - O.K.	

Check Bearing:

$\mathbf{P}_{max}$	=	=	**771**	PSI
$\mathbf{Ps}_{Allowable}$	=		**2,145**	PSI
			Check - O.K.	

Check Bending:

$\mathbf{M}_{max}$	=	=	**2,972**	$inches^3$
$\mathbf{M}_{Allowable}$	=		**3,000**	$inches^3$
			Check - O.K.	

Check Deflection:

$D_{max.}$	=			**0.06** inches
$D_{Allowable}$	=	**L**/350	=	**0.53** inches
				Check - O.K.

TJI - FLOOR JOIST DESIGN

Green Roof

Assumptions:

1. 9.5" TJI 110 Floor Joists
2. Supports spaced @: **8.92** feet
3. Floor Joist Spacing: **2.00** feet O.C.
4. Minimum Bearing Area: 2.31" x 3.5" = **8.09** $inches^2$

9.5" TJI 110 Floor Joists

Weight	=	**2.30**	PLF
EI	=	**157,000,000**	PSI
$\mathbf{M}_{Allowable}$	=	**2,500**	$inches^4$
$\mathbf{V}_{Allowable}$	=	**1,220**	$inches^3$
$\mathbf{Pe}_{Allowable}$	=	**1220**	PSI
$\mathbf{Ps}_{Allowable}$	=	**1935**	PSI

Reactions: From Uniform Loading:

W	=	Floor Dead Load	=	12.5	PSF
		Green Roof Dead Load	=	45	PSF
		Floor Live Load	=	40	PSF
		Factored Floor DL x 1.2 +LL x 1.4		**125**	PSF

$\mathbf{W}_{max}$	=	W x Joist Spacing	=	**250** PLF
$\mathbf{V}_{max}$	=	**WL /2**	=	**1,115** lbs
$\mathbf{P}_{max}$	=	$\mathbf{V}_{max}$	=	**1,115** lbs
$\mathbf{M}_{max}$	=	$(\mathbf{WL}^2)/8$	=	**2,486** ft-lbs
$\mathbf{D}_{max}$	=	$5\mathbf{WL}^4/ 384\mathbf{EI}$	=	**0.02** inches

Check Shear:

$\mathbf{V}_{max}$	=	=	**1,115** PSI
$\mathbf{V}_{Allowable}$	=	=	**1,220** PSI
			Check - O.K.

Check Bearing:

$\mathbf{P}_{max}$	=	=	**1,115** PSI
$\mathbf{Ps}_{Allowable}$	=		**1,935** PSI
			Check - O.K.

Check Bending:

$\mathbf{M}_{max}$	=	=	**2,486** $inches^3$
$\mathbf{M}_{Allowable}$	=		**2,500** $inches^3$
			Check - O.K.

Check Deflection:

$D_{max.}$	=			**0.02** inches
$D_{Allowable}$	=	**L**/350	=	**0.31** inches

Check - O.K.

LUCIA ENGINEERING, INC.
12527 Huckleberry Lane
Arlington, Washington 98223
Phone: 206.790-8039

SECOND FLOOR LOADS:

FLOOR D.L. = 10.5 PSF

FLOOR L.L. = 40 PSF

$\Rightarrow$ DESIGN LOAD = $10.5^{PSF} \times 1.2 + 40^{PSF} \times 1.4$

$= 58.1^{PSF}$

USE RED-L OPEN W/EB TRUSS JOIST
SET @ 12" O.C.

$W_{MAX} = 59^{PLF} \Rightarrow$ USE 14" TRUSS JOISTS

PER RED-L TABLE

14" TRUSS @ 2'-0" O.C. LOAD = $59^{PLF} \times 2 = 118^{PLF}$

ALLOW LOAD = 215^{PLF} $\checkmark$ OK

L/480 Live Load Deflection

Depth	TJI®	40 PSF Live Load / 10 PSF Dead Load				40 PSF Live Load / 20 PSF Dead Load			
		12" o.c.	16" o.c.	19.2" o.c.	24" o.c.	12" o.c.	16" o.c.	19.2" o.c.	24" o.c.
9½"	110	16'-11"	15'-6"	14'-7"	13'-7"	16'-11"	15'-6"	14'-3"	12'-9"
	210	17'-9"	16'-3"	15'-4"	14'-3"	17'-9"	16'-3"	15'-4"	14'-0"
	230	18'-3"	16'-8"	15'-9"	14'-8"	18'-3"	16'-8"	15'-9"	14'-8"
11⅞"	110	20'-2"	18'-5"	17'-4"	15'-9"(1)	20'-2"	17'-8"	16'-1"(1)	14'-4"(1)
	210	21'-1"	19'-3"	18'-2"	16'-11"	21'-1"	19'-3"	17'-8"	15'-9"(1)
	230	21'-8"	19'-10"	18'-8"	17'-5"	21'-8"	19'-10"	18'-7"	16'-7"(1)
	360	22'-11"	20'-11"	19'-8"	18'-4"	22'-11"	20'-11"	19'-8"	17'-10"(1)
	560	26'-1"	23'-8"	22'-4"	20'-9"	26'-1"	23'-8"	22'-4"	20'-9"(1)
14"	110	22'-10"	20'-11"	19'-2"	17'-2"(1)	22'-2"	19'-2"	17'-6"(1)	15'-0"(1)
	210	23'-11"	21'-10"	20'-8"	18'-10"(1)	23'-11"	21'-1"	19'-2"(1)	16'-7"(1)
	230	24'-8"	22'-6"	21'-2"	19'-9"(1)	24'-8"	22'-2"	20'-3"(1)	17'-6"(1)
	360	26'-0"	23'-8"	22'-4"	20'-9"(1)	26'-0"	23'-8"	22'-4"(1)	17'-10"(1)
	560	29'-6"	26'-10"	25'-4"	23'-6"	***29'-6"***	***26'-10"***	25'-4"(1)	20'-11"(1)
16"	110	25'-4"	22'-6"	20'-7"(1)	18'-1"(1)	23'-9"	20'-7"(1)	18'-9"(1)	15'-0"(1)
	210	26'-6"	24'-3"	22'-6"(1)	19'-11"(1)	26'-0"	22'-6"(1)	20'-7"(1)	16'-7"(1)
	230	27'-3"	24'-10"	23'-6"	21'-1"(1)	***27'-3"***	23'-9"	21'-8"(1)	17'-6"(1)
	360	28'-9"	26'-3"	24'-8"(1)	21'-5"(1)	***28'-9"***	26'-3"(1)	22'-4"(1)	17'-10"(1)
	560	32'-8"	29'-8"	28'-0"	25'-2"(1)	***32'-8"***	***29'-8"***	26'-3"(1)	20'-11"(1)

L/360 Live Load Deflection (Minimum Criteria per Code)

Depth	TJI®	40 PSF Live Load / 10 PSF Dead Load				40 PSF Live Load / 20 PSF Dead Load			
		12" o.c.	16" o.c.	19.2" o.c.	24" o.c.	12" o.c.	16" o.c.	19.2" o.c.	24" o.c.
9½"	110	18'-9"	17'-2"	15'-8"	14'-0"	18'-1"	15'-8"	14'-3"	12'-9"
	210	19'-8"	18'-0"	17'-0"	15'-4"	19'-8"	17'-2"	15'-8"	14'-0"
	230	20'-3"	18'-6"	17'-5"	16'-2"	***20'-3"***	18'-1"	16'-6"	14'-9"
11⅞"	110	22'-3"	19'-4"	17'-8"	15'-9"(1)	20'-5"	17'-8"	16'-1"(1)	14'-4"(1)
	210	23'-4"	21'-2"	19'-4"	17'-3"(1)	***22'-4"***	19'-4"	17'-8"	15'-9"(1)
	230	24'-0"	21'-11"	20'-5"	18'-3"	***23'-7"***	20'-5"	18'-7"	16'-7"(1)
	360	25'-4"	23'-2"	21'-10"	20'-4"(1)	***25'-4"***	***23'-2"***	***21'-10"***(1)	17'-10"(1)
	560	28'-10"	26'-3"	24'-9"	23'-0"	***28'-10"***	***26'-3"***	***24'-9"***	20'-11"(1)
14"	110	24'-4"	21'-0"	19'-2"	17'-2"(1)	22'-2"	19'-2"	17'-6"(1)	15'-0"(1)
	210	26'-6"	23'-1"	21'-1"	18'-10"(1)	24'-4"	21'-1"	19'-2"(1)	16'-7"(1)
	230	27'-3"	24'-4"	22'-2"	19'-10"(1)	***25'-8"***	22'-2"	20'-3"(1)	17'-6"(1)
	360	28'-9"	26'-3"	24'-9"(1)	21'-5"(1)	***28'-9"***	***26'-3"***(1)	22'-4"(1)	17'-10"(1)
	560	32'-8"	29'-9"	28'-0"	25'-2"(1)	***32'-8"***	***29'-9"***	***26'-3"***(1)	20'-11"(1)
16"	110	26'-0"	22'-6"	20'-7"(1)	18'-1"(1)	23'-9"	20'-7"(1)	18'-9"(1)	15'-0"(1)
	210	28'-6"	24'-8"	22'-6"(1)	19'-11"(1)	26'-0"	22'-6"(1)	20'-7"(1)	16'-7"(1)
	230	30'-1"	26'-0"	23'-9"	21'-1"(1)	***27'-5"***	23'-9"	21'-8"(1)	17'-6"(1)
	360	31'-10"	29'-0"	26'-10"(1)	21'-5"(1)	***31'-10"***	***26'-10"***(1)	22'-4"(1)	17'-10"(1)
	560	36'-1"	32'-11"	31'-0"(1)	25'-2"(1)	***36'-1"***	***31'-6"***(1)	26'-3"(1)	20'-11"(1)

(1) Web stiffeners are required at intermediate supports of continuous-span joists when the intermediate bearing length is ***less*** than 5¼" and the span on either side of the intermediate bearing is greater than the following spans:

TJI®	40 PSF Live Load / 10 PSF Dead Load				40 PSF Live Load / 20 PSF Dead Load			
	12" o.c.	16" o.c.	19.2" o.c.	24" o.c.	12" o.c.	16" o.c.	19.2" o.c.	24" o.c.
110	Not Req.	Not Req.	19'-2"	15'-4"	Not Req.	19'-2"	16'-0"	12'-9"
210			21'-4"	17'-0"		21'-4"	17'-9"	14'-2"
230			Not Req.	19'-2"		Not Req.	19'-11"	15'-11"
360			24'-5"	19'-6"		24'-5"	20'-4"	16'-3"
560			29'-10"	23'-10"		29'-10"	24'-10"	19'-10"

- Long-term deflection under dead load, which includes the effect of creep, has not been considered. Bold italic spans reflect initial dead load deflection exceeding 0.33".

How to Use These Tables

1. Determine the appropriate live load deflection criteria.
2. Identify the live and dead load condition.
3. Select on-center spacing.
4. Scan down the column until you meet or exceed the span of your application.
5. Select TJI® joist and depth.

General Notes

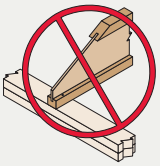
- Tables are based on:
  - Uniform loads.
  - More restrictive of simple or continuous span.
  - Clear distance between supports
  - Minimum bearing length of 1¾" end (no web stiffeners) and 3½" intermediate.
- Assumed composite action with a single layer of 24" on-center span-rated, glue-nailed floor panels for deflection only. **When subfloor adhesive is not applied, spans shall be reduced 6" for nails and 12" for proprietary fasteners.**
- For continuous spans, ratio of short span to long span should be 0.4 or greater to prevent uplift.
- Spans generated from Weyerhaeuser software may exceed the spans shown in these tables because software reflects actual design conditions.
- For multi-family applications and other loading conditions not shown, refer to Weyerhaeuser software or to the load table on page 8.

Live load deflection is not the only factor that affects how a floor will perform. To more accurately predict floor performance, use our TJ-Pro™ Ratings.

These Conditions Are NOT Permitted:



DO NOT use sawn lumber for rim board or blocking as it may shrink after installation. Use only engineered lumber



DO NOT bevel cut joist beyond inside face of wall.



DO NOT install hanger overhanging face of plate or beam. Flush bearing plate with inside face of wall or beam.

HEADER BEAM DESIGN:

Assumptions:

1. 3.5 x 12.0 GLULAM Beam Western Species				**12.00**	feet span
	Therefore:	L_1	=	144	inches
2. Span 1 Supposrted				**1.00**	feet
Span 2 Supported				**19.25**	feet
4. Assume bearing area = (5.5" x 5.5")				30.25	$inches^2$

3.50" x 12.0" GluLam Girder Properties:

Reference: 2020 APA Glued Laminated Beam Design Tables

Western Species GluLam Beam

ANSI Design Values

A	=	**42.00**	$inches^2$
E	=	**1,800,000**	PSI
S	=	**84**	inches3
I	=	**504**	inches4
$Fv_{Allowable}$	=	**6,720**	LBS
$Fp_{Allowable}$	=	**405**	PSI
Fb	=	**2,400**	PSI
$M_{Allowable}$	=	**16,800**	FT-LBS
W	=	**10.2**	LBS

Reactions:

W	=	Floor Framing DL	12.3	PSF
		Glulam DL	10.2	PLF
		Floor LL	40	PSF
		Design Load = DL x 1.2 + LL x 1.6 =	809.69	PLF

Reactions:

V_{max}	=	0.5**WL**	=	**4,858**	lbs
P_{max}	=	0.5**WL**	=	**4,858**	lbs
M_{max}	=	**(WL^2)**/8	=	**14,574**	ft-lbs
D_{max}	=	0.013**WL^4 / EI**	=	**0.42**	inches

Check Shear:

Fv	=	**V_{max}**(1.5) / **A**	=	**4,858**	lbs
$Fv_{Allowable}$	=			**6,720**	lbs
				Check - O.K.	

Check Bearing:

Fp	=	**P_{max} / A**	=	**160.60**	PSI
$Fp_{Allowable}$	=			**405**	PSI
				Check - O.K.	

Check Bending:

M_{max}	=		=	**14,574**	FT-LBS
$Mm_{Allowable}$	=			**16,800**	FT-LBS
				Check - O.K.	

Check Deflection:

$D_{max.}$	=			**0.4157**	inches
$D_{Allowable}$	=	**L**/350	=	**0.4114**	inches
				Check - O.K.	

HEADER BEAM DESIGN:

Assumptions:

1. 5.5 x 15.0 GLULAM Beam Western Species				**16.33**	feet span
	Therefore:	L_1	=	195.96	inches
2. Span 1 Supposrted				**6.50**	feet
Span 2 Supported				**19.25**	feet
4. Assume bearing area = (5.5" x 5.5")				30.25	$inches^2$

5.50" x 15.0" GluLam Girder Properties:

Reference: 2020 APA Glued Laminated Beam Design Tables

Western Species GluLam Beam

ANSI Design Values

A	=	**82.50**	$inches^2$
E	=	**1,800,000**	PSI
S	=	**206**	inches3
I	=	**1,547**	inches4
$Fv_{Allowable}$	=	**13,200**	LBS
$Fp_{Allowable}$	=	**405**	PSI
Fb	=	**2,400**	PSI
$M_{Allowable}$	=	**41,250**	FT-LBS
W	=	**20.1**	LBS

Reactions:

W	=	Floor Framing DL	12.3	PSF
		Glulam DL	20.1	PLF
		Floor LL	40	PSF
		Design Load = DL x 1.2 + LL x 1.6 =	1,038.16	PLF

Reactions:

V_{max}	=	0.5**WL**	=	**8,477**	lbs
P_{max}	=	0.5**WL**	=	**8,477**	lbs
M_{max}	=	$(WL^2)/8$	=	**34,605**	ft-lbs
D_{max}	=	$0.013WL^4 / EI$	=	**0.60**	inches

Check Shear:

Fv	=	$V_{max}(1.5) / A$	=	**8,477**	lbs
$Fv_{Allowable}$	=			**13,200**	lbs
				Check - O.K.	

Check Bearing:

Fp	=	P_{max} / A	=	**280.22**	PSI
$Fp_{Allowable}$	=			**405**	PSI
				Check - O.K.	

Check Bending:

M_{max}	=		=	**34,605**	FT-LBS
$Mm_{Allowable}$	=			**41,250**	FT-LBS
				Check - O.K.	

Check Deflection:

$D_{max.}$	=			**0.5956**	inches
$D_{Allowable}$	=	**L**/350	=	**0.5599**	inches
				Header is supported in three places	
				Check - O.K.	

Second Level Header Beam - 12' Span

TABLE 1

DOUGLAS-FIR GLUED LAMINATED BEAM SECTION PROPERTIES AND CAPACITIES
F_b = 2,400 PSI, E = 1,800,000 PSI, F_v = 240 PSI

3-1/8-INCH WIDTH															
Depth (in.)	6	7-1/2	9	10-1/2	12	13-1/2	15	16-1/2	18	19-1/2	21	22-1/2	24	25-1/2	27
Beam Weight (lb/ft)[1]	4.6	5.7	6.8	8.0	9.1	10.3	11.4	12.5	13.7	14.8	16.0	17.1	18.2	19.4	20.5
A (in.^{2})	18.8	23.4	28.1	32.8	37.5	42.2	46.9	51.6	56.3	60.9	65.6	70.3	75.0	79.7	84.4
S (in.^{3})	19	29	42	57	75	95	117	142	169	198	230	264	300	339	380
I (in.^{4})	56	110	190	301	450	641	879	1170	1519	1931	2412	2966	3600	4318	5126
EI (106 lb-in.^{2})	101	198	342	543	810	1153	1582	2106	2734	3476	4341	5339	6480	7773	9226
Moment Capacity (lb-ft)[2][3]	3750	5859	8438	11484	15000	18984	23438	28359	33750	39609	45938	52734	60000	67734	75938
Shear Capacity (lb)[3]	3000	3750	4500	5250	6000	6750	7500	8250	9000	9750	10500	11250	12000	12750	13500
3-1/2-INCH WIDTH															
Depth (in.)	6	7-1/2	9	10-1/2	12	13-1/2	15	16-1/2	18	19-1/2	21	22-1/2	24	25-1/2	27
Beam Weight (lb/ft)[1]	5.1	6.4	7.7	8.9	10.2	11.5	12.8	14.0	15.3	16.6	17.9	19.1	20.4	21.7	23.0
A (in.^{2})	21.0	26.3	31.5	36.8	42.0	47.3	52.5	57.8	63.0	68.3	73.5	78.8	84.0	89.3	94.5
S (in.^{3})	21	33	47	64	84	106	131	159	189	222	257	295	336	379	425
I (in.^{4})	63	123	213	338	504	718	984	1310	1701	2163	2701	3322	4032	4836	5741
EI (106 lb-in.^{2})	113	221	383	608	907	1292	1772	2358	3062	3893	4862	5980	7258	8705	10334
Moment Capacity (lb-ft)[2][3]	4200	6563	9450	12863	16800	21263	26250	31763	37800	44363	51450	59063	67200	75863	85050
Shear Capacity (lb)[3]	3360	4200	5040	5880	6720	7560	8400	9240	10080	10920	11760	12600	13440	14280	15120
5-1/8-INCH WIDTH															
Depth (in.)	12	13-1/2	15	16-1/2	18	19-1/2	21	22-1/2	24	25-1/2	27	28-1/2	30	31-1/2	33
Beam Weight (lb/ft)[1]	14.9	16.8	18.7	20.6	22.4	24.3	26.2	28.0	29.9	31.8	33.6	35.5	37.4	39.2	41.1
A (in.^{2})	61.5	69.2	76.9	84.6	92.3	99.9	107.6	115.3	123.0	130.7	138.4	146.1	153.8	161.4	169.1
S (in.^{3})	123	156	192	233	277	325	377	432	492	555	623	694	769	848	930
I (in.^{4})	738	1051	1441	1919	2491	3167	3955	4865	5904	7082	8406	9887	11531	13349	15348
EI (106 lb-in.^{2})	1328	1891	2595	3453	4483	5700	7119	8757	10627	12747	15131	17796	20756	24028	27627
Moment Capacity (lb-ft)[2][3]	24600	31134	38438	46509	55350	64959	75338	86484	98400	111084	124538	138759	153750	169509	186038
Shear Capacity (lb)[3]	9840	11070	12300	13530	14760	15990	17220	18450	19680	20910	22140	23370	24600	25830	27060
5-1/2-INCH WIDTH															
Depth (in.)	12	13-1/2	15	16-1/2	18	19-1/2	21	22-1/2	24	25-1/2	27	28-1/2	30	31-1/2	33
Beam Weight (lb/ft)[1]	16.0	18.0	20.1	22.1	24.1	26.1	28.1	30.1	32.1	34.1	36.1	38.1	40.1	42.1	44.1
A (in.^{2})	66.0	74.3	82.5	90.8	99.0	107.3	115.5	123.8	132.0	140.3	148.5	156.8	165.0	173.3	181.5
S (in.^{3})	132	167	206	250	297	349	404	464	528	596	668	745	825	910	998
I (in.^{4})	792	1128	1547	2059	2673	3398	4245	5221	6336	7600	9021	10610	12375	14326	16471
EI (106 lb-in.^{2})	1426	2030	2784	3706	4811	6117	7640	9397	11405	13680	16238	19098	22275	25786	29648
Moment Capacity (lb-ft)[2][3]	26400	33413	41250	49913	59400	69713	80850	92813	105600	119213	133650	148913	165000	181913	199650
Shear Capacity (lb)[3]	10560	11880	13200	14520	15840	17160	18480	19800	21120	22440	23760	25080	26400	27720	29040
6-3/4-INCH WIDTH															
Depth (in.)	18	19-1/2	21	22-1/2	24	25-1/2	27	28-1/2	30	31-1/2	33	34-1/2	36	37-1/2	39
Beam Weight (lb/ft)[1]	29.5	32.0	34.5	36.9	39.4	41.8	44.3	46.8	49.2	51.7	54.1	56.6	59.1	61.5	64.0
A (in.^{2})	121.5	131.6	141.8	151.9	162.0	172.1	182.3	192.4	202.5	212.6	222.8	232.9	243.0	253.1	263.3
S (in.^{3})	365	428	496	570	648	732	820	914	1013	1116	1225	1339	1458	1582	1711
I (in.^{4})	3281	4171	5209	6407	7776	9327	11072	13021	15188	17581	20215	23098	26244	29663	33367
EI (106 lb-in.^{2})	5905	7508	9377	11533	13997	16789	19929	23438	27338	31647	36386	41577	47239	53394	60060
Moment Capacity (lb-ft)[2][3]	72900	85556	99225	113906	129600	146306	164025	182756	202500	223256	245025	267806	291600	316406	342225
Shear Capacity (lb)[3]	19440	21060	22680	24300	25920	27540	29160	30780	32400	34020	35640	37260	38880	40500	42120
8-3/4-INCH WIDTH															
Depth (in.)	24	25-1/2	27	28-1/2	30	31-1/2	33	34-1/2	36	37-1/2	39	40-1/2	42	43-1/2	45
Beam Weight (lb/ft)[1]	51.0	54.2	57.4	60.6	63.8	67.0	70.2	73.4	76.6	79.8	82.9	86.1	89.3	92.5	95.7
A (in.^{2})	210.0	223.1	236.3	249.4	262.5	275.6	288.8	301.9	315.0	328.1	341.3	354.4	367.5	380.6	393.8
S (in.^{3})	840	948	1063	1185	1313	1447	1588	1736	1890	2051	2218	2392	2573	2760	2953
I (in.^{4})	10080	12091	14352	16880	19688	22791	26204	29942	34020	38452	43253	48439	54023	60020	66445
EI (106 lb-in.^{2})	18144	21763	25834	30383	35438	41023	47167	53896	61236	69214	77856	87190	97241	108036	119602
Moment Capacity (lb-ft)[2][3]	168000	189656	212625	236906	262500	289406	317625	347156	378000	410156	443625	478406	514500	551906	590625
Shear Capacity (lb)[3]	33600	35700	37800	39900	42000	44100	46200	48300	50400	52500	54600	56700	58800	60900	63000

Notes:
(1) Beam weight is based on density of 35 pcf.
(2) Moment capacity must be adjusted for volume effect. The volume factor for various glulam sizes and simple spans, as well as the complete formula, is given in Appendix A.
(3) Moment and shear capacities are based on a normal (10 years) duration of load and should be adjusted for the design duration of load per the applicable building code.

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Second Level Header Beam - 16' Span

TABLE 1

DOUGLAS-FIR GLUED LAMINATED BEAM SECTION PROPERTIES AND CAPACITIES
F_b = 2,400 PSI, E = 1,800,000 PSI, F_v = 240 PSI

3-1/8-INCH WIDTH															
Depth (in.)	6	7-1/2	9	10-1/2	12	13-1/2	15	16-1/2	18	19-1/2	21	22-1/2	24	25-1/2	27
Beam Weight (lb/ft)[1]	4.6	5.7	6.8	8.0	9.1	10.3	11.4	12.5	13.7	14.8	16.0	17.1	18.2	19.4	20.5
A (in.^{2})	18.8	23.4	28.1	32.8	37.5	42.2	46.9	51.6	56.3	60.9	65.6	70.3	75.0	79.7	84.4
S (in.^{3})	19	29	42	57	75	95	117	142	169	198	230	264	300	339	380
I (in.^{4})	56	110	190	301	450	641	879	1170	1519	1931	2412	2966	3600	4318	5126
EI (106 lb-in.^{2})	101	198	342	543	810	1153	1582	2106	2734	3476	4341	5339	6480	7773	9226
Moment Capacity (lb-ft)[2][3]	3750	5859	8438	11484	15000	18984	23438	28359	33750	39609	45938	52734	60000	67734	75938
Shear Capacity (lb)[3]	3000	3750	4500	5250	6000	6750	7500	8250	9000	9750	10500	11250	12000	12750	13500
3-1/2-INCH WIDTH															
Depth (in.)	6	7-1/2	9	10-1/2	12	13-1/2	15	16-1/2	18	19-1/2	21	22-1/2	24	25-1/2	27
Beam Weight (lb/ft)[1]	5.1	6.4	7.7	8.9	10.2	11.5	12.8	14.0	15.3	16.6	17.9	19.1	20.4	21.7	23.0
A (in.^{2})	21.0	26.3	31.5	36.8	42.0	47.3	52.5	57.8	63.0	68.3	73.5	78.8	84.0	89.3	94.5
S (in.^{3})	21	33	47	64	84	106	131	159	189	222	257	295	336	379	425
I (in.^{4})	63	123	213	338	504	718	984	1310	1701	2163	2701	3322	4032	4836	5741
EI (106 lb-in.^{2})	113	221	383	608	907	1292	1772	2358	3062	3893	4862	5980	7258	8705	10334
Moment Capacity (lb-ft)[2][3]	4200	6563	9450	12863	16800	21263	26250	31763	37800	44363	51450	59063	67200	75863	85050
Shear Capacity (lb)[3]	3360	4200	5040	5880	6720	7560	8400	9240	10080	10920	11760	12600	13440	14280	15120
5-1/8-INCH WIDTH															
Depth (in.)	12	13-1/2	15	16-1/2	18	19-1/2	21	22-1/2	24	25-1/2	27	28-1/2	30	31-1/2	33
Beam Weight (lb/ft)[1]	14.9	16.8	18.7	20.6	22.4	24.3	26.2	28.0	29.9	31.8	33.6	35.5	37.4	39.2	41.1
A (in.^{2})	61.5	69.2	76.9	84.6	92.3	99.9	107.6	115.3	123.0	130.7	138.4	146.1	153.8	161.4	169.1
S (in.^{3})	123	156	192	233	277	325	377	432	492	555	623	694	769	848	930
I (in.^{4})	738	1051	1441	1919	2491	3167	3955	4865	5904	7082	8406	9887	11531	13349	15348
EI (106 lb-in.^{2})	1328	1891	2595	3453	4483	5700	7119	8757	10627	12747	15131	17796	20756	24028	27627
Moment Capacity (lb-ft)[2][3]	24600	31134	38438	46509	55350	64959	75338	86484	98400	111084	124538	138759	153750	169509	186038
Shear Capacity (lb)[3]	9840	11070	12300	13530	14760	15990	17220	18450	19680	20910	22140	23370	24600	25830	27060
5-1/2-INCH WIDTH															
Depth (in.)	12	13-1/2	15	16-1/2	18	19-1/2	21	22-1/2	24	25-1/2	27	28-1/2	30	31-1/2	33
Beam Weight (lb/ft)[1]	16.0	18.0	20.1	22.1	24.1	26.1	28.1	30.1	32.1	34.1	36.1	38.1	40.1	42.1	44.1
A (in.^{2})	66.0	74.3	82.5	90.8	99.0	107.3	115.5	123.8	132.0	140.3	148.5	156.8	165.0	173.3	181.5
S (in.^{3})	132	167	206	250	297	349	404	464	528	596	668	745	825	910	998
I (in.^{4})	792	1128	1547	2059	2673	3398	4245	5221	6336	7600	9021	10610	12375	14326	16471
EI (106 lb-in.^{2})	1426	2030	2784	3706	4811	6117	7640	9397	11405	13680	16238	19098	22275	25786	29648
Moment Capacity (lb-ft)[2][3]	26400	33413	41250	49913	59400	69713	80850	92813	105600	119213	133650	148913	165000	181913	199650
Shear Capacity (lb)[3]	10560	11880	13200	14520	15840	17160	18480	19800	21120	22440	23760	25080	26400	27720	29040
6-3/4-INCH WIDTH															
Depth (in.)	18	19-1/2	21	22-1/2	24	25-1/2	27	28-1/2	30	31-1/2	33	34-1/2	36	37-1/2	39
Beam Weight (lb/ft)[1]	29.5	32.0	34.5	36.9	39.4	41.8	44.3	46.8	49.2	51.7	54.1	56.6	59.1	61.5	64.0
A (in.^{2})	121.5	131.6	141.8	151.9	162.0	172.1	182.3	192.4	202.5	212.6	222.8	232.9	243.0	253.1	263.3
S (in.^{3})	365	428	496	570	648	732	820	914	1013	1116	1225	1339	1458	1582	1711
I (in.^{4})	3281	4171	5209	6407	7776	9327	11072	13021	15188	17581	20215	23098	26244	29663	33367
EI (106 lb-in.^{2})	5905	7508	9377	11533	13997	16789	19929	23438	27338	31647	36386	41577	47239	53394	60060
Moment Capacity (lb-ft)[2][3]	72900	85556	99225	113906	129600	146306	164025	182756	202500	223256	245025	267806	291600	316406	342225
Shear Capacity (lb)[3]	19440	21060	22680	24300	25920	27540	29160	30780	32400	34020	35640	37260	38880	40500	42120
8-3/4-INCH WIDTH															
Depth (in.)	24	25-1/2	27	28-1/2	30	31-1/2	33	34-1/2	36	37-1/2	39	40-1/2	42	43-1/2	45
Beam Weight (lb/ft)[1]	51.0	54.2	57.4	60.6	63.8	67.0	70.2	73.4	76.6	79.8	82.9	86.1	89.3	92.5	95.7
A (in.^{2})	210.0	223.1	236.3	249.4	262.5	275.6	288.8	301.9	315.0	328.1	341.3	354.4	367.5	380.6	393.8
S (in.^{3})	840	948	1063	1185	1313	1447	1588	1736	1890	2051	2218	2392	2573	2760	2953
I (in.^{4})	10080	12091	14352	16880	19688	22791	26204	29942	34020	38452	43253	48439	54023	60020	66445
EI (106 lb-in.^{2})	18144	21763	25834	30383	35438	41023	47167	53896	61236	69214	77856	87190	97241	108036	119602
Moment Capacity (lb-ft)[2][3]	168000	189656	212625	236906	262500	289406	317625	347156	378000	410156	443625	478406	514500	551906	590625
Shear Capacity (lb)[3]	33600	35700	37800	39900	42000	44100	46200	48300	50400	52500	54600	56700	58800	60900	63000

Notes:
(1) Beam weight is based on density of 35 pcf.
(2) Moment capacity must be adjusted for volume effect. The volume factor for various glulam sizes and simple spans, as well as the complete formula, is given in Appendix A.
(3) Moment and shear capacities are based on a normal (10 years) duration of load and should be adjusted for the design duration of load per the applicable building code.

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TJI - FLOOR JOIST DESIGN

Second Floor Level

Assumptions:

1. 9.5 210 Floor Joists
2. Supports spaced @: **23** feet
3. Floor Joist Spacing: **2.00** feet O.C.
4. Minimum Bearing Area: 2.31" x 3.5" = **8.09** $inches^2$

9.5" 210 Properties:

Weight	=	**2.60**	PLF
EI	=	**186,000,000**	PSI
$\mathbf{M}_{Allowable}$	=	**3,000**	$inches^4$
$\mathbf{V}_{Allowable}$	=	**1,005**	$inches^3$
$\mathbf{Pe}_{Allowable}$	=	**1330**	PSI
$\mathbf{Ps}_{Allowable}$	=	**1330**	PSI

Reactions: From Uniform Loading:

W = Factored Floor DL +LL = **80** PSF

$\mathbf{W_{max}}$	=	W x Joist Spacing	=	**160**	PLF
$\mathbf{V_{max}}$	=	$\mathbf{WL/2 + P_L}$	=	**1,840**	lbs
$\mathbf{P_{max}}$	=	$\mathbf{V_{max} + P_L}$	=	**1,840**	lbs
$\mathbf{M_{max}}$	=	$\mathbf{(WL^2)/8 + (P_L*L/4)}$	=	**10,580**	ft-lbs
$\mathbf{D_{max}}$	=	$(5\mathbf{WL}^4/384\mathbf{EI})+(PL^3/48EI)$	=	**0.45**	inches
			=		

Check Shear:

$\mathbf{V_{max}}$	=	=	**1,840**	PSI
$\mathbf{V}_{Allowable}$	=	=	**1,330**	PSI
			Check - O.K.	

Check Bearing:

$\mathbf{P_{max}}$	=	=	**1,840**	PSI
$\mathbf{Ps}_{Allowable}$	=		**1,330**	PSI
			Check - O.K.	

Check Bending:

$\mathbf{M_{max}}$	=	=	**10,580**	$inches^3$
$\mathbf{M}_{Allowable}$	=		**3,000**	$inches^3$
			Check - O.K.	

Check Deflection:

$D_{max.}$	=			**0.45**	inches
$D_{Allowable}$	=	**L**/350	=	**0.79**	inches
				Check - O.K.	

Steel Header Beam Design

Assumptions:

1. W14 x 25 Beam			**21.17**	Feet =	254.04	Inches
		Supports are:	**3.00**	inches from each end of the beam		
	Therefore:		**L**	=	248.04	Inches
2. Beam designed as a simple span.						
	Therefore:		**L_d**	=	248.04	Inches
3. Bearing area:	(Assume: 6"x6" Plate)		**N**	=	**6**	Inches
4. Span 1 Supposrted				**13.67**	feet	
Span 2 Supported				**30.17**	feet	

W12 x 45 Beam Properties:

A	=	**13.20** inches^2		
A_{web}	=	**4.04** inches2	(Web area from top of beam to bottom of beam x web width.)	
E	=	**29,000,000** PSI		
I	=	**350** inches^4		
S	=	**58.1** inches^3		
$Fv_{Allowable}$	=	**9,500** PSI		SDOT 6-02.3(17)B For Salvaged Steel
		30,000 PSI	0.6**F_y**	AISC (Chapter G 16.1-64)
$Fp_{Allowable}$	=	**22,500** PSI		SDOT 6-02.3(17)B For Salvaged Steel
Fb	=	**33,000.00** PSI	0.66**F_y**	AISC (F3-1)
t_w	=	**0.335** inches		
d	=	**12.06** inches		
t_f	=	**0.575** inches		
F_y	=	**50,000** PSI		
k	=	**0.81** inches		
b_f	=	**8.045** inches		
W_t	=	**45** lbs/lf		

Reactions:

W_{design}	=	Design Load = DL x 1.2 + LL x 1.4 =	=	**1,293**	plf
V_{max}	=		=	**13,689**	lbs
P_{max}	=		=	**13,689**	lbs
M_{max}	=		=	**72,451**	ft-lbs
D_{max}	=		=	**0.576**	inches

Check Shear:

Fv	=	**V_{max}** / **A_{web}**	=	**3,398**	PSI
$Fv_{Allowable}$	=	0.6**F_y**	=	**30,000**	PSI
				Check - O.K.	
		V_{max}	=	**13,689**	LBS
$Fv_{Allowable}$	=	$0.66F_w t_w(N + 2.5k)$	=	**88,785**	LBS
		ASD Section K1.3		**Check - O.K.**	

Check Bearing-Web Crippling:

Fp	=	**P_{max}** / **A_{web}**	=	**3,388**	PSI
$Fp_{Allowable}$	=	0.6**F_y**	=	**30,000**	PSI
				Check - O.K.	
		V_{max}	=	**13,689**	LBS
$Fp_{Allowable}$	=	$0.66F_w t_w(N + 2.5k)$	=	**88,785**	LBS
		ASD Section K1.3		**Check - O.K.**	

Check Bending:

$S_{Req'd}$	=	**M_{max}**(12) / **Fb**	=	**26.3**	inches^3
$S_{Furnished}$	=			**58.1**	inches^3
				Check - O.K.	

Check Deflection:

$D_{max.}$	=	**D_{max}**	=	**0.576**	inches
$D_{Allowable}$	=	L/350	=	**0.726**	inches
				Check - O.K.	

HEADER BEAM DESIGN:

Assumptions:

1. 3.5 x 12.0 GLULAM Beam Western Species				**12.00**	feet span
	Therefore:	L_1	=	144	inches
2. Span 1 Supposrted				**1.00**	feet
Span 2 Supported				**19.25**	feet
4. Assume bearing area = (5.5" x 5.5")				30.25	$inches^2$

3.50" x 12.0" GluLam Girder Properties:

Reference: 2020 APA Glued Laminated Beam Design Tables

Western Species GluLam Beam

ANSI Design Values

A	=	**42.00**	$inches^2$
E	=	**1,800,000**	PSI
S	=	**84**	inches3
I	=	**504**	inches4
$Fv_{Allowable}$	=	**6,720**	LBS
$Fp_{Allowable}$	=	**405**	PSI
Fb	=	**2,400**	PSI
$M_{Allowable}$	=	**16,800**	FT-LBS
W	=	**10.2**	LBS

Reactions:

W	=	Floor Framing DL	12.3	PSF
		Glulam DL	10.2	PLF
		Floor LL	40	PSF
		Design Load = DL x 1.2 + LL x 1.6 =	809.69	PLF

Reactions:

V_{max}	=	0.5**WL**	=	**4,858**	lbs
P_{max}	=	0.5**WL**	=	**4,858**	lbs
M_{max}	=	**(WL^2)**/8	=	**14,574**	ft-lbs
D_{max}	=	0.013**WL^4 / EI**	=	**0.42**	inches

Check Shear:

Fv	=	**V_{max}**(1.5) / **A**	=	**4,858**	lbs
$Fv_{Allowable}$	=			**6,720**	lbs
				Check - O.K.	

Check Bearing:

Fp	=	**P_{max} / A**	=	**160.60**	PSI
$Fp_{Allowable}$	=			**405**	PSI
				Check - O.K.	

Check Bending:

M_{max}	=		=	**14,574**	FT-LBS
$Mm_{Allowable}$	=			**16,800**	FT-LBS
				Check - O.K.	

Check Deflection:

$D_{max.}$	=			**0.4157**	inches
$D_{Allowable}$	=	**L**/350	=	**0.4114**	inches
				Check - O.K.	

HEADER BEAM DESIGN:

Assumptions:

1. 5.5 x 15.0 GLULAM Beam Western Species				**16.33**	feet span
	Therefore:	L_1	=	195.96	inches
2. Span 1 Supposrted				**6.50**	feet
Span 2 Supported				**19.25**	feet
4. Assume bearing area = (5.5" x 5.5")				30.25	$inches^2$

5.50" x 15.0" GluLam Girder Properties:

Reference: 2020 APA Glued Laminated Beam Design Tables

Western Species GluLam Beam

ANSI Design Values

A	=	**82.50**	$inches^2$
E	=	**1,800,000**	PSI
S	=	**206**	inches3
I	=	**1,547**	inches4
$Fv_{Allowable}$	=	**13,200**	LBS
$Fp_{Allowable}$	=	**405**	PSI
Fb	=	**2,400**	PSI
$M_{Allowable}$	=	**41,250**	FT-LBS
W	=	**20.1**	LBS

Reactions:

W	=	Floor Framing DL	12.3	PSF
		Glulam DL	20.1	PLF
		Floor LL	40	PSF
		Design Load = DL x 1.2 + LL x 1.6 =	1,038.16	PLF

Reactions:

V_{max}	=	0.5**WL**	=	**8,477**	lbs
P_{max}	=	0.5**WL**	=	**8,477**	lbs
M_{max}	=	$(WL^2)/8$	=	**34,605**	ft-lbs
D_{max}	=	$0.013WL^4 / EI$	=	**0.60**	inches

Check Shear:

Fv	=	$V_{max}(1.5) / A$	=	**8,477**	lbs
$Fv_{Allowable}$	=			**13,200**	lbs
				Check - O.K.	

Check Bearing:

Fp	=	P_{max} / A	=	**280.22**	PSI
$Fp_{Allowable}$	=			**405**	PSI
				Check - O.K.	

Check Bending:

M_{max}	=		=	**34,605**	FT-LBS
$Mm_{Allowable}$	=			**41,250**	FT-LBS
				Check - O.K.	

Check Deflection:

$D_{max.}$	=			**0.5956**	inches
$D_{Allowable}$	=	**L**/350	=	**0.5599**	inches
				Header is supported in three places	
				Check - O.K.	

Second Level Header Beam - 12' Span

TABLE 1

DOUGLAS-FIR GLUED LAMINATED BEAM SECTION PROPERTIES AND CAPACITIES
F_b = 2,400 PSI, E = 1,800,000 PSI, F_v = 240 PSI

3-1/8-INCH WIDTH															
Depth (in.)	6	7-1/2	9	10-1/2	12	13-1/2	15	16-1/2	18	19-1/2	21	22-1/2	24	25-1/2	27
Beam Weight (lb/ft)[1]	4.6	5.7	6.8	8.0	9.1	10.3	11.4	12.5	13.7	14.8	16.0	17.1	18.2	19.4	20.5
A (in.^{2})	18.8	23.4	28.1	32.8	37.5	42.2	46.9	51.6	56.3	60.9	65.6	70.3	75.0	79.7	84.4
S (in.^{3})	19	29	42	57	75	95	117	142	169	198	230	264	300	339	380
I (in.^{4})	56	110	190	301	450	641	879	1170	1519	1931	2412	2966	3600	4318	5126
EI (106 lb-in.^{2})	101	198	342	543	810	1153	1582	2106	2734	3476	4341	5339	6480	7773	9226
Moment Capacity (lb-ft)[2][3]	3750	5859	8438	11484	15000	18984	23438	28359	33750	39609	45938	52734	60000	67734	75938
Shear Capacity (lb)[3]	3000	3750	4500	5250	6000	6750	7500	8250	9000	9750	10500	11250	12000	12750	13500
3-1/2-INCH WIDTH															
Depth (in.)	6	7-1/2	9	10-1/2	12	13-1/2	15	16-1/2	18	19-1/2	21	22-1/2	24	25-1/2	27
Beam Weight (lb/ft)[1]	5.1	6.4	7.7	8.9	10.2	11.5	12.8	14.0	15.3	16.6	17.9	19.1	20.4	21.7	23.0
A (in.^{2})	21.0	26.3	31.5	36.8	42.0	47.3	52.5	57.8	63.0	68.3	73.5	78.8	84.0	89.3	94.5
S (in.^{3})	21	33	47	64	84	106	131	159	189	222	257	295	336	379	425
I (in.^{4})	63	123	213	338	504	718	984	1310	1701	2163	2701	3322	4032	4836	5741
EI (106 lb-in.^{2})	113	221	383	608	907	1292	1772	2358	3062	3893	4862	5980	7258	8705	10334
Moment Capacity (lb-ft)[2][3]	4200	6563	9450	12863	16800	21263	26250	31763	37800	44363	51450	59063	67200	75863	85050
Shear Capacity (lb)[3]	3360	4200	5040	5880	6720	7560	8400	9240	10080	10920	11760	12600	13440	14280	15120
5-1/8-INCH WIDTH															
Depth (in.)	12	13-1/2	15	16-1/2	18	19-1/2	21	22-1/2	24	25-1/2	27	28-1/2	30	31-1/2	33
Beam Weight (lb/ft)[1]	14.9	16.8	18.7	20.6	22.4	24.3	26.2	28.0	29.9	31.8	33.6	35.5	37.4	39.2	41.1
A (in.^{2})	61.5	69.2	76.9	84.6	92.3	99.9	107.6	115.3	123.0	130.7	138.4	146.1	153.8	161.4	169.1
S (in.^{3})	123	156	192	233	277	325	377	432	492	555	623	694	769	848	930
I (in.^{4})	738	1051	1441	1919	2491	3167	3955	4865	5904	7082	8406	9887	11531	13349	15348
EI (106 lb-in.^{2})	1328	1891	2595	3453	4483	5700	7119	8757	10627	12747	15131	17796	20756	24028	27627
Moment Capacity (lb-ft)[2][3]	24600	31134	38438	46509	55350	64959	75338	86484	98400	111084	124538	138759	153750	169509	186038
Shear Capacity (lb)[3]	9840	11070	12300	13530	14760	15990	17220	18450	19680	20910	22140	23370	24600	25830	27060
5-1/2-INCH WIDTH															
Depth (in.)	12	13-1/2	15	16-1/2	18	19-1/2	21	22-1/2	24	25-1/2	27	28-1/2	30	31-1/2	33
Beam Weight (lb/ft)[1]	16.0	18.0	20.1	22.1	24.1	26.1	28.1	30.1	32.1	34.1	36.1	38.1	40.1	42.1	44.1
A (in.^{2})	66.0	74.3	82.5	90.8	99.0	107.3	115.5	123.8	132.0	140.3	148.5	156.8	165.0	173.3	181.5
S (in.^{3})	132	167	206	250	297	349	404	464	528	596	668	745	825	910	998
I (in.^{4})	792	1128	1547	2059	2673	3398	4245	5221	6336	7600	9021	10610	12375	14326	16471
EI (106 lb-in.^{2})	1426	2030	2784	3706	4811	6117	7640	9397	11405	13680	16238	19098	22275	25786	29648
Moment Capacity (lb-ft)[2][3]	26400	33413	41250	49913	59400	69713	80850	92813	105600	119213	133650	148913	165000	181913	199650
Shear Capacity (lb)[3]	10560	11880	13200	14520	15840	17160	18480	19800	21120	22440	23760	25080	26400	27720	29040
6-3/4-INCH WIDTH															
Depth (in.)	18	19-1/2	21	22-1/2	24	25-1/2	27	28-1/2	30	31-1/2	33	34-1/2	36	37-1/2	39
Beam Weight (lb/ft)[1]	29.5	32.0	34.5	36.9	39.4	41.8	44.3	46.8	49.2	51.7	54.1	56.6	59.1	61.5	64.0
A (in.^{2})	121.5	131.6	141.8	151.9	162.0	172.1	182.3	192.4	202.5	212.6	222.8	232.9	243.0	253.1	263.3
S (in.^{3})	365	428	496	570	648	732	820	914	1013	1116	1225	1339	1458	1582	1711
I (in.^{4})	3281	4171	5209	6407	7776	9327	11072	13021	15188	17581	20215	23098	26244	29663	33367
EI (106 lb-in.^{2})	5905	7508	9377	11533	13997	16789	19929	23438	27338	31647	36386	41577	47239	53394	60060
Moment Capacity (lb-ft)[2][3]	72900	85556	99225	113906	129600	146306	164025	182756	202500	223256	245025	267806	291600	316406	342225
Shear Capacity (lb)[3]	19440	21060	22680	24300	25920	27540	29160	30780	32400	34020	35640	37260	38880	40500	42120
8-3/4-INCH WIDTH															
Depth (in.)	24	25-1/2	27	28-1/2	30	31-1/2	33	34-1/2	36	37-1/2	39	40-1/2	42	43-1/2	45
Beam Weight (lb/ft)[1]	51.0	54.2	57.4	60.6	63.8	67.0	70.2	73.4	76.6	79.8	82.9	86.1	89.3	92.5	95.7
A (in.^{2})	210.0	223.1	236.3	249.4	262.5	275.6	288.8	301.9	315.0	328.1	341.3	354.4	367.5	380.6	393.8
S (in.^{3})	840	948	1063	1185	1313	1447	1588	1736	1890	2051	2218	2392	2573	2760	2953
I (in.^{4})	10080	12091	14352	16880	19688	22791	26204	29942	34020	38452	43253	48439	54023	60020	66445
EI (106 lb-in.^{2})	18144	21763	25834	30383	35438	41023	47167	53896	61236	69214	77856	87190	97241	108036	119602
Moment Capacity (lb-ft)[2][3]	168000	189656	212625	236906	262500	289406	317625	347156	378000	410156	443625	478406	514500	551906	590625
Shear Capacity (lb)[3]	33600	35700	37800	39900	42000	44100	46200	48300	50400	52500	54600	56700	58800	60900	63000

Notes:
(1) Beam weight is based on density of 35 pcf.
(2) Moment capacity must be adjusted for volume effect. The volume factor for various glulam sizes and simple spans, as well as the complete formula, is given in Appendix A.
(3) Moment and shear capacities are based on a normal (10 years) duration of load and should be adjusted for the design duration of load per the applicable building code.

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Second Level Header Beam - 16' Span

TABLE 1

DOUGLAS-FIR GLUED LAMINATED BEAM SECTION PROPERTIES AND CAPACITIES
F_b = 2,400 PSI, E = 1,800,000 PSI, F_v = 240 PSI

3-1/8-INCH WIDTH															
Depth (in.)	6	7-1/2	9	10-1/2	12	13-1/2	15	16-1/2	18	19-1/2	21	22-1/2	24	25-1/2	27
Beam Weight (lb/ft)[1]	4.6	5.7	6.8	8.0	9.1	10.3	11.4	12.5	13.7	14.8	16.0	17.1	18.2	19.4	20.5
A (in.^{2})	18.8	23.4	28.1	32.8	37.5	42.2	46.9	51.6	56.3	60.9	65.6	70.3	75.0	79.7	84.4
S (in.^{3})	19	29	42	57	75	95	117	142	169	198	230	264	300	339	380
I (in.^{4})	56	110	190	301	450	641	879	1170	1519	1931	2412	2966	3600	4318	5126
EI (106 lb-in.^{2})	101	198	342	543	810	1153	1582	2106	2734	3476	4341	5339	6480	7773	9226
Moment Capacity (lb-ft)[2][3]	3750	5859	8438	11484	15000	18984	23438	28359	33750	39609	45938	52734	60000	67734	75938
Shear Capacity (lb)[3]	3000	3750	4500	5250	6000	6750	7500	8250	9000	9750	10500	11250	12000	12750	13500
3-1/2-INCH WIDTH															
Depth (in.)	6	7-1/2	9	10-1/2	12	13-1/2	15	16-1/2	18	19-1/2	21	22-1/2	24	25-1/2	27
Beam Weight (lb/ft)[1]	5.1	6.4	7.7	8.9	10.2	11.5	12.8	14.0	15.3	16.6	17.9	19.1	20.4	21.7	23.0
A (in.^{2})	21.0	26.3	31.5	36.8	42.0	47.3	52.5	57.8	63.0	68.3	73.5	78.8	84.0	89.3	94.5
S (in.^{3})	21	33	47	64	84	106	131	159	189	222	257	295	336	379	425
I (in.^{4})	63	123	213	338	504	718	984	1310	1701	2163	2701	3322	4032	4836	5741
EI (106 lb-in.^{2})	113	221	383	608	907	1292	1772	2358	3062	3893	4862	5980	7258	8705	10334
Moment Capacity (lb-ft)[2][3]	4200	6563	9450	12863	16800	21263	26250	31763	37800	44363	51450	59063	67200	75863	85050
Shear Capacity (lb)[3]	3360	4200	5040	5880	6720	7560	8400	9240	10080	10920	11760	12600	13440	14280	15120
5-1/8-INCH WIDTH															
Depth (in.)	12	13-1/2	15	16-1/2	18	19-1/2	21	22-1/2	24	25-1/2	27	28-1/2	30	31-1/2	33
Beam Weight (lb/ft)[1]	14.9	16.8	18.7	20.6	22.4	24.3	26.2	28.0	29.9	31.8	33.6	35.5	37.4	39.2	41.1
A (in.^{2})	61.5	69.2	76.9	84.6	92.3	99.9	107.6	115.3	123.0	130.7	138.4	146.1	153.8	161.4	169.1
S (in.^{3})	123	156	192	233	277	325	377	432	492	555	623	694	769	848	930
I (in.^{4})	738	1051	1441	1919	2491	3167	3955	4865	5904	7082	8406	9887	11531	13349	15348
EI (106 lb-in.^{2})	1328	1891	2595	3453	4483	5700	7119	8757	10627	12747	15131	17796	20756	24028	27627
Moment Capacity (lb-ft)[2][3]	24600	31134	38438	46509	55350	64959	75338	86484	98400	111084	124538	138759	153750	169509	186038
Shear Capacity (lb)[3]	9840	11070	12300	13530	14760	15990	17220	18450	19680	20910	22140	23370	24600	25830	27060
5-1/2-INCH WIDTH															
Depth (in.)	12	13-1/2	15	16-1/2	18	19-1/2	21	22-1/2	24	25-1/2	27	28-1/2	30	31-1/2	33
Beam Weight (lb/ft)[1]	16.0	18.0	20.1	22.1	24.1	26.1	28.1	30.1	32.1	34.1	36.1	38.1	40.1	42.1	44.1
A (in.^{2})	66.0	74.3	82.5	90.8	99.0	107.3	115.5	123.8	132.0	140.3	148.5	156.8	165.0	173.3	181.5
S (in.^{3})	132	167	206	250	297	349	404	464	528	596	668	745	825	910	998
I (in.^{4})	792	1128	1547	2059	2673	3398	4245	5221	6336	7600	9021	10610	12375	14326	16471
EI (106 lb-in.^{2})	1426	2030	2784	3706	4811	6117	7640	9397	11405	13680	16238	19098	22275	25786	29648
Moment Capacity (lb-ft)[2][3]	26400	33413	41250	49913	59400	69713	80850	92813	105600	119213	133650	148913	165000	181913	199650
Shear Capacity (lb)[3]	10560	11880	13200	14520	15840	17160	18480	19800	21120	22440	23760	25080	26400	27720	29040
6-3/4-INCH WIDTH															
Depth (in.)	18	19-1/2	21	22-1/2	24	25-1/2	27	28-1/2	30	31-1/2	33	34-1/2	36	37-1/2	39
Beam Weight (lb/ft)[1]	29.5	32.0	34.5	36.9	39.4	41.8	44.3	46.8	49.2	51.7	54.1	56.6	59.1	61.5	64.0
A (in.^{2})	121.5	131.6	141.8	151.9	162.0	172.1	182.3	192.4	202.5	212.6	222.8	232.9	243.0	253.1	263.3
S (in.^{3})	365	428	496	570	648	732	820	914	1013	1116	1225	1339	1458	1582	1711
I (in.^{4})	3281	4171	5209	6407	7776	9327	11072	13021	15188	17581	20215	23098	26244	29663	33367
EI (106 lb-in.^{2})	5905	7508	9377	11533	13997	16789	19929	23438	27338	31647	36386	41577	47239	53394	60060
Moment Capacity (lb-ft)[2][3]	72900	85556	99225	113906	129600	146306	164025	182756	202500	223256	245025	267806	291600	316406	342225
Shear Capacity (lb)[3]	19440	21060	22680	24300	25920	27540	29160	30780	32400	34020	35640	37260	38880	40500	42120
8-3/4-INCH WIDTH															
Depth (in.)	24	25-1/2	27	28-1/2	30	31-1/2	33	34-1/2	36	37-1/2	39	40-1/2	42	43-1/2	45
Beam Weight (lb/ft)[1]	51.0	54.2	57.4	60.6	63.8	67.0	70.2	73.4	76.6	79.8	82.9	86.1	89.3	92.5	95.7
A (in.^{2})	210.0	223.1	236.3	249.4	262.5	275.6	288.8	301.9	315.0	328.1	341.3	354.4	367.5	380.6	393.8
S (in.^{3})	840	948	1063	1185	1313	1447	1588	1736	1890	2051	2218	2392	2573	2760	2953
I (in.^{4})	10080	12091	14352	16880	19688	22791	26204	29942	34020	38452	43253	48439	54023	60020	66445
EI (106 lb-in.^{2})	18144	21763	25834	30383	35438	41023	47167	53896	61236	69214	77856	87190	97241	108036	119602
Moment Capacity (lb-ft)[2][3]	168000	189656	212625	236906	262500	289406	317625	347156	378000	410156	443625	478406	514500	551906	590625
Shear Capacity (lb)[3]	33600	35700	37800	39900	42000	44100	46200	48300	50400	52500	54600	56700	58800	60900	63000

Notes:
(1) Beam weight is based on density of 35 pcf.
(2) Moment capacity must be adjusted for volume effect. The volume factor for various glulam sizes and simple spans, as well as the complete formula, is given in Appendix A.
(3) Moment and shear capacities are based on a normal (10 years) duration of load and should be adjusted for the design duration of load per the applicable building code.

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TJI - FLOOR JOIST DESIGN

Second Floor Level

Assumptions:

1. 9.5 210 Floor Joists
2. Supports spaced @: **23** feet
3. Floor Joist Spacing: **2.00** feet O.C.
4. Minimum Bearing Area: 2.31" x 3.5" = **8.09** $inches^2$

9.5" 210 Properties:

Weight	=	**2.60**	PLF
EI	=	**186,000,000**	PSI
$\mathbf{M}_{Allowable}$	=	**3,000**	$inches^4$
$\mathbf{V}_{Allowable}$	=	**1,005**	$inches^3$
$\mathbf{Pe}_{Allowable}$	=	**1330**	PSI
$\mathbf{Ps}_{Allowable}$	=	**1330**	PSI

Reactions: From Uniform Loading:

W = Factored Floor DL +LL = **80** PSF

$\mathbf{W_{max}}$	=	W x Joist Spacing	=	**160**	PLF
$\mathbf{V_{max}}$	=	$\mathbf{WL/2 + P_L}$	=	**1,840**	lbs
$\mathbf{P_{max}}$	=	$\mathbf{V_{max} + P_L}$	=	**1,840**	lbs
$\mathbf{M_{max}}$	=	$\mathbf{(WL^2)/8 + (P_L*L/4)}$	=	**10,580**	ft-lbs
$\mathbf{D_{max}}$	=	$(5\mathbf{WL}^4/384\mathbf{EI})+(PL^3/48EI)$	=	**0.45**	inches
			=		

Check Shear:

$\mathbf{V_{max}}$	=	=	**1,840**	PSI
$\mathbf{V}_{Allowable}$	=	=	**1,330**	PSI
			Check - O.K.	

Check Bearing:

$\mathbf{P_{max}}$	=	=	**1,840**	PSI
$\mathbf{Ps}_{Allowable}$	=		**1,330**	PSI
			Check - O.K.	

Check Bending:

$\mathbf{M_{max}}$	=	=	**10,580**	$inches^3$
$\mathbf{M}_{Allowable}$	=		**3,000**	$inches^3$
			Check - O.K.	

Check Deflection:

$D_{max.}$	=			**0.45**	inches
$D_{Allowable}$	=	**L**/350	=	**0.79**	inches
				Check - O.K.	

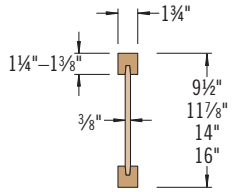
SECTION 1: 9½"–16" TJI® JOISTS

This section contains design information for 9½"–16" deep Trus Joist® TJI® joists.
These standard-size TJI® joists are readily available through your local Weyerhaeuser dealer or distributor.
Offered with the flange sizes shown below, they come in lengths up to 60' (in 1' increments).

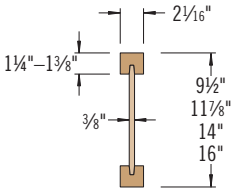
Design Properties (100% Load Duration)

Depth	TJI®	Basic Properties				Reaction Properties					
		Joist Weight (lbs/ft)	Maximum Resistive Moment(1) (ft-lbs)	Joist Only EI x 10^6 ($in.^2$-lbs)	Maximum Vertical Shear (lbs)	1¾" End Reaction (lbs)	3½" End Reaction (lbs)	3½" Intermediate Reaction (lbs)		5¼" Intermediate Reaction (lbs)	
								No Web Stiffeners	With Web Stiffeners(2)	No Web Stiffeners	With Web Stiffeners(2)
9½"	110	2.3	2,500	157	1,220	910	1,220	1,935	N.A.	2,350	N.A.
	210	2.6	3,000	186	1,330	1,005	1,330	2,145	N.A.	2,565	N.A.
	230	2.7	3,330	206	1,330	1,060	1,330	2,410	N.A.	2,790	N.A.
11⅞"	110	2.5	3,160	267	1,560	910	1,375	1,935	2,295	2,350	2,705
	210	2.8	3,795	315	1,655	1,005	1,460	2,145	2,505	2,565	2,925
	230	3.0	4,215	347	1,655	1,060	1,485	2,410	2,765	2,790	3,150
	360	3.0	6,180	419	1,705	1,080	1,505	2,460	2,815	3,000	3,360
	560	4.0	9,500	636	2,050	1,265	1,725	3,000	3,475	3,455	3,930
14"	110	2.8	3,740	392	1,860	910	1,375	1,935	2,295	2,350	2,705
	210	3.1	4,490	462	1,945	1,005	1,460	2,145	2,505	2,565	2,925
	230	3.3	4,990	509	1,945	1,060	1,485	2,410	2,765	2,790	3,150
	360	3.3	7,335	612	1,955	1,080	1,505	2,460	2,815	3,000	3,360
	560	4.2	11,275	926	2,390	1,265	1,725	3,000	3,475	3,455	3,930
16"	110	3.0	4,280	535	2,145	910	1,375	1,935	2,295	2,350	2,705
	210	3.3	5,140	629	2,190	1,005	1,460	2,145	2,505	2,565	2,925
	230	3.5	5,710	691	2,190	1,060	1,485	2,410	2,765	2,790	3,150
	360	3.5	8,405	830	2,190	1,080	1,505	2,460	2,815	3,000	3,360
	560	4.5	12,925	1,252	2,710	1,265	1,725	3,000	3,475	3,455	3,930

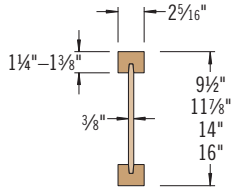
(1) **Caution: Do not** increase joist moment design properties by a repetitive member use factor.
(2) See detail W on page 27 for web stiffener requirements and nailing information.



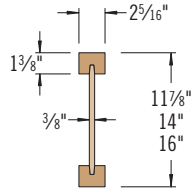
TJI® 110 joists



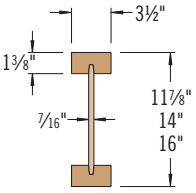
TJI® 210 joists



TJI® 230 joists



TJI® 360 joists



TJI® 560 joists

General Notes

- Design reaction includes all loads on the joist. Design shear is computed at the inside face of supports and includes all loads on the span(s). Allowable shear may sometimes be increased at interior supports in accordance with ICC ES ESR-1153, and these increases are reflected in span tables.
- The formulas at right approximate the uniform load deflection of Δ (inches).

TJI® joists are intended for dry-use applications

Some TJI® joist series may not be available in your region. Contact your Weyerhaeuser representative for information.

For TJI® 110, 210, and 360 Joists

$$\Delta = \frac{22.5\ wL^4}{EI} + \frac{2.67\ wL^2}{d \times 10^5}$$

For TJI® 560 Joists

$$\Delta = \frac{22.5\ wL^4}{EI} + \frac{2.29\ wL^2}{d \times 10^5}$$

w = uniform load in pounds per linear foot
L = span in feet
d = out-to-out depth of the joist in inches
EI = value from table above


DO NOT walk on joists until braced. INJURY MAY RESULT.


DO NOT stack building materials on unsheathed joists. Stack only over beams or walls.


DO NOT walk on joists that are lying flat.

WARNING

Joists are unstable until braced laterally

Bracing Includes:
- **Blocking**
- **Hangers**
- **Rim Board**
- **Sheathing**
- **Rim Joist**
- **Strut Lines**

WARNING NOTES: *Lack of proper bracing during construction can result in serious accidents. Observe the following guidelines:*

1. All blocking, hangers, rim boards, and rim joists at the end supports of the TJI® joists must be completely installed and properly nailed.
2. Lateral strength, like a braced end wall or an existing deck, must be established at the ends of the bay. This can also be accomplished by a temporary or permanent deck (sheathing) fastened to the first 4 feet of joists at the end of the bay.
3. Safety bracing of 1x4 (minimum) must be nailed to a braced end wall or sheathed area (as in note 2) and to each joist. Without this bracing, buckling sideways or rollover is highly probable under light construction loads—such as a worker or one layer of unnailed sheathing.
4. Sheathing must be completely attached to each TJI® joist before additional loads can be placed on the system.
5. Ends of cantilevers require safety bracing on both the top and bottom flanges.
6. The flanges must remain straight within a tolerance of ½" from true alignment.

TJI - FLOOR JOIST DESIGN

Green Roof

Assumptions:

1. 9.5 210 Floor Joists
2. Supports spaced @: **23** feet
3. Floor Joist Spacing: **2.00** feet O.C.
4. Minimum Bearing Area: 2.31" x 3.5" = **8.09** $inches^2$

9.5" 210 Properties:

Weight	=	**2.60**	PLF
EI	=	**186,000,000**	PSI
$\mathbf{M}_{Allowable}$	=	**3,000**	$inches^4$
$\mathbf{V}_{Allowable}$	=	**1,005**	$inches^3$
$\mathbf{Pe}_{Allowable}$	=	**1330**	PSI
$\mathbf{Ps}_{Allowable}$	=	**1330**	PSI

Reactions: From Uniform Loading:

W = Factored Floor DL +LL = **154** PSF

$\mathbf{W_{max}}$	=	W x Joist Spacing	=	**308**	PLF
$\mathbf{V_{max}}$	=	$\mathbf{WL/2 + P_L}$	=	**3,542**	lbs
$\mathbf{P_{max}}$	=	$\mathbf{V_{max} + P_L}$	=	**3,542**	lbs
$\mathbf{M_{max}}$	=	$\mathbf{(WL^2)/8 + (P_L*L/4)}$	=	**20,367**	ft-lbs
$\mathbf{D_{max}}$	=	$(5\mathbf{WL}^4/384\mathbf{EI}) + (PL^3/48EI)$	=	**0.87**	inches
			=		

Check Shear:

$\mathbf{V_{max}}$	=	=	**3,542**	PSI
$\mathbf{V}_{Allowable}$	=	=	**1,330**	PSI
			Check - O.K.	

Check Bearing:

$\mathbf{P_{max}}$	=	=	**3,542**	PSI
$\mathbf{Ps}_{Allowable}$	=		**1,330**	PSI
			Check - O.K.	

Check Bending:

$\mathbf{M_{max}}$	=	=	**20,367**	$inches^3$
$\mathbf{M}_{Allowable}$	=		**3,000**	$inches^3$
			Check - O.K.	

Check Deflection:

$D_{max.}$	=			**0.87**	inches
$D_{Allowable}$	=	**L**/350	=	**0.79**	inches
				Check - O.K.	

TJI - FLOOR JOIST DESIGN

First Floor Level

Assumptions:

1. 9.5 210 Floor Joists
2. Supports spaced @: **23** feet
3. Floor Joist Spacing: **2.00** feet O.C.
4. Minimum Bearing Area: 2.31" x 3.5" = **8.09** $inches^2$

9.5" 210 Properties:

Weight	=	**2.60**	PLF
EI	=	**186,000,000**	PSI
$\mathbf{M}_{Allowable}$	=	**3,000**	$inches^4$
$\mathbf{V}_{Allowable}$	=	**1,005**	$inches^3$
$\mathbf{Pe}_{Allowable}$	=	**1330**	PSI
$\mathbf{Ps}_{Allowable}$	=	**1330**	PSI

Reactions: From Uniform Loading:

W = Factored Floor DL +LL = **80** PSF

$\mathbf{W_{max}}$	=	W x Joist Spacing	=	**160**	PLF
$\mathbf{V_{max}}$	=	$\mathbf{WL/2 + P_L}$	=	**1,840**	lbs
$\mathbf{P_{max}}$	=	$\mathbf{V_{max} + P_L}$	=	**1,840**	lbs
$\mathbf{M_{max}}$	=	$\mathbf{(WL^2)/8 + (P_L*L/4)}$	=	**10,580**	ft-lbs
$\mathbf{D_{max}}$	=	$(5\mathbf{WL}^4/384\mathbf{EI})+(PL^3/48EI)$	=	**0.45**	inches
			=		

Check Shear:

$\mathbf{V_{max}}$	=	=	**1,840**	PSI
$\mathbf{V}_{Allowable}$	=	=	**1,330**	PSI
			Check - O.K.	

Check Bearing:

$\mathbf{P_{max}}$	=	=	**1,840**	PSI
$\mathbf{Ps}_{Allowable}$	=		**1,330**	PSI
			Check - O.K.	

Check Bending:

$\mathbf{M_{max}}$	=	=	**10,580**	$inches^3$
$\mathbf{M}_{Allowable}$	=		**3,000**	$inches^3$
			Check - O.K.	

Check Deflection:

$D_{max.}$	=			**0.45**	inches
$D_{Allowable}$	=	**L**/350	=	**0.79**	inches
				Check - O.K.	

Second Level Header Beam - 16' Span

TABLE 1

DOUGLAS-FIR GLUED LAMINATED BEAM SECTION PROPERTIES AND CAPACITIES
F_b = 2,400 PSI, E = 1,800,000 PSI, F_v = 240 PSI

3-1/8-INCH WIDTH															
Depth (in.)	6	7-1/2	9	10-1/2	12	13-1/2	15	16-1/2	18	19-1/2	21	22-1/2	24	25-1/2	27
Beam Weight (lb/ft)[1]	4.6	5.7	6.8	8.0	9.1	10.3	11.4	12.5	13.7	14.8	16.0	17.1	18.2	19.4	20.5
A (in.^{2})	18.8	23.4	28.1	32.8	37.5	42.2	46.9	51.6	56.3	60.9	65.6	70.3	75.0	79.7	84.4
S (in.^{3})	19	29	42	57	75	95	117	142	169	198	230	264	300	339	380
I (in.^{4})	56	110	190	301	450	641	879	1170	1519	1931	2412	2966	3600	4318	5126
EI (106 lb-in.^{2})	101	198	342	543	810	1153	1582	2106	2734	3476	4341	5339	6480	7773	9226
Moment Capacity (lb-ft)[2][3]	3750	5859	8438	11484	15000	18984	23438	28359	33750	39609	45938	52734	60000	67734	75938
Shear Capacity (lb)[3]	3000	3750	4500	5250	6000	6750	7500	8250	9000	9750	10500	11250	12000	12750	13500
3-1/2-INCH WIDTH															
Depth (in.)	6	7-1/2	9	10-1/2	12	13-1/2	15	16-1/2	18	19-1/2	21	22-1/2	24	25-1/2	27
Beam Weight (lb/ft)[1]	5.1	6.4	7.7	8.9	10.2	11.5	12.8	14.0	15.3	16.6	17.9	19.1	20.4	21.7	23.0
A (in.^{2})	21.0	26.3	31.5	36.8	42.0	47.3	52.5	57.8	63.0	68.3	73.5	78.8	84.0	89.3	94.5
S (in.^{3})	21	33	47	64	84	106	131	159	189	222	257	295	336	379	425
I (in.^{4})	63	123	213	338	504	718	984	1310	1701	2163	2701	3322	4032	4836	5741
EI (106 lb-in.^{2})	113	221	383	608	907	1292	1772	2358	3062	3893	4862	5980	7258	8705	10334
Moment Capacity (lb-ft)[2][3]	4200	6563	9450	12863	16800	21263	26250	31763	37800	44363	51450	59063	67200	75863	85050
Shear Capacity (lb)[3]	3360	4200	5040	5880	6720	7560	8400	9240	10080	10920	11760	12600	13440	14280	15120
5-1/8-INCH WIDTH															
Depth (in.)	12	13-1/2	15	16-1/2	18	19-1/2	21	22-1/2	24	25-1/2	27	28-1/2	30	31-1/2	33
Beam Weight (lb/ft)[1]	14.9	16.8	18.7	20.6	22.4	24.3	26.2	28.0	29.9	31.8	33.6	35.5	37.4	39.2	41.1
A (in.^{2})	61.5	69.2	76.9	84.6	92.3	99.9	107.6	115.3	123.0	130.7	138.4	146.1	153.8	161.4	169.1
S (in.^{3})	123	156	192	233	277	325	377	432	492	555	623	694	769	848	930
I (in.^{4})	738	1051	1441	1919	2491	3167	3955	4865	5904	7082	8406	9887	11531	13349	15348
EI (106 lb-in.^{2})	1328	1891	2595	3453	4483	5700	7119	8757	10627	12747	15131	17796	20756	24028	27627
Moment Capacity (lb-ft)[2][3]	24600	31134	38438	46509	55350	64959	75338	86484	98400	111084	124538	138759	153750	169509	186038
Shear Capacity (lb)[3]	9840	11070	12300	13530	14760	15990	17220	18450	19680	20910	22140	23370	24600	25830	27060
5-1/2-INCH WIDTH															
Depth (in.)	12	13-1/2	15	16-1/2	18	19-1/2	21	22-1/2	24	25-1/2	27	28-1/2	30	31-1/2	33
Beam Weight (lb/ft)[1]	16.0	18.0	20.1	22.1	24.1	26.1	28.1	30.1	32.1	34.1	36.1	38.1	40.1	42.1	44.1
A (in.^{2})	66.0	74.3	82.5	90.8	99.0	107.3	115.5	123.8	132.0	140.3	148.5	156.8	165.0	173.3	181.5
S (in.^{3})	132	167	206	250	297	349	404	464	528	596	668	745	825	910	998
I (in.^{4})	792	1128	1547	2059	2673	3398	4245	5221	6336	7600	9021	10610	12375	14326	16471
EI (106 lb-in.^{2})	1426	2030	2784	3706	4811	6117	7640	9397	11405	13680	16238	19098	22275	25786	29648
Moment Capacity (lb-ft)[2][3]	26400	33413	41250	49913	59400	69713	80850	92813	105600	119213	133650	148913	165000	181913	199650
Shear Capacity (lb)[3]	10560	11880	13200	14520	15840	17160	18480	19800	21120	22440	23760	25080	26400	27720	29040
6-3/4-INCH WIDTH															
Depth (in.)	18	19-1/2	21	22-1/2	24	25-1/2	27	28-1/2	30	31-1/2	33	34-1/2	36	37-1/2	39
Beam Weight (lb/ft)[1]	29.5	32.0	34.5	36.9	39.4	41.8	44.3	46.8	49.2	51.7	54.1	56.6	59.1	61.5	64.0
A (in.^{2})	121.5	131.6	141.8	151.9	162.0	172.1	182.3	192.4	202.5	212.6	222.8	232.9	243.0	253.1	263.3
S (in.^{3})	365	428	496	570	648	732	820	914	1013	1116	1225	1339	1458	1582	1711
I (in.^{4})	3281	4171	5209	6407	7776	9327	11072	13021	15188	17581	20215	23098	26244	29663	33367
EI (106 lb-in.^{2})	5905	7508	9377	11533	13997	16789	19929	23438	27338	31647	36386	41577	47239	53394	60060
Moment Capacity (lb-ft)[2][3]	72900	85556	99225	113906	129600	146306	164025	182756	202500	223256	245025	267806	291600	316406	342225
Shear Capacity (lb)[3]	19440	21060	22680	24300	25920	27540	29160	30780	32400	34020	35640	37260	38880	40500	42120
8-3/4-INCH WIDTH															
Depth (in.)	24	25-1/2	27	28-1/2	30	31-1/2	33	34-1/2	36	37-1/2	39	40-1/2	42	43-1/2	45
Beam Weight (lb/ft)[1]	51.0	54.2	57.4	60.6	63.8	67.0	70.2	73.4	76.6	79.8	82.9	86.1	89.3	92.5	95.7
A (in.^{2})	210.0	223.1	236.3	249.4	262.5	275.6	288.8	301.9	315.0	328.1	341.3	354.4	367.5	380.6	393.8
S (in.^{3})	840	948	1063	1185	1313	1447	1588	1736	1890	2051	2218	2392	2573	2760	2953
I (in.^{4})	10080	12091	14352	16880	19688	22791	26204	29942	34020	38452	43253	48439	54023	60020	66445
EI (106 lb-in.^{2})	18144	21763	25834	30383	35438	41023	47167	53896	61236	69214	77856	87190	97241	108036	119602
Moment Capacity (lb-ft)[2][3]	168000	189656	212625	236906	262500	289406	317625	347156	378000	410156	443625	478406	514500	551906	590625
Shear Capacity (lb)[3]	33600	35700	37800	39900	42000	44100	46200	48300	50400	52500	54600	56700	58800	60900	63000

Notes:
(1) Beam weight is based on density of 35 pcf.
(2) Moment capacity must be adjusted for volume effect. The volume factor for various glulam sizes and simple spans, as well as the complete formula, is given in Appendix A.
(3) Moment and shear capacities are based on a normal (10 years) duration of load and should be adjusted for the design duration of load per the applicable building code.

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HEADER BEAM DESIGN:

Assumptions:

1. 3.5 x 12.0 GLULAM Beam Western Species				**12.00**	feet span
	Therefore:	L_1	=	144	inches
2. Span 1 Supposrted				**1.00**	feet
Span 2 Supported				**19.25**	feet
4. Assume bearing area = (5.5" x 5.5")				30.25	$inches^2$

3.50" x 12.0" GluLam Girder Properties:

Reference: 2020 APA Glued Laminated Beam Design Tables

Western Species GluLam Beam

ANSI Design Values

A	=	**42.00**	$inches^2$
E	=	**1,800,000**	PSI
S	=	**84**	inches3
I	=	**504**	inches4
$Fv_{Allowable}$	=	**6,720**	LBS
$Fp_{Allowable}$	=	**405**	PSI
Fb	=	**2,400**	PSI
$M_{Allowable}$	=	**16,800**	FT-LBS
W	=	**10.2**	LBS

Reactions:

W	=	Floor Framing DL	12.3	PSF
		Glulam DL	10.2	PLF
		Floor LL	40	PSF
		Design Load = DL x 1.2 + LL x 1.6 =	809.69	PLF

Reactions:

V_{max}	=	0.5**WL**	=	**4,858**	lbs
P_{max}	=	0.5**WL**	=	**4,858**	lbs
M_{max}	=	**(WL^2)**/8	=	**14,574**	ft-lbs
D_{max}	=	0.013**WL^4 / EI**	=	**0.42**	inches

Check Shear:

Fv	=	**V_{max}**(1.5) / **A**	=	**4,858**	lbs
$Fv_{Allowable}$	=			**6,720**	lbs
				Check - O.K.	

Check Bearing:

Fp	=	**P_{max} / A**	=	**160.60**	PSI
$Fp_{Allowable}$	=			**405**	PSI
				Check - O.K.	

Check Bending:

M_{max}	=		=	**14,574**	FT-LBS
$Mm_{Allowable}$	=			**16,800**	FT-LBS
				Check - O.K.	

Check Deflection:

$D_{max.}$	=			**0.4157**	inches
$D_{Allowable}$	=	**L**/350	=	**0.4114**	inches
				Check - O.K.	

HEADER BEAM DESIGN:

Assumptions:

1. 3.5 x 12.0 GLULAM Beam Western Species				**12.00**	feet span
	Therefore:	L_1	=	144	inches
2. Span 1 Supposrted				**1.00**	feet
Span 2 Supported				**19.25**	feet
4. Assume bearing area = (5.5" x 5.5")				30.25	$inches^2$

3.50" x 12.0" GluLam Girder Properties:

Reference: 2020 APA Glued Laminated Beam Design Tables

Western Species GluLam Beam

ANSI Design Values

A	=	**42.00**	$inches^2$
E	=	**1,800,000**	PSI
S	=	**84**	inches3
I	=	**504**	inches4
$Fv_{Allowable}$	=	**6,720**	LBS
$Fp_{Allowable}$	=	**405**	PSI
Fb	=	**2,400**	PSI
$M_{Allowable}$	=	**16,800**	FT-LBS
W	=	**10.2**	LBS

Reactions:

W	=	Floor Framing DL	12.3	PSF
		Glulam DL	10.2	PLF
		Floor LL	40	PSF
		Design Load = DL x 1.2 + LL x 1.6 =	809.69	PLF

Reactions:

V_{max}	=	0.5**WL**	=	**4,858**	lbs
P_{max}	=	0.5**WL**	=	**4,858**	lbs
M_{max}	=	**(WL^2)**/8	=	**14,574**	ft-lbs
D_{max}	=	0.013**WL^4 / EI**	=	**0.42**	inches

Check Shear:

Fv	=	**V_{max}**(1.5) / **A**	=	**4,858**	lbs
$Fv_{Allowable}$	=			**6,720**	lbs
				Check - O.K.	

Check Bearing:

Fp	=	**P_{max} / A**	=	**160.60**	PSI
$Fp_{Allowable}$	=			**405**	PSI
				Check - O.K.	

Check Bending:

M_{max}	=		=	**14,574**	FT-LBS
$Mm_{Allowable}$	=			**16,800**	FT-LBS
				Check - O.K.	

Check Deflection:

$D_{max.}$	=			**0.4157**	inches
$D_{Allowable}$	=	**L**/350	=	**0.4114**	inches
				Check - O.K.	

HEADER BEAM DESIGN:

Assumptions:

1. 5.5 x 15.0 GLULAM Beam Western Species				**16.33**	feet span
	Therefore:	L_1	=	195.96	inches
2. Span 1 Supposrted				**6.50**	feet
Span 2 Supported				**19.25**	feet
4. Assume bearing area = (5.5" x 5.5")				30.25	$inches^2$

5.50" x 15.0" GluLam Girder Properties:

Reference: 2020 APA Glued Laminated Beam Design Tables

Western Species GluLam Beam

ANSI Design Values

A	=	**82.50**	$inches^2$
E	=	**1,800,000**	PSI
S	=	**206**	inches3
I	=	**1,547**	inches4
$Fv_{Allowable}$	=	**13,200**	LBS
$Fp_{Allowable}$	=	**405**	PSI
Fb	=	**2,400**	PSI
$M_{Allowable}$	=	**41,250**	FT-LBS
W	=	**20.1**	LBS

Reactions:

W	=	Floor Framing DL	12.3	PSF
		Glulam DL	20.1	PLF
		Floor LL	40	PSF
		Design Load = DL x 1.2 + LL x 1.6 =	1,038.16	PLF

Reactions:

V_{max}	=	0.5**WL**	=	**8,477**	lbs
P_{max}	=	0.5**WL**	=	**8,477**	lbs
M_{max}	=	$(WL^2)/8$	=	**34,605**	ft-lbs
D_{max}	=	$0.013WL^4 / EI$	=	**0.60**	inches

Check Shear:

Fv	=	$V_{max}(1.5) / A$	=	**8,477**	lbs
$Fv_{Allowable}$	=			**13,200**	lbs
				Check - O.K.	

Check Bearing:

Fp	=	P_{max} / A	=	**280.22**	PSI
$Fp_{Allowable}$	=			**405**	PSI
				Check - O.K.	

Check Bending:

M_{max}	=		=	**34,605**	FT-LBS
$Mm_{Allowable}$	=			**41,250**	FT-LBS
				Check - O.K.	

Check Deflection:

$D_{max.}$	=			**0.5956**	inches
$D_{Allowable}$	=	**L**/350	=	**0.5599**	inches
				Header is supported in three places	
				Check - O.K.	

Second Level Header Beam - 12' Span

TABLE 1

DOUGLAS-FIR GLUED LAMINATED BEAM SECTION PROPERTIES AND CAPACITIES
F_b = 2,400 PSI, E = 1,800,000 PSI, F_v = 240 PSI

3-1/8-INCH WIDTH															
Depth (in.)	6	7-1/2	9	10-1/2	12	13-1/2	15	16-1/2	18	19-1/2	21	22-1/2	24	25-1/2	27
Beam Weight (lb/ft)[1]	4.6	5.7	6.8	8.0	9.1	10.3	11.4	12.5	13.7	14.8	16.0	17.1	18.2	19.4	20.5
A (in.^{2})	18.8	23.4	28.1	32.8	37.5	42.2	46.9	51.6	56.3	60.9	65.6	70.3	75.0	79.7	84.4
S (in.^{3})	19	29	42	57	75	95	117	142	169	198	230	264	300	339	380
I (in.^{4})	56	110	190	301	450	641	879	1170	1519	1931	2412	2966	3600	4318	5126
EI (106 lb-in.^{2})	101	198	342	543	810	1153	1582	2106	2734	3476	4341	5339	6480	7773	9226
Moment Capacity (lb-ft)[2][3]	3750	5859	8438	11484	15000	18984	23438	28359	33750	39609	45938	52734	60000	67734	75938
Shear Capacity (lb)[3]	3000	3750	4500	5250	6000	6750	7500	8250	9000	9750	10500	11250	12000	12750	13500
3-1/2-INCH WIDTH															
Depth (in.)	6	7-1/2	9	10-1/2	12	13-1/2	15	16-1/2	18	19-1/2	21	22-1/2	24	25-1/2	27
Beam Weight (lb/ft)[1]	5.1	6.4	7.7	8.9	10.2	11.5	12.8	14.0	15.3	16.6	17.9	19.1	20.4	21.7	23.0
A (in.^{2})	21.0	26.3	31.5	36.8	42.0	47.3	52.5	57.8	63.0	68.3	73.5	78.8	84.0	89.3	94.5
S (in.^{3})	21	33	47	64	84	106	131	159	189	222	257	295	336	379	425
I (in.^{4})	63	123	213	338	504	718	984	1310	1701	2163	2701	3322	4032	4836	5741
EI (106 lb-in.^{2})	113	221	383	608	907	1292	1772	2358	3062	3893	4862	5980	7258	8705	10334
Moment Capacity (lb-ft)[2][3]	4200	6563	9450	12863	16800	21263	26250	31763	37800	44363	51450	59063	67200	75863	85050
Shear Capacity (lb)[3]	3360	4200	5040	5880	6720	7560	8400	9240	10080	10920	11760	12600	13440	14280	15120
5-1/8-INCH WIDTH															
Depth (in.)	12	13-1/2	15	16-1/2	18	19-1/2	21	22-1/2	24	25-1/2	27	28-1/2	30	31-1/2	33
Beam Weight (lb/ft)[1]	14.9	16.8	18.7	20.6	22.4	24.3	26.2	28.0	29.9	31.8	33.6	35.5	37.4	39.2	41.1
A (in.^{2})	61.5	69.2	76.9	84.6	92.3	99.9	107.6	115.3	123.0	130.7	138.4	146.1	153.8	161.4	169.1
S (in.^{3})	123	156	192	233	277	325	377	432	492	555	623	694	769	848	930
I (in.^{4})	738	1051	1441	1919	2491	3167	3955	4865	5904	7082	8406	9887	11531	13349	15348
EI (106 lb-in.^{2})	1328	1891	2595	3453	4483	5700	7119	8757	10627	12747	15131	17796	20756	24028	27627
Moment Capacity (lb-ft)[2][3]	24600	31134	38438	46509	55350	64959	75338	86484	98400	111084	124538	138759	153750	169509	186038
Shear Capacity (lb)[3]	9840	11070	12300	13530	14760	15990	17220	18450	19680	20910	22140	23370	24600	25830	27060
5-1/2-INCH WIDTH															
Depth (in.)	12	13-1/2	15	16-1/2	18	19-1/2	21	22-1/2	24	25-1/2	27	28-1/2	30	31-1/2	33
Beam Weight (lb/ft)[1]	16.0	18.0	20.1	22.1	24.1	26.1	28.1	30.1	32.1	34.1	36.1	38.1	40.1	42.1	44.1
A (in.^{2})	66.0	74.3	82.5	90.8	99.0	107.3	115.5	123.8	132.0	140.3	148.5	156.8	165.0	173.3	181.5
S (in.^{3})	132	167	206	250	297	349	404	464	528	596	668	745	825	910	998
I (in.^{4})	792	1128	1547	2059	2673	3398	4245	5221	6336	7600	9021	10610	12375	14326	16471
EI (106 lb-in.^{2})	1426	2030	2784	3706	4811	6117	7640	9397	11405	13680	16238	19098	22275	25786	29648
Moment Capacity (lb-ft)[2][3]	26400	33413	41250	49913	59400	69713	80850	92813	105600	119213	133650	148913	165000	181913	199650
Shear Capacity (lb)[3]	10560	11880	13200	14520	15840	17160	18480	19800	21120	22440	23760	25080	26400	27720	29040
6-3/4-INCH WIDTH															
Depth (in.)	18	19-1/2	21	22-1/2	24	25-1/2	27	28-1/2	30	31-1/2	33	34-1/2	36	37-1/2	39
Beam Weight (lb/ft)[1]	29.5	32.0	34.5	36.9	39.4	41.8	44.3	46.8	49.2	51.7	54.1	56.6	59.1	61.5	64.0
A (in.^{2})	121.5	131.6	141.8	151.9	162.0	172.1	182.3	192.4	202.5	212.6	222.8	232.9	243.0	253.1	263.3
S (in.^{3})	365	428	496	570	648	732	820	914	1013	1116	1225	1339	1458	1582	1711
I (in.^{4})	3281	4171	5209	6407	7776	9327	11072	13021	15188	17581	20215	23098	26244	29663	33367
EI (106 lb-in.^{2})	5905	7508	9377	11533	13997	16789	19929	23438	27338	31647	36386	41577	47239	53394	60060
Moment Capacity (lb-ft)[2][3]	72900	85556	99225	113906	129600	146306	164025	182756	202500	223256	245025	267806	291600	316406	342225
Shear Capacity (lb)[3]	19440	21060	22680	24300	25920	27540	29160	30780	32400	34020	35640	37260	38880	40500	42120
8-3/4-INCH WIDTH															
Depth (in.)	24	25-1/2	27	28-1/2	30	31-1/2	33	34-1/2	36	37-1/2	39	40-1/2	42	43-1/2	45
Beam Weight (lb/ft)[1]	51.0	54.2	57.4	60.6	63.8	67.0	70.2	73.4	76.6	79.8	82.9	86.1	89.3	92.5	95.7
A (in.^{2})	210.0	223.1	236.3	249.4	262.5	275.6	288.8	301.9	315.0	328.1	341.3	354.4	367.5	380.6	393.8
S (in.^{3})	840	948	1063	1185	1313	1447	1588	1736	1890	2051	2218	2392	2573	2760	2953
I (in.^{4})	10080	12091	14352	16880	19688	22791	26204	29942	34020	38452	43253	48439	54023	60020	66445
EI (106 lb-in.^{2})	18144	21763	25834	30383	35438	41023	47167	53896	61236	69214	77856	87190	97241	108036	119602
Moment Capacity (lb-ft)[2][3]	168000	189656	212625	236906	262500	289406	317625	347156	378000	410156	443625	478406	514500	551906	590625
Shear Capacity (lb)[3]	33600	35700	37800	39900	42000	44100	46200	48300	50400	52500	54600	56700	58800	60900	63000

Notes:
(1) Beam weight is based on density of 35 pcf.
(2) Moment capacity must be adjusted for volume effect. The volume factor for various glulam sizes and simple spans, as well as the complete formula, is given in Appendix A.
(3) Moment and shear capacities are based on a normal (10 years) duration of load and should be adjusted for the design duration of load per the applicable building code.

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Second Level Header Beam - 16' Span

TABLE 1

DOUGLAS-FIR GLUED LAMINATED BEAM SECTION PROPERTIES AND CAPACITIES
F_b = 2,400 PSI, E = 1,800,000 PSI, F_v = 240 PSI

3-1/8-INCH WIDTH															
Depth (in.)	6	7-1/2	9	10-1/2	12	13-1/2	15	16-1/2	18	19-1/2	21	22-1/2	24	25-1/2	27
Beam Weight (lb/ft)[1]	4.6	5.7	6.8	8.0	9.1	10.3	11.4	12.5	13.7	14.8	16.0	17.1	18.2	19.4	20.5
A (in.^{2})	18.8	23.4	28.1	32.8	37.5	42.2	46.9	51.6	56.3	60.9	65.6	70.3	75.0	79.7	84.4
S (in.^{3})	19	29	42	57	75	95	117	142	169	198	230	264	300	339	380
I (in.^{4})	56	110	190	301	450	641	879	1170	1519	1931	2412	2966	3600	4318	5126
EI (106 lb-in.^{2})	101	198	342	543	810	1153	1582	2106	2734	3476	4341	5339	6480	7773	9226
Moment Capacity (lb-ft)[2][3]	3750	5859	8438	11484	15000	18984	23438	28359	33750	39609	45938	52734	60000	67734	75938
Shear Capacity (lb)[3]	3000	3750	4500	5250	6000	6750	7500	8250	9000	9750	10500	11250	12000	12750	13500
3-1/2-INCH WIDTH															
Depth (in.)	6	7-1/2	9	10-1/2	12	13-1/2	15	16-1/2	18	19-1/2	21	22-1/2	24	25-1/2	27
Beam Weight (lb/ft)[1]	5.1	6.4	7.7	8.9	10.2	11.5	12.8	14.0	15.3	16.6	17.9	19.1	20.4	21.7	23.0
A (in.^{2})	21.0	26.3	31.5	36.8	42.0	47.3	52.5	57.8	63.0	68.3	73.5	78.8	84.0	89.3	94.5
S (in.^{3})	21	33	47	64	84	106	131	159	189	222	257	295	336	379	425
I (in.^{4})	63	123	213	338	504	718	984	1310	1701	2163	2701	3322	4032	4836	5741
EI (106 lb-in.^{2})	113	221	383	608	907	1292	1772	2358	3062	3893	4862	5980	7258	8705	10334
Moment Capacity (lb-ft)[2][3]	4200	6563	9450	12863	16800	21263	26250	31763	37800	44363	51450	59063	67200	75863	85050
Shear Capacity (lb)[3]	3360	4200	5040	5880	6720	7560	8400	9240	10080	10920	11760	12600	13440	14280	15120
5-1/8-INCH WIDTH															
Depth (in.)	12	13-1/2	15	16-1/2	18	19-1/2	21	22-1/2	24	25-1/2	27	28-1/2	30	31-1/2	33
Beam Weight (lb/ft)[1]	14.9	16.8	18.7	20.6	22.4	24.3	26.2	28.0	29.9	31.8	33.6	35.5	37.4	39.2	41.1
A (in.^{2})	61.5	69.2	76.9	84.6	92.3	99.9	107.6	115.3	123.0	130.7	138.4	146.1	153.8	161.4	169.1
S (in.^{3})	123	156	192	233	277	325	377	432	492	555	623	694	769	848	930
I (in.^{4})	738	1051	1441	1919	2491	3167	3955	4865	5904	7082	8406	9887	11531	13349	15348
EI (106 lb-in.^{2})	1328	1891	2595	3453	4483	5700	7119	8757	10627	12747	15131	17796	20756	24028	27627
Moment Capacity (lb-ft)[2][3]	24600	31134	38438	46509	55350	64959	75338	86484	98400	111084	124538	138759	153750	169509	186038
Shear Capacity (lb)[3]	9840	11070	12300	13530	14760	15990	17220	18450	19680	20910	22140	23370	24600	25830	27060
5-1/2-INCH WIDTH															
Depth (in.)	12	13-1/2	15	16-1/2	18	19-1/2	21	22-1/2	24	25-1/2	27	28-1/2	30	31-1/2	33
Beam Weight (lb/ft)[1]	16.0	18.0	20.1	22.1	24.1	26.1	28.1	30.1	32.1	34.1	36.1	38.1	40.1	42.1	44.1
A (in.^{2})	66.0	74.3	82.5	90.8	99.0	107.3	115.5	123.8	132.0	140.3	148.5	156.8	165.0	173.3	181.5
S (in.^{3})	132	167	206	250	297	349	404	464	528	596	668	745	825	910	998
I (in.^{4})	792	1128	1547	2059	2673	3398	4245	5221	6336	7600	9021	10610	12375	14326	16471
EI (106 lb-in.^{2})	1426	2030	2784	3706	4811	6117	7640	9397	11405	13680	16238	19098	22275	25786	29648
Moment Capacity (lb-ft)[2][3]	26400	33413	41250	49913	59400	69713	80850	92813	105600	119213	133650	148913	165000	181913	199650
Shear Capacity (lb)[3]	10560	11880	13200	14520	15840	17160	18480	19800	21120	22440	23760	25080	26400	27720	29040
6-3/4-INCH WIDTH															
Depth (in.)	18	19-1/2	21	22-1/2	24	25-1/2	27	28-1/2	30	31-1/2	33	34-1/2	36	37-1/2	39
Beam Weight (lb/ft)[1]	29.5	32.0	34.5	36.9	39.4	41.8	44.3	46.8	49.2	51.7	54.1	56.6	59.1	61.5	64.0
A (in.^{2})	121.5	131.6	141.8	151.9	162.0	172.1	182.3	192.4	202.5	212.6	222.8	232.9	243.0	253.1	263.3
S (in.^{3})	365	428	496	570	648	732	820	914	1013	1116	1225	1339	1458	1582	1711
I (in.^{4})	3281	4171	5209	6407	7776	9327	11072	13021	15188	17581	20215	23098	26244	29663	33367
EI (106 lb-in.^{2})	5905	7508	9377	11533	13997	16789	19929	23438	27338	31647	36386	41577	47239	53394	60060
Moment Capacity (lb-ft)[2][3]	72900	85556	99225	113906	129600	146306	164025	182756	202500	223256	245025	267806	291600	316406	342225
Shear Capacity (lb)[3]	19440	21060	22680	24300	25920	27540	29160	30780	32400	34020	35640	37260	38880	40500	42120
8-3/4-INCH WIDTH															
Depth (in.)	24	25-1/2	27	28-1/2	30	31-1/2	33	34-1/2	36	37-1/2	39	40-1/2	42	43-1/2	45
Beam Weight (lb/ft)[1]	51.0	54.2	57.4	60.6	63.8	67.0	70.2	73.4	76.6	79.8	82.9	86.1	89.3	92.5	95.7
A (in.^{2})	210.0	223.1	236.3	249.4	262.5	275.6	288.8	301.9	315.0	328.1	341.3	354.4	367.5	380.6	393.8
S (in.^{3})	840	948	1063	1185	1313	1447	1588	1736	1890	2051	2218	2392	2573	2760	2953
I (in.^{4})	10080	12091	14352	16880	19688	22791	26204	29942	34020	38452	43253	48439	54023	60020	66445
EI (106 lb-in.^{2})	18144	21763	25834	30383	35438	41023	47167	53896	61236	69214	77856	87190	97241	108036	119602
Moment Capacity (lb-ft)[2][3]	168000	189656	212625	236906	262500	289406	317625	347156	378000	410156	443625	478406	514500	551906	590625
Shear Capacity (lb)[3]	33600	35700	37800	39900	42000	44100	46200	48300	50400	52500	54600	56700	58800	60900	63000

Notes:
(1) Beam weight is based on density of 35 pcf.
(2) Moment capacity must be adjusted for volume effect. The volume factor for various glulam sizes and simple spans, as well as the complete formula, is given in Appendix A.
(3) Moment and shear capacities are based on a normal (10 years) duration of load and should be adjusted for the design duration of load per the applicable building code.

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Steel Floor Beam Design

Max Span - First Floor Level

Assumptions:

1. W10 x 100 Beam		**27.00**	Feet =	324.00	Inches
	Supports are:	**3.00**	inches from each end of the beam		
	Therefore:	**L**	=	318.00	Inches
2. Beam designed as a simple span.					
	Therefore:	**L_d**	**=**	318	Inches
3. Bearing area:	(Assume: 6"x6" Plate)	**N**	=	**6**	Inches

W10 x 100 Beam Properties:

A	=	**29.40** inches^2		
A_{web}	=	**4.16** inches2	(Web area from top of beam to bottom of beam x web width.)	
E	=	**29,000,000** PSI		
I	=	**623** inches^4		
S	=	**112** inches^3		
$Fv_{Allowable}$	=	**9,500** PSI		SDOT 6-02.3(17)B For Salvaged Steel
		30,000 PSI	0.6**F_y**	AISC (Chapter G 16.1-64)
$Fp_{Allowable}$	=	**22,500** PSI		SDOT 6-02.3(17)B For Salvaged Steel
Fb	=	**33,000.00** PSI	0.66**F_y**	AISC (F3-1)
t_w	=	**0.375** inches		
d	=	**11.1** inches		
t_f	=	**1.12** inches		
F_y	=	**50,000** PSI		
k	=	**1.00** inches		
b_f	=	**10.3** inches		
W_t	=	**100** lbs/lf		

Reactions:

W_{design}	=	=	**1,325**	plf
V_{max}	=	=	**17,888**	lbs
P_{max}	=	=	**17,888**	lbs
M_{max}	=	=	**120,741**	ft-lbs
D_{max}	=	=	**0.877**	inches

Check Shear:

Fv	=	**V_{max}** / **A_{web}**	=	**4,307**	PSI
$Fv_{Allowable}$	=	0.6**F_y**	=	**30,000**	PSI
				Check - O.K.	
		V_{max}	=	**17,888**	LBS
$Fv_{Allowable}$	=	$0.66F_w t_w(N + 2.5k)$	=	**105,188**	LBS
		ASD Section K1.3		**Check - O.K.**	

Check Bearing-Web Crippling:

Fp	=	**P_{max}** / **A_{web}**	=	**4,297**	PSI
$Fp_{Allowable}$	=	0.6**F_y**	=	**30,000**	PSI
				Check - O.K.	
		V_{max}	=	**17,888**	LBS
$F_{PAllowable}$	=	$0.66F_w t_w(N + 2.5k)$	=	**105,188**	LBS
		ASD Section K1.3		**Check - O.K.**	

Check Bending:

$S_{Req'd}$	=	**M_{max}**(12) / **Fb**	=	**43.9**	inches^3
$S_{Furnished}$	=			**112**	inches^3
				Check - O.K.	

Check Deflection:

$D_{max.}$	=	**D_{max}**	=	**0.877**	inches
$D_{Allowable}$	=	L/350	=	**0.926**	inches
				Check - O.K.	

Steel Floor Beam Design

Max Span - First Floor Level

Assumptions:

1. W16 x 89 Beam		**33.00**	Feet =	396.00	Inches
	Supports are:	**3.00**	inches from each end of the beam		
	Therefore:	**L**	=	390.00	Inches
2. Beam designed as a simple span.					
	Therefore:	**L_d**	**=**	390	Inches
3. Bearing area:	(Assume: 6"x6" Plate)	**N**	=	**6**	Inches

W16 x 89 Beam Properties:

A	=	**26.20** inches^2		
A_{web}	=	**8.79** inches2	(Web area from top of beam to bottom of beam x web width.)	
E	=	**29,000,000** PSI		
I	=	**1300** inches^4		
S	=	**155** inches^3		
$Fv_{Allowable}$	=	**9,500** PSI		SDOT 6-02.3(17)B For Salvaged Steel
		30,000 PSI	0.6**F_y**	AISC (Chapter G 16.1-64)
$Fp_{Allowable}$	=	**22,500** PSI		SDOT 6-02.3(17)B For Salvaged Steel
Fb	=	**33,000.00** PSI	0.66**F_y**	AISC (F3-1)
t_w	=	**0.525** inches		
d	=	**16.75** inches		
t_f	=	**0.875** inches		
F_y	=	**50,000** PSI		
k	=	**0.88** inches		
b_f	=	**10.365** inches		
W_t	=	**89** lbs/lf		

Reactions:

W_{design}	=	=	**1,325**	plf
V_{max}	=	=	**21,863**	lbs
P_{max}	=	=	**21,863**	lbs
M_{max}	=	=	**180,366**	ft-lbs
D_{max}	=	=	**0.938**	inches

Check Shear:

Fv	=	**V_{max}** / **A_{web}**	=	**2,496**	PSI
$Fv_{Allowable}$	=	0.6**F_y**	=	**30,000**	PSI
				Check - O.K.	
		V_{max}	=	**21,863**	LBS
$Fv_{Allowable}$	=	$0.66F_w t_w(N + 2.5k)$	=	**141,848**	LBS
		ASD Section K1.3		**Check - O.K.**	

Check Bearing-Web Crippling:

Fp	=	**P_{max}** / **A_{web}**	=	**2,486**	PSI
$Fp_{Allowable}$	=	0.6**F_y**	=	**30,000**	PSI
				Check - O.K.	
		V_{max}	=	**21,863**	LBS
$F_{PAllowable}$	=	$0.66F_w t_w(N + 2.5k)$	=	**141,848**	LBS
		ASD Section K1.3		**Check - O.K.**	

Check Bending:

$S_{Req'd}$	=	**M_{max}**(12) / **Fb**	=	**65.6**	inches^3
$S_{Furnished}$	=			**155**	inches^3
				Check - O.K.	

Check Deflection:

$D_{max.}$	=	**D_{max}**	=	**0.938**	inches
$D_{Allowable}$	=	L/350	=	**1.131**	inches
				Check - O.K.	

ROOF SHEATHING DESIGN:

Assumptions:

1. Plywood shall be: **1/2 inch** APA Rated Sheathing Ext.[3]
2. 2 x _ Trusses/TJI's spaced at: **16** inches o.c.. maximum

Therefore:

L_1	=	16	
L_2	=	14.5	
L_3	=	14.75	

3. Plywood spans over three or more supports.
4. Roof Design Live Load: **LL** = **16** (See UBC Table 16-C)
5. Roof Design Snow Load: **SL** = **25**

1/2 inch Plywood Properties:

		APA Design Values
t_s	=	**0.5** inches
A	=	**2.884** $inches^2$
I	=	**0.075** $inches^3$
KS	=	**0.267** $inches^4$
lb/Q	=	**4.891** $inches^2$
F_b	=	**1930** PSI
F_s	=	**72** PSI
E_e	=	**1,500,000** PSI
E	=	**1,650,000** PSI

Check Plywood Stresses:

A) Check Bending:

$KS_{Req'd}$ = $W(L_1)^2 / 120(F_b)$ = **0.059** $inches^4$

Where **W** = D.L + Roof L.L. + Roof Snow L 53.3 PSF

$KS_{Furnished}$ = **0.267** $inches^4$

Check - O.K.

B) Check Rolling Shear:

$Fs_{Req'd}$ = $W(L_2) / 20(lb/Q)$ = **5.53** PSI

$Fs_{Furnished}$ = $L_2/360$ = **72** PSI

Check - O.K.

C) Check Shear Deflection:

$D_{maximum}$ = $[WC(t_s)^2(L_2)^2] / (1270\ E_e\ I)$ = **0.0008** inches

$D_{Allowable}$ = $L_2/360$ = **0.0403** inches

Check - O.K.

D) Check Bending Deflection:

$D_{maximum}$ = $[W(L_3)^4] / 1743\ E\ I$ = **0.0007** inches

$D_{Allowable}$ = $L_3/360$ = **0.0410** inches

Check - O.K.

ROOF SPAN TABLE

Maximum Horizontal Clear Spans—Roof

O.C. Spacing	Depth	TJI®	Design Live Load (LL) and Dead Load (DL) in PSF											
			Non-Snow (125%)				Snow Load Area (115%)							
			20LL + 15DL		20LL + 20DL		25LL + 15DL		30LL + 15DL		40LL + 15DL		50LL + 15DL	
			Low	High	Low	High	Low	High	Low	High	Low	High	Low	High
16"	9½"	110	20'-0"	17'-10"	19'-1"	16'-11"	19'-2"	17'-2"	18'-5"	16'-7"	17'-2"	15'-7"	15'-11"	14'-9"
		210	21'-2"	18'-10"	20'-2"	17'-10"	20'-3"	18'-2"	19'-6"	17'-6"	18'-2"	16'-6"	17'-2"	15'-7"
		230	21'-11"	19'-6"	20'-10"	18'-6"	20'-11"	18'-9"	20'-2"	18'-1"	18'-10"	17'-0"	17'-9"	16'-2"
	11⅞"	110	23'-11"	21'-4"	22'-9"	20'-2"	22'-8"	20'-6"	21'-5"	19'-10"	19'-5"	18'-7"	17'-11"	17'-4"
		210	25'-3"	22'-6"	24'-1"	21'-4"	24'-2"	21'-8"	23'-3"	20'-11"	21'-4"	19'-8"	19'-8"	18'-8"
		230	26'-1"	23'-3"	24'-10"	22'-0"	24'-11"	22'-4"	24'-0"	21'-7"	22'-5"	20'-4"	20'-9"	19'-3"
		360	27'-9"	24'-9"	26'-5"	23'-5"	26'-7"	23'-10"	25'-6"	23'- 0"	23'-11"	21'-7"	22'-7"	20'-6"
		560	31'-11"	28'-6"	30'-5"	27'- 0"	30'-7"	27'-5"	29'-5"	26'-5"	27'-6"	24'-10"	26'- 0"	23'-7"
	14"	110	27'-2"	24'-3"	25'-7"	23'-0"	24'-9"	23'-4"	23'-4"	22'-4"	21'-2"	20'-5"	19'-6"	18'-11"
		210	28'-9"	25'-7"	27'-4"	24'-3"	27'-1"	24'-8"	25'-7"	23'-9"	23'-3"	22'-4"	21'-5"	20'-9"
		230	29'-8"	26'-6"	28'-3"	25'-1"	28'-5"	25'-5"	27'-0"	24'-7"	24'-6"	23'-1"	22'-7"	21'-10"
		360	31'-6"	28'-2"	30'-0"	26'-8"	30'-2"	27'-1"	29'-0"	26'-1"	27'-2"	24'-7"	25'-8"	23'-4"
		560	36'-3"	32'-4"	34'-6"	30'-7"	34'-8"	31'-1"	33'-4"	30'-0"	31'-2"	28'-3"	29'-6"	26'-9"
	16"	110	29'-5"	26'-11"	27'-5"	25'-6"	26'-5"	25'-2"	25'-0"	23'-10"	22'-8"	21'-10"	20'-5"	20'-3"
		210	31'-10"	28'-5"	30'-0"	26'-11"	29'-0"	27'-4"	27'-5"	26'-2"	24'-10"	23'-11"	22'-8"	22'-2"
		230	32'-10"	29'-4"	31'-4"	27'-9"	30'-7"	28'-2"	28'-11"	27'-3"	26'-2"	25'-3"	24'-2"	23'-5"
		360	34'-11"	31'-2"	33'-3"	29'-6"	33'-5"	30'- 0"	32'-2"	28'-11"	30'-1"	27'-2"	26'- 0"	25'-10"
		560	40'-1"	35'-9"	38'-2"	33'-11"	38'-4"	34'-5"	36'-11"	33'-2"	34'-6"	31'-3"	31'-8"	29'-8"
19.2"	9½"	110	18'-9"	16'-9"	17'-11"	15'-10"	18'-0"	16'-1"	17'-3"	15'-7"	15'-9"	14'-7"	14'-6"	13'-10"
		210	19'-10"	17'-9"	18'-11"	16'-9"	19'-0"	17'-0"	18'-3"	16'-5"	17'-1"	15'-5"	15'-11"	14'-8"
		230	20'-7"	18'-4"	19'-7"	17'-4"	19'-8"	17'-7"	18'-11"	17'-0"	17'-8"	16'-0"	16'-8"	15'-2"
	11⅞"	110	22'-5"	20'-0"	21'-5"	19'-0"	20'-9"	19'-3"	19'-7"	18'-7"	17'-9"	17'-1"	16'-4"	15'-10"
		210	23'-9"	21'-2"	22'-7"	20'-0"	22'-8"	20'-4"	21'-5"	19'-8"	19'-6"	18'-6"	17'-11"	17'-4"
		230	24'-6"	21'-10"	23'-4"	20'-8"	23'-5"	21'-0"	22'-6"	20'-3"	20'-6"	19'-1"	18'-11"	18'-1"
		360	26'-1"	23'-3"	24'-10"	22'-0"	24'-11"	22'-4"	24'- 0"	21'-7"	22'-5"	20'-3"	21'-2"	19'-3"
		560	30'- 0"	26'-9"	28'-7"	25'-4"	28'-8"	25'-9"	27'-7"	24'-10"	25'-9"	23'-4"	24'-4"	22'-2"
	14"	110	25'-1"	22'-10"	23'-4"	21'-7"	22'-7"	21'-5"	21'-4"	20'-4"	19'-4"	18'-7"	17'-0"	17'-3"
		210	27'-0"	24'-1"	25'-7"	22'-10"	24'-9"	23'-2"	23'-4"	22'-4"	21'-2"	20'-5"	18'-10"	18'-11"
		230	27'-10"	24'-10"	26'-6"	23'-7"	26'-1"	23'-11"	24'-7"	23'-1"	22'-4"	21'-6"	20'-7"	19'-11"
		360	29'-7"	26'-5"	28'-2"	25'-0"	28'-4"	25'-5"	27'-3"	24'-6"	25'-6"	23'-1"	21'-7"	21'-8"
		560	34'-0"	30'-4"	32'-5"	28'-9"	32'-7"	29'-2"	31'-4"	28'-2"	29'-3"	26'-6"	26'-5"	25'-2"
	16"	110	26'-10"	25'-4"	25'-0"	23'-5"	24'-2"	22'-11"	22'-10"	21'-9"	20'-1"	19'-11"	17'-0"	18'-3"
		210	29'-5"	26'-8"	27'-5"	25'-4"	26'-5"	25'-2"	25'-0"	23'-11"	22'-3"	21'-10"	18'-10"	20'-2"
		230	30'-11"	27'-7"	28'-11"	26'-1"	27'-11"	26'-6"	26'-4"	25'-2"	23'-11"	23'-0"	21'-2"	21'-3"
		360	32'-10"	29'-3"	31'-3"	27'-9"	31'-5"	28'-2"	30'-2"	27'-2"	25'-7"	25'-3"	21'-7"	21'-8"
		560	37'-8"	33'-7"	35'-10"	31'-10"	36'-0"	32'-4"	34'-8"	31'-2"	31'-3"	29'-4"	26'-5"	25'-5"
24"	9½"	110	17'-5"	15'-6"	16'-7"	14'-8"	16'-5"	14'-11"	15'-6"	14'-5"	14'-1"	13'-6"	13'-0"	12'-7"
		210	18'-5"	16'-5"	17'-6"	15'-6"	17'-7"	15'-9"	16'-11"	15'-3"	15'-5"	14'-4"	14'-3"	13'-7"
		230	19'-0"	17'-0"	18'-1"	16'-1"	18'-2"	16'-4"	17'-6"	15'-9"	16'-3"	14'-10"	15'-0"	14'-0"
	11⅞"	110	20'-7"	18'-7"	19'-2"	17'-7"	18'-6"	17'-7"	17'-6"	16'-8"	15'-10"	15'-3"	13'-7"	14'-2"
		210	21'-11"	19'-7"	20'-11"	18'-7"	20'-4"	18'-10"	19'-2"	18'-2"	17'-5"	16'-9"	15'-0"	15'-6"
		230	22'-8"	20'-3"	21'-7"	19'-2"	21'-5"	19'-5"	20'-3"	18'-9"	18'-4"	17'-8"	16'-11"	16'-4"
		360	24'-1"	21'-6"	23'- 0"	20'-5"	23'-1"	20'-8"	22'-2"	20'-0"	20'-5"	18'-9"	17'-3"	17'-4"
		560	27'-9"	24'-9"	26'-5"	23'-6"	26'-7"	23'-10"	25'-6"	23'-0"	23'-10"	21'-7"	21'-1"	20'-3"
	14"	110	22'-5"	21'-1"	20'-10"	19'-6"	20'-2"	19'-2"	19'-0"	18'-2"	16'-0"	16'-7"	13'-7"	14'-7"
		210	24'-7"	22'-4"	22'-11"	21'-1"	22'-1"	21'-0"	20'-10"	19'-11"	17'-10"	18'-3"	15'-0"	16'-1"
		230	25'-9"	23'-0"	24'-1"	21'-10"	23'-4"	22'-2"	22'-0"	21'-0"	20'-0"	19'-3"	16'-11"	17'-0"
		360	27'-5"	24'-6"	26'-1"	23'-2"	26'-3"	23'-6"	25'-0"	22'-8"	20'-5"	20'-2"	17'-3"	17'-4"
		560	31'-6"	28'-1"	30'-0"	26'-8"	30'-2"	27'-0"	29'-0"	26'-1"	24'-11"	23'-7"	21'-1"	20'-3"
	16"	110	24'-0"	22'-8"	22'-4"	20'-11"	21'-7"	20'-6"	19'-8"	19'-6"	16'-0"	16'-11"	13'-7"	14'-7"
		210	26'-3"	24'-9"	24'-6"	22'-11"	23'-8"	22'-6"	21'-9"	21'-4"	17'-10"	18'-9"	15'-0"	16'-1"
		230	27'-9"	25'-6"	25'-10"	24'-2"	24'-11"	23'-8"	23'-7"	22'-6"	20'-0"	19'-9"	16'-11"	17'-0"
		360	30'-4"	27'-1"	28'-11"	25'-8"	28'-2"	26'-1"	25'-0"	24'-1"	20'-5"	20'-2"	17'-3"	17'-4"
		560	34'-10"	31'-2"	33'-2"	29'-6"	33'-4"	29'-11"	30'-6"	28'-3"	24'-11"	23'-7"	21'-1"	20'-3"

How to Use This Table

1. Determine appropriate live and dead load, and the load duration factor.
2. If your slope is 6:12 or less, use the **Low** slope column. If it is between 6:12 and 12:12, use the **High** column.
3. Scan down the column until you find a span that meets or exceeds the span of your application.
4. Select TJI® joist and on-center spacing.

General Notes

- Table is based on:
  - Minimum bearing length of 1¾" end and 3½" intermediate, without web stiffeners.
  - Uniform loads.
  - More restrictive of simple or continuous span.
  - Minimum roof slope of ¼:12.
- Total load values are limited to deflection of L/180 and live load is based on joist deflection of L/240.
- For continuous spans, ratio of short span to long span should be 0.4 or greater to prevent uplift.
- A support beam or wall at the high end is required. Ridge board applications do not provide adequate support.
- For flat roofs or other loading conditions not shown, refer to Weyerhaeuser software.

ROOF HEADER BEAM DESIGN:

Assumptions:

1. 5.5 x 12.0 GLULAM Beam Western Species				**12.00**	feet span
	Therefore:	L_1	=	144	inches
2. Span 1 Supposrted				**5.00**	feet
Span 2 Supported				**19.00**	feet
4. Assume bearing area	= (5.5" x 5.5")			30.25	$inches^2$

5.50" x 12.0" GluLam Girder Properties:

Reference: 2020 APA Glued Laminated Beam Design Tables

Western Species GluLam Beam

ANSI Design Values

A	=	**66.00**	$inches^2$
E	=	**1,800,000**	PSI
S	=	**132**	inches3
I	=	**792**	inches4
$Fv_{Allowable}$	=	**10,560**	LBS
$Fp_{Allowable}$	=	**405**	PSI
Fb	=	**2,400**	PSI
$M_{Allowable}$	=	**26,400**	FT-LBS
W	=	**16.0**	LBS

Reactions:

W	=	Roof Framing DL	10.0	PSF
		Glulam DL	16.0	PLF
		Roof LL (Snow)	25	PSF
		Design Load = DL x 1.2 + LL x 1.6 =	643.20	PLF

Reactions:

V_{max}	=	0.5**WL**	=	**3,859**	lbs
P_{max}	=	0.5**WL**	=	**3,859**	lbs
M_{max}	=	**(WL^2)**/8	=	**11,578**	ft-lbs
D_{max}	=	0.013**WL^4 / EI**	=	**0.21**	inches

Check Shear:

Fv	=	**V_{max}**(1.5) / **A**	=	**3,859**	lbs
$Fv_{Allowable}$	=			**10,560**	lbs
				Check - O.K.	

Check Bearing:

Fp	=	**P_{max}** / **A**	=	**127.58**	PSI
$Fp_{Allowable}$	=			**405**	PSI
				Check - O.K.	

Check Bending:

M_{max}	=		=	**11,578**	FT-LBS
$Mm_{Allowable}$	=			**26,400**	FT-LBS
				Check - O.K.	

Check Deflection:

$D_{max.}$	=			**0.2102**	inches
$D_{Allowable}$	=	**L**/350	=	**0.4114**	inches
				Check - O.K.	

Roof Header Beam - 12' Span

TABLE 1

DOUGLAS-FIR GLUED LAMINATED BEAM SECTION PROPERTIES AND CAPACITIES
F_b = 2,400 PSI, E = 1,800,000 PSI, F_v = 240 PSI

3-1/8-INCH WIDTH

Depth (in.)	6	7-1/2	9	10-1/2	12	13-1/2	15	16-1/2	18	19-1/2	21	22-1/2	24	25-1/2	27
Beam Weight (lb/ft)[1]	4.6	5.7	6.8	8.0	9.1	10.3	11.4	12.5	13.7	14.8	16.0	17.1	18.2	19.4	20.5
A (in.^{2})	18.8	23.4	28.1	32.8	37.5	42.2	46.9	51.6	56.3	60.9	65.6	70.3	75.0	79.7	84.4
S (in.^{3})	19	29	42	57	75	95	117	142	169	198	230	264	300	339	380
I (in.^{4})	56	110	190	301	450	641	879	1170	1519	1931	2412	2966	3600	4318	5126
EI (106 lb-in.^{2})	101	198	342	543	810	1153	1582	2106	2734	3476	4341	5339	6480	7773	9226
Moment Capacity (lb-ft)[2][3]	3750	5859	8438	11484	15000	18984	23438	28359	33750	39609	45938	52734	60000	67734	75938
Shear Capacity (lb)[3]	3000	3750	4500	5250	6000	6750	7500	8250	9000	9750	10500	11250	12000	12750	13500

3-1/2-INCH WIDTH

Depth (in.)	6	7-1/2	9	10-1/2	12	13-1/2	15	16-1/2	18	19-1/2	21	22-1/2	24	25-1/2	27
Beam Weight (lb/ft)[1]	5.1	6.4	7.7	8.9	10.2	11.5	12.8	14.0	15.3	16.6	17.9	19.1	20.4	21.7	23.0
A (in.^{2})	21.0	26.3	31.5	36.8	42.0	47.3	52.5	57.8	63.0	68.3	73.5	78.8	84.0	89.3	94.5
S (in.^{3})	21	33	47	64	84	106	131	159	189	222	257	295	336	379	425
I (in.^{4})	63	123	213	338	504	718	984	1310	1701	2163	2701	3322	4032	4836	5741
EI (106 lb-in.^{2})	113	221	383	608	907	1292	1772	2358	3062	3893	4862	5980	7258	8705	10334
Moment Capacity (lb-ft)[2][3]	4200	6563	9450	12863	16800	21263	26250	31763	37800	44363	51450	59063	67200	75863	85050
Shear Capacity (lb)[3]	3360	4200	5040	5880	6720	7560	8400	9240	10080	10920	11760	12600	13440	14280	15120

5-1/8-INCH WIDTH

Depth (in.)	12	13-1/2	15	16-1/2	18	19-1/2	21	22-1/2	24	25-1/2	27	28-1/2	30	31-1/2	33
Beam Weight (lb/ft)[1]	14.9	16.8	18.7	20.6	22.4	24.3	26.2	28.0	29.9	31.8	33.6	35.5	37.4	39.2	41.1
A (in.^{2})	61.5	69.2	76.9	84.6	92.3	99.9	107.6	115.3	123.0	130.7	138.4	146.1	153.8	161.4	169.1
S (in.^{3})	123	156	192	233	277	325	377	432	492	555	623	694	769	848	930
I (in.^{4})	738	1051	1441	1919	2491	3167	3955	4865	5904	7082	8406	9887	11531	13349	15348
EI (106 lb-in.^{2})	1328	1891	2595	3453	4483	5700	7119	8757	10627	12747	15131	17796	20756	24028	27627
Moment Capacity (lb-ft)[2][3]	24600	31134	38438	46509	55350	64959	75338	86484	98400	111084	124538	138759	153750	169509	186038
Shear Capacity (lb)[3]	9840	11070	12300	13530	14760	15990	17220	18450	19680	20910	22140	23370	24600	25830	27060

5-1/2-INCH WIDTH

Depth (in.)	12	13-1/2	15	16-1/2	18	19-1/2	21	22-1/2	24	25-1/2	27	28-1/2	30	31-1/2	33
Beam Weight (lb/ft)[1]	16.0	18.0	20.1	22.1	24.1	26.1	28.1	30.1	32.1	34.1	36.1	38.1	40.1	42.1	44.1
A (in.^{2})	66.0	74.3	82.5	90.8	99.0	107.3	115.5	123.8	132.0	140.3	148.5	156.8	165.0	173.3	181.5
S (in.^{3})	132	167	206	250	297	349	404	464	528	596	668	745	825	910	998
I (in.^{4})	792	1128	1547	2059	2673	3398	4245	5221	6336	7600	9021	10610	12375	14326	16471
EI (106 lb-in.^{2})	1426	2030	2784	3706	4811	6117	7640	9397	11405	13680	16238	19098	22275	25786	29648
Moment Capacity (lb-ft)[2][3]	26400	33413	41250	49913	59400	69713	80850	92813	105600	119213	133650	148913	165000	181913	199650
Shear Capacity (lb)[3]	10560	11880	13200	14520	15840	17160	18480	19800	21120	22440	23760	25080	26400	27720	29040

6-3/4-INCH WIDTH

Depth (in.)	18	19-1/2	21	22-1/2	24	25-1/2	27	28-1/2	30	31-1/2	33	34-1/2	36	37-1/2	39
Beam Weight (lb/ft)[1]	29.5	32.0	34.5	36.9	39.4	41.8	44.3	46.8	49.2	51.7	54.1	56.6	59.1	61.5	64.0
A (in.^{2})	121.5	131.6	141.8	151.9	162.0	172.1	182.3	192.4	202.5	212.6	222.8	232.9	243.0	253.1	263.3
S (in.^{3})	365	428	496	570	648	732	820	914	1013	1116	1225	1339	1458	1582	1711
I (in.^{4})	3281	4171	5209	6407	7776	9327	11072	13021	15188	17581	20215	23098	26244	29663	33367
EI (106 lb-in.^{2})	5905	7508	9377	11533	13997	16789	19929	23438	27338	31647	36386	41577	47239	53394	60060
Moment Capacity (lb-ft)[2][3]	72900	85556	99225	113906	129600	146306	164025	182756	202500	223256	245025	267806	291600	316406	342225
Shear Capacity (lb)[3]	19440	21060	22680	24300	25920	27540	29160	30780	32400	34020	35640	37260	38880	40500	42120

8-3/4-INCH WIDTH

Depth (in.)	24	25-1/2	27	28-1/2	30	31-1/2	33	34-1/2	36	37-1/2	39	40-1/2	42	43-1/2	45
Beam Weight (lb/ft)[1]	51.0	54.2	57.4	60.6	63.8	67.0	70.2	73.4	76.6	79.8	82.9	86.1	89.3	92.5	95.7
A (in.^{2})	210.0	223.1	236.3	249.4	262.5	275.6	288.8	301.9	315.0	328.1	341.3	354.4	367.5	380.6	393.8
S (in.^{3})	840	948	1063	1185	1313	1447	1588	1736	1890	2051	2218	2392	2573	2760	2953
I (in.^{4})	10080	12091	14352	16880	19688	22791	26204	29942	34020	38452	43253	48439	54023	60020	66445
EI (106 lb-in.^{2})	18144	21763	25834	30383	35438	41023	47167	53896	61236	69214	77856	87190	97241	108036	119602
Moment Capacity (lb-ft)[2][3]	168000	189656	212625	236906	262500	289406	317625	347156	378000	410156	443625	478406	514500	551906	590625
Shear Capacity (lb)[3]	33600	35700	37800	39900	42000	44100	46200	48300	50400	52500	54600	56700	58800	60900	63000

Notes:
(1) Beam weight is based on density of 35 pcf.
(2) Moment capacity must be adjusted for volume effect. The volume factor for various glulam sizes and simple spans, as well as the complete formula, is given in Appendix A.
(3) Moment and shear capacities are based on a normal (10 years) duration of load and should be adjusted for the design duration of load per the applicable building code.

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ROOF HEADER BEAM DESIGN:

Assumptions:

1. 5.5 x 15.0 GLULAM Beam Western Species				**20.00**	feet span
	Therefore:	L_1	=	240	inches
2. Span 1 Supposrted				**5.00**	feet
Span 2 Supported				**19.00**	feet
4. Assume bearing area	= (5.5" x 5.5")			30.25	$inches^2$

5.50" x 15.0" GluLam Girder Properties:

Reference: 2020 APA Glued Laminated Beam Design Tables

Western Species GluLam Beam

ANSI Design Values

A	=	**82.50**	$inches^2$
E	=	**1,800,000**	PSI
S	=	**206**	inches3
I	=	**1,128**	inches4
$Fv_{Allowable}$	=	**13,200**	LBS
$Fp_{Allowable}$	=	**405**	PSI
Fb	=	**2,400**	PSI
$M_{Allowable}$	=	**41,250**	FT-LBS
W	=	**20.1**	LBS

Reactions:

W	=	Roof Framing DL	10.0	PSF
		Glulam DL	20.1	PLF
		Roof LL (Snow)	25	PSF
		Design Load = DL x 1.2 + LL x 1.6 =	648.12	PLF

Reactions:

V_{max}	=	0.5**WL**	=	**6,481**	lbs
P_{max}	=	0.5**WL**	=	**6,481**	lbs
M_{max}	=	**(WL^2)**/8	=	**32,406**	ft-lbs
D_{max}	=	0.013**WL^4** / **EI**	=	**1.15**	inches

Check Shear:

Fv	=	**V_{max}**(1.5) / **A**	=	**6,481**	lbs
$Fv_{Allowable}$	=			**13,200**	lbs
				Check - O.K.	

Check Bearing:

Fp	=	**P_{max}** / **A**	=	**214.25**	PSI
$Fp_{Allowable}$	=			**405**	PSI
				Check - O.K.	

Check Bending:

M_{max}	=		=	**32,406**	FT-LBS
$Mm_{Allowable}$	=			**41,250**	FT-LBS
				Check - O.K.	

Check Deflection:

$D_{max.}$	=			**1.1473**	inches
$D_{Allowable}$	=	**L**/350	=	**0.6857**	inches
				Check - O.K.	

Roof Header Beam - 20' Span

TABLE 1

DOUGLAS-FIR GLUED LAMINATED BEAM SECTION PROPERTIES AND CAPACITIES
F_b = 2,400 PSI, E = 1,800,000 PSI, F_v = 240 PSI

3-1/8-INCH WIDTH															
Depth (in.)	6	7-1/2	9	10-1/2	12	13-1/2	15	16-1/2	18	19-1/2	21	22-1/2	24	25-1/2	27
Beam Weight (lb/ft)(1)	4.6	5.7	6.8	8.0	9.1	10.3	11.4	12.5	13.7	14.8	16.0	17.1	18.2	19.4	20.5
A (in.^{2})	18.8	23.4	28.1	32.8	37.5	42.2	46.9	51.6	56.3	60.9	65.6	70.3	75.0	79.7	84.4
S (in.^{3})	19	29	42	57	75	95	117	142	169	198	230	264	300	339	380
I (in.^{4})	56	110	190	301	450	641	879	1170	1519	1931	2412	2966	3600	4318	5126
EI (106 lb-in.^{2})	101	198	342	543	810	1153	1582	2106	2734	3476	4341	5339	6480	7773	9226
Moment Capacity (lb-ft)(2)(3)	3750	5859	8438	11484	15000	18984	23438	28359	33750	39609	45938	52734	60000	67734	75938
Shear Capacity (lb)(3)	3000	3750	4500	5250	6000	6750	7500	8250	9000	9750	10500	11250	12000	12750	13500

3-1/2-INCH WIDTH															
Depth (in.)	6	7-1/2	9	10-1/2	12	13-1/2	15	16-1/2	18	19-1/2	21	22-1/2	24	25-1/2	27
Beam Weight (lb/ft)(1)	5.1	6.4	7.7	8.9	10.2	11.5	12.8	14.0	15.3	16.6	17.9	19.1	20.4	21.7	23.0
A (in.^{2})	21.0	26.3	31.5	36.8	42.0	47.3	52.5	57.8	63.0	68.3	73.5	78.8	84.0	89.3	94.5
S (in.^{3})	21	33	47	64	84	106	131	159	189	222	257	295	336	379	425
I (in.^{4})	63	123	213	338	504	718	984	1310	1701	2163	2701	3322	4032	4836	5741
EI (106 lb-in.^{2})	113	221	383	608	907	1292	1772	2358	3062	3893	4862	5980	7258	8705	10334
Moment Capacity (lb-ft)(2)(3)	4200	6563	9450	12863	16800	21263	26250	31763	37800	44363	51450	59063	67200	75863	85050
Shear Capacity (lb)(3)	3360	4200	5040	5880	6720	7560	8400	9240	10080	10920	11760	12600	13440	14280	15120

5-1/8-INCH WIDTH															
Depth (in.)	12	13-1/2	15	16-1/2	18	19-1/2	21	22-1/2	24	25-1/2	27	28-1/2	30	31-1/2	33
Beam Weight (lb/ft)(1)	14.9	16.8	18.7	20.6	22.4	24.3	26.2	28.0	29.9	31.8	33.6	35.5	37.4	39.2	41.1
A (in.^{2})	61.5	69.2	76.9	84.6	92.3	99.9	107.6	115.3	123.0	130.7	138.4	146.1	153.8	161.4	169.1
S (in.^{3})	123	156	192	233	277	325	377	432	492	555	623	694	769	848	930
I (in.^{4})	738	1051	1441	1919	2491	3167	3955	4865	5904	7082	8406	9887	11531	13349	15348
EI (106 lb-in.^{2})	1328	1891	2595	3453	4483	5700	7119	8757	10627	12747	15131	17796	20756	24028	27627
Moment Capacity (lb-ft)(2)(3)	24600	31134	38438	46509	55350	64959	75338	86484	98400	111084	124538	138759	153750	169509	186038
Shear Capacity (lb)(3)	9840	11070	12300	13530	14760	15990	17220	18450	19680	20910	22140	23370	24600	25830	27060

5-1/2-INCH WIDTH															
Depth (in.)	12	13-1/2	15	16-1/2	18	19-1/2	21	22-1/2	24	25-1/2	27	28-1/2	30	31-1/2	33
Beam Weight (lb/ft)(1)	16.0	18.0	20.1	22.1	24.1	26.1	28.1	30.1	32.1	34.1	36.1	38.1	40.1	42.1	44.1
A (in.^{2})	66.0	74.3	82.5	90.8	99.0	107.3	115.5	123.8	132.0	140.3	148.5	156.8	165.0	173.3	181.5
S (in.^{3})	132	167	206	250	297	349	404	464	528	596	668	745	825	910	998
I (in.^{4})	792	1128	1547	2059	2673	3398	4245	5221	6336	7600	9021	10610	12375	14326	16471
EI (106 lb-in.^{2})	1426	2030	2784	3706	4811	6117	7640	9397	11405	13680	16238	19098	22275	25786	29648
Moment Capacity (lb-ft)(2)(3)	26400	33413	41250	49913	59400	69713	80850	92813	105600	119213	133650	148913	165000	181913	199650
Shear Capacity (lb)(3)	10560	11880	13200	14520	15840	17160	18480	19800	21120	22440	23760	25080	26400	27720	29040

6-3/4-INCH WIDTH															
Depth (in.)	18	19-1/2	21	22-1/2	24	25-1/2	27	28-1/2	30	31-1/2	33	34-1/2	36	37-1/2	39
Beam Weight (lb/ft)(1)	29.5	32.0	34.5	36.9	39.4	41.8	44.3	46.8	49.2	51.7	54.1	56.6	59.1	61.5	64.0
A (in.^{2})	121.5	131.6	141.8	151.9	162.0	172.1	182.3	192.4	202.5	212.6	222.8	232.9	243.0	253.1	263.3
S (in.^{3})	365	428	496	570	648	732	820	914	1013	1116	1225	1339	1458	1582	1711
I (in.^{4})	3281	4171	5209	6407	7776	9327	11072	13021	15188	17581	20215	23098	26244	29663	33367
EI (106 lb-in.^{2})	5905	7508	9377	11533	13997	16789	19929	23438	27338	31647	36386	41577	47239	53394	60060
Moment Capacity (lb-ft)(2)(3)	72900	85556	99225	113906	129600	146306	164025	182756	202500	223256	245025	267806	291600	316406	342225
Shear Capacity (lb)(3)	19440	21060	22680	24300	25920	27540	29160	30780	32400	34020	35640	37260	38880	40500	42120

8-3/4-INCH WIDTH															
Depth (in.)	24	25-1/2	27	28-1/2	30	31-1/2	33	34-1/2	36	37-1/2	39	40-1/2	42	43-1/2	45
Beam Weight (lb/ft)(1)	51.0	54.2	57.4	60.6	63.8	67.0	70.2	73.4	76.6	79.8	82.9	86.1	89.3	92.5	95.7
A (in.^{2})	210.0	223.1	236.3	249.4	262.5	275.6	288.8	301.9	315.0	328.1	341.3	354.4	367.5	380.6	393.8
S (in.^{3})	840	948	1063	1185	1313	1447	1588	1736	1890	2051	2218	2392	2573	2760	2953
I (in.^{4})	10080	12091	14352	16880	19688	22791	26204	29942	34020	38452	43253	48439	54023	60020	66445
EI (106 lb-in.^{2})	18144	21763	25834	30383	35438	41023	47167	53896	61236	69214	77856	87190	97241	108036	119602
Moment Capacity (lb-ft)(2)(3)	168000	189656	212625	236906	262500	289406	317625	347156	378000	410156	443625	478406	514500	551906	590625
Shear Capacity (lb)(3)	33600	35700	37800	39900	42000	44100	46200	48300	50400	52500	54600	56700	58800	60900	63000

Notes:
(1) Beam weight is based on density of 35 pcf.
(2) Moment capacity must be adjusted for volume effect. The volume factor for various glulam sizes and simple spans, as well as the complete formula, is given in Appendix A.
(3) Moment and shear capacities are based on a normal (10 years) duration of load and should be adjusted for the design duration of load per the applicable building code.

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ROOF HEADER BEAM DESIGN:

Assumptions:

1. 6" x 12" Timber Header H.F. No. 2 or Better				**12.00**	feet span
	Therefore:	L_1	=	144	inches
2. Span 1 Supposrted				**17.00**	feet
Span 2 Supported				**1.00**	feet
3. Minimum Bearing Area: (5.5" x 5.5")				30.25	inches

6" x 12" Timber Beam Properties:

		ANSI Design Valu	Hem-Fir (North)
A	=	**39.38**	$inches^2$
E	=	**1,700,000**	PSI
I	=	**415**	$inches^4$
S	=	**74**	$inches^3$
$Fv_{Allowable}$	=	**145**	PSI
$Fp_{Allowable}$	=	**405**	PSI
Fb	=	**1,300**	PSI

Reactions:

W	=	Roof Framing DL	=	10.0	PSF
		Header DL	=	16.0	PLF
		Roof LL (Snow)	=	25	PSF
		Design Load = DL x 1.2 + LL x 1.6 =		487.20	PSF

V_{max}	=	0.5**WL**	=	**2,923**	lbs
P_{max}	=	**0.5WL**	=	**2,923**	lbs
M_{max}	=	$(WL^2)/8$	=	**8,770**	ft-lbs
D_{max}	=	0.013**WL**^{4} / **EI**	=	**0.15**	inches

Check Shear:

Fv	=	$V_{max}(1.5)$ / **A**	=	**111.36**	PSI
$Fv_{Allowable}$	=			**145**	PSI
				Check - O.K.	

Check Bearing:

Fp	=	P_{max} / **A**	=	**97**	PSI
$Fp_{Allowable}$	=			405	PSI
				Check - O.K.	

Check Bending:

$S_{Req'd}$	=	$M_{max}(12)$ / **Fb**	=	**81**	$inches^3$
$S_{Furnished}$	=			**74**	$inches^3$
				Check - O.K.	

Check Deflection:

$D_{max.}$	=			**0.1507**	inches
$D_{Allowable}$	=	**L**/350	=	**0.4114**	inches
				Check - O.K.	

ROOF HEADER BEAM DESIGN:

Assumptions:

1. 6" x 8" Timber Header H.F. No. 2 or Better				**7.00**	feet span
	Therefore:	L_1	=	84	inches
2. Span 1 Supposrted				**17.00**	feet
Span 2 Supported				**1.00**	feet
3. Minimum Bearing Area: (5.5" x 5.5")				30.25	inches

6" x 8" Timber Beam Properties:

		ANSI Design Valu	Hem-Fir (North)
A	=	**25.38**	$inches^2$
E	=	**1,700,000**	PSI
I	=	**111**	$inches^4$
S	=	**31**	$inches^3$
$Fv_{Allowable}$	=	**145**	PSI
$Fp_{Allowable}$	=	**405**	PSI
Fb	=	**1,300**	PSI

Reactions:

W	=	Roof Framing DL	=	10.0	PSF
		Header DL	=	16.0	PLF
		Roof LL (Snow)	=	25	PSF
		Design Load = DL x 1.2 + LL x 1.6 =		487.20	PSF

V_{max}	=	0.5**WL**	=	**1,705**	lbs
P_{max}	=	**0.5WL**	=	**1,705**	lbs
M_{max}	=	$(WL^2)/8$	=	**2,984**	ft-lbs
D_{max}	=	$0.013WL^4 / EI$	=	**0.04**	inches

Check Shear:

Fv	=	$V_{max}(1.5) / A$	=	**101**	PSI
$Fv_{Allowable}$	=			**145**	PSI
				Check - O.K.	

Check Bearing:

Fp	=	P_{max} / A	=	**56**	PSI
$Fp_{Allowable}$	=			405	PSI
				Check - O.K.	

Check Bending:

$S_{Req'd}$	=	$M_{max}(12) / Fb$	=	**28**	$inches^3$
$S_{Furnished}$	=			**31**	$inches^3$
				Check - O.K.	

Check Deflection:

$D_{max.}$	=			**0.0420**	inches
$D_{Allowable}$	=	**L**/350	=	**0.2400**	inches
				Check - O.K.	

ROOF HEADER BEAM DESIGN:

Assumptions:

1. 6" x 10" Timber Header H.F. No. 2 or Better				**9.00**	feet span
	Therefore:	L_1	=	108	inches
2. Span 1 Supposrted				**17.00**	feet
Span 2 Supported				**1.00**	feet
3. Minimum Bearing Area: (5.5" x 5.5")				30.25	inches

6" x 10" Timber Beam Properties:

		ANSI Design Valu	Hem-Fir (North)
A	=	**32.38**	$inches^2$
E	=	**1,700,000**	PSI
I	=	**231**	$inches^4$
S	=	**50**	$inches^3$
$Fv_{Allowable}$	=	**145**	PSI
$Fp_{Allowable}$	=	**405**	PSI
Fb	=	**1,300**	PSI

Reactions:

W	=	Roof Framing DL	=	10.0	PSF
		Header DL	=	16.0	PLF
		Roof LL (Snow)	=	25	PSF
		Design Load = DL x 1.2 + LL x 1.6 =		487.20	PSF

V_{max}	=	0.5**WL**	=	**2,192**	lbs
P_{max}	=	**0.5WL**	=	**2,192**	lbs
M_{max}	=	$(WL^2)/8$	=	**4,933**	ft-lbs
D_{max}	=	$0.013WL^4$ / **EI**	=	**0.07**	inches

Check Shear:

Fv	=	$V_{max}(1.5)$ / **A**	=	**102**	PSI
$Fv_{Allowable}$	=			**145**	PSI
				Check - O.K.	

Check Bearing:

Fp	=	P_{max} / **A**	=	**72**	PSI
$Fp_{Allowable}$	=			405	PSI
				Check - O.K.	

Check Bending:

$S_{Req'd}$	=	$M_{max}(12)$ / **Fb**	=	**46**	$inches^3$
$S_{Furnished}$	=			**50**	$inches^3$
				Check - O.K.	

Check Deflection:

$D_{max.}$	=			**0.0705**	inches
$D_{Allowable}$	=	**L**/350	=	**0.3086**	inches

Roof Header Beam - 12' Span

TABLE 1

DOUGLAS-FIR GLUED LAMINATED BEAM SECTION PROPERTIES AND CAPACITIES
F_b = 2,400 PSI, E = 1,800,000 PSI, F_v = 240 PSI

3-1/8-INCH WIDTH

Depth (in.)	6	7-1/2	9	10-1/2	12	13-1/2	15	16-1/2	18	19-1/2	21	22-1/2	24	25-1/2	27
Beam Weight (lb/ft)[1]	4.6	5.7	6.8	8.0	9.1	10.3	11.4	12.5	13.7	14.8	16.0	17.1	18.2	19.4	20.5
A (in.^{2})	18.8	23.4	28.1	32.8	37.5	42.2	46.9	51.6	56.3	60.9	65.6	70.3	75.0	79.7	84.4
S (in.^{3})	19	29	42	57	75	95	117	142	169	198	230	264	300	339	380
I (in.^{4})	56	110	190	301	450	641	879	1170	1519	1931	2412	2966	3600	4318	5126
EI (106 lb-in.^{2})	101	198	342	543	810	1153	1582	2106	2734	3476	4341	5339	6480	7773	9226
Moment Capacity (lb-ft)[2][3]	3750	5859	8438	11484	15000	18984	23438	28359	33750	39609	45938	52734	60000	67734	75938
Shear Capacity (lb)[3]	3000	3750	4500	5250	6000	6750	7500	8250	9000	9750	10500	11250	12000	12750	13500

3-1/2-INCH WIDTH

Depth (in.)	6	7-1/2	9	10-1/2	12	13-1/2	15	16-1/2	18	19-1/2	21	22-1/2	24	25-1/2	27
Beam Weight (lb/ft)[1]	5.1	6.4	7.7	8.9	10.2	11.5	12.8	14.0	15.3	16.6	17.9	19.1	20.4	21.7	23.0
A (in.^{2})	21.0	26.3	31.5	36.8	42.0	47.3	52.5	57.8	63.0	68.3	73.5	78.8	84.0	89.3	94.5
S (in.^{3})	21	33	47	64	84	106	131	159	189	222	257	295	336	379	425
I (in.^{4})	63	123	213	338	504	718	984	1310	1701	2163	2701	3322	4032	4836	5741
EI (106 lb-in.^{2})	113	221	383	608	907	1292	1772	2358	3062	3893	4862	5980	7258	8705	10334
Moment Capacity (lb-ft)[2][3]	4200	6563	9450	12863	16800	21263	26250	31763	37800	44363	51450	59063	67200	75863	85050
Shear Capacity (lb)[3]	3360	4200	5040	5880	6720	7560	8400	9240	10080	10920	11760	12600	13440	14280	15120

5-1/8-INCH WIDTH

Depth (in.)	12	13-1/2	15	16-1/2	18	19-1/2	21	22-1/2	24	25-1/2	27	28-1/2	30	31-1/2	33
Beam Weight (lb/ft)[1]	14.9	16.8	18.7	20.6	22.4	24.3	26.2	28.0	29.9	31.8	33.6	35.5	37.4	39.2	41.1
A (in.^{2})	61.5	69.2	76.9	84.6	92.3	99.9	107.6	115.3	123.0	130.7	138.4	146.1	153.8	161.4	169.1
S (in.^{3})	123	156	192	233	277	325	377	432	492	555	623	694	769	848	930
I (in.^{4})	738	1051	1441	1919	2491	3167	3955	4865	5904	7082	8406	9887	11531	13349	15348
EI (106 lb-in.^{2})	1328	1891	2595	3453	4483	5700	7119	8757	10627	12747	15131	17796	20756	24028	27627
Moment Capacity (lb-ft)[2][3]	24600	31134	38438	46509	55350	64959	75338	86484	98400	111084	124538	138759	153750	169509	186038
Shear Capacity (lb)[3]	9840	11070	12300	13530	14760	15990	17220	18450	19680	20910	22140	23370	24600	25830	27060

5-1/2-INCH WIDTH

Depth (in.)	12	13-1/2	15	16-1/2	18	19-1/2	21	22-1/2	24	25-1/2	27	28-1/2	30	31-1/2	33
Beam Weight (lb/ft)[1]	16.0	18.0	20.1	22.1	24.1	26.1	28.1	30.1	32.1	34.1	36.1	38.1	40.1	42.1	44.1
A (in.^{2})	66.0	74.3	82.5	90.8	99.0	107.3	115.5	123.8	132.0	140.3	148.5	156.8	165.0	173.3	181.5
S (in.^{3})	132	167	206	250	297	349	404	464	528	596	668	745	825	910	998
I (in.^{4})	792	1128	1547	2059	2673	3398	4245	5221	6336	7600	9021	10610	12375	14326	16471
EI (106 lb-in.^{2})	1426	2030	2784	3706	4811	6117	7640	9397	11405	13680	16238	19098	22275	25786	29648
Moment Capacity (lb-ft)[2][3]	26400	33413	41250	49913	59400	69713	80850	92813	105600	119213	133650	148913	165000	181913	199650
Shear Capacity (lb)[3]	10560	11880	13200	14520	15840	17160	18480	19800	21120	22440	23760	25080	26400	27720	29040

6-3/4-INCH WIDTH

Depth (in.)	18	19-1/2	21	22-1/2	24	25-1/2	27	28-1/2	30	31-1/2	33	34-1/2	36	37-1/2	39
Beam Weight (lb/ft)[1]	29.5	32.0	34.5	36.9	39.4	41.8	44.3	46.8	49.2	51.7	54.1	56.6	59.1	61.5	64.0
A (in.^{2})	121.5	131.6	141.8	151.9	162.0	172.1	182.3	192.4	202.5	212.6	222.8	232.9	243.0	253.1	263.3
S (in.^{3})	365	428	496	570	648	732	820	914	1013	1116	1225	1339	1458	1582	1711
I (in.^{4})	3281	4171	5209	6407	7776	9327	11072	13021	15188	17581	20215	23098	26244	29663	33367
EI (106 lb-in.^{2})	5905	7508	9377	11533	13997	16789	19929	23438	27338	31647	36386	41577	47239	53394	60060
Moment Capacity (lb-ft)[2][3]	72900	85556	99225	113906	129600	146306	164025	182756	202500	223256	245025	267806	291600	316406	342225
Shear Capacity (lb)[3]	19440	21060	22680	24300	25920	27540	29160	30780	32400	34020	35640	37260	38880	40500	42120

8-3/4-INCH WIDTH

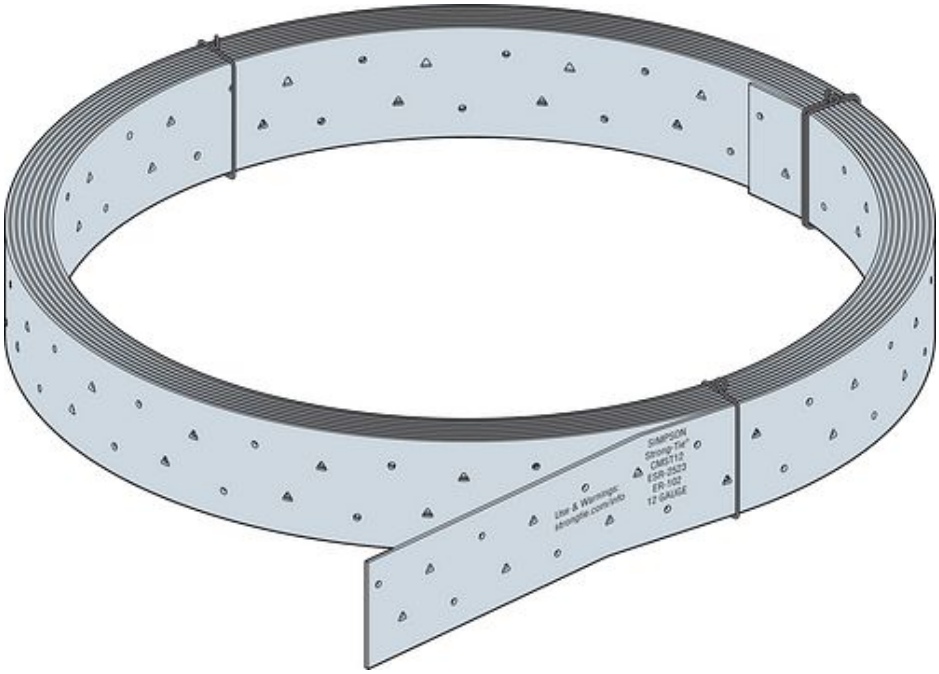
Depth (in.)	24	25-1/2	27	28-1/2	30	31-1/2	33	34-1/2	36	37-1/2	39	40-1/2	42	43-1/2	45
Beam Weight (lb/ft)[1]	51.0	54.2	57.4	60.6	63.8	67.0	70.2	73.4	76.6	79.8	82.9	86.1	89.3	92.5	95.7
A (in.^{2})	210.0	223.1	236.3	249.4	262.5	275.6	288.8	301.9	315.0	328.1	341.3	354.4	367.5	380.6	393.8
S (in.^{3})	840	948	1063	1185	1313	1447	1588	1736	1890	2051	2218	2392	2573	2760	2953
I (in.^{4})	10080	12091	14352	16880	19688	22791	26204	29942	34020	38452	43253	48439	54023	60020	66445
EI (106 lb-in.^{2})	18144	21763	25834	30383	35438	41023	47167	53896	61236	69214	77856	87190	97241	108036	119602
Moment Capacity (lb-ft)[2][3]	168000	189656	212625	236906	262500	289406	317625	347156	378000	410156	443625	478406	514500	551906	590625
Shear Capacity (lb)[3]	33600	35700	37800	39900	42000	44100	46200	48300	50400	52500	54600	56700	58800	60900	63000

Notes:
(1) Beam weight is based on density of 35 pcf.
(2) Moment capacity must be adjusted for volume effect. The volume factor for various glulam sizes and simple spans, as well as the complete formula, is given in Appendix A.
(3) Moment and shear capacities are based on a normal (10 years) duration of load and should be adjusted for the design duration of load per the applicable building code.

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CMST Coiled Medium Strap Tie

Select Media



Model No.	Ga.	Length (ft.)	Coating/Material

Coiled Straps

Load Tables

This product's information may differ depending on the category of use. You are currently viewing details related to **Coiled Straps**. You can also view product information related to the category: Lateral Connectors, Ties and Straps for Cold-Formed Steel Construction

Product Details

The CMST is a continuous utility strap designed to resist tension and uplift loads. This coiled strap can be cut to length on the jobsite and used in a double 2x stud or post application. The CMST is ideal for many applications including resisting uplift in a floor-to-floor application and also resisting tension in a horizontal application.

Key Features

- CMSTC option available, which comes with countersunk nail slots for lower profile when installed with 0.148" x 3 1/4" sinkers
- Used for roof uplift and shearwall overturning resistance
- A cost-effective alternative to pre-cut straps

Material

- See table

Finish

- Galvanized (G90)

Installation

- Use all specified fasteners; see General Notes.
- Wood shrinkage after strap installation across horizontal wood members may cause strap to buckle outward.
- Refer to the applicable code for minimum nail penetration and minimum wood edge and end distances.
- The table shows the maximum allowable loads and the nails required to attain them. Fewer nails may be used; reduce the allowable load as shown in the Straps and Ties General Notes.
- For lap splice and alternative nailing information, see tables below.
- The cut length of the strap shall be equal to twice the “End Length” noted in the table plus the clear span dimension.
- Use only every other round hole if the wood tends to split.
- Fill round and triangle holes for loads shown. If wood tends to split, fill only round hole and double the end length listed for full load.

Lap Splicing

Lap splicing of coiled straps can be used to extend standard strap lengths for designing continuous drag elements and diaphragm chord members. The Strap Lap Splices table

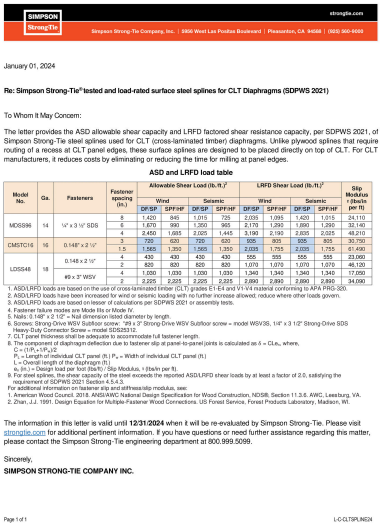
Model No.	Ga.	Length (ft.)	Coating/Material

A manual pocket installation guide for

A technical bulletin providing information on

Load Tables

with a concealed connection using a combination of Simpson Strong-Tie anchors and threaded rods or coil straps.



January 01, 2024

Re: Simpson Strong-Tie® tested and load-rated surface steel splines for CLT Diaphragms (SDPWS 2021)

To Whom It May Concern:

The letter provides the ASD allowable shear capacity and LRFD factored shear resistance capacity, per SDPWS 2021, of Simpson Strong-Tie steel splines used for CLT (cross-laminated timber) diaphragms. Unlike plywood splines that require routing of a recess at CLT panel edges, these surface splines are designed to be placed directly on top of CLT. For CLT manufacturers, it reduces costs by eliminating or reducing the time for milling at panel edges.

ASD and LRFD load table

Model No.	Ga.	Fasteners	Fastener spacing (in.)	Allowable Shear Load (lb./ft.)[2]				LRFD Shear Load (lb./ft.)[2]				Slip Modulus (lb/in per ft)
				Wind		Seismic		Wind		Seismic		
				DF/SP	SPF/HF	DF/SP	SPF/HF	DF/SP	SPF/HF	DF/SP	SPF/HF	
MDSS96	14	¼" x 3 ½" SDS	8	1,420	845	1,015	725	2,035	1,095	1,420	1,015	24,110
			6	1,870	990	1,350	965	2,170	1,295	1,890	1,295	32,140
			4	2,450	1,685	2,025	1,445	3,190	2,190	2,835	2,025	48,210
CMSTC16	16	0.148" x 2 ½"	3	720	630	720	630	935	815	935	815	30,750
			1.5	1,565	1,350	1,565	1,350	2,035	1,755	2,035	1,755	61,490
LDSS48	18	0.148 x 2 ½"	4	430	430	430	430	555	555	555	555	23,060
			2	820	820	820	820	1,070	1,070	1,070	1,070	46,120
		#9 x 3" WSV	4	1,030	1,030	1,030	1,030	1,340	1,340	1,340	1,340	17,050
			2	2,225	2,225	2,225	2,225	2,890	2,890	2,890	2,890	34,090

1. ASD/LRFD loads are based on the use of cross-laminated timber (CLT) grades E1-E4 and V1-V4 material conforming to APA PRG-320.
2. ASD/LRFD loads have been increased for wind or seismic loading with no further increase allowed; reduce where other loads govern.
3. ASD/LRFD loads are based on lesser of calculations per SDPWS 2021 or assembly tests.
4. Fastener failure modes are Mode IIIs or Mode IV.
5. Nails: 0.148" x 2 1/2" = Nail dimension listed diameter by length.
6. Screws: Strong-Drive WSV SubRoof screw: "#9 x 3" Strong-Drive WSV SubRoof screw = model WSV3S, 1/4" x 3 1/2" Strong-Drive SDS Heavy-Duty Connector Screw = model SDS25312.
7. CLT panel thickness shall be adequate to accommodate full fastener length.
8. The component of diaphragm deflection due to fastener slip at panel-to-panel joints is calculated as $\delta_s = CL_e$, where,
   $C = (1/P_x + 1/P_y)/2$
   P_x = Length of individual CLT panel (ft.) P_y = Width of individual CLT panel (ft.)
   L_e = Overall length of the diaphragm (ft.)
   e_n (in.) = Design load per foot (lbs/ft) / Slip Modulus, γ (lbs/in per ft).
9. For steel splines, the applied shear capacity or shear demand exceeds the reported ASD/LRFD shear loads by at least a factor of 2.0, satisfying the requirements of SDPWS 2021 Section 4.5.4.

For additional information on fastener slip and stiffness/slip modulus, see:
1. American Wood Council. 2018. ANSI/AWC National Design Specification for Wood Construction, NDS®, Section 11.3.6. AWC, Leesburg, VA.
2. Zhan, J.J. 1991. Design Equation for Multiple Fastener Wood Connections. US Forest Service, Forest Products Laboratory, Madison, WI.

The information in this letter is valid until **12/31/2024** when it will be re-evaluated by Simpson Strong-Tie. Please visit strongtie.com for additional pertinent information. If you have questions or need further assistance regarding this matter, please contact the Simpson Strong-Tie engineering department at 800.999.5099.

Sincerely,

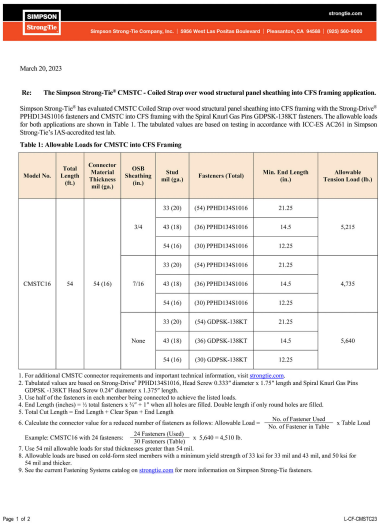
SIMPSON STRONG-TIE COMPANY INC.

Page 1 of 1 L-C-CLTSPLINE24

Simpson Strong-Tie Tested and Load-Rated Surface Steel Splines for CLT Diaphragms (SDPWS 2021)

L-C-CLTSPLINE24 — *Engineering Letter*

This engineering letter provides both LRFD and ASD values for the steel splines of CLT diaphragms satisfying the requirements of SDPWS 2021 Section 4.5.4.



March 20, 2023

Re: The Simpson Strong-Tie® CMSTC - Coiled Strap over wood structural panel sheathing into CFS framing application.

Simpson Strong-Tie® has evaluated CMSTC Coiled Strap over wood structural panel sheathing into CFS framing with the Strong-Drive® PPHD134S1016 fasteners and CMSTC into CFS framing with the Spiral Knurl Gas Pins GDPSK-138KT fasteners. The allowable loads for both applications are shown in Table 1. The tabulated values are based on testing in accordance with ICC-ES AC261 in Simpson Strong-Tie's IAS-accredited test lab.

Table 1: Allowable Loads for CMSTC into CFS Framing

Model No.	Total Length (ft.)	Connector Material Thickness mil (ga.)	OSB Sheathing (in.)	Stud mil (ga.)	Fasteners (Total)	Min. End Length (in.)	Allowable Tension Load (lb.)
CMSTC16	54	54 (16)	3/4	33 (20)	(54) PPHD134S1016	21.25	5,215
				43 (18)	(36) PPHD134S1016	14.5	
				54 (16)	(30) PPHD134S1016	12.25	
			7/16	33 (20)	(54) PPHD134S1016	21.25	4,735
				43 (18)	(36) PPHD134S1016	14.5	
				54 (16)	(30) PPHD134S1016	12.25	
			None	33 (20)	(54) GDPSK-138KT	21.25	5,640
				43 (18)	(36) GDPSK-138KT	14.5	
				54 (16)	(30) GDPSK-138KT	12.25	

1. For additional CMSTC connector requirements and important technical information, visit strongtie.com.
2. Tabulated values are based on Strong-Drive® PPHD134S1016, Head Screw 0.333" diameter x 1.75" length and Spiral Knurl Gas Pins GDPSK-138KT Head Screw 0.34" diameter x 1.375" length.
3. Use half of the fasteners in each member being connected to achieve the listed loads.
4. End Length (inches) = ½ total fasteners x ½" + 1" when all holes are filled. Double length if only round holes are filled.
5. Total Cut Length = End Length + Clear Span + End Length
6. Calculate the connector value for a reduced number of fasteners as follows: Allowable Load = $\frac{\text{No. of Fastener Used}}{\text{No. of Fastener in Table}}$ x Table Load
   Example: CMSTC16 with 24 fasteners: $\frac{24 \text{ Fasteners (Used)}}{30 \text{ Fasteners (Table)}}$ x 5,640 = 4,510 lb.
7. Use 54 mil allowable loads for stud thicknesses greater than 54 mil.
8. Allowable loads are based on cold-form steel members with a minimum yield strength of 33 ksi for 33 mil and 43 mil, and 50 ksi for 54 mil and thicker.
9. See the current Fastening Systems catalog on strongtie.com for more information on Simpson Strong-Tie fasteners.

Page 1 of 2 L-CF-CMSTC23

The Simpson Strong-Tie® CMSTC — Coiled Strap over Wood Structural Panel Sheathing into CFS Framing Application

L-CF-CMSTC23 — *Engineering Letter*

The engineering letter provides capacities of CMSTC coiled strap as installed over wood structural panel sheathing into CFS framing with Strong-Drive® PPSD134S1016 fasteners.

Product Information Table

Model No.	Ga.	Length (ft.)	Coating/Material	Packaging Qty.
CMST12	12	40	Zinc Galvanized, G90	1
CMST14	14	52 1/2	Zinc Galvanized, G90	1
CMSTC16	16	54	Zinc Galvanized, G90	1

Load Tables

Model No.	Total L	Ga.	DF/SP		SPF/HF		Allowable Tension Load (160)
			Fasteners (in.)	End Length (in.)	Fasteners (in.)	End Length (in.)	
CMST12	40'	12	(74) 0.162 x 2½	33	(84) 0.162 x 2½	38	9,215
			(86) 0.148 x 2½	39	(98) 0.148 x 2½	44	9,215
CMST14	52½'	14	(56) 0.162 x 2½	26	(66) 0.162 x 2½	30	6,475
			(66) 0.148 x 2½	30	(76) 0.148 x 2½	34	6,475
CMSTC16	54'	16	(50) 0.148 x 3¼	20	(58) 0.148 x 3¼	25	4,690

1. See Straps and Ties General Notes.
2. Calculate the connector value for a reduced number of nails as follows:

$$\text{Allowable Load} = \frac{\text{No. of Nails Used}}{\text{No. of Nails in Table}} \times \text{Table Load}$$

Example: CMSTC16 in DF/SP with 40 nails total.
(Half of the nails in each member being connected)

$$\text{Allowable Load} = \frac{40 \text{ nails (Used)}}{50 \text{ Nails (Table)}} \times 4{,}690 \text{ lb.} = 3{,}752 \text{ lb.}$$

3. See table below for alternate nailing and lap splice information.
4. Fasteners: Nail dimensions in the table are listed diameter by length. For additional information, see Fastener Types and Sizes Specified for Simpson Strong-Tie Connectors.

Strap Lap Splices

Model No.	Ga.	Strap Lap Splice	
		Minimum Fasteners per Splice	Min. Splice Length, L_{sp} (in.)
CMST12	12	(18) 0.162 x 2½	18
		(22) 0.148 x 2½	21
CMST14	14	(13) 0.162 x 2½	14
		(15) 0.148 x 2½	15
CMSTC16	16	(11) 0.162 x 2½	10
		(11) 0.148 x 2½	10

1. See Straps and Ties General Notes.

SIMPSON StrongTie
Model No. | Ga. | Length (ft.) | Coating/Material
Load Tables

2. 0.148" x 3 1/2" nails can be replaced by 0.148" x 3 1/4" nails. No other nail substitution is
3. Refer to the applicable code for minimum edge distance and minimum end distance.
4. No strap modification is allowed and the splice must meet both the minimum number of fasteners and the minimum splice length.

Allowable Loads for Alternative Nailing

Model No.	Ga.	Total Coil Length (ft.)	Fasteners (in.)	DF/SP Allowable Tension Loads	End Length (in.)	
				(160)	Nail Installed in Every Hole	Nail Installed in Every Other Hole
CMST12	12	40	(66) 0.162 x 2½	8,415	30	58
			(58) 0.162 x 2½	7,395	27	51
			(50) 0.162 x 2½	6,375	23	44
			(76) 0.148 x 2½	8,320	35	66
			(68) 0.148 x 2½	7,445	31	59
			(60) 0.148 x 2½	6,570	28	52
CMST14	14	52.5	(48) 0.162 x 2½	5,615	22	42
			(40) 0.162 x 2½	4,680	19	35
			(32) 0.162 x 2½	3,745	15	28
			(58) 0.148 x 2½	5,770	27	51
			(50) 0.148 x 2½	4,975	23	44
			(42) 0.148 x 2½	4,180	20	37
CMSTC16	16	54	(42) 0.162 x 2½	4,690	17	32
			(34) 0.162 x 2½	3,875	14	26
			(26) 0.162 x 2½	2,965	11	20
			(18) 0.162 x 2½	2,050	8	14
			(48) 0.148 x 2½	4,610	19	35
			(40) 0.148 x 2½	3,840	16	29
			(32) 0.148 x 2½	3,070	13	23
			(24) 0.148 x 2½	2,305	10	17
			(16) 0.148 x 2½	1,535	7	11

1. See Straps and Ties General Notes.

Model No.	Ga.	Length (ft.)	Coating/Material

Fasteners: Nail dimensions in the table are listed diameter x length. For additional information, see Fastener Types and Sizes Specified for Simpson Strong-Tie Connectors.

Load Tables

Code Reports & Compliance

Product Series	Compliance/Certification	Report/Approval
CMST	International Building Code International Residential Code City of Los Angeles Building Code City of Los Angeles Residential Code	ER-124 (CFS)
	International Building Code International Residential Code City of Los Angeles Building Code City of Los Angeles Residential Code	ESR-2105
	Florida Building Code	FL10456
	Florida Building Code	FL13872

Footnotes

1. Please review compliance documents for information about specific product models. In some cases, Compliance documents cover most but not all models associated with a product series.
2. For additional information regarding Florida's Statewide Product Approval System and Miami-Dade County Notice of Acceptance (NOA), click here.

Related Products



CS
Coiled Strap

Model No.	Ga.	Length (ft.)	Coating/Material	

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Product Use & Corrosion Info
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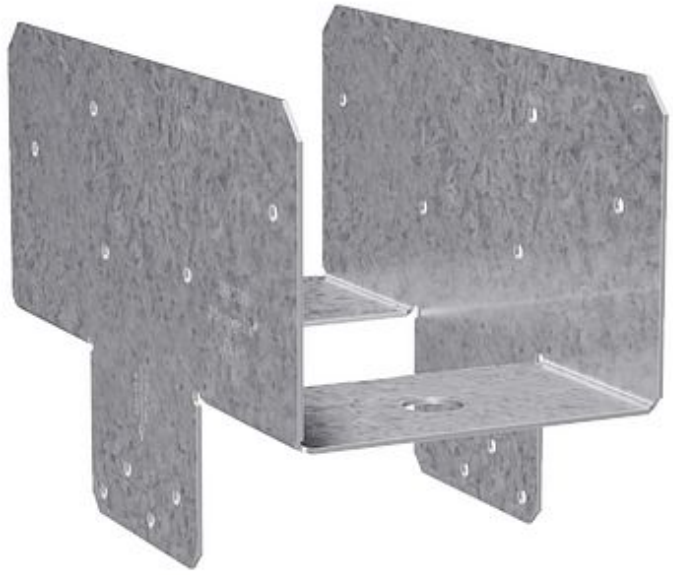
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PCZ™ Post Cap

Select Media



Product Details

Product Information Table

The PCZ is designed as a fast and easy post-to-beam connection. With the post and beam flanges in line with each other, this post cap can accommodate several post sizes. The PCZ features a ZMAX® finish to meet exposure conditions in many environments.

Key Features

- ZMAX finish is standard
- Easy install with 10d (0.148" x 3") common nails or Strong-Drive® SD Connector screws
- In-line flanges accommodate multiple beam sizes

Material

- 16 gauge

Finish

- ZMAX coating; see Corrosion Information

Installation

- Use all specified fasteners; see General Notes
- Center PCZ legs on post
- Do not install bolts into pilot holes

Options

- For end conditions, see EPCZ post caps
- For heavy-duty applications, see CC, ECC, CCQ, and ECCQ
- For retrofit applications, see AC and LCE series

Related Links

- Wood Construction Connectors Technical and Installation Notes
- General Notes
- Corrosion Information

Catalog Pages

- C-C-2024 (Wood Construction Connectors), page 91

SIMPSON Strong-Tie

g-Tie® connectors on SCL columns as well as reduction factors for narrow-face SCL installations.

Product Information Table

Product Information Table

Model No.	Ga.	W (in.)	Coating/Material	Packaging Qty.
PC4RZ	16	4	ZMAX®	1
PC4Z	16	3 9/16	ZMAX®	1
PC6RZ	16	6	ZMAX®	1
PC6Z	16	5 1/2	ZMAX®	1
PC8RZ	16	8	ZMAX®	1
PC8Z	16	7 1/2	ZMAX®	1

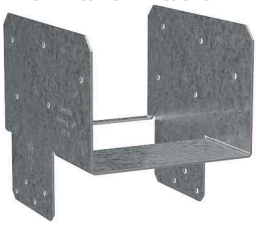
Load Tables

These products are available with additional corrosion protection.

SD Many of these products are approved for installation with Strong-Drive® SD Connector screws.

Model No.	W (in.)	Fasteners (in.)		Post Size	DF/SP Allowable Loads	
		Beam	Post		Uplift (160)	Lateral (160)
PC4Z	3 9/16	(10) 0.148 x 3	(8) 0.148 x 3	(2) 2x4	1,480	1,120
				4x4	1,480	1,260
				4x6	1,480	1,260
				4x8	1,480	1,380
PC6Z	5 1/2	(10) 0.148 x 3	(8) 0.148 x 3	4x6	1,480	1,260
				6x6	1,480	1,295
				6x8	1,480	1,380
PC8Z	7 1/2	(10) 0.148 x 3	(8) 0.148 x 3	4x8	1,480	1,260
				6x8	1,480	1,295
				8x8	1,480	1,380

Product Information Table



EPCZ™
End Post Cap



Post-to-Beam
Selector
PBS



Strong-Drive® SD
CONNECTOR Screw

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Product Use & Corrosion Info

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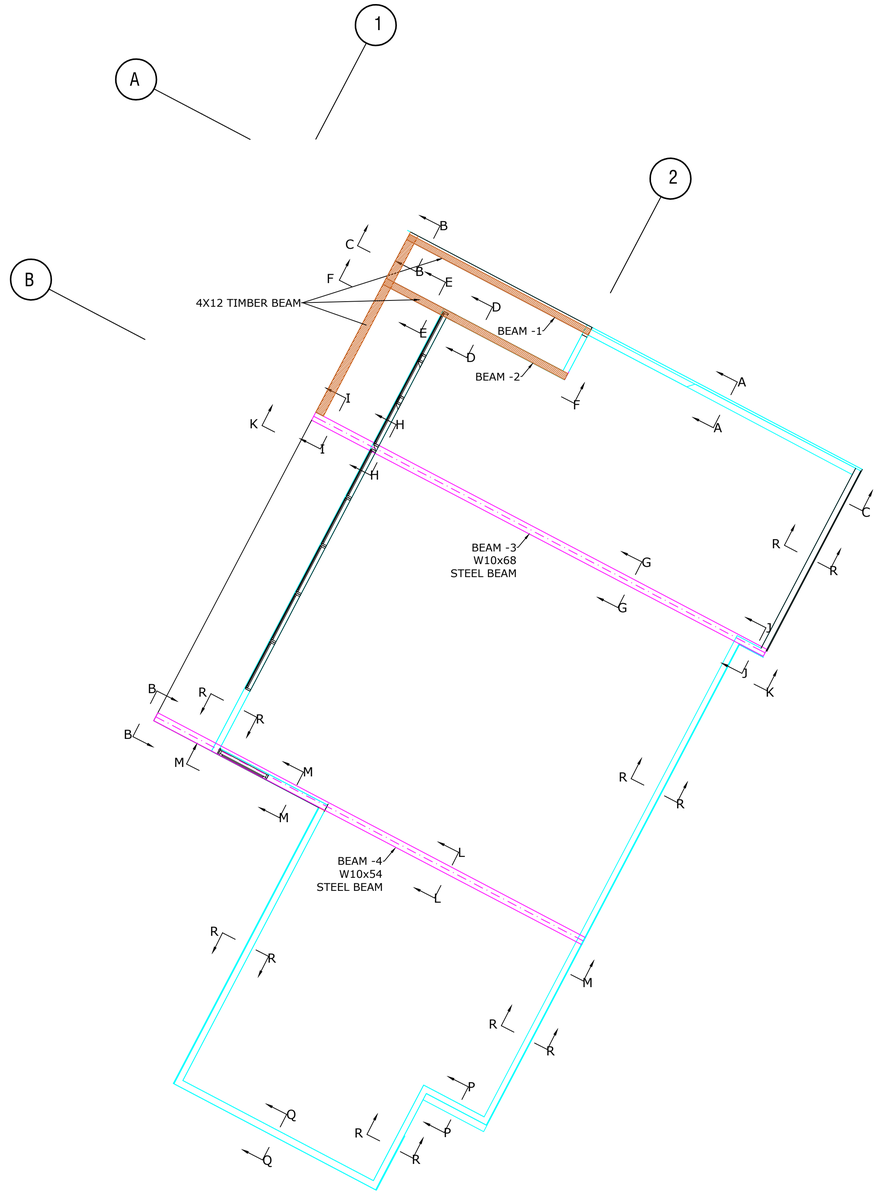
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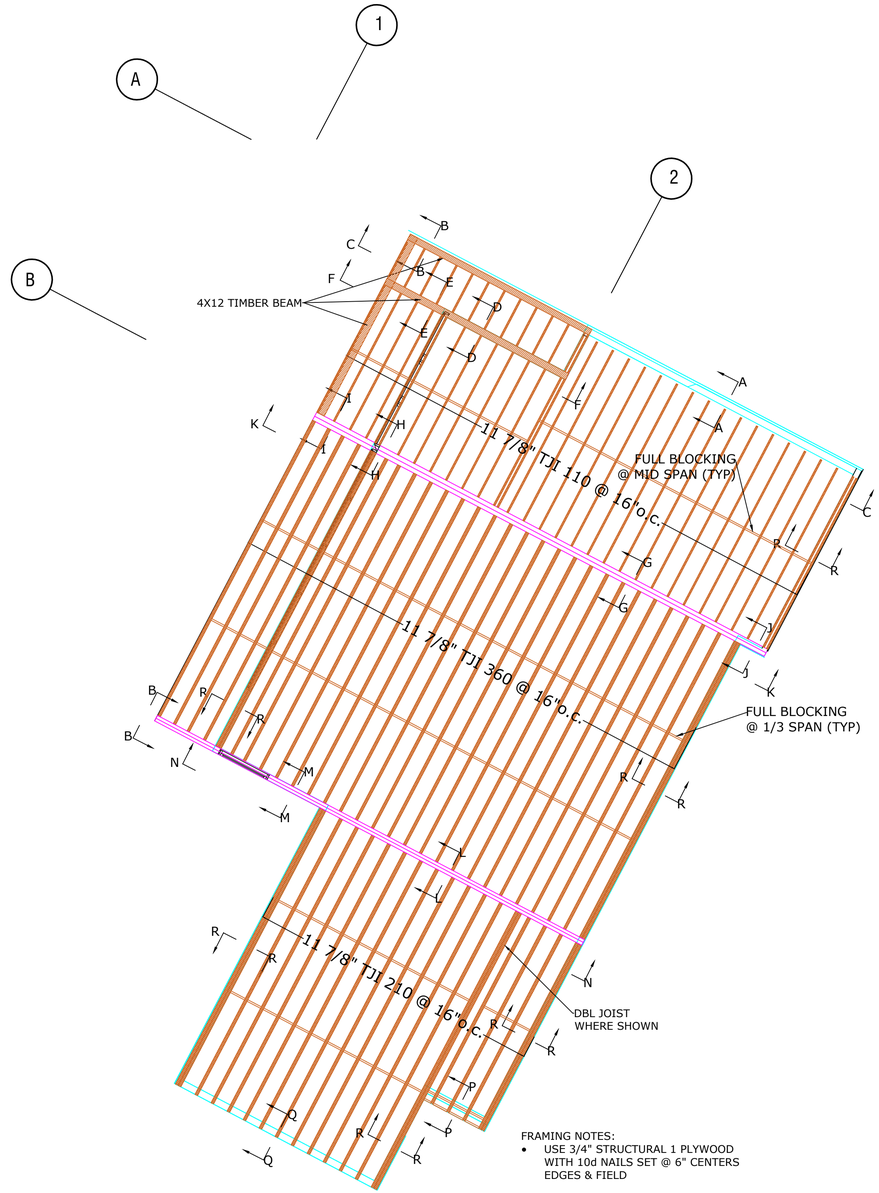
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Product Information Table

PLAN SET





FRAMING NOTES:

- USE 3/4" STRUCTURAL 1 PLYWOOD WITH 10d NAILS SET @ 6" CENTERS EDGES & FIELD

ROOF FRAMING

LANZ RESIDENCE
8020 SE 57th Street
Mercier Island, WA 98040

Roof Framing Details

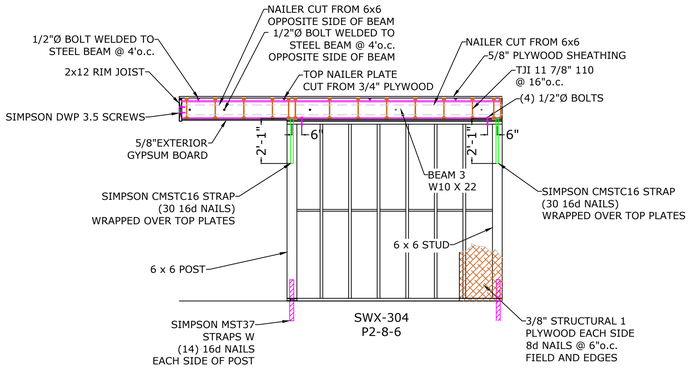
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12527 Huckleberry Lane
Arlington, Washington 98223
PHONE: (206) 790-8039
E-MAIL: joe@luciaeng.com

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STATE OF WASHINGTON
23314
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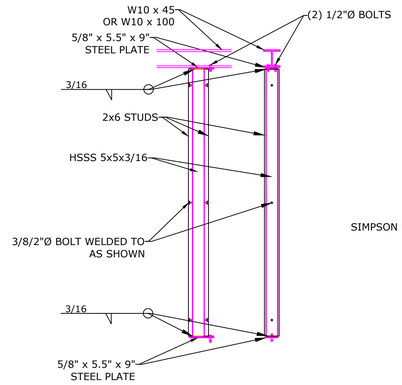
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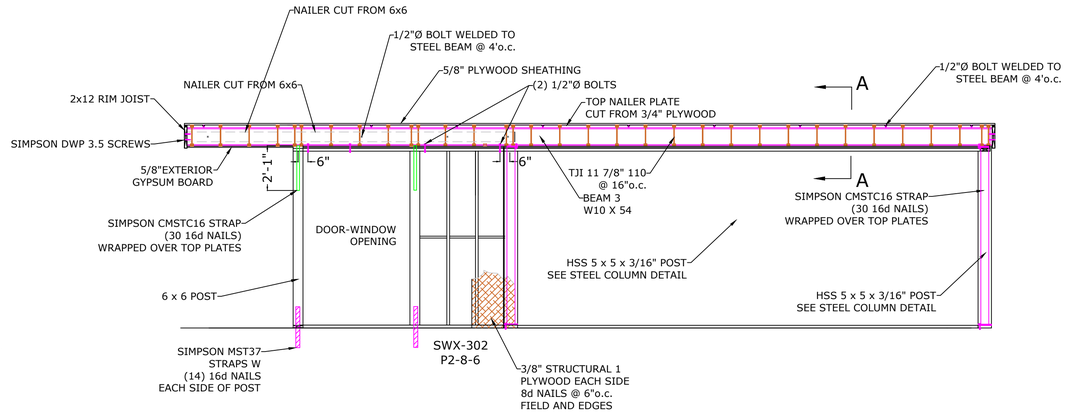
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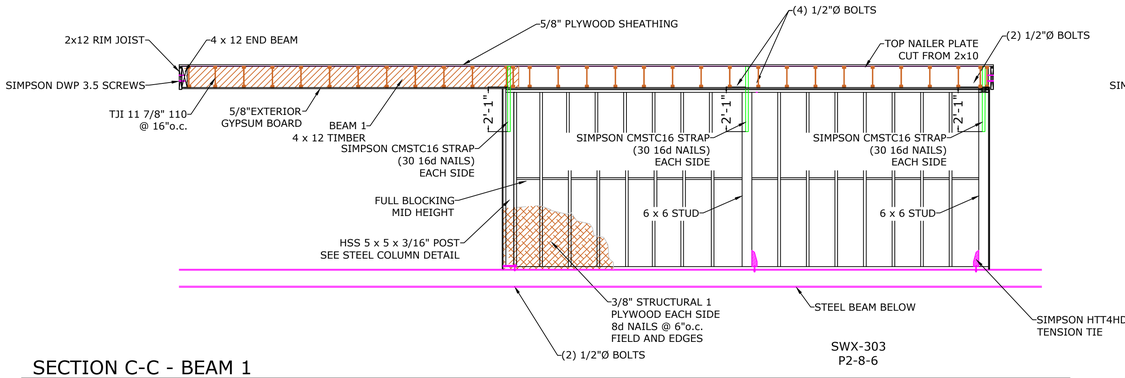
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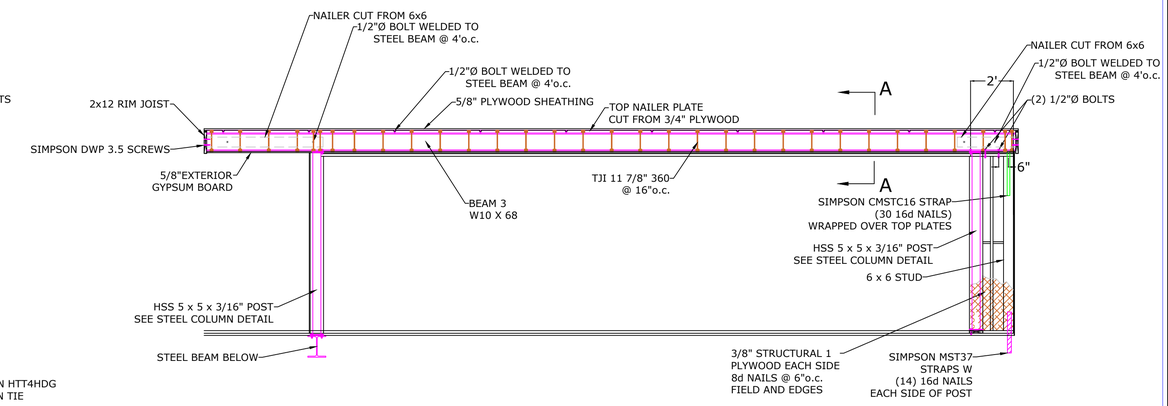
STEEL COLUMN DETAIL



SECTION N-N - BEAM 4



SECTION C-C - BEAM 1



SECTION K-K - BEAM 3

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8020 SE 57th Street
Mercer Island, WA 98040

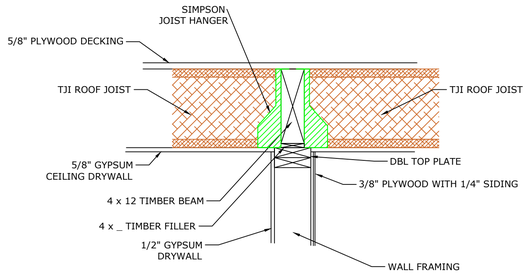
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E-MAIL: joe@luciaeng.com

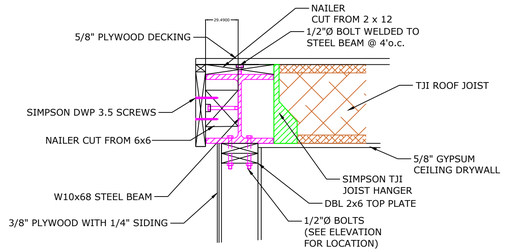
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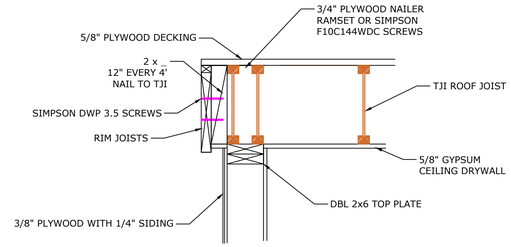
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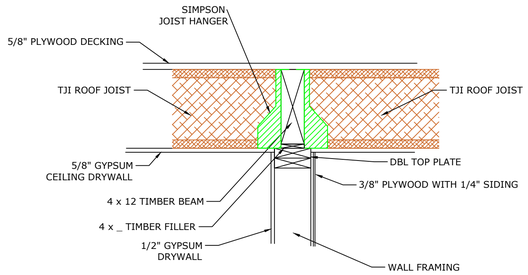
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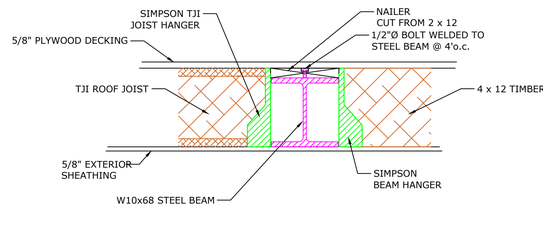
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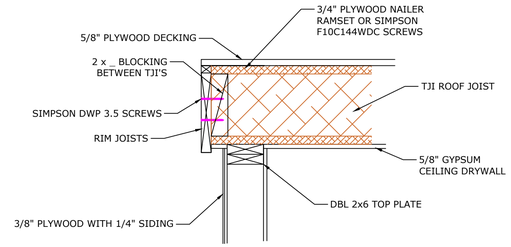
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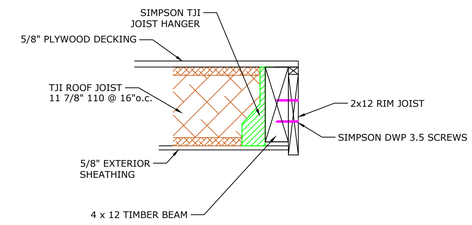
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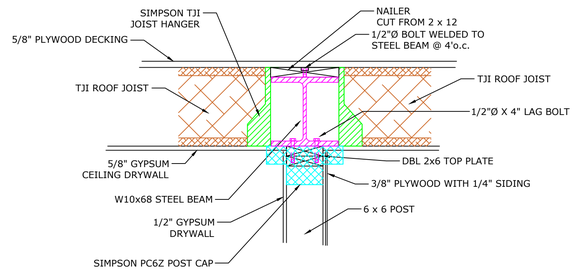
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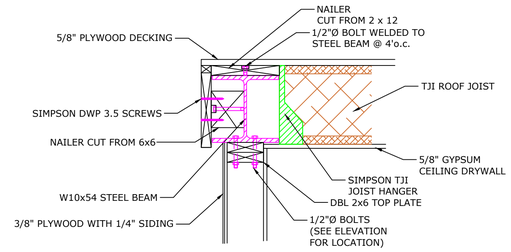
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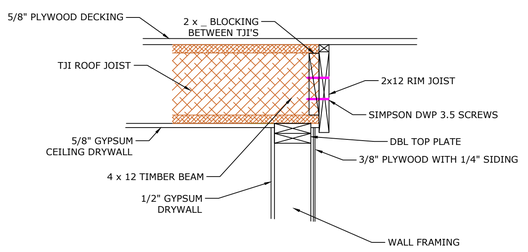
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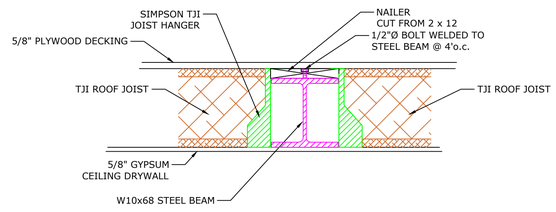
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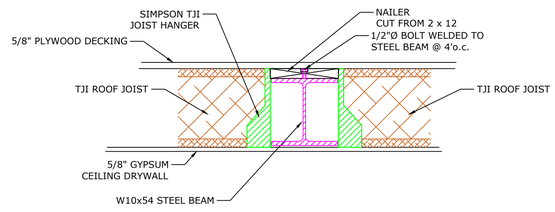
SECTION M-M



SECTION A-A



SECTION G-G



SECTION L-L

LANZ RESIDENCE
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Mercer Island, WA 98040

Framing Details

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23314
REGISTERED
PROFESSIONAL ENGINEER
08-15-24

Number	Date	By	Description
5	08-15-24	JML	

SHEET
S-10.2



SECOND FLOOR - FLOOR FRAMING

LANZ RESIDENCE
8020 SE 57th Street
Mercer Island, WA 98040

Second Level
Framing Details

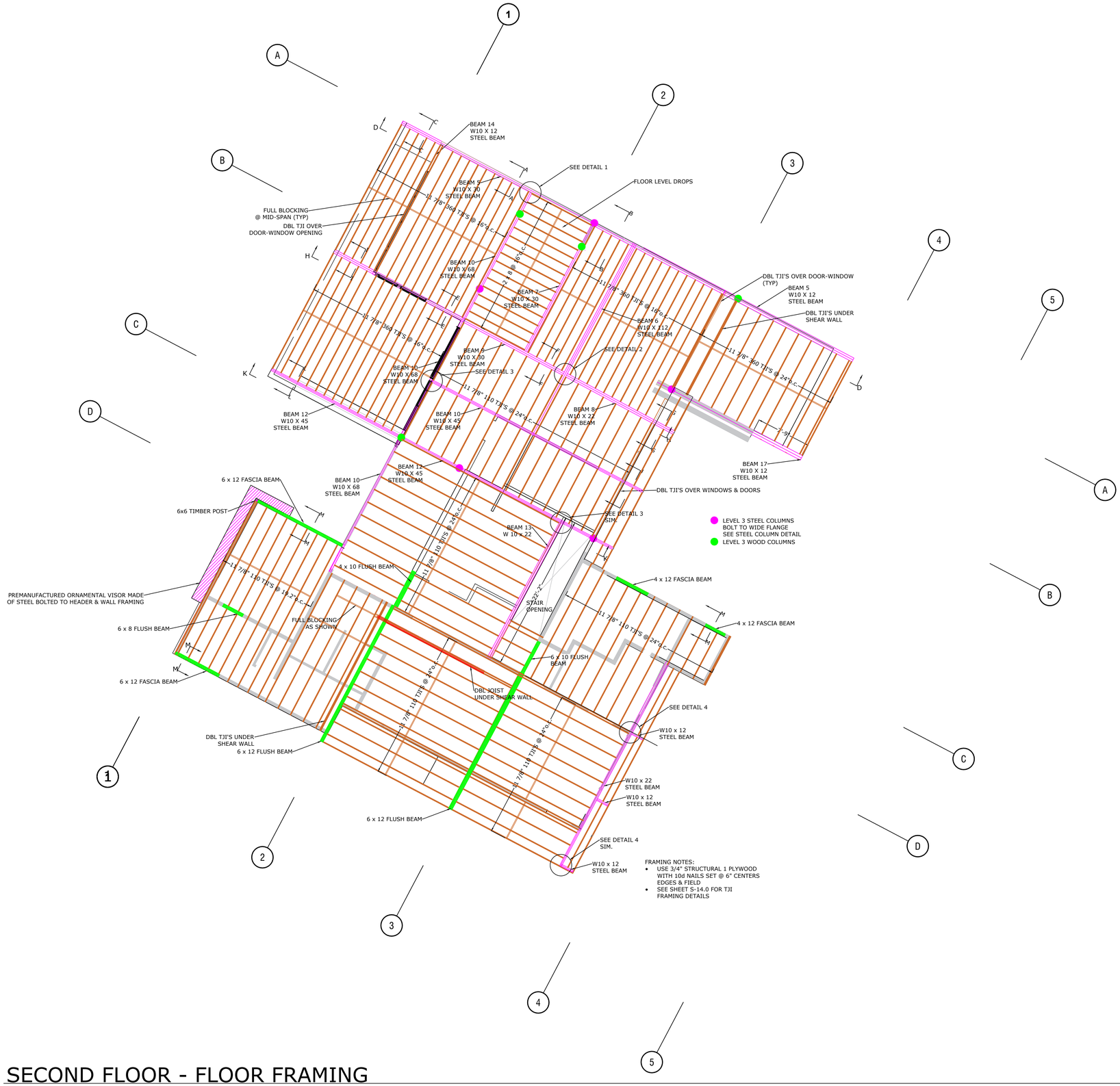
LUCIA ENGINEERING, INC.
12527 Huckleberry Lane
Arlington, Washington 98223
PHONE: (206) 790-8039
E-MAIL: joe@luciaeng.com

JOSEPH M. LUCIA
STATE OF WASHINGTON
23314
REGISTERED
PROFESSIONAL ENGINEER

08-15-24

Number	Date	By	Description
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SHEET
S-11.0



SECOND FLOOR - FLOOR FRAMING

LANZ RESIDENCE
8020 SE 57th Street
Mercer Island, WA 98040

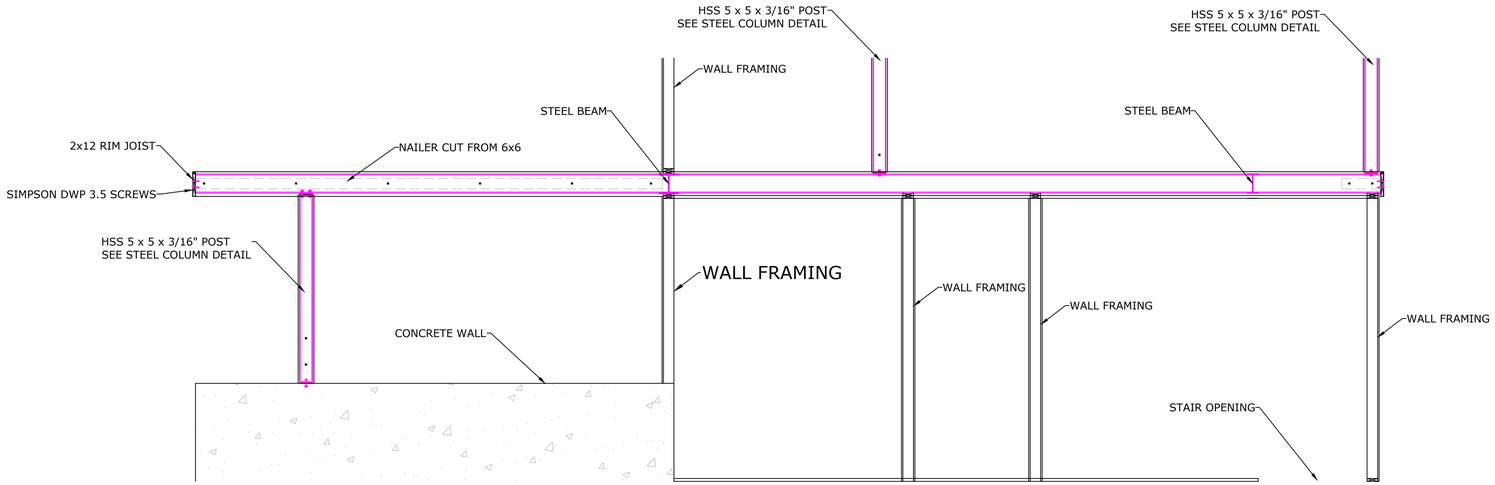
Second Level
Floor Framing

LUCIA ENGINEERING, INC.
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Arlington, Washington 98223
PHONE: (206) 790-8039
E-MAIL: joe@luciaeng.com

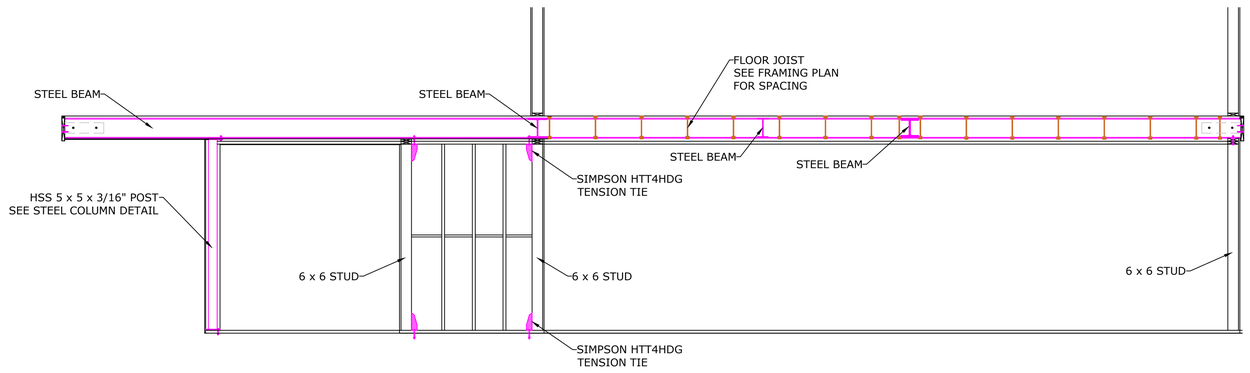
JOSEPH M. LUCIA
STATE OF WASHINGTON
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REGISTERED
PROFESSIONAL ENGINEER
08-15-24

Number	Date	By	Description
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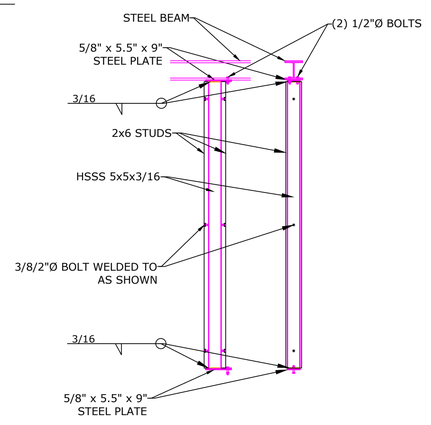
SHEET
S-11.1



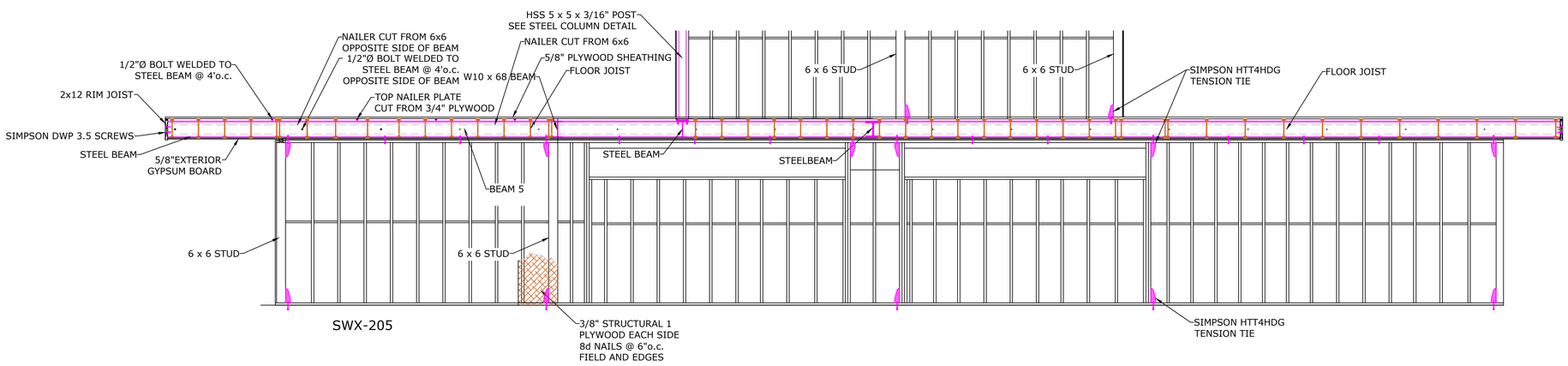
SECTION K-K - BEAM 12



SECTION H-H - BEAM 9



STEEL COLUMN DETAIL



SECTION F-F - BEAM 5

LANZ RESIDENCE
8020 SE 57th Street
Mercer Island, WA 98040

Second Level
Sections

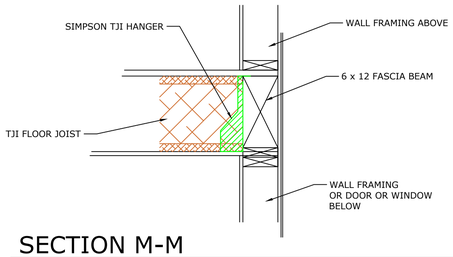
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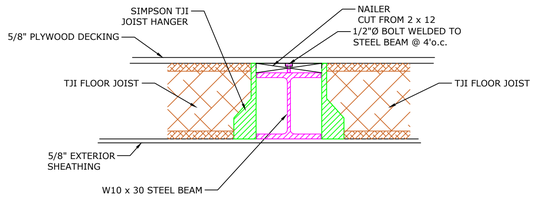
08-15-24

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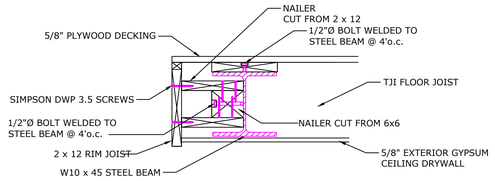
SHEET
S-11.2



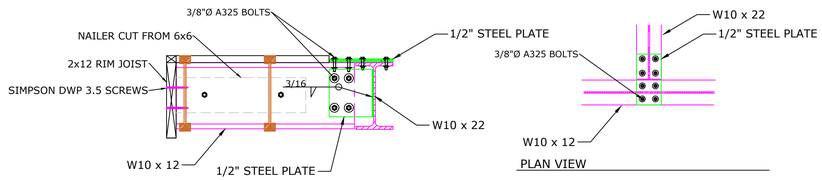
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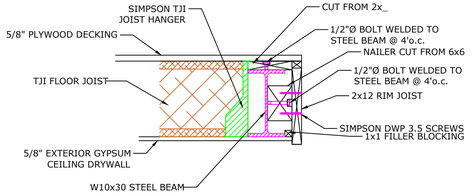
SECTION I-I



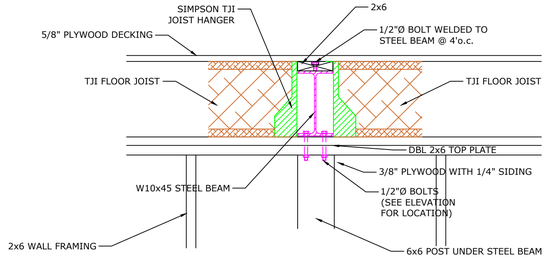
SECTION L-L



DETAIL 4

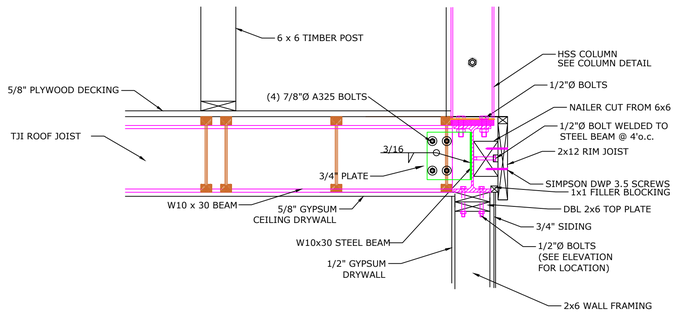


SECTION C-C

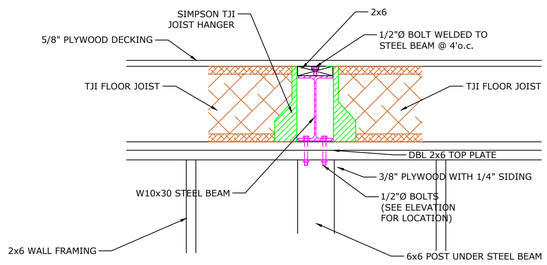


SECTION J-J

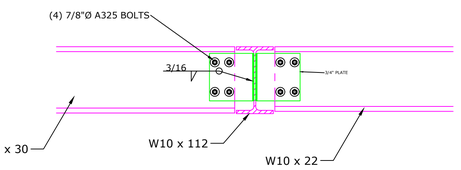
DETAIL 3



SECTION B-B



SECTION G-G



DETAIL 2

SECTION A-A

SECTION E-E & F-F

DETAIL 1

LANZ RESIDENCE
8020 SE 57th Street
Mercer Island, WA 98040

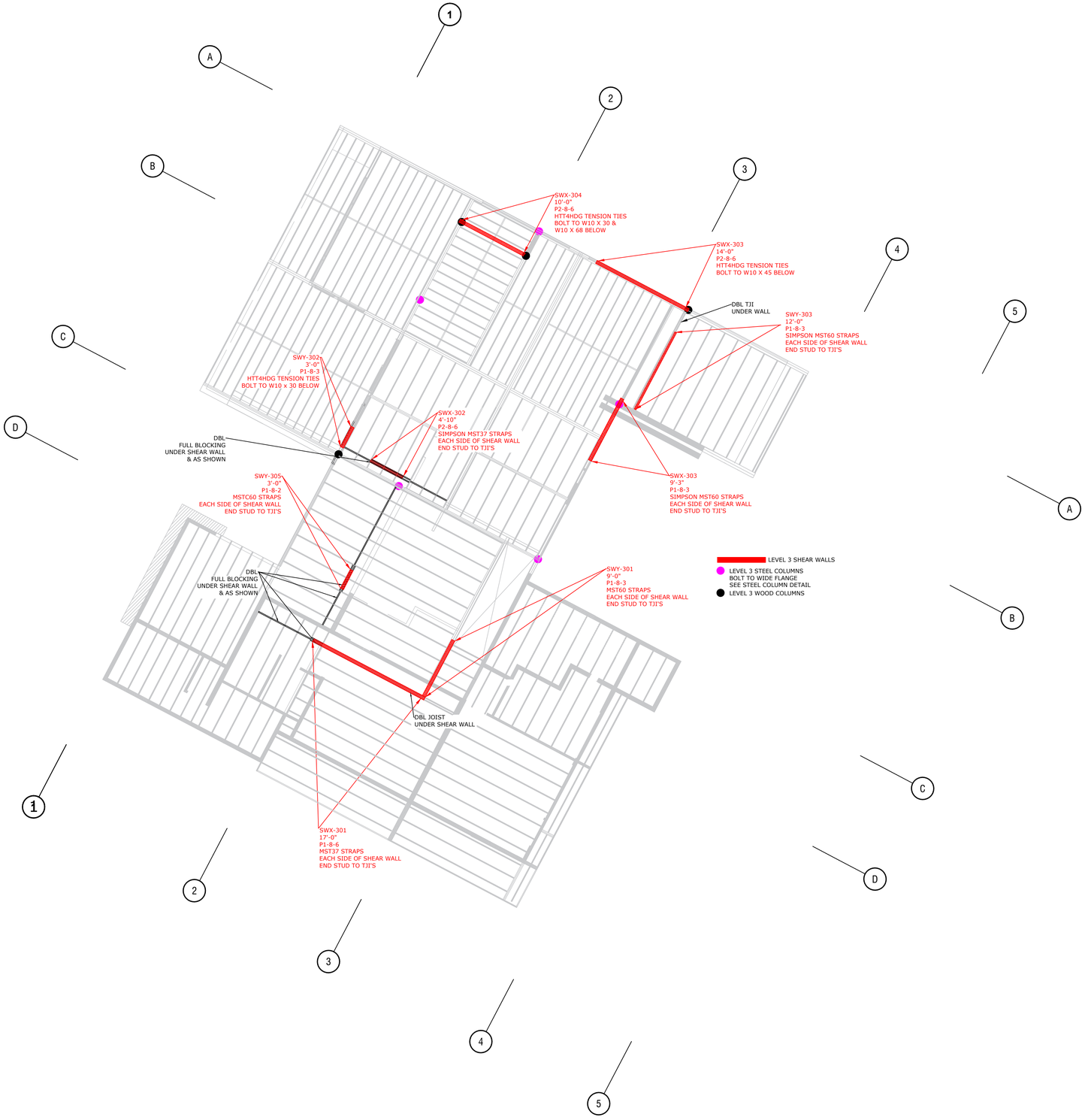
Second Level
Framing Details

LUCIA ENGINEERING, INC.
12527 Huckleberry Lane
Arlington, Washington 98223
PHONE: (206) 790-8039
E-MAIL: joe@luciaeng.com

JOSEPH M. LUCIA
STATE OF WASHINGTON
23314
REGISTERED
PROFESSIONAL ENGINEER
08-15-24

Number	Date	By	Description
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SHEET
S-11.3



SECOND FLOOR - FLOOR FRAMING - SHEAR WALL CONNECTIONS

LANZ RESIDENCE
8020 SE 57th Street
Mercier Island, WA 98040

Second Level
Shear Wall
Connections

LUCIA ENGINEERING, INC.
12527 Huckleberry Lane
Arlington, Washington 98223
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SHEET
S-11.4

FIRST FLOOR - FLOOR FRAMING

LANZ RESIDENCE
8020 SE 57th Street
Mercer Island, WA 98040

First Level
Framing

LUCIA ENGINEERING, INC.
12527 Huckleberry Lane
Arlington, Washington 98223
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SHEET
S-12.0

FIRST FLOOR - FLOOR FRAMING

LANZ RESIDENCE
8020 SE 57th Street
Mercer Island, WA 98040

First Level
Floor Framing

LUCIA ENGINEERING, INC.
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SHEET
S-12.1

FIRST FLOOR - FLOOR FRAMING - SHEAR WALL CONNECTIONS

LANZ RESIDENCE
8020 SE 57th Street
Mercer Island, WA 98040

First Level
Shear Wall
Connections

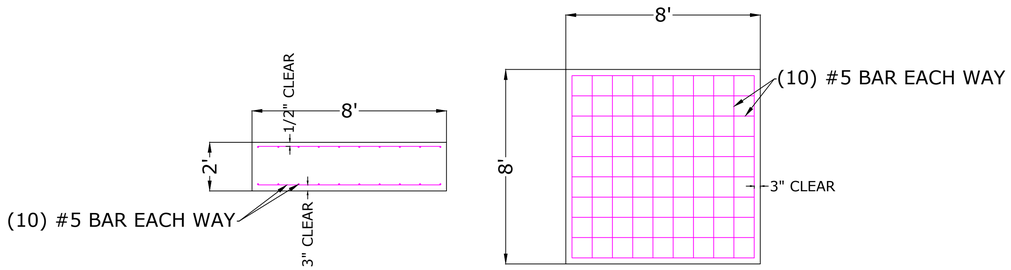
LUCIA ENGINEERING, INC.
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Arlington, Washington 98223
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JOSEPH M. LUCIA
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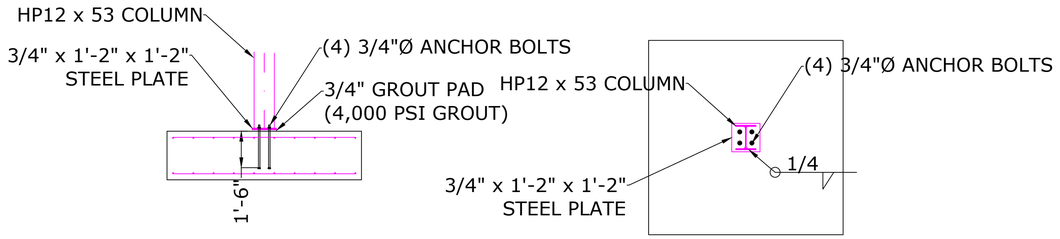
08-15-24

Number	Date	By	Description
5	08-15-24	JML	

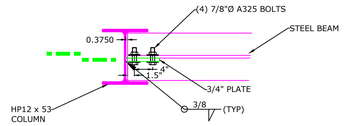
SHEET
S-12.3



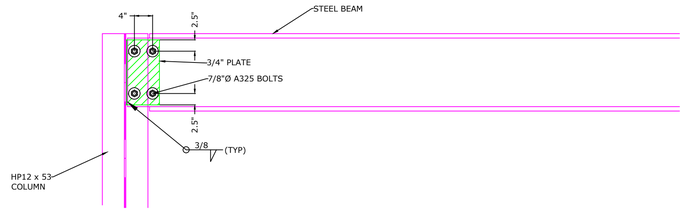
COLUMN FOOTING DETAIL



COLUMN TO FOOTING DETAIL

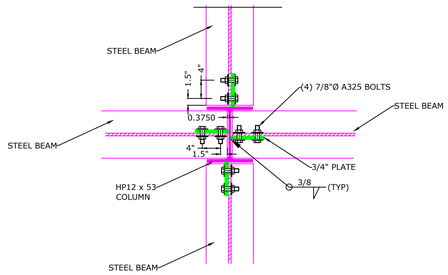


PLAN

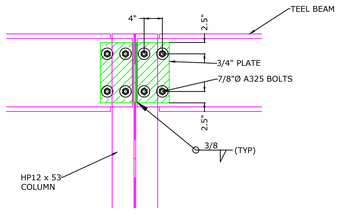


ELEVATION

DETAIL D

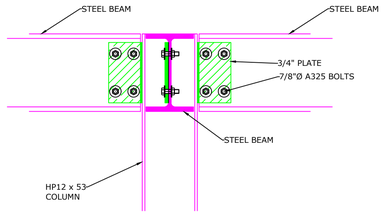


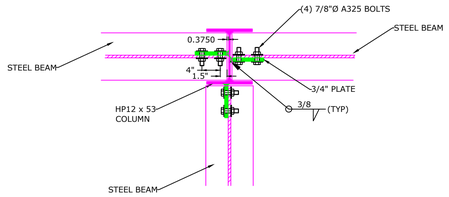
PLAN



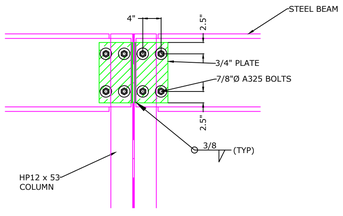
ELEVATION

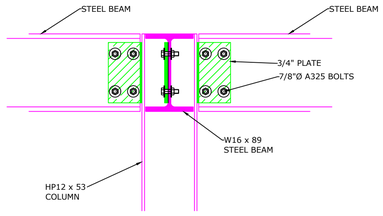
DETAIL B





PLAN





ELEVATION

DETAIL C

LANZ RESIDENCE
8020 SE 57th Street
Mercier Island, WA 98040

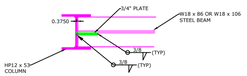
Steel Framing Details

LUCIA ENGINEERING, INC.
12527 Huckleberry Lane
Arlington, Washington 98223
PHONE: (206) 790-8039
E-MAIL: joe@luciaeng.com

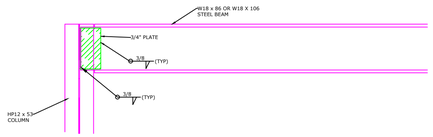
JOSEPH M. LUCIA
STATE OF WASHINGTON
23314
REGISTERED PROFESSIONAL ENGINEER
08-15-24

Number	Date	By	Description
5	08-15-24	JML	

SHEET
S-12.4

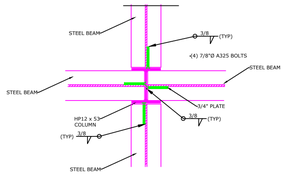


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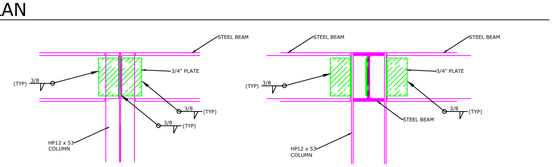


ELEVATION

DETAIL D

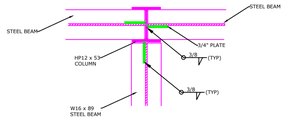


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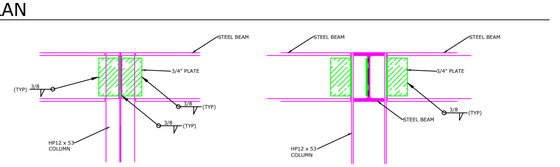


ELEVATION

DETAIL B

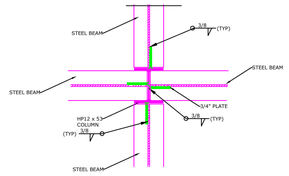


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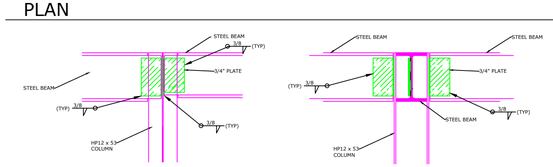


ELEVATION

DETAIL C

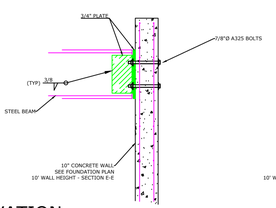


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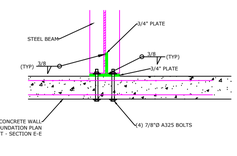


ELEVATION

DETAIL D

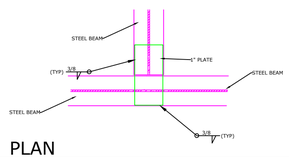


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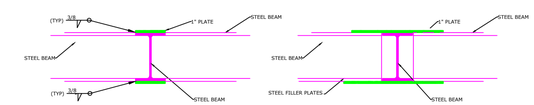


PLAN

DETAIL E



PLAN



ELEVATION

DETAIL F

ALTERNATE WELDED CONNECTIONS

LUCIA ENGINEERING, INC.
12527 Huckleberry Lane
Arlington, Washington 98223
PHONE: (206) 790-8039
E-MAIL: joe@luciaeng.com

LANZ RESIDENCE
8020 SE 57th Street
Mercer Island, WA 98040

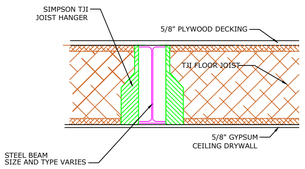
Steel Framing Details

JOSEPH M. LUCIA
STATE OF WASHINGTON
23314
REGISTERED
PROFESSIONAL ENGINEER

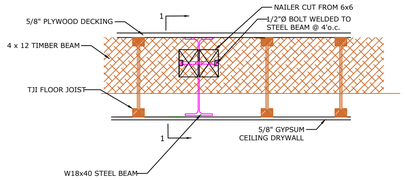
08-15-24

Number	Date	By	Description
5	08-15-24	JML	

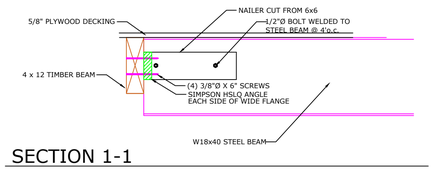
SHEET
S-12.5



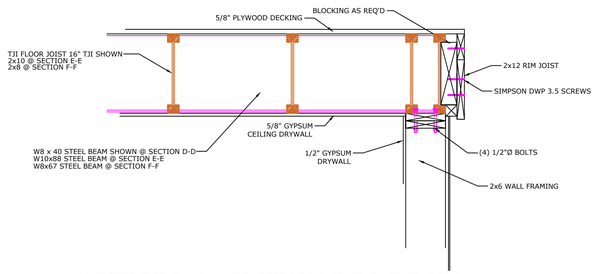
SECTION H-H



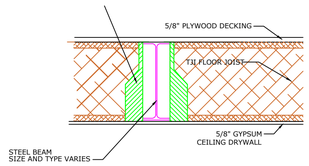
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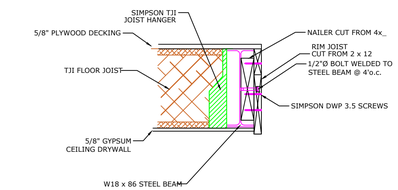
SECTION 1-1



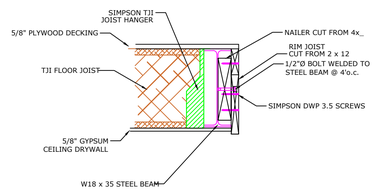
SECTION D-D (SECTIONS E-E & F-F SIM.)



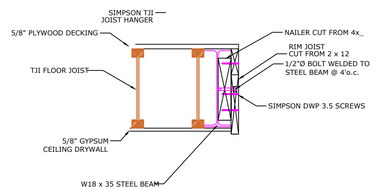
SECTION J-J



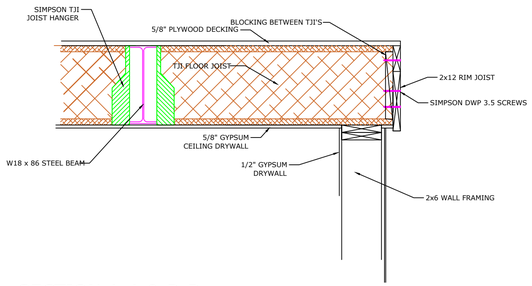
SECTION C-C



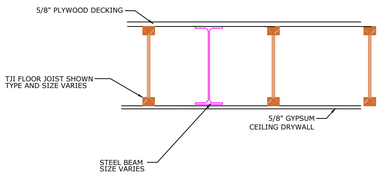
SECTION I-I (SECTION J-J, OPPOSITE HAND)



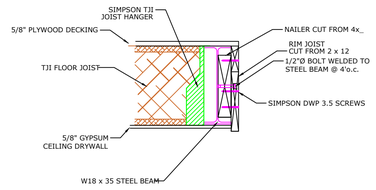
SECTION L-L



SECTION A-A & B-B



SECTION I-I



SECTION K-K

LANZ RESIDENCE
8020 SE 57th Street
Mercer Island, WA 98040

First Level
Framing Details

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JOSEPH M. LUCIA
STATE OF WASHINGTON
23314
REGISTERED
PROFESSIONAL ENGINEER

08-15-24

Number	Date	By	Description
5	08-15-24	JML	

SHEET
S-12.6