LUCIA ENGINEERING, INC.

LNL Builds
317 4th Street
Kirkland, Washington 98003

April 27, 0224

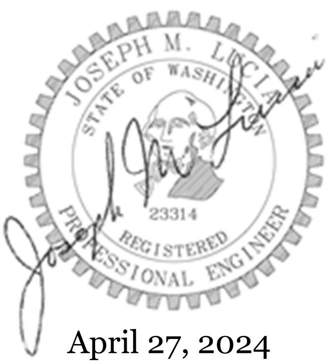
Attention: **Vann Lanz**

Reference: Lanz Residence
<u>Soldier Pile & Timber Lagging Shoring Wall Design</u>

Vann

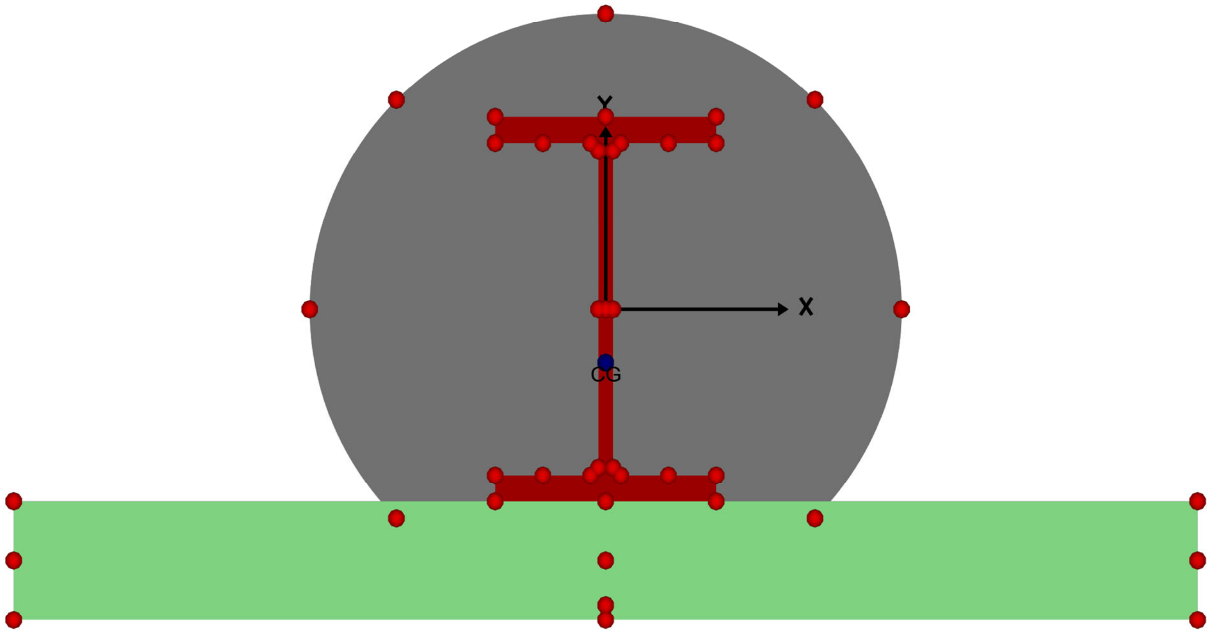
Attached are the calculations and plans required for the soldier pile shoring wall.

Please contact me with any questions.



April 27, 2024
Joseph M. Lucia, PE

7307 12th Avenue NE, Seattle, Washington 98115
155 Bovet Road, Suite 550, San Mateo, California 94402


Y
X
CG

Geometric Properties

Area*	143.391	in^2
Ix*	10819.225	in^4
Ixy*	0.000	in^4
Iy*	15938.295	in^4
Sx+*	611.081	in^3
Sx-*	829.380	in^3
Sy+*	531.276	in^3
Sy-*	531.276	in^3
Xc*	0.000	in
Yc*	-2.705	in
rx*	8.686	in
ry*	10.543	in

* means transformed by E/E_base

Overall Properties

Depth	30.750	in
E_base	29000.000	Ksi
Perimeter	177.559	in
Weight	1.124	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	7.460	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	38.756	11626.759	116.268	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

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

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.

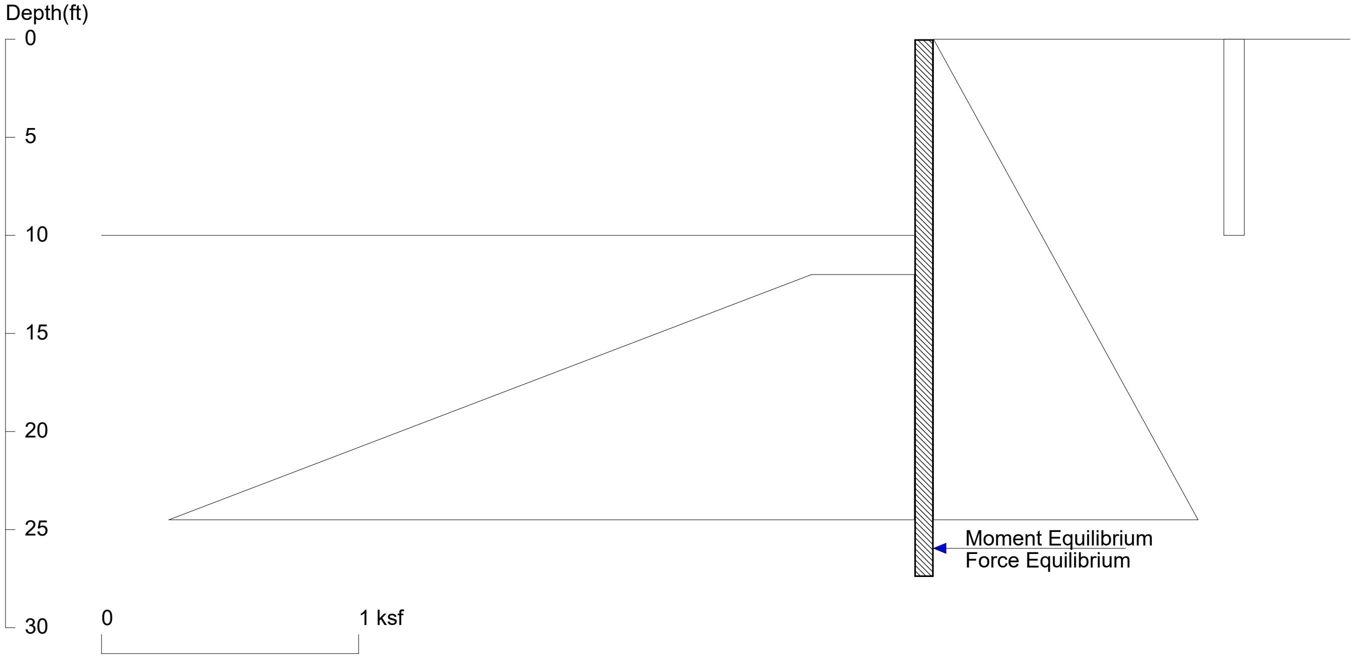
Drainage should be provided behind retaining walls such that hydrostatic pressures do not develop. If drainage is not provided, hydrostatic pressures should be included in the wall design. A perforated drainpipe should be placed along the base of the wall and connected to an approved discharge location.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



<ShoringSuite> **CIVILTECH SOFTWARE USA www.civiltech.com**

Licensed to 4324324234 3424343 Date: 3/4/2024

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

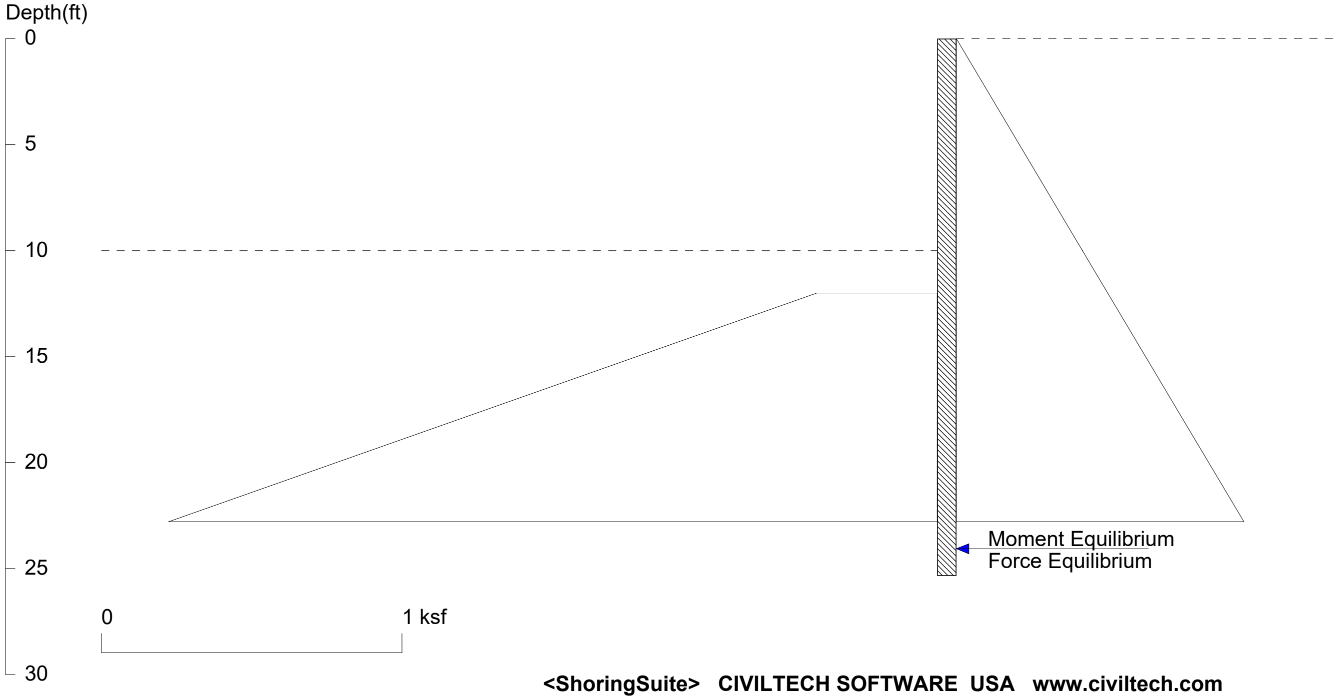
PASSIVE SPACING:

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



Licensed to 4324324234 3424343 Date: 3/4/2024

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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Licensed to   4324324234     3424343
Date: 3/4/2024  File: C:\Lucia Engineering 2024\Clients\LNL Builds\4425 Fremont
Aver N\10' 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: 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=0.00
        |
        |
        |
      ==|==    D2=10.00
        |
        |
        |      D3=27.41
```

```
        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):

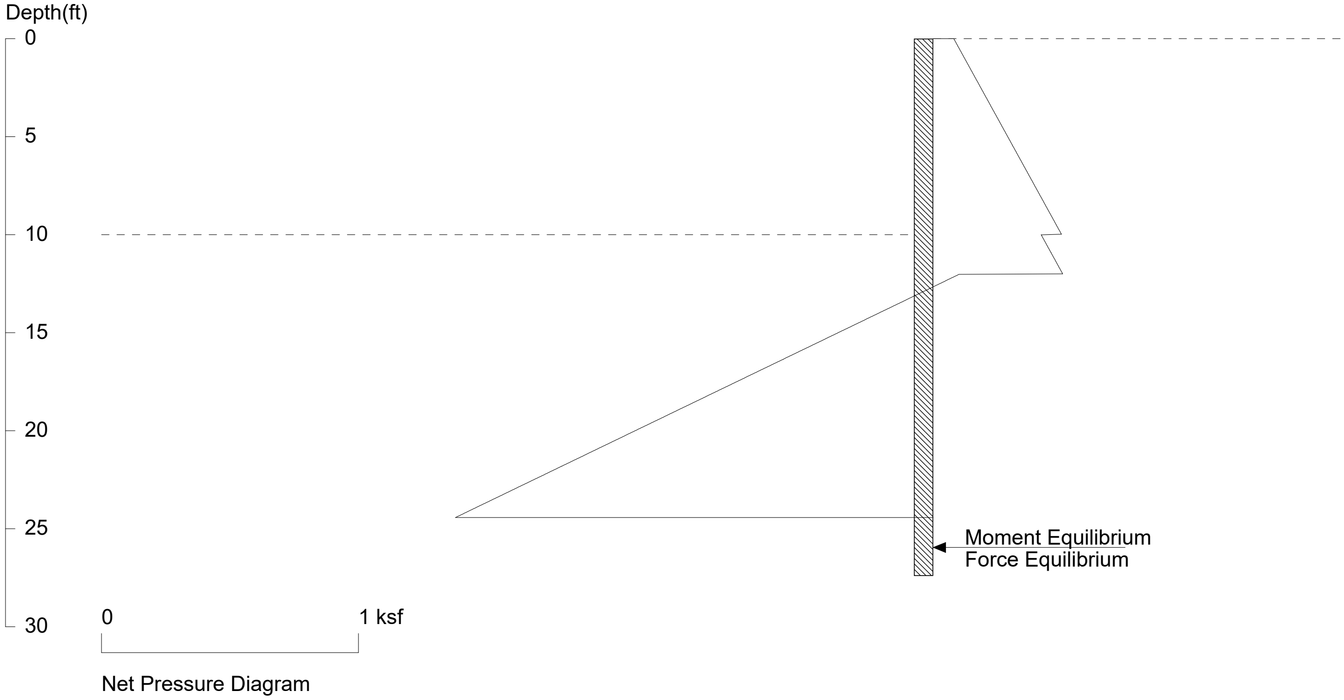
```
   Area= 35.7 in.   Depth= 15.75 in.  Width= 15.875 in.  Height= 16 in.
   Flange thickness= 0.75 in.   Web thickness= 0.75 in.
   Ix= 1578 in4/pile   Sx= 200 in3/pile   Iy= 501 in4/pile   Sy= 63.1 in3/pile
   (Metric Units):
   Ix= 656.76 x100cm4/pile   Sx= 3277.40 cm3/pile   Iy= 208.52 x100cm4/pile   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)

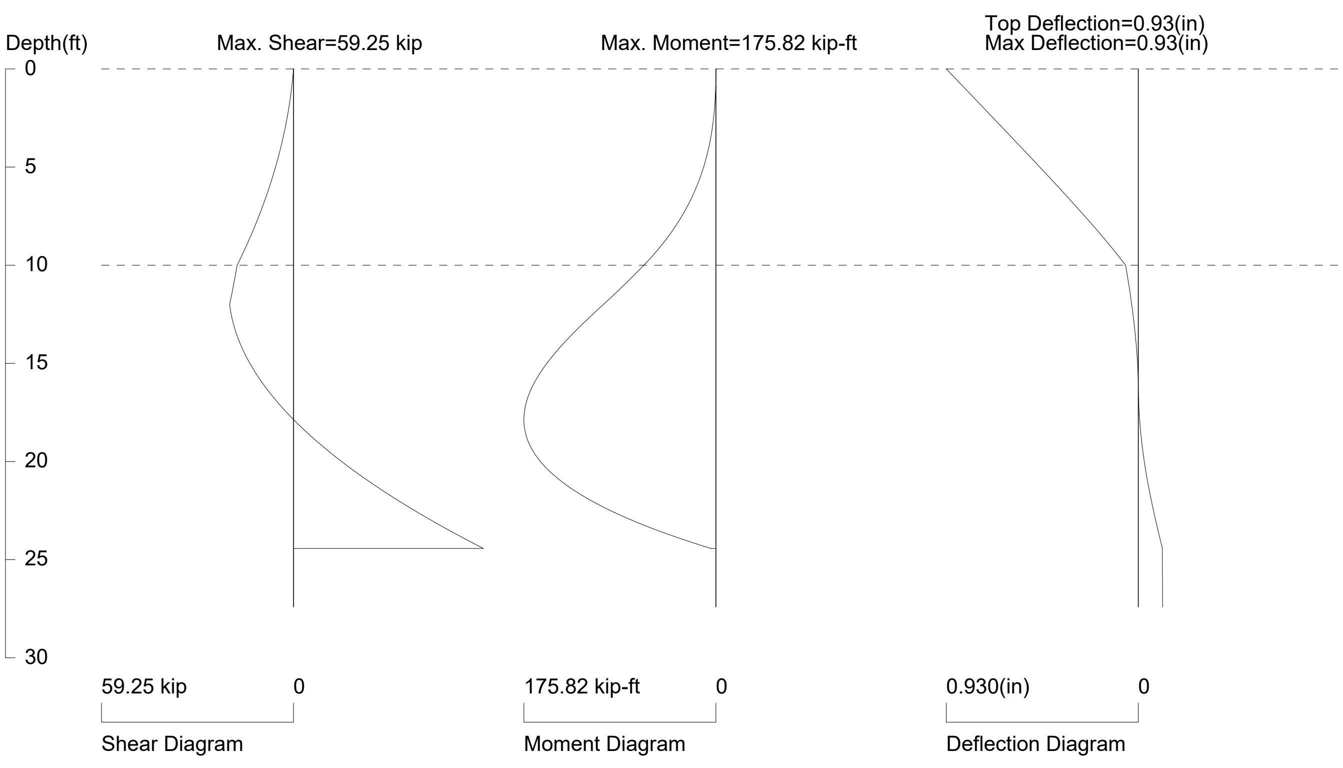
*********************** LAGGING SIZE ESTIMATION ***********************
Max. Pressure above base = 0.50
Piles are more rigid than timber lagging, due to arching, only portion of pressures
are acting to lagging, 30-50% loading is suggested.
     If 50% loading is used for lagging design, Design Pressure = 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





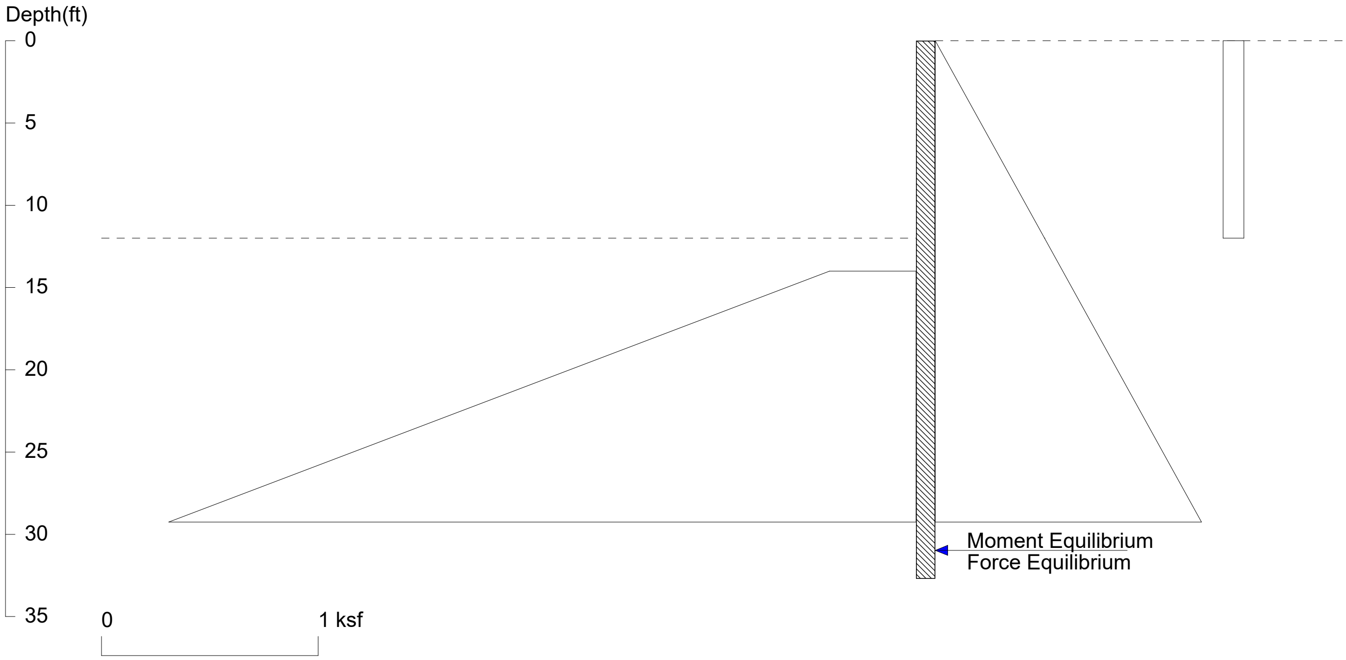
PRESSURE, SHEAR, MOMENT, AND DEFLECTION DIAGRAMS

Based on pile spacing: 6.0 foot or meter

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

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

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



<ShoringSuite> CIVILTECH SOFTWARE USA www.civiltech.com

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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:

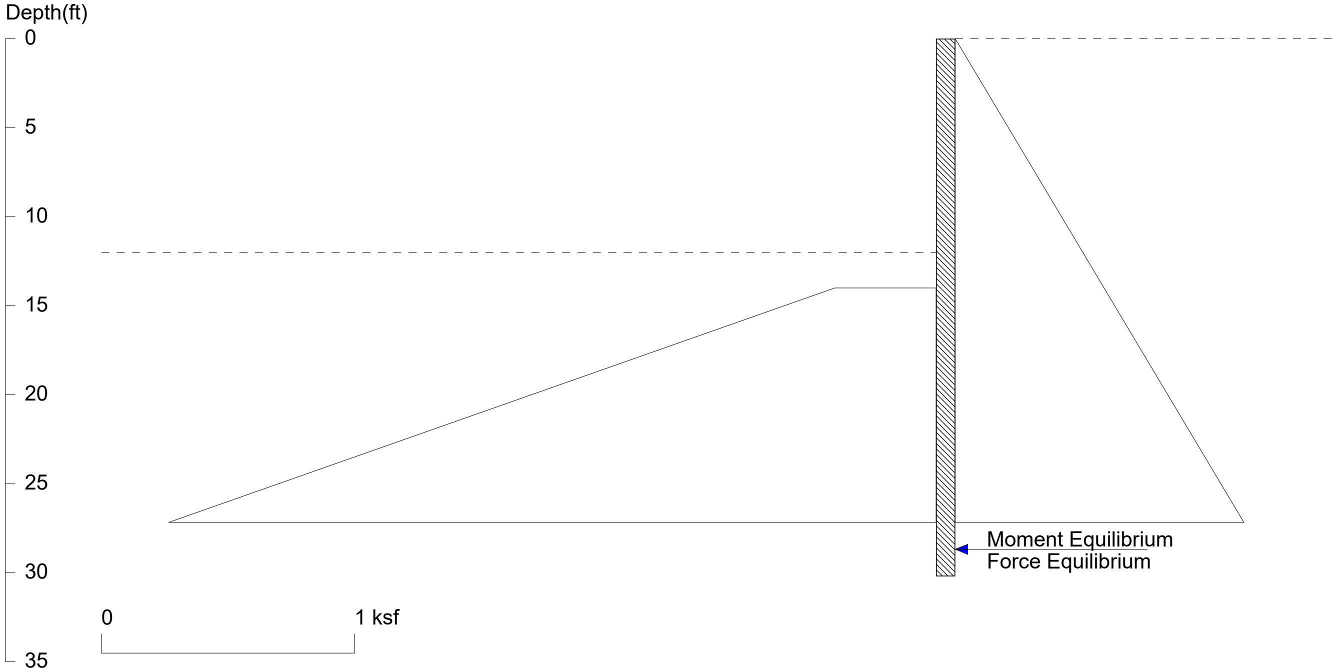
No.	Z depth	Spacing
1	0.00	6.00
2	12.00	2.50

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
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    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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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=0.00
        |
        |
        |
      ==|==    D2=12.00
        |
        |
        |      D3=32.70
```

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)

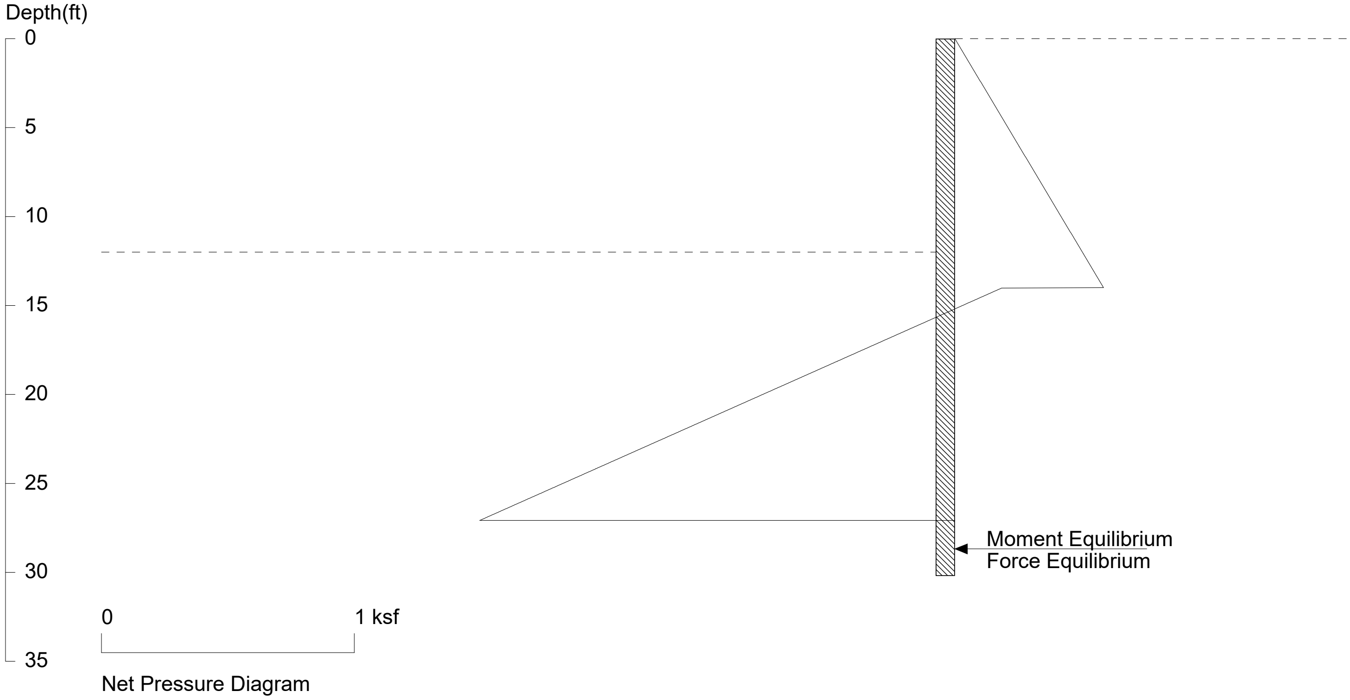
********************** LAGGING SIZE ESTIMATION **********************
Max. Pressure above base = 0.60
Piles are more rigid than timber lagging, due to arching, only portion of pressures are acting to lagging, 30-50% loading is suggested.
If 50% loading is used for lagging design, Design Pressure = 0.30
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

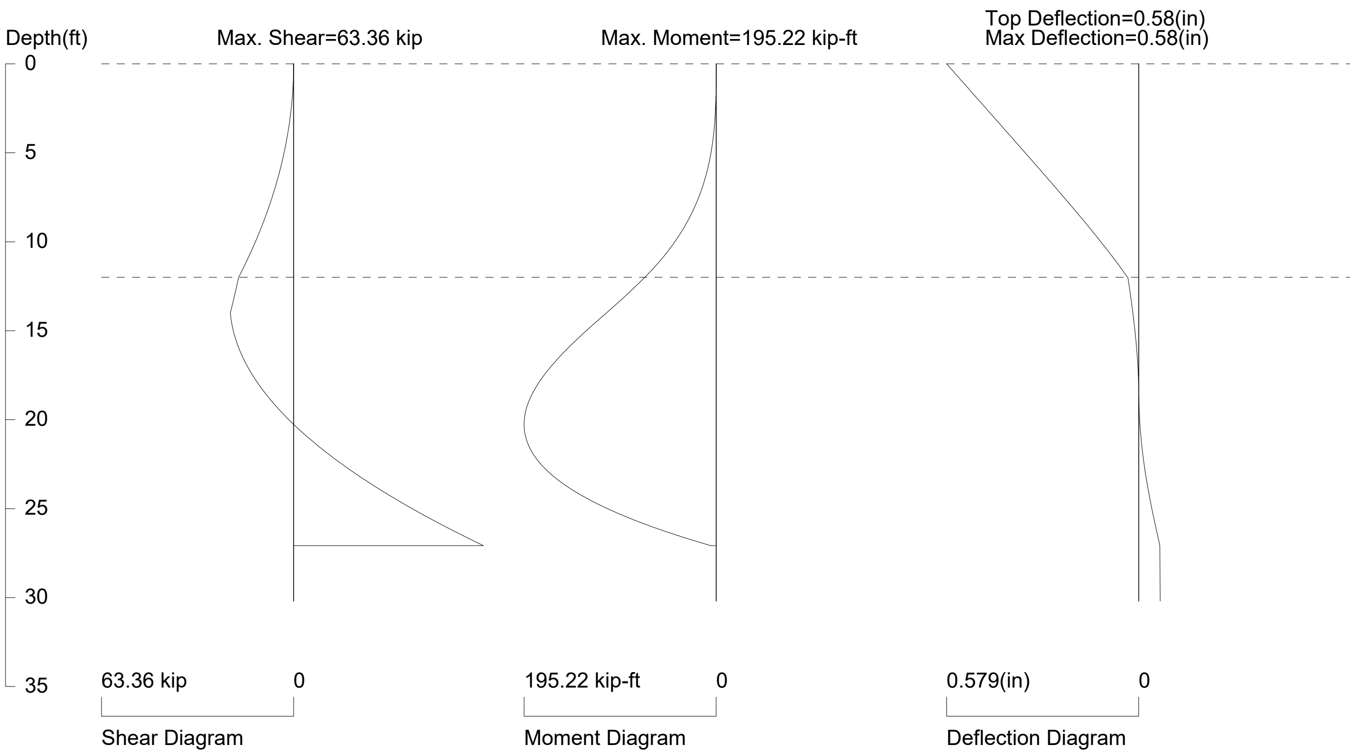
If 30% loading is used for lagging design, Design Pressure = 0.18
Pile Spacing =6.0, Max. Moment in lagging = 0.81
For 4"x12" Timber, Section Modules S=23.47 in3. The request allowable bending strength, fb=M/S=0.41
For 6"x12" Timber, Section Modules S=57.98 in3. The request allowable bending strength,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

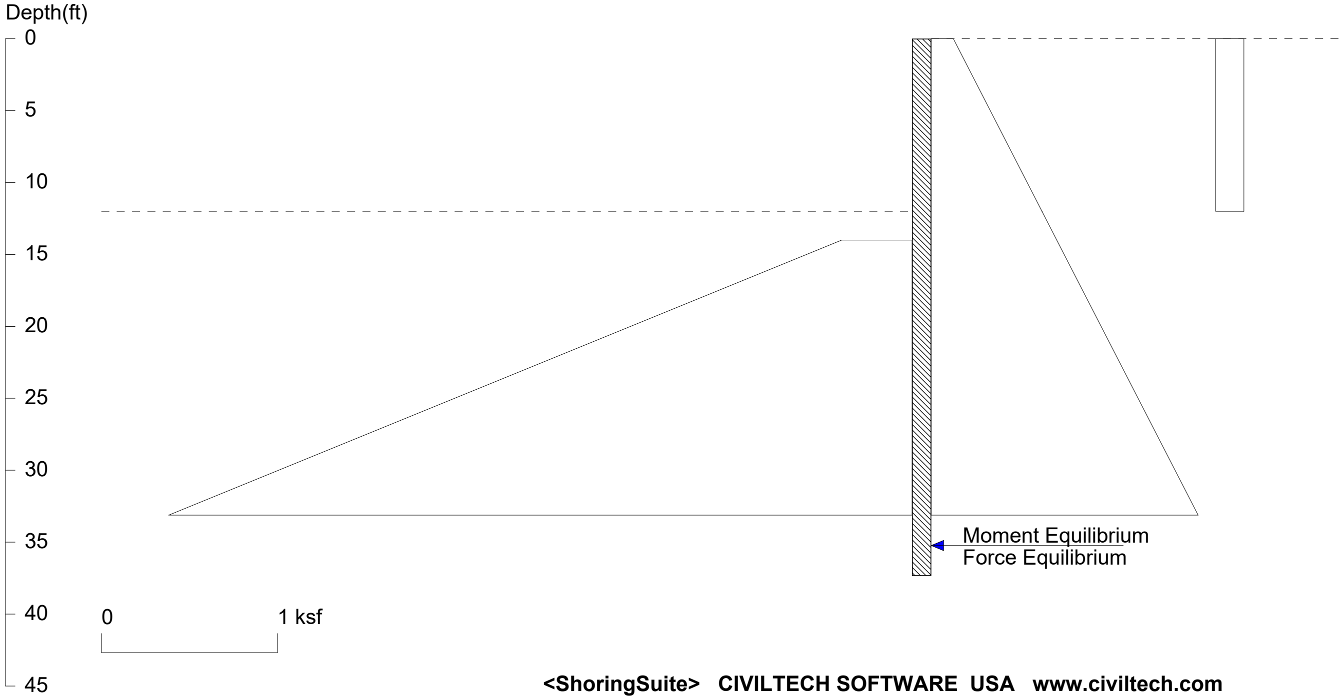
Based on pile spacing: 6.0 foot or meter

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

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

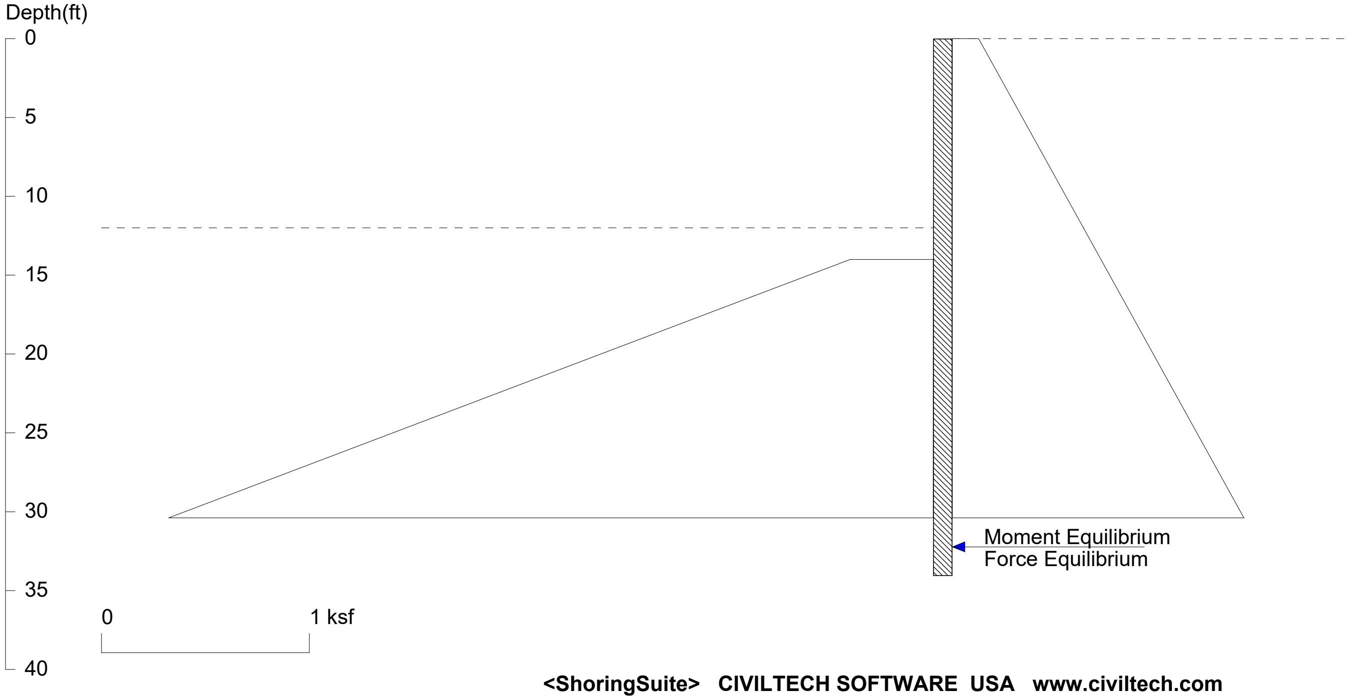
PASSIVE SPACING:

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
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    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
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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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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=0.00
        |
        |
        |
      ==|==    D2=12.00
        |
        |
        |      D3=37.34
```

```
        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)

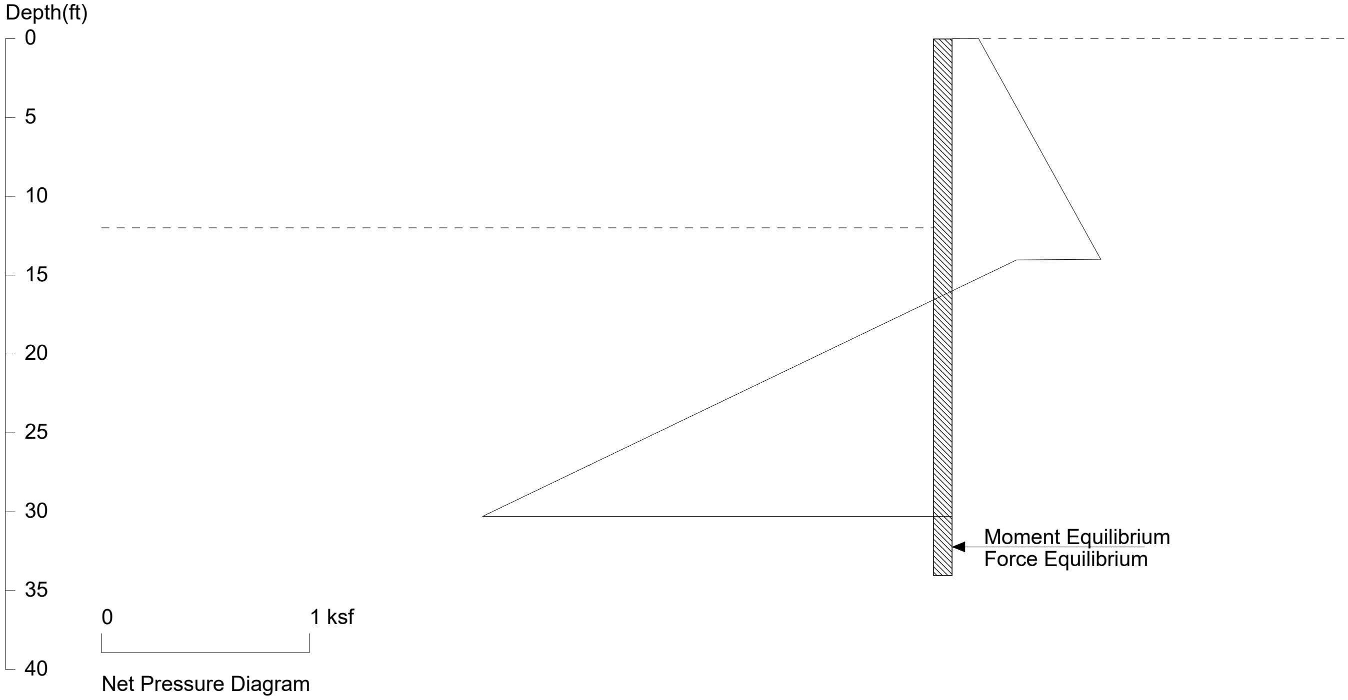
********************** LAGGING SIZE ESTIMATION **********************
Max. Pressure above base = 0.79
Piles are more rigid than timber lagging, due to arching, only portion of pressures are acting to lagging, 30-50% loading is suggested.
If 50% loading is used for lagging design, Design Pressure = 0.39
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

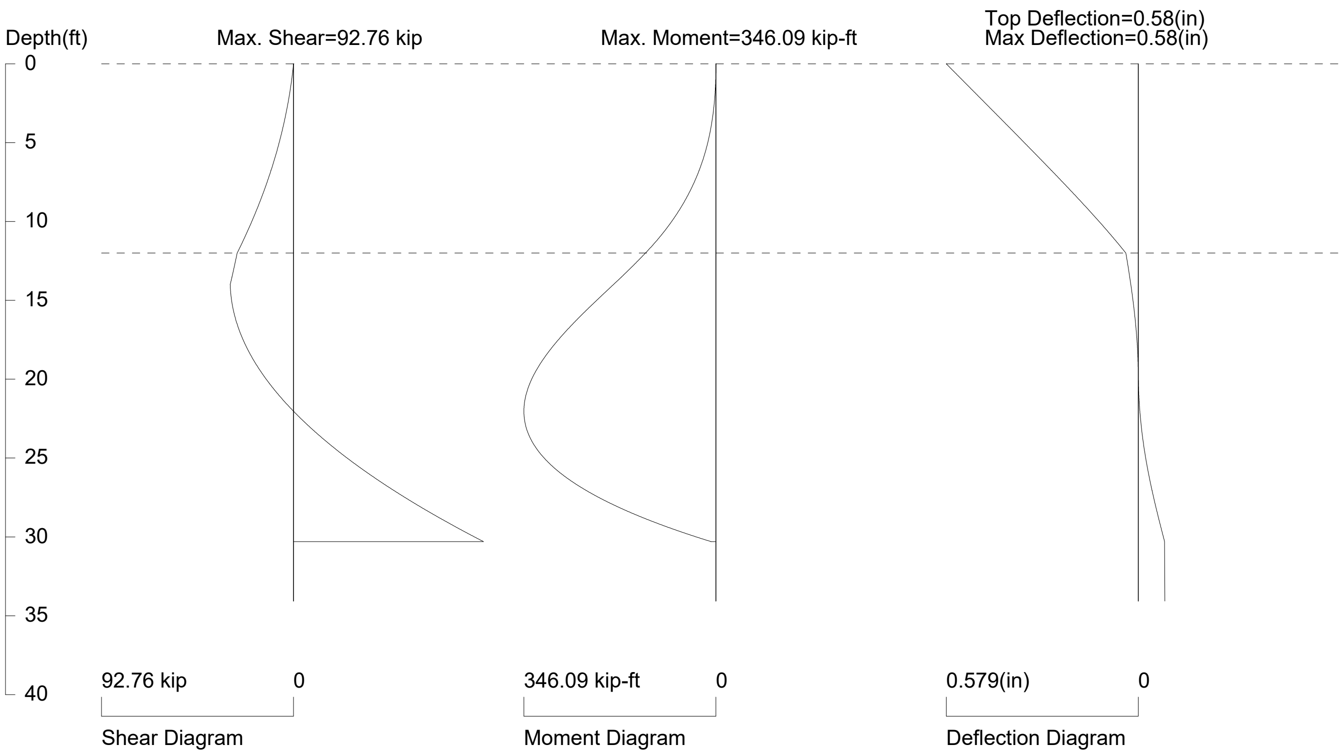
If 30% loading is used for lagging design, Design Pressure = 0.24
Pile Spacing =6.0, Max. Moment in lagging = 1.07
For 4"x12" Timber, Section Modules S=23.47 in3. The request allowable bending strength, fb=M/S=0.54
For 6"x12" Timber, Section Modules S=57.98 in3. The request allowable bending strength,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

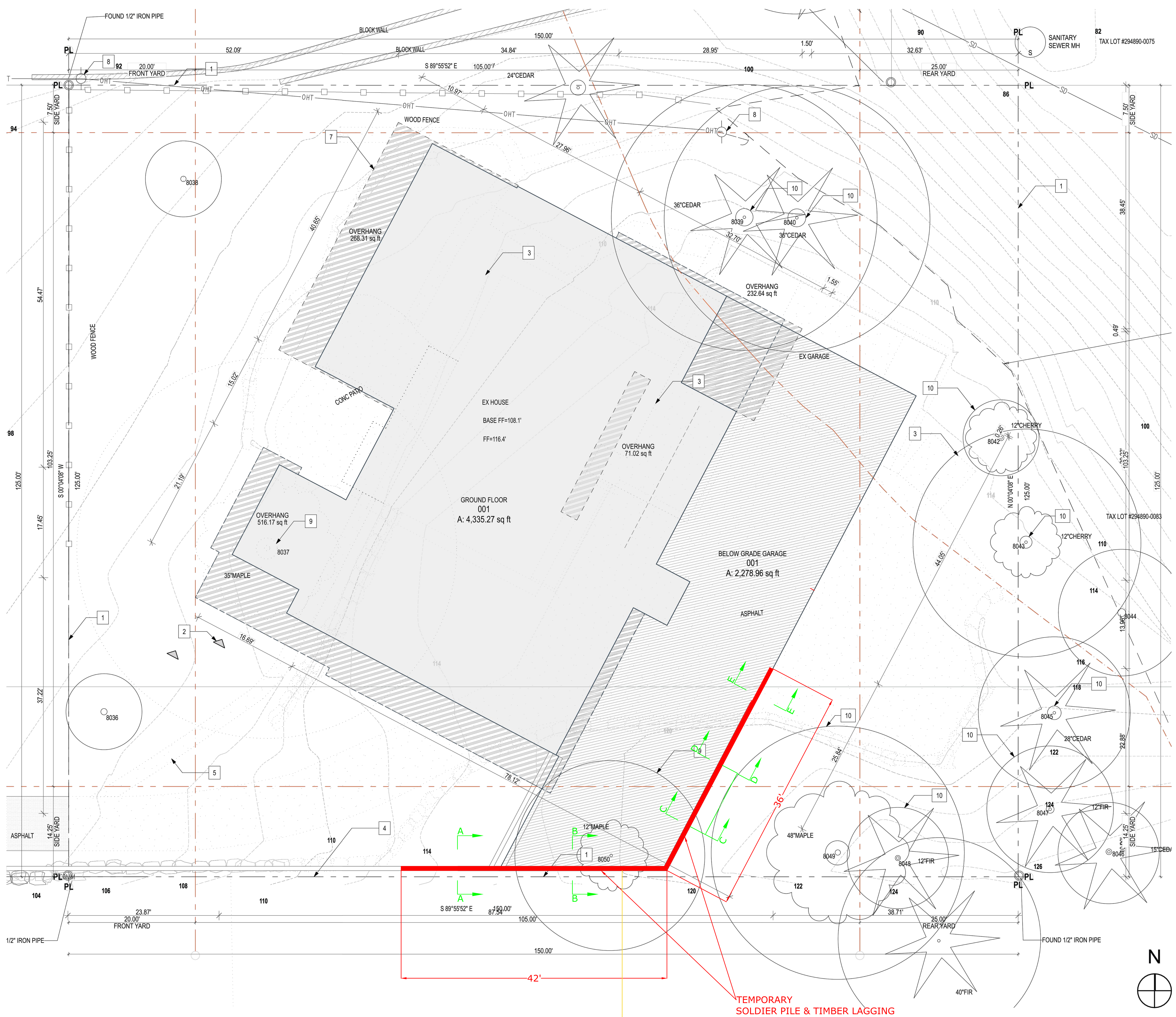
File: C:\Lucia Engineering 2024\Clients\LNL Builds\4425 Fremont Aver N\12' BS Tall Soldier Pile Shoring Wall - DL.sh8

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 - 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
04-27-24

Number	Date	By	Description
3	04-27-24	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:

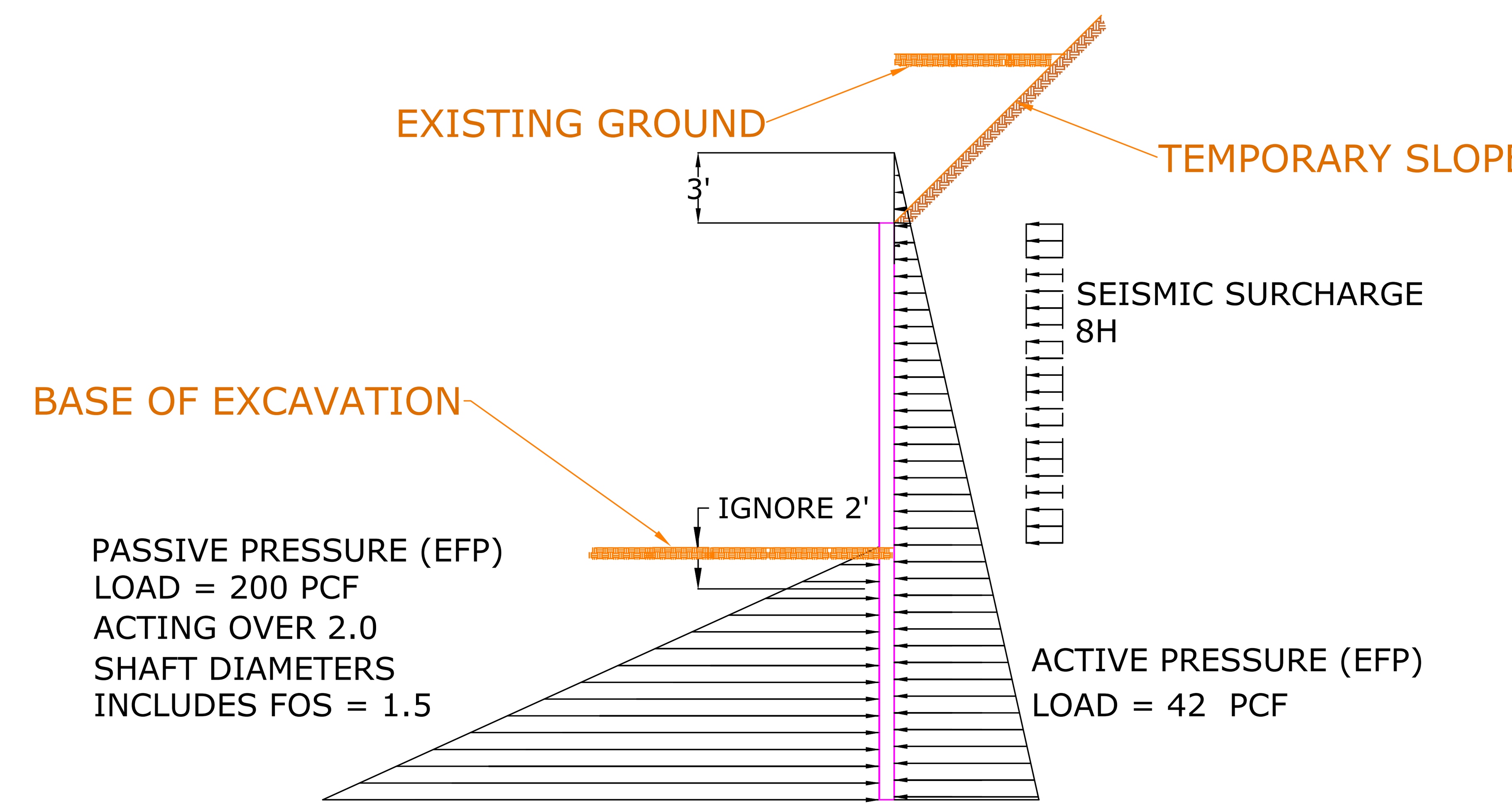
ALL EXISTING UTILITIES AND OTHER OBJECTS WHICH MAY INTERFERE WITH THE INSTALLATION OF THE SHORING SYSTEM ARE TO BE LOCATED PRIOR TO BEGINNING CONSTRUCTION.

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

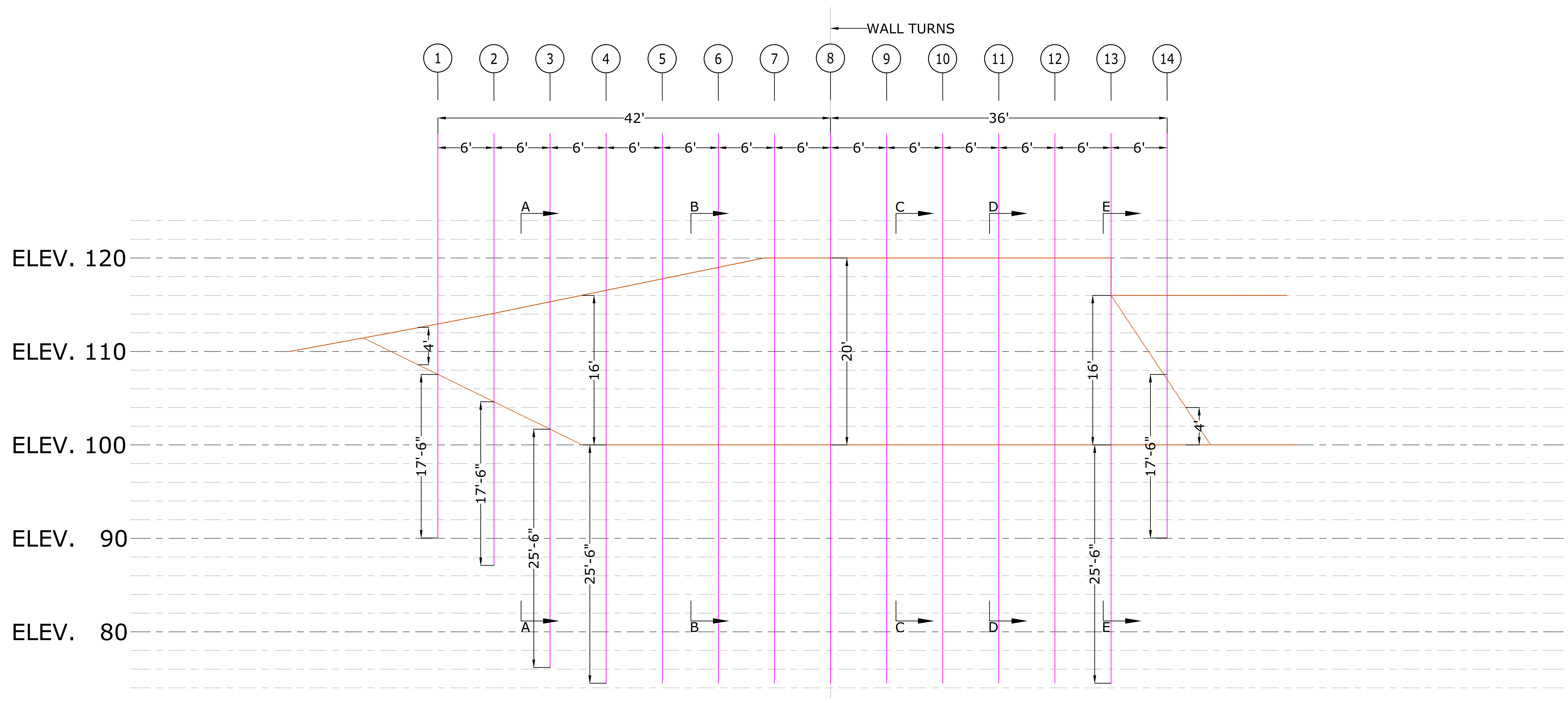
Permanent Soldier Pile
& Timber Lagging
Retaining Wall

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

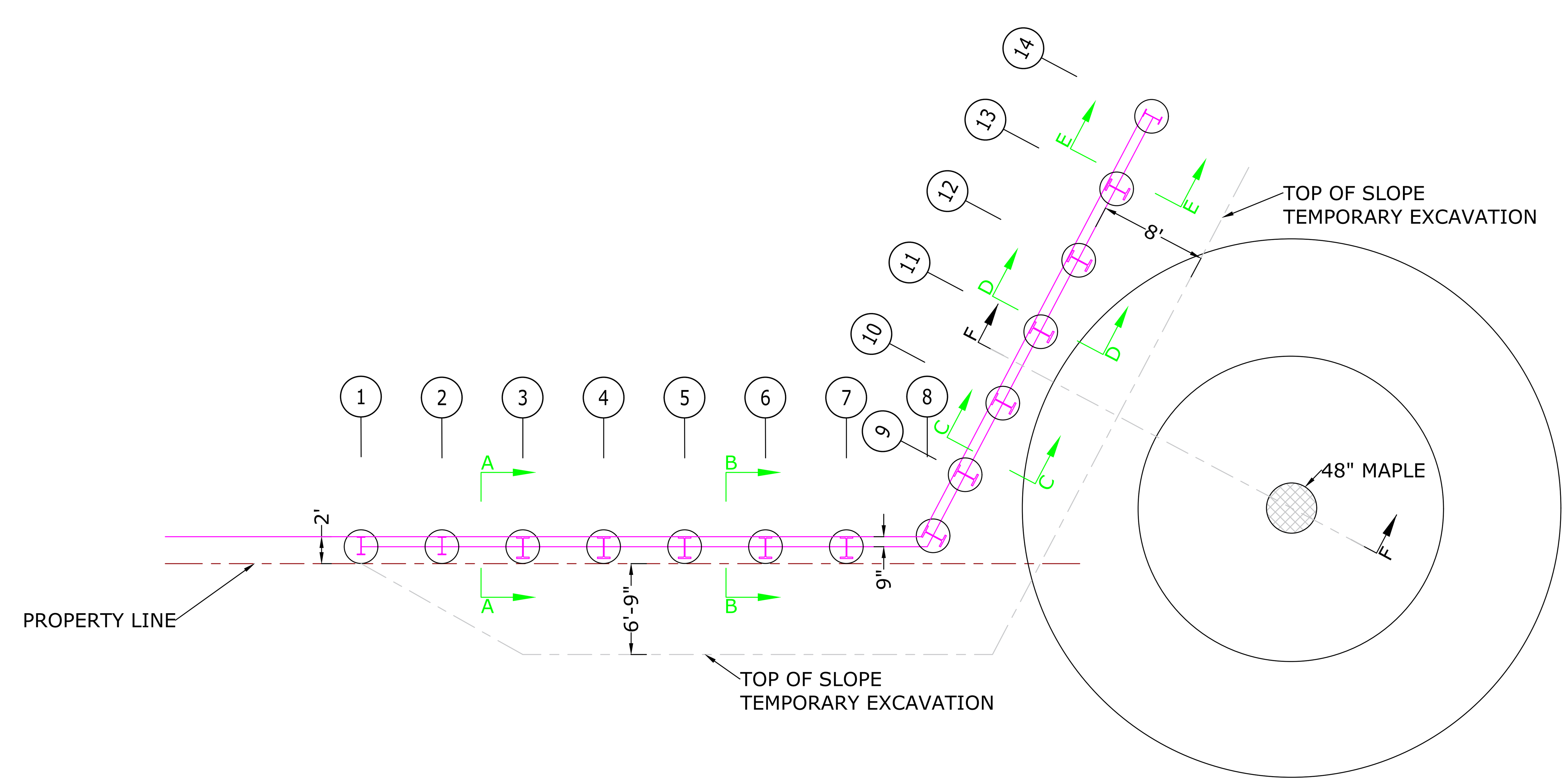
JOSEPH M. LUCIA
STATE OF WASHINGTON
23314
REGISTERED PROFESSIONAL ENGINEER
04-27-24

Number	Date	By	Description
3	04-27-24	JML	

SHEET
S-2.0



ELEVATION



PLAN

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

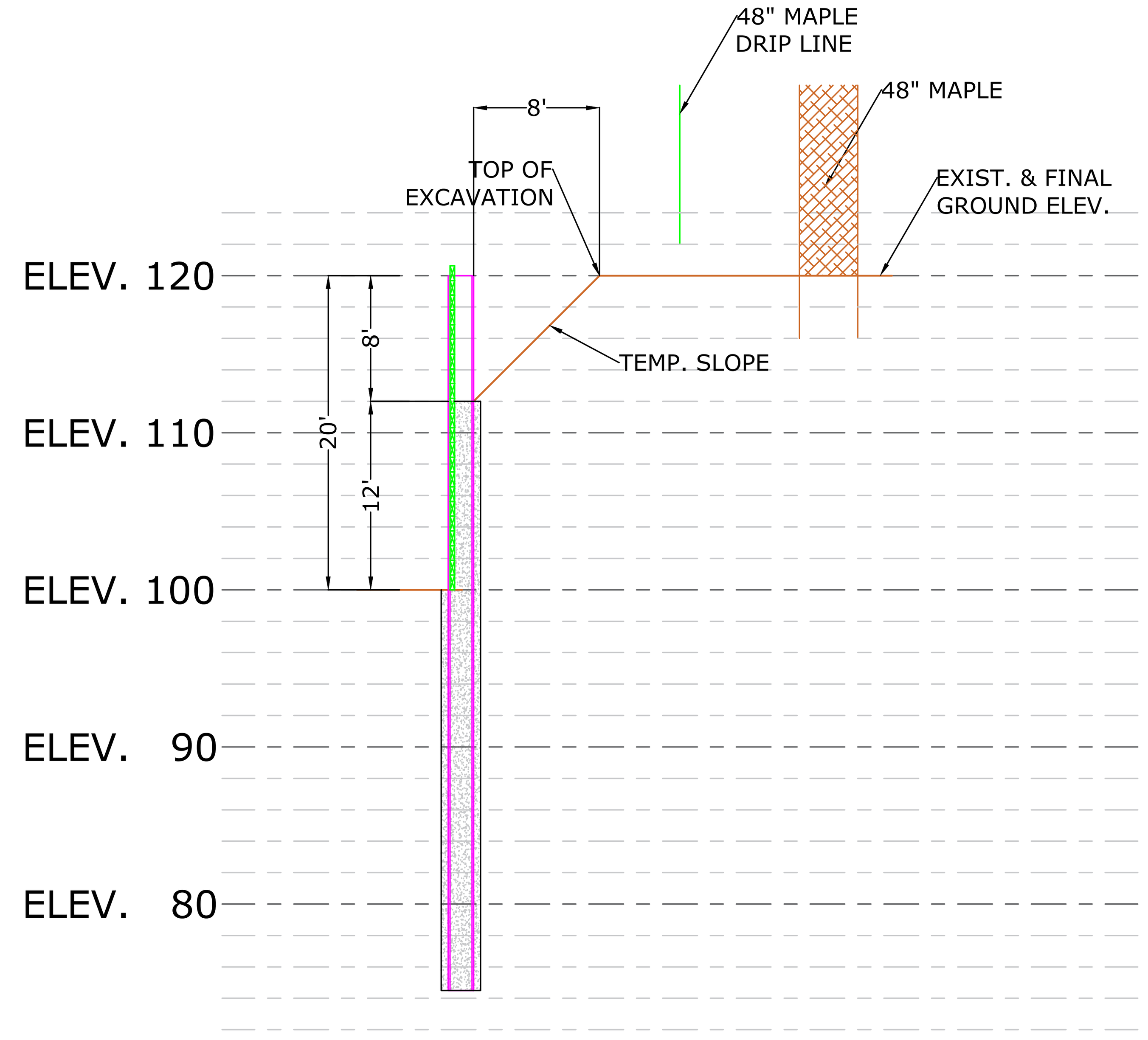
Permanent Soldier Pile
& Timber Lagging
Retaining Wall

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

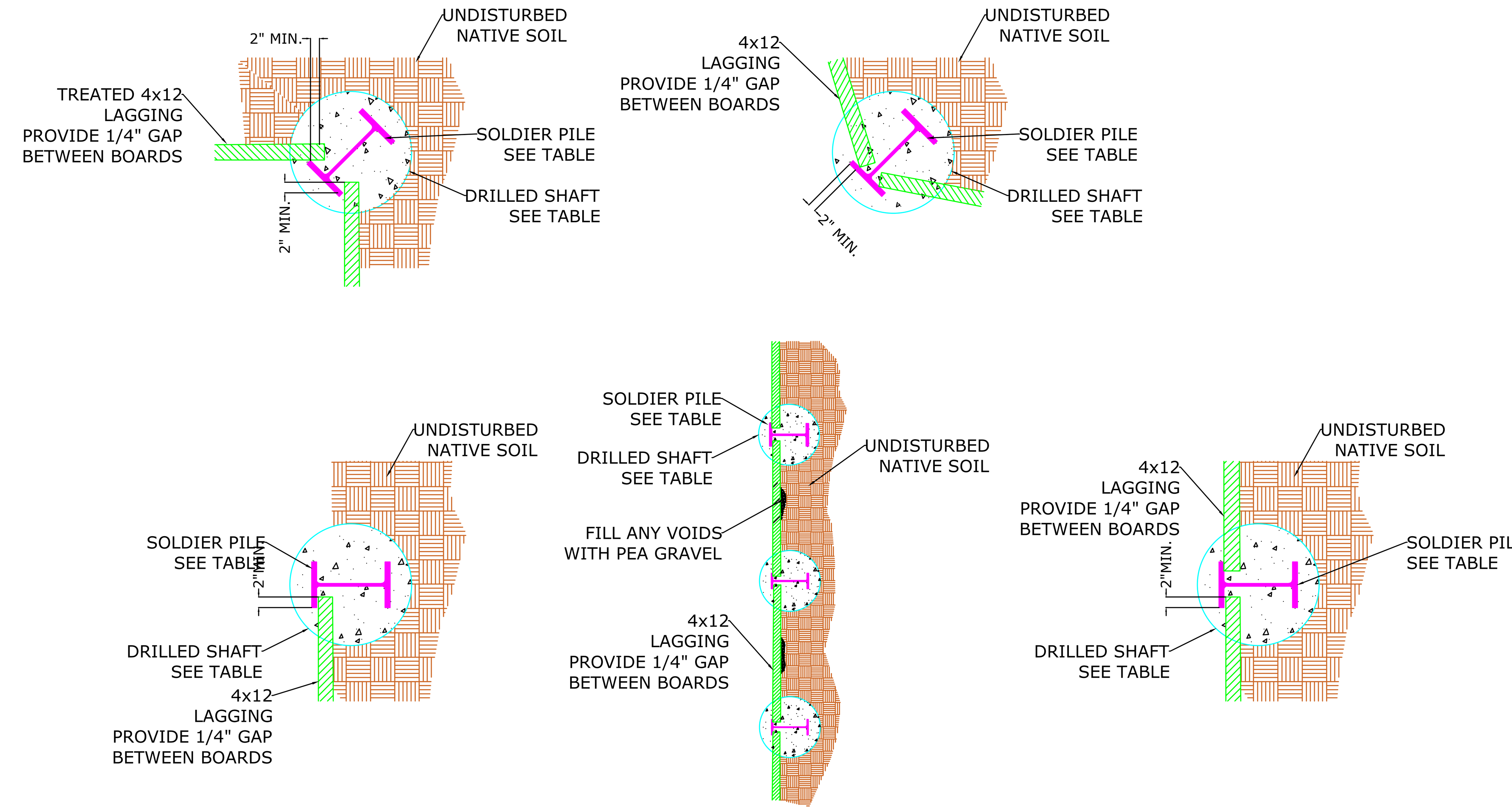
JOSEPH M. LUCIA
STATE OF WASHINGTON
23314
REGISTERED PROFESSIONAL ENGINEER
04-27-24

Number	Date	By	Description
3	04-27-24	JML	

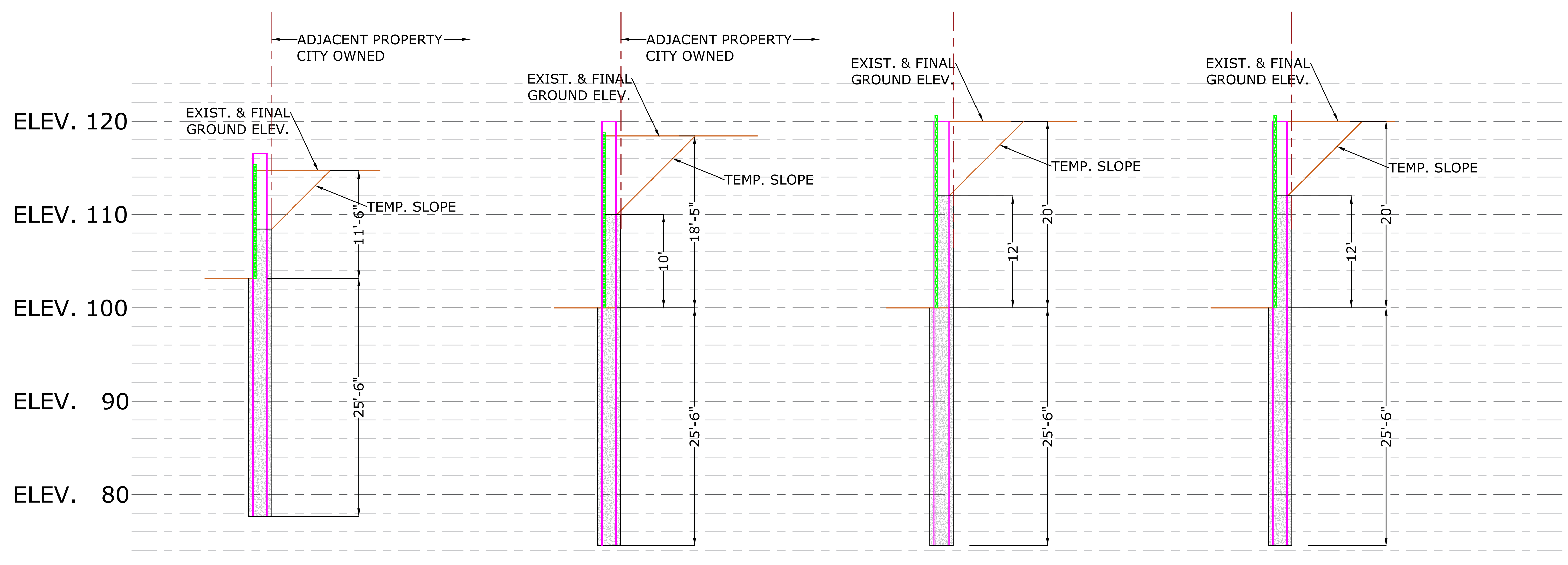
SHEET
S-3.0



SECTION F-F



TYPICAL DETAILS - SOLDIER PILE & TIMBER LAGGING

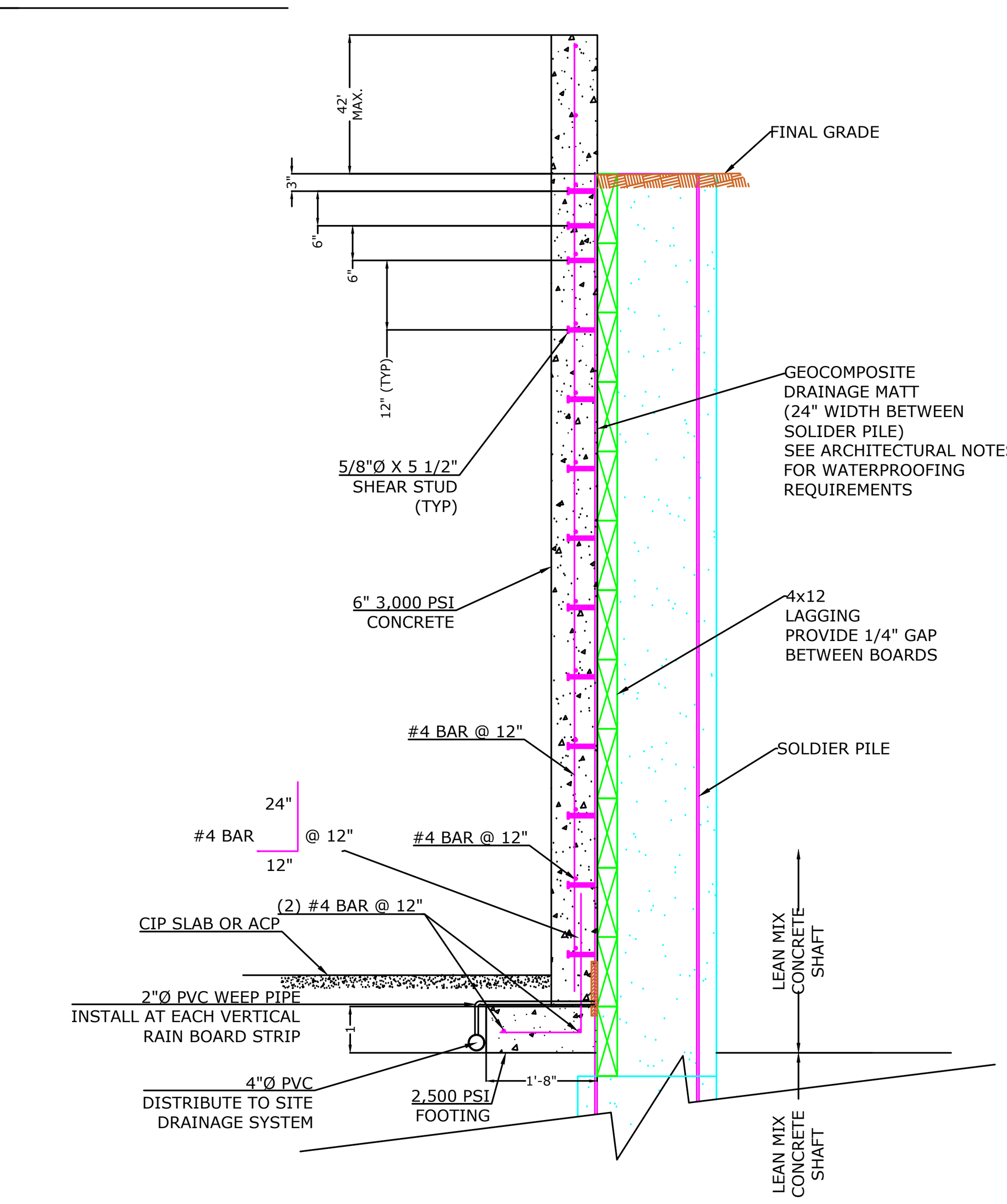


SECTION A-A

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
8020 SE 57th Street
Mercer Island, WA 98040

Permanent Soldier Pile & Timber Lagging Retaining Wall

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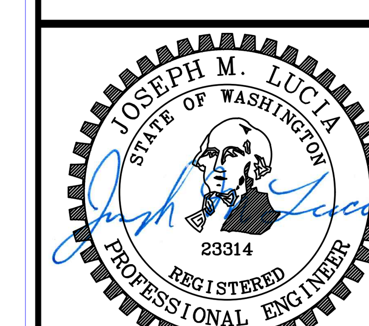


04-27-24

Number	Date	By	Description
3	04-27-24	JML	

SHEET
S-4.0

LNL Builds
317 4th Street
Kirkland, Washington 98003

April 21, 2024

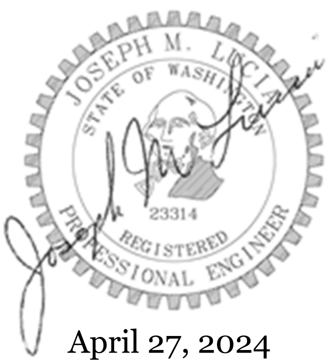
Attention: **Vann Lanz**

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.



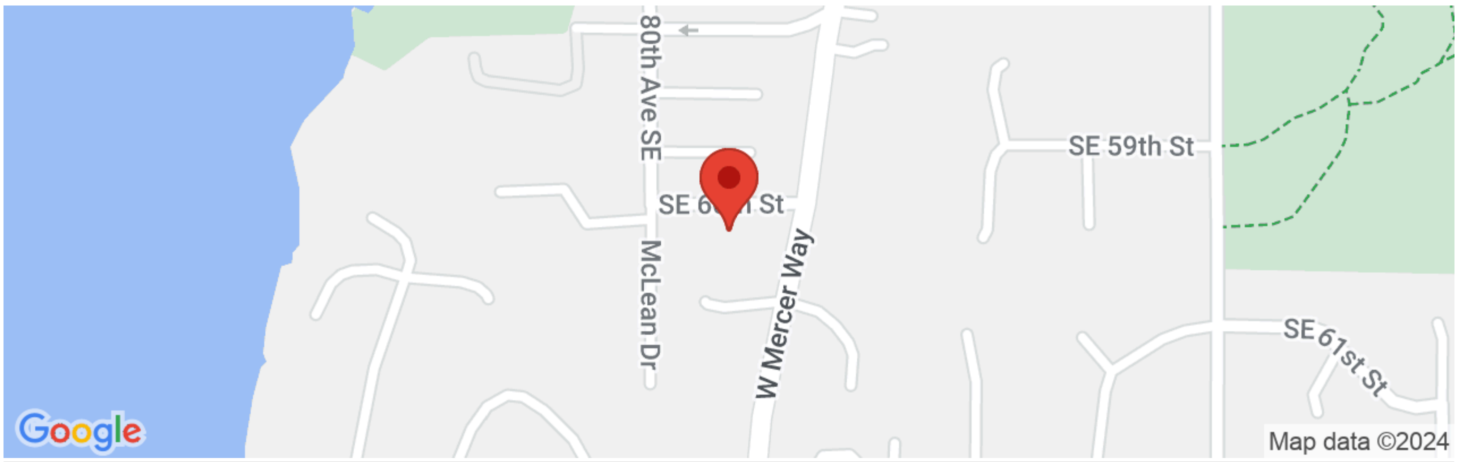
April 27, 2024
Joseph M. Lucia, PE

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

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

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

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 C_t & x
C_{t1} =	0.016	ASCE 7-10, Table 12.8-2 Values of Approximate Period Parameters C_t & x b (Concrete)
x =	0.75	ASCE 7-10, Table 12.8-2 Values of Approximate Period Parameters C_t & x
x_1 =	0.90	ASCE 7-10, Table 12.8-2 Values of Approximate Period Parameters C_t & x (Concrete)

Roof	Roof Area:	
Roof Area:	1910.00	$feet^2$
Deck Area:	0.00	$feet^2$
Diaphragm Area:	1910.00	$feet^2$
Floor	**Floor Area:**	
Floor Area	1690.00	$feet^2$
Diaphragm Area:	1690.00	$feet^2$
Second Floor	**Floor Area:**	
Floor Area	4960.00	$feet^2$
Diaphragm Area:	4960.00	$feet^2$
Bottom Floor	**Floor Area:**	
Floor Area:	6,790.00	$feet^2$
Diaphragm Area:	6790.00	$feet^2$
Basement Floor	**Floor Area:**	
Floor Area:	0.00	$feet^2$
Diaphragm Area:	0.00	$feet^2$
	Floor Area:	
Floor Area:	0.00	$feet^2$
Diaphragm Area:	0.00	$feet^2$

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

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 & SEISMIC LOADING:

WIND LOADING:

I	=	1.00	Importance Factor	Roof Level:			Ave. Elev. =	29.98	q_z	=	24.21	psf *
V	=	110.00 MPH	Basic Wind Speed	Thrid Floor	8.00	feet	Elev. =	21.98	q_z	=	24.21	psf *
K_{ZT}	=	1.38	Per City of Seattle	Second Floor	11.44	feet	Elev. =	10.54	q_z	=	24.21	psf *
h_e	=	34.00 FT	Height to Eave	Bottom Floor	10.54	feet	Elev. =	0.00	q_z	=	24.21	psf *
h_r	=	36.00 FT	Height to Ridge	Basement Floor	0.00	feet	Elev. =	0.00	q_z	=	24.21	psf *
B	=	82.67 FT	Building Length		0.00	feet	Elev. =	0.00	q_z	=		psf
L	=	83.25 FT	Building Width	* 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_t & 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_t & 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_t & x
I =	1.00	Importance Factor	x_1 =	0.90	ASCE 7-10, Table 12.8-2 Values of Approximate Period Parameters C_t & x (Concrete)

Seismic Base Shear:

For Wood Framed Story Levels:

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

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

Where: $T = T_a = C_t h_n^x$ (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 $\le$ 0.000 or 0.000 $\ge$ 1.00

For Concrete Story Levels:

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

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

Where: $T = T_a = C_t h_n^x$ (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 $\le$ 0.000 or 0.000 $\ge$ 1.00

Vertical Distribution of Shear forces

$F_x = C_{vx}V$ Where:

$C_{vx} = w_x h_x^k / \text{sum } w_i h_i^k$ 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)	$w_x h_x^k$	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.)

		Wind:		Seismic:		DesignShear Load:		
Vy Design Shear in the "Y" Direction								
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" enges & 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 Informatio
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} x FT_{max}	=	3.59 kips / Wall Length =	211		P1-8-6
SWX	202	V_{max}	=	F_{max} x FT_{max}	=	3.59 kips / Wall Length =	211		P1-8-6
SWX	203	V_{max}	=	F_{max} x FT_{max}	=	3.46 kips / Wall Length =	289		P1-8-6
SWX	204	V_{max}	=	F_{max} x FT_{max}	=	3.53 kips / Wall Length =	235		P1-8-6
SWX	205	V_{max}	=	F_{max} x FT_{max}	=	3.53 kips / Wall Length =	235		P1-8-6
SWX	206	V_{max}	=	F_{max} x 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} x FT_{max}	=	7.37 kips / Wall Length =	614		P1-8-3
SWY	201A	V_{max}	=	F_{max} x FT_{max}	=	2.46 kips / Wall Length =	982		P2-8-4
SWY	203	V_{max}	=	F_{max} x FT_{max}	=	10.44 kips / Wall Length =	696		P1-8-3
SWY	204	V_{max}	=	F_{max} x FT_{max}	=	4.91 kips / Wall Length =	1637		P2-8-2
SWY	205	V_{max}	=	F_{max} x FT_{max}	=	7.98 kips / Wall Length =	639		P1-8-3
SWY	206	V_{max}	=	F_{max} x 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
Ib/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:

$Fs_{Req'd}$ = $W(L_2) / 20(Ib/Q)$ = **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.

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.

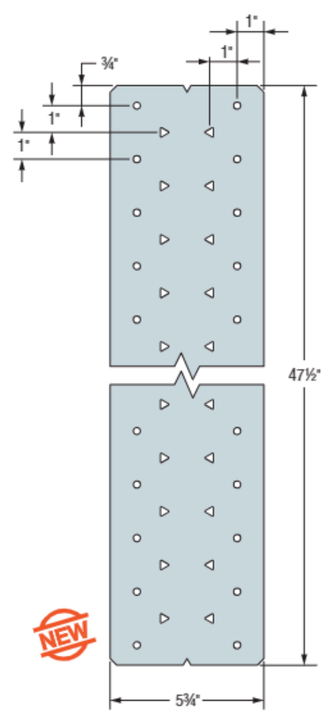
MSTC — A high-capacity strap that utilizes a staggered nail pattern to help minimize wood splitting. Nail slots have been countersunk to provide a lower nail head profile.

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.

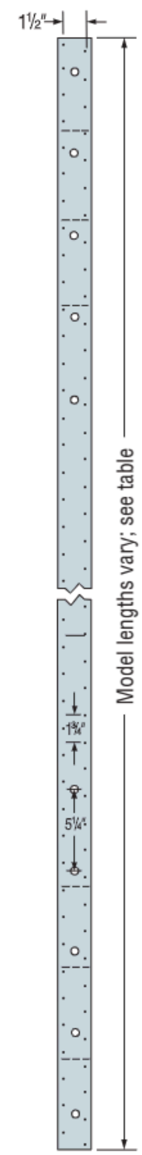
MST — A high capacity strap that uses nails and is suitable for a variety of CLT applications. Suitable for double 2x member connections or greater, and also for a variety of CLT applications.

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

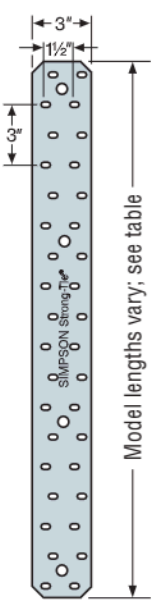
Installation: Use all specified fasteners. See General Notes.



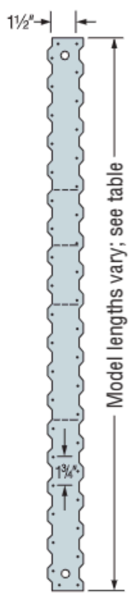
MDCST48



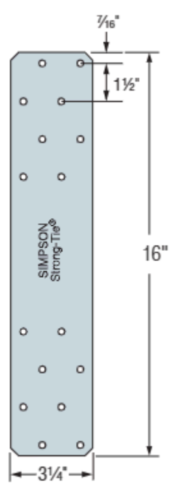
MST



MSTC



ST



HRS416Z

SIMPSON Strong-Tie

Value-Engineered Design Options for CLT

Factored Tensile Resistance Table

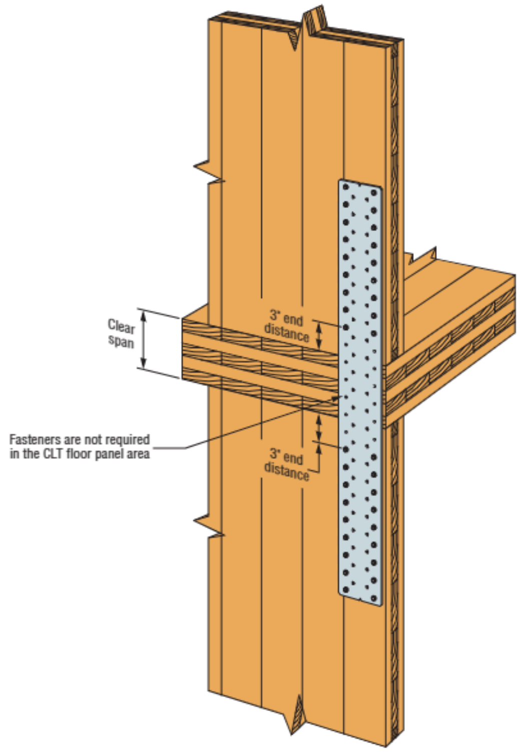
Model No.	Ga.	Dimensions (in.)		Fasteners (Total)	Factored Tensile Resistance							
					D.Fir-L				S-P-F			
					K_D = 1.00		K_D = 1.15		K_D = 1.00		K_D = 1.15	
		W	L		lb.	kN	lb.	kN	lb.	kN	lb.	kN
ST6215	16	2 1/16	16 5/16	(16) 8d x 2½"	1365	6.07	1565	6.98	1265	5.64	1455	6.49
ST6224		2 1/16	23 5/16	(24) 8d x 2½"	2245	10.00	2580	11.50	2065	9.20	2375	10.58
MSTC28		3	28¼	(32) 10d x 2½"	3850	17.14	4425	19.71	3430	15.28	3945	17.57
MSTC40		3	40¼	(48) 10d x 2½"	5770	25.71	6640	29.57	5145	22.92	5915	26.36
MSTC52		3	52¼	(54) 10d x 2½"	6495	28.93	6940	30.87	5790	25.79	6655	29.65
MSTC66	14	3	65¾	(66) 10d x 2½"	8085	36.02	8570	38.11	7075	31.52	8135	36.24
MSTC78		3	77¾	(66) 10d x 2½"	8085	36.02	8570	38.11	7075	31.52	8135	36.24
ST6236		2 1/16	33 13/16	(36) 8d x 2½"	3550	15.82	4085	18.19	3105	13.84	3575	15.91
MDCST48		5¾	47½	(44) ¼" x 3" SDS	12350	55.00	14200	63.26	11320	50.43	13020	58.00
MDCST48 (Double/ Overlapped)		5¾	47½	(88) ¼" x 3" SDS	34670	154.42	35505	157.94	31110	138.57	35505	157.94
HRS416Z	12	3¼	16	(16) ¼" x 1½" SDS	3095	13.80	3560	15.87	2920	13.00	3355	14.95
MST27		2 1/16	27	(26) 8d x 2½"	2555	11.38	2940	13.09	2235	9.97	2575	11.46
MST37		2 1/16	37½	(38) 8d x 2½"	3735	16.63	4295	19.13	3270	14.57	3760	16.75
MST48		2 1/16	48	(50) 8d x 2½"	4915	21.90	5655	25.18	4305	19.17	4950	22.05
MST60	10	2 1/16	60	(64) 8d x 2½"	6290	28.03	7235	32.23	5510	24.54	6335	28.22
MST72		2 1/16	72	(78) 8d x 2½"	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 2½" long, 10d = 0.148" x 2½" long. Screws: ¼" x 1½" long SDS, ¼" x 3" long SDS. All nails shown are common wire.

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

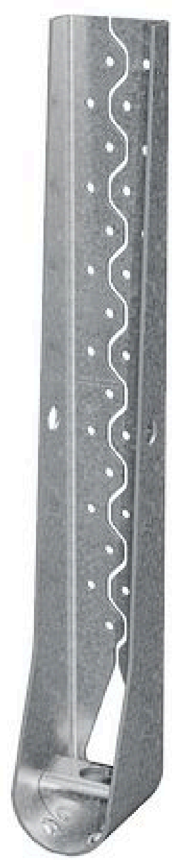
(other strap options similar; other CLT ply combinations similar)

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.

HDU Holdown

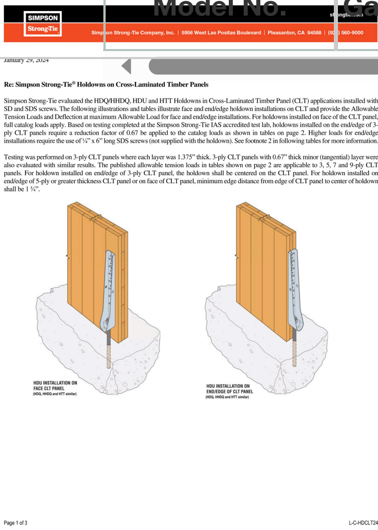
Select Media



HDU11-SDS2.5

Model No.	Ga.	W (in.)	H (in.)	B (in.)	CL (in.)	SO (in.)	Coating/Material

Load Tables



Simpson Strong-Tie Company, Inc. | 5956 West Las Positas Boulevard | Pleasanton, CA 94588 | (9[illegible]) 560-9000

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 end/edge holdown installations on CLT and provide the Allowable Tension Loads and Deflection at maximum Allowable Load for face and end/edge 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 end/edge 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 end/edge 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 end/edge of 3-ply CLT panel, the holdown shall be centered on the CLT panel. For holdown installed on end/edge 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 ¾".

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.
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

Load Tables

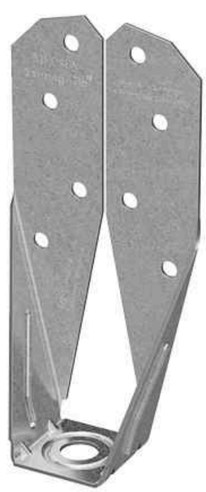
Product Series	Compliance/Certification	Report/Approval
HDU	International Building Code International Residential Code City of Los Angeles Building Code City of Los Angeles Residential Code	ESR-2330
	Florida Building Code	FL10441

Model No.	Ga.	W (in.)	H (in.)	B (in.)	CL (in.)	SO (in.)	Coating/Material

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

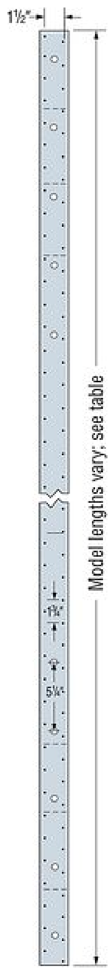


DTT™
Deck Tension Tie

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

...high-wind-induced damage to existing ...s in Hawaii.

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

tnotes 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 2½	4	½	3,700	2,165	3,210	2,000
MST37		2 1/16	37½	(42) 0.162 x 2½	6	½	5,070	3,030	4,495	2,800
MST48		2 1/16	48	(50) 0.162 x 2½	8	½	5,310	3,675	5,190	3,395
MST60	10	2 1/16	60	(68) 0.162 x 2½	10	½	6,730	4,490	6,475	4,150
MST72		2 1/16	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	(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.

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:
S_S = 1.466
S_1 = 0.508 SITE CLASS = D
S_{ds} = 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
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Permanent Soldier Pile
& Timber Lagging
Retaining Wall

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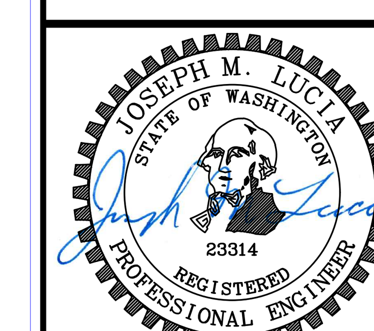


04-21-24

Number	Date	By	Description
2	04-21-24	JML	

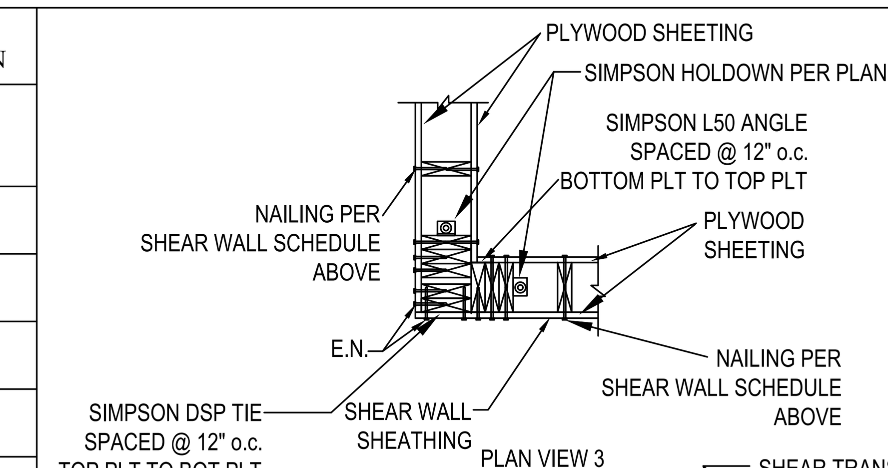
SHEET
S-5.0

SHEAR WALL SCHEDULE

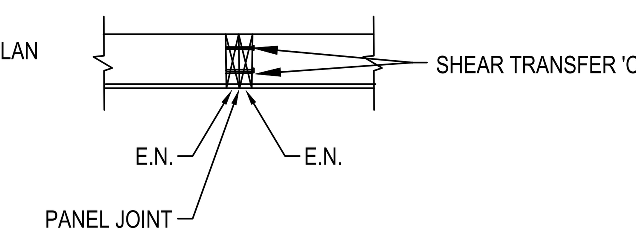
MARK	SHEATHING	NAILING (5)		LUMBER			SHEAR TRANSFER					1.4 INCREASE FOR WIND
		EDGE (E.N.)	FIELD	ALLOWABLE SHEAR	SILL PL	TOP PL'S	"A" SILL PL TO CONC.	"B" BLKG TO TOP PL.	"C" SILL PL RIM/JST/BLKG (F.N.)	"D" SHEAR WALL INTERSECTIONS	CAPACITY	CAPACITY
P1-8-6	3/8" APA RATED SHEATHING, ONE SIDE	8d@6"	8d@ 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

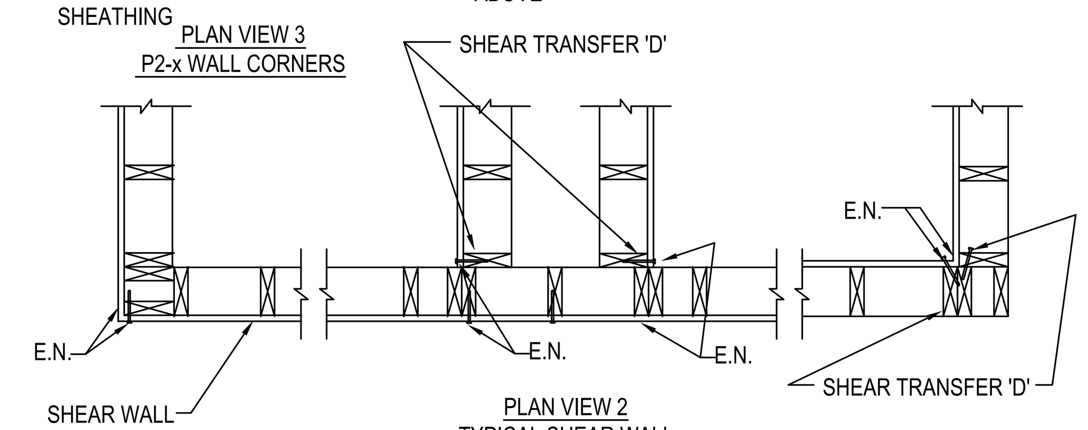
DIA. #	DIAPHRAGM SHEATHING	NAILING (INCHES o.c.) 15/32" SHEATHING W/ 10d COMMON			
		EDGE (E.N.)	FIELD	ALLOWABLE SHEAR (KLF)	NOTES
	UNBLOCKED, OTHER		6	0.20	2x
	UNBLOCKED CASE#1		6	0.28	2x
1	BLOCKED	6	6	0.32	3x
2	BLOCKED	4	6	0.43	3x
3	BLOCKED	2.5	4	0.67	3x
4	BLOCKED	2	3	0.73	3x
5	BLOCKED	2	3	0.82	3x



PLAN VIEW 3
P2-x WALL CORNERS



PLAN VIEW I
OPTIONAL (2)2x IN LIEU OF 3x



PLAN VIEW 2
TYPICAL SHEAR WALL INTERSECTIONS

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

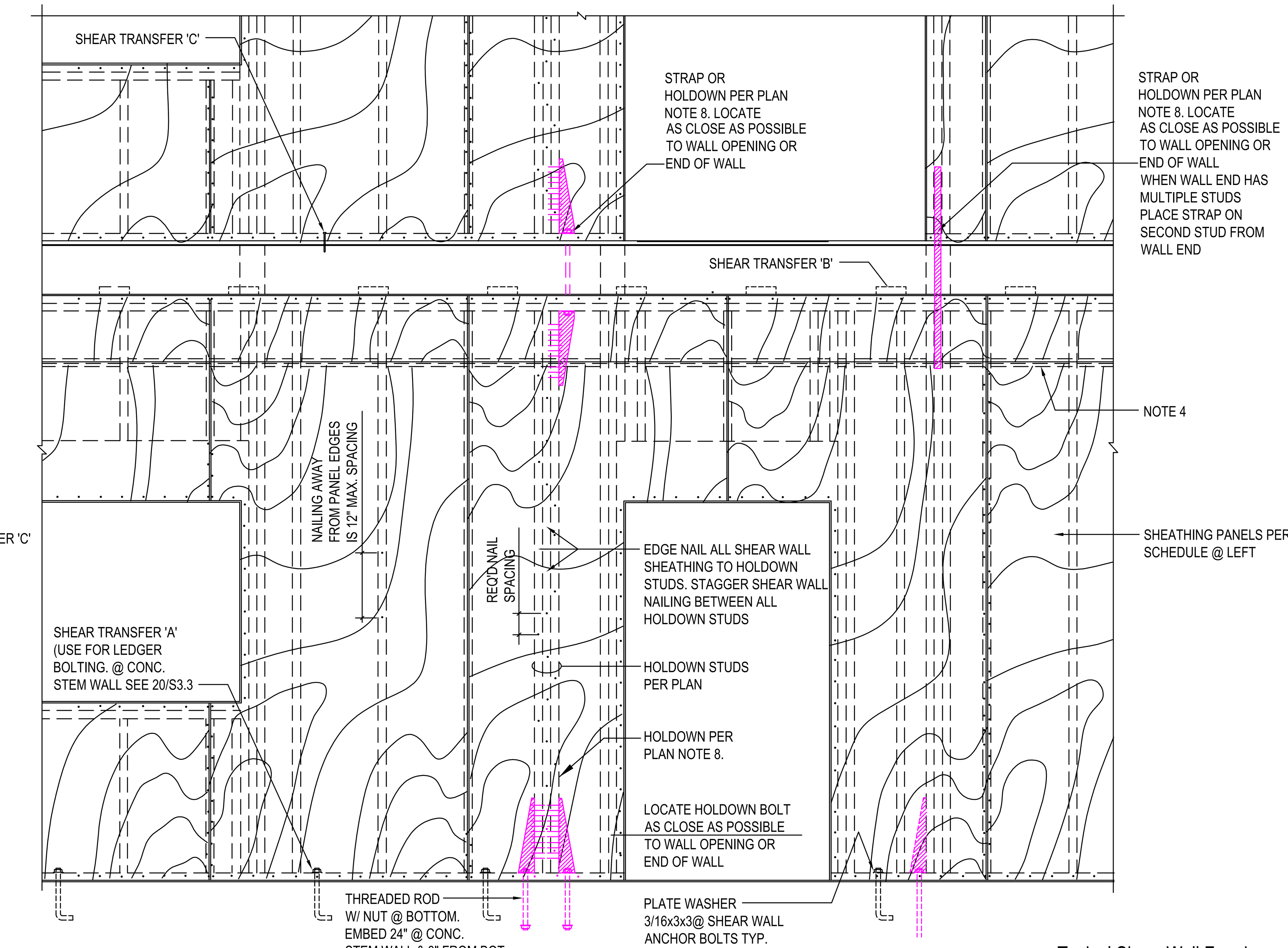
NOMINAL DEPTH	END ATTACHMENT
4	(4)16d
6	(6)16d
8	(8)16d
10	(10)16d
12	(12)16d
14	(14)16d
16	(16)16d
18	(18)16d

ROUGH WINDOW SILL

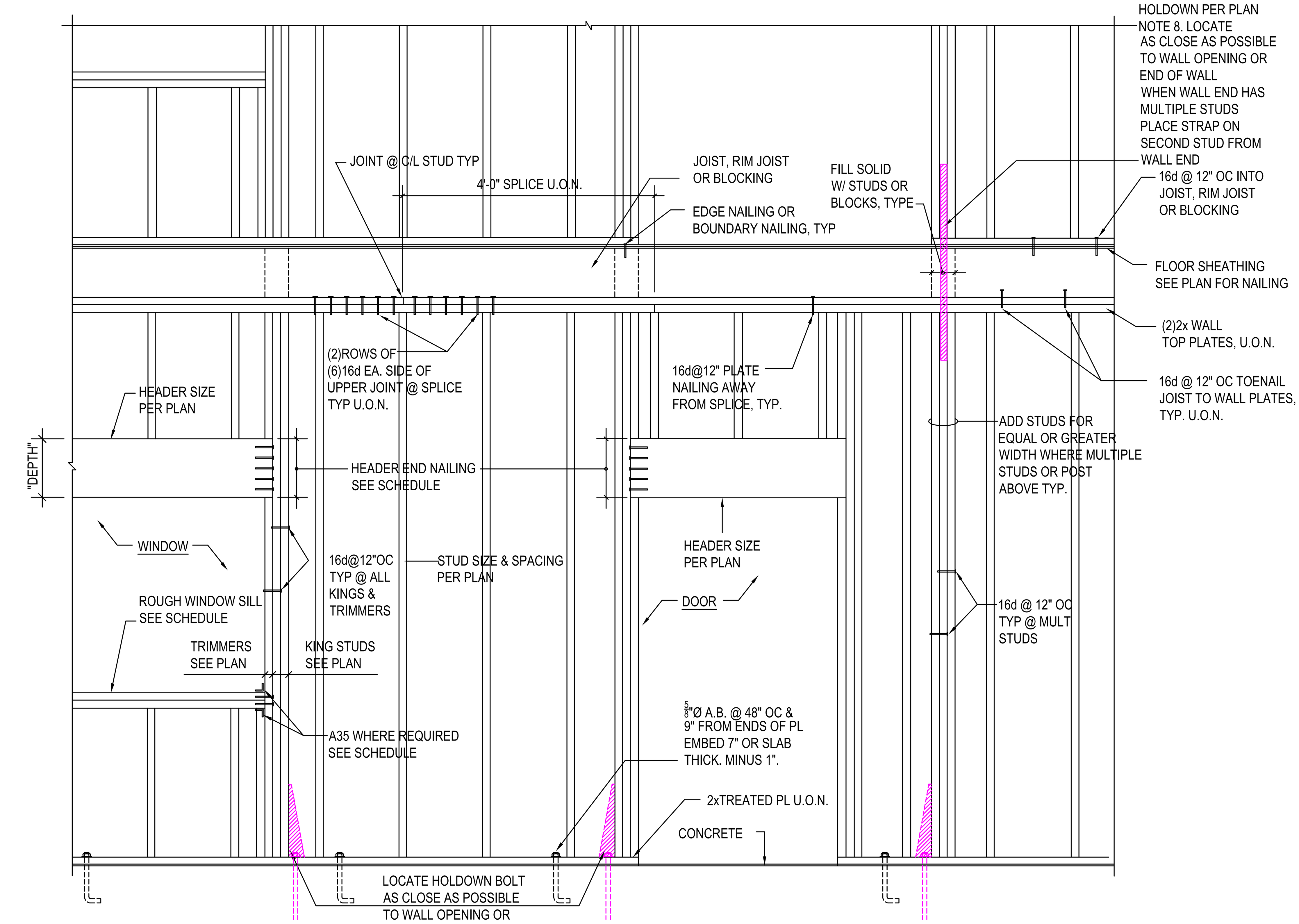
HORIZ ROUGH OPENING	NUMBER OF SILLS REQUIRED	END ATTACHMENT	REF.
0 TO 6'	1	(2)16d END NAIL	20/S6.1
> 6'	2	(2)16d END NAIL, +A35 EA END @ EA SILL	20/S6.1

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.

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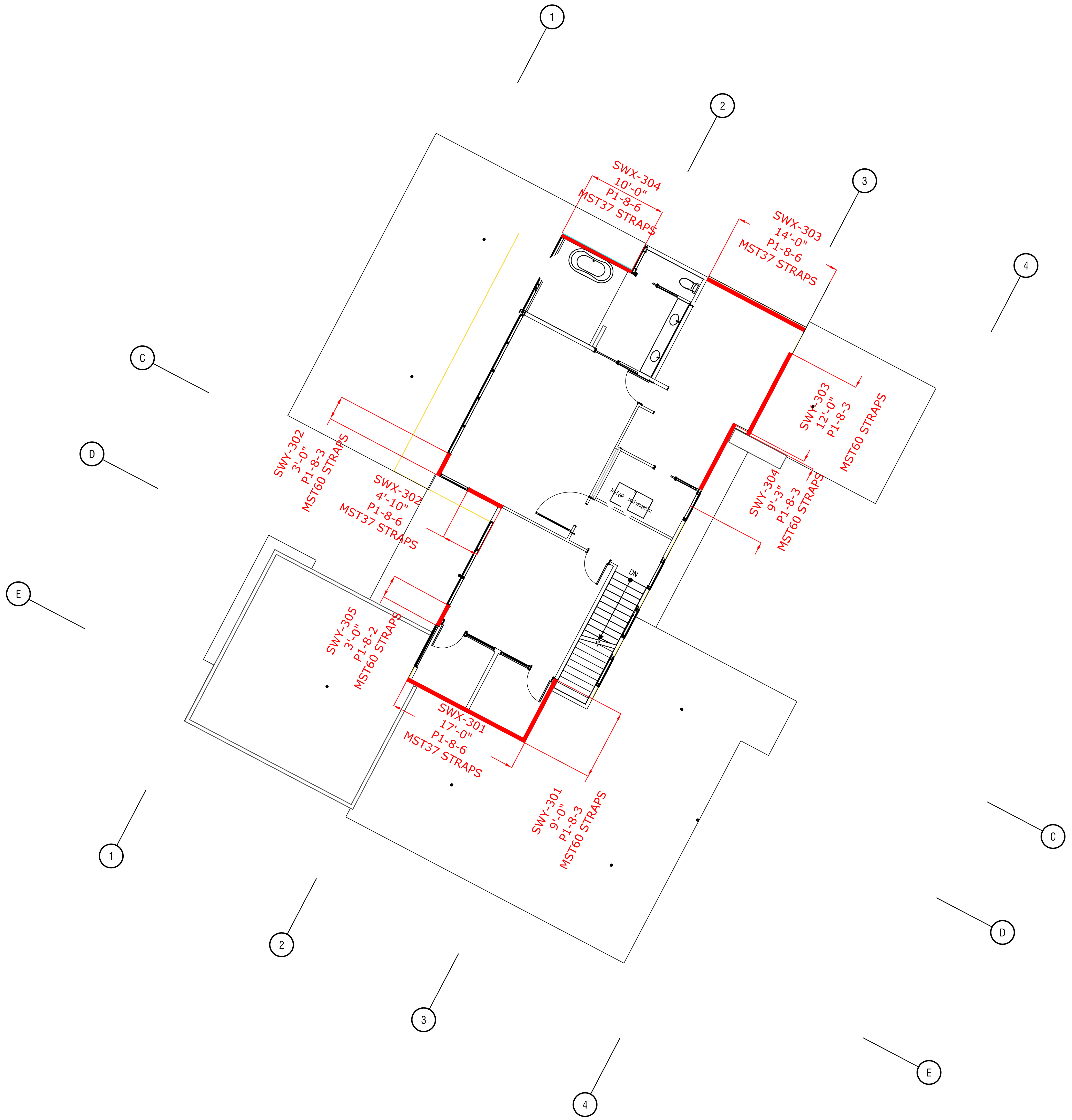
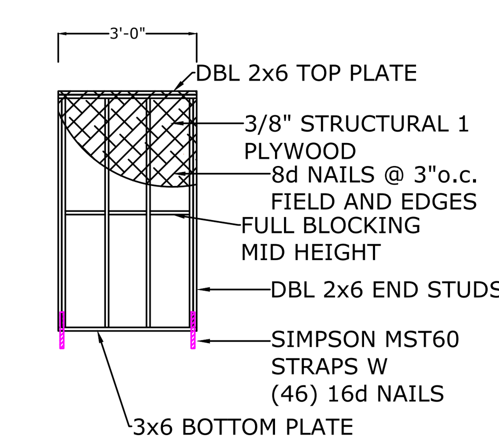
JOSEPH M. LUCIA
STATE OF WASHINGTON
23314
REGISTERED PROFESSIONAL ENGINEER
04-27-24

Number	Date	By	Description
3	04-27-24	JML	

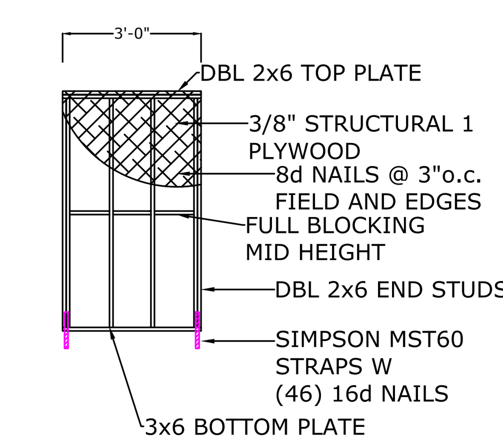
SHEET
S-6.0



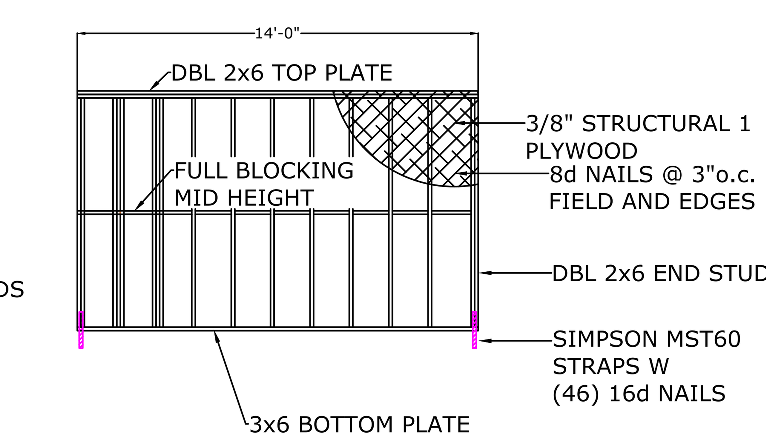
SECOND FLOOR LEVEL - SHEAR WALLS



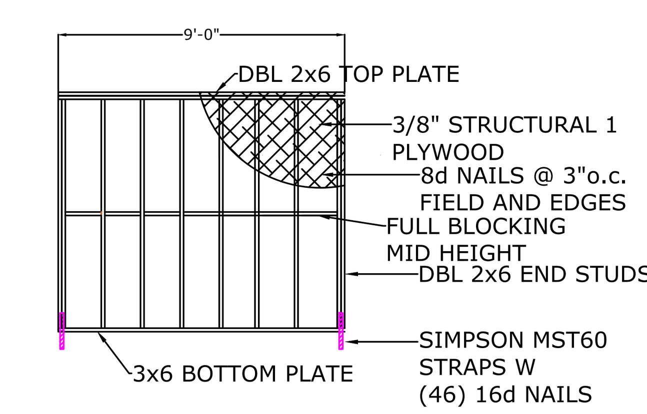
SWY-302
P1-8-6



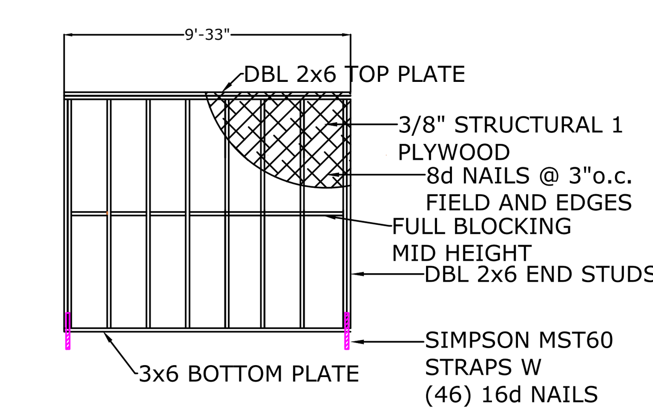
SWY-305
P1-8-6



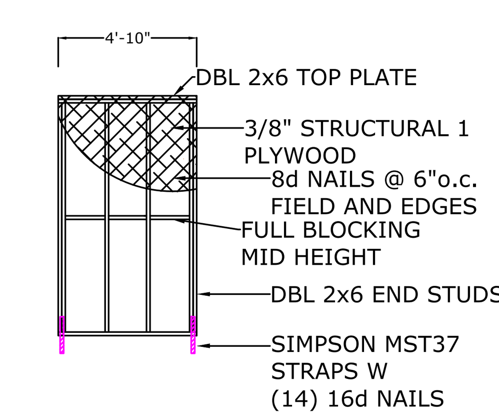
SWY-303
P1-8-6



SWY-301
P1-8-6



SWY-304
P1-8-6



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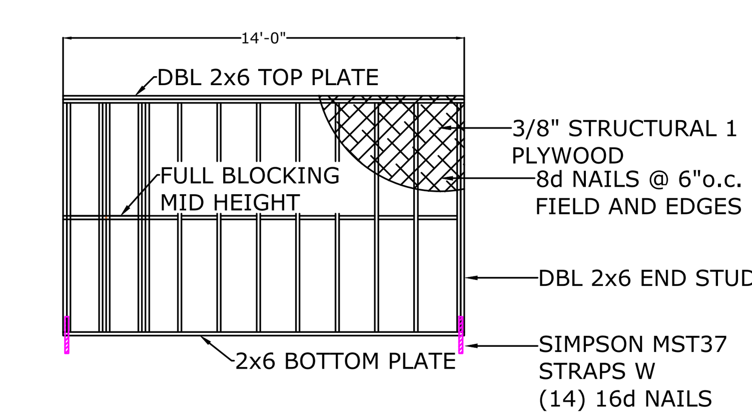
P1-8-6



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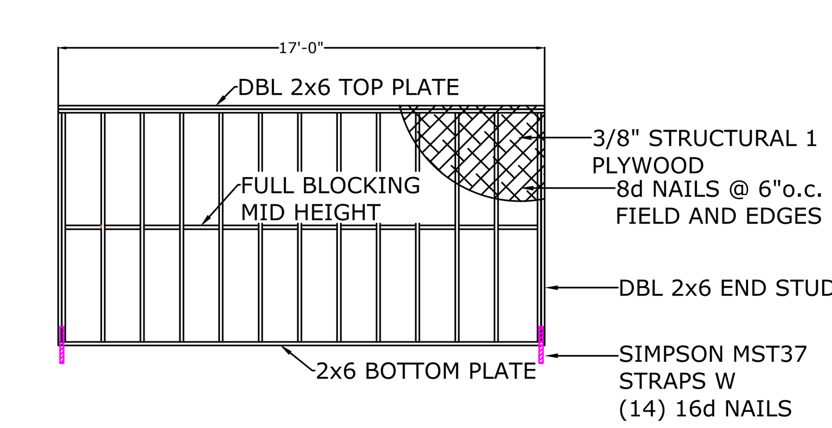
P1-8-6



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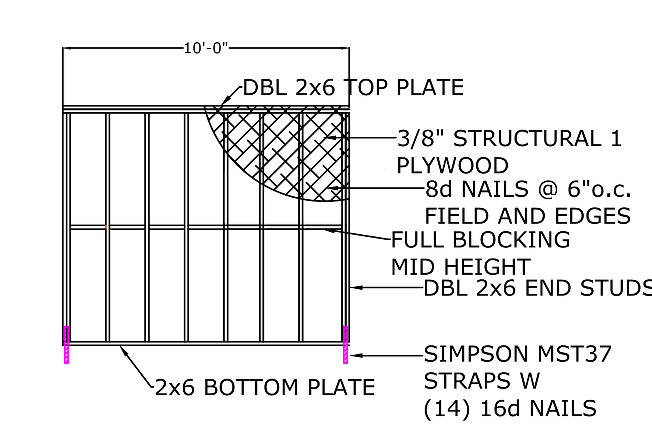
P1-8-6



SWX-303
P1-8-6

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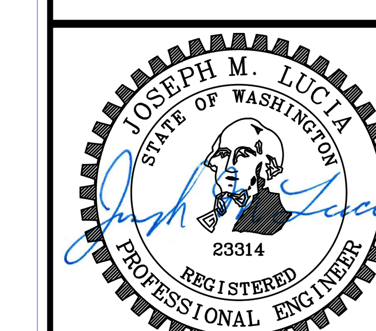

04-27-24

Number	Date	By	Description
32	04-27-24	JML	

SHEET
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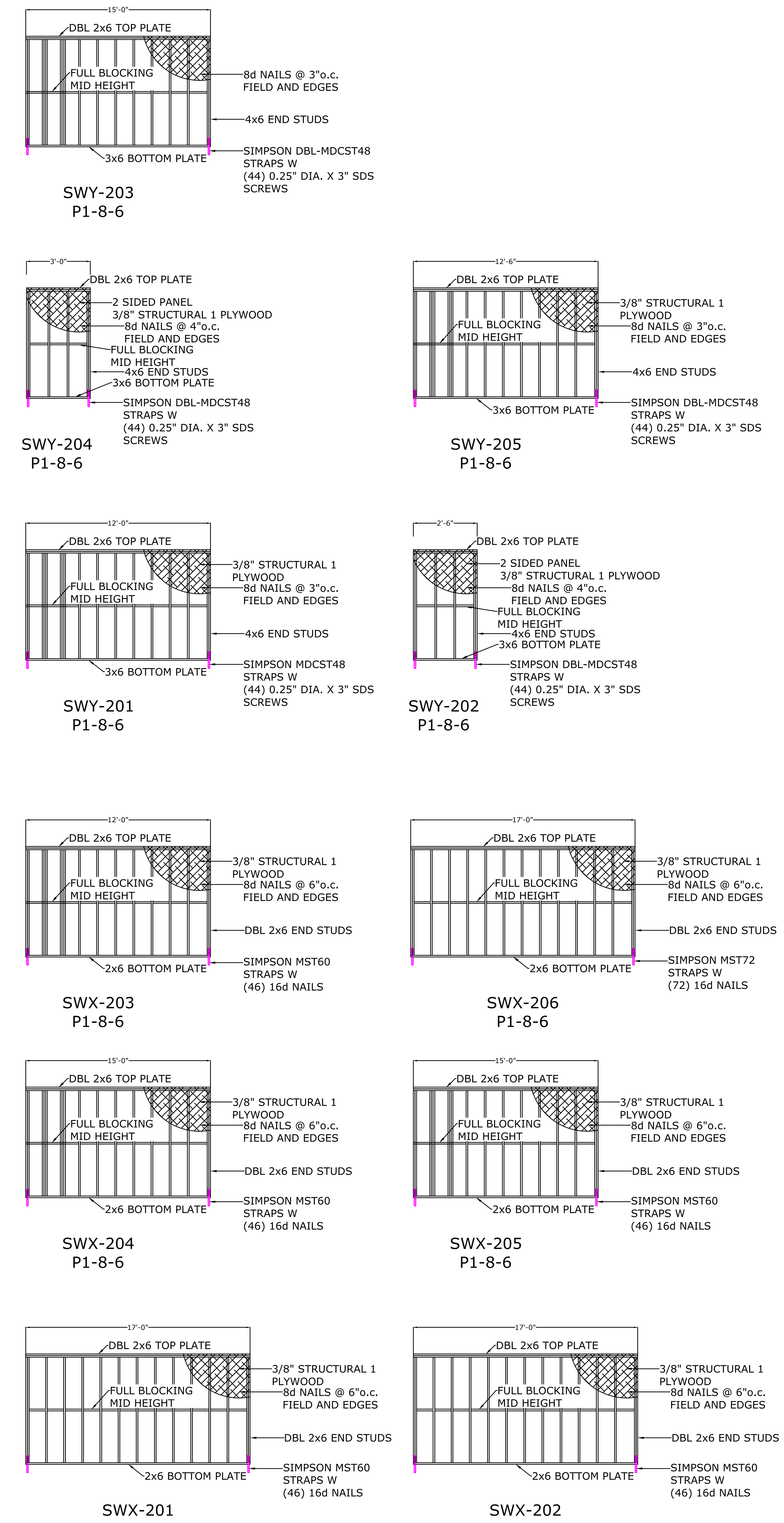
FIRST FLOOR LEVEL - SHEAR WALLS



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04-27-24

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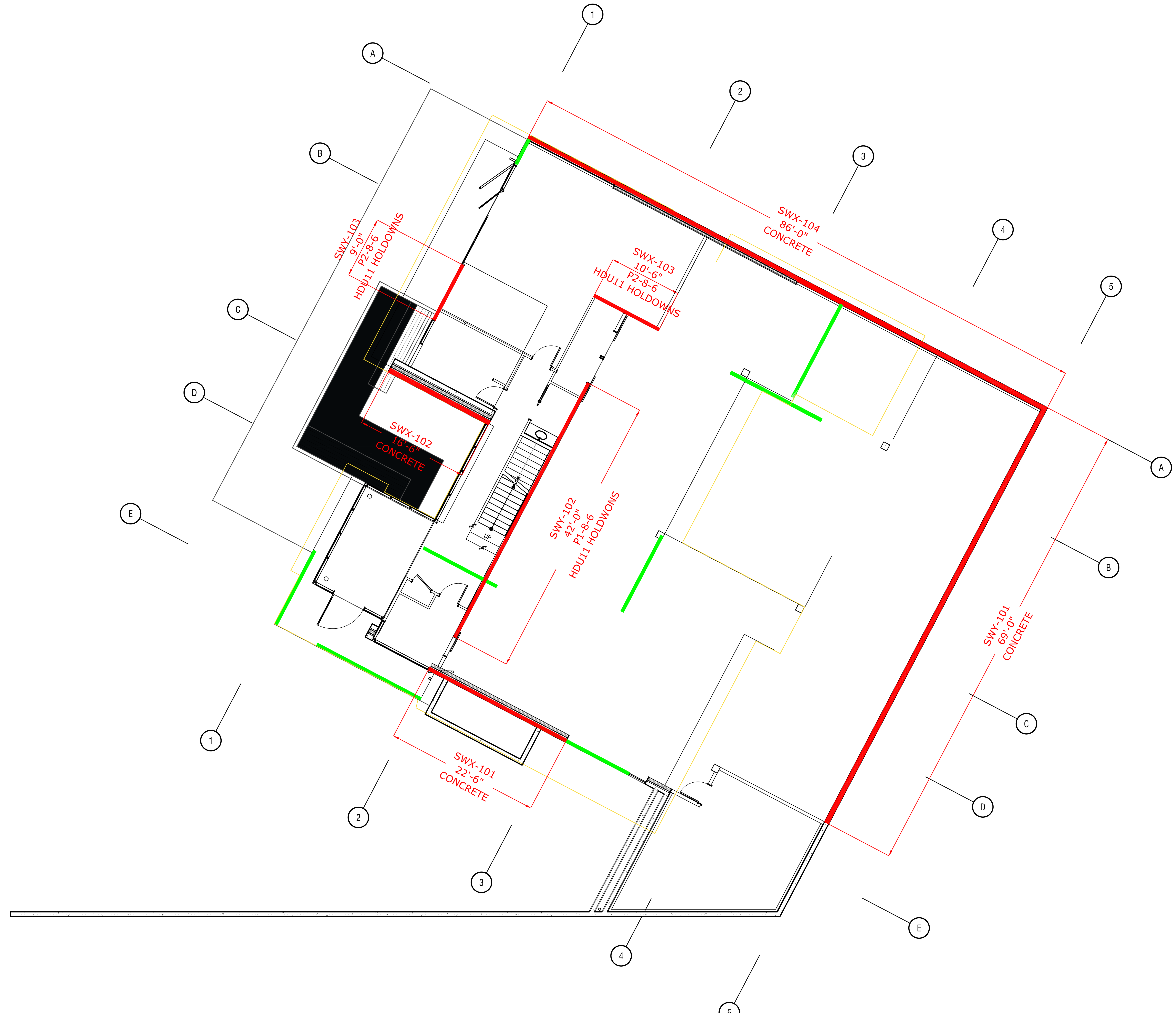
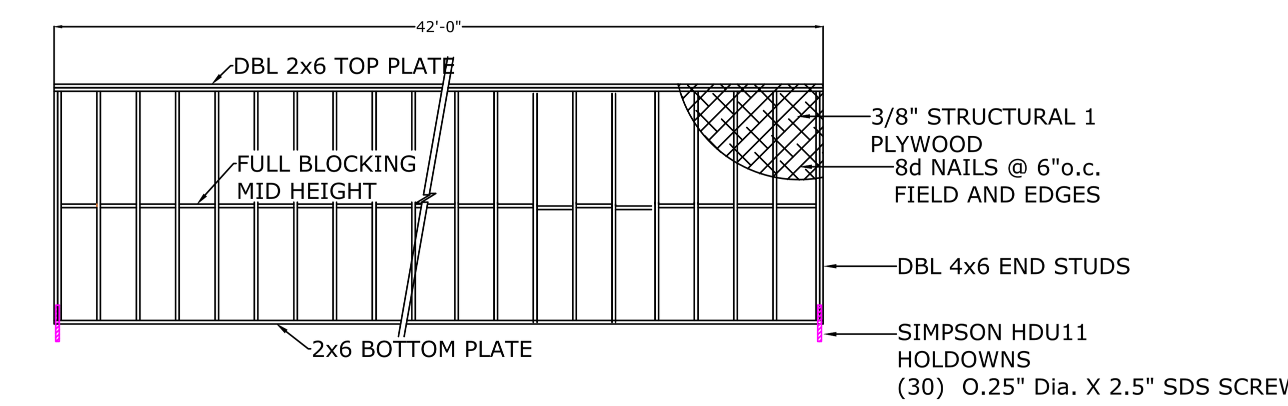
3	04-27-24	JML	

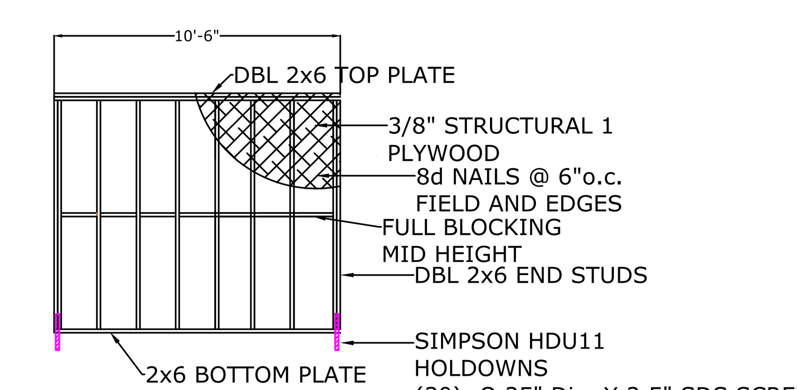
SHEET
S-8.0



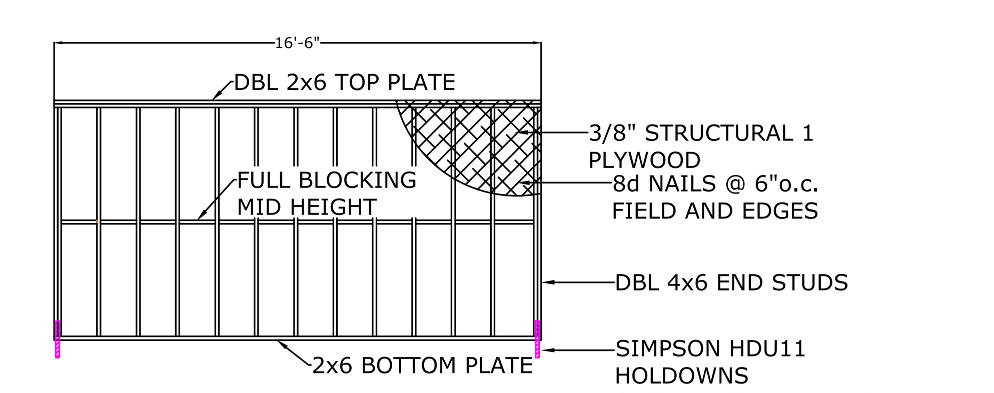
GARAGE-BASEMENT LEVEL - SHEAR WALLS



SWY-102
P1-8-6



SWX-103
P1-8-6



SWX-102
P1-8-6

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SHEET
S-9.0

FOUNDATION & STEEL STRUCTURE MEMBER DESIGN

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.

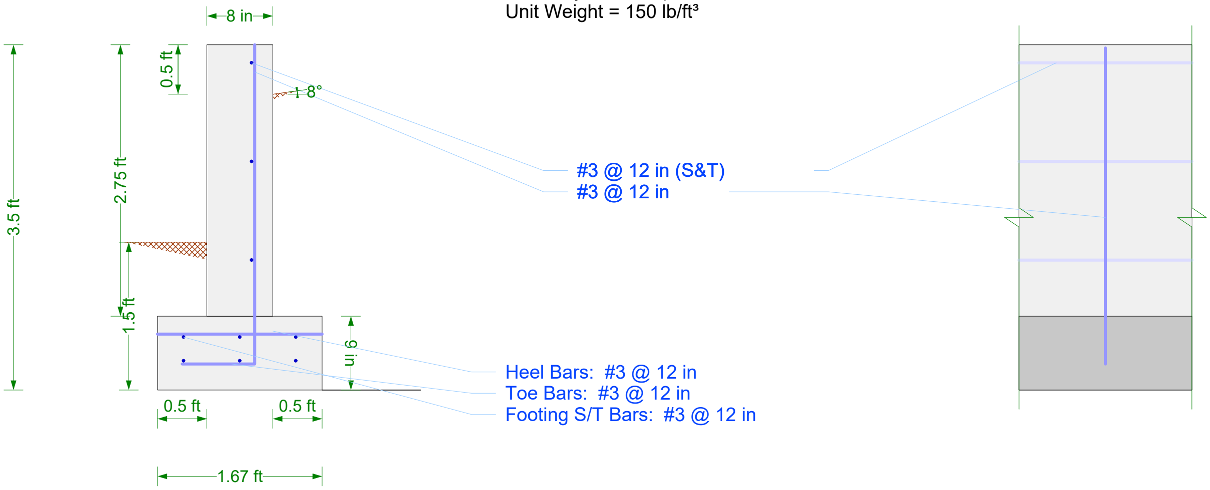
Drainage should be provided behind retaining walls such that hydrostatic pressures do not develop. If drainage is not provided, hydrostatic pressures should be included in the wall design. A perforated drainpipe should be placed along the base of the wall and connected to an approved discharge location.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

Design Detail

Concrete f'c = 3000 psi
Rebar Fy = 60000 psi
Unit Weight = 150 lb/ft³



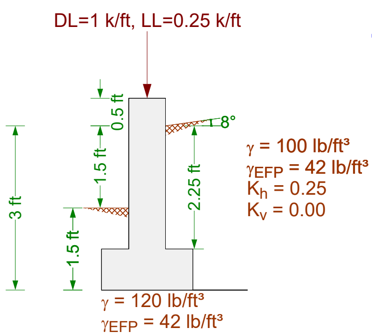
Check Summary

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

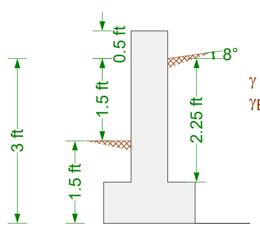
Load Combination	Stem Actual steel (in²/in)	Heel M-applied (ft·k/ft)	Heel M-allow (ft·k/ft)	Heel V-applied (k/ft)	Heel V-allow (k/ft)	Toe M-applied (ft·k/ft)	Toe M-allow (ft·k/ft)	Toe V-applied (k/ft)	Toe V-allow (k/ft)
1.2D + 1.6L + 1.6H	0.01	0.05	3.32	0.21	6.72	0.24	2.82	0.03	5.73
1.2D + 1.6L + 0.9H	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

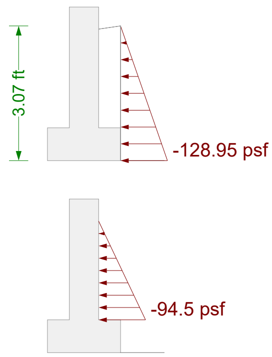
Stability Check Results Summary

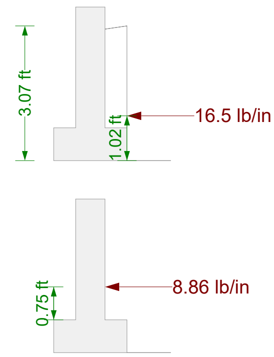
Load Combination	Overturning Moment (ft·k/ft)	Resisting Moment (ft·k/ft)	Overturning F.S.	Overturning F.S. Req'd	Overturning F.S. Req'd Seismic	Sliding Force (lb/in)	Resisting Force (lb/in)	Sliding F.S.
1.0D + 1.0L + 1.0H + 0.7E	0.2	1.42	6.988	1.500	1.200	16.5	59.79	3.625
1.0D + 1.0L + 1.0H	0.2	1.42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

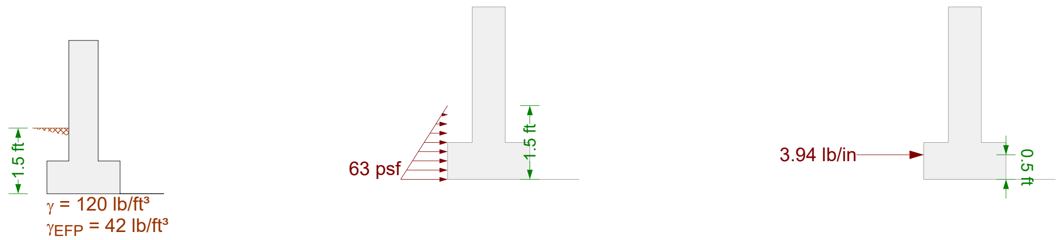
Equivalent Fluid Pressure

$\sigma_h = H\,\gamma_{fluid} = (3.07\text{ ft})(42\text{ lb / ft}^3) = 129\text{ psf}$

Lateral Earth Pressure (stem only)

$\sigma_h = H\,\gamma_{fluid} = (2.25\text{ ft})(42\text{ lb / ft}^3) = 94.5\text{ psf}$

Passive Pressure

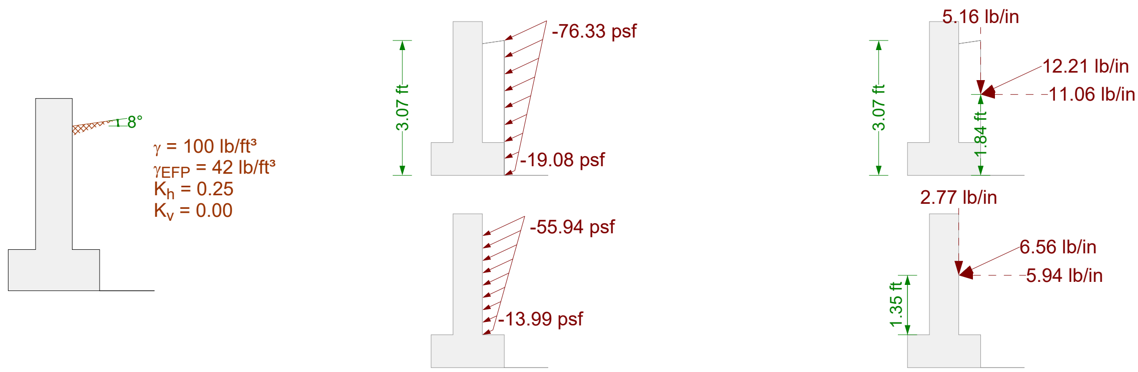


Lateral Earth Pressure

Equivalent Fluid Pressure

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

Seismic Pressure



Seismic Pressure

Dynamic + static force (Mononobe - Okabe equation)

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

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

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

$= 0.6403$

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

Static - only force (Coulomb equation)

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

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

$= 0.3295$

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

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)

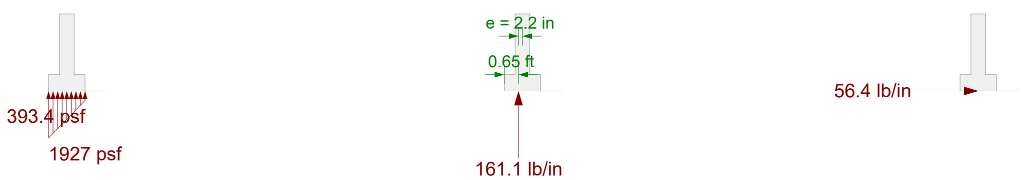
To arrive at the pressure distribution illustrated above (used to determine stem moments) ,
apply inverted triangular pressure plus a uniform portion to bring resultant to 0.6H

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

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

Wall/Soil Weights

Bearing Pressure



Friction

F = μ R = (0.350) (161.1 lb / in) = 56.4 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:

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

Including seismic loads:

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

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	55.86 lb/in
Total:	59.79 lb/in

Without seismic loads:

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

Including seismic loads:

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

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:

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

Including seismic loads:

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

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	48.57 lb/in
Total:	52.5 lb/in

Without seismic loads:

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

Including seismic loads:

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

Bearing Capacity Check

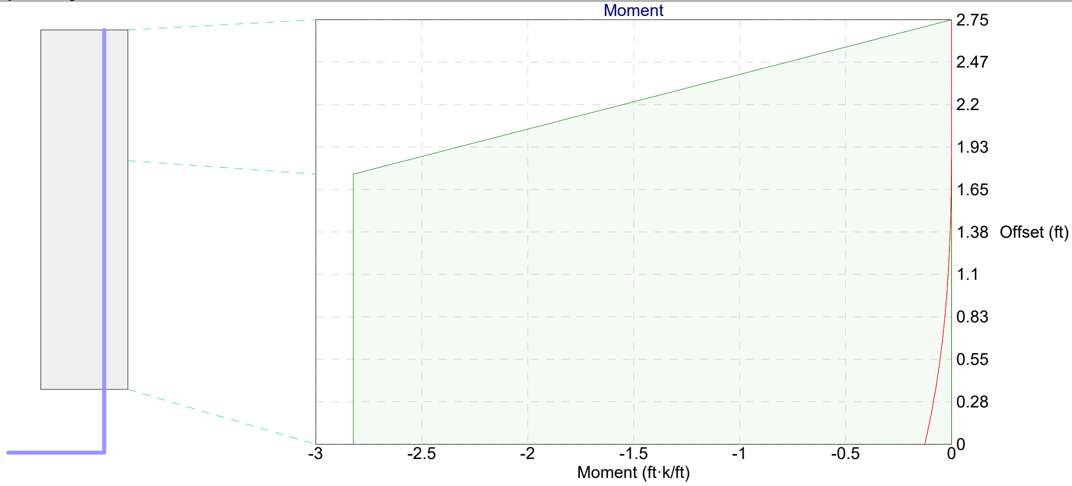
Bearing pressure < allowable (1659 psf < 2500 psf) - OK
Bearing resultant eccentricity < allowable (2.2 in < 10 in) - OK

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)

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

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

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)

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

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

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 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 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)

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

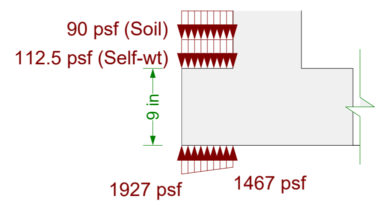
$$l_d = \left(\frac{3.}{40}\,\frac{f_y}{\lambda\sqrt{F'_c}}\,\frac{\psi_t\,\psi_e\,\psi_s}{2.5}\right) d_b = \left[\frac{3.}{40}\,\frac{(60000\text{ psi})}{(1.0)\sqrt{3000\text{ psi}}}\,\frac{(1.0)(1.0)(0.80)}{2.5}\right](0.38\text{ in}) = 9.86\text{ in}$$

12 inch minimum controls

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

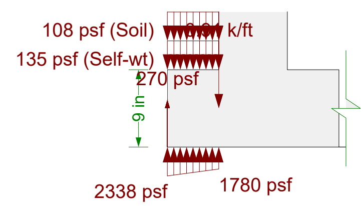
Toe Unfactored Loads

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)

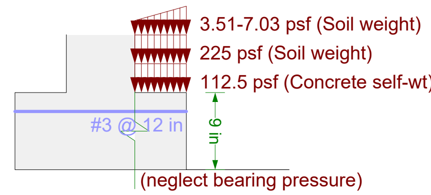
6 inch minimum controls

l_{dh_prov} = 6 in ≥ l_{dh} = 6 in ✓

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

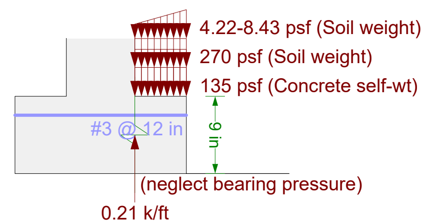
Heel Unfactored Loads

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)

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

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

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)

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

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

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)

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

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)

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

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

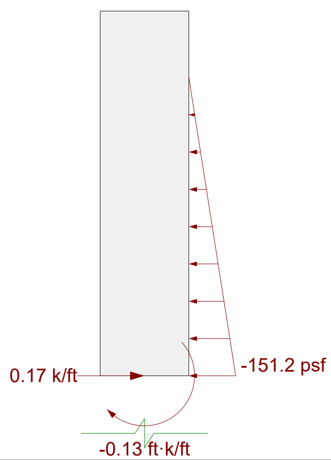
Factoring l_d by the excess reinforcement ratio (0.0155) per 25.4.10: l_d = 0.15 in

12 inch minimum controls

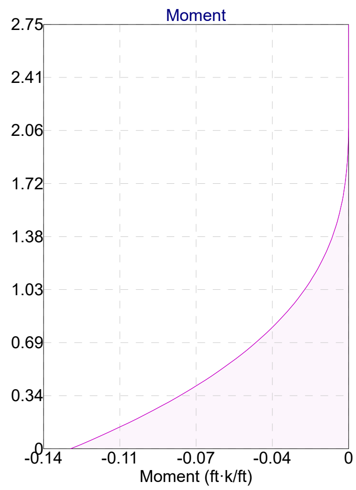
l_{d_prov} = 12 in ≥ l_d = 12 in ✓

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

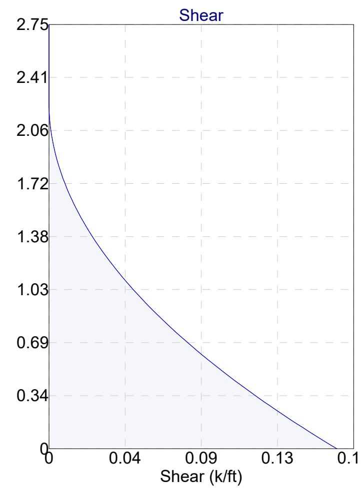
Stem Internal Forces



Stem Internal Forces



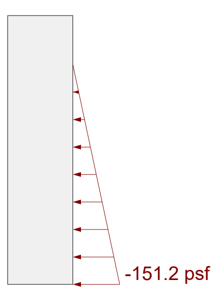
Stem Internal Forces



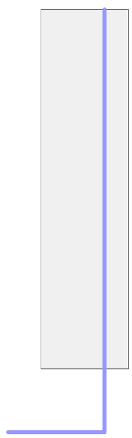
Stem Joint Force Transfer

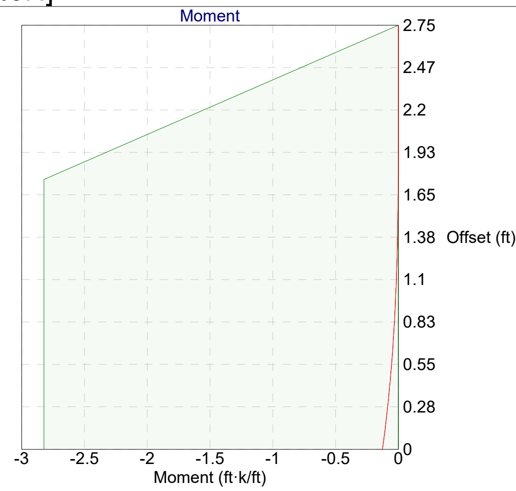
Location	Force
@ stem base	0.17 k/ft

Stem Internal Forces



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





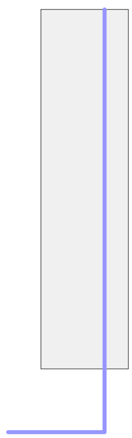
Check (ACI 318-14 11.5.5.1b) @ 0 ft from base

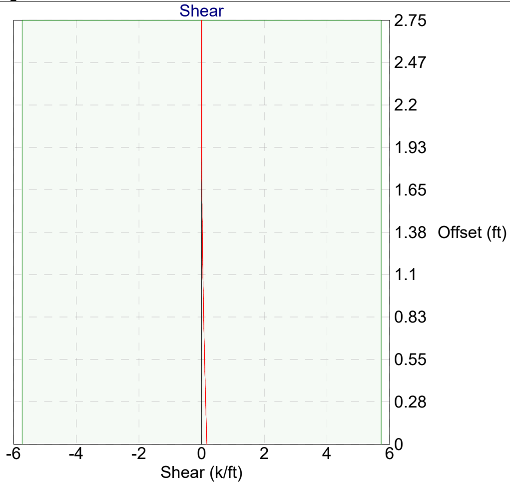
ϕM_n = 2.82 ft·k / ft ≥ M_u = 0.13 ft·k / ft ✓

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 $\geq$ 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]

$\phi M_n = 2.82 \text{ ft·k / ft} \geq (4/3) M_u = [4/3](0.13 \text{ ft·k / ft}) = 0.17 \text{ ft·k / ft}$

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]

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

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]

$\beta_1 = 0.850 \quad (F'_c \leq 4000 \text{ psi})$

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

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

$\varepsilon_t = 0.0657 \geq 0.004$ ✓

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

$\beta_1 = 0.850 \quad (F'_c \leq 4000 \text{ psi})$

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

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

$\varepsilon_t = 0.0657 \geq 0.004$ ✓

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 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)

$$\frac{M_u}{\phi M_n} = \frac{(0.13 \text{ ft·k / ft})}{(2.82 \text{ ft·k / ft})} = 0.0452$$ (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 (0.0452) per 25.4.10: $l_{dh} = 0.26$ in

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

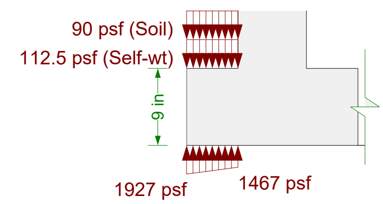
6 inch minimum controls

$l_{dh\_prov} = 6 \text{ in} \geq l_{dh} = 6 \text{ in}$ ✓

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

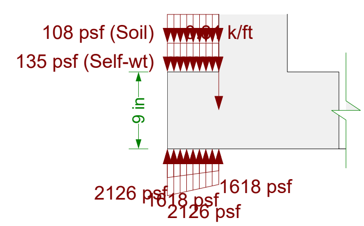
Toe Unfactored Loads

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]

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)

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

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

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)

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

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

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)

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

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

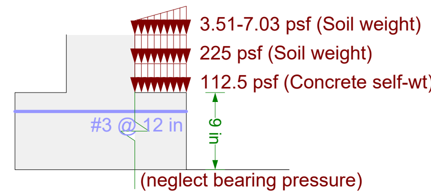
Factoring l_d by the excess reinforcement ratio (0.0155) per 25.4.10: l_d = 0.15 in

12 inch minimum controls

l_{d_prov} = 12 in ≥ l_d = 12 in ✓

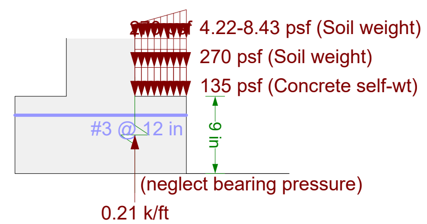
Heel Unfactored Loads

Unfactored Loads



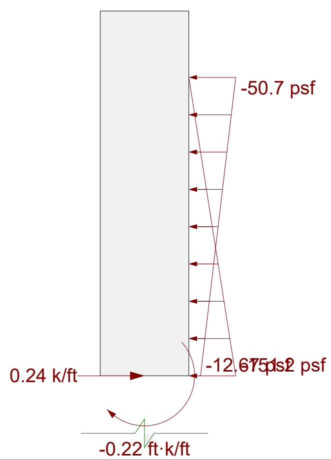
Heel Factored Loads

1.2D + 0.5L + 1.6H + 1.0E

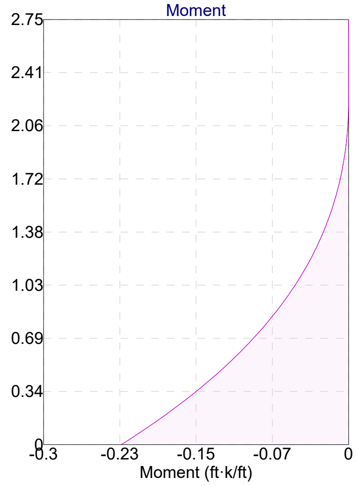


Stem Forces [1.2D + 0.5L + 1.6H + 1.0E]

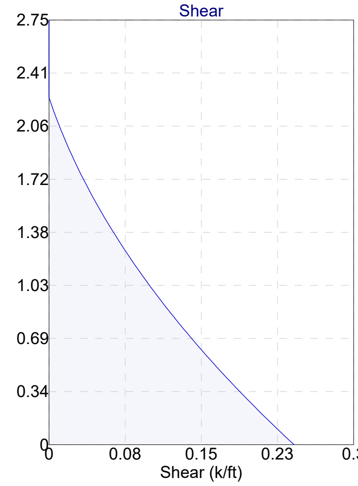
Stem Internal Forces



Stem Internal Forces



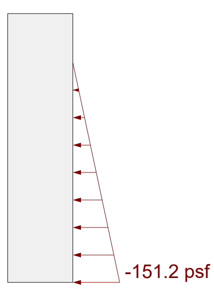
Stem Internal Forces

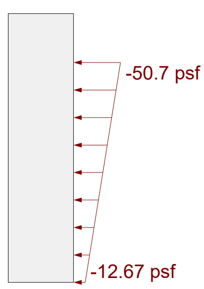


Stem Joint Force Transfer

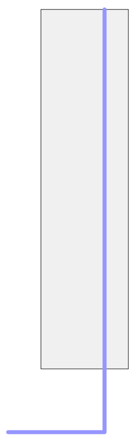
Location	Force
@ stem base	0.24 k/ft

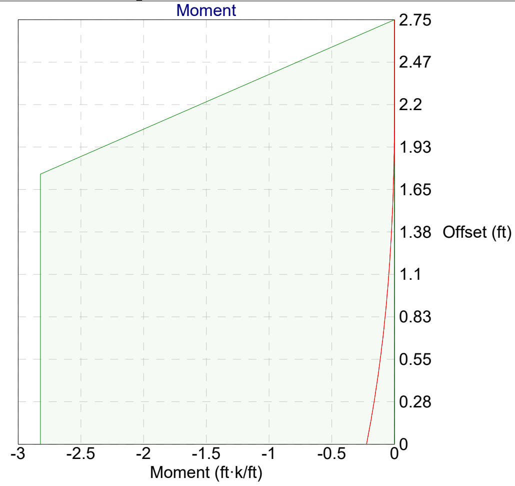
Stem Internal Forces





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





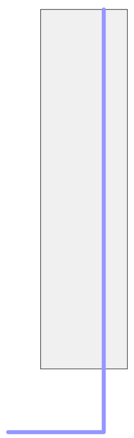
Check (ACI 318-14 11.5.5.1b) @ 0 ft from base

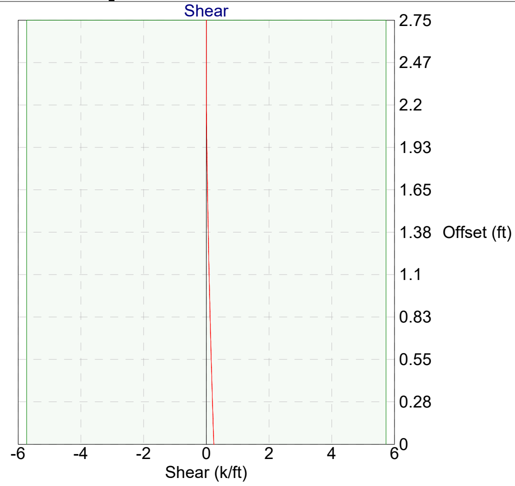
ϕM_n = 2.82 ft·k / ft ≥ M_u = 0.22 ft·k / ft ✓

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]

ϕM_n = 2.82 ft·k / ft ≥ (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]

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]

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

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

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

$\varepsilon_t = 0.0657 \ge 0.004$ ✓

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

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

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

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

$\varepsilon_t = 0.0657 \ge 0.004$ ✓

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 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)

$$\frac{M_u}{\phi M_n} = \frac{(0.22 \text{ ft·k / ft})}{(2.82 \text{ ft·k / ft})} = 0.0793$$ (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 in) = 3.0 (minimum limit, does not control)

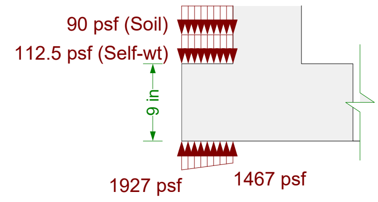
6 inch minimum controls

l_{dh_prov} = 6 in ≥ l_{dh} = 6 in ✓

Toe Checks [1.4D]

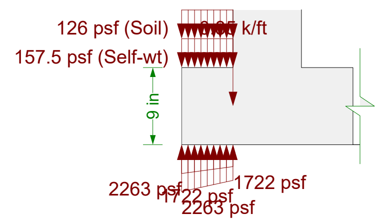
Toe Unfactored Loads

Unfactored Loads



Toe Factored Loads

1.4D



Controlling Moment

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

M_{toe} = 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]

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)

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

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

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)

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

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

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)

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

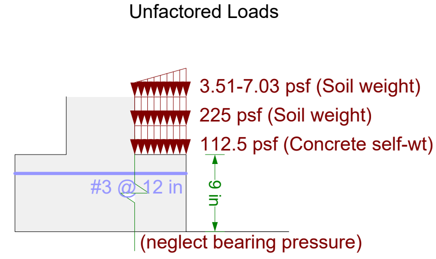
$$l_d = \left(\frac{3.}{40}\,\frac{f_y}{\lambda\sqrt{F'_c}}\,\frac{\psi_t\,\psi_e\,\psi_s}{2.5}\right) d_b = \left[\frac{3.}{40}\,\frac{(60000 \text{ psi})}{(1.0)\sqrt{3000 \text{ psi}}}\,\frac{(1.0)(1.0)(0.80)}{2.5}\right](0.38 \text{ in}) = 9.86 \text{ in}$$

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

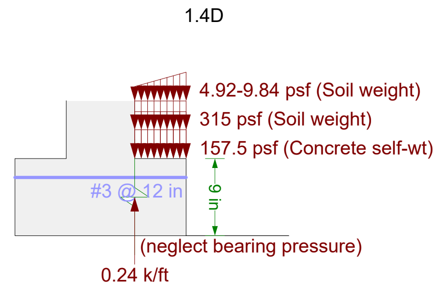
12 inch minimum controls

l_{d_prov} = 12 in ≥ l_d = 12 in ✓

Heel Unfactored Loads

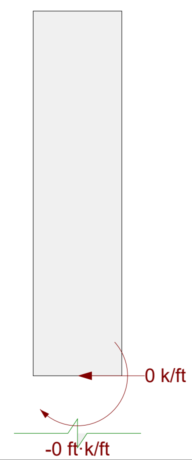


Heel Factored Loads

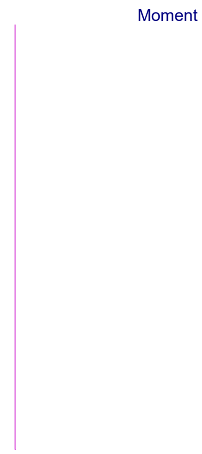


Stem Forces [1.4D]

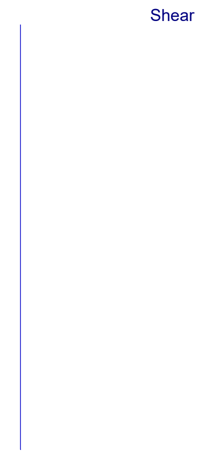
Stem Internal Forces



Stem Internal Forces



Stem Internal Forces

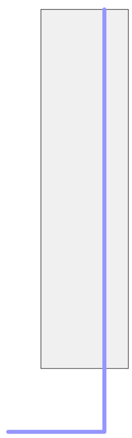


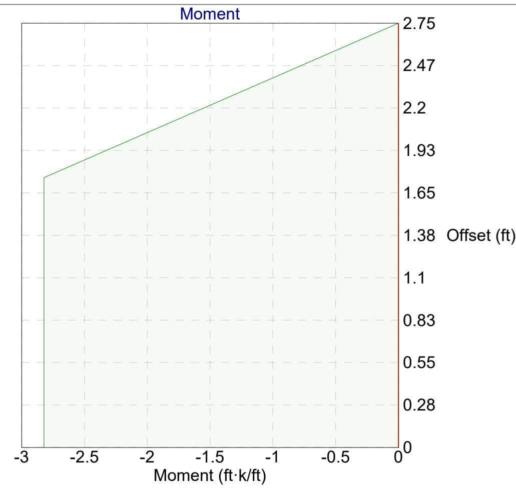
Stem Joint Force Transfer

Location	Force
@ stem base	0 k/ft

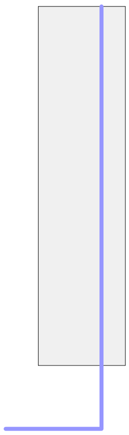
Stem Internal Forces

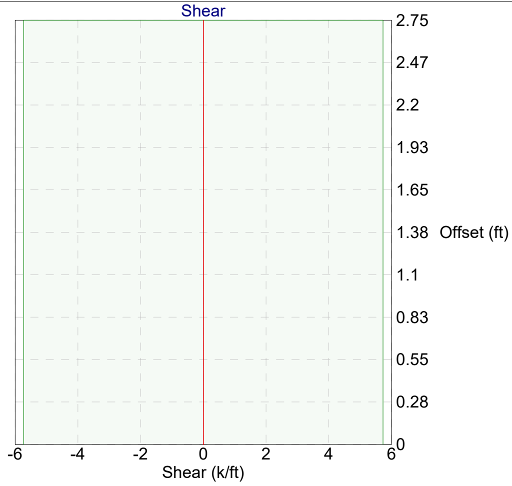
Stem Moment Checks [1.4D]





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]

$\phi M_n = 2.82 \text{ ft·k / ft} \geq (4/3) M_u = [4/3](0 \text{ ft·k / ft}) = 0 \text{ ft·k / ft}$

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]

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

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]

$\beta_1 = 0.850 \quad (F'_c \leq 4000 \text{ psi})$

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

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

$\varepsilon_t = 0.0657 \geq 0.004$ ✓

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

$\beta_1 = 0.850 \quad (F'_c \leq 4000 \text{ psi})$

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

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

$\varepsilon_t = 0.0657 \geq 0.004$ ✓

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

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

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

$\rho_t = 0.0011 < \rho_{t\_min} = 0.0020$ ✗

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

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)

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

$\psi_e = 1.0 \quad$ (uncoated hooked bars)

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

$\psi_r = 1.0 \quad$ (no confining reinforcement)

$\lambda = 1.0 \quad$ (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

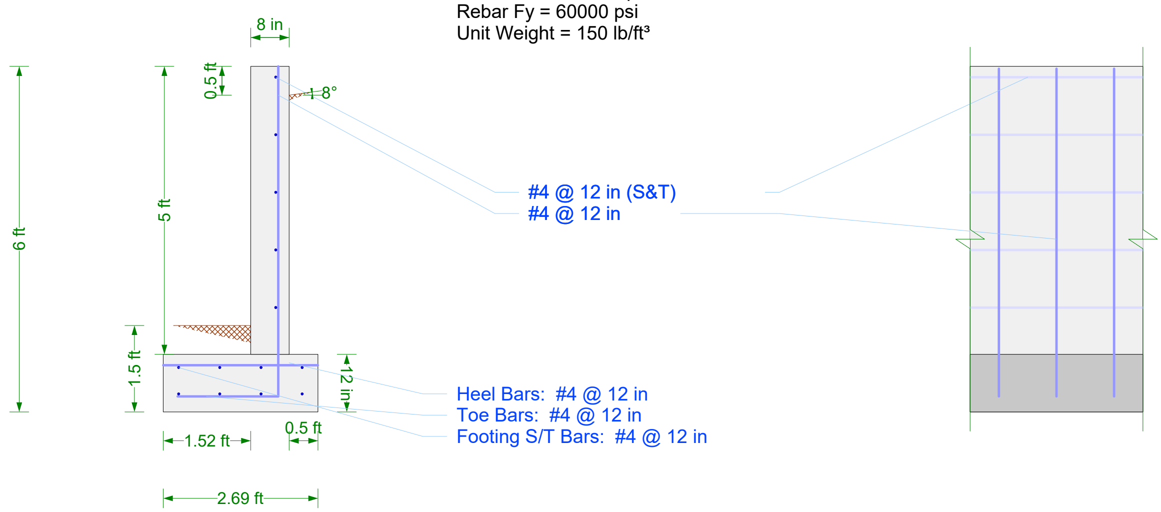
$8 d_b = 8(0.38 \text{ in}) = 3.0 \quad$ (minimum limit, does not control)

6 inch minimum controls

$l_{dh\_prov} = 6 \text{ in} \geq l_{dh} = 6 \text{ in}$ ✓

Design Detail

Concrete f'c = 3000 psi
Rebar Fy = 60000 psi
Unit Weight = 150 lb/ft³



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

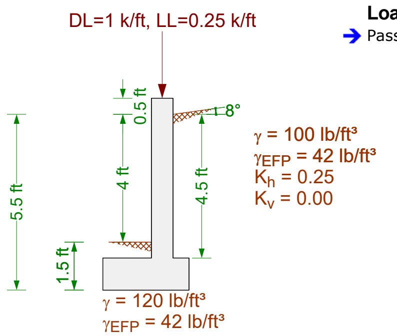
Criteria

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

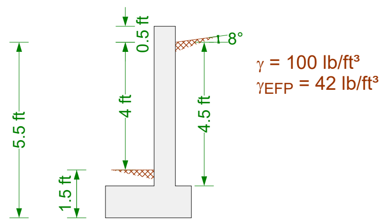
Load Combination	Stem Actual steel (in²/in)	Heel M-applied (ft·k/ft)	Heel M-allow (ft·k/ft)	Heel V-applied (k/ft)	Heel V-allow (k/ft)	Toe M-applied (ft·k/ft)	Toe M-allow (ft·k/ft)	Toe V-applied (k/ft)	Toe V-allow (k/ft)
1.2D + 1.6L + 1.6H	0.02	0.09	8.6	0.36	9.61	1.93	7.7	1.41	8.63
1.2D + 1.6L + 0.9H	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

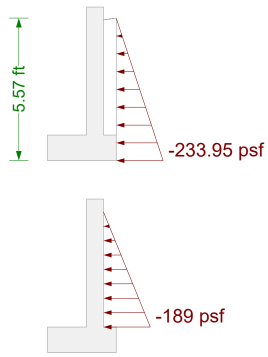
Stability Check Results Summary

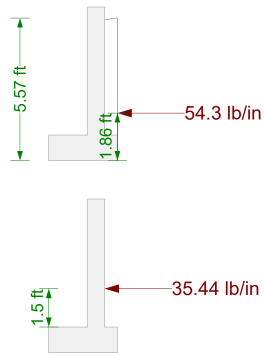
Load Combination	Overturning Moment (ft·k/ft)	Resisting Moment (ft·k/ft)	Overturning F.S.	Overturning F.S. Req'd	Overturning F.S. Req'd Seismic	Sliding Force (lb/in)	Resisting Force (lb/in)	Sliding F.S.
1.0D + 1.0L + 1.0H + 0.7E	1.21	3.97	3.279	1.500	1.200	54.3	80.17	1.476
1.0D + 1.0L + 1.0H	1.21	3.97	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

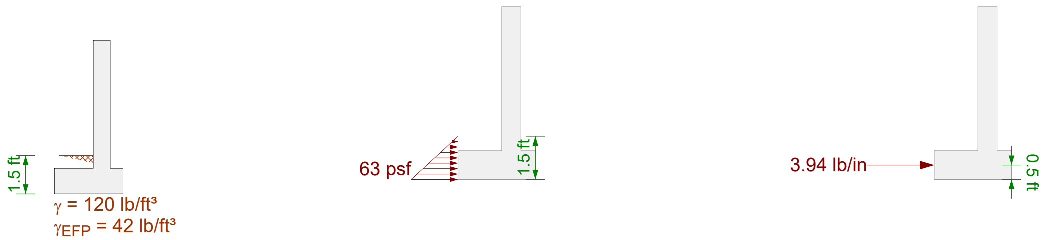
Equivalent Fluid Pressure

$\sigma_h = H\,\gamma_{fluid} = (5.57\text{ ft})(42\text{ lb / ft}^3) = 234\text{ psf}$

Lateral Earth Pressure (stem only)

$\sigma_h = H\,\gamma_{fluid} = (4.5\text{ ft})(42\text{ lb / ft}^3) = 189\text{ psf}$

Passive Pressure

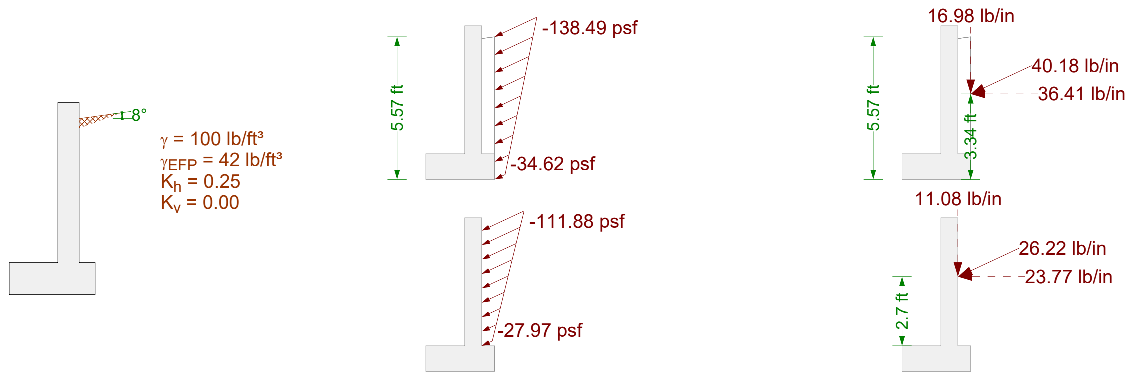


Lateral Earth Pressure

Equivalent Fluid Pressure

$\sigma_h = H\,\gamma_{fluid}$ = (1.5 ft)(42 lb / ft³) = 63 psf

Seismic Pressure



Seismic Pressure

Dynamic + static force (Mononobe - Okabe equation)

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

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

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

$$= 0.6403$$

$$P_{ae} = \frac{1}{2} K_{ae}\,\gamma\,H^2(1 - k_v) = \frac{1}{2}(0.6403)(100\ \mathrm{lb/ft^3})(5.57\ \mathrm{ft})^2[1 - (0.0)] = 82.78\ \mathrm{lb/in}$$

Static - only force (Coulomb equation)

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

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

$$= 0.3295$$

$$P_a = \frac{1}{2} K_a\,\gamma\,H^2 = \frac{1}{2}(0.3295)(100\ \mathrm{lb/ft^3})(5.57\ \mathrm{ft})^2 = 42.6\ \mathrm{lb/in}$$

Net dynamic force

$$\Delta P_{ae} = P_{ae} - P_a = (82.78\ \mathrm{lb/in}) - (42.6\ \mathrm{lb/in}) = 40.18\ \mathrm{lb/in}$$

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

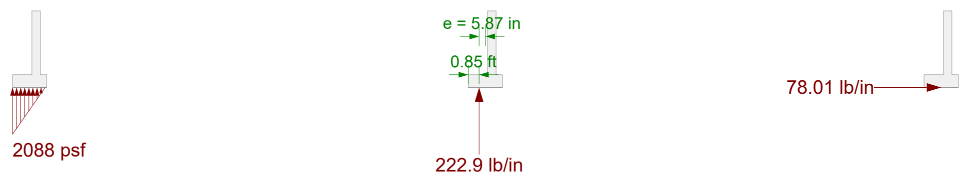
To arrive at the pressure distribution illustrated above (used to determine stem moments), apply inverted triangular pressure plus a uniform portion to bring resultant to 0.6H

$$\sigma_{e\_top} = \frac{8}{5}\frac{\Delta P_{ae}}{H} = \frac{8}{5}\frac{(40.18\ \mathrm{lb/in})}{(5.57\ \mathrm{ft})} = 138.5\ \mathrm{psf}$$

$$\sigma_{e\_bot} = \frac{2}{5}\frac{\Delta P_{ae}}{H} = \frac{2}{5}\frac{(40.18\ \mathrm{lb/in})}{(5.57\ \mathrm{ft})} = 34.62\ \mathrm{psf}$$

Wall/Soil Weights

Bearing Pressure



Friction

F = μ 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

$$\frac{-\ 27406.43\ \text{in·lb / ft}}{-\ 222.89\ \text{lb / in}} = 0.85\ \text{ft}$$

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:

$$\text{F.S.} = \frac{\text{RM}}{\text{OTM}} = \frac{47604 \text{ in·lb / ft}}{14518 \text{ in·lb / ft}} = 3.279 > 1.50 \text{ (OK)}$$

Including seismic loads:

$$\text{F.S.} = \frac{\text{RM}}{\text{OTM}} = \frac{47604 \text{ in·lb / ft}}{26786 \text{ in·lb / ft}} = 1.777 > 1.20 \text{ (OK)}$$

Sliding Check

Check not performed; restrained against sliding.

Bearing Capacity Check

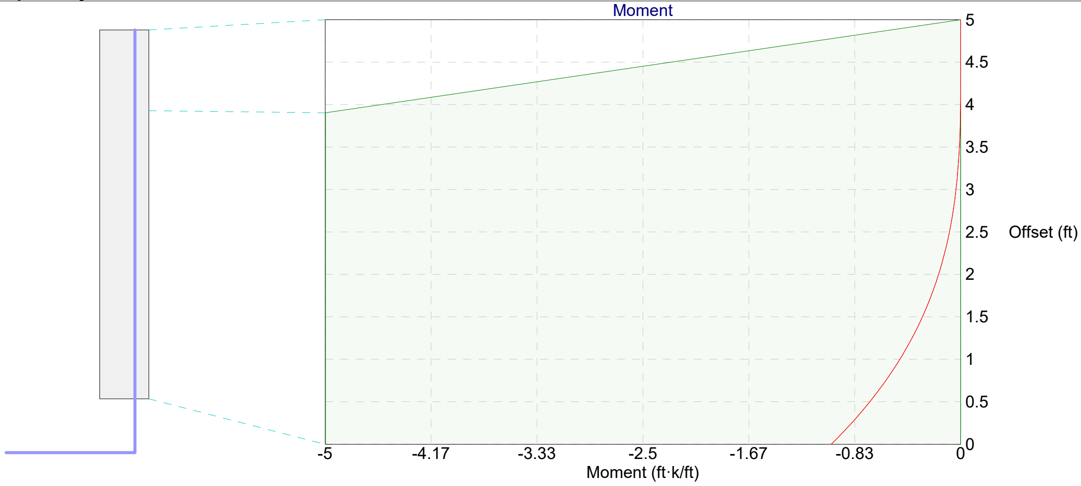
Bearing pressure < allowable (2041 psf < 2500 psf) - OK
Bearing resultant eccentricity < allowable (5.87 in < 16.12 in) - OK

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)

$V_c = 2\,\lambda\,\sqrt{F'_c}\,d = 2\,(1.0)\,\sqrt{3000\text{ psi}}\,(5.75\text{ in}) = 7.56\text{ k / ft}$

$\phi V_n = \phi\,V_c = (0.750)\,(7.56\text{ k / ft}) = 5.67\text{ k / ft}$

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)

$V_c = 2\,\lambda\,\sqrt{F'_c}\,d = 2\,(1.0)\,\sqrt{3000\text{ psi}}\,(5.75\text{ in}) = 7.56\text{ k / ft}$

$\phi V_n = \phi\,V_c = (0.750)\,(7.56\text{ k / ft}) = 5.67\text{ k / ft}$

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)

$\psi_e = 1.0$ (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)

$$l_{dh} = \left(\frac{f_y \, \psi_e \, \psi_c \, \psi_r}{50 \, \lambda \, \sqrt{F'_c}}\right) d_b = \left[\frac{(60000 \text{ psi})(1.0)(0.70)(1.0)}{50 \, (1.0)\sqrt{3000 \text{ psi}}}\right](0.5 \text{ in}) = 7.67 \text{ in}$$

$8 \, d_b = 8 \, (0.5 \text{ in}) = 4.0$ (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)

$\psi_t = 1.0$ (bars are not horizontal)

$\psi_e = 1.0$ (bar not epoxy coated)

$\psi_s = 0.80$ (bars are #6 or smaller)

$\lambda = 1.0$ (normal weight concrete)

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

$\text{cover} + d_b / 2 = (2 \text{ in}) + (0.5 \text{ in}) / 2 = 2.25 \text{ in}$

$c_b = 2.25 \text{ in}$ (lesser of half spacing, ctr to surface)

$K_{tr} = 0.0$ (no transverse reinforcement)

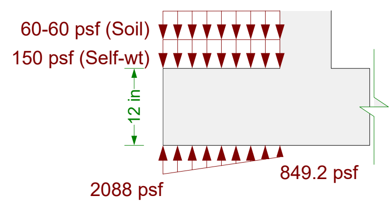
$$\frac{c_b + K_{tr}}{d_b} = \frac{(2.25 \text{ in}) + (0.0)}{(0.5 \text{ in})} = 4.50$$

$$l_d = \left(\frac{3.}{40} \, \frac{f_y}{\lambda \sqrt{F'_c}} \, \frac{\psi_t \, \psi_e \, \psi_s}{2.5}\right) d_b = \left[\frac{3.}{40} \, \frac{(60000 \text{ psi})}{(1.0)\sqrt{3000 \text{ psi}}} \, \frac{(1.0)(1.0)(0.80)}{2.5}\right](0.5 \text{ in}) = 13.15 \text{ in}$$

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

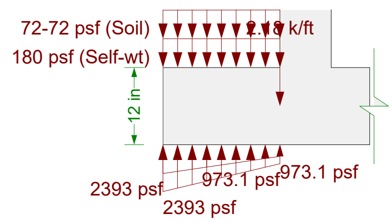
Toe Unfactored Loads

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)

$$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0.02 \text{ in}^2/\text{in})(60000 \text{ psi})}{0.85(3000 \text{ psi})} = 0.39 \text{ in}$$

$$\phi M_n = \phi A_s f_y (d - a/2) = (0.90)(0.02 \text{ in}^2/\text{in})(60000 \text{ psi})[(8.75 \text{ in}) - (0.39 \text{ in})/2] = 7.7 \text{ ft·k/ft}$$

ϕM_n = 7.7 ft·k / ft ≥ M_u = 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)

$$V_c = 2\lambda\sqrt{F'_c}\, d = 2(1.0)\sqrt{3000 \text{ psi}}(8.75 \text{ in}) = 11.5 \text{ k/ft}$$

$$\phi V_n = \phi V_c = (0.750)(11.5 \text{ k/ft}) = 8.63 \text{ k/ft}$$

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)

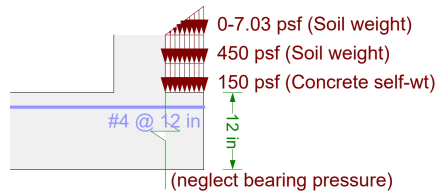
6 inch minimum controls

l_{dh_prov} = 6 in ≥ l_{dh} = 6 in ✓

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

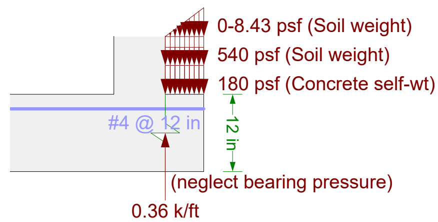
Heel Unfactored Loads

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 ≥ $(4/3) M_u$ = $[4/3](0.09 \text{ ft·k/ft})$ = 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)

$\frac{c_b + K_{tr}}{d_b} = \frac{(2.25 \text{ in}) + (0.0)}{(0.5 \text{ in})} = 4.50$

$l_d = \left(\frac{3.}{40}\, \frac{f_y}{\lambda\sqrt{F'_c}}\, \frac{\psi_t \psi_e \psi_s}{2.5}\right) d_b = \left[\frac{3.}{40}\, \frac{(60000 \text{ psi})}{(1.0)\sqrt{3000 \text{ psi}}}\, \frac{(1.0)(1.0)(0.80)}{2.5}\right](0.5 \text{ in}) = 13.15 \text{ in}$

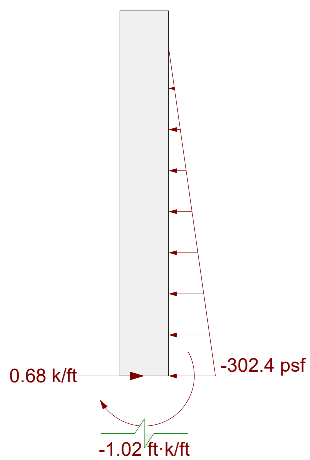
Factoring l_d by the excess reinforcement ratio (0.0105) per 25.4.10: l_d = 0.14 in

12 inch minimum controls

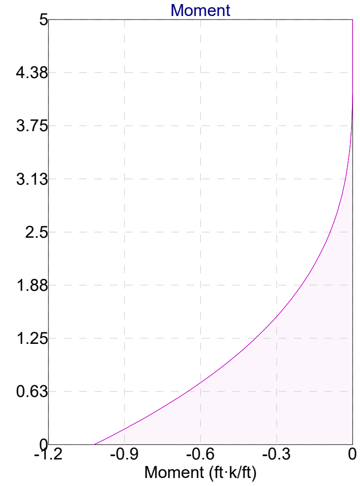
l_{d_prov} = 24.24 in ≥ l_d = 12 in ✓

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

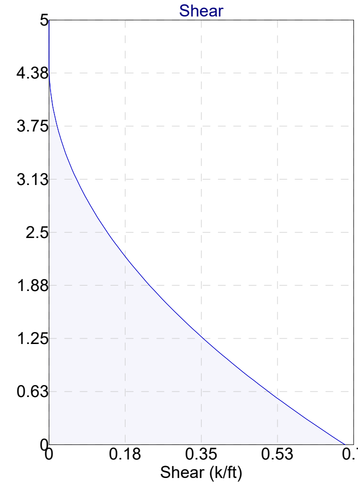
Stem Internal Forces



Stem Internal Forces



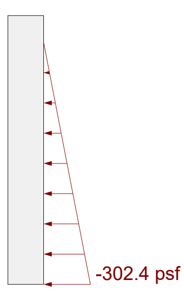
Stem Internal Forces



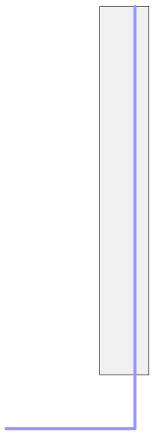
Stem Joint Force Transfer

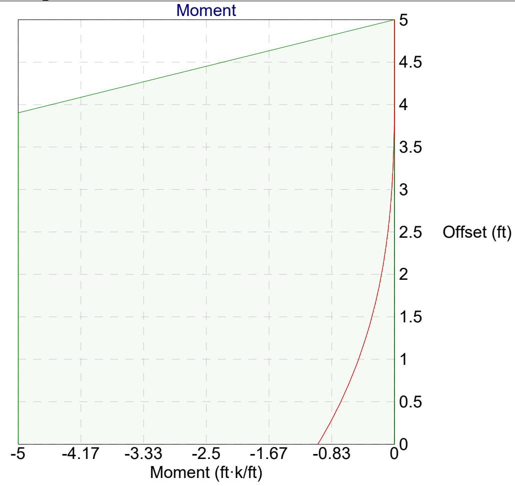
Location	Force
@ stem base	0.68 k/ft

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 ✓

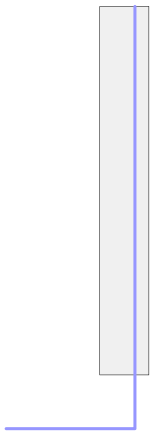
Check (ACI 318-14 11.5.5.1b) @ 3.9 ft from base

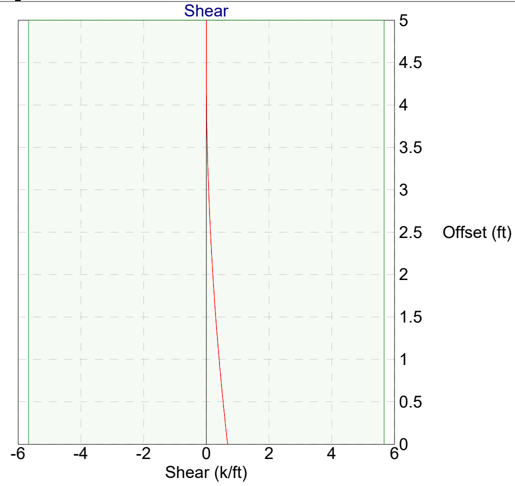
ϕM_n = 5 ft·k / ft ≥ M_u = 0 ft·k / ft ✓

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 $\geq$ 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]

$\phi M_n = 5 \text{ ft·k / ft} \geq (4/3) M_u = [4/3](1.02 \text{ ft·k / ft}) = 1.36 \text{ ft·k / ft}$

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]

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

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]

$\beta_1 = 0.850 \quad (F'_c \leq 4000 \text{ psi})$

$$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0.02 \text{ in}^2/\text{in})(60000 \text{ psi})}{0.85(3000 \text{ psi})} = 0.39 \text{ in}$$

$$\varepsilon_t = 0.003\left(\frac{d}{a/\beta_1} - 1\right) = 0.003\left[\frac{(5.75 \text{ in})}{(0.39 \text{ in})/(0.850)} - 1\right] = 0.0344$$

$\varepsilon_t = 0.0344 \geq 0.004$ ✓

Maximum Steel Check (ACI 318-14 9.3.3.1) @ 5 ft from base [Stem in negative flexure]

$\beta_1 = 0.850 \quad (F'_c \leq 4000 \text{ psi})$

$$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0.02 \text{ in}^2/\text{in})(60000 \text{ psi})}{0.85(3000 \text{ psi})} = 0.39 \text{ in}$$

$$\varepsilon_t = 0.003\left(\frac{d}{a/\beta_1} - 1\right) = 0.003\left[\frac{(5.75 \text{ in})}{(0.39 \text{ in})/(0.850)} - 1\right] = 0.0344$$

$\varepsilon_t = 0.0344 \geq 0.004$ ✓

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

$$\rho_t = \frac{A_{s\_horz} / s_{horz}}{t} = \frac{(0.2 \text{ in}^2)/(12 \text{ in})}{(8 \text{ in})} = 0.0021$$

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

$\rho_t = 0.0021 \geq \rho_{t\_min} = 0.0020$ ✓

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

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)

$$\frac{M_u}{\phi M_n} = \frac{(1.02 \text{ ft·k / ft})}{(5 \text{ ft·k / ft})} = 0.2042 \quad \text{(ratio to represent excess reinforcement)}$$

$\psi_e = 1.0 \quad$ (uncoated hooked bars)

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

$\psi_r = 1.0 \quad$ (no confining reinforcement)

$\lambda = 1.0 \quad$ (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: $l_{dh} = 1.57 \text{ in}$

$8 d_b = 8(0.5 \text{ in}) = 4.0 \quad$ (minimum limit, does not control)

6 inch minimum controls

$l_{dh\_prov} = 9 \text{ in} \geq l_{dh} = 6 \text{ in}$ ✓

Toe Checks [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)

$$a = \frac{A_s\, f_y}{0.85\, F'_c} = \frac{(0.02\ \text{in}^2/\text{in})\,(60000\ \text{psi})}{0.85\,(3000\ \text{psi})} = 0.39\ \text{in}$$

$$\phi M_n = \phi\, A_s\, f_y\,(d - a/2) = (0.90)\,(0.02\ \text{in}^2/\text{in})\,(60000\ \text{psi})\,[(8.75\ \text{in}) - (0.39\ \text{in})/2] = 7.7\ \text{ft·k/ft}$$

ϕM_n = 7.7 ft·k / ft ≥ M_u = 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)

$$V_c = 2\,\lambda\,\sqrt{F'_c}\,d = 2\,(1.0)\,\sqrt{3000\ \text{psi}}\,(8.75\ \text{in}) = 11.5\ \text{k/ft}$$

$$\phi V_n = \phi\, V_c = (0.750)\,(11.5\ \text{k/ft}) = 8.63\ \text{k/ft}$$

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

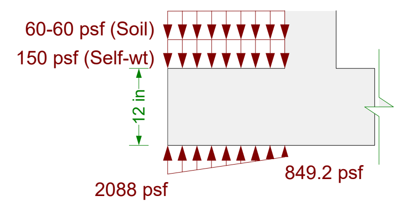
8 d_b = 8 (0.5 in) = 4.0 (minimum limit, does not control)

6 inch minimum controls

l_{dh_prov} = 6 in ≥ l_{dh} = 6 in ✓

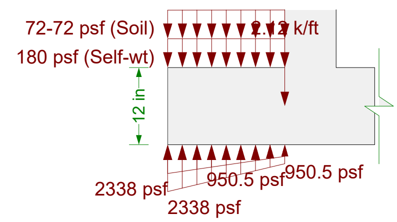
Toe Unfactored Loads

Unfactored Loads



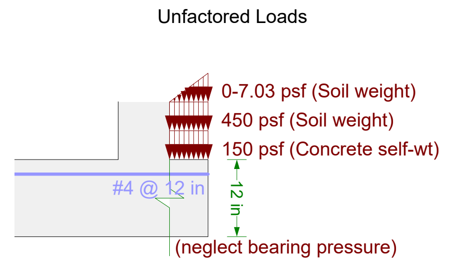
Toe Factored Loads

1.2D + 0.5L + 1.6H + 1.0E

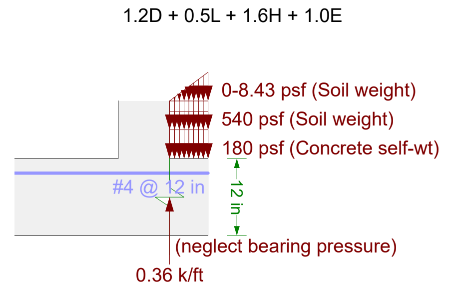


Heel Checks [1.2D + 0.5L + 1.6H + 1.0E]

Heel Unfactored Loads



Heel Factored Loads



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)

$$\frac{c_b + K_{tr}}{d_b} = \frac{(2.25 \text{ in}) + (0.0)}{(0.5 \text{ in})} = 4.50$$

$$l_d = \left(\frac{3.}{40}\,\frac{f_y}{\lambda\sqrt{F'_c}}\,\frac{\psi_t \psi_e \psi_s}{2.5}\right) d_b = \left[\frac{3.}{40}\,\frac{(60000 \text{ psi})}{(1.0)\sqrt{3000 \text{ psi}}}\,\frac{(1.0)(1.0)(0.80)}{2.5}\right](0.5 \text{ in}) = 13.15 \text{ in}$$

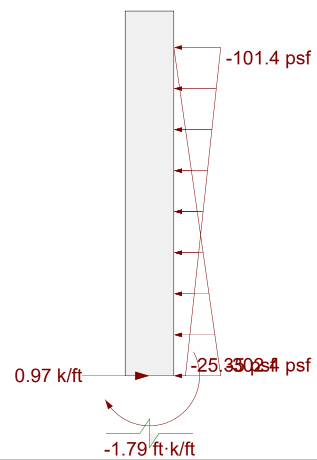
Factoring l_d by the excess reinforcement ratio (0.0105) per 25.4.10: l_d = 0.14 in

12 inch minimum controls

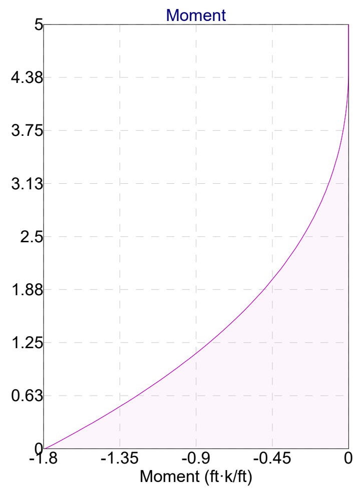
l_{d_prov} = 24.24 in ≥ l_d = 12 in ✓

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

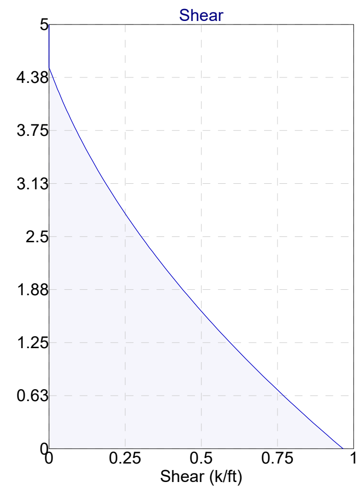
Stem Internal Forces



Stem Internal Forces



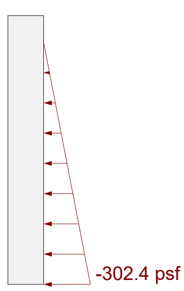
Stem Internal Forces

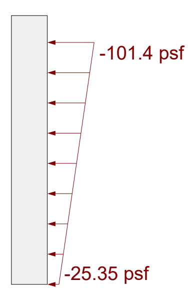


Stem Joint Force Transfer

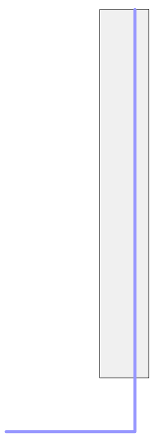
Location	Force
@ stem base	0.97 k/ft

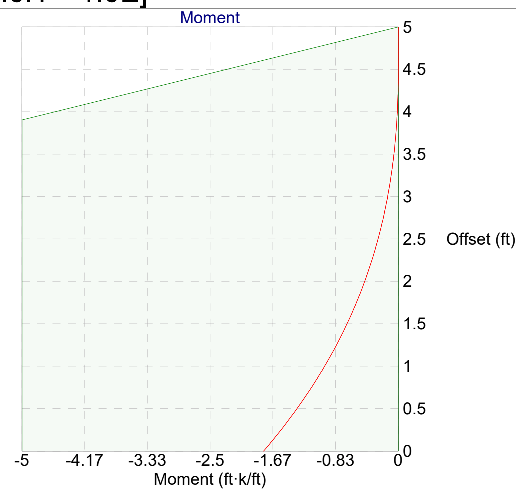
Stem Internal Forces





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





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

$\phi M_n = 5$ ft·k / ft $\geq$ $M_u = 1.79$ ft·k / ft ✓

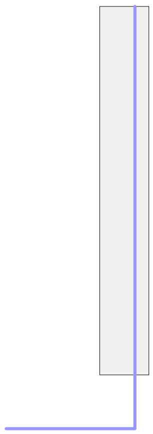
Check (ACI 318-14 11.5.5.1b) @ 3.9 ft from base

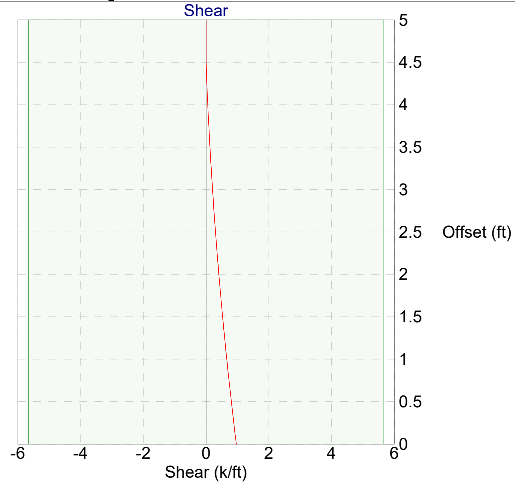
$\phi M_n = 5$ ft·k / ft $\geq$ $M_u = 0.02$ ft·k / ft ✓

Check (ACI 318-14 11.5.5.1b) @ 3.94 ft from base

$\phi M_n = 4.84$ ft·k / ft $\geq$ $M_u = 0.02$ 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 $\geq$ 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]

$\phi M_n = 5\ ft \cdot k / ft \quad \geq \quad (4/3)\, M_u = [4/3](1.79\ ft \cdot k / ft) = 2.39\ ft \cdot k / ft$

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]

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

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]

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

$$a = \frac{A_s f_y}{0.85\, F'_c} = \frac{(0.02\ in^2 / in)(60000\ psi)}{0.85\,(3000\ psi)} = 0.39\ in$$

$$\varepsilon_t = 0.003 \left(\frac{d}{a / \beta_1} - 1\right) = 0.003 \left[\frac{(5.75\ in)}{(0.39\ in) / (0.850)} - 1\right] = 0.0344$$

$\varepsilon_t = 0.0344 \quad \geq \quad 0.004$ ✓

Maximum Steel Check (ACI 318-14 9.3.3.1) @ 5 ft from base [Stem in negative flexure]

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

$$a = \frac{A_s f_y}{0.85\, F'_c} = \frac{(0.02\ in^2 / in)(60000\ psi)}{0.85\,(3000\ psi)} = 0.39\ in$$

$$\varepsilon_t = 0.003 \left(\frac{d}{a / \beta_1} - 1\right) = 0.003 \left[\frac{(5.75\ in)}{(0.39\ in) / (0.850)} - 1\right] = 0.0344$$

$\varepsilon_t = 0.0344 \quad \geq \quad 0.004$ ✓

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

$$\rho_t = \frac{A_{s\_horz} / s_{horz}}{t} = \frac{(0.2\ in^2) / (12\ in)}{(8\ in)} = 0.0021$$

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

$\rho_t = 0.0021 \quad \geq \quad \rho_{t\_min} = 0.0020$ ✓

$3\,h = 3\,(8\ in) = 24\ in$

18 inch limit governs

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

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

$$\frac{M_u}{\phi M_n} = \frac{(1.79\ ft \cdot k / ft)}{(5\ ft \cdot k / ft)} = 0.3582 \qquad \text{(ratio to represent excess reinforcement)}$$

$\psi_e = 1.0 \qquad$ (uncoated hooked bars)

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

$\psi_r = 1.0 \qquad$ (no confining reinforcement)

$\lambda = 1.0 \qquad$ (normal weight concrete)

$$l_{dh} = \left(\frac{f_y\, \psi_e\, \psi_c\, \psi_r}{50\, \lambda \sqrt{F'_c}}\right) d_b = \left[\frac{(60000\ psi)(1.0)(0.70)(1.0)}{50\,(1.0)\sqrt{3000\ psi}}\right](0.5\ in) = 7.67\ in$$

Factoring l_{dh} by the excess reinforcement ratio (0.3582) per 25.4.10: $l_{dh} = 2.75\ in$

$8\, d_b = 8\,(0.5\ in) = 4.0 \qquad$ (minimum limit, does not control)

6 inch minimum controls

$l_{dh\_prov} = 9\ in \quad \geq \quad l_{dh} = 6\ in$ ✓

Toe Checks [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)

$a = \dfrac{A_s\, f_y}{0.85\, F'_c} = \dfrac{(0.02\ in^2 / in)\,(60000\ psi)}{0.85\,(3000\ psi)} = 0.39\ in$

$\phi M_n = \phi\, A_s\, f_y\, (d - a / 2) = (0.90)\,(0.02\ in^2 / in)\,(60000\ psi)\,[(8.75\ in) - (0.39\ in) / 2] = 7.7\ ft{\cdot}k / ft$

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)

$V_c = 2\,\lambda\,\sqrt{F'_c}\, d = 2\,(1.0)\,\sqrt{3000\ psi}\,(8.75\ in) = 11.5\ k / ft$

$\phi V_n = \phi\, V_c = (0.750)\,(11.5\ k / ft) = 8.63\ k / ft$

ϕV_n = 8.63 k / ft ≥ V_u = 1.4 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)

$a = \dfrac{A_s\, f_y}{0.85\, F'_c} = \dfrac{(0.02\ in^2 / in)\,(60000\ psi)}{0.85\,(3000\ psi)} = 0.39\ in$

$\varepsilon_t = 0.003\left(\dfrac{d}{a / \beta_1} - 1\right) = 0.003\left[\dfrac{(8.75\ in)}{(0.39\ in) / (0.850)} - 1\right] = 0.0539$

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 ≥ $(4 / 3)\, M_u$ = $[4 / 3]\,(-0\ ft{\cdot}k / ft)$ = - 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)

$\rho_{ST\_prov} = \dfrac{A_{ST}}{t\, s_{ST}} = \dfrac{(0.4\ in^2 / in)}{(12\ in)\,(12\ in)} = 0.0028$

$\rho_{ST\_prov} = \dfrac{A_{ST}}{t\, s_{ST}} = \dfrac{(0.4\ in^2 / in)}{(12\ in)\,(12\ in)} = 0.0028$

$\dfrac{0.0018\,(60000)}{f_y} = \dfrac{0.0018\,(60000)}{(60000\ psi)} = 0.0018$

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)

$\dfrac{M_u}{\phi M_n} = \dfrac{(-0\ ft{\cdot}k / ft)}{(7.7\ ft{\cdot}k / ft)} = -0.0$ (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)

$l_{dh} = \left(\dfrac{f_y\, \psi_e\, \psi_c\, \psi_r}{50\,\lambda\,\sqrt{F'_c}}\right) d_b = \left[\dfrac{(60000\ psi)\,(1.0)\,(0.70)\,(1.0)}{50\,(1.0)\,\sqrt{3000\ psi}}\right](0.5\ in) = 7.67\ in$

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

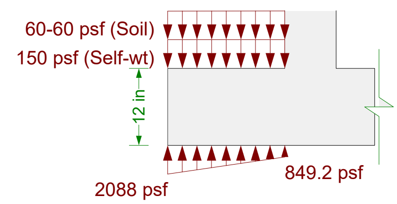
$8\, d_b = 8\,(0.5\ in) = 4.0$ (minimum limit, does not control)

6 inch minimum controls

l_{dh_prov} = 6 in ≥ l_{dh} = 6 in ✓

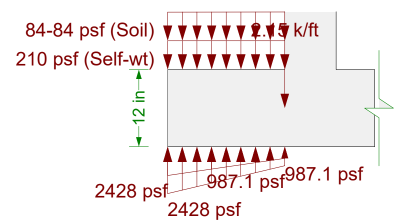
Toe Unfactored Loads

Unfactored Loads



Toe Factored Loads

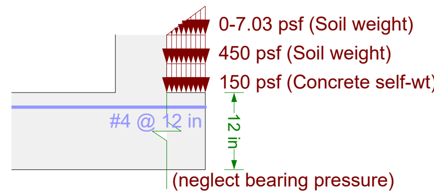
1.4D



Heel Checks [1.4D]

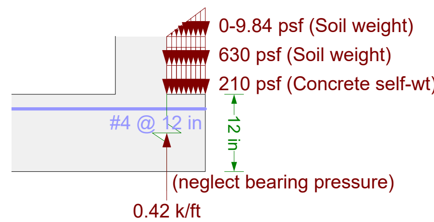
Heel Unfactored Loads

Unfactored Loads



Heel Factored Loads

1.4D



Controlling Moment

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

$M_{heel} = 0.11 \text{ ft·k / ft} \geq M_{stem} = -0 \text{ ft·k / ft}$

$M_u = -0 \text{ ft·k / ft}$ (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)

$a = \dfrac{A_s f_y}{0.85 F'_c} = \dfrac{(0.02 \text{ in}^2/\text{in})(60000 \text{ psi})}{0.85(3000 \text{ psi})} = 0.39 \text{ in}$

$\phi M_n = \phi A_s f_y (d - a/2) = (0.90)(0.02 \text{ in}^2/\text{in})(60000 \text{ psi})[(9.75 \text{ in}) - (0.39 \text{ in})/2] = 8.6 \text{ ft·k / ft}$

$\phi M_n = 8.6 \text{ ft·k / ft} \geq M_u = -0 \text{ ft·k / ft}$ ✓

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

$\lambda = 1.0$ (normal weight concrete)

$V_c = 2\lambda\sqrt{F'_c}\, d = 2(1.0)\sqrt{3000 \text{ psi}}(9.75 \text{ in}) = 12.82 \text{ k / ft}$

$\phi V_n = \phi V_c = (0.750)(12.82 \text{ k / ft}) = 9.61 \text{ k / ft}$

$\phi V_n = 9.61 \text{ k / ft} \geq V_u = 0.42 \text{ k / ft}$ ✓

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

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

$a = \dfrac{A_s f_y}{0.85 F'_c} = \dfrac{(0.02 \text{ in}^2/\text{in})(60000 \text{ psi})}{0.85(3000 \text{ psi})} = 0.39 \text{ in}$

$\varepsilon_t = 0.003\left(\dfrac{d}{a/\beta_1} - 1\right) = 0.003\left[\dfrac{(9.75 \text{ in})}{(0.39 \text{ in})/(0.850)} - 1\right] = 0.0604$

$\varepsilon_t = 0.0604 \geq 0.004$ ✓

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

$\phi M_n = 8.6 \text{ ft·k / ft} \geq (4/3) M_u = [4/3](-0 \text{ ft·k / ft}) = -0 \text{ ft·k / ft}$

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)

$\rho_{ST\_prov} = \dfrac{A_{ST}}{t\, s_{ST}} = \dfrac{(0.4 \text{ in}^2/\text{in})}{(12 \text{ in})(12 \text{ in})} = 0.0028$

$\rho_{ST\_prov} = \dfrac{A_{ST}}{t\, s_{ST}} = \dfrac{(0.4 \text{ in}^2/\text{in})}{(12 \text{ in})(12 \text{ in})} = 0.0028$

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

$\rho_{ST\_min} = 0.0018$

$\rho_{ST\_prov} = 0.0028 \geq \rho_{ST\_min} = 0.0018$ ✓

18 inch limit governs

$s_{ST\_max} = 18 \text{ in}$

$s_{ST} = 12 \text{ in} \leq s_{ST\_max} = 18 \text{ in}$ ✓

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

$\dfrac{M_u}{\phi M_n} = \dfrac{(-0 \text{ ft·k / ft})}{(8.6 \text{ ft·k / ft})} = -0.0$ (ratio to represent excess reinforcement)

$\psi_t = 1.0$ (12 inches or less cast below - 9.50 inches)

$\psi_e = 1.0$ (bar not epoxy coated)

$\psi_s = 0.80$ (bars are #6 or smaller)

$\lambda = 1.0$ (normal weight concrete)

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

$\text{cover} + d_b/2 = (2 \text{ in}) + (0.5 \text{ in})/2 = 2.25 \text{ in}$

$c_b = 2.25 \text{ in}$ (lesser of half spacing, ctr to surface)

$K_{tr} = 0.0$ (no transverse reinforcement)

$\dfrac{c_b + K_{tr}}{d_b} = \dfrac{(2.25 \text{ in}) + (0.0)}{(0.5 \text{ in})} = 4.50$

$l_d = \left(\dfrac{3.}{40}\dfrac{f_y}{\lambda\sqrt{F'_c}}\dfrac{\psi_t\psi_e\psi_s}{2.5}\right)d_b = \left[\dfrac{3.}{40}\dfrac{(60000 \text{ psi})}{(1.0)\sqrt{3000 \text{ psi}}}\dfrac{(1.0)(1.0)(0.80)}{2.5}\right](0.5 \text{ in}) = 13.15 \text{ in}$

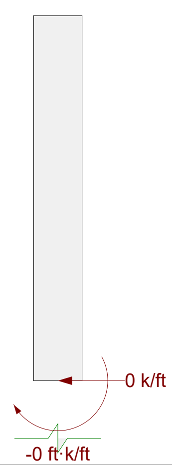
Factoring l_d by the excess reinforcement ratio (- 0.0000) per 25.4.10: $l_d = -0 \text{ in}$

12 inch minimum controls

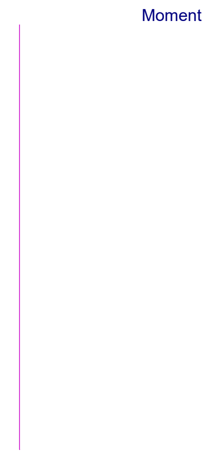
$l_{d\_prov} = 24.24 \text{ in} \geq l_d = 12 \text{ in}$ ✓

Stem Forces [1.4D]

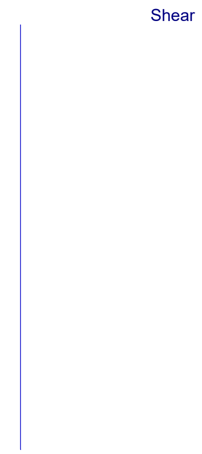
Stem Internal Forces



Stem Internal Forces



Stem Internal Forces

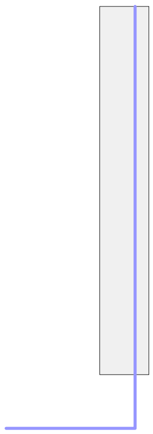


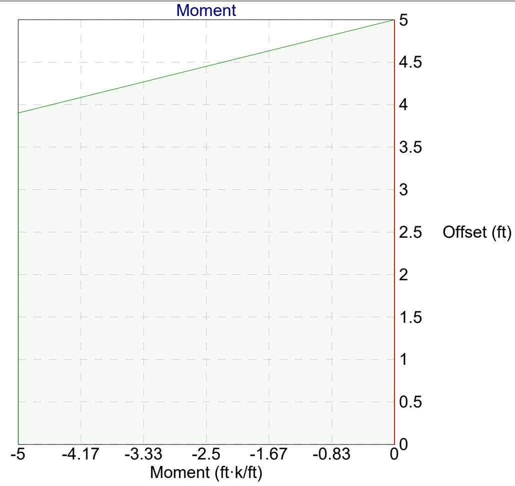
Stem Joint Force Transfer

Location	**Force**
@ stem base	0 k/ft

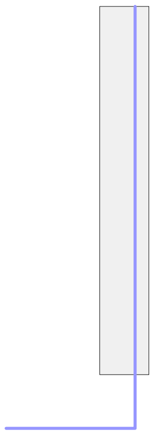
Stem Internal Forces

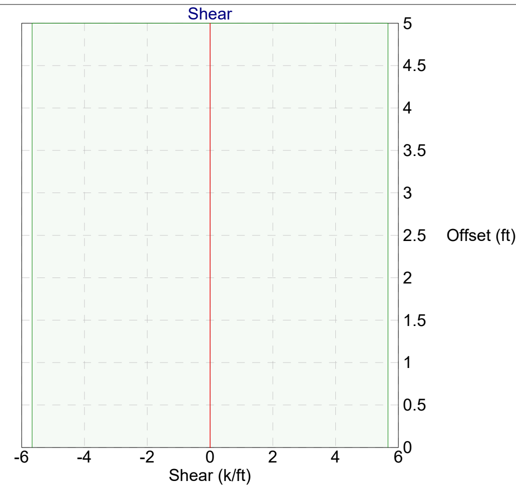
Stem Moment Checks [1.4D]





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]

$\phi M_n = 5 \text{ ft·k / ft} \geq (4/3)\, M_u = [4/3](0 \text{ ft·k / ft}) = 0 \text{ ft·k / ft}$

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]

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

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]

$\beta_1 = 0.850 \quad (F'_c \leq 4000 \text{ psi})$

$$a = \frac{A_s\, f_y}{0.85\, F'_c} = \frac{(0.02 \text{ in}^2/\text{in})(60000 \text{ psi})}{0.85\,(3000 \text{ psi})} = 0.39 \text{ in}$$

$$\varepsilon_t = 0.003\left(\frac{d}{a/\beta_1} - 1\right) = 0.003\left[\frac{(5.75 \text{ in})}{(0.39 \text{ in})/(0.850)} - 1\right] = 0.0344$$

$\varepsilon_t = 0.0344 \geq 0.004$ ✓

Maximum Steel Check (ACI 318-14 9.3.3.1) @ 5 ft from base [Stem in negative flexure]

$\beta_1 = 0.850 \quad (F'_c \leq 4000 \text{ psi})$

$$a = \frac{A_s\, f_y}{0.85\, F'_c} = \frac{(0.02 \text{ in}^2/\text{in})(60000 \text{ psi})}{0.85\,(3000 \text{ psi})} = 0.39 \text{ in}$$

$$\varepsilon_t = 0.003\left(\frac{d}{a/\beta_1} - 1\right) = 0.003\left[\frac{(5.75 \text{ in})}{(0.39 \text{ in})/(0.850)} - 1\right] = 0.0344$$

$\varepsilon_t = 0.0344 \geq 0.004$ ✓

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

$$\rho_t = \frac{A_{s\_horz} / s_{horz}}{t} = \frac{(0.2 \text{ in}^2)/(12 \text{ in})}{(8 \text{ in})} = 0.0021$$

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

$\rho_t = 0.0021 \geq \rho_{t\_min} = 0.0020$ ✓

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

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)

$$\frac{M_u}{\phi M_n} = \frac{(0 \text{ ft·k / ft})}{(5 \text{ ft·k / ft})} = 0.0 \quad \text{(ratio to represent excess reinforcement)}$$

$\psi_e = 1.0 \quad$ (uncoated hooked bars)

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

$\psi_r = 1.0 \quad$ (no confining reinforcement)

$\lambda = 1.0 \quad$ (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

$8\, d_b = 8\,(0.5 \text{ in}) = 4.0 \quad$ (minimum limit, does not control)

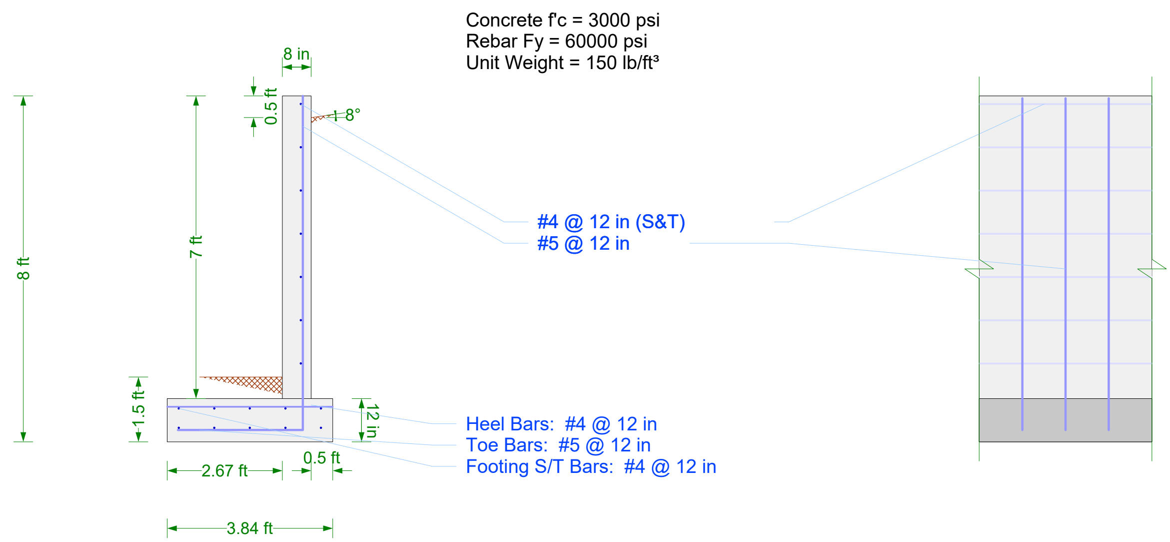
6 inch minimum controls

$l_{dh\_prov} = 9 \text{ in} \geq l_{dh} = 6 \text{ in}$ ✓

JOSEPH LUCIA
LUCIA ENGINEERING, INC.

Cantilever Wall 6' Tall

Design Detail



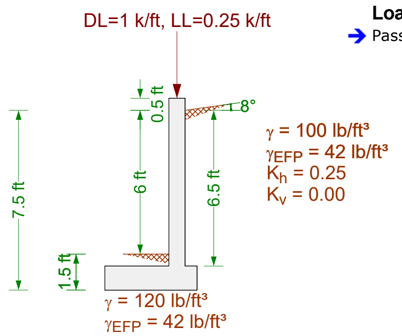
Check Summary

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

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

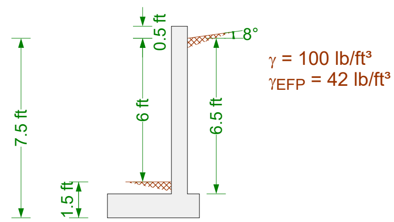
Load Combination	Stem Actual steel (in²/in)	Heel M-applied (ft·k/ft)	Heel M-allow (ft·k/ft)	Heel V-applied (k/ft)	Heel V-allow (k/ft)	Toe M-applied (ft·k/ft)	Toe M-allow (ft·k/ft)	Toe V-applied (k/ft)	Toe V-allow (k/ft)
1.2D + 1.6L + 1.6H	0.03	0.12	8.6	0.48	9.61	5.64	11.7	2.91	8.57
1.2D + 1.6L + 0.9H	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

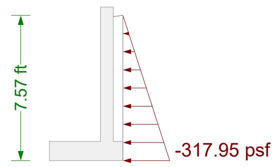
Stability Check Results Summary

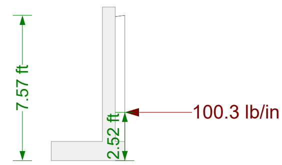
Load Combination	Overturning Moment (ft·k/ft)	Resisting Moment (ft·k/ft)	Overturning F.S.	Overturning F.S. Req'd	Overturning F.S. Req'd Seismic	Sliding Force (lb/in)	Resisting Force (lb/in)	Sliding F.S.
1.0D + 1.0L + 1.0H + 0.7E	3.04	7.62	2.509	1.500	1.200	100.3	99.48	0.992
1.0D + 1.0L + 1.0H	3.04	7.62	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

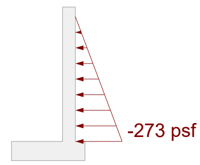
Load Combination	Sliding F.S. Req'd	Sliding F.S. Req'd Seismic	Bearing Pressure Actual (psf)	Bearing Pressure Allowable (psf)	Bearing Eccentricity Actual (in)	Bearing Eccentricity Allowable (in)	Wall Top Actual Deflection (in)
1.0D + 1.0L + 1.0H + 0.7E	1.500	1.200	2397	2500	12.09	23.02	0.36
1.0D + 1.0L + 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

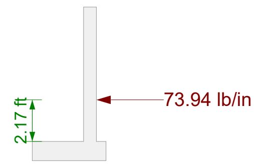
Backfill Pressure











Lateral Earth Pressure

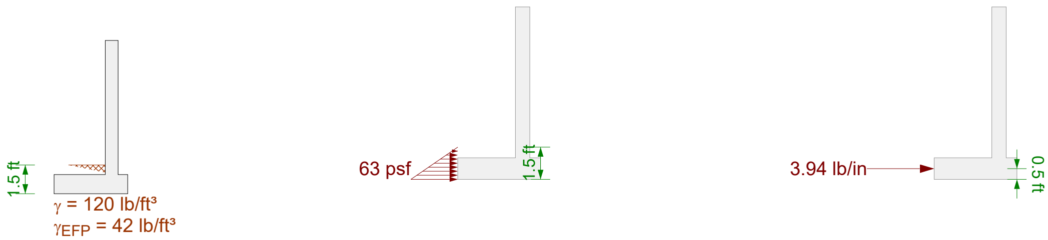
Equivalent Fluid Pressure

$\sigma_h = H\ \gamma_{fluid} = (7.57\ \text{ft})(42\ \text{lb / ft}^3) = 318\ \text{psf}$

Lateral Earth Pressure (stem only)

$\sigma_h = H\ \gamma_{fluid} = (6.5\ \text{ft})(42\ \text{lb / ft}^3) = 273\ \text{psf}$

Passive Pressure

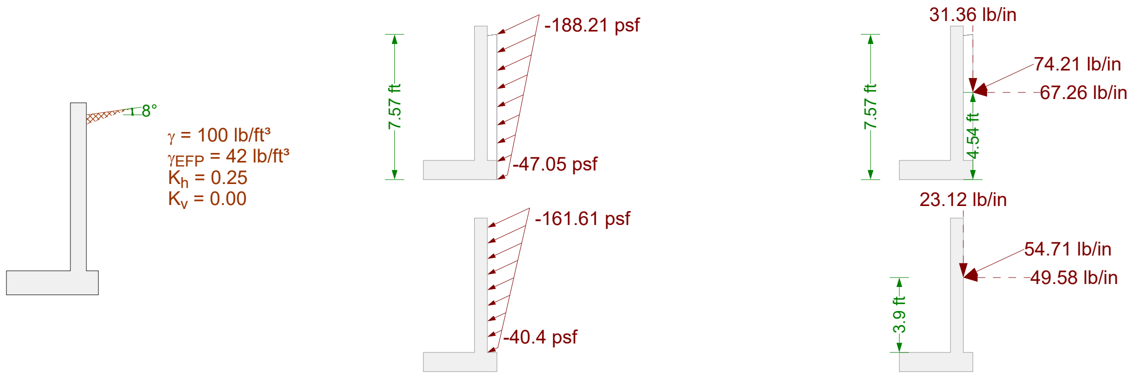


Lateral Earth Pressure

Equivalent Fluid Pressure

$\sigma_h = H\ \gamma_{fluid}$ = (1.5 ft)(42 lb / ft³) = 63 psf

Seismic Pressure



Seismic Pressure

Dynamic + static force (Mononobe - Okabe equation)

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

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

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

$$= 0.6403$$

$$P_{ae} = \frac{1}{2}K_{ae}\,\gamma\,H^{\wedge}2\,(1 - k_v) = \frac{1}{2}(0.6403)(100\ lb / ft^3)(7.57\ ft)^{\wedge}2\,[1 - (0.0)] = 152.9\ lb / in$$

Static - only force (Coulomb equation)

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

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

$$= 0.3295$$

$$P_a = \frac{1}{2}K_a\,\gamma\,H^{\wedge}2 = \frac{1}{2}(0.3295)(100\ lb / ft^3)(7.57\ ft)^{\wedge}2 = 78.68\ lb / in$$

Net dynamic force

$$\Delta P_{ae} = P_{ae} - P_a = (152.9\ lb / in) - (78.68\ lb / in) = 74.21\ lb / in$$

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

To arrive at the pressure distribution illustrated above (used to determine stem moments),

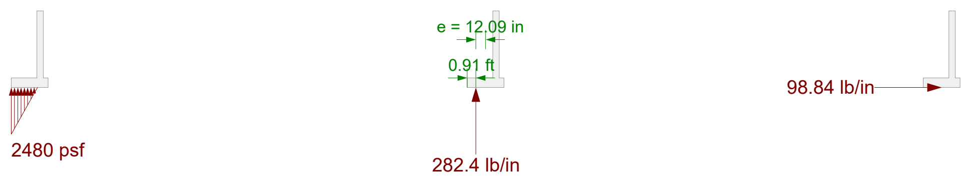
apply inverted triangular pressure plus a uniform portion to bring resultant to 0.6H

$$\sigma_{e\_top} = \frac{8}{5}\frac{\Delta P_{ae}}{H} = \frac{8}{5}\frac{(74.21\ lb / in)}{(7.57\ ft)} = 188.2\ psf$$

$$\sigma_{e\_bot} = \frac{2}{5}\frac{\Delta P_{ae}}{H} = \frac{2}{5}\frac{(74.21\ lb / in)}{(7.57\ ft)} = 47.05\ psf$$

Wall/Soil Weights

Bearing Pressure



Friction

F = μ 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

$$\frac{-\ 37050.99\ \text{in·lb / ft}}{-\ 282.4\ \text{lb / in}} = 0.91\ \text{ft}$$

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

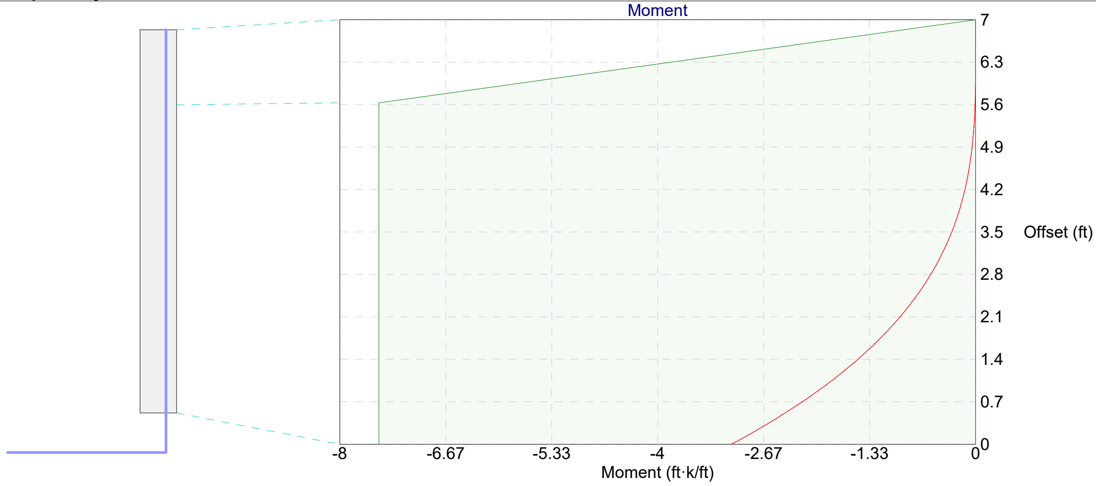
Bearing pressure < allowable (2397 psf < 2500 psf) - OK
Bearing resultant eccentricity < allowable (12.09 in < 23.02 in) - OK

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

$$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0.03 \text{ in}^2 / \text{in})(60000 \text{ psi})}{0.85 (3000 \text{ psi})} = 0.61 \text{ in}$$

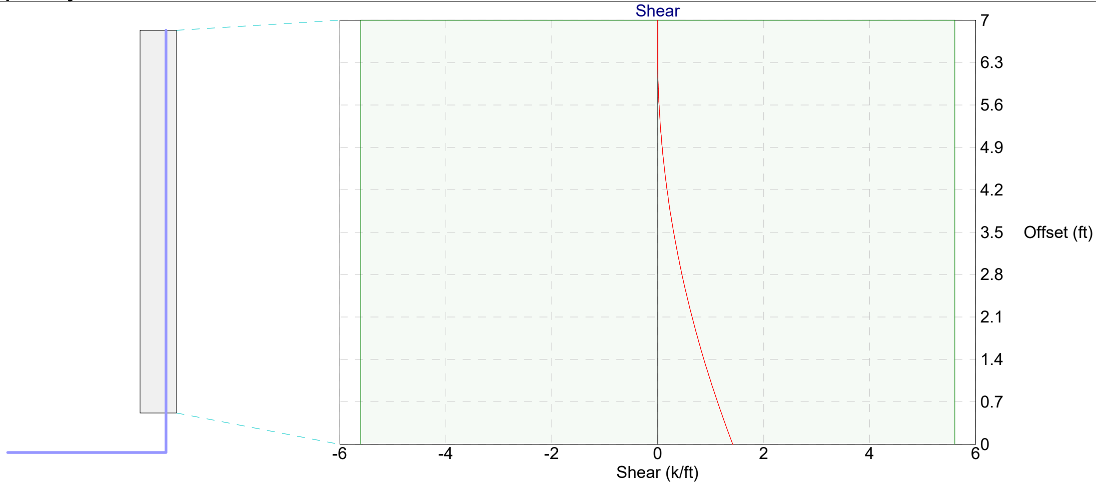
$$\phi M_n = \phi A_s f_y (d - a / 2) = (0.90)(0.03 \text{ in}^2 / \text{in})(60000 \text{ psi})[(5.69 \text{ in}) - (0.61 \text{ in}) / 2] = 7.51 \text{ ft·k / ft}$$

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)

$V_c = 2\,\lambda\,\sqrt{F'_c}\,d \;=\; 2\,(1.0)\,\sqrt{3000\text{ psi}}\,(5.69\text{ in}) \;=\; 7.48\text{ k / ft}$

$\phi V_n = \phi\,V_c \;=\; (0.750)\,(7.48\text{ k / ft}) \;=\; 5.61\text{ k / ft}$

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)

$V_c = 2\,\lambda\,\sqrt{F'_c}\,d \;=\; 2\,(1.0)\,\sqrt{3000\text{ psi}}\,(5.69\text{ in}) \;=\; 7.48\text{ k / ft}$

$\phi V_n = \phi\,V_c \;=\; (0.750)\,(7.48\text{ k / ft}) \;=\; 5.61\text{ k / ft}$

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)

$\psi_e = 1.0$ (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)

$$l_{dh} = \left(\frac{f_y\,\psi_e\,\psi_c\,\psi_r}{50\,\lambda\,\sqrt{F'_c}}\right) d_b = \left[\frac{(60000\text{ psi})(1.0)(0.70)(1.0)}{50\,(1.0)\sqrt{3000\text{ psi}}}\right](0.63\text{ in}) = 9.59\text{ in}$$

$8\,d_b = 8\,(0.63\text{ in}) = 5.0$ (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)

$\psi_t = 1.0$ (bars are not horizontal)

$\psi_e = 1.0$ (bar not epoxy coated)

$\psi_s = 0.80$ (bars are #6 or smaller)

$\lambda = 1.0$ (normal weight concrete)

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

$\text{cover} + d_b / 2 = (2\text{ in}) + (0.63\text{ in}) / 2 = 2.31\text{ in}$

$c_b = 2.31\text{ in}$ (lesser of half spacing, ctr to surface)

$K_{tr} = 0.0$ (no transverse reinforcement)

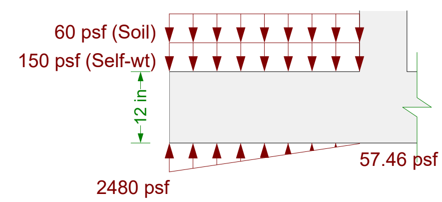
$$\frac{c_b + K_{tr}}{d_b} = \frac{(2.31\text{ in}) + (0.0)}{(0.63\text{ in})} = 3.70$$

$$l_d = \left(\frac{3.}{40}\,\frac{f_y}{\lambda\,\sqrt{F'_c}}\,\frac{\psi_t\,\psi_e\,\psi_s}{2.5}\right) d_b = \left[\frac{3.}{40}\,\frac{(60000\text{ psi})}{(1.0)\sqrt{3000\text{ psi}}}\,\frac{(1.0)(1.0)(0.80)}{2.5}\right](0.63\text{ in}) = 16.43\text{ in}$$

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

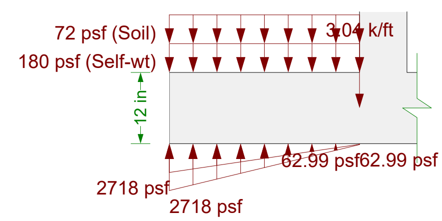
Toe Unfactored Loads

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)

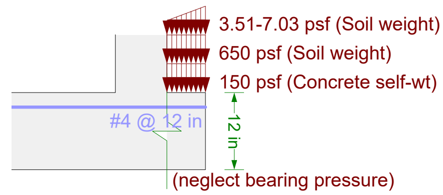
6 inch minimum controls

l_{dh_prov} = 6 in ≥ l_{dh} = 6 in ✓

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

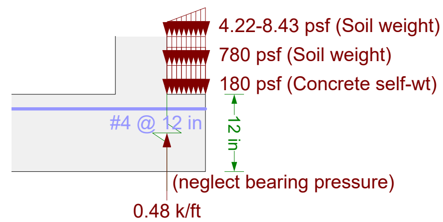
Heel Unfactored Loads

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)

$$\frac{c_b + K_{tr}}{d_b} = \frac{(2.25\text{ in}) + (0.0)}{(0.5\text{ in})} = 4.50$$

$$l_d = \left(\frac{3.}{40}\,\frac{f_y}{\lambda\,\sqrt{F'_c}}\,\frac{\psi_t\,\psi_e\,\psi_s}{2.5}\right) d_b = \left[\frac{3.}{40}\,\frac{(60000\text{ psi})}{(1.0)\,\sqrt{3000\text{ psi}}}\,\frac{(1.0)\,(1.0)\,(0.80)}{2.5}\right](0.5\text{ in}) = 13.15\text{ in}$$

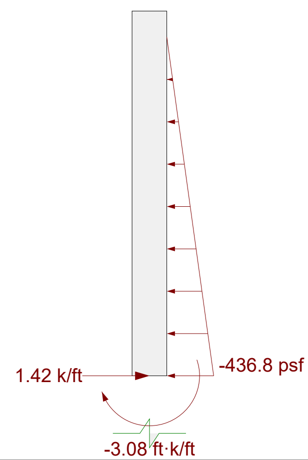
Factoring l_d by the excess reinforcement ratio (0.0141) per 25.4.10: l_d = 0.18 in

12 inch minimum controls

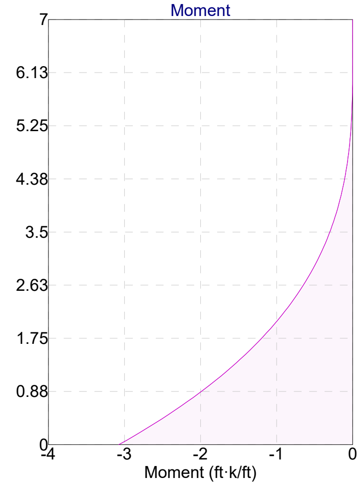
l_{d_prov} = 38.04 in ≥ l_d = 12 in ✓

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

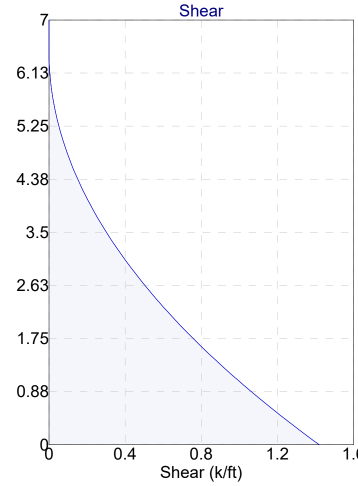
Stem Internal Forces



Stem Internal Forces



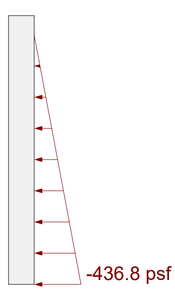
Stem Internal Forces



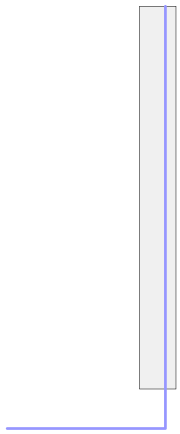
Stem Joint Force Transfer

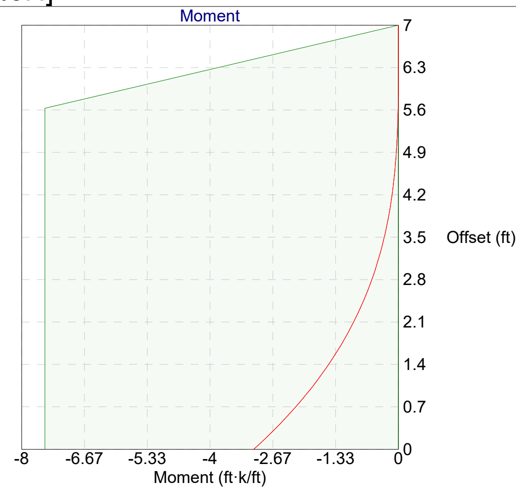
Location	Force
@ stem base	1.42 k/ft

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 ✓

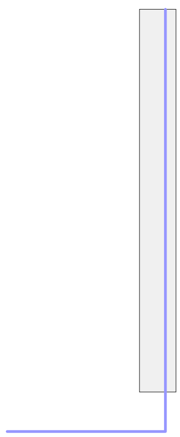
Check (ACI 318-14 11.5.5.1b) @ 5.63 ft from base

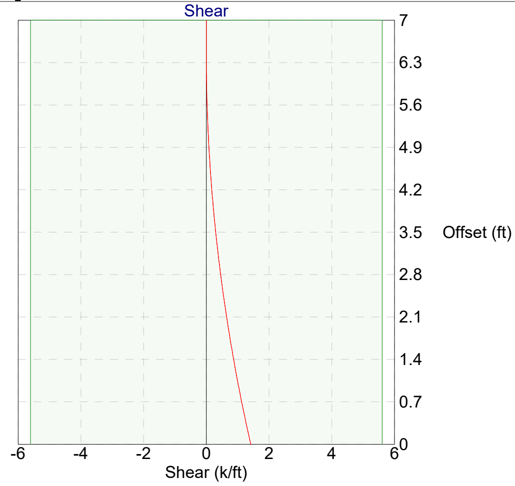
ϕM_n = 7.51 ft·k / ft ≥ M_u = 0.01 ft·k / ft ✓

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]

$\phi M_n = 7.51$ ft·k / ft $\geq$ $(4/3)\,M_u = [4/3]\,(3.08$ 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]

$\phi M_n = 0$ 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)

$\rho_t = 0.0021 \geq \rho_{t\_min} = 0.0020$ ✓

$3\,h = 3\,(8$ 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)

$$\frac{M_u}{\phi M_n} = \frac{(3.08\text{ ft·k/ft})}{(7.51\text{ ft·k/ft})} = 0.4096$$ (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 (0.4096) per 25.4.10: $l_{dh} = 3.93$ in

$8\,d_b = 8\,(0.63$ in$) = 5.0$ (minimum limit, does not control)

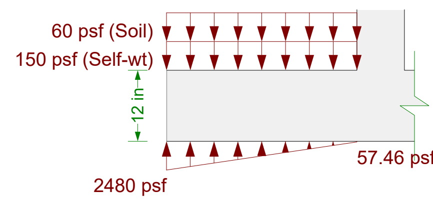
6 inch minimum controls

$l_{dh\_prov} = 9$ in $\geq$ $l_{dh} = 6$ in ✓

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

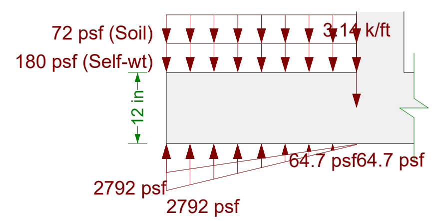
Toe Unfactored Loads

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} = 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)

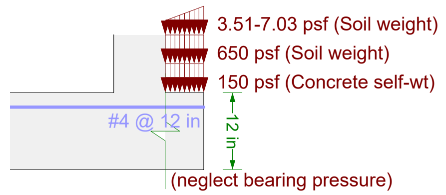
6 inch minimum controls

l_{dh_prov} = 6 in ≥ l_{dh} = 6 in ✓

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

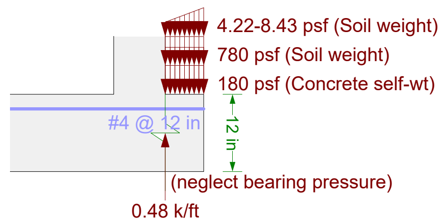
Heel Unfactored Loads

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)

$a = \dfrac{A_s\,f_y}{0.85\,F'_c} = \dfrac{(0.02\text{ in}^2/\text{in})\,(60000\text{ psi})}{0.85\,(3000\text{ psi})} = 0.39\text{ in}$

$\phi M_n = \phi\,A_s\,f_y\,(d - a/2) = (0.90)\,(0.02\text{ in}^2/\text{in})\,(60000\text{ psi})\,[(9.75\text{ in}) - (0.39\text{ in})/2] = 8.6\text{ ft·k/ft}$

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)

$a = \dfrac{A_s\,f_y}{0.85\,F'_c} = \dfrac{(0.02\text{ in}^2/\text{in})\,(60000\text{ psi})}{0.85\,(3000\text{ psi})} = 0.39\text{ in}$

$\varepsilon_t = 0.003\left(\dfrac{d}{a/\beta_1} - 1\right) = 0.003\left[\dfrac{(9.75\text{ in})}{(0.39\text{ in})/(0.850)} - 1\right] = 0.0604$

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)

$\rho_{ST\_prov} = \dfrac{A_{ST}}{t\,s_{ST}} = \dfrac{(0.4\text{ in}^2/\text{in})}{(12\text{ in})\,(12\text{ in})} = 0.0028$

$\rho_{ST\_prov} = \dfrac{A_{ST}}{t\,s_{ST}} = \dfrac{(0.4\text{ in}^2/\text{in})}{(12\text{ in})\,(12\text{ in})} = 0.0028$

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

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)

$\dfrac{M_u}{\phi M_n} = \dfrac{(0.12\text{ ft·k/ft})}{(8.6\text{ ft·k/ft})} = 0.0141$ (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)

$\dfrac{c_b + K_{tr}}{d_b} = \dfrac{(2.25\text{ in}) + (0.0)}{(0.5\text{ in})} = 4.50$

$l_d = \left(\dfrac{3.}{40}\,\dfrac{f_y}{\lambda\,\sqrt{F'_c}}\,\dfrac{\psi_t\,\psi_e\,\psi_s}{2.5}\right) d_b = \left[\dfrac{3.}{40}\,\dfrac{(60000\text{ psi})}{(1.0)\,\sqrt{3000\text{ psi}}}\,\dfrac{(1.0)\,(1.0)\,(0.80)}{2.5}\right](0.5\text{ in}) = 13.15\text{ in}$

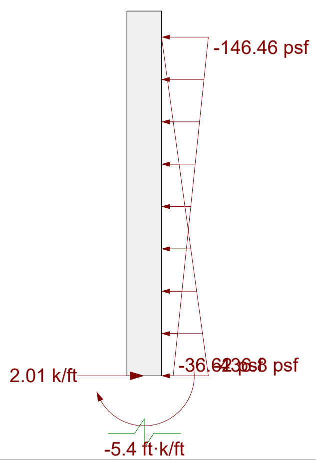
Factoring l_d by the excess reinforcement ratio (0.0141) per 25.4.10: l_d = 0.18 in

12 inch minimum controls

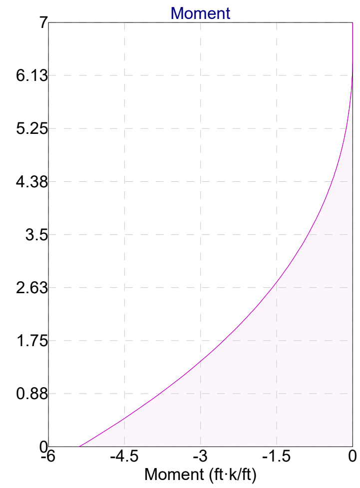
l_{d_prov} = 38.04 in ≥ l_d = 12 in ✓

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

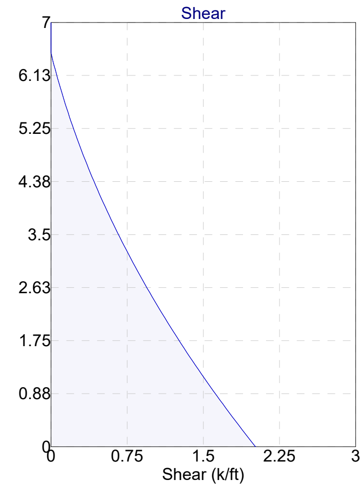
Stem Internal Forces



Stem Internal Forces



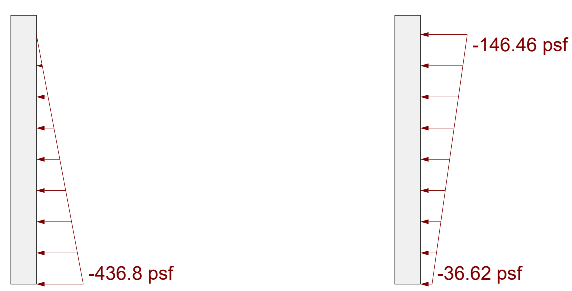
Stem Internal Forces



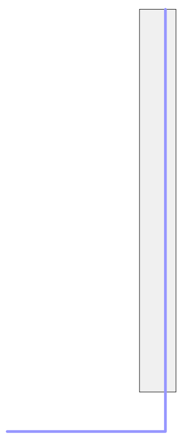
Stem Joint Force Transfer

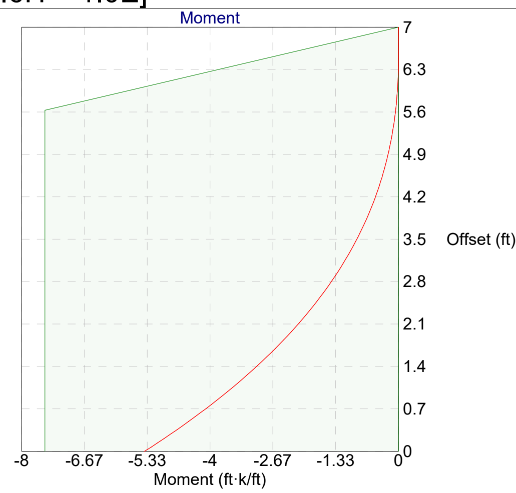
Location	Force
@ stem base	2.01 k/ft

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 ✓

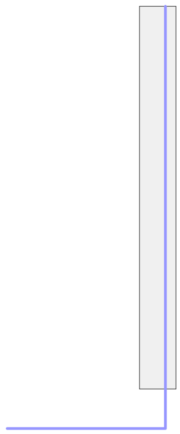
Check (ACI 318-14 11.5.5.1b) @ 5.63 ft from base

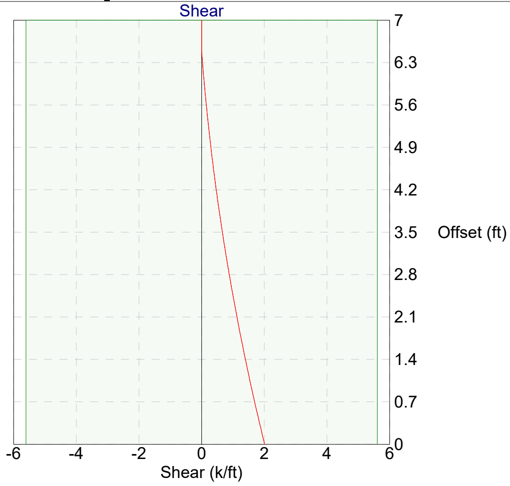
ϕM_n = 7.51 ft·k / ft ≥ M_u = 0.06 ft·k / ft ✓

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]

$\phi M_n = 7.51 \text{ ft·k / ft} \geq (4/3)\, M_u = [4/3]\,(5.4 \text{ ft·k / ft}) = 7.2 \text{ ft·k / ft}$

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]

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

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]

$\beta_1 = 0.850 \quad (F'_c \leq 4000 \text{ psi})$

$$a = \frac{A_s\, f_y}{0.85\, F'_c} = \frac{(0.03 \text{ in}^2/\text{in})\,(60000 \text{ psi})}{0.85\,(3000 \text{ psi})} = 0.61 \text{ in}$$

$$\varepsilon_t = 0.003\left(\frac{d}{a/\beta_1} - 1\right) = 0.003\left[\frac{(5.69 \text{ in})}{(0.61 \text{ in})/(0.850)} - 1\right] = 0.0209$$

$\varepsilon_t = 0.0209 \geq 0.004$ ✓

Maximum Steel Check (ACI 318-14 9.3.3.1) @ 7 ft from base [Stem in negative flexure]

$\beta_1 = 0.850 \quad (F'_c \leq 4000 \text{ psi})$

$$a = \frac{A_s\, f_y}{0.85\, F'_c} = \frac{(0.03 \text{ in}^2/\text{in})\,(60000 \text{ psi})}{0.85\,(3000 \text{ psi})} = 0.61 \text{ in}$$

$$\varepsilon_t = 0.003\left(\frac{d}{a/\beta_1} - 1\right) = 0.003\left[\frac{(5.69 \text{ in})}{(0.61 \text{ in})/(0.850)} - 1\right] = 0.0209$$

$\varepsilon_t = 0.0209 \geq 0.004$ ✓

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

$$\rho_t = \frac{A_{s\_horz} / s_{horz}}{t} = \frac{(0.2 \text{ in}^2)/(12 \text{ in})}{(8 \text{ in})} = 0.0021$$

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

$\rho_t = 0.0021 \geq \rho_{t\_min} = 0.0020$ ✓

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

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)

$$\frac{M_u}{\phi M_n} = \frac{(5.4 \text{ ft·k / ft})}{(7.51 \text{ ft·k / ft})} = 0.7185 \quad \text{(ratio to represent excess reinforcement)}$$

$\psi_e = 1.0 \quad$ (uncoated hooked bars)

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

$\psi_r = 1.0 \quad$ (no confining reinforcement)

$\lambda = 1.0 \quad$ (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: $l_{dh} = 6.89 \text{ in}$

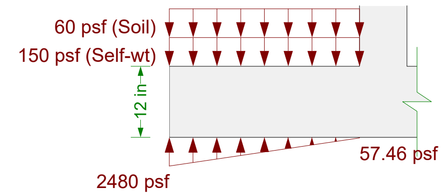
$8\,d_b = 8\,(0.63 \text{ in}) = 5.0 \quad$ (minimum limit, does not control)

$l_{dh\_prov} = 9 \text{ in} \geq l_{dh} = 6.89 \text{ in}$ ✓

Toe Checks [1.4D]

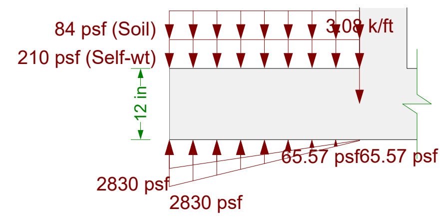
Toe Unfactored Loads

Unfactored Loads



Toe Factored Loads

1.4D



Controlling Moment

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

M_{toe} = 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

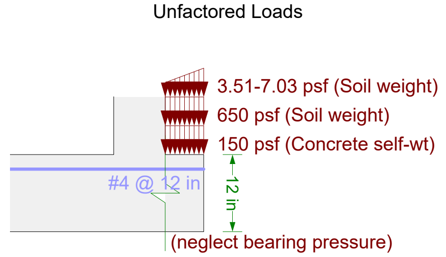
8 d_b = 8 (0.63 in) = 5.0 (minimum limit, does not control)

6 inch minimum controls

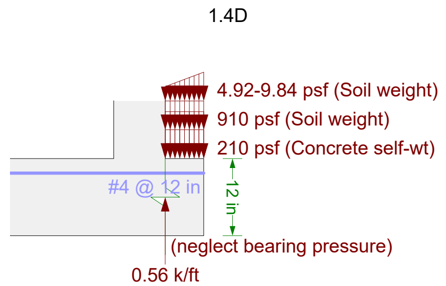
l_{dh_prov} = 6 in ≥ l_{dh} = 6 in ✓

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)

$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0.02 \text{ in}^2/\text{in})(60000 \text{ psi})}{0.85 (3000 \text{ psi})} = 0.39 \text{ in}$

$\phi M_n = \phi A_s f_y (d - a/2) = (0.90)(0.02 \text{ in}^2/\text{in})(60000 \text{ psi})[(9.75 \text{ in}) - (0.39 \text{ in})/2] = 8.6 \text{ ft·k/ft}$

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 = 0.56 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)

$\frac{c_b + K_{tr}}{d_b} = \frac{(2.25 \text{ in}) + (0.0)}{(0.5 \text{ in})} = 4.50$

$l_d = \left(\frac{3.}{40} \frac{f_y}{\lambda \sqrt{F'_c}} \frac{\psi_t \psi_e \psi_s}{2.5}\right) d_b = \left[\frac{3.}{40} \frac{(60000 \text{ psi})}{(1.0)\sqrt{3000 \text{ psi}}} \frac{(1.0)(1.0)(0.80)}{2.5}\right] (0.5 \text{ in}) = 13.15 \text{ in}$

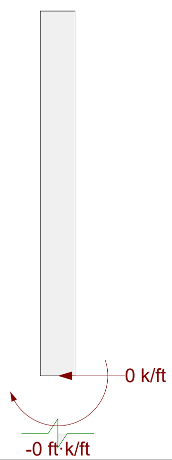
Factoring l_d by the excess reinforcement ratio (- 0.0000) per 25.4.10: l_d = - 0 in

12 inch minimum controls

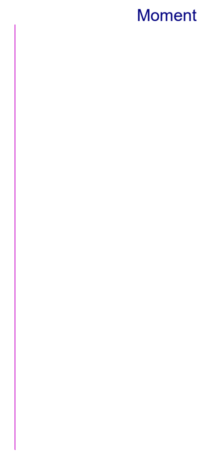
l_{d_prov} = 38.04 in ≥ l_d = 12 in ✓

Stem Forces [1.4D]

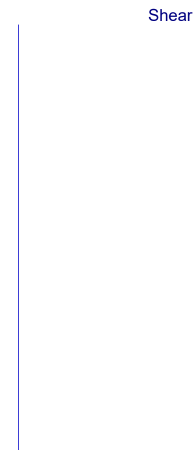
Stem Internal Forces



Stem Internal Forces



Stem Internal Forces

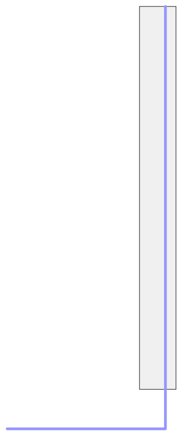


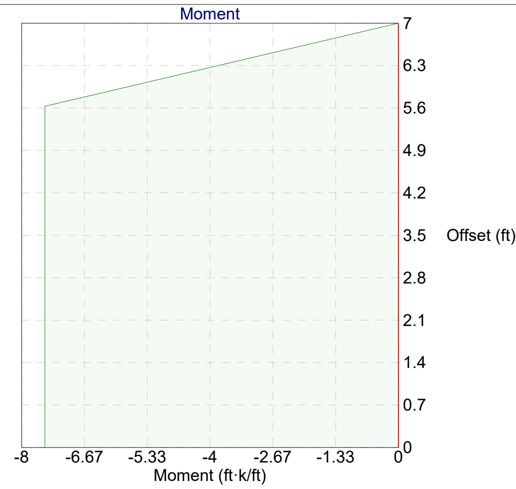
Stem Joint Force Transfer

Location	Force
@ stem base	0 k/ft

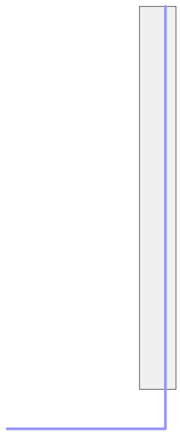
Stem Internal Forces

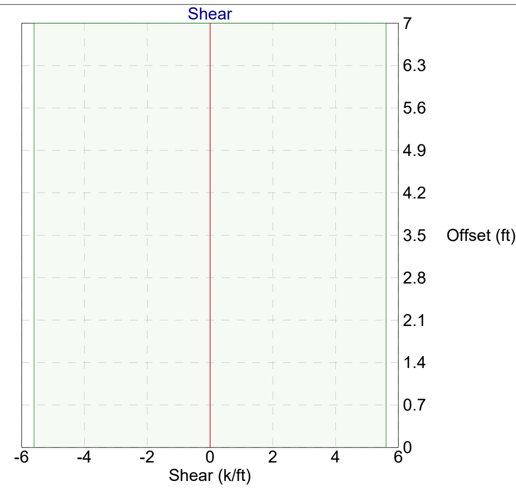
Stem Moment Checks [1.4D]





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]

$\phi M_n = 7.51\ \text{ft·k / ft} \quad \geq \quad (4/3)\,M_u = [4/3]\,(0\ \text{ft·k / ft}) = 0\ \text{ft·k / ft}$

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]

$\phi M_n = 0\ \text{ft·k / ft} \quad \geq \quad (4/3)\,M_u = [4/3]\,(0\ \text{ft·k / ft}) = 0\ \text{ft·k / ft}$

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]

$\beta_1 = 0.850 \qquad (F'_c \leq 4000\ \text{psi})$

$$a = \frac{A_s\,f_y}{0.85\,F'_c} = \frac{(0.03\ \text{in}^2/\text{in})\,(60000\ \text{psi})}{0.85\,(3000\ \text{psi})} = 0.61\ \text{in}$$

$$\varepsilon_t = 0.003\left(\frac{d}{a/\beta_1} - 1\right) = 0.003\left[\frac{(5.69\ \text{in})}{(0.61\ \text{in})/(0.850)} - 1\right] = 0.0209$$

$\varepsilon_t = 0.0209 \quad \geq \quad 0.004$ ✓

Maximum Steel Check (ACI 318-14 9.3.3.1) @ 7 ft from base [Stem in negative flexure]

$\beta_1 = 0.850 \qquad (F'_c \leq 4000\ \text{psi})$

$$a = \frac{A_s\,f_y}{0.85\,F'_c} = \frac{(0.03\ \text{in}^2/\text{in})\,(60000\ \text{psi})}{0.85\,(3000\ \text{psi})} = 0.61\ \text{in}$$

$$\varepsilon_t = 0.003\left(\frac{d}{a/\beta_1} - 1\right) = 0.003\left[\frac{(5.69\ \text{in})}{(0.61\ \text{in})/(0.850)} - 1\right] = 0.0209$$

$\varepsilon_t = 0.0209 \quad \geq \quad 0.004$ ✓

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

$$\rho_t = \frac{A_{s\_horz}/s_{horz}}{t} = \frac{(0.2\ \text{in}^2)/(12\ \text{in})}{(8\ \text{in})} = 0.0021$$

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

$\rho_t = 0.0021 \quad \geq \quad \rho_{t\_min} = 0.0020$ ✓

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

18 inch limit governs

$s_{horz} = 12\ \text{in} \quad \leq \quad s_{horz\_max} = 18\ \text{in}$ ✓

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

$$\frac{M_u}{\phi M_n} = \frac{(0\ \text{ft·k / ft})}{(7.51\ \text{ft·k / ft})} = 0.0 \qquad \text{(ratio to represent excess reinforcement)}$$

$\psi_e = 1.0 \qquad$ (uncoated hooked bars)

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

$\psi_r = 1.0 \qquad$ (no confining reinforcement)

$\lambda = 1.0 \qquad$ (normal weight concrete)

$$l_{dh} = \left(\frac{f_y\,\psi_e\,\psi_c\,\psi_r}{50\,\lambda\,\sqrt{F'_c}}\right) d_b = \left[\frac{(60000\ \text{psi})\,(1.0)\,(0.70)\,(1.0)}{50\,(1.0)\,\sqrt{3000\ \text{psi}}}\right](0.63\ \text{in}) = 9.59\ \text{in}$$

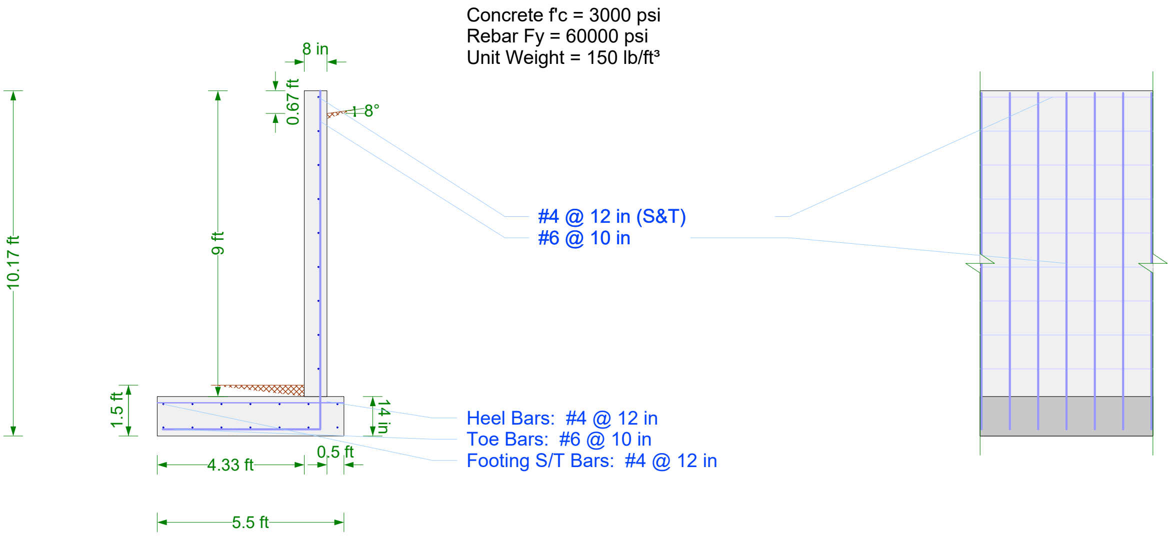
Factoring l_{dh} by the excess reinforcement ratio (0.0000) per 25.4.10: $l_{dh} = 0\ \text{in}$

$8\,d_b = 8\,(0.63\ \text{in}) = 5.0 \qquad$ (minimum limit, does not control)

6 inch minimum controls

$l_{dh\_prov} = 9\ \text{in} \quad \geq \quad l_{dh} = 6\ \text{in}$ ✓

Design Detail



Check Summary

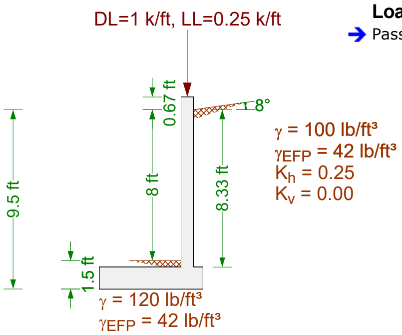
Ratio	Check	Provided	Required	Combination
----- Stability Checks -----				
✓ 0.964	Overturning	1.25	1.20	1.0D + 1.0L + 1.0H + 0.7E
✓ 0.946	Bearing Pressure	2500 psf	2366 psf	1.0D + 1.0L + 1.0H + 0.7E
✓ 0.577	Bearing Eccentricity	19.03 in	32.98 in	1.0D + 1.0L + 1.0H + 0.7E
----- Toe Checks -----				
✓ 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

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

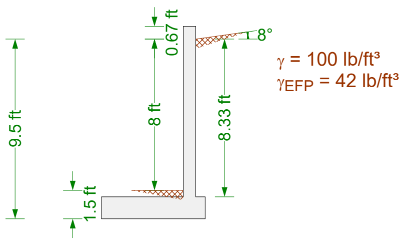
Load Combination	Stem Actual steel (in²/in)	Heel M-applied (ft·k/ft)	Heel M-allow (ft·k/ft)	Heel V-applied (k/ft)	Heel V-allow (k/ft)	Toe M-applied (ft·k/ft)	Toe M-allow (ft·k/ft)	Toe V-applied (k/ft)	Toe V-allow (k/ft)
1.2D + 1.6L + 1.6H	0.04	0.15	10.4	0.61	11.58	11.98	26.39	3.67	11.46
1.2D + 1.6L + 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



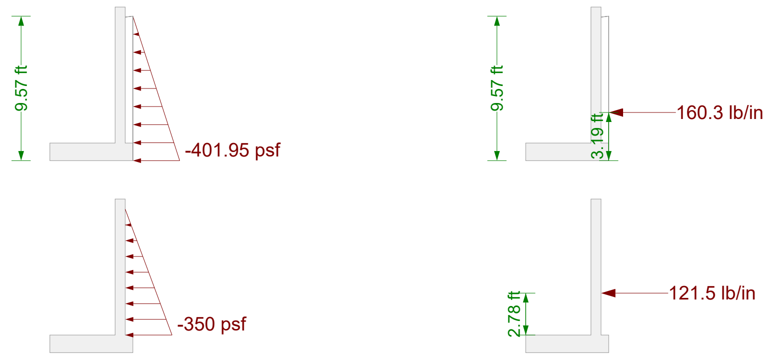
Lateral Earth Pressure

Equivalent Fluid Pressure

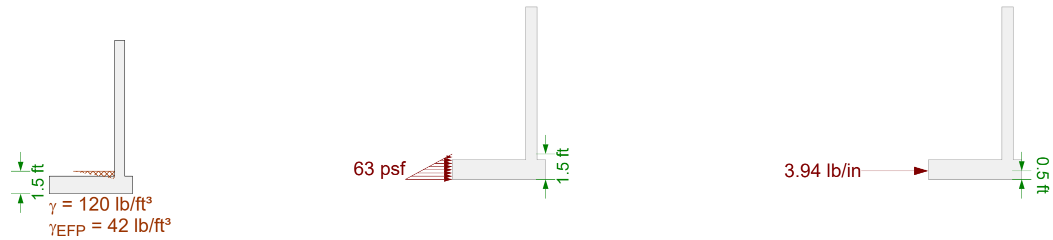
$\sigma_h = H\,\gamma_{fluid} = (9.57\text{ ft})(42\text{ lb / ft}^3) = 402\text{ psf}$

Lateral Earth Pressure (stem only)

$\sigma_h = H\,\gamma_{fluid} = (8.33\text{ ft})(42\text{ lb / ft}^3) = 350\text{ psf}$



Passive Pressure

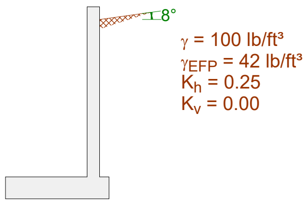


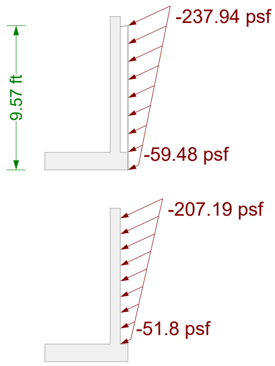
Lateral Earth Pressure

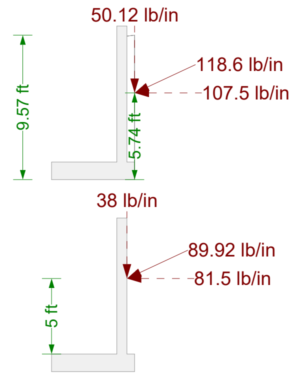
Equivalent Fluid Pressure

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

Seismic Pressure







Seismic Pressure

Dynamic + static force (Mononobe - Okabe equation)

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

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

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

$= 0.6403$

$P_{ae} = \frac{1}{2} K_{ae}\gamma H^2 (1 - k_v) = \frac{1}{2}(0.6403)(100\ \text{lb}/\text{ft}^3)(9.57\ \text{ft})^2[1 - (0.0)] = 244.4\ \text{lb}/\text{in}$

Static - only force (Coulomb equation)

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

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

$= 0.3295$

$P_a = \frac{1}{2} K_a \gamma H^2 = \frac{1}{2}(0.3295)(100\ \text{lb}/\text{ft}^3)(9.57\ \text{ft})^2 = 125.8\ \text{lb}/\text{in}$

Net dynamic force

$\Delta P_{ae} = P_{ae} - P_a = (244.4\ \text{lb}/\text{in}) - (125.8\ \text{lb}/\text{in}) = 118.6\ \text{lb}/\text{in}$

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

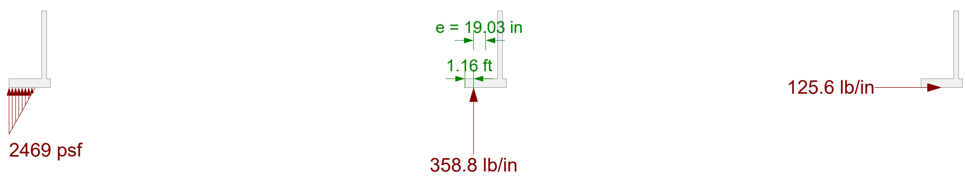
To arrive at the pressure distribution illustrated above (used to determine stem moments),
apply inverted triangular pressure plus a uniform portion to bring resultant to 0.6H

$\sigma_{e\_top} = \frac{8}{5}\frac{\Delta P_{ae}}{H} = \frac{8}{5}\frac{(118.6\ \text{lb}/\text{in})}{(9.57\ \text{ft})} = 237.9\ \text{psf}$

$\sigma_{e\_bot} = \frac{2}{5}\frac{\Delta P_{ae}}{H} = \frac{2}{5}\frac{(118.6\ \text{lb}/\text{in})}{(9.57\ \text{ft})} = 59.48\ \text{psf}$

Wall/Soil Weights

Bearing Pressure



Friction

F = μ 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

$$\frac{-\ 60047.59\ \text{in·lb / ft}}{-\ 358.75\ \text{lb / in}} = 1.16\ \text{ft}$$

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

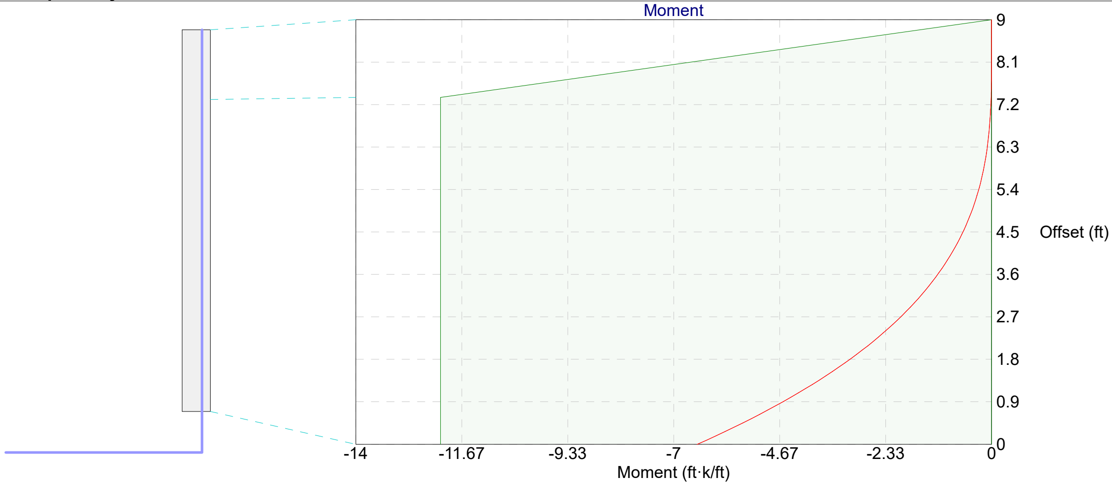
Bearing pressure < allowable (2366 psf < 2500 psf) - OK
Bearing resultant eccentricity < allowable (19.03 in < 32.98 in) - OK

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

$$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0.04 \text{ in}^2 / \text{in})(60000 \text{ psi})}{0.85 (3000 \text{ psi})} = 1.04 \text{ in}$$

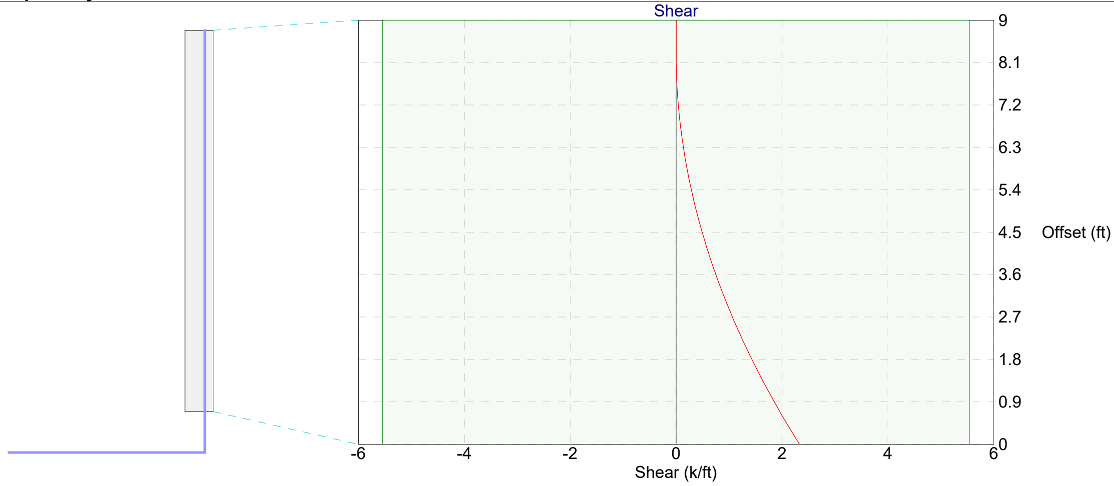
$$\phi M_n = \phi A_s f_y (d - a / 2) = (0.90)(0.04 \text{ in}^2 / \text{in})(60000 \text{ psi})[(5.63 \text{ in}) - (1.04 \text{ in}) / 2] = 12.14 \text{ ft·k / ft}$$

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)

$V_c = 2\,\lambda\,\sqrt{F'_c}\,d \;=\; 2\,(1.0)\,\sqrt{3000\text{ psi}}\,(5.63\text{ in}) \;=\; 7.39\text{ k / ft}$

$\phi V_n = \phi\,V_c \;=\; (0.750)\,(7.39\text{ k / ft}) \;=\; 5.55\text{ k / ft}$

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)

$V_c = 2\,\lambda\,\sqrt{F'_c}\,d \;=\; 2\,(1.0)\,\sqrt{3000\text{ psi}}\,(5.63\text{ in}) \;=\; 7.39\text{ k / ft}$

$\phi V_n = \phi\,V_c \;=\; (0.750)\,(7.39\text{ k / ft}) \;=\; 5.55\text{ k / ft}$

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)

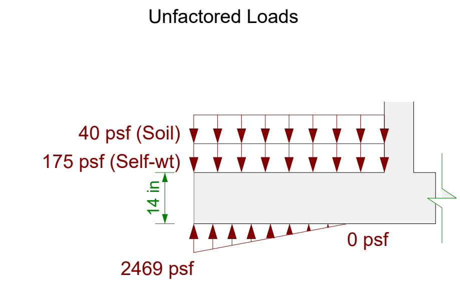
K_{tr} = 0.0 (no transverse reinforcement)

$$\frac{c_b + K_{tr}}{d_b} = \frac{(2.38\text{ in}) + (0.0)}{(0.75\text{ in})} = 3.1667$$

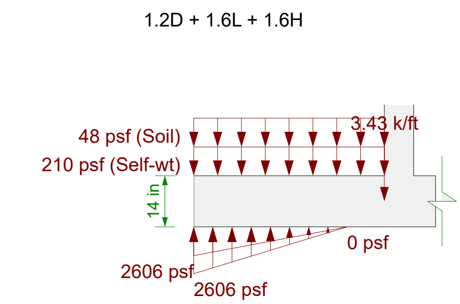
$$l_d = \left(\frac{3.}{40} \frac{f_y}{\lambda \sqrt{F'_c}} \frac{\psi_t\, \psi_e\, \psi_s}{2.5} \right) d_b = \left[\frac{3.}{40} \frac{(60000\text{ psi})}{(1.0)\sqrt{3000\text{ psi}}} \frac{(1.0)\,(1.0)\,(0.80)}{2.5} \right] (0.75\text{ in}) = 19.72\text{ in}$$

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

Toe Unfactored Loads



Toe Factored Loads



Controlling Moment

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

$M_{toe} = 11.98$ ft·k / ft $\geq$ $M_{stem} = 6.48$ 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)

$$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0.04 \text{ in}^2/\text{in})(60000 \text{ psi})}{0.85 (3000 \text{ psi})} = 1.04 \text{ in}$$

$$\phi M_n = \phi A_s f_y (d - a/2) = (0.90)(0.04 \text{ in}^2/\text{in})(60000 \text{ psi})[(11.63 \text{ in}) - (1.04 \text{ in})/2] = 26.39 \text{ ft·k/ft}$$

$\phi M_n = 26.39$ ft·k / ft $\geq$ $M_u = 6.48$ 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)

$\lambda = 1.0$ (normal weight concrete)

$$V_c = 2 \lambda \sqrt{F'_c}\, d = 2 (1.0) \sqrt{3000 \text{ psi}} (11.63 \text{ in}) = 15.28 \text{ k/ft}$$

$$\phi V_n = \phi V_c = (0.750)(15.28 \text{ k/ft}) = 11.46 \text{ k/ft}$$

$\phi V_n = 11.46$ k / ft $\geq$ $V_u = 3.67$ k / ft ✓

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

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

$$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0.04 \text{ in}^2/\text{in})(60000 \text{ psi})}{0.85 (3000 \text{ psi})} = 1.04 \text{ in}$$

$$\varepsilon_t = 0.003 \left(\frac{d}{a/\beta_1} - 1\right) = 0.003 \left[\frac{(11.63 \text{ in})}{(1.04 \text{ in})/(0.850)} - 1\right] = 0.0256$$

$\varepsilon_t = 0.0256 \geq 0.004$ ✓

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

$\phi M_n = 26.39$ ft·k / ft $\geq$ $(4/3) M_u = [4/3](6.48 \text{ ft·k/ft}) = 8.64$ 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)

$$\rho_{ST\_prov} = \frac{A_{ST}}{t\, s_{ST}} = \frac{(0.4 \text{ in}^2/\text{in})}{(14 \text{ in})(12 \text{ in})} = 0.0024$$

$$\rho_{ST\_prov} = \frac{A_{ST}}{t\, s_{ST}} = \frac{(0.4 \text{ in}^2/\text{in})}{(14 \text{ in})(12 \text{ in})} = 0.0024$$

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

$\rho_{ST\_min} = 0.0018$

$\rho_{ST\_prov} = 0.0024 \geq \rho_{ST\_min} = 0.0018$ ✓

18 inch limit governs

$s_{ST\_max} = 18$ in

$s_{ST} = 12$ in $\leq$ $s_{ST\_max} = 18$ 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)

$\psi_e = 1.0$ (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 (0.2456) per 25.4.10: $l_{dh} = 2.82$ in

$8\, d_b = 8 (0.75 \text{ in}) = 6.0$ (minimum limit, does not control)

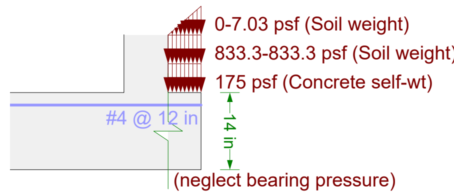
6 inch minimum controls

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

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

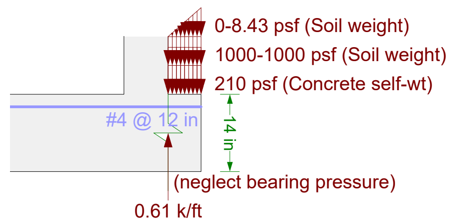
Heel Unfactored Loads

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)

$$\frac{c_b + K_{tr}}{d_b} = \frac{(2.25\ \text{in}) + (0.0)}{(0.5\ \text{in})} = 4.50$$

$$l_d = \left(\frac{3.}{40}\,\frac{f_y}{\lambda\sqrt{F'_c}}\,\frac{\psi_t\,\psi_e\,\psi_s}{2.5}\right) d_b = \left[\frac{3.}{40}\,\frac{(60000\ \text{psi})}{(1.0)\sqrt{3000\ \text{psi}}}\,\frac{(1.0)(1.0)(0.80)}{2.5}\right](0.5\ \text{in}) = 13.15\ \text{in}$$

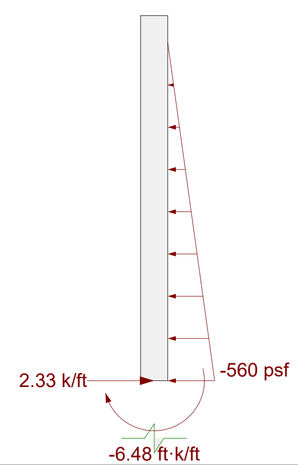
Factoring l_d by the excess reinforcement ratio (0.0146) per 25.4.10: l_d = 0.19 in

12 inch minimum controls

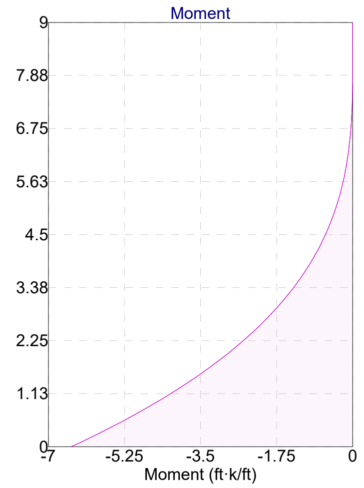
l_{d_prov} = 57.96 in ≥ l_d = 12 in ✓

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

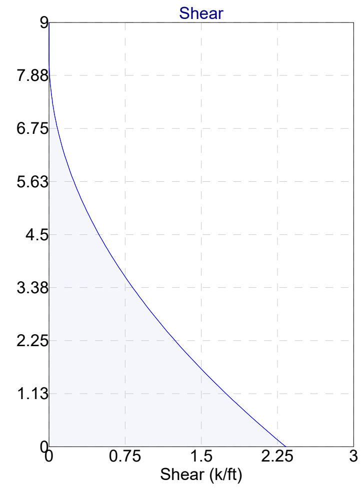
Stem Internal Forces



Stem Internal Forces



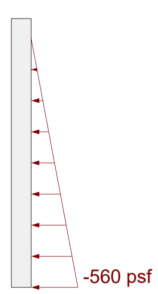
Stem Internal Forces



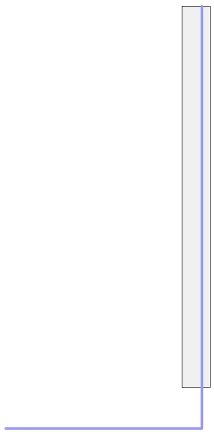
Stem Joint Force Transfer

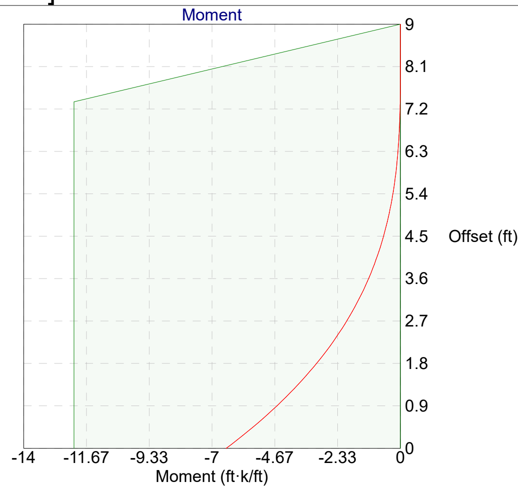
Location	Force
@ stem base	2.33 k/ft

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 ✓

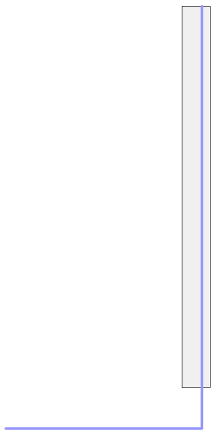
Check (ACI 318-14 11.5.5.1b) @ 7.36 ft from base

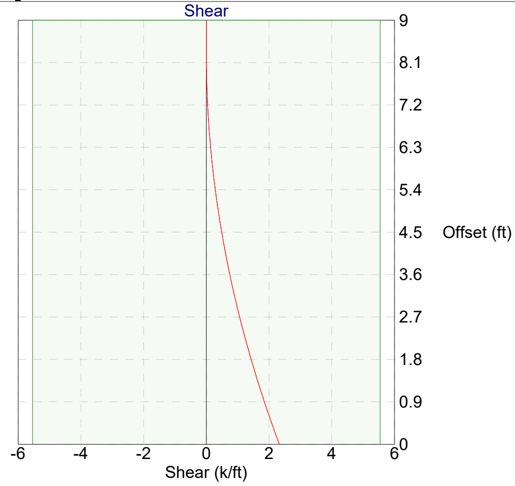
ϕM_n = 12.14 ft·k / ft ≥ M_u = 0.01 ft·k / ft ✓

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 $\geq$ 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]

$\phi M_n = 12.14$ ft·k / ft $\geq$ $(4 / 3)\, M_u = [4 / 3]\,(6.48$ 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]

$\phi M_n = 0$ 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]

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

$$a = \frac{A_s\, f_y}{0.85\, F'_c} = \frac{(0.04 \text{ in}^2 / \text{in})\,(60000 \text{ psi})}{0.85\,(3000 \text{ psi})} = 1.04 \text{ in}$$

$$\varepsilon_t = 0.003\left(\frac{d}{a / \beta_1} - 1\right) = 0.003\left[\frac{(5.63 \text{ in})}{(1.04 \text{ in}) / (0.850)} - 1\right] = 0.0109$$

$\varepsilon_t = 0.0109 \geq 0.004$ ✓

Maximum Steel Check (ACI 318-14 9.3.3.1) @ 9 ft from base [Stem in negative flexure]

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

$$a = \frac{A_s\, f_y}{0.85\, F'_c} = \frac{(0.04 \text{ in}^2 / \text{in})\,(60000 \text{ psi})}{0.85\,(3000 \text{ psi})} = 1.04 \text{ in}$$

$$\varepsilon_t = 0.003\left(\frac{d}{a / \beta_1} - 1\right) = 0.003\left[\frac{(5.63 \text{ in})}{(1.04 \text{ in}) / (0.850)} - 1\right] = 0.0109$$

$\varepsilon_t = 0.0109 \geq 0.004$ ✓

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)

$\rho_t = 0.0021 \geq \rho_{t\_min} = 0.0020$ ✓

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

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)

$$\frac{M_u}{\phi M_n} = \frac{(6.48 \text{ ft·k / ft})}{(12.14 \text{ ft·k / ft})} = 0.5341$$ (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)

$$l_{dh} = \left(\frac{f_y\, \psi_e\, \psi_c\, \psi_r}{50\, \lambda\, \sqrt{F'_c}}\right) d_b = \left[\frac{(60000 \text{ psi})\,(1.0)\,(0.70)\,(1.0)}{50\,(1.0)\,\sqrt{3000 \text{ psi}}}\right](0.75 \text{ in}) = 11.5 \text{ in}$$

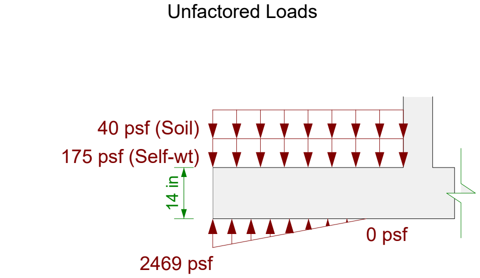
Factoring l_{dh} by the excess reinforcement ratio (0.5341) per 25.4.10: $l_{dh} = 6.14$ in

$8\, d_b = 8\,(0.75 \text{ in}) = 6.0$ (minimum limit, does not control)

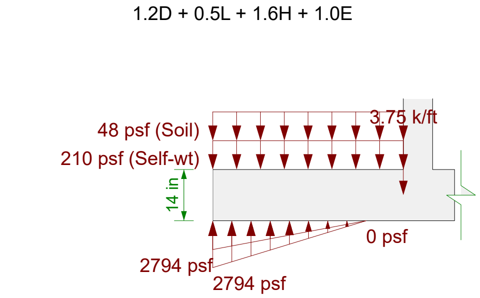
$l_{dh\_prov} = 12 \text{ in} \geq l_{dh} = 6.14 \text{ in}$ ✓

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

Toe Unfactored Loads



Toe Factored Loads



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)

$$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0.04 \text{ in}^2/\text{in})(60000 \text{ psi})}{0.85 (3000 \text{ psi})} = 1.04 \text{ in}$$

$$\phi M_n = \phi A_s f_y (d - a/2) = (0.90)(0.04 \text{ in}^2/\text{in})(60000 \text{ psi})[(11.63 \text{ in}) - (1.04 \text{ in})/2] = 26.39 \text{ ft·k/ft}$$

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)

$$V_c = 2 \lambda \sqrt{F'_c}\, d = 2 (1.0) \sqrt{3000 \text{ psi}} (11.63 \text{ in}) = 15.28 \text{ k/ft}$$

$$\phi V_n = \phi V_c = (0.750)(15.28 \text{ k/ft}) = 11.46 \text{ k/ft}$$

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]

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)

$$a = \frac{A_s\, f_y}{0.85\, F'_c} = \frac{(0.02 \text{ in}^2/\text{in})(60000 \text{ psi})}{0.85\,(3000 \text{ psi})} = 0.39 \text{ in}$$

$$\phi M_n = \phi\, A_s\, f_y\,(d - a/2) = (0.90)(0.02 \text{ in}^2/\text{in})(60000 \text{ psi})\,[(11.75 \text{ in}) - (0.39 \text{ in})/2] = 10.4 \text{ ft·k/ft}$$

$\phi M_n = 10.4$ ft·k / ft $\geq$ $M_u = 0.15$ 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)

$\lambda = 1.0$ (normal weight concrete)

$$V_c = 2\,\lambda\,\sqrt{F'_c}\, d = 2\,(1.0)\,\sqrt{3000 \text{ psi}}\,(11.75 \text{ in}) = 15.45 \text{ k/ft}$$

$$\phi V_n = \phi\, V_c = (0.750)(15.45 \text{ k/ft}) = 11.58 \text{ k/ft}$$

$\phi V_n = 11.58$ k / ft $\geq$ $V_u = 0.61$ k / ft ✓

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

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

$$a = \frac{A_s\, f_y}{0.85\, F'_c} = \frac{(0.02 \text{ in}^2/\text{in})(60000 \text{ psi})}{0.85\,(3000 \text{ psi})} = 0.39 \text{ in}$$

$$\varepsilon_t = 0.003\left(\frac{d}{a/\beta_1} - 1\right) = 0.003\left[\frac{(11.75 \text{ in})}{(0.39 \text{ in})/(0.850)} - 1\right] = 0.0734$$

$\varepsilon_t = 0.0734$ $\geq$ 0.004 ✓

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

$\phi M_n = 10.4$ ft·k / ft $\geq$ $(4/3)\, M_u = [4/3]\,(0.15$ 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)

$$\rho_{ST\_prov} = \frac{A_{ST}}{t\, s_{ST}} = \frac{(0.4 \text{ in}^2/\text{in})}{(14 \text{ in})(12 \text{ in})} = 0.0024$$

$$\rho_{ST\_prov} = \frac{A_{ST}}{t\, s_{ST}} = \frac{(0.4 \text{ in}^2/\text{in})}{(14 \text{ in})(12 \text{ in})} = 0.0024$$

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

$\rho_{ST\_min} = 0.0018$

$\rho_{ST\_prov} = 0.0024$ $\geq$ $\rho_{ST\_min} = 0.0018$ ✓

18 inch limit governs

$s_{ST\_max} = 18$ in

$s_{ST} = 12$ in $\leq$ $s_{ST\_max} = 18$ in ✓

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

$$\frac{M_u}{\phi M_n} = \frac{(0.15 \text{ ft·k/ft})}{(10.4 \text{ ft·k/ft})} = 0.0146 \quad \text{(ratio to represent excess reinforcement)}$$

$\psi_t = 1.0$ (12 inches or less cast below - 11.50 inches)

$\psi_e = 1.0$ (bar not epoxy coated)

$\psi_s = 0.80$ (bars are #6 or smaller)

$\lambda = 1.0$ (normal weight concrete)

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

cover + $d_b / 2 = (2 \text{ in}) + (0.5 \text{ in}) / 2 = 2.25$ in

$c_b = 2.25$ in (lesser of half spacing, ctr to surface)

$K_{tr} = 0.0$ (no transverse reinforcement)

$$\frac{c_b + K_{tr}}{d_b} = \frac{(2.25 \text{ in}) + (0.0)}{(0.5 \text{ in})} = 4.50$$

$$l_d = \left(\frac{3.}{40}\,\frac{f_y}{\lambda\,\sqrt{F'_c}}\,\frac{\psi_t\,\psi_e\,\psi_s}{2.5}\right) d_b = \left[\frac{3.}{40}\,\frac{(60000 \text{ psi})}{(1.0)\,\sqrt{3000 \text{ psi}}}\,\frac{(1.0)(1.0)(0.80)}{2.5}\right](0.5 \text{ in}) = 13.15 \text{ in}$$

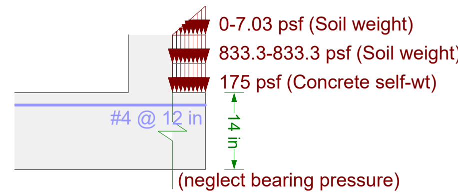
Factoring l_d by the excess reinforcement ratio (0.0146) per 25.4.10: $l_d = 0.19$ in

12 inch minimum controls

$l_{d\_prov} = 57.96$ in $\geq$ $l_d = 12$ in ✓

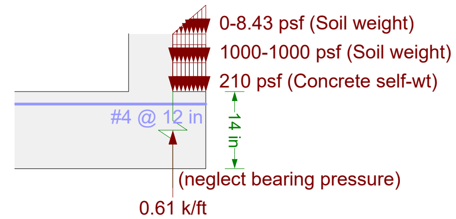
Heel Unfactored Loads

Unfactored Loads



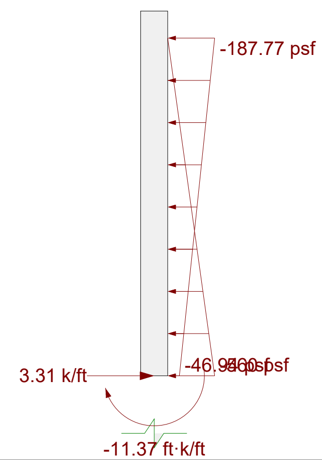
Heel Factored Loads

1.2D + 0.5L + 1.6H + 1.0E

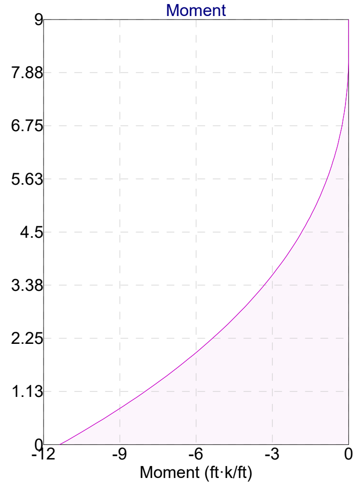


Stem Forces [1.2D + 0.5L + 1.6H + 1.0E]

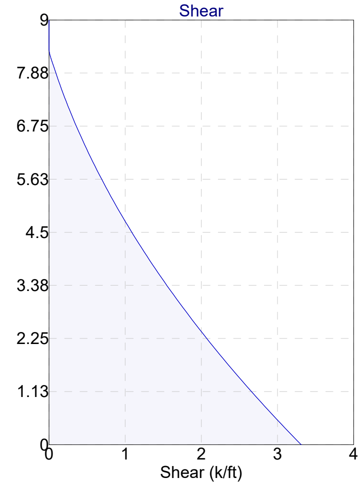
Stem Internal Forces



Stem Internal Forces



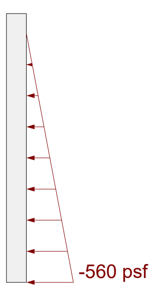
Stem Internal Forces

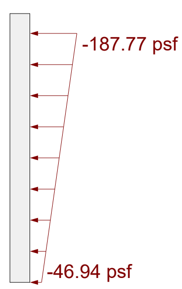


Stem Joint Force Transfer

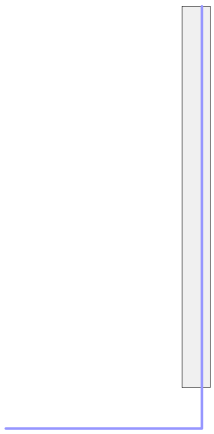
Location	Force
@ stem base	3.31 k/ft

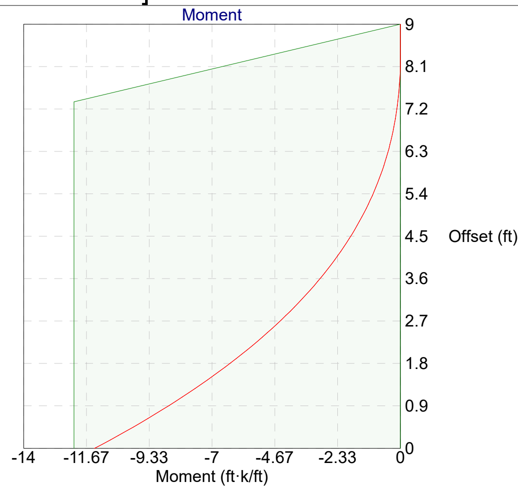
Stem Internal Forces





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



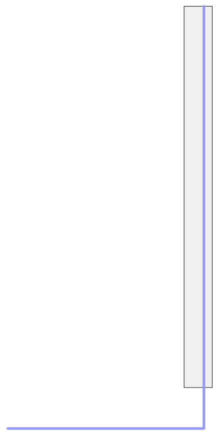


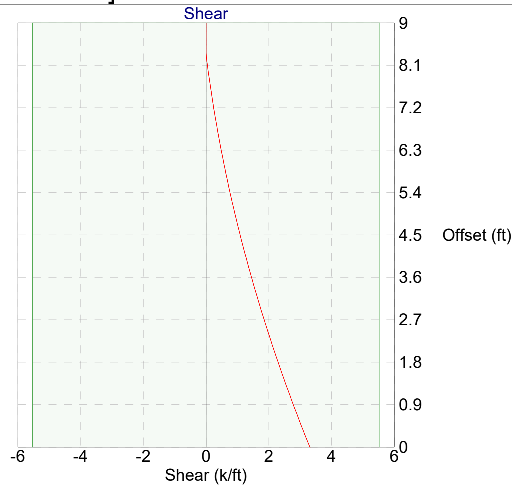
Check (ACI 318-14 11.5.5.1b) @ 0 ft from base
ϕM_n = 12.14 ft·k / ft ≥ M_u = 11.37 ft·k / ft ✓

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]

ϕM_n = 12.14 ft·k / ft < (4 / 3) M_u = [4 / 3] (11.37 ft·k / ft) = 15.16 ft·k / ft

$$A_{s\_min} = \frac{3\sqrt{F'_c}}{f_y} d = \frac{3\sqrt{3000\text{ psi}}}{(60000\text{ psi})}(5.63\text{ in}) = 0.02\text{ in}^2/\text{in}$$

200 d / f_y = 200 (5.63 in) / (60000 psi) = 0.02 in² / in

A_{s_min} = 0.02 in² / in

A_s = 0.04 in² / in ≥ A_{s_min} = 0.02 in² / in ✓

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)

$$\frac{M_u}{\phi M_n} = \frac{(11.37\text{ ft·k / ft})}{(12.14\text{ ft·k / ft})} = 0.9371$$ (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 (0.9371) per 25.4.10: l_{dh} = 10.78 in

8 d_b = 8 (0.75 in) = 6.0 (minimum limit, does not control)

l_{dh_prov} = 12 in ≥ l_{dh} = 10.78 in ✓

Toe Checks [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)

$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0.04 \text{ in}^2/\text{in})(60000 \text{ psi})}{0.85\,(3000 \text{ psi})} = 1.04 \text{ in}$

$\phi M_n = \phi A_s f_y (d - a/2) = (0.90)(0.04 \text{ in}^2/\text{in})(60000 \text{ psi})[(11.63 \text{ in}) - (1.04 \text{ in})/2] = 26.39 \text{ ft·k/ft}$

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)

$V_c = 2\lambda\sqrt{F'_c}\,d = 2\,(1.0)\sqrt{3000 \text{ psi}}\,(11.63 \text{ in}) = 15.28 \text{ k/ft}$

$\phi V_n = \phi V_c = (0.750)(15.28 \text{ k/ft}) = 11.46 \text{ k/ft}$

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)

$l_{dh} = \left(\frac{f_y\,\psi_e\,\psi_c\,\psi_r}{50\,\lambda\sqrt{F'_c}}\right) d_b = \left[\frac{(60000 \text{ psi})(1.0)(0.70)(1.0)}{50\,(1.0)\sqrt{3000 \text{ psi}}}\right](0.75 \text{ in}) = 11.5 \text{ in}$

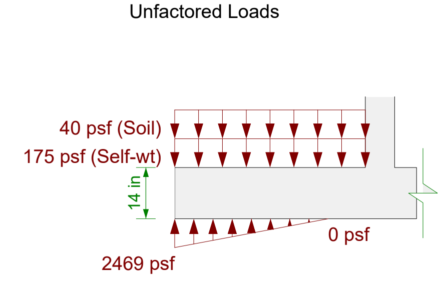
Factoring l_{dh} by the excess reinforcement ratio (- 0.0000) per 25.4.10: l_{dh} = - 0 in

$8\,d_b = 8\,(0.75 \text{ in}) = 6.0$ (minimum limit, does not control)

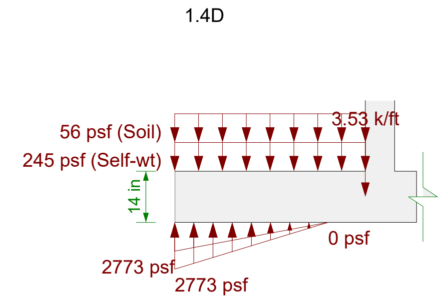
6 inch minimum controls

l_{dh_prov} = 6 in ≥ l_{dh} = 6 in ✓

Toe Unfactored Loads



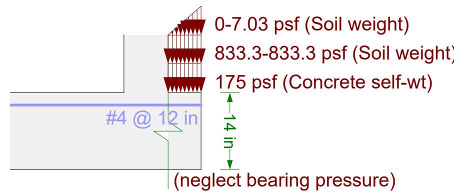
Toe Factored Loads



Heel Checks [1.4D]

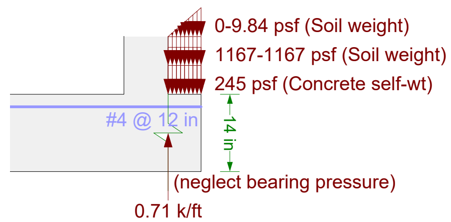
Heel Unfactored Loads

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)

$a = \dfrac{A_s\, f_y}{0.85\, F'_c} = \dfrac{(0.02 \text{ in}^2 / \text{in})(60000 \text{ psi})}{0.85\,(3000 \text{ psi})} = 0.39 \text{ in}$

$\phi M_n = \phi\, A_s\, f_y\, (d - a / 2) = (0.90)(0.02 \text{ in}^2 / \text{in})(60000 \text{ psi})\,[(11.75 \text{ in}) - (0.39 \text{ in}) / 2] = 10.4 \text{ ft·k / ft}$

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)

$a = \dfrac{A_s\, f_y}{0.85\, F'_c} = \dfrac{(0.02 \text{ in}^2 / \text{in})(60000 \text{ psi})}{0.85\,(3000 \text{ psi})} = 0.39 \text{ in}$

$\varepsilon_t = 0.003 \left(\dfrac{d}{a / \beta_1} - 1\right) = 0.003 \left[\dfrac{(11.75 \text{ in})}{(0.39 \text{ in}) / (0.850)} - 1\right] = 0.0734$

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 ≥ $(4 / 3)\, M_u = [4 / 3]\,(- 0 \text{ ft·k / ft}) = - 0 \text{ ft·k / ft}$

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)

$\rho_{ST\_prov} = \dfrac{A_{ST}}{t\, s_{ST}} = \dfrac{(0.4 \text{ in}^2 / \text{in})}{(14 \text{ in})(12 \text{ in})} = 0.0024$

$\rho_{ST\_prov} = \dfrac{A_{ST}}{t\, s_{ST}} = \dfrac{(0.4 \text{ in}^2 / \text{in})}{(14 \text{ in})(12 \text{ in})} = 0.0024$

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

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)

$\dfrac{M_u}{\phi M_n} = \dfrac{(- 0 \text{ ft·k / ft})}{(10.4 \text{ ft·k / ft})} = - 0.0$ (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)

$\dfrac{c_b + K_{tr}}{d_b} = \dfrac{(2.25 \text{ in}) + (0.0)}{(0.5 \text{ in})} = 4.50$

$l_d = \left(\dfrac{3.}{40}\,\dfrac{f_y}{\lambda\,\sqrt{F'_c}}\,\dfrac{\psi_t\,\psi_e\,\psi_s}{2.5}\right) d_b = \left[\dfrac{3.}{40}\,\dfrac{(60000 \text{ psi})}{(1.0)\,\sqrt{3000 \text{ psi}}}\,\dfrac{(1.0)(1.0)(0.80)}{2.5}\right](0.5 \text{ in}) = 13.15 \text{ in}$

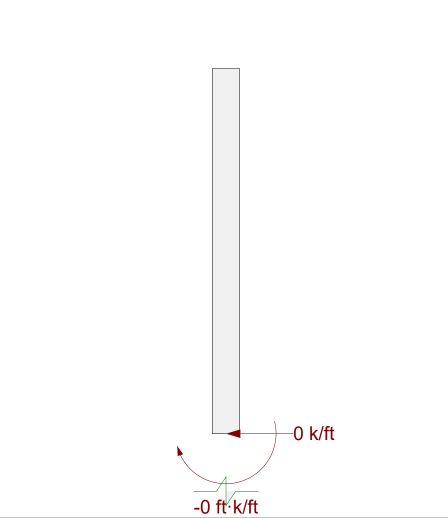
Factoring l_d by the excess reinforcement ratio (- 0.0000) per 25.4.10: l_d = - 0 in

12 inch minimum controls

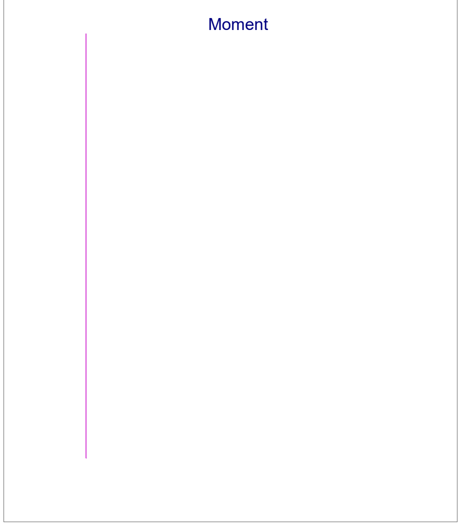
l_{d_prov} = 57.96 in ≥ l_d = 12 in ✓

Stem Forces [1.4D]

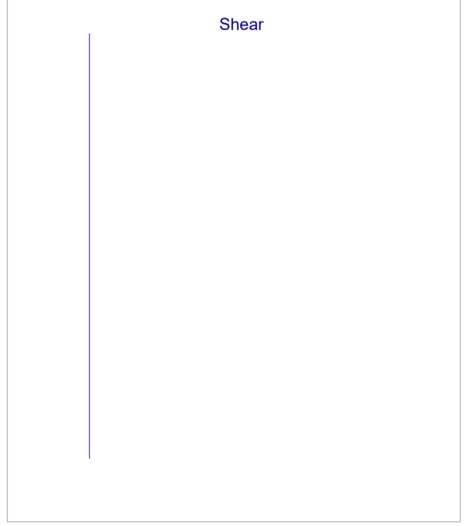
Stem Internal Forces



Stem Internal Forces



Stem Internal Forces

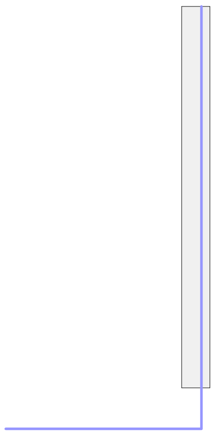


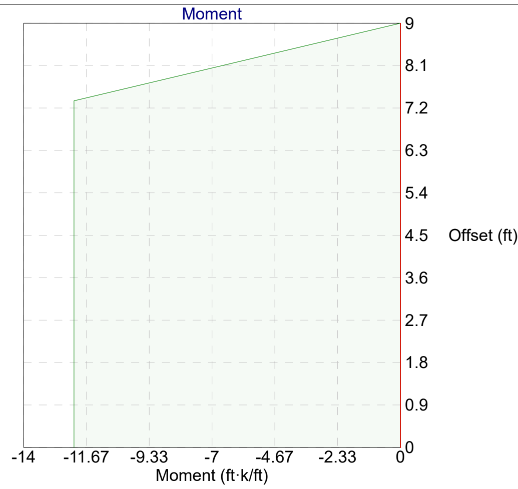
Stem Joint Force Transfer

Location	Force
@ stem base	0 k/ft

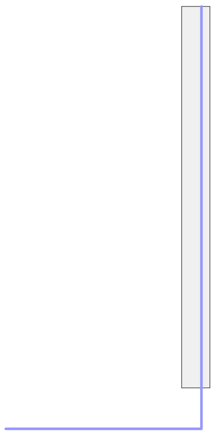
Stem Internal Forces

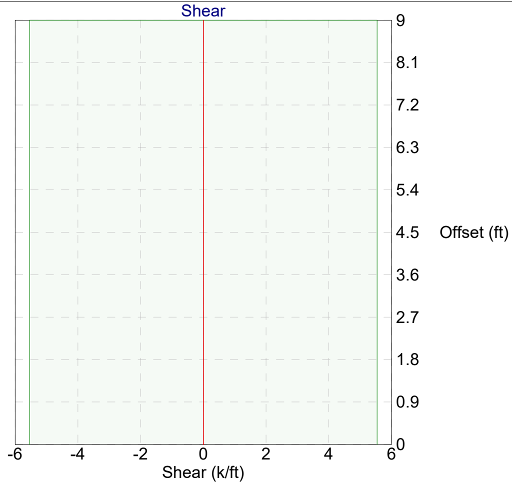
Stem Moment Checks [1.4D]





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]

ϕM_n = 12.14 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) @ 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)

$$\frac{M_u}{\phi M_n} = \frac{(0 \text{ ft·k / ft})}{(12.14 \text{ ft·k / ft})} = 0.0$$ (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)

$$l_{dh} = \left(\frac{f_y \psi_e \psi_c \psi_r}{50 \lambda \sqrt{F'_c}}\right) d_b = \left[\frac{(60000 \text{ psi})(1.0)(0.70)(1.0)}{50 (1.0) \sqrt{3000 \text{ psi}}}\right] (0.75 \text{ in}) = 11.5 \text{ in}$$

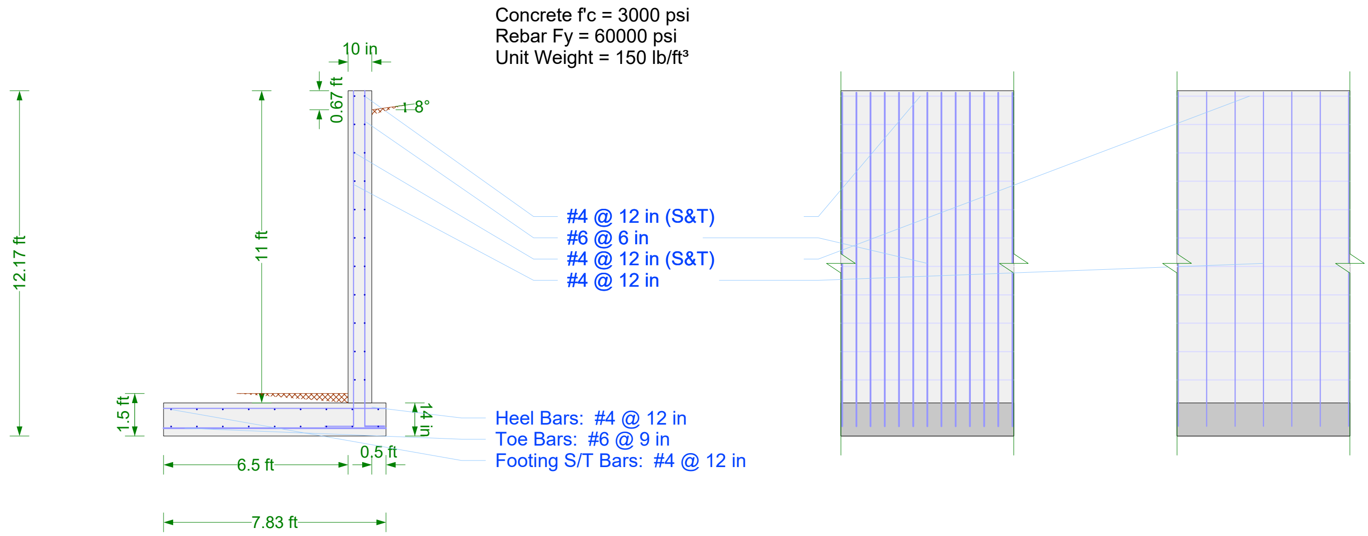
Factoring l_{dh} by the excess reinforcement ratio (0.0000) per 25.4.10: l_{dh} = 0 in

8 d_b = 8 (0.75 in) = 6.0 (minimum limit, does not control)

6 inch minimum controls

l_{dh_prov} = 12 in ≥ l_{dh} = 6 in ✓

Design Detail



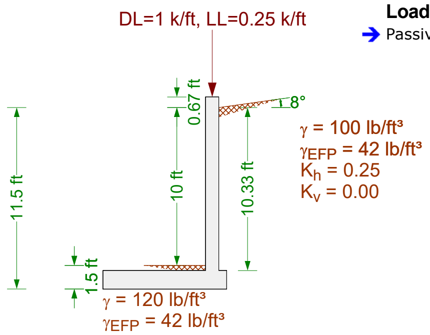
Check Summary

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

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

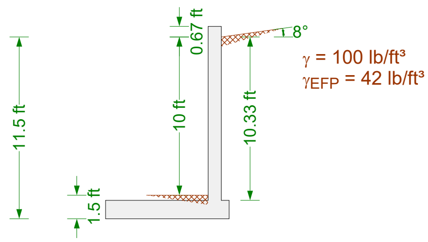
Load Combination	Stem Actual steel (in²/in)	Heel M-applied (ft·k/ft)	Heel M-allow (ft·k/ft)	Heel V-applied (k/ft)	Heel V-allow (k/ft)	Toe M-applied (ft·k/ft)	Toe M-allow (ft·k/ft)	Toe V-applied (k/ft)	Toe V-allow (k/ft)
1.2D + 1.6L + 1.6H	0.07	0.18	10.4	0.73	11.58	20.83	26.53	4.35	10.48
1.2D + 1.6L + 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

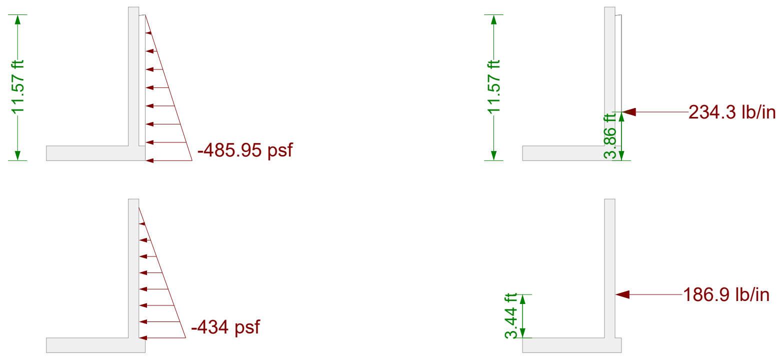
Stability Check Results Summary

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

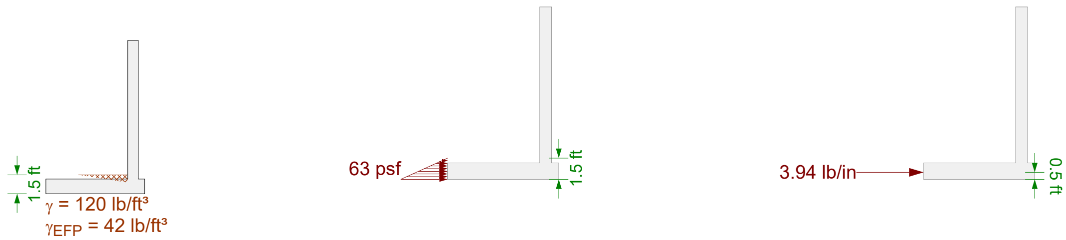
Equivalent Fluid Pressure

$\sigma_h = H\,\gamma_{fluid} = (11.57\text{ ft})(42\text{ lb / ft}^3) = 486\text{ psf}$

Lateral Earth Pressure (stem only)

$\sigma_h = H\,\gamma_{fluid} = (10.33\text{ ft})(42\text{ lb / ft}^3) = 434\text{ psf}$

Passive Pressure

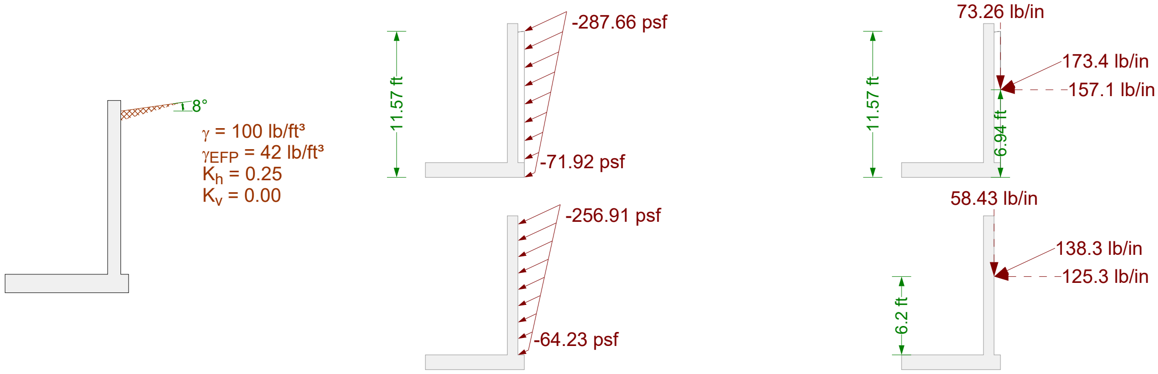


Lateral Earth Pressure

Equivalent Fluid Pressure

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

Seismic Pressure



Seismic Pressure

Dynamic + static force (Mononobe - Okabe equation)

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

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

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

$= 0.6403$

$P_{ae} = \frac{1}{2} K_{ae} \gamma H^{\wedge}2 (1 - k_v) = \frac{1}{2}(0.6403)(100\ lb/ft^3)(11.57\ ft)^{\wedge}2 [1 - (0.0)] = 357.2\ lb/in$

Static - only force (Coulomb equation)

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

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

$= 0.3295$

$P_a = \frac{1}{2} K_a \gamma H^{\wedge}2 = \frac{1}{2}(0.3295)(100\ lb/ft^3)(11.57\ ft)^{\wedge}2 = 183.8\ lb/in$

Net dynamic force

$\Delta P_{ae} = P_{ae} - P_a = (357.2\ lb/in) - (183.8\ lb/in) = 173.4\ lb/in$

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

To arrive at the pressure distribution illustrated above (used to determine stem moments),

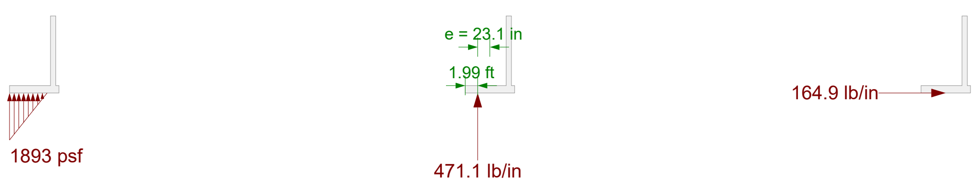
apply inverted triangular pressure plus a uniform portion to bring resultant to 0.6H

$\sigma_{e\_top} = \frac{8}{5}\frac{\Delta P_{ae}}{H} = \frac{8}{5}\frac{(173.4\ lb/in)}{(11.57\ ft)} = 287.7\ psf$

$\sigma_{e\_bot} = \frac{2}{5}\frac{\Delta P_{ae}}{H} = \frac{2}{5}\frac{(173.4\ lb/in)}{(11.57\ ft)} = 71.92\ psf$

Wall/Soil Weights

Bearing Pressure



Friction

$F = \mu R = (0.350)(471.1\ lb / in) = 164.9\ lb / in$

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

$$\frac{-\ 135093.51\ in\cdot lb\ /\ ft}{-\ 471.12\ lb\ /\ in} = 1.99\ ft$$

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

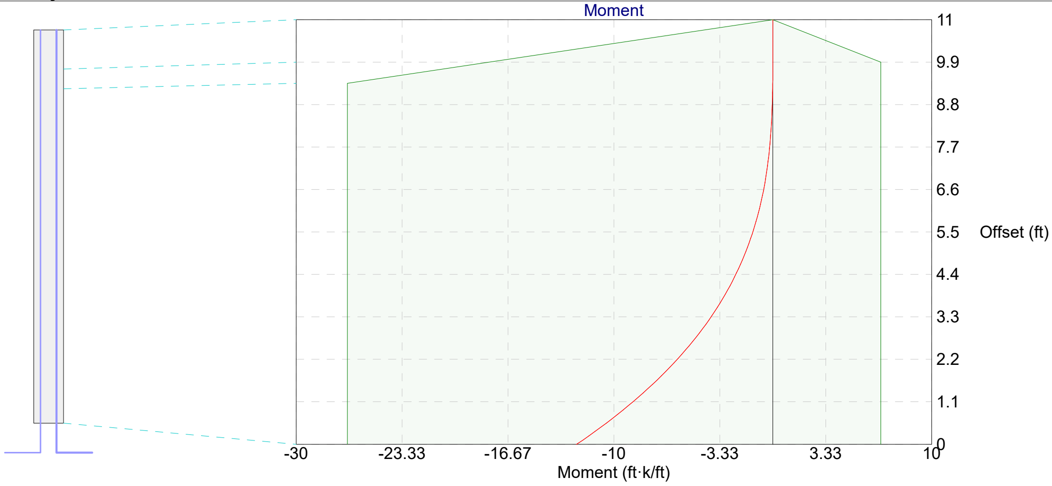
Bearing pressure < allowable (1804 psf < 2500 psf) - OK
Bearing resultant eccentricity < allowable (23.1 in < 47 in) - OK

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]

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

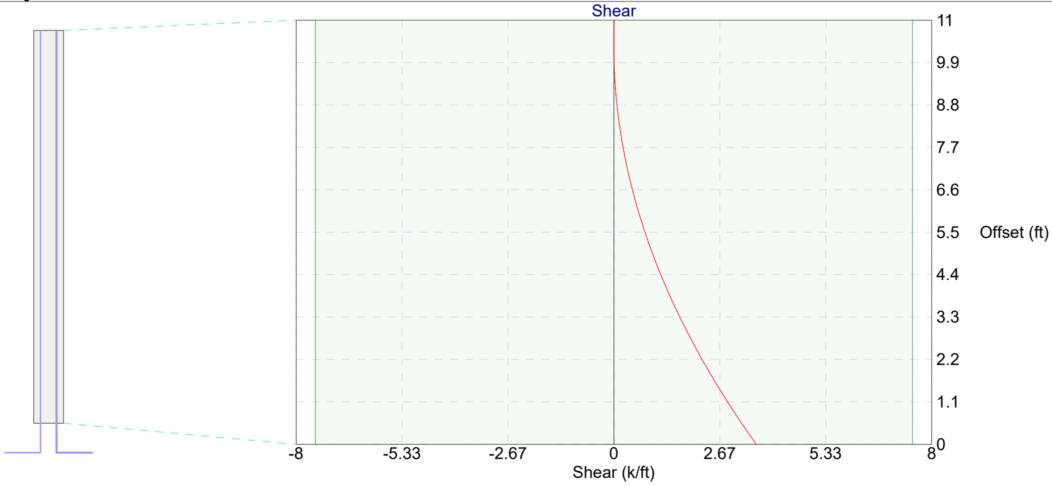
$$\phi M_n = \phi A_s f_y (d - a/2) = (0.90)(0 \text{ in}^2/\text{in})(60000 \text{ psi})[(7.63 \text{ in}) - (0 \text{ in})/2] = 0 \text{ ft·k/ft}$$

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)

$V_c = 2\,\lambda\,\sqrt{F'_c}\,d = 2\,(1.0)\sqrt{3000\text{ psi}}\,(7.63\text{ in}) = 10.02\text{ k / ft}$

$\phi V_n = \phi\,V_c = (0.750)\,(10.02\text{ k / ft}) = 7.52\text{ k / ft}$

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)

$V_c = 2\,\lambda\,\sqrt{F'_c}\,d = 2\,(1.0)\sqrt{3000\text{ psi}}\,(7.63\text{ in}) = 10.02\text{ k / ft}$

$\phi V_n = \phi\,V_c = (0.750)\,(10.02\text{ k / ft}) = 7.52\text{ k / ft}$

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)

$V_c = 2\,\lambda\,\sqrt{F'_c}\,d = 2\,(1.0)\sqrt{3000\text{ psi}}\,(7.63\text{ in}) = 10.02\text{ k / ft}$

$\phi V_n = \phi\,V_c = (0.750)\,(10.02\text{ k / ft}) = 7.52\text{ k / ft}$

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)

$V_c = 2\,\lambda\,\sqrt{F'_c}\,d = 2\,(1.0)\sqrt{3000\text{ psi}}\,(7.63\text{ in}) = 10.02\text{ k / ft}$

$\phi V_n = \phi\,V_c = (0.750)\,(10.02\text{ k / ft}) = 7.52\text{ k / ft}$

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)

$\psi_e = 1.0$ (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)

$$l_{dh} = \left(\frac{f_y \psi_e \psi_c \psi_r}{50 \lambda \sqrt{F'_c}}\right) d_b = \left[\frac{(60000\text{ psi})(1.0)(0.70)(1.0)}{50(1.0)\sqrt{3000\text{ psi}}}\right](0.75\text{ in}) = 11.5\text{ in}$$

$8 d_b = 8(0.75\text{ in}) = 6.0$ (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)

$\psi_t = 1.0$ (bars are not horizontal)

$\psi_e = 1.0$ (bar not epoxy coated)

$\psi_s = 0.80$ (bars are #6 or smaller)

$\lambda = 1.0$ (normal weight concrete)

$s / 2 = (6\text{ in}) / 2 = 3\text{ in}$

$\text{cover} + d_b / 2 = (2\text{ in}) + (0.75\text{ in}) / 2 = 2.38\text{ in}$

$c_b = 2.38\text{ in}$ (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)

$\psi_t = 1.0$ (bars are not horizontal)

$\psi_e = 1.0$ (bar not epoxy coated)

$\psi_s = 0.80$ (bars are #6 or smaller)

$\lambda = 1.0$ (normal weight concrete)

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

$\text{cover} + d_b / 2 = (2\text{ in}) + (0.5\text{ in}) / 2 = 2.25\text{ in}$

$c_b = 2.25\text{ in}$ (lesser of half spacing, ctr to surface)

$K_{tr} = 0.0$ (no transverse reinforcement)

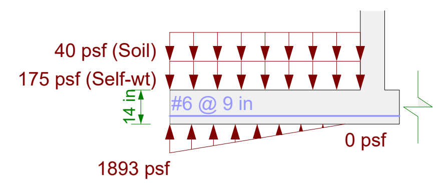
$$\frac{c_b + K_{tr}}{d_b} = \frac{(2.25\text{ in}) + (0.0)}{(0.5\text{ in})} = 4.50$$

$$l_d = \left(\frac{3.}{40} \frac{f_y}{\lambda \sqrt{F'_c}} \frac{\psi_t \psi_e \psi_s}{2.5}\right) d_b = \left[\frac{3.}{40} \frac{(60000\text{ psi})}{(1.0)\sqrt{3000\text{ psi}}} \frac{(1.0)(1.0)(0.80)}{2.5}\right](0.5\text{ in}) = 13.15\text{ in}$$

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

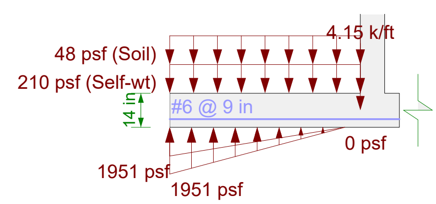
Toe Unfactored Loads

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)

$$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0.05 \text{ in}^2/\text{in})(60000 \text{ psi})}{0.85(3000 \text{ psi})} = 1.15 \text{ in}$$

$$\phi M_n = \phi A_s f_y (d - a/2) = (0.90)(0.05 \text{ in}^2/\text{in})(60000 \text{ psi})[(10.63 \text{ in}) - (1.15 \text{ in})/2] = 26.53 \text{ ft·k/ft}$$

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)

$$V_c = 2\lambda\sqrt{F'_c}\,d = 2(1.0)\sqrt{3000 \text{ psi}}(10.63 \text{ in}) = 13.97 \text{ k/ft}$$

$$\phi V_n = \phi V_c = (0.750)(13.97 \text{ k/ft}) = 10.48 \text{ k/ft}$$

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

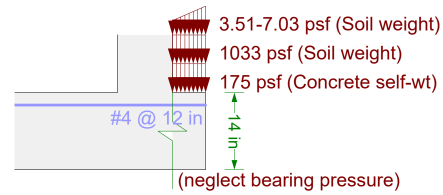
12 inch minimum controls

l_{d_prov} = 13 in ≥ l_d = 12 in ✓

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

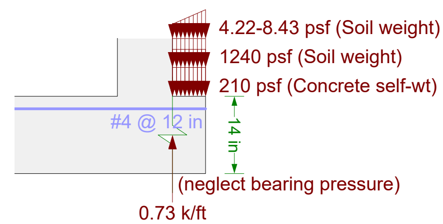
Heel Unfactored Loads

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)

$$a = \frac{A_s\, f_y}{0.85\, F'_c} = \frac{(0.02\ \text{in}^2 / \text{in})\,(60000\ \text{psi})}{0.85\,(3000\ \text{psi})} = 0.39\ \text{in}$$

$$\phi M_n = \phi\, A_s\, f_y\,(d - a/2) = (0.90)\,(0.02\ \text{in}^2/\text{in})\,(60000\ \text{psi})\,[(11.75\ \text{in}) - (0.39\ \text{in})/2] = 10.4\ \text{ft·k / ft}$$

ϕM_n = 10.4 ft·k / ft ≥ M_u = 0.18 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)

$$V_c = 2\,\lambda\,\sqrt{F'_c}\,d = 2\,(1.0)\,\sqrt{3000\ \text{psi}}\,(11.75\ \text{in}) = 15.45\ \text{k / ft}$$

$$\phi V_n = \phi\, V_c = (0.750)\,(15.45\ \text{k / ft}) = 11.58\ \text{k / ft}$$

ϕV_n = 11.58 k / ft ≥ V_u = 0.73 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)

$$\frac{c_b + K_{tr}}{d_b} = \frac{(2.25\ \text{in}) + (0.0)}{(0.5\ \text{in})} = 4.50$$

$$l_d = \left(\frac{3.}{40}\,\frac{f_y}{\lambda\,\sqrt{F'_c}}\,\frac{\psi_t\,\psi_e\,\psi_s}{2.5}\right) d_b = \left[\frac{3.}{40}\,\frac{(60000\ \text{psi})}{(1.0)\,\sqrt{3000\ \text{psi}}}\,\frac{(1.0)\,(1.0)\,(0.80)}{2.5}\right](0.5\ \text{in}) = 13.15\ \text{in}$$

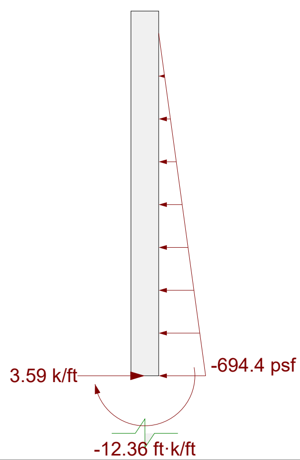
Factoring l_d by the excess reinforcement ratio (0.0175) per 25.4.10: l_d = 0.23 in

12 inch minimum controls

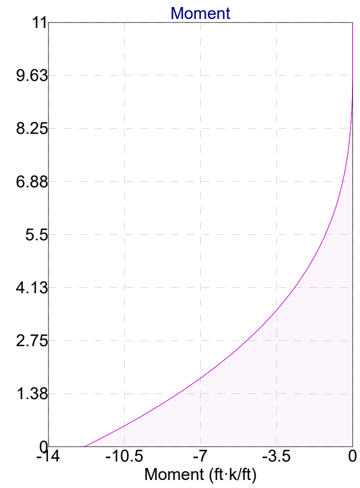
l_{d_prov} = 86 in ≥ l_d = 12 in ✓

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

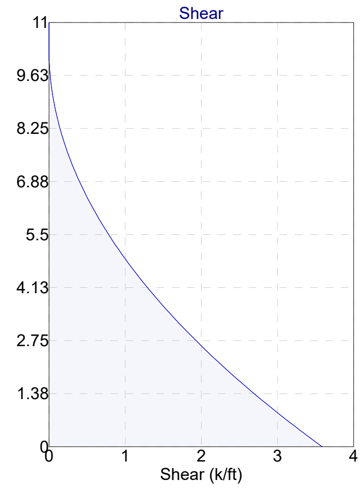
Stem Internal Forces



Stem Internal Forces



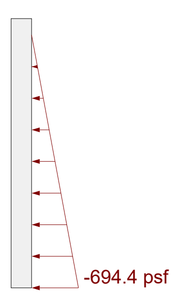
Stem Internal Forces



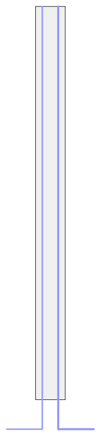
Stem Joint Force Transfer

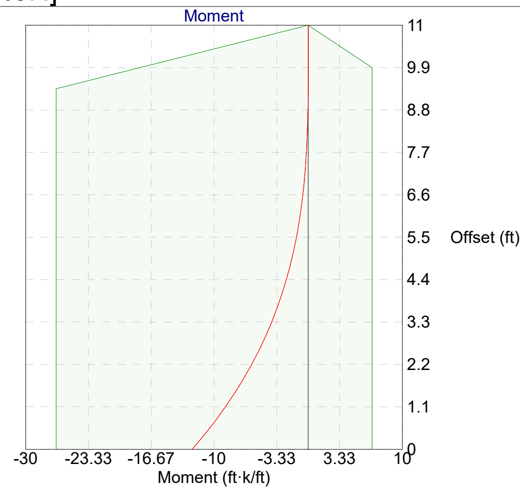
Location	Force
@ stem base	3.59 k/ft

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 ✓

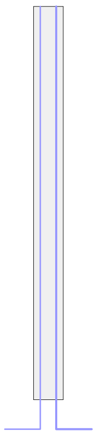
Check (ACI 318-14 11.5.5.1b) @ 9.36 ft from base

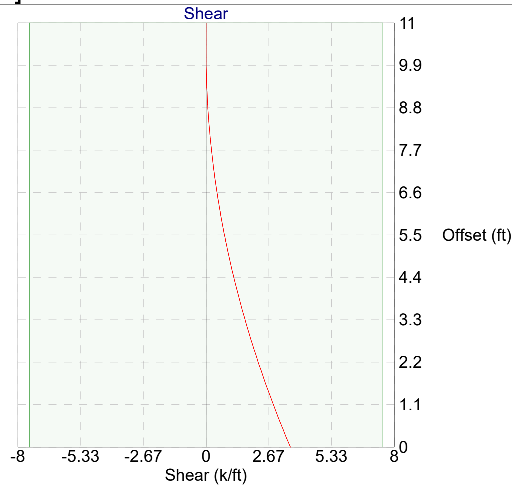
ϕM_n = 26.78 ft·k / ft ≥ M_u = 0.01 ft·k / ft ✓

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 $\geq$ 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]

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

$$a = \frac{A_s\,f_y}{0.85\,F'_c} = \frac{(0.07\text{ in}^2/\text{in})\,(60000\text{ psi})}{0.85\,(3000\text{ psi})} = 1.73\text{ in}$$

$$\varepsilon_t = 0.003\left(\frac{d}{a/\beta_1} - 1\right) = 0.003\left[\frac{(7.63\text{ in})}{(1.73\text{ in})/(0.850)} - 1\right] = 0.0083$$

$\varepsilon_t = 0.0083 \geq 0.004$ ✓

Maximum Steel Check (ACI 318-14 9.3.3.1) @ 11 ft from base [Stem in negative flexure]

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

$$a = \frac{A_s\,f_y}{0.85\,F'_c} = \frac{(0.07\text{ in}^2/\text{in})\,(60000\text{ psi})}{0.85\,(3000\text{ psi})} = 1.73\text{ in}$$

$$\varepsilon_t = 0.003\left(\frac{d}{a/\beta_1} - 1\right) = 0.003\left[\frac{(7.63\text{ in})}{(1.73\text{ in})/(0.850)} - 1\right] = 0.0083$$

$\varepsilon_t = 0.0083 \geq 0.004$ ✓

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)

$\rho_t = 0.0033 \geq \rho_{t\_min} = 0.0020$ ✓

$3\,h = 3\,(10\text{ in}) = 30\text{ in}$

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)

$$\frac{M_u}{\phi M_n} = \frac{(12.36\text{ ft·k/ft})}{(26.78\text{ ft·k/ft})} = 0.4615$$ (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)

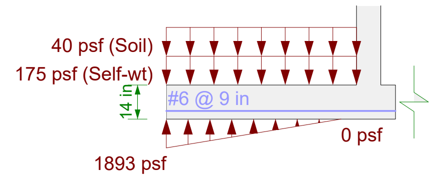
6 inch minimum controls

$l_{dh\_prov} = 11\text{ in} \geq l_{dh} = 6\text{ in}$ ✓

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

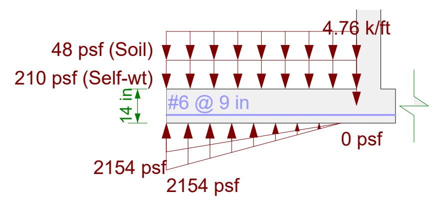
Toe Unfactored Loads

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)

$$a = \frac{A_s\, f_y}{0.85\, F'_c} = \frac{(0.05\text{ in}^2/\text{in})\,(60000\text{ psi})}{0.85\,(3000\text{ psi})} = 1.15\text{ in}$$

$$\phi M_n = \phi\, A_s\, f_y\,(d - a/2) = (0.90)\,(0.05\text{ in}^2/\text{in})\,(60000\text{ psi})\,[(10.63\text{ in}) - (1.15\text{ in})/2] = 26.53\text{ ft·k/ft}$$

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)

$$V_c = 2\,\lambda\,\sqrt{F'_c}\,d = 2\,(1.0)\,\sqrt{3000\text{ psi}}\,(10.63\text{ in}) = 13.97\text{ k/ft}$$

$$\phi V_n = \phi\, V_c = (0.750)\,(13.97\text{ k/ft}) = 10.48\text{ k/ft}$$

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]

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)

$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0.02 \text{ in}^2/\text{in})(60000 \text{ psi})}{0.85 (3000 \text{ psi})} = 0.39 \text{ in}$

$\phi M_n = \phi A_s f_y (d - a/2) = (0.90)(0.02 \text{ in}^2/\text{in})(60000 \text{ psi})[(11.75 \text{ in}) - (0.39 \text{ in})/2] = 10.4 \text{ ft·k/ft}$

ϕM_n = 10.4 ft·k / ft ≥ M_u = 0.18 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)

$V_c = 2 \lambda \sqrt{F'_c}\, d = 2 (1.0) \sqrt{3000 \text{ psi}} (11.75 \text{ in}) = 15.45 \text{ k/ft}$

$\phi V_n = \phi V_c = (0.750)(15.45 \text{ k/ft}) = 11.58 \text{ k/ft}$

ϕV_n = 11.58 k / ft ≥ V_u = 0.73 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)

$\frac{c_b + K_{tr}}{d_b} = \frac{(2.25 \text{ in}) + (0.0)}{(0.5 \text{ in})} = 4.50$

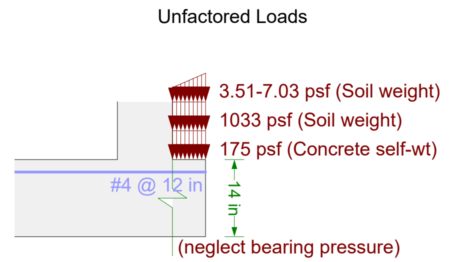
$l_d = \left(\frac{3.}{40} \frac{f_y}{\lambda \sqrt{F'_c}} \frac{\psi_t \psi_e \psi_s}{2.5}\right) d_b = \left[\frac{3.}{40} \frac{(60000 \text{ psi})}{(1.0)\sqrt{3000 \text{ psi}}} \frac{(1.0)(1.0)(0.80)}{2.5}\right](0.5 \text{ in}) = 13.15 \text{ in}$

Factoring l_d by the excess reinforcement ratio (0.0175) per 25.4.10: l_d = 0.23 in

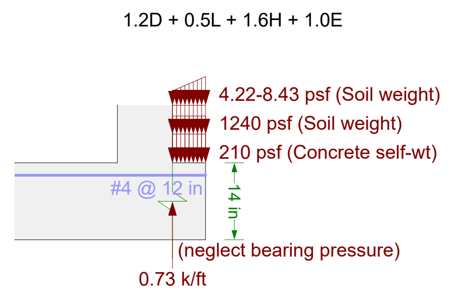
12 inch minimum controls

l_{d_prov} = 86 in ≥ l_d = 12 in ✓

Heel Unfactored Loads

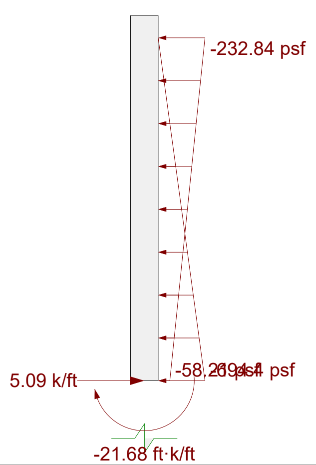


Heel Factored Loads

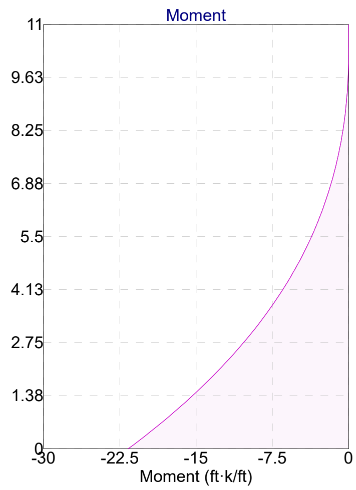


Stem Forces [1.2D + 0.5L + 1.6H + 1.0E]

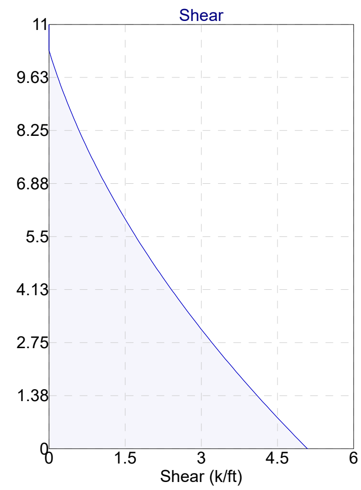
Stem Internal Forces



Stem Internal Forces



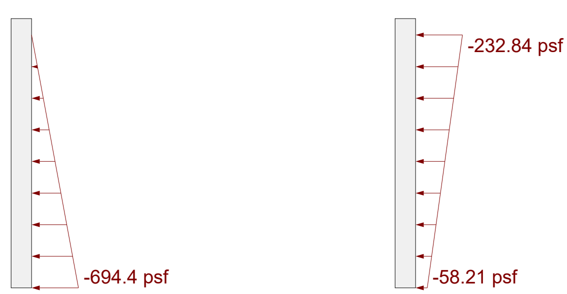
Stem Internal Forces



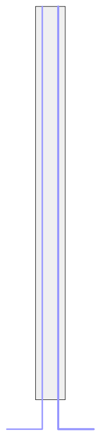
Stem Joint Force Transfer

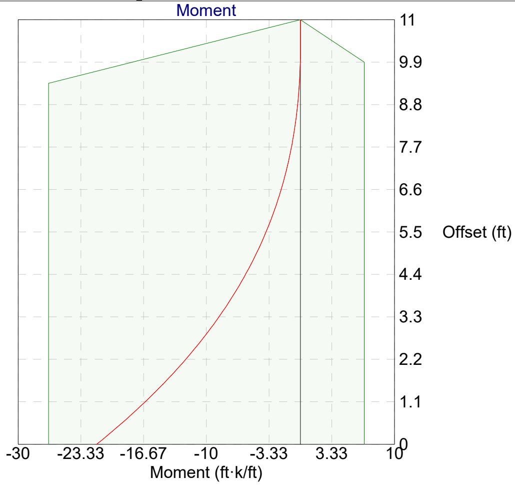
Location	**Force**
@ stem base	5.09 k/ft

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 ✓

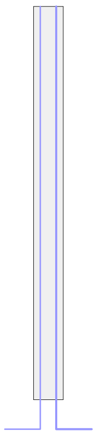
Check (ACI 318-14 11.5.5.1b) @ 9.36 ft from base

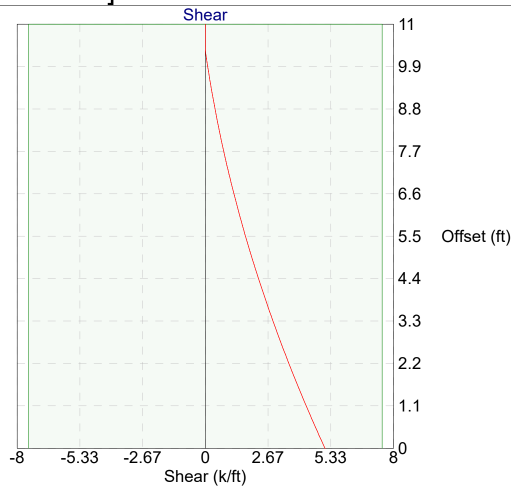
ϕM_n = 26.78 ft·k / ft ≥ M_u = 0.12 ft·k / ft ✓

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]

$\phi M_n = 26.78 \text{ ft·k / ft} < (4/3)\,M_u = [4/3]\,(21.68 \text{ ft·k / ft}) = 28.91 \text{ ft·k / ft}$

$$A_{s\_min} = \frac{3\sqrt{F'_c}}{f_y}\,d = \frac{3\sqrt{3000 \text{ psi}}}{(60000 \text{ psi})}(7.63 \text{ in}) = 0.02 \text{ in}^2/\text{in}$$

$200\,d / f_y = 200\,(7.63 \text{ in}) / (60000 \text{ psi}) = 0.03 \text{ in}^2/\text{in}$

$A_{s\_min} = 0.03 \text{ in}^2/\text{in}$

$A_s = 0.07 \text{ in}^2/\text{in} \geq A_{s\_min} = 0.03 \text{ in}^2/\text{in}$ ✓

Minimum Steel Check (ACI 318-14 9.6.1) @ 11 ft from base [Stem in negative flexure]

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

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]

$\beta_1 = 0.850 \qquad (F'_c \leq 4000 \text{ psi})$

$$a = \frac{A_s\,f_y}{0.85\,F'_c} = \frac{(0.07 \text{ in}^2/\text{in})\,(60000 \text{ psi})}{0.85\,(3000 \text{ psi})} = 1.73 \text{ in}$$

$$\varepsilon_t = 0.003\left(\frac{d}{a/\beta_1} - 1\right) = 0.003\left[\frac{(7.63 \text{ in})}{(1.73 \text{ in})/(0.850)} - 1\right] = 0.0083$$

$\varepsilon_t = 0.0083 \geq 0.004$ ✓

Maximum Steel Check (ACI 318-14 9.3.3.1) @ 11 ft from base [Stem in negative flexure]

$\beta_1 = 0.850 \qquad (F'_c \leq 4000 \text{ psi})$

$$a = \frac{A_s\,f_y}{0.85\,F'_c} = \frac{(0.07 \text{ in}^2/\text{in})\,(60000 \text{ psi})}{0.85\,(3000 \text{ psi})} = 1.73 \text{ in}$$

$$\varepsilon_t = 0.003\left(\frac{d}{a/\beta_1} - 1\right) = 0.003\left[\frac{(7.63 \text{ in})}{(1.73 \text{ in})/(0.850)} - 1\right] = 0.0083$$

$\varepsilon_t = 0.0083 \geq 0.004$ ✓

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

$$\rho_t = \frac{A_{s\_horz} / s_{horz}}{t} = \frac{(0.4 \text{ in}^2)/(12 \text{ in})}{(10 \text{ in})} = 0.0033$$

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

$\rho_t = 0.0033 \geq \rho_{t\_min} = 0.0020$ ✓

$3\,h = 3\,(10 \text{ in}) = 30 \text{ in}$

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)

$$\frac{M_u}{\phi M_n} = \frac{(21.68 \text{ ft·k / ft})}{(26.78 \text{ ft·k / ft})} = 0.8096 \qquad \text{(ratio to represent excess reinforcement)}$$

$\psi_e = 1.0 \qquad$ (uncoated hooked bars)

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

$\psi_r = 1.0 \qquad$ (no confining reinforcement)

$\lambda = 1.0 \qquad$ (normal weight concrete)

$$l_{dh} = \left(\frac{f_y\,\psi_e\,\psi_c\,\psi_r}{50\,\lambda\,\sqrt{F'_c}}\right)d_b = \left[\frac{(60000 \text{ psi})\,(1.0)\,(0.70)\,(1.0)}{50\,(1.0)\,\sqrt{3000 \text{ psi}}}\right](0.75 \text{ in}) = 11.5 \text{ in}$$

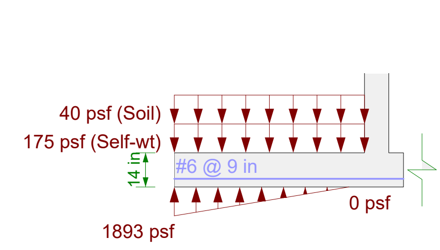
Factoring l_{dh} by the excess reinforcement ratio (0.8096) per 25.4.10: $l_{dh} = 9.31 \text{ in}$

$8\,d_b = 8\,(0.75 \text{ in}) = 6.0 \qquad$ (minimum limit, does not control)

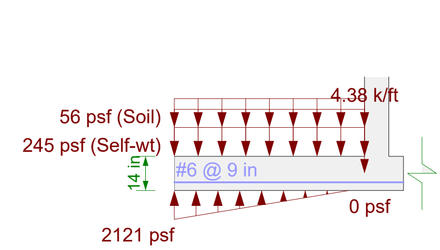
$l_{dh\_prov} = 11 \text{ in} \geq l_{dh} = 9.31 \text{ in}$ ✓

Toe Checks [1.4D]

Toe Unfactored Loads



Toe Factored Loads



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]

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)

$$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0.02 \text{ in}^2/\text{in})(60000 \text{ psi})}{0.85 (3000 \text{ psi})} = 0.39 \text{ in}$$

$$\varepsilon_t = 0.003 \left(\frac{d}{a/\beta_1} - 1\right) = 0.003 \left[\frac{(11.75 \text{ in})}{(0.39 \text{ in})/(0.850)} - 1\right] = 0.0734$$

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)

$$\frac{c_b + K_{tr}}{d_b} = \frac{(2.25 \text{ in}) + (0.0)}{(0.5 \text{ in})} = 4.50$$

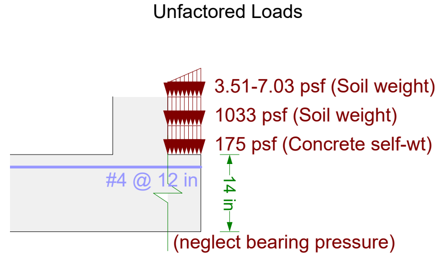
$$l_d = \left(\frac{3.}{40} \frac{f_y}{\lambda \sqrt{F'_c}} \frac{\psi_t \psi_e \psi_s}{2.5}\right) d_b = \left[\frac{3.}{40} \frac{(60000 \text{ psi})}{(1.0)\sqrt{3000 \text{ psi}}} \frac{(1.0)(1.0)(0.80)}{2.5}\right](0.5 \text{ in}) = 13.15 \text{ in}$$

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

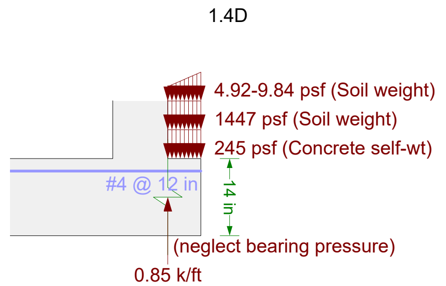
12 inch minimum controls

l_{d_prov} = 86 in ≥ l_d = 12 in ✓

Heel Unfactored Loads

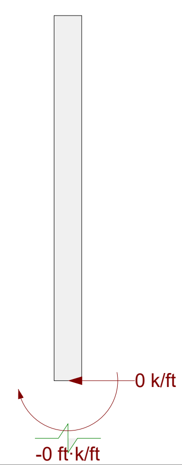


Heel Factored Loads

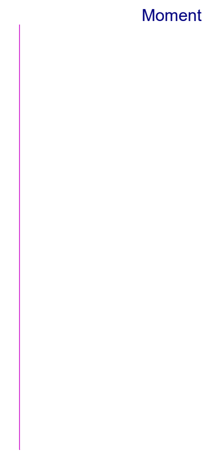


Stem Forces [1.4D]

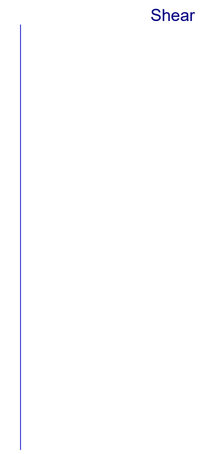
Stem Internal Forces



Stem Internal Forces



Stem Internal Forces

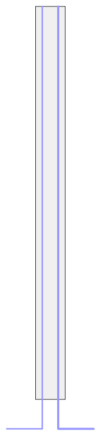


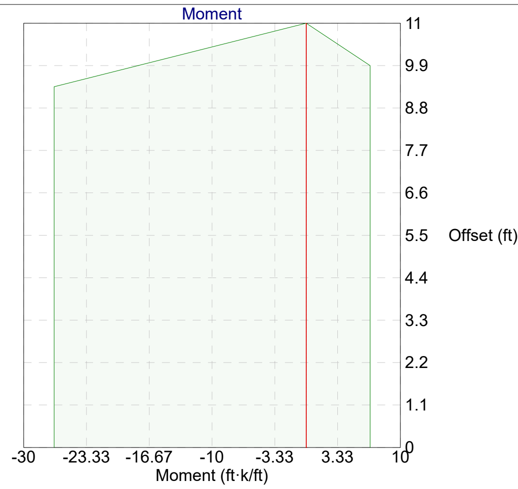
Stem Joint Force Transfer

Location	Force
@ stem base	0 k/ft

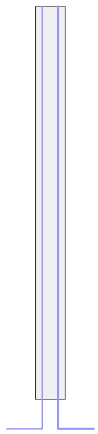
Stem Internal Forces

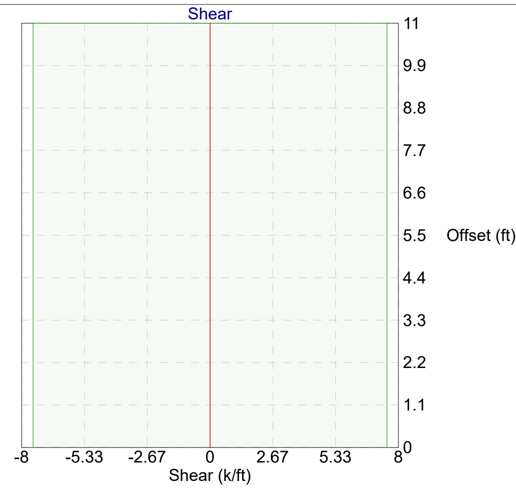
Stem Moment Checks [1.4D]





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]

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

$$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0.07 \text{ in}^2/\text{in})(60000 \text{ psi})}{0.85(3000 \text{ psi})} = 1.73 \text{ in}$$

$$\varepsilon_t = 0.003\left(\frac{d}{a/\beta_1} - 1\right) = 0.003\left[\frac{(7.63 \text{ in})}{(1.73 \text{ in})/(0.850)} - 1\right] = 0.0083$$

$\varepsilon_t = 0.0083 \geq 0.004$ ✓

Maximum Steel Check (ACI 318-14 9.3.3.1) @ 11 ft from base [Stem in negative flexure]

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

$$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0.07 \text{ in}^2/\text{in})(60000 \text{ psi})}{0.85(3000 \text{ psi})} = 1.73 \text{ in}$$

$$\varepsilon_t = 0.003\left(\frac{d}{a/\beta_1} - 1\right) = 0.003\left[\frac{(7.63 \text{ in})}{(1.73 \text{ in})/(0.850)} - 1\right] = 0.0083$$

$\varepsilon_t = 0.0083 \geq 0.004$ ✓

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)

$\rho_t = 0.0033 \geq \rho_{t\_min} = 0.0020$ ✓

$3\,h = 3\,(10 \text{ in}) = 30 \text{ in}$

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)

$$\frac{M_u}{\phi M_n} = \frac{(0 \text{ ft·k/ft})}{(26.78 \text{ ft·k/ft})} = 0.0$$ (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

$l_{dh\_prov} = 11 \text{ in} \geq l_{dh} = 6 \text{ in}$ ✓

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LOADS TO STEEL BEAMS

2ND LEVEL

ROOF D.L. = $12.3^{PSF} \times 1.2 = 14.76^{PLF}$
ROOF L.L. = $25^{PSF} \times 1.6 = 40^{PLF}$ } 55^{PLF}
WALL D.L. = $10^{PSF} \times 8.0' = 80^{PLF} \times 1.2 = 96^{PLF}$

MAX ROOF RAFTER SPAN = 20'

⇒ LOAD TO STEEL BEAM = $(55^{PLF} \times 20'/2) + 96^{PLF}$
$\cong 650^{PLF}$

1ST LEVEL

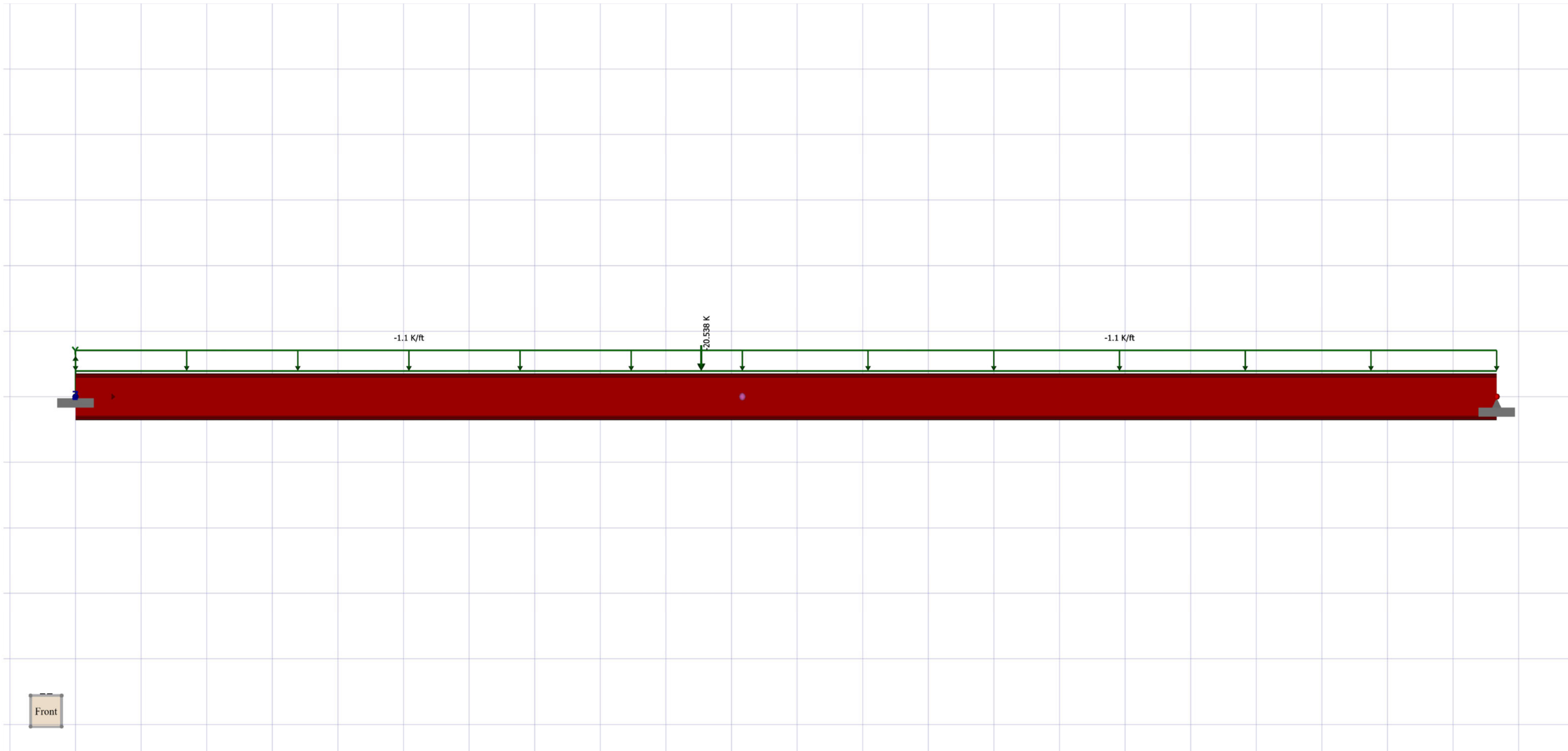
FLOOR D.L. = $12.3^{PSF} \times 1.2 = 14.76^{PLF}$
FLOOR L.L. = $40^{PSF} \times 1.6 = 64^{PLF}$ } 79^{PLF}
WALL D.L. = $10^{PSF} \times 8.0' = 80^{PLF} \times 1.2 = 96^{PLF}$

MAX FLOOR JOIST SPAN = 31'

⇒ LOAD TO STEEL BEAM = $(79^{PLF} \times 31'/2) + 96^{PLF}$
$\cong 1,325^{PLF}$

MAX SPAN BETWEEN STEEL BEAMS = 27'

⇒ LOAD TO STEEL BEAM = $79^{PLF} \times 27'/2 \cong 1,100^{PLF}$


-1.1 K/ft
20.538 K
-1.1 K/ft
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

Section Properties

Section	Source	Area in^2	Iz in^4	Iy in^4	J in^4	Alpha, α deg
W16X100	Database Shape	29.40000	1490.00000	186.00000	7.73000	0.00000

Member Loads, Concentrated

Member	Service Case	Direction	Magnitude	Offset ft
BmX002	D	Shear y	-20.53800 K	19.07726

Member Loads, Uniform

Member	Service Case	Direction	Magnitude	Full Length?	Start Offset ft	End Offset ft	Projected?	Predefined Load
BmX002	D	Shear y	-1.10000 K/ft	Yes	0.00000	20.33000	No	N.A.
BmX003	D	Shear y	-1.10000 K/ft	Yes	0.00000	23.00000	No	N.A.

Member Displacements

(extreme rows: max and min)

Member	Dy Min in	Dy Max in	Dz Min in	Dz Max in
BmX002	**-1.95713 (2)**	0.00000 (4)	0.00000 (4)	0.00000 (4)
BmX003	**-2.06678 (2)**	0.00000 (4)	0.00000 (4)	0.00000 (4)

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)	**-43.38586 (2)**	**43.38586 (2)**	**-43.38586 (2)**	**43.38586 (2)**

Member Stresses (continued)

(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)	**-24.15361 (2)**	**24.15361 (2)**	**-24.15361 (2)**	**24.15361 (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	**67.12358**	0.00000	0.00000	0.00000	**633.77389**
N001	5. 0.9D+W	0.00000	43.15088	0.00000	0.00000	0.00000	**407.42607**
N002	5. 0.9D+W	0.00000	**22.13734**	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

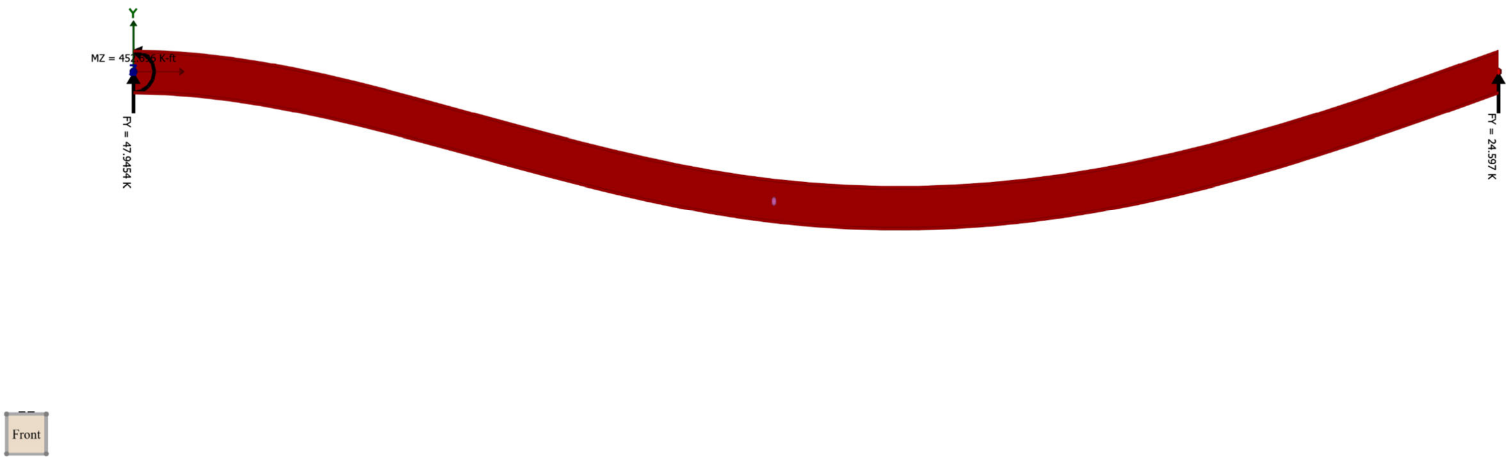
Steel_Beam X_G 1: Strong Flexure Check

(extreme rows: max)

Member	Section	Offset ft	Result Case	Demand Mz K-ft	Capacity Mz K-ft	Code Reference	Unity Check	Details
BmX002	W16X100	0.00000	1. 1.4D	-633.77389	742.50000	F2-1	**0.85357**	Lb = 20.33 ft, Cb = 2.42743

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	W16X100	0.00000	1. 1.4D	67.12358	298.35000	G2-1	**0.22498**	Shear Area = 9.945 in^2, Cv = 1, h/tw = 24.3077


Y
MZ = 452.636 K-ft
FY = 47.9454 K
FY = 24.597 K
Front

Steel Floor Beam Design

Assumptions:

1. W14 x 89 Beam			**31.00**	Feet =	372.00	Inches
		Supports are:	**3.00**	inches from each end of the beam		
	Therefore:		**L**	=	366.00	Inches
2. Beam designed as a simple span.						
	Therefore:		$\mathbf{L_d}$	**=**	366	Inches
3. Bearing area:	(Assume: 6"x6" Plate)		**N**	=	**6**	Inches

W14 x 89 Beam Properties:

A	=	**26.20**	inches^2		
$\mathbf{A_{web}}$	=	**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		
$\mathbf{Fv_{Allowable}}$	=	**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	=	**33,000.00**	PSI	$0.66\mathbf{F_y}$	AISC (F3-1)
$\mathbf{t_w}$	=	**0.525**	inches		
d	=	**16.75**	inches		
$\mathbf{t_f}$	=	**0.875**	inches		
$\mathbf{F_y}$	=	**50,000**	PSI		
k	=	**0.88**	inches		
$\mathbf{b_f}$	=	**10.365**	inches		
$\mathbf{W_t}$	=	**89**	lbs/lf		

Reactions:

$\mathbf{W_{design}}$	=		=	**1,325**	plf
$\mathbf{V_{max}}$	=		=	**20,538**	lbs
$\mathbf{P_{max}}$	=		=	**20,538**	lbs
$\mathbf{M_{max}}$	=		=	**159,166**	ft-lbs
$\mathbf{D_{max}}$	=		=	**0.730**	inches

Check Shear:

Fv	=	$\mathbf{V_{max}} / \mathbf{A_{web}}$		=	**2,345**	PSI
$\text{Fv}_{\text{Allowable}}$	=	$0.6\mathbf{F_y}$		=	**30,000**	PSI
					Check - O.K.	
			$\mathbf{V_{max}}$	=	**20,538**	LBS
$\text{Fv}_{\text{Allowable}}$	=	$0.66F_wt_w(N + 2.5k)$		=	**141,848**	LBS
		ASD Section K1.3			**Check - O.K.**	

Check Bearing-Web Crippling:

Fp	=	$\mathbf{P_{max}} / \mathbf{A_{web}}$		=	**2,335**	PSI
$\text{Fp}_{\text{Allowable}}$	=	$0.6\mathbf{F_y}$		=	**30,000**	PSI
					Check - O.K.	
			$\mathbf{V_{max}}$	=	**20,538**	LBS
$\text{Fp}_{\text{Allowable}}$	=	$0.66F_wt_w(N + 2.5k)$		=	**141,848**	LBS
		ASD Section K1.3			**Check - O.K.**	

Check Bending:

$\text{S}_{\text{Req'd}}$	=	$\mathbf{M_{max}}(12) / \mathbf{Fb}$	=	**57.9**	inches^3
$\text{S}_{\text{Furnished}}$	=			**155**	inches^3
				Check - O.K.	

Check Deflection:

$\mathbf{D_{max.}}$	=	$\mathbf{D_{max}}$	=	**0.730**	inches
$\mathbf{D_{Allowable}}$	=	L/350	=	**1.063**	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	$0.6F_y$	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	=	**0.335**	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
$Fv_{Allowable}$	=	$0.6F_y$	=	**30,000**	PSI
				Check - O.K.	
		V_{max}	=	**13,689**	LBS
$Fv_{Allowable}$	=	$0.66F_W t_W(N + 2.5k)$ ASD Section K1.3	=	**88,785**	LBS
				Check - O.K.	

Check Bearing-Web Crippling:

Fp	=	P_{max} / A_{web}	=	**3,388**	PSI
$Fp_{Allowable}$	=	$0.6F_y$	=	**30,000**	PSI
				Check - O.K.	
		V_{max}	=	**13,689**	LBS
$Fp_{Allowable}$	=	$0.66F_W t_W(N + 2.5k)$ ASD Section K1.3	=	**88,785**	LBS
				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.	

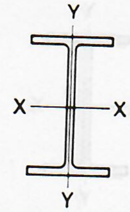
Check Deflection:

$D_{max.}$	=	D_{max}	=	**0.576**	inches
$D_{Allowable}$	=	L/350	=	**0.726**	inches
				Check - O.K.	

F_y = 36 ksi

F_y = 50 ksi

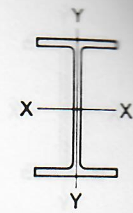
COLUMNS
W shapes
Allowable axial loads in kips



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 factors		0.218		0.221		0.227		0.227		0.227	
B_y Bending 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	
$F'_{ex}(K_xL_x)^2/10^2$ (kips)		289		284		278		275		273	
$F'_{ey}(K_yL_y)^2/10^2$ (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.



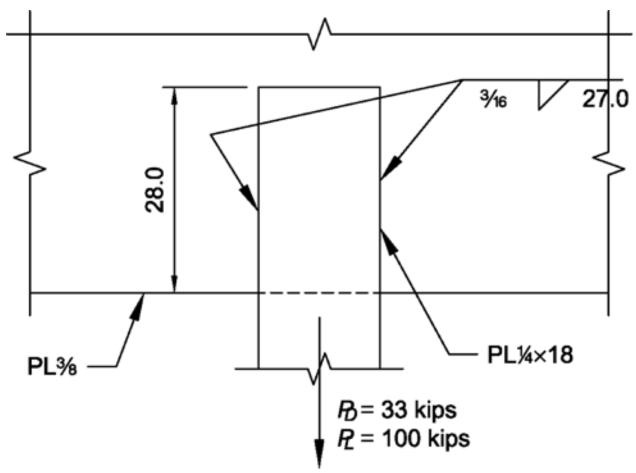
Designation		
Wt./ft		1
F_y		36
Effective length in ft KL with respect to least radius of gyration r_y	0	711
	6	663
	7	653
	8	642
	9	631
	10	619
	11	606
	12	593
	13	579
	14	565
	15	550
	16	535
	17	519
	18	503
	19	486
	20	469
	22	433
	24	395
	26	355
	28	313
	30	272
	32	239
	34	212
	36	189
	38	170
	40	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 factors		0.2
B_y Bending factors		0.7
$a_x/10^6$		106
$a_y/10^6$		35
$F'_{ex}(K_xL_x)^2/10^2$ (kips)		2.
$F'_{ey}(K_yL_y)^2/10^2$ (kips)		74

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 3⁄16-in. fillet weld. A 3⁄16-in. fillet weld can be deposited in the flat or horizontal position in a single pass (true up to 5⁄16-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 3⁄16-in. fillet weld is ϕR_n = 1.392 (3) = 4.17 kips/ in. $\dfrac{P_u}{\phi R_n} = \dfrac{200\text{ kips}}{4.17\text{ kips/in.}} = 48\text{ in.}$ or 24 in. of weld on each side	The allowable strength per inch of a 3⁄16-in. fillet weld is R_n/Ω= 0.928 (3) = 2.78 kips/ in. $\dfrac{P_a\Omega}{R_n} = \dfrac{133\text{ kips}}{2.78\text{ kips/in.}} = 48\text{ in.}$ or 24 in. of weld on each side.

Manual Part 8

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12527 Huckleberry Lane
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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

$L_w = (3)(1.392) = 4.17^{k/IN}$

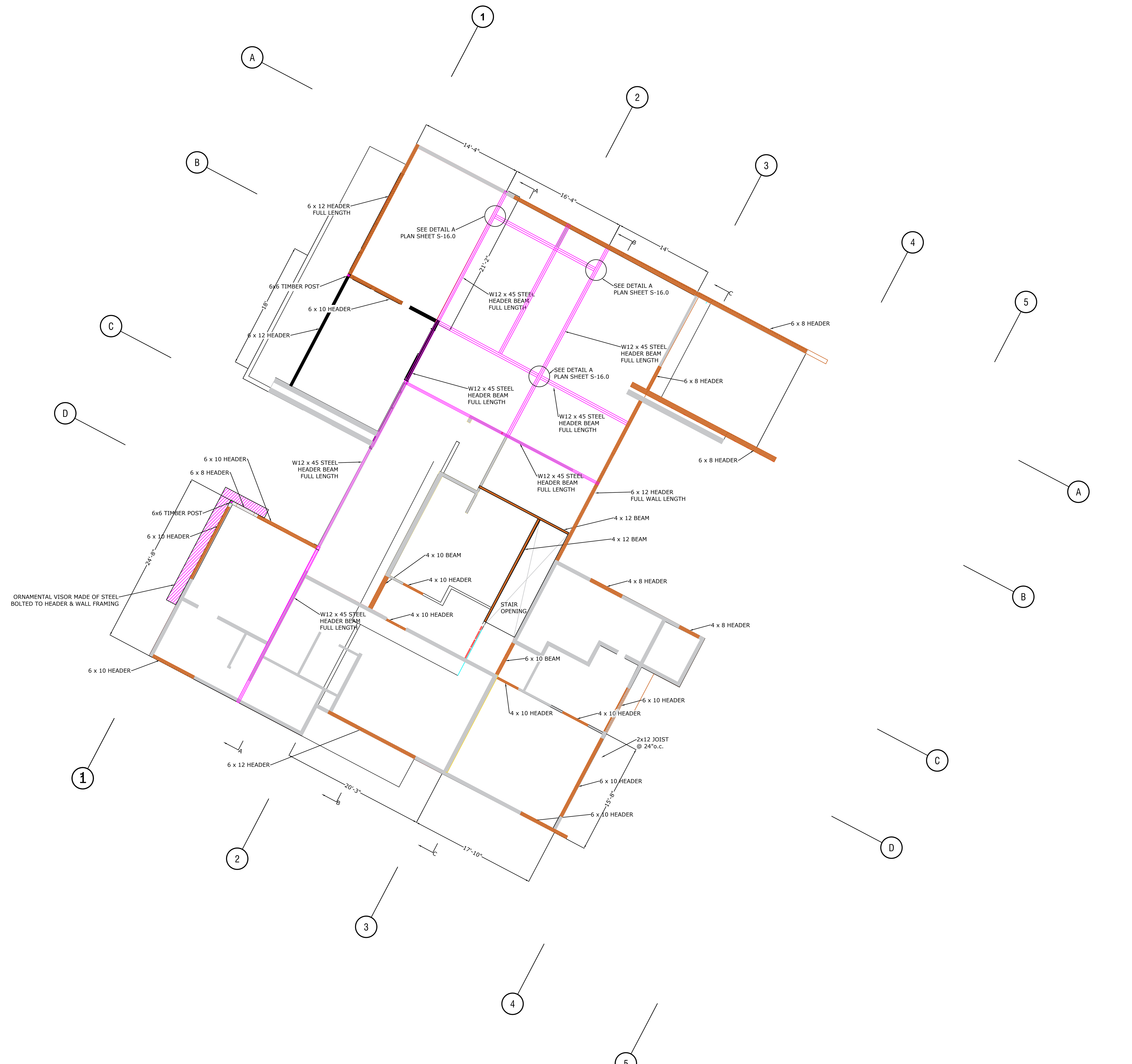
$\Rightarrow 33^k / 4.17^{k/IN} \cong 8"$ OF WELD REQ'D.

CHECK BOLTS

7/8" Ø A325 BOLTS

$V_{ALLOW} = 21.6^{KIPS}$

$33^k / 21.6^{k/BOLT} \cong 2$ BOLT REQ'D.



SECOND FLOOR - FLOOR FRAMING

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

Permanent Soldier Pile & Timber Lagging Retaining Wall

LUCIA E N G I N E E R I N G, I N C.
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
04-27-24

Number	Date	By	Description
3	04-27-24	JML	

SHEET
S-15.0

FRAMING NOTES:

- USE 3/4" STRUCTURAL 1 PLYWOOD WITH 10d NAILS SET @ 6" CENTERS EDGES & FIELD

FIRST FLOOR - FLOOR FRAMING

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JOSEPH M. LUCIA
STATE OF WASHINGTON
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32	04-27-24	JML	

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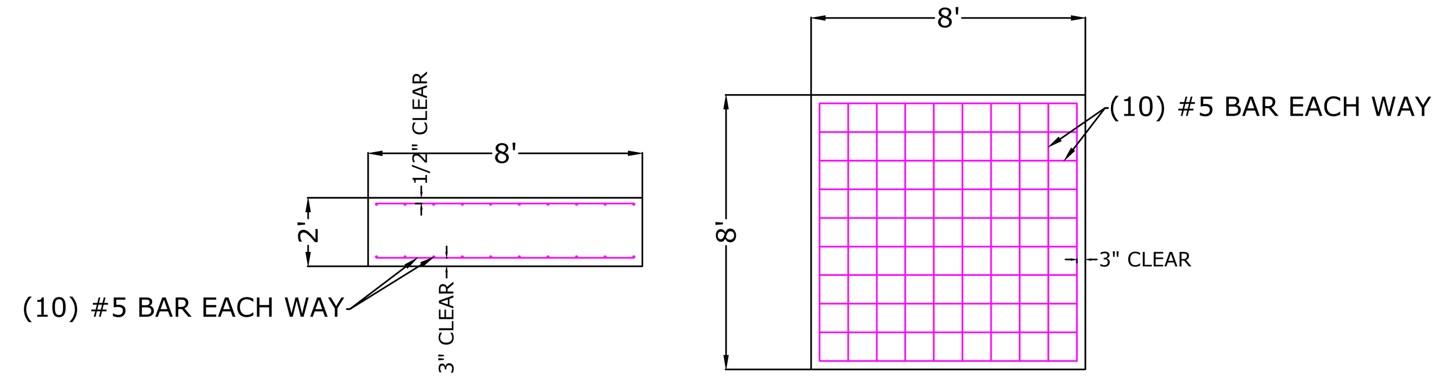
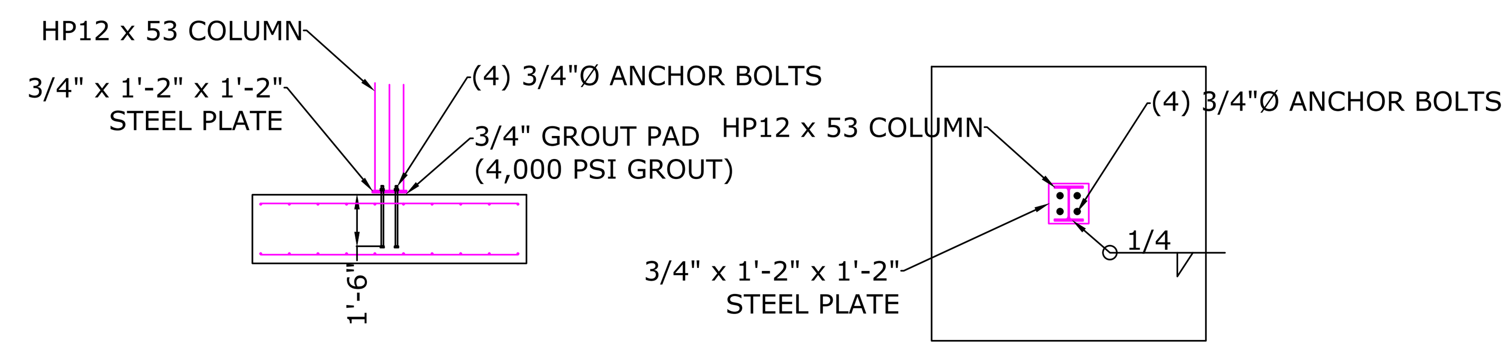
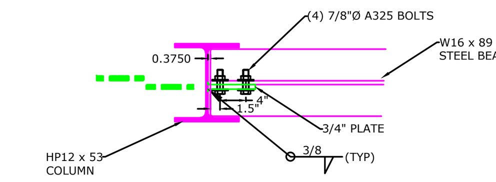


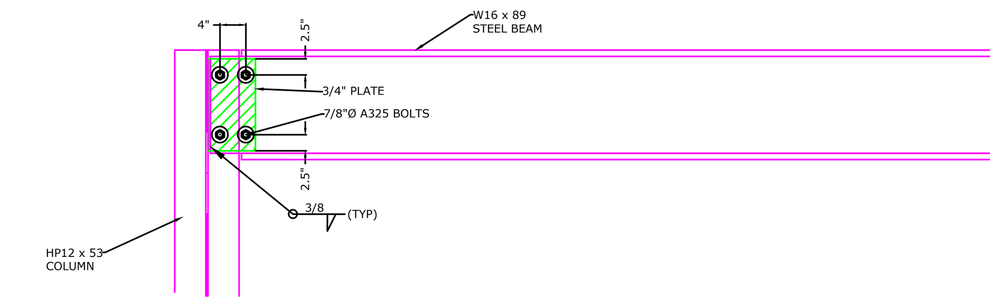
COLUMN FOOTING DETAIL



COLUMN TO FOOTING DETAIL

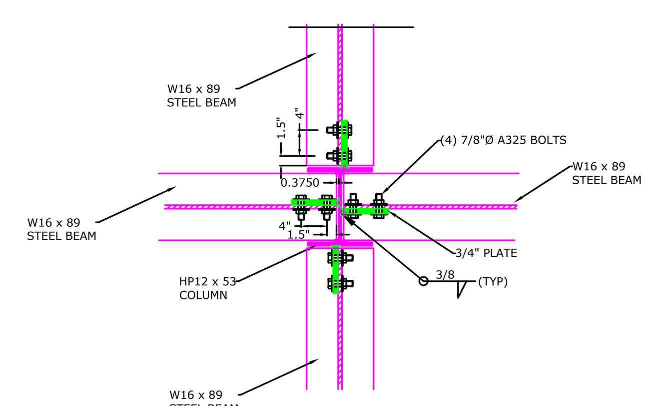


PLAN

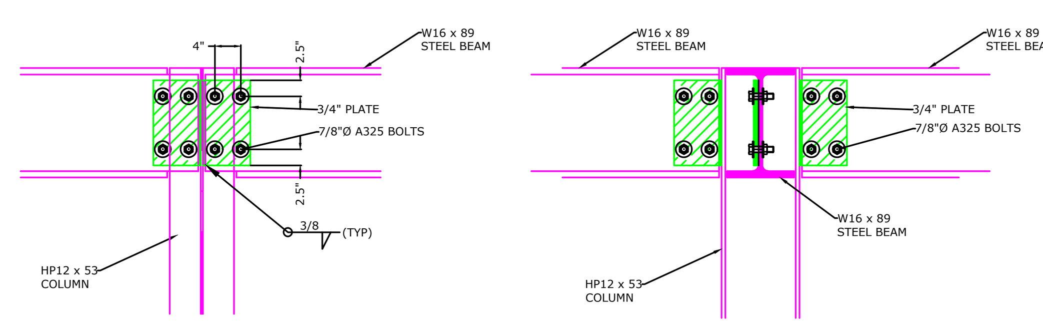


ELEVATION

DETAIL D

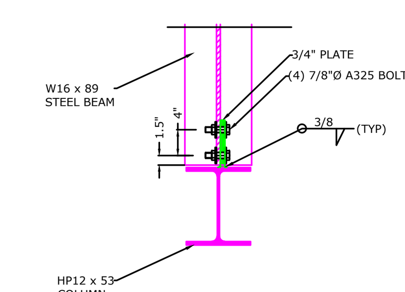


PLAN

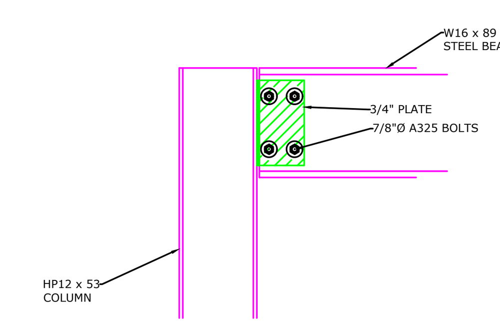


ELEVATION

DETAIL B

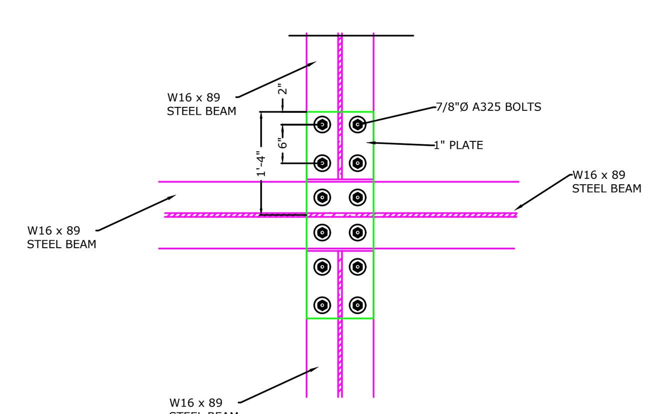


PLAN

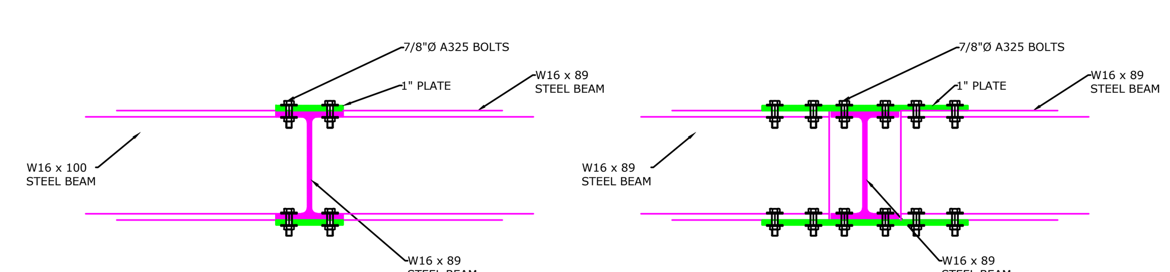


ELEVATION

DETAIL C

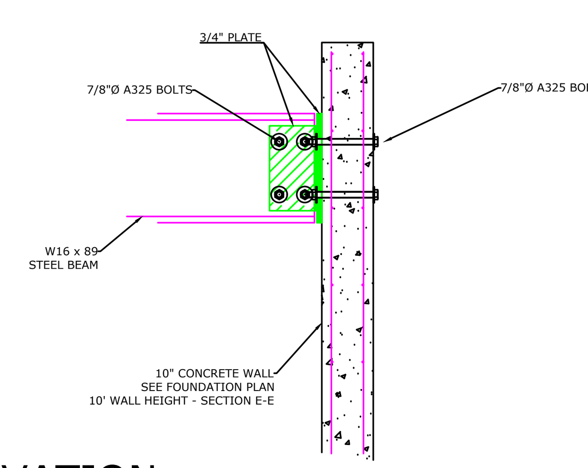


PLAN

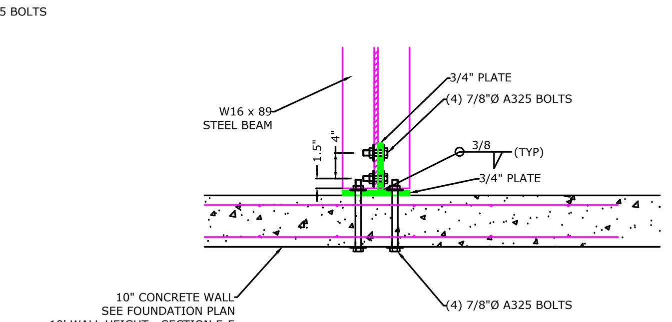


ELEVATION

DETAIL A



ELEVATION



PLAN

DETAIL E

ALTERNATES TO THE W16x89 STEEL BEAM SHOWN
W14 X 109, W18 X 97

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Mercer Island, WA 98040

Permanent Soldier Pile
& Timber Lagging
Retaining Wall

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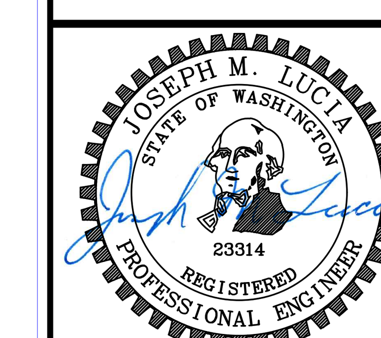
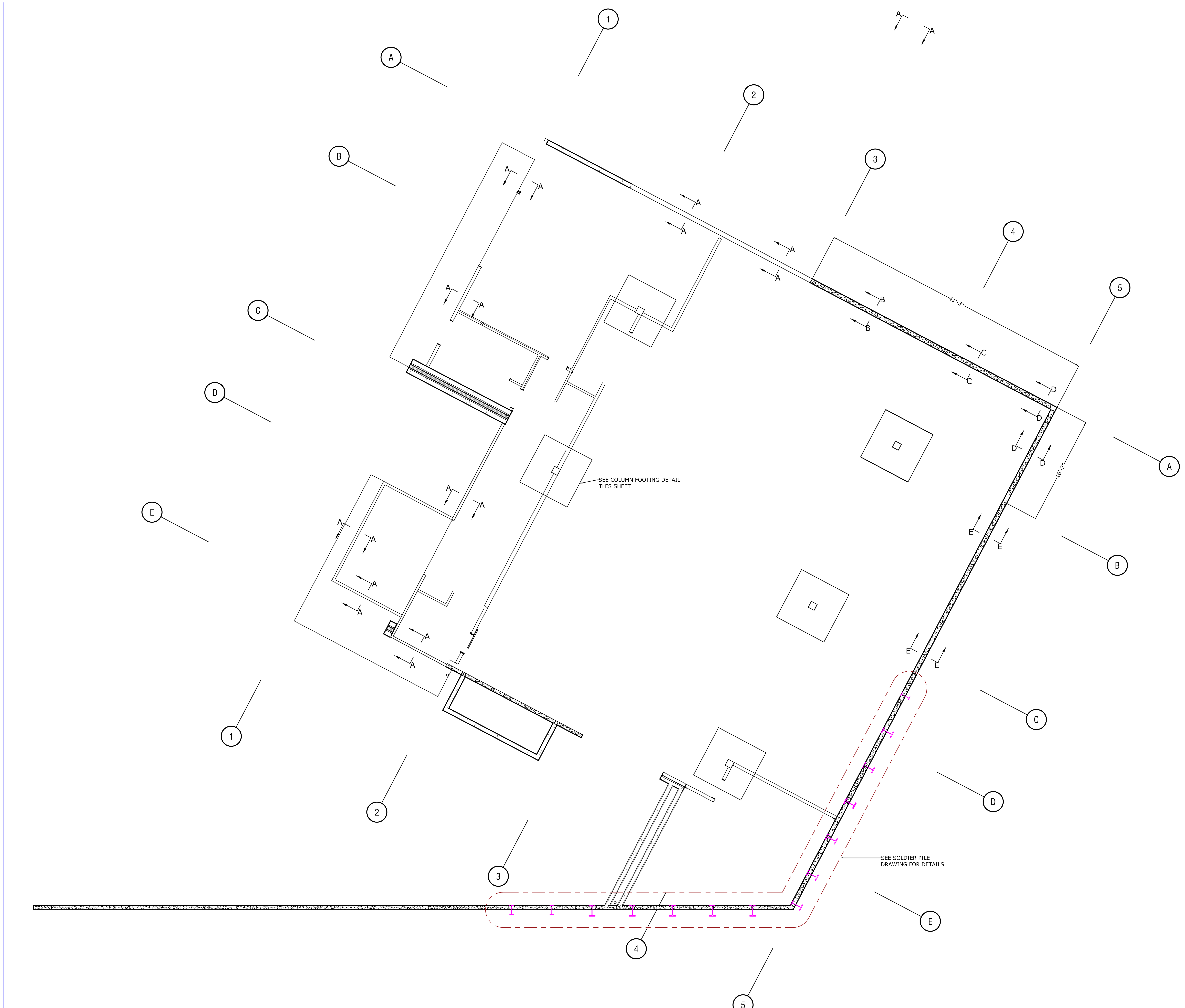


04-27-24

Number	Date	By	Description
3	04-27-24	JML	

SHEET
S-17.0



FOUNDATION PLAN

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

Permanent Soldier Pile
& Timber Lagging
Retaining Wall

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STATE OF WASHINGTON
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REGISTERED
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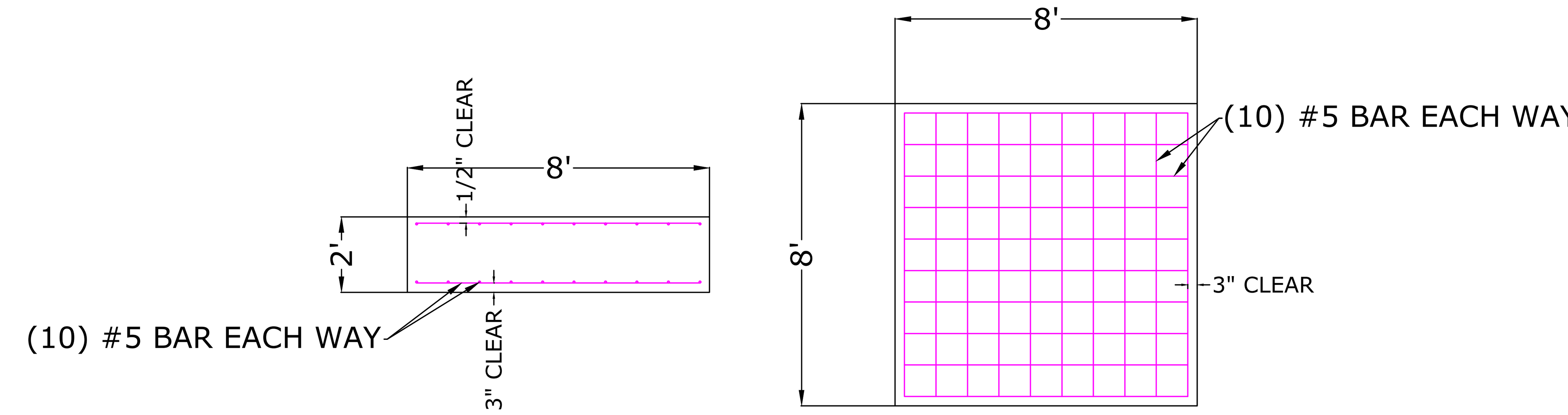
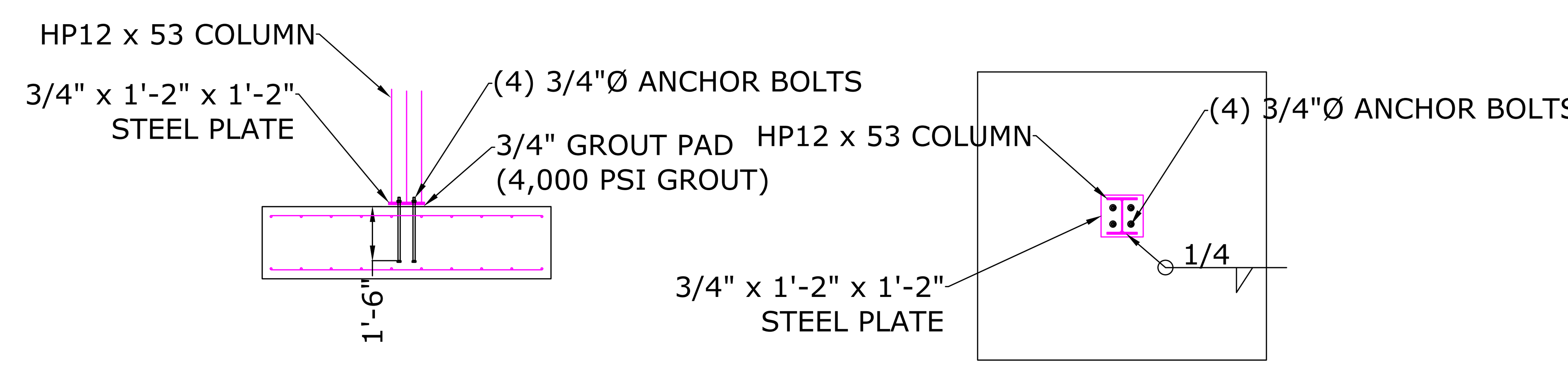
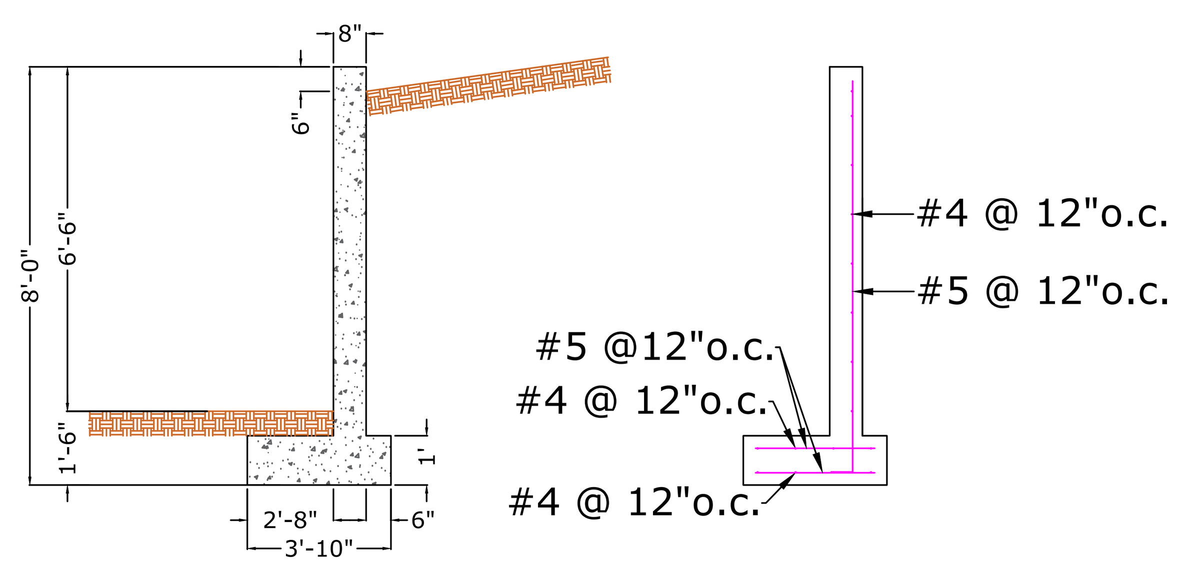


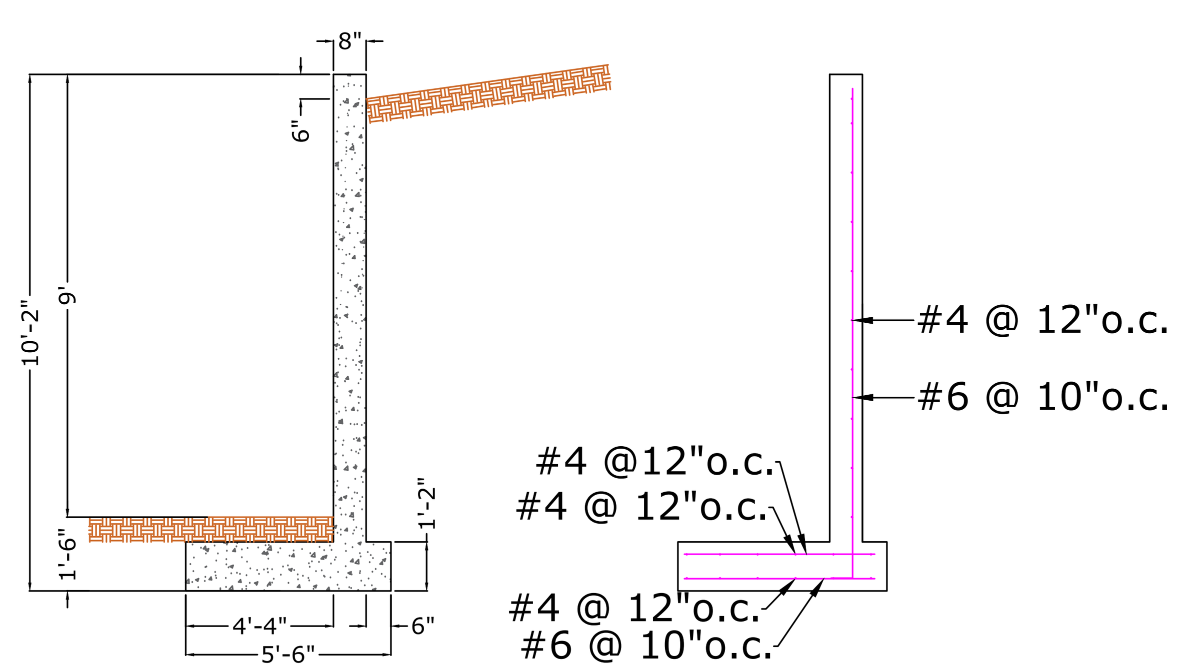
COLUMN FOOTING DETAIL



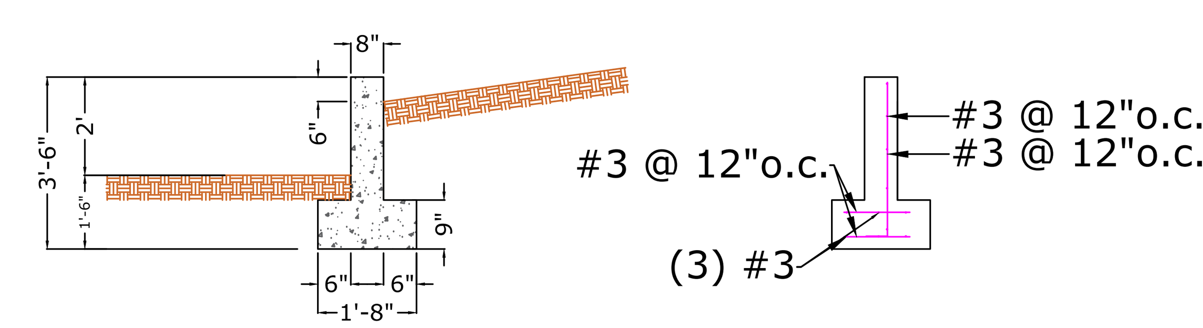
COLUMN TO FOOTING DETAIL



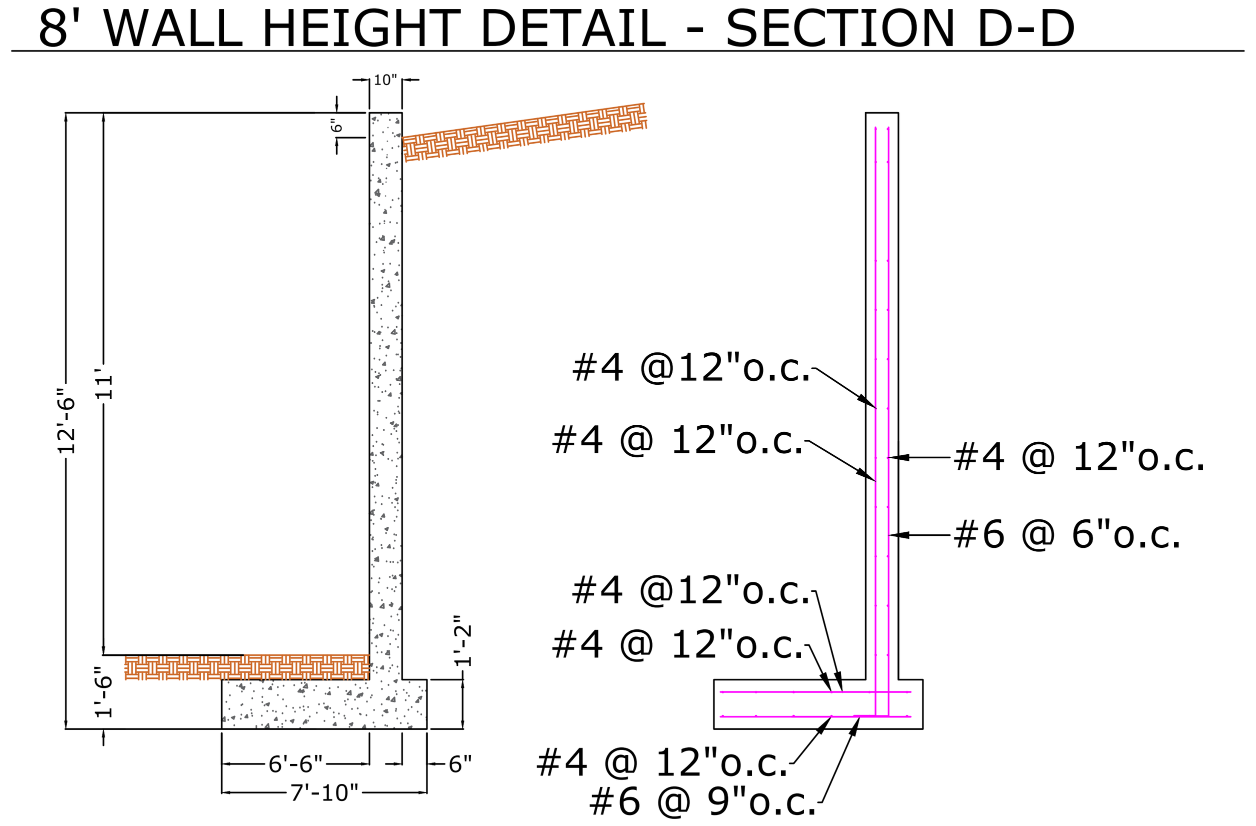
6' WALL HEIGHT DETAIL - SECTION C-C



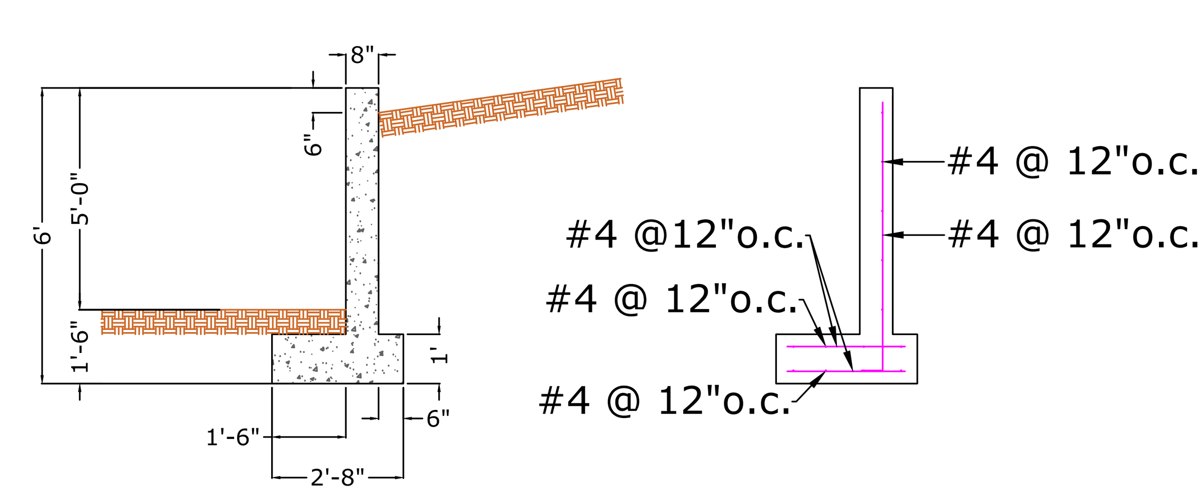
8' WALL HEIGHT DETAIL - SECTION D-D



STANDARD FOOTING DETAIL - SECTION A-A



10' WALL HEIGHT DETAIL - SECTION E-E



4' WALL HEIGHT DETAIL - SECTION B-B

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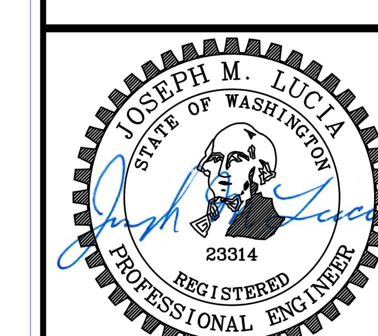

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FIRST FLOOR LEVEL FRAMING MEMBER DESIGN

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GREEN ROOF LOADING.

FRAMING D.L = 25^{PSF}

ROOF MEMBRANE D.L. = 0.05^{PSF}

COVER BOARD D.L. = 2.50^{PSF}

3.5 MIL RIDGID INSULATION D.L. = 1.5^{PSF}

4.5^{IN} SOIL D.L. = 45^{PSF}

TOTA D.L. $\cong 75^{PSF}$

L.L. = 40^{PSF}

DESIGN LOAD = $75 \times 1.2 + 40 \times 1.6 = 154^{PSF}$

MAX TJI SPAN = $23'$

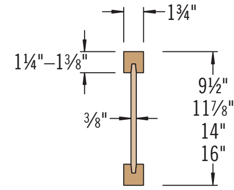
SECTION 1: 9½"–16" TJI® JOISTS

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).

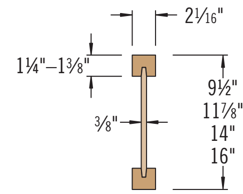
Design Properties (100% Load Duration)

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

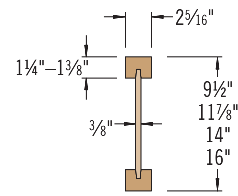
(1) **Caution: Do not** increase joist moment design properties by a repetitive member use factor.
(2) See detail W on page 27 for web stiffener requirements and nailing information.



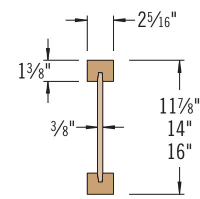
TJI® 110 joists



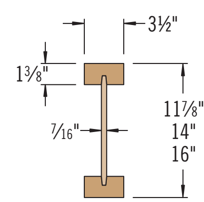
TJI® 210 joists



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, 230, 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 cantilever
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
$\mathbf{M}_{Allowable}$	=	**3,000**	$inches^4$
$\mathbf{V}_{Allowable}$	=	**1,330**	$inches^3$
$\mathbf{Pe}_{Allowable}$	=	**1330**	PSI
$\mathbf{Ps}_{Allowable}$	=	**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

$\mathbf{W_{max}}$	=	W x Joist Spacing	=	**147**	PLF
$\mathbf{V_{max}}$	=	$\mathbf{WL/2 + P_L}$	=	**1,323**	lbs
$\mathbf{P_{max}}$	=	$\mathbf{V_{max} + P_L}$	=	**1,323**	lbs
$\mathbf{M_1}$	=	$\mathbf{((W / (L^2 \times 8)) \times ((L + A)^2 * (L-A)^2)}$	=	**132**	ft-lbs
$\mathbf{M_2}$	=	$\mathbf{WA^2 / 2}$	=	**2,223**	ft-lbs
$\mathbf{D_{max}}$	=	$(5\mathbf{WL}^4/ 384\mathbf{EI})+(PL^3/48EI)$	=	**0.00**	inches
			=		

Check Shear:

$\mathbf{V_{max}}$	=	=	**1,323**	PSI
$\mathbf{V}_{Allowable}$	=	=	**1,330**	PSI
			Check - O.K.	

Check Bearing:

$\mathbf{P_{max}}$	=	=	**1,323**	PSI
$\mathbf{Ps}_{Allowable}$	=		**2,656**	PSI
			Check - O.K.	

Check Bending:

$\mathbf{M_{max}}$	=	=	**2,223**	$inches^3$
$\mathbf{M}_{Allowable}$	=		**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
$M_{Allowable}$	=	**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}	=	$WL/2 + P_L$	=	**1,628**	lbs
P_{max}	=	$V_{max} + P_L$	=	**1,628**	lbs
M_1	=	$((W / (L^2 \times 8)) \times ((L+A)^2 * (L-A)^2)$	=	**4,486**	ft-lbs
M_2	=	$WA^2 / 2$	=	**1,588**	ft-lbs
D_{max}	=	$(5WL^4/384EI)+(PL^3/48EI)$	=	**0.17**	inches
			=		

Check Shear:

V_{max}	=	=	**1,628**	PSI
$V_{Allowable}$	=	=	**2,190**	PSI

Check - O.K.

Check Bearing:

P_{max}	=	=	**1,628**	PSI
$Ps_{Allowable}$	=		**2,145**	PSI

Check - O.K.

Check Bending:

M_{max}	=	=	**4,486**	$inches^3$
$M_{Allowable}$	=		**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
$M_{Allowable}$	=	**3,000**	$inches^4$
$V_{Allowable}$	=	**1,330**	$inches^3$
$Pe_{Allowable}$	=	**1330**	PSI
$Ps_{Allowable}$	=	**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

W_{max}	=	W x Joist Spacing	=	**100**	PLF
V_{max}	=	**WL /2**	=	**771**	lbs
P_{max}	=	V_{max}	=	**771**	lbs
M_{max}	=	$(WL^2)/8$	=	**2,972**	ft-lbs
D_{max}	=	$5WL^4/384EI$	=	**0.06**	inches

Check Shear:

V_{max} = = **771** PSI

$V_{Allowable}$ = = **1,330** PSI

Check - O.K.

Check Bearing:

P_{max} = = **771** PSI

$Ps_{Allowable}$ = **2,145** PSI

Check - O.K.

Check Bending:

M_{max} = = **2,972** $inches^3$

$M_{Allowable}$ = **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
$M_{Allowable}$	=	**2,500**	$inches^4$
$V_{Allowable}$	=	**1,220**	$inches^3$
$Pe_{Allowable}$	=	**1220**	PSI
$Ps_{Allowable}$	=	**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

W_{max}	=	W x Joist Spacing	=	**250**	PLF
V_{max}	=	**WL /2**	=	**1,115**	lbs
P_{max}	=	V_{max}	=	**1,115**	lbs
M_{max}	=	$(WL^2)/8$	=	**2,486**	ft-lbs
D_{max}	=	$5WL^4/384EI$	=	**0.02**	inches

Check Shear:

V_{max} = = **1,115** PSI

$V_{Allowable}$ = = **1,220** PSI

Check - O.K.

Check Bearing:

P_{max} = = **1,115** PSI

$Ps_{Allowable}$ = **1,935** PSI

Check - O.K.

Check Bending:

M_{max} = = **2,486** $inches^3$

$M_{Allowable}$ = **2,500** $inches^3$

Check - O.K.

Check Deflection:

$D_{max.}$	=			**0.02** inches
$D_{Allowable}$	=	**L**/350	=	**0.31** inches
				Check - O.K.

SECOND FLOOR LEVEL FRAMING MEMBER DESIGN

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 $= 10.5^{PSF} \times 1.2 + 40^{PSF} \times 1.4$

$= \underline{58.1^{PSF}}$

USE RED-L OPEN WEB 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 $= 215^{PLF}$ ✓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

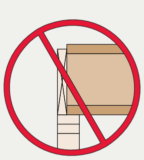
1. Determine the appropriate live load deflection criteria.
2. Identify the live and dead load condition.
3. Select on-center spacing.
4. Scan down the column until you meet or exceed the span of your application.
5. Select TJI® joist and depth.

General Notes

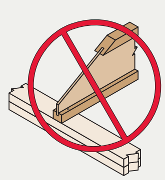
- Tables are based on:
  - Uniform loads.
  - More restrictive of simple or continuous span.
  - Clear distance between supports
  - Minimum bearing length of 1¾" end (no web stiffeners) and 3½" intermediate.
- Assumed composite action with a single layer of 24" on-center span-rated, glue-nailed floor 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.

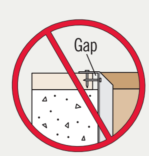
These Conditions Are NOT Permitted:



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}	=	$0.5WL$	=	**4,858**	lbs
P_{max}	=	$0.5WL$	=	**4,858**	lbs
M_{max}	=	$(WL^2)/8$	=	**14,574**	ft-lbs
D_{max}	=	$0.013WL^4 / EI$	=	**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.

Second Level Header Beam - 16' Span

24F DOUGLAS-FIR

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.

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
$\mathbf{V}_{max}$	=	**WL /2 +** $\mathbf{P_L}$	=	**1,840**	lbs
$\mathbf{P}_{max}$	=	$\mathbf{V}_{max}$ **+**$\mathbf{P_L}$	=	**1,840**	lbs
$\mathbf{M}_{max}$	=	**(WL^2)**/8 + **(P$_L$*L/4)**	=	**10,580**	ft-lbs
$\mathbf{D}_{max}$	=	(5**WL**^{4}/ 384**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	$0.6\mathbf{F_y}$	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	$0.66\mathbf{F_y}$	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
$Fv_{Allowable}$	=	$0.6\mathbf{F_y}$	=	**30,000**	PSI
				Check - O.K.	
		V_{max}	=	**13,689**	LBS
$Fv_{Allowable}$	=	$0.66F_yt_W(N + 2.5k)$ ASD Section K1.3	=	**88,785**	LBS
				Check - O.K.	

Check Bearing-Web Crippling:

Fp	=	**P_{max} / A_{web}**	=	**3,388**	PSI
$Fp_{Allowable}$	=	$0.6\mathbf{F_y}$	=	**30,000**	PSI
				Check - O.K.	
		V_{max}	=	**13,689**	LBS
$Fp_{Allowable}$	=	$0.66F_yt_W(N + 2.5k)$ ASD Section K1.3	=	**88,785**	LBS
				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}	=	$0.013WL^4 / EI$	=	**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

24F DOUGLAS-FIR

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.

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.

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.

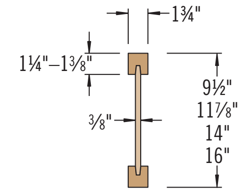
SECTION 1: 9½"–16" TJI® JOISTS

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).

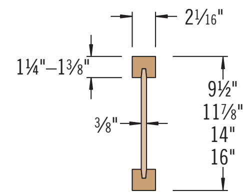
Design Properties (100% Load Duration)

Depth	TJI®	Basic Properties				Reaction Properties					
		Joist Weight (lbs/ft)	Maximum Resistive Moment(1) (ft-lbs)	Joist Only EI x 10⁶ (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

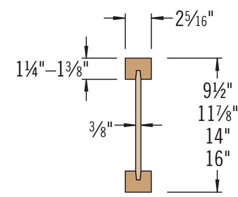
(1) **Caution: Do not** increase joist moment design properties by a repetitive member use factor.
(2) See detail W on page 27 for web stiffener requirements and nailing information.



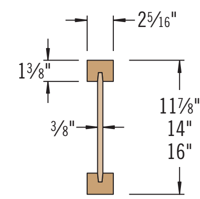
TJI® 110 joists



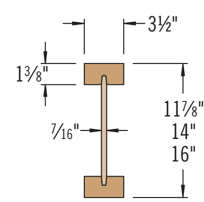
TJI® 210 joists



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).

For TJI® 110, 210, 230, 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

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.

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.

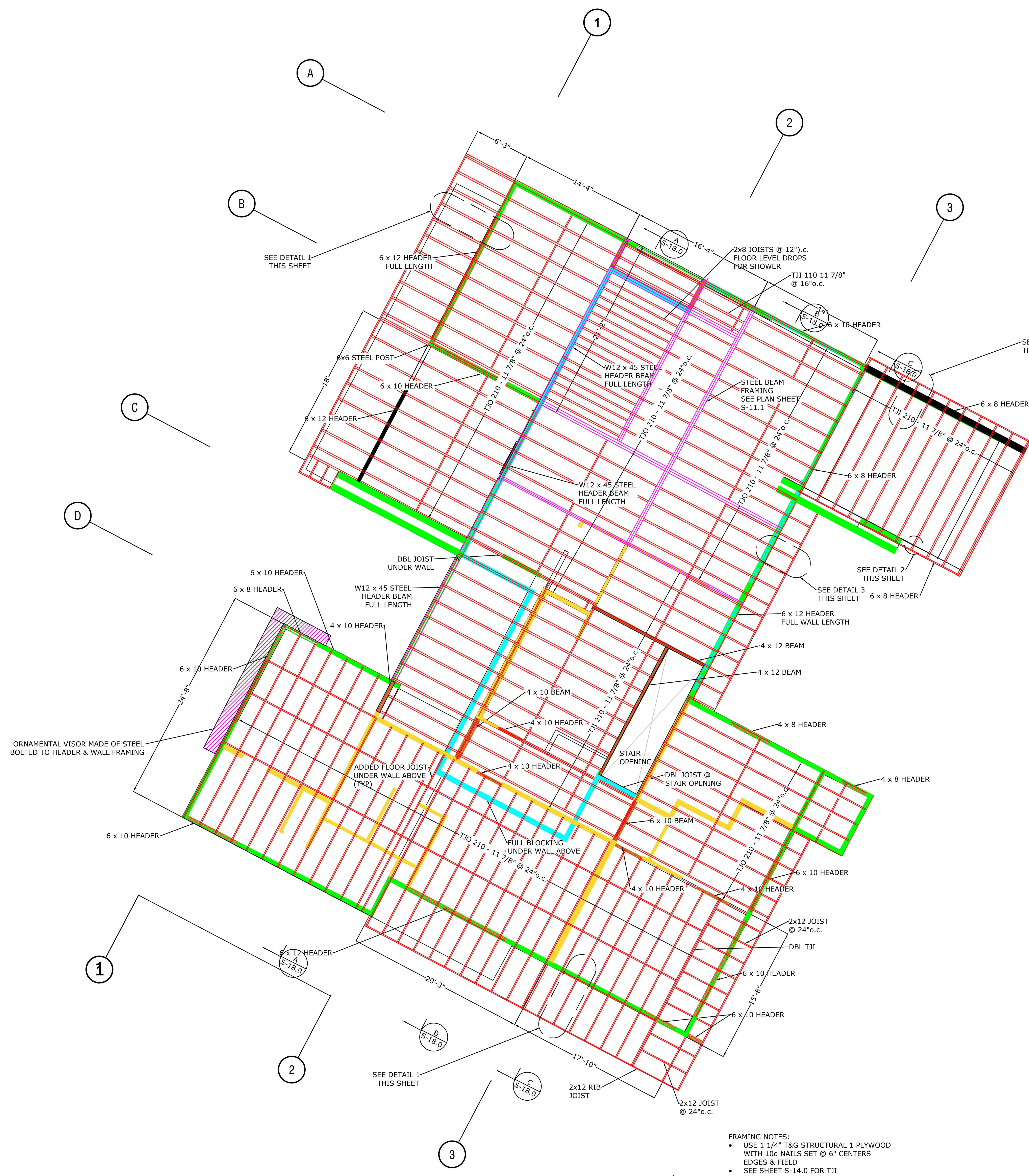
WARNING

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:*

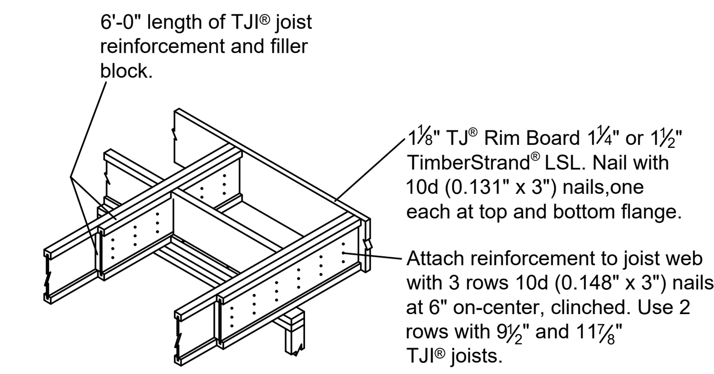
1. All blocking, hangers, rim boards, and rim joists at the end supports of the TJI® joists must be completely installed and properly nailed.
2. Lateral strength, like a braced end wall or an existing deck, must be established at the ends of the bay. This can also be accomplished by a temporary or permanent 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.



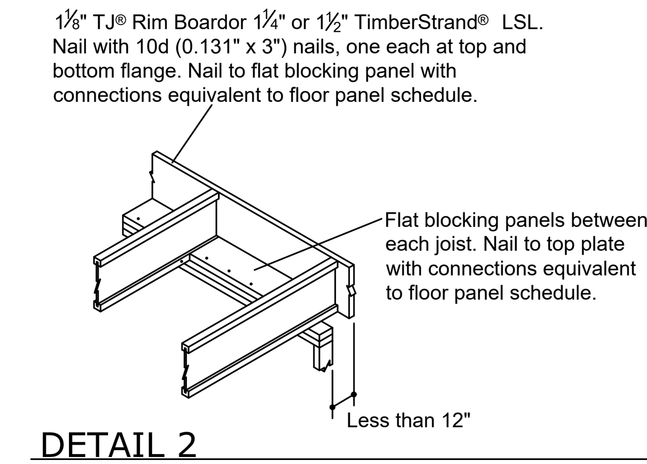
FRAMING NOTES:

- USE 1 1/4" T&G STRUCTURAL 1 PLYWOOD WITH 10d NAILS SET @ 6" CENTERS EDGES & FIELD
- SEE SHEET S-14.0 FOR TJI FRAMING DETAILS

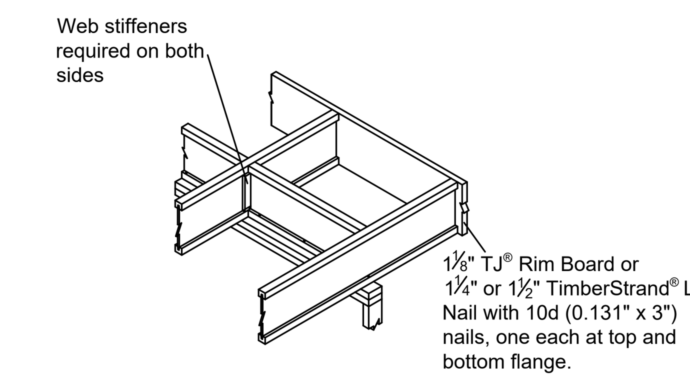
SECOND FLOOR - FLOOR FRAMING



DETAIL 1



DETAIL 2



DETAIL 3

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

Permanent Soldier Pile & Timber Lagging Retaining Wall

LUCIA E N G I N E E R I N G, I N C.
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
04-27-24

Number	Date	By	Description
3	04-27-24	JML	

SHEET
S-11.0

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
$M_{Allowable}$	=	**3,000**	$inches^4$
$V_{Allowable}$	=	**1,005**	$inches^3$
$Pe_{Allowable}$	=	**1330**	PSI
$Ps_{Allowable}$	=	**1330**	PSI

Reactions: From Uniform Loading:

W = Factored Floor DL +LL = **80** PSF

W_{max}	=	W x Joist Spacing	=	**160**	PLF
V_{max}	=	$WL/2 + P_L$	=	**1,840**	lbs
P_{max}	=	$V_{max} + P_L$	=	**1,840**	lbs
M_{max}	=	$(WL^2)/8 + (P_L*L/4)$	=	**10,580**	ft-lbs
D_{max}	=	$(5WL^4/384EI)+(PL^3/48EI)$	=	**0.45**	inches
			=		

Check Shear:

V_{max}	=		=	**1,840**	PSI
$V_{Allowable}$	=		=	**1,330**	PSI

Check - O.K.

Check Bearing:

P_{max}	=		=	**1,840**	PSI
$Ps_{Allowable}$	=			**1,330**	PSI

Check - O.K.

Check Bending:

M_{max}	=		=	**10,580**	$inches^3$
$M_{Allowable}$	=			**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.

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}	=	$0.013WL^4 / EI$	=	**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}	=	$0.013WL^4 / EI$	=	**0.42**	inches

Check Shear:

F_v	=	$V_{max}(1.5) / A$	=	**4,858**	lbs
$Fv_{Allowable}$	=			**6,720**	lbs
				Check - O.K.	

Check Bearing:

F_p	=	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

24F DOUGLAS-FIR

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.

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.

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:	$\mathbf{L_d}$	**=**	318	Inches
3. Bearing area:	(Assume: 6"x6" Plate)	**N**	=	**6**	Inches

W10 x 100 Beam Properties:

A	=	**29.40**	inches^2		
$\mathbf{A_{web}}$	=	**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		
$\mathbf{Fv_{Allowable}}$	=	**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	=	**33,000.00**	PSI	$0.66\mathbf{F_y}$	AISC (F3-1)
$\mathbf{t_w}$	=	**0.375**	inches		
d	=	**11.1**	inches		
$\mathbf{t_f}$	=	**1.12**	inches		
$\mathbf{F_y}$	=	**50,000**	PSI		
k	=	**1.00**	inches		
$\mathbf{b_f}$	=	**10.3**	inches		
$\mathbf{W_t}$	=	**100**	lbs/lf		

Reactions:

$\mathbf{W_{design}}$	=	=	**1,325**	plf
$\mathbf{V_{max}}$	=	=	**17,888**	lbs
$\mathbf{P_{max}}$	=	=	**17,888**	lbs
$\mathbf{M_{max}}$	=	=	**120,741**	ft-lbs
$\mathbf{D_{max}}$	=	=	**0.877**	inches

Check Shear:

F_v	=	$\mathbf{V_{max} / A_{web}}$	=	**4,307**	PSI
$Fv_{Allowable}$	=	$0.6\mathbf{F_y}$	=	**30,000**	PSI
				Check - O.K.	
		$\mathbf{V_{max}}$	=	**17,888**	LBS
$Fv_{Allowable}$	=	$0.66F_wt_w(N + 2.5k)$ ASD Section K1.3	=	**105,188**	LBS
				Check - O.K.	

Check Bearing-Web Crippling:

F_p	=	$\mathbf{P_{max} / A_{web}}$	=	**4,297**	PSI
$Fp_{Allowable}$	=	$0.6\mathbf{F_y}$	=	**30,000**	PSI
				Check - O.K.	
		$\mathbf{V_{max}}$	=	**17,888**	LBS
$Fp_{Allowable}$	=	$0.66F_wt_w(N + 2.5k)$ ASD Section K1.3	=	**105,188**	LBS
				Check - O.K.	

Check Bending:

$S_{Req'd}$	=	$\mathbf{M_{max}}(12) / \mathbf{Fb}$	=	**43.9**	inches^3
$S_{Furnished}$	=			**112**	inches^3
				Check - O.K.	

Check Deflection:

$\mathbf{D_{max.}}$	=	$\mathbf{D_{max}}$	=	**0.877**	inches
$\mathbf{D_{Allowable}}$	=	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	$0.6\mathbf{F_y}$	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	$0.66\mathbf{F_y}$	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
$Fv_{Allowable}$	=	$0.6\mathbf{F_y}$	=	**30,000**	PSI
				Check - O.K.	
		V_{max}	=	**21,863**	LBS
$Fv_{Allowable}$	=	$0.66F_wt_w(N + 2.5k)$ ASD Section K1.3	=	**141,848**	LBS
				Check - O.K.	

Check Bearing-Web Crippling:

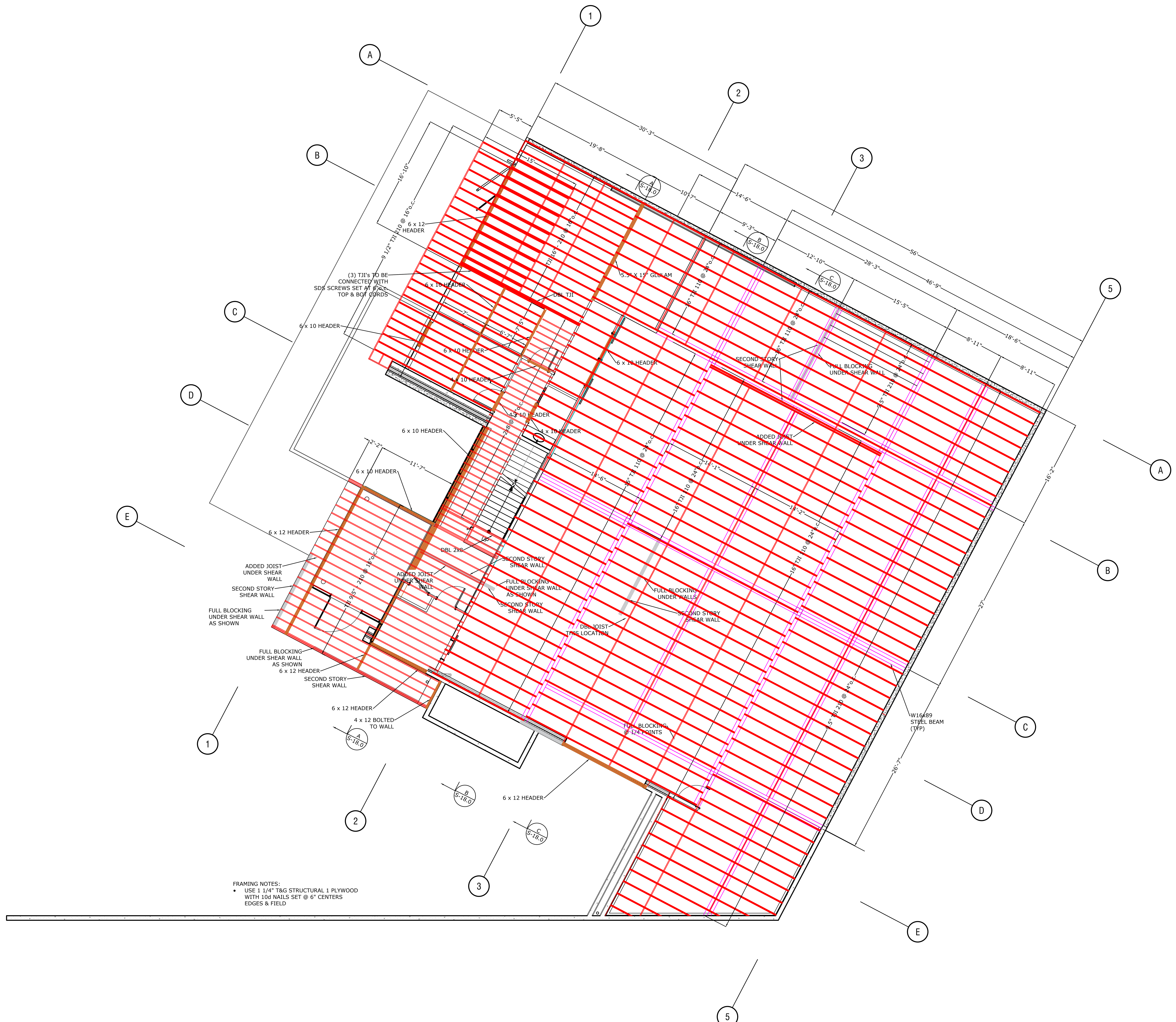
Fp	=	**P_{max} / A_{web}**	=	**2,486**	PSI
$Fp_{Allowable}$	=	$0.6\mathbf{F_y}$	=	**30,000**	PSI
				Check - O.K.	
		V_{max}	=	**21,863**	LBS
$Fp_{Allowable}$	=	$0.66F_wt_w(N + 2.5k)$ ASD Section K1.3	=	**141,848**	LBS
				Check - O.K.	

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.	



FRAMING NOTES:

- USE 1 1/4" T&G STRUCTURAL 1 PLYWOOD WITH 10d NAILS SET @ 6" CENTERS EDGES & FIELD

FIRST FLOOR - FLOOR FRAMING

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
04-21-24

Number	Date	By	Description
2	04-21-24	JML	

SHEET
S-12.0

ROOF MEMBER DESIGN

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:	$\mathbf{L_1}$	=	16
	$\mathbf{L_2}$	=	14.5
	$\mathbf{L_3}$	=	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
$\mathbf{t_s}$	=	**0.5** inches
A	=	**2.884** inches^2
I	=	**0.075** inches^3
KS	=	**0.267** inches^4
lb/Q	=	**4.891** inches^2
$\mathbf{F_b}$	=	**1930** PSI
$\mathbf{F_s}$	=	**72** PSI
$\mathbf{E_e}$	=	**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 I 53.3 PSF

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

Check - O.K.

B) Check Rolling Shear:

$Fs_{Req'd}$ = $W(L_2) / 20(lb/Q)$ = **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}	=	$0.013WL^4 / EI$	=	**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.

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}	=	$0.013WL^4 / EI$	=	**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

24F DOUGLAS-FIR

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.

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}	=	$0.013\mathbf{WL^4} / \mathbf{EI}$	=	**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
$\mathbf{Fv}_{\text{Allowable}}$	=	**145**	PSI
$\mathbf{Fp}_{\text{Allowable}}$	=	**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

$\mathbf{V}_{\mathbf{max}}$	=	0.5**WL**	=	**1,705**	lbs
$\mathbf{P}_{\mathbf{max}}$	=	**0.5WL**	=	**1,705**	lbs
$\mathbf{M}_{\mathbf{max}}$	=	$(WL^2)/8$	=	**2,984**	ft-lbs
$\mathbf{D}_{\mathbf{max}}$	=	$0.013\mathbf{WL}^4 / \mathbf{EI}$	=	**0.04**	inches

Check Shear:

Fv = $V_{max}(1.5) / \mathbf{A}$ = **101** PSI

$Fv_{Allowable}$ = **145** PSI

Check - O.K.

Check Bearing:

Fp = $P_{max} / \mathbf{A}$ = **56** PSI

$Fp_{Allowable}$ = 405 PSI

Check - O.K.

Check Bending:

$S_{Req'd}$ = $M_{max}(12) / \mathbf{Fb}$ = **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}	=	0.013**WL**^{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

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

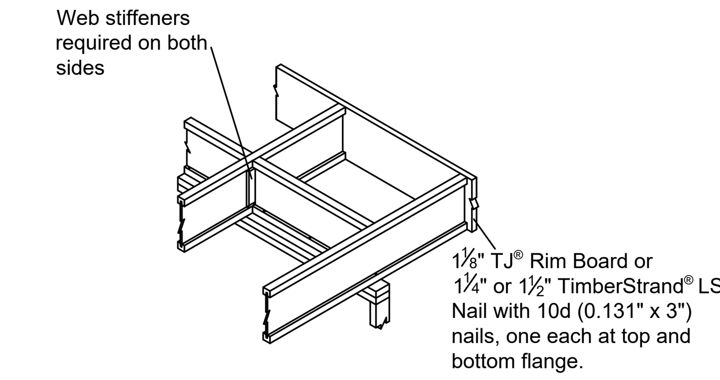
6-3/4-INCH WIDTH

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

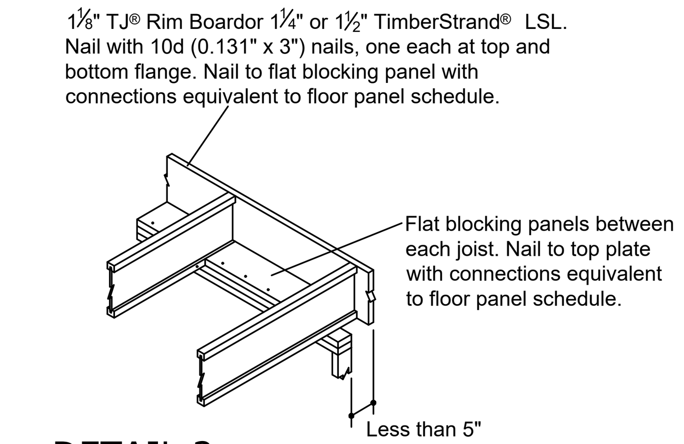
8-3/4-INCH WIDTH

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

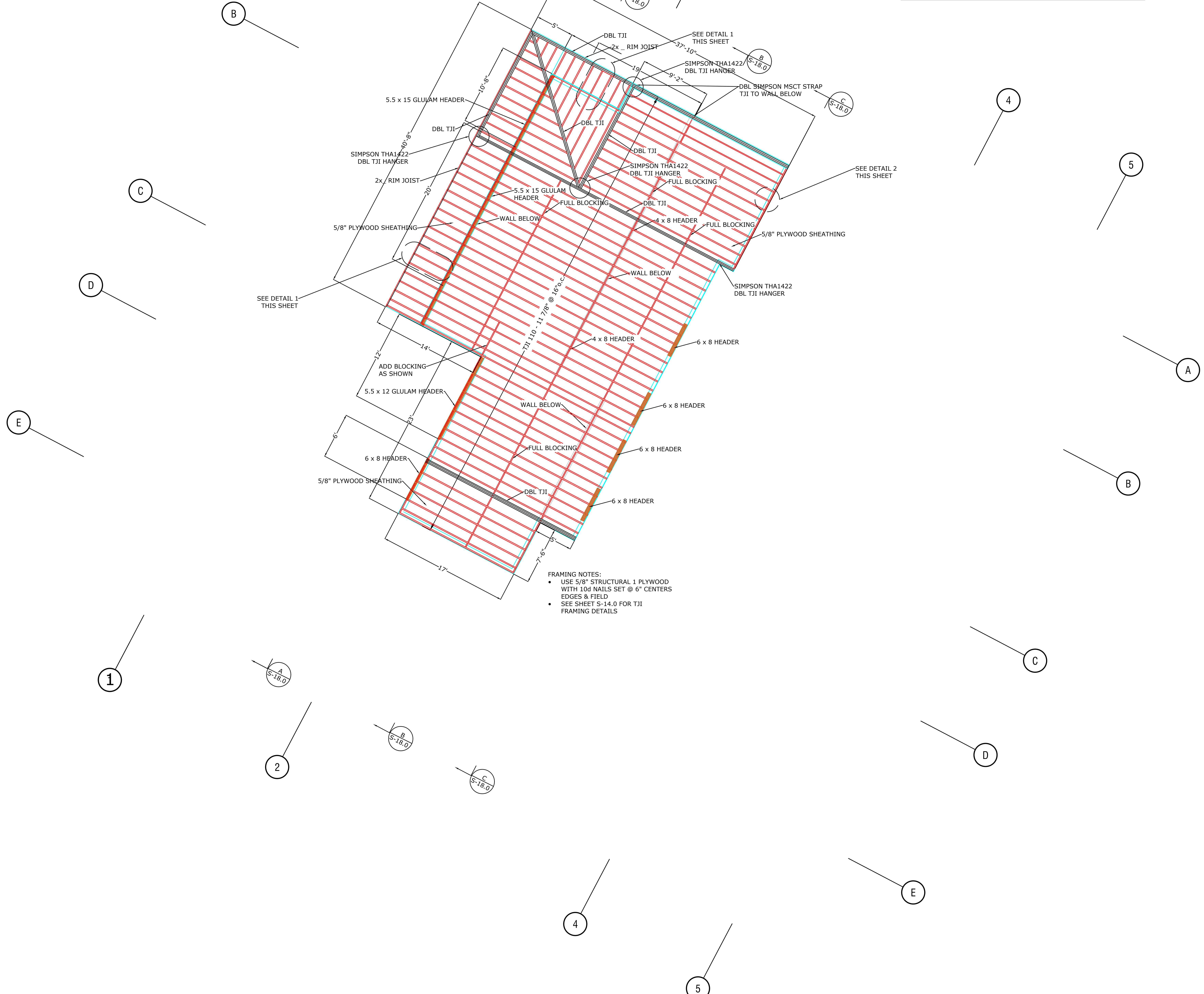
Notes:
(1) Beam weight is based on density of 35 pcf.
(2) Moment capacity must be adjusted for volume effect. The volume factor for various glulam sizes and simple spans, as well as the complete formula, is given in Appendix A.
(3) Moment and shear capacities are based on a normal (10 years) duration of load and should be adjusted for the design duration of load per the applicable building code.



DETAIL 1



DETAIL 2



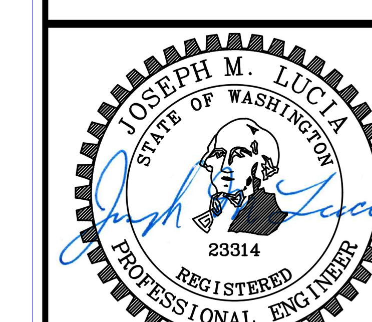
FRAMING NOTES:
- USE 5/8" STRUCTURAL 1 PLYWOOD WITH 10d NAILS SET @ 6" CENTERS EDGES & FIELD
- SEE SHEET S-14.0 FOR TJI FRAMING DETAILS

ROOF FRAMING

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

Permanent Soldier Pile
& Timber Lagging
Retaining Wall

LUCIA E N G I N E E R I N G, I N C.
12527 Huckleberry Lane
Arlington, Washington 98223
PHONE: (206) 790-8039
E-MAIL: joe@luciaeng.com



04-27-24

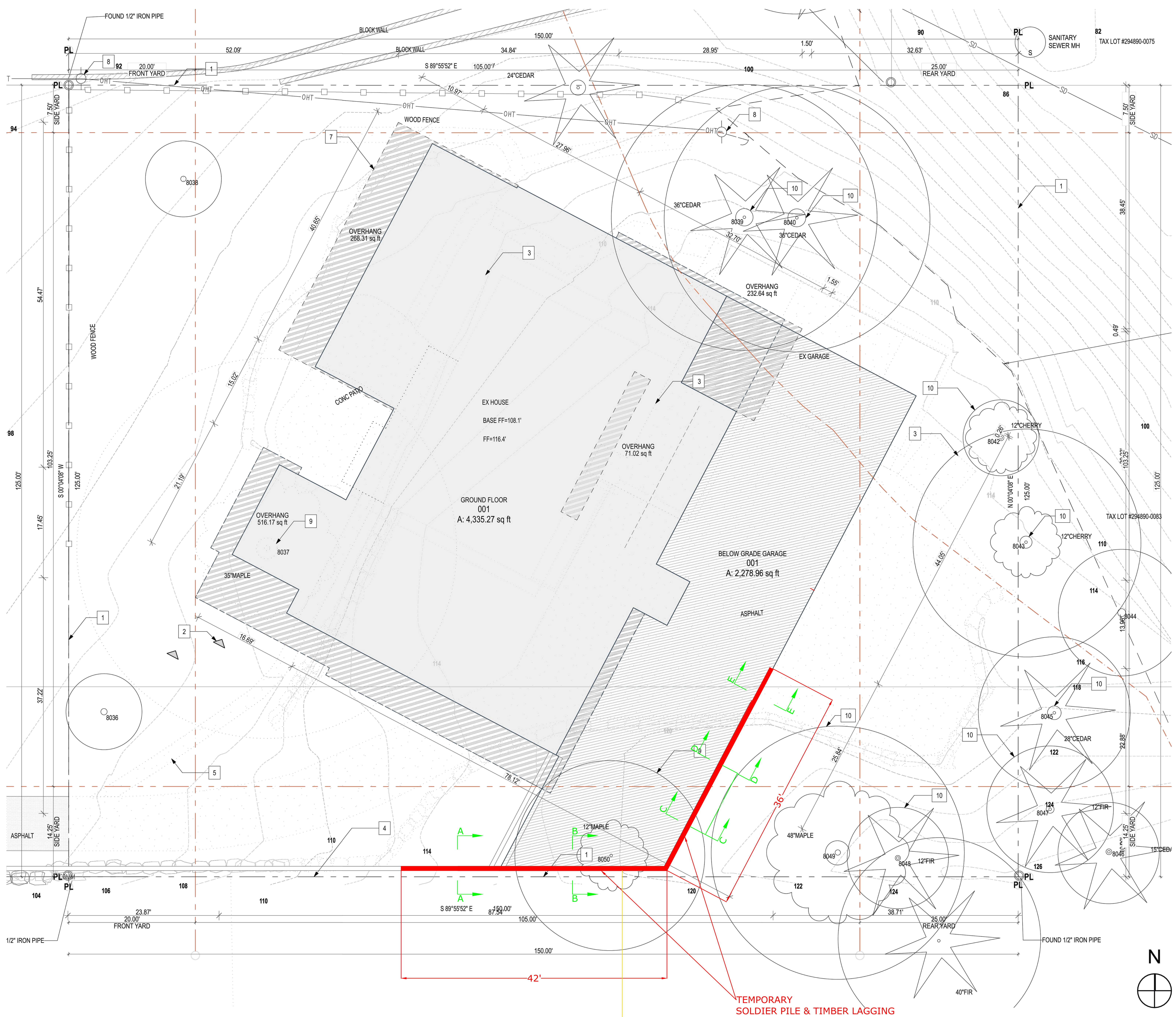
Number	Date	By	Description
3	04-27-24	JML	

SHEET
S-10.0

PLAN SET

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

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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
04-27-24

Number	Date	By	Description
3	04-27-24	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:

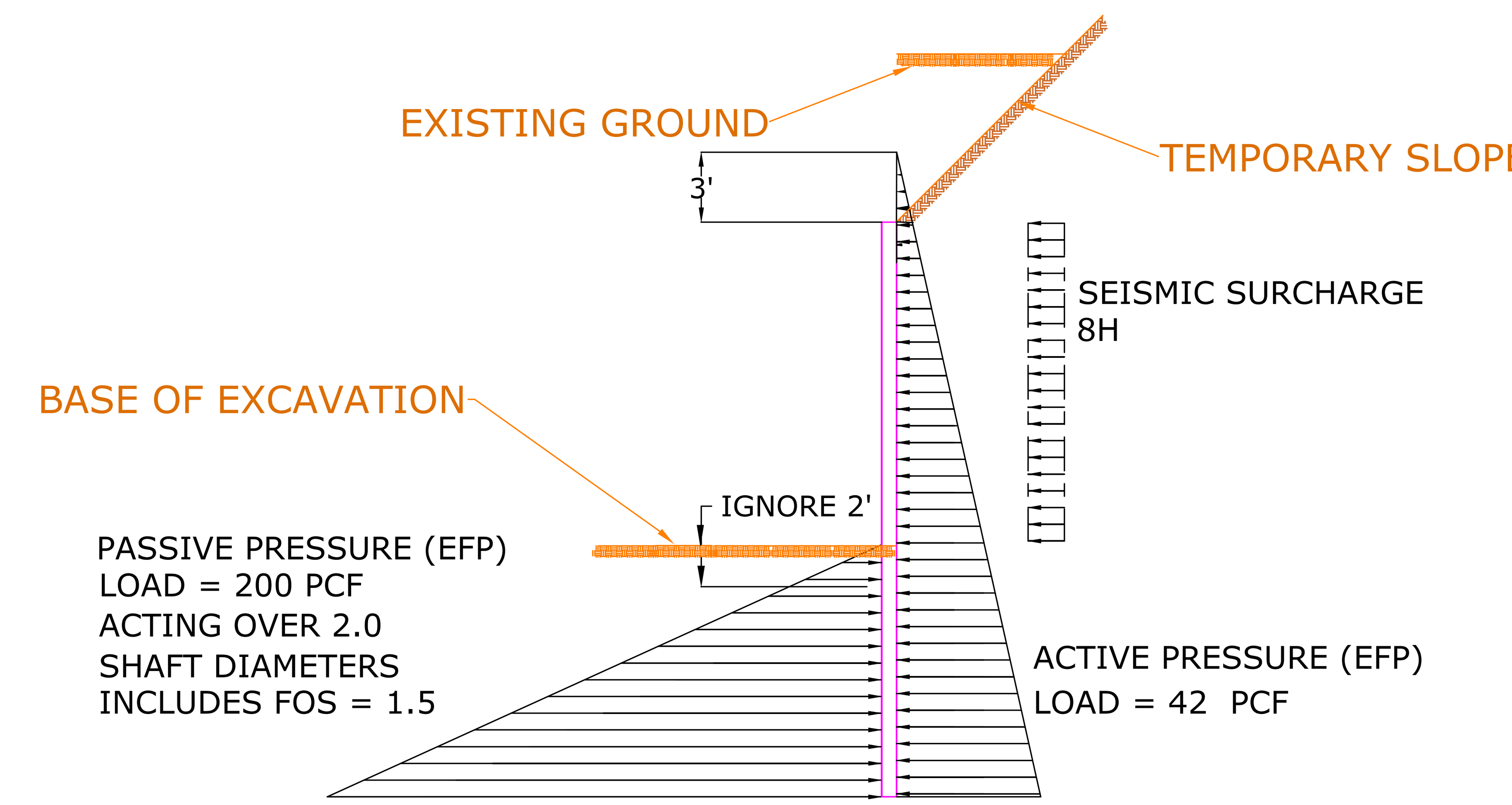
ALL EXISTING UTILITIES AND OTHER OBJECTS WHICH MAY INTERFERE WITH THE INSTALLATION OF THE SHORING SYSTEM ARE TO BE LOCATED PRIOR TO BEGINNING CONSTRUCTION.

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
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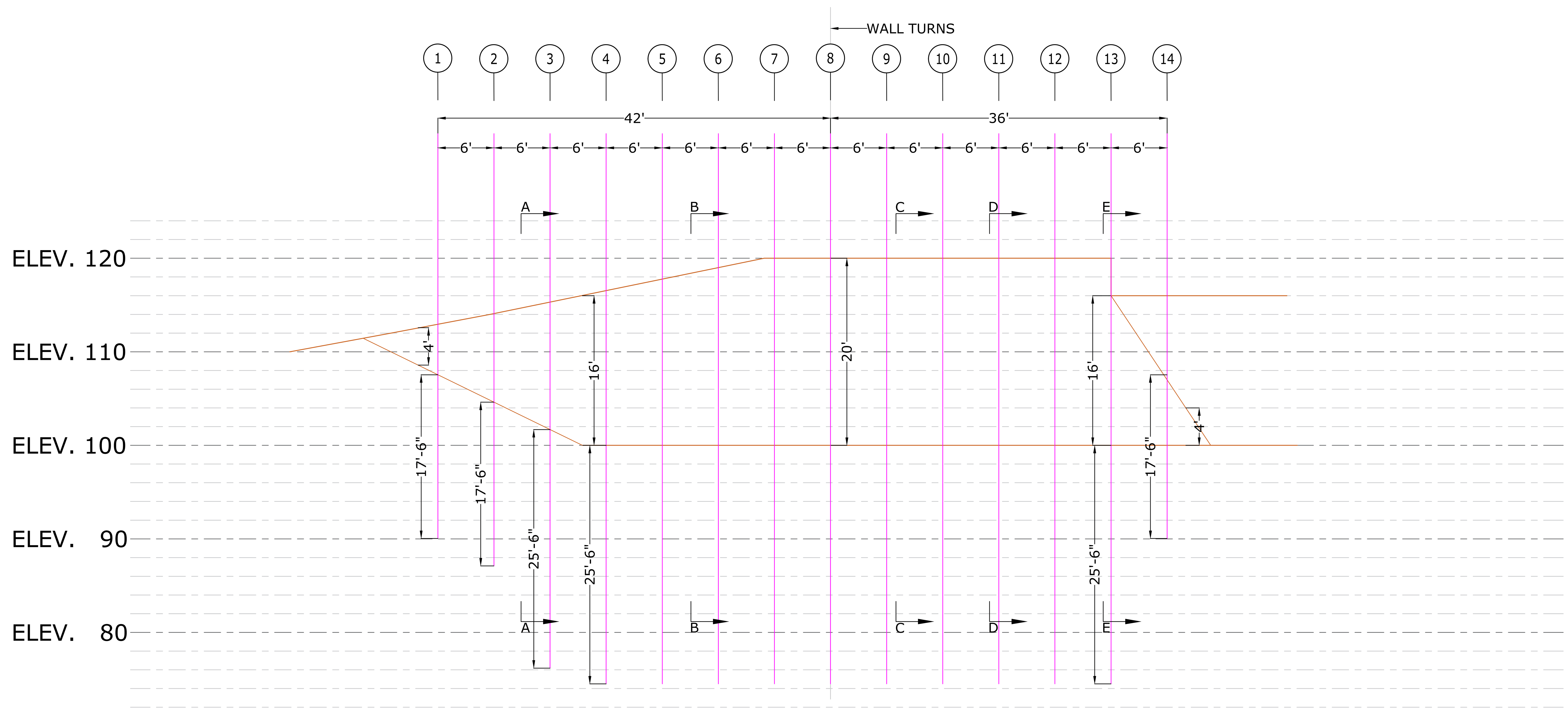
E-MAIL: joe@luciaeng.com

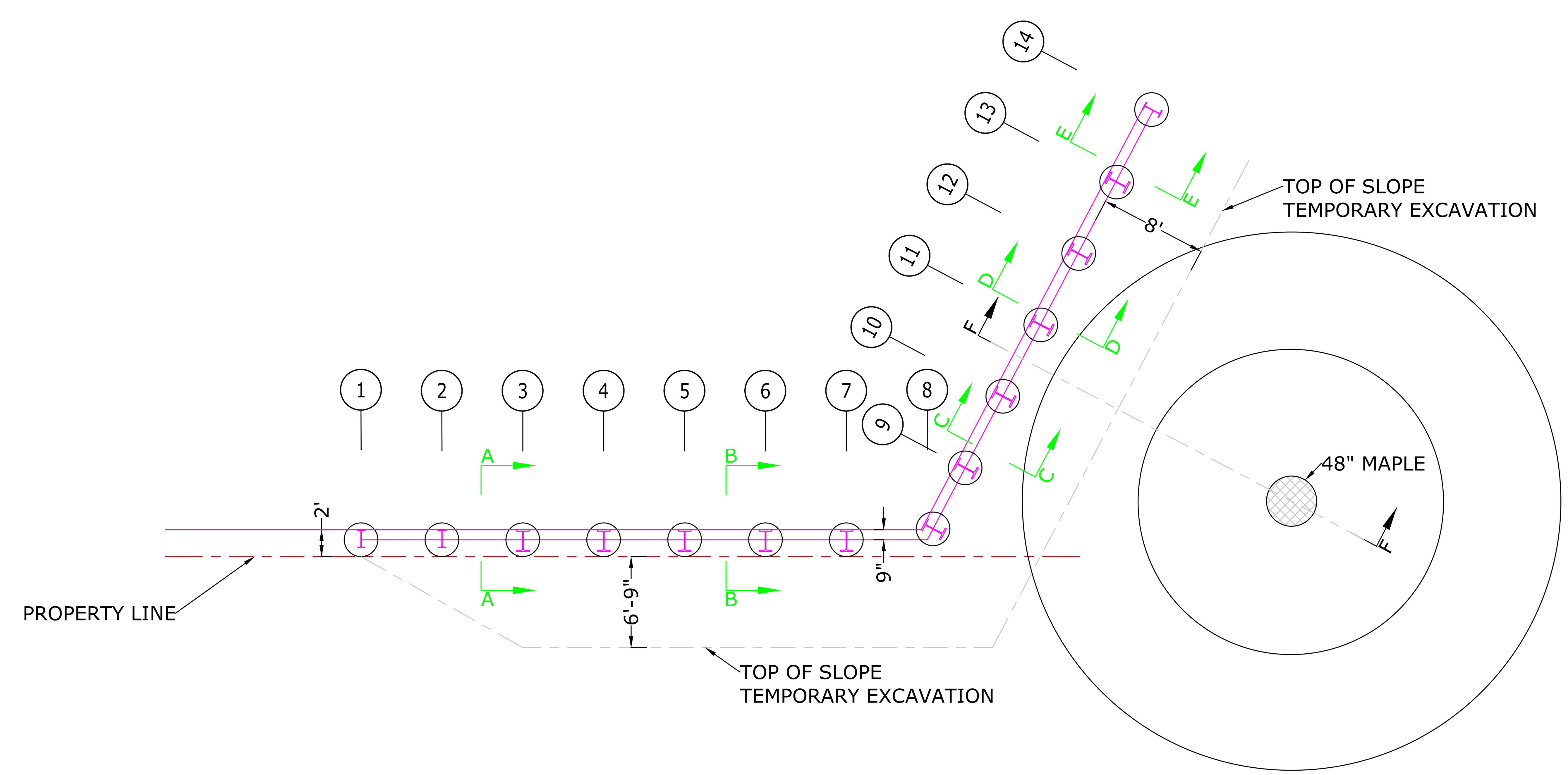
JOSEPH M. LUCIA
STATE OF WASHINGTON
23314
REGISTERED PROFESSIONAL ENGINEER
04-27-24

Number	Date	By	Description
3	04-27-24	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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JOSEPH M. LUCIA
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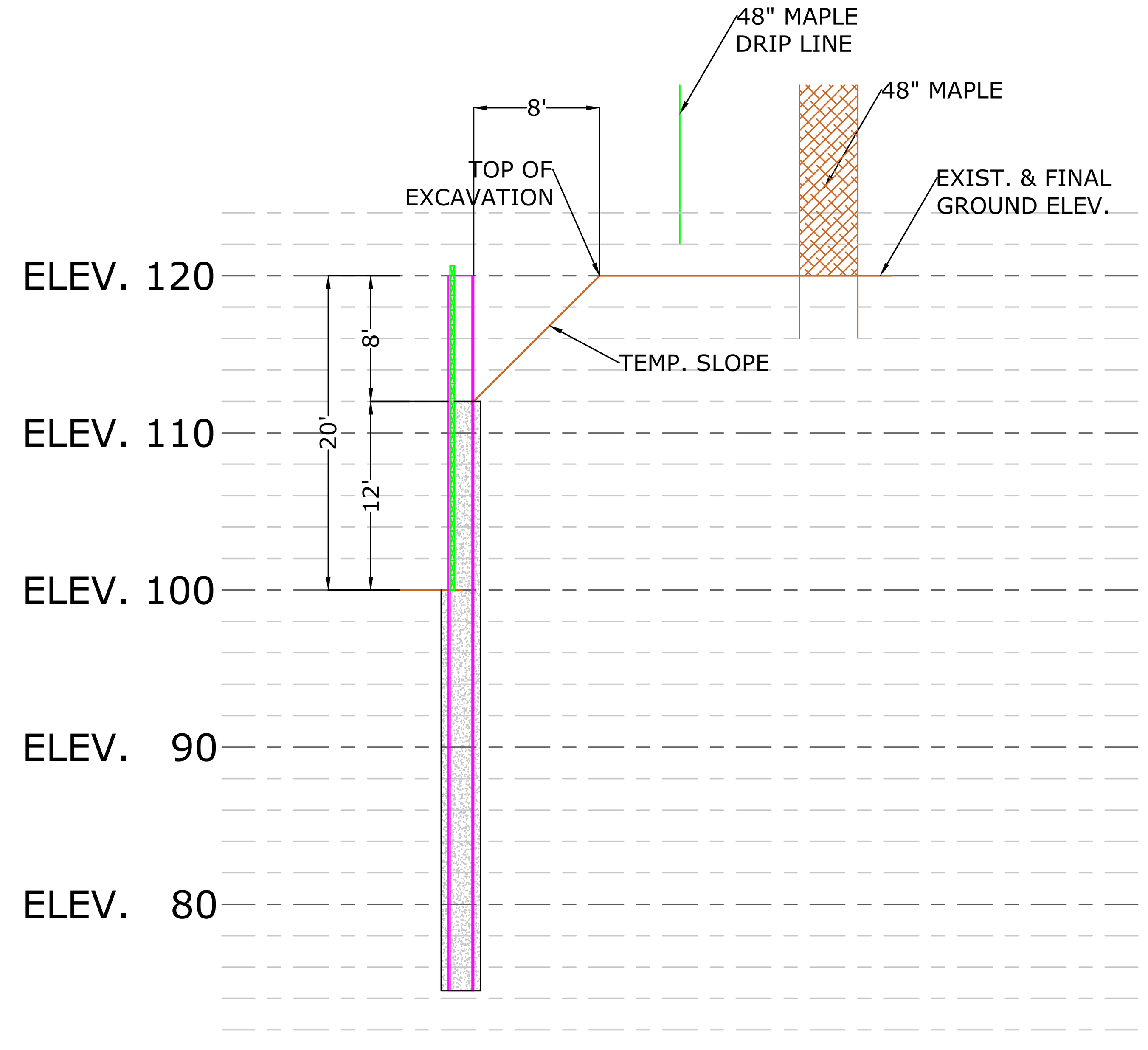
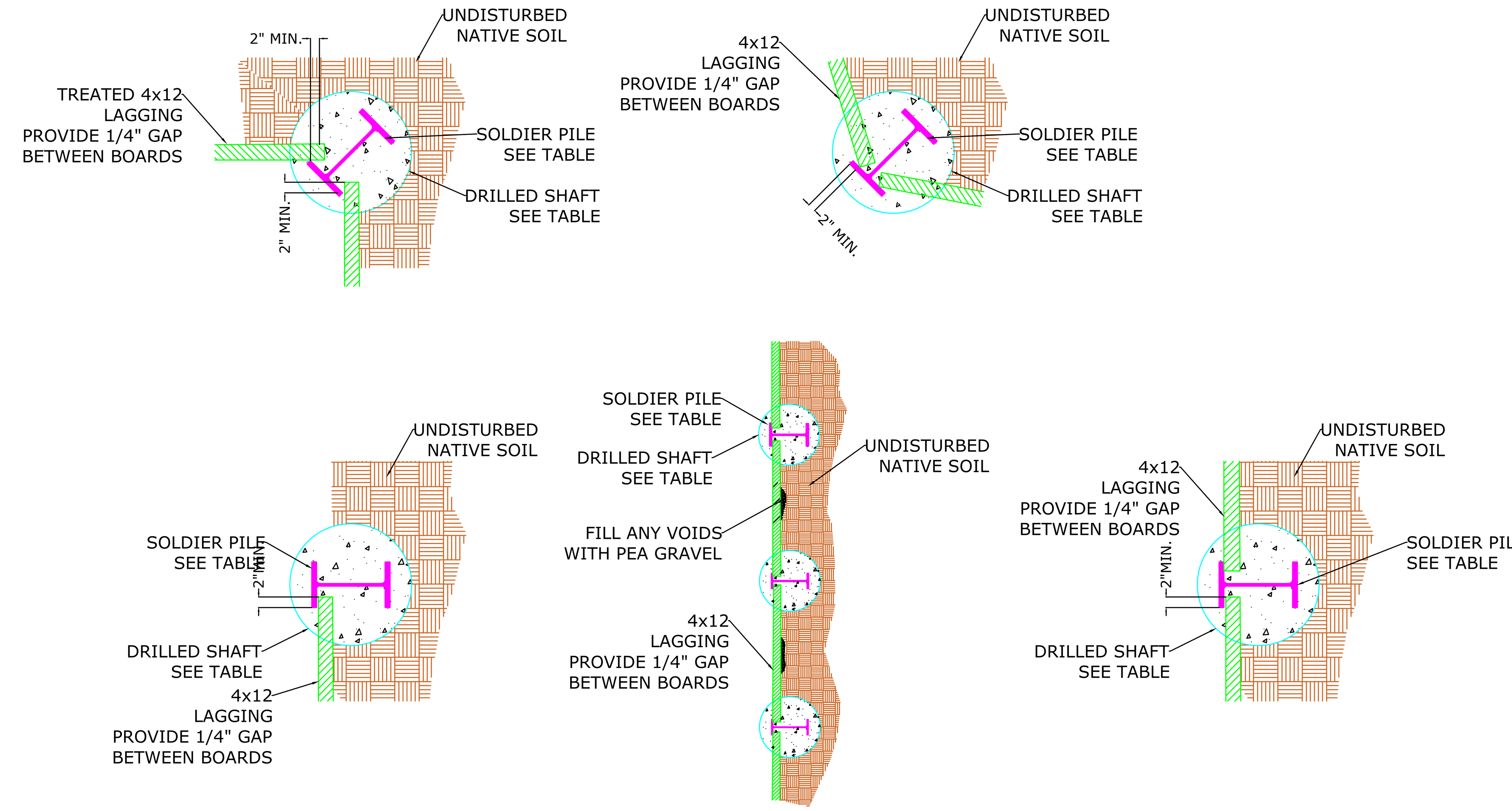
04-27-24

Number	Date	By	Description
3	04-27-24	JML	

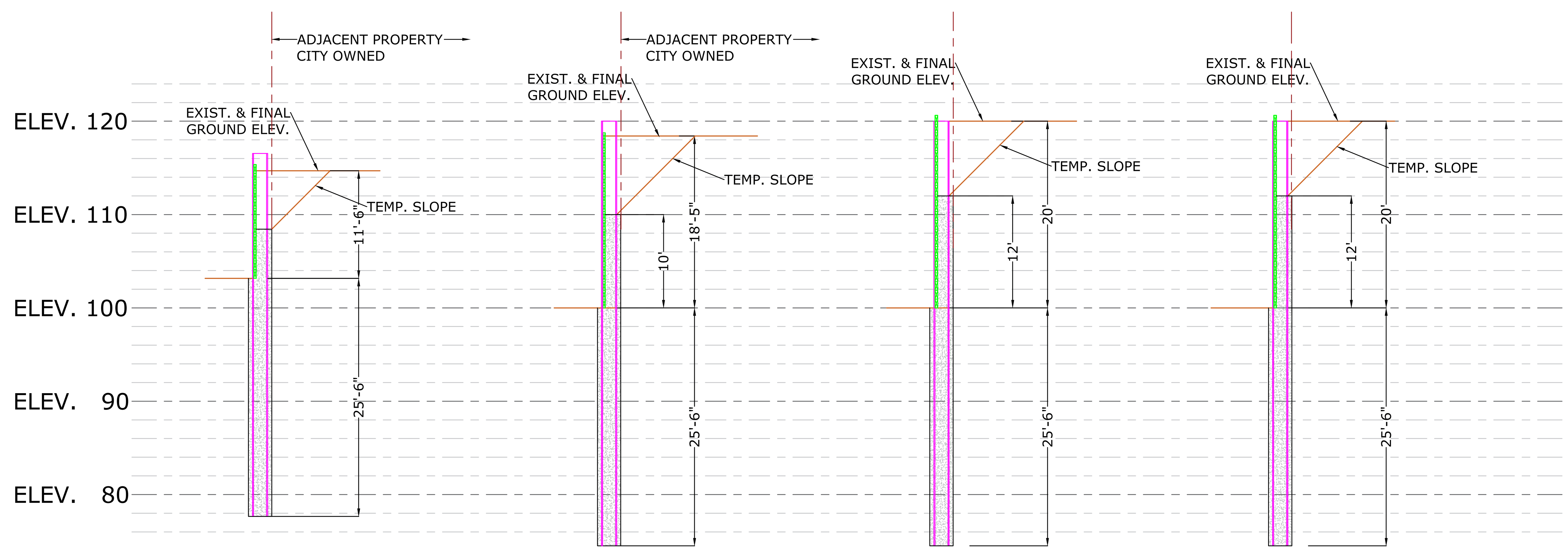
SHEET
S-3.0



SECTION F-F



TYPICAL DETAILS - SOLDIER PILE & TIMBER LAGGING

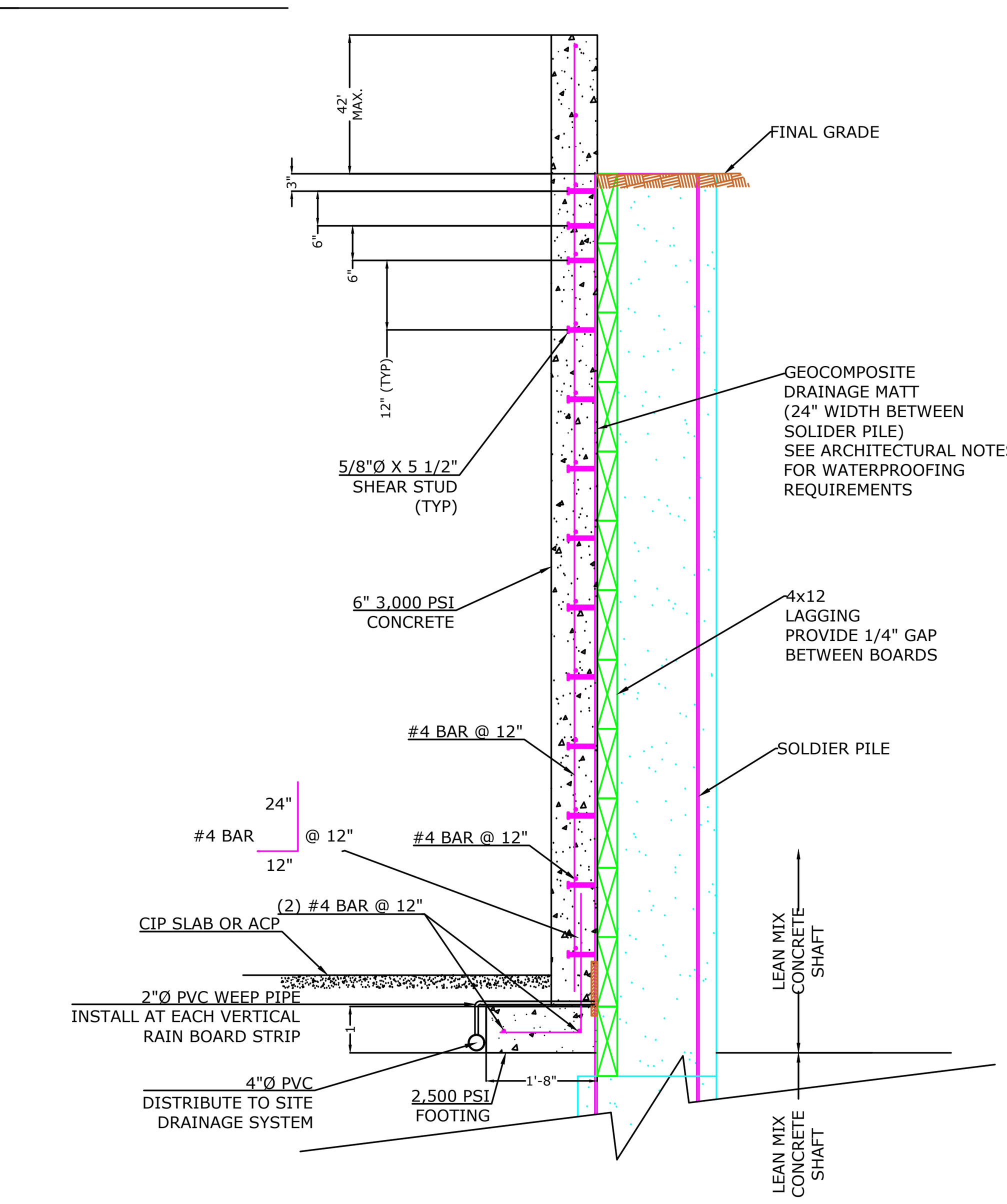


SECTION A-A

SECTION B-B

SECTION C-C & D-D

SECTION E-E



FUTURE FASCIA WALL DETAIL

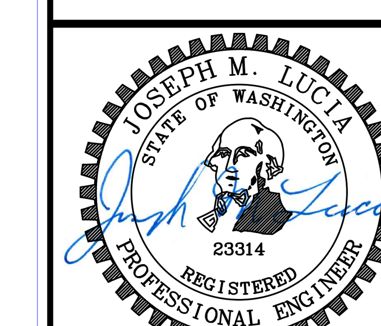
PILE INFORMATION

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
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



04-27-24

Number	Date	By	Description
3	04-27-24	JML	

SHEET
S-4.0

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:
$S_s = 1.466$
$S_1 = 0.508$ SITE CLASS = D
$S_{ds} = 1.173$ 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

Permanent Soldier Pile
& Timber Lagging
Retaining Wall

LUCIA E N G I N E E R I N G, I N C.
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JOSEPH M. LUCIA
STATE OF WASHINGTON
23314
REGISTERED
PROFESSIONAL ENGINEER
04-27-24

Number	Date	By	Description
32	04-27-24	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:

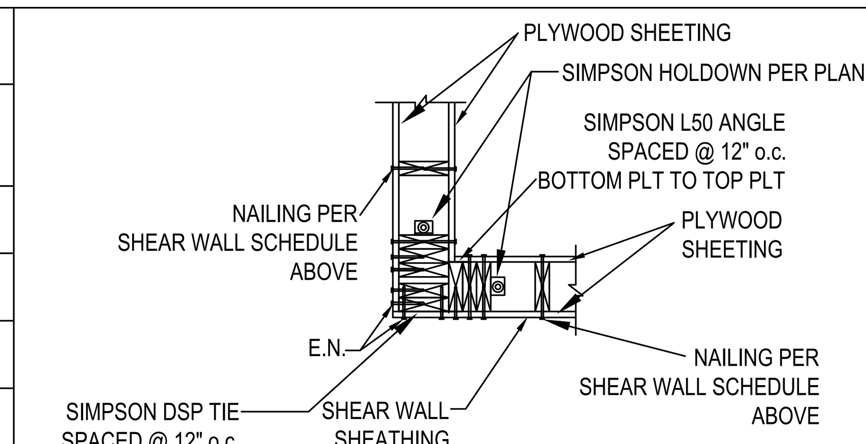
1. ROOF AND FLOOR DIAPHRAGMS ARE UNBLOCKED, U.L.N. AND NAILED ACCORDING TO THE FASTENING SCHEDULE OF IBC TABLE 2304.9.1.

HEADER END NAILING

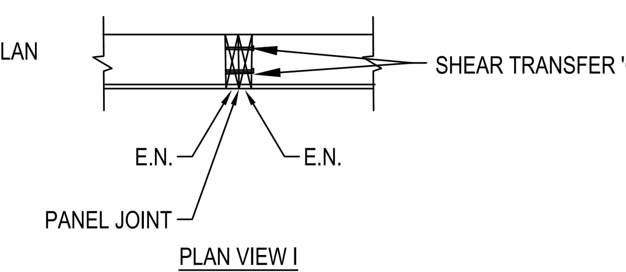
NOMINAL DEPTH	END ATTACHMENT
4	(4)16d
6	(6)16d
8	(8)16d
10	(10)16d
12	(12)16d
14	(14)16d
16	(16)16d
18	(18)16d

ROUGH WINDOW SILL

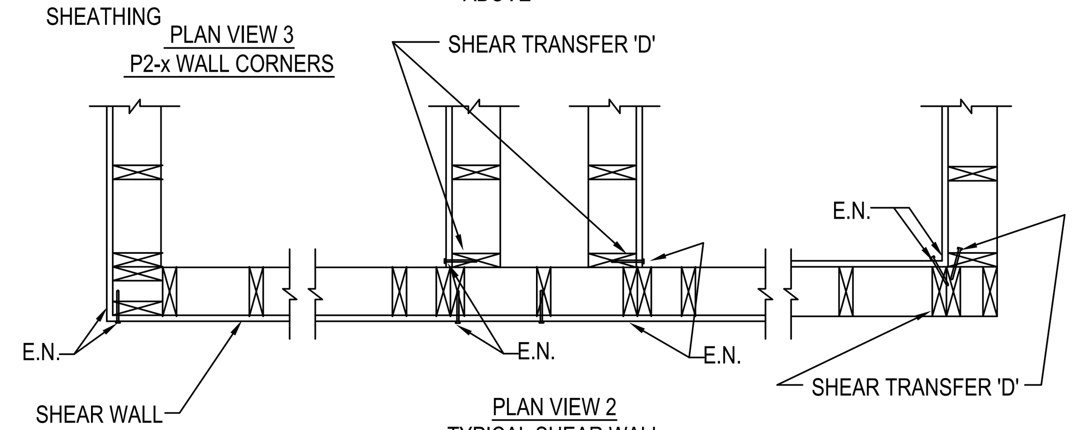
HORIZ ROUGH OPENING	NUMBER OF SILLS REQUIRED	END ATTACHMENT	REF.
0 TO 6'	1	(2)16d END NAIL	20/S6.1
> 6'	2	(2)16d END NAIL, +A35 EA END @ EA SILL	20/S6.1



PLAN VIEW 3
P2-x WALL CORNERS



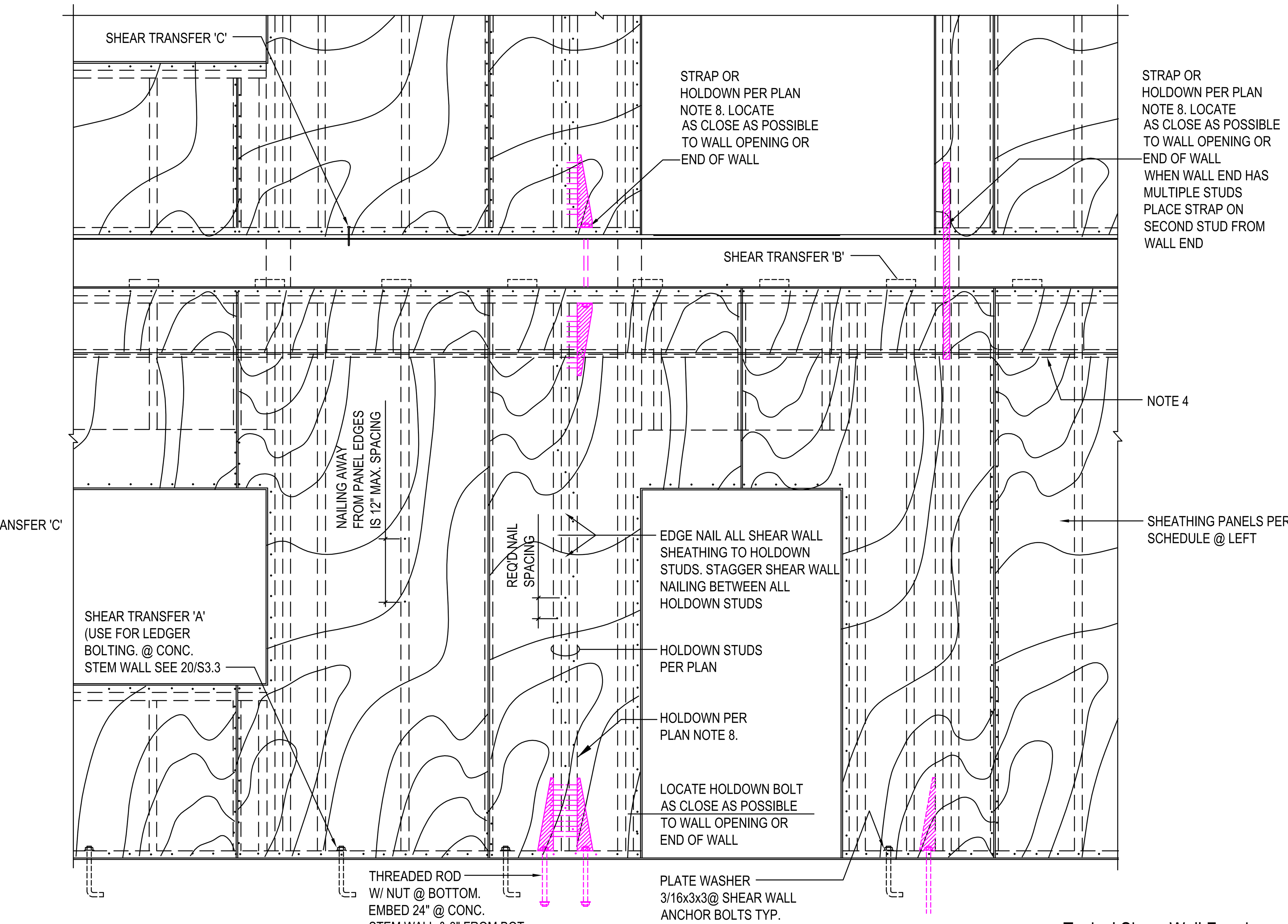
PLAN VIEW I
OPTIONAL (2)2x IN LIEU OF 3x



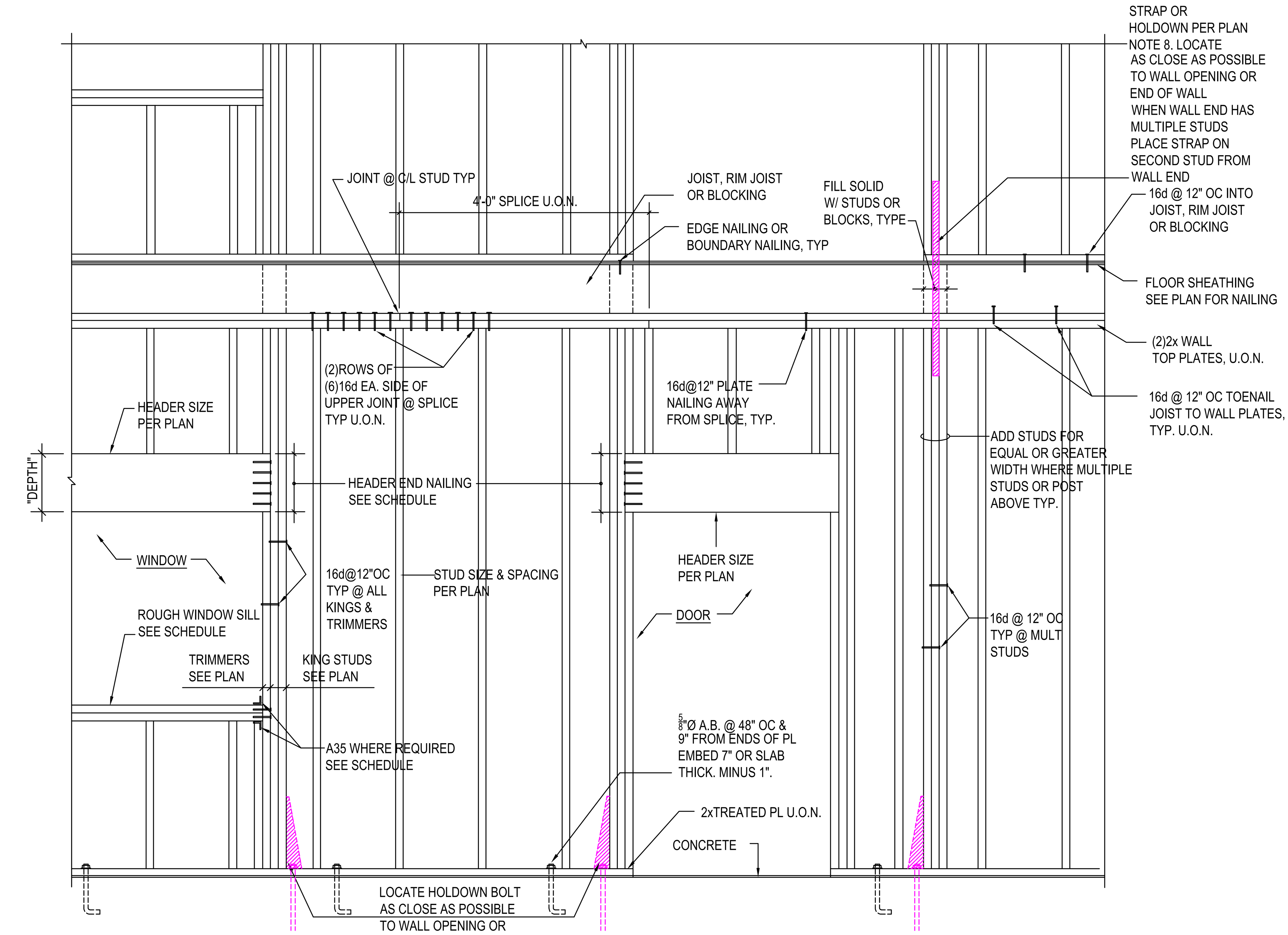
PLAN VIEW 2
TYPICAL SHEAR WALL INTERSECTIONS

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.

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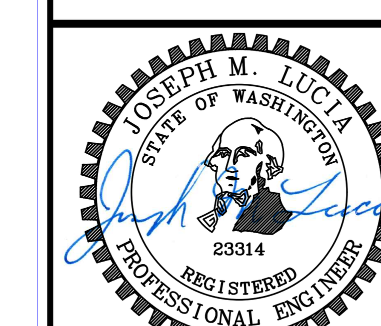

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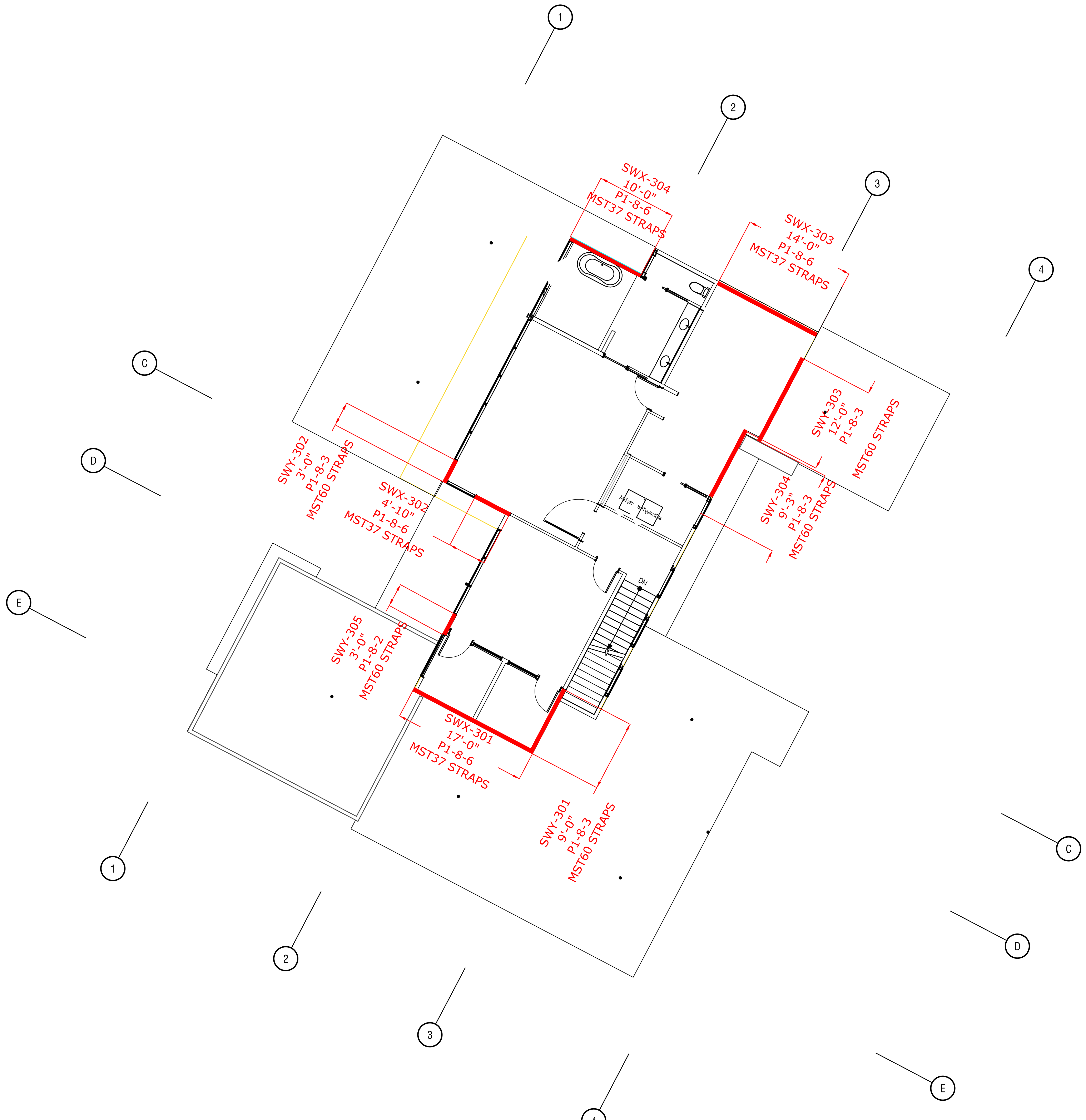
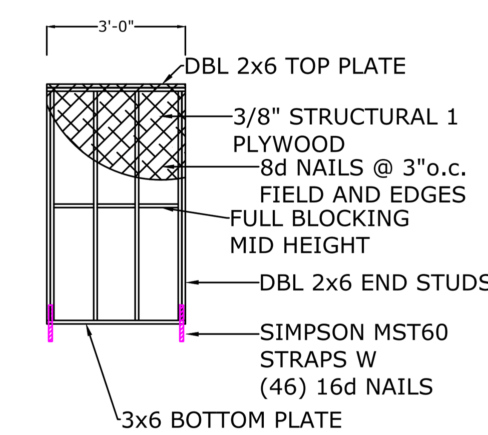
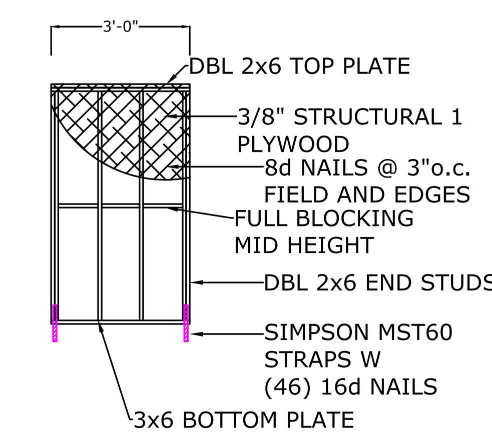
SHEET
S-6.0



SECOND FLOOR LEVEL - SHEAR WALLS



SWY-302
P1-8-6



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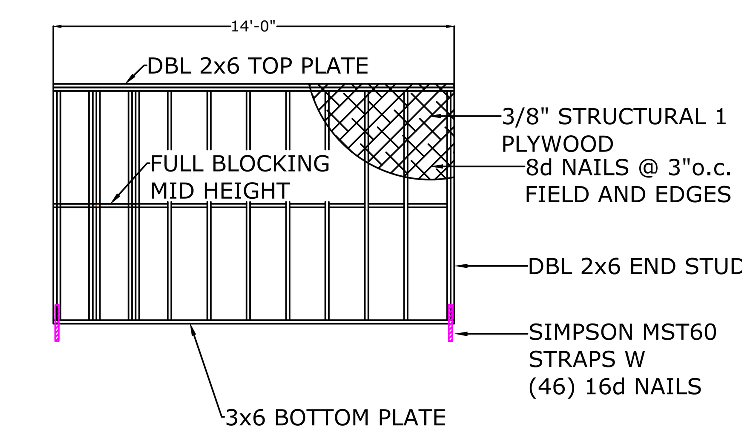
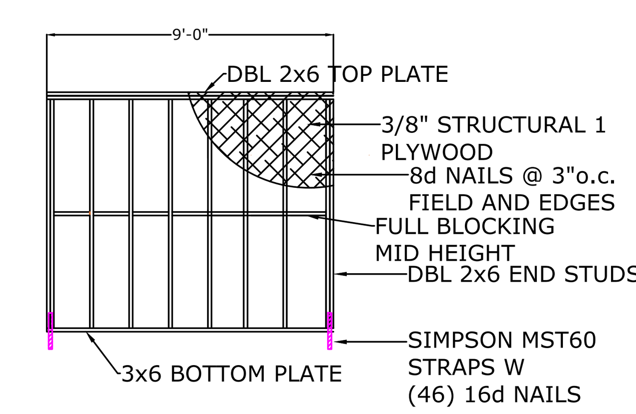


SWY-303
P1-8-6



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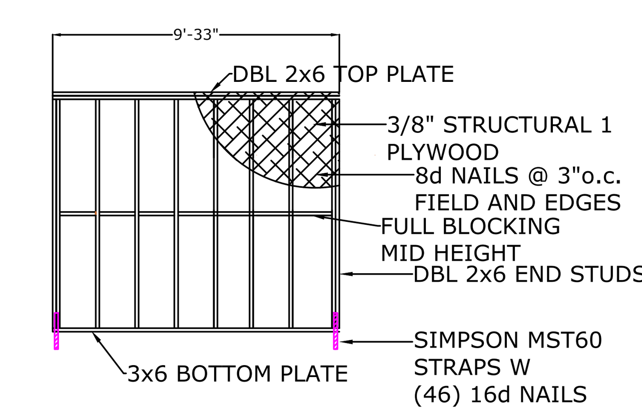

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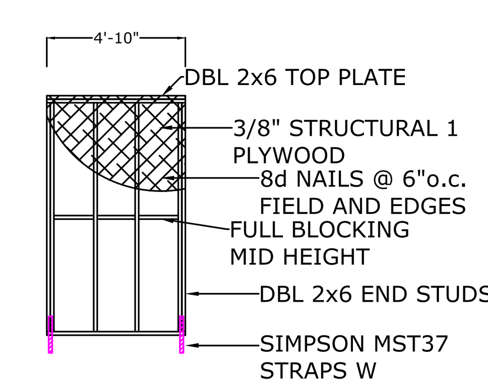

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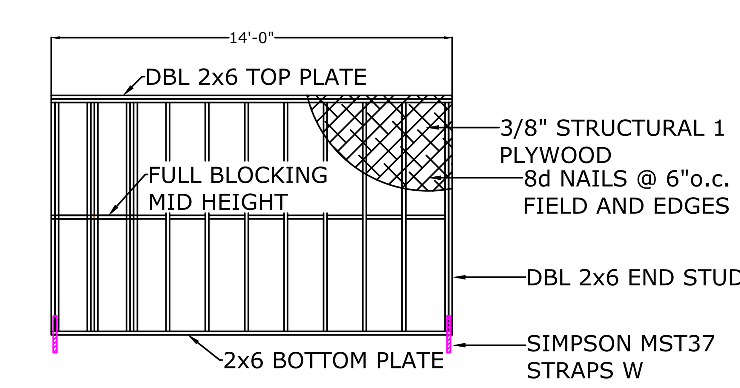
P1-8-6



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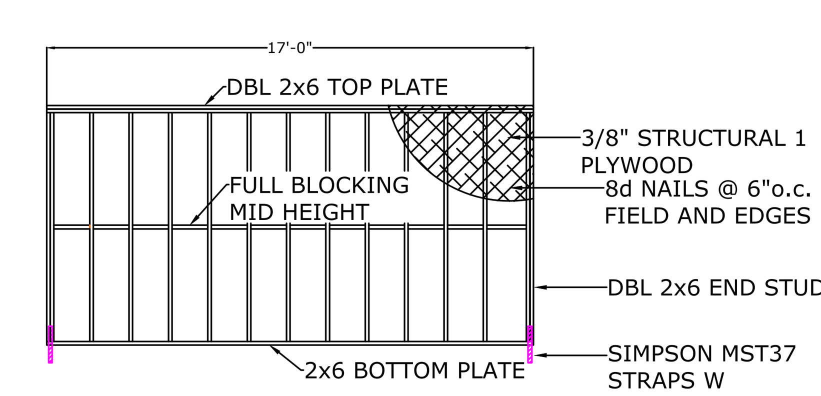

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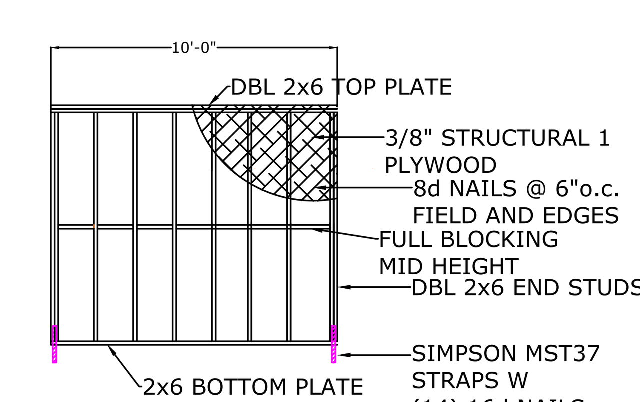

SWX-303
P1-8-6

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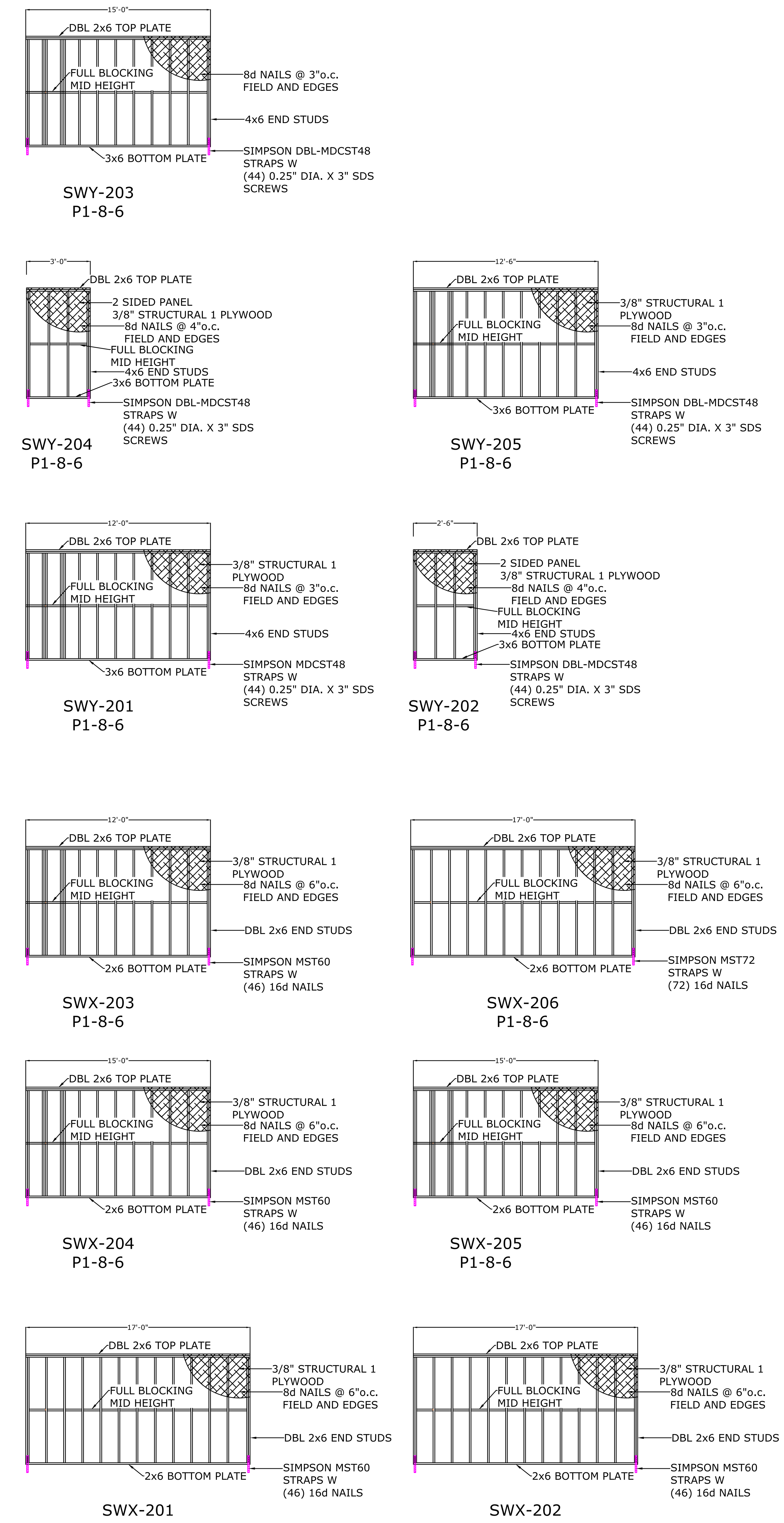
SHEET
S-7.0

FIRST FLOOR LEVEL - SHEAR WALLS



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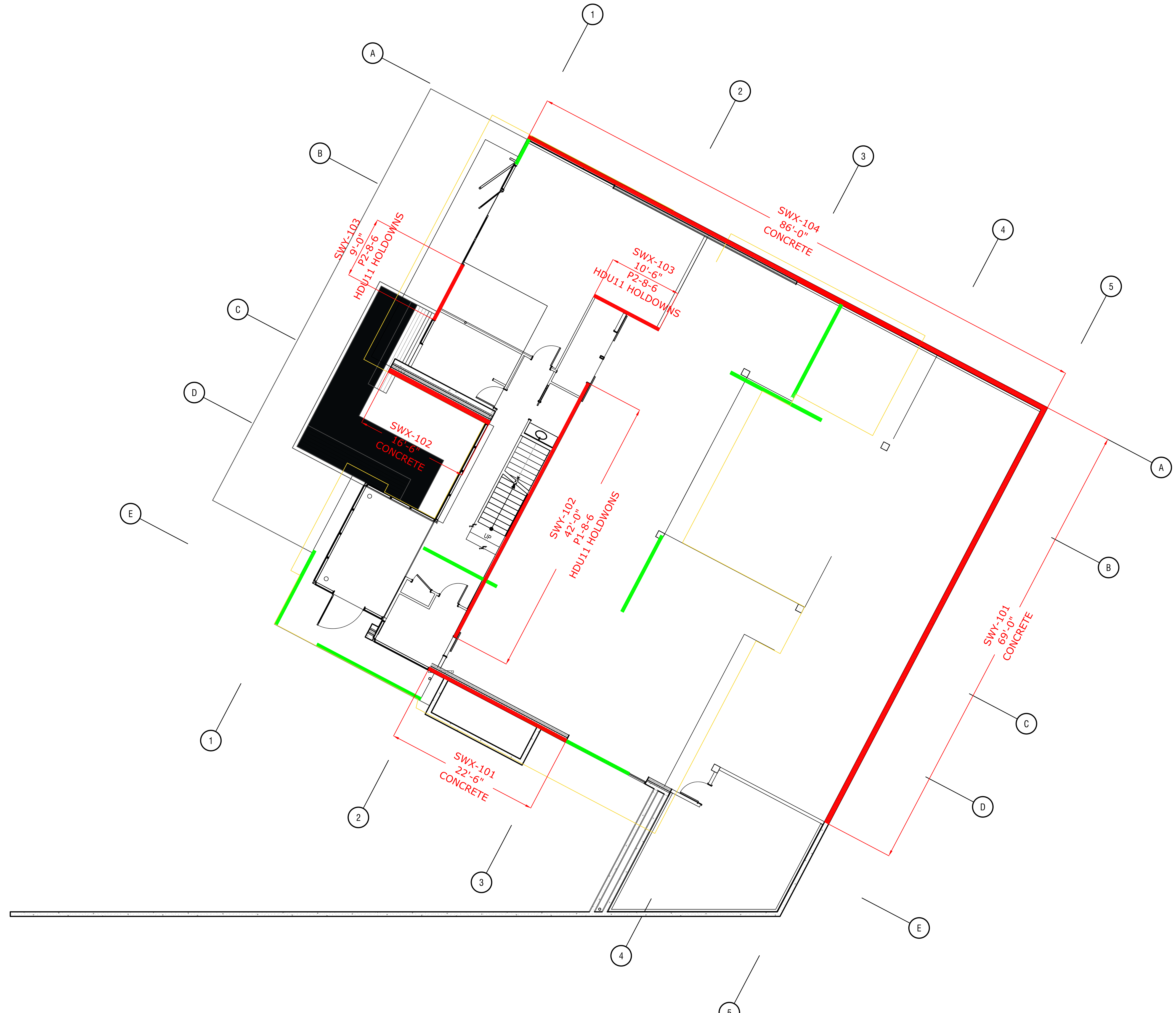
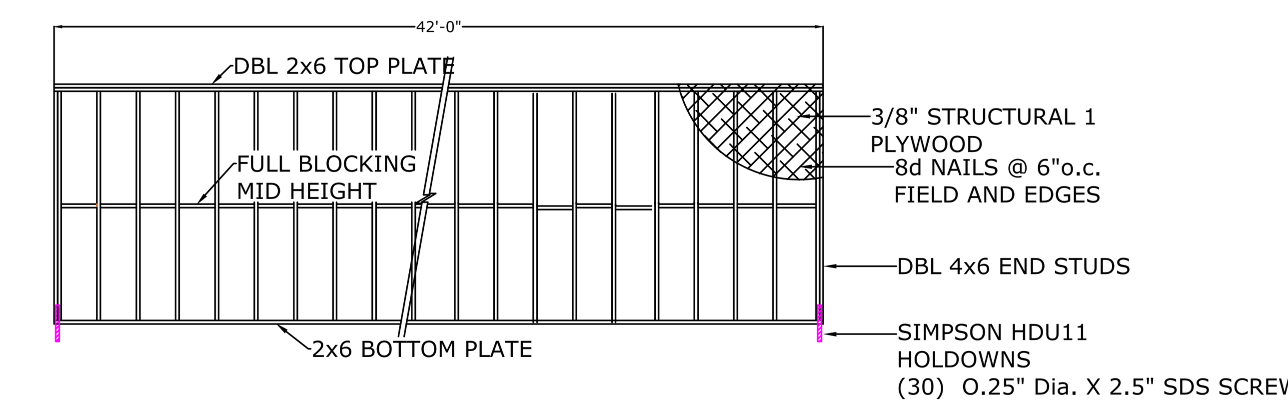
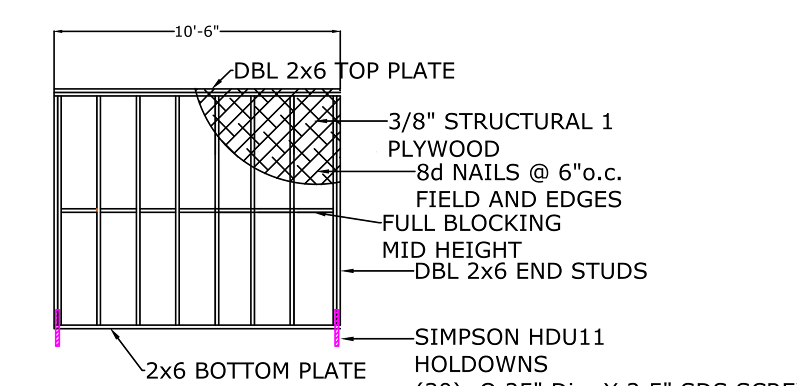


GARAGE-BASEMENT LEVEL - SHEAR WALLS



SWY-102
P1-8-6



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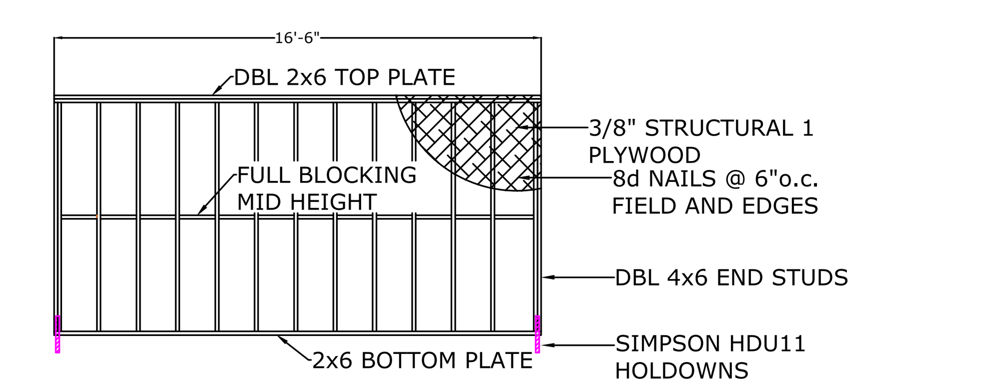
P1-8-6



SWX-102
P1-8-6

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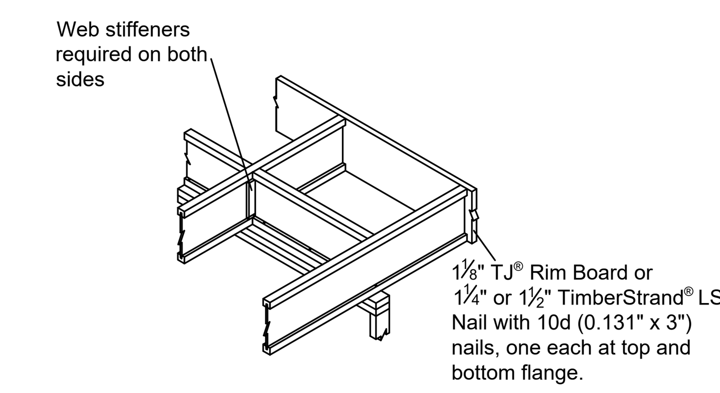
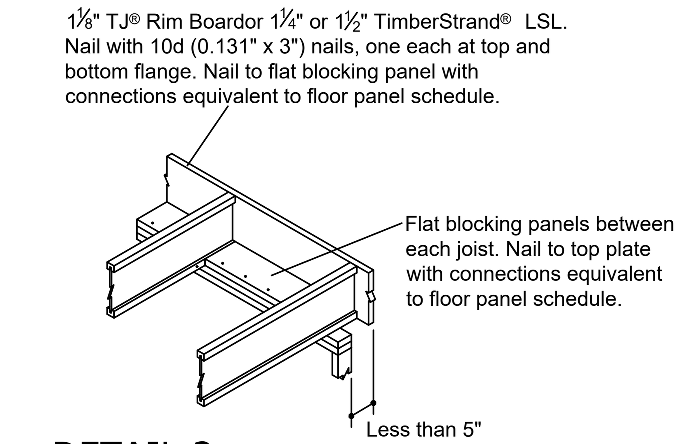
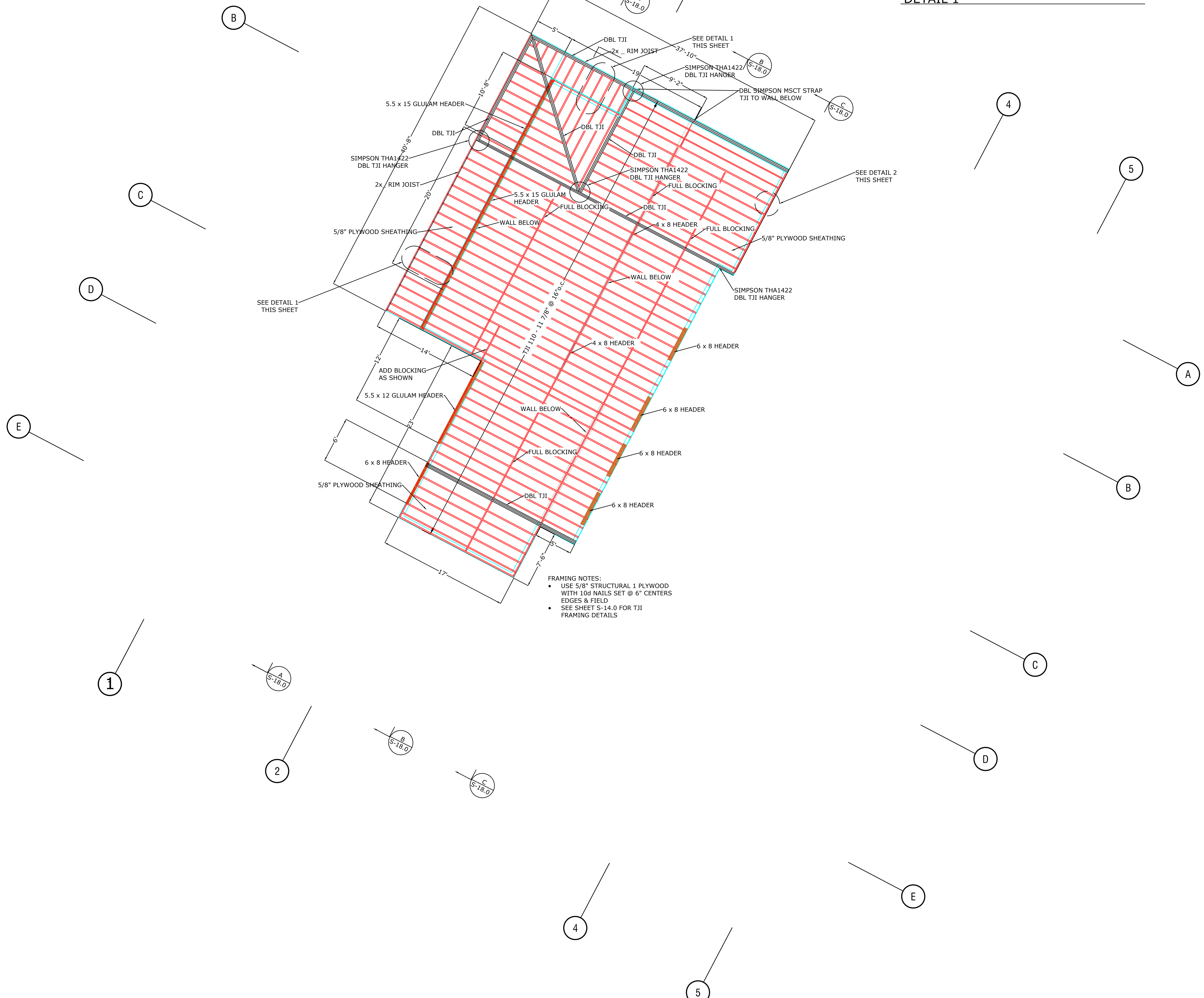


DETAIL 1



DETAIL 2



ROOF FRAMING

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SHEET
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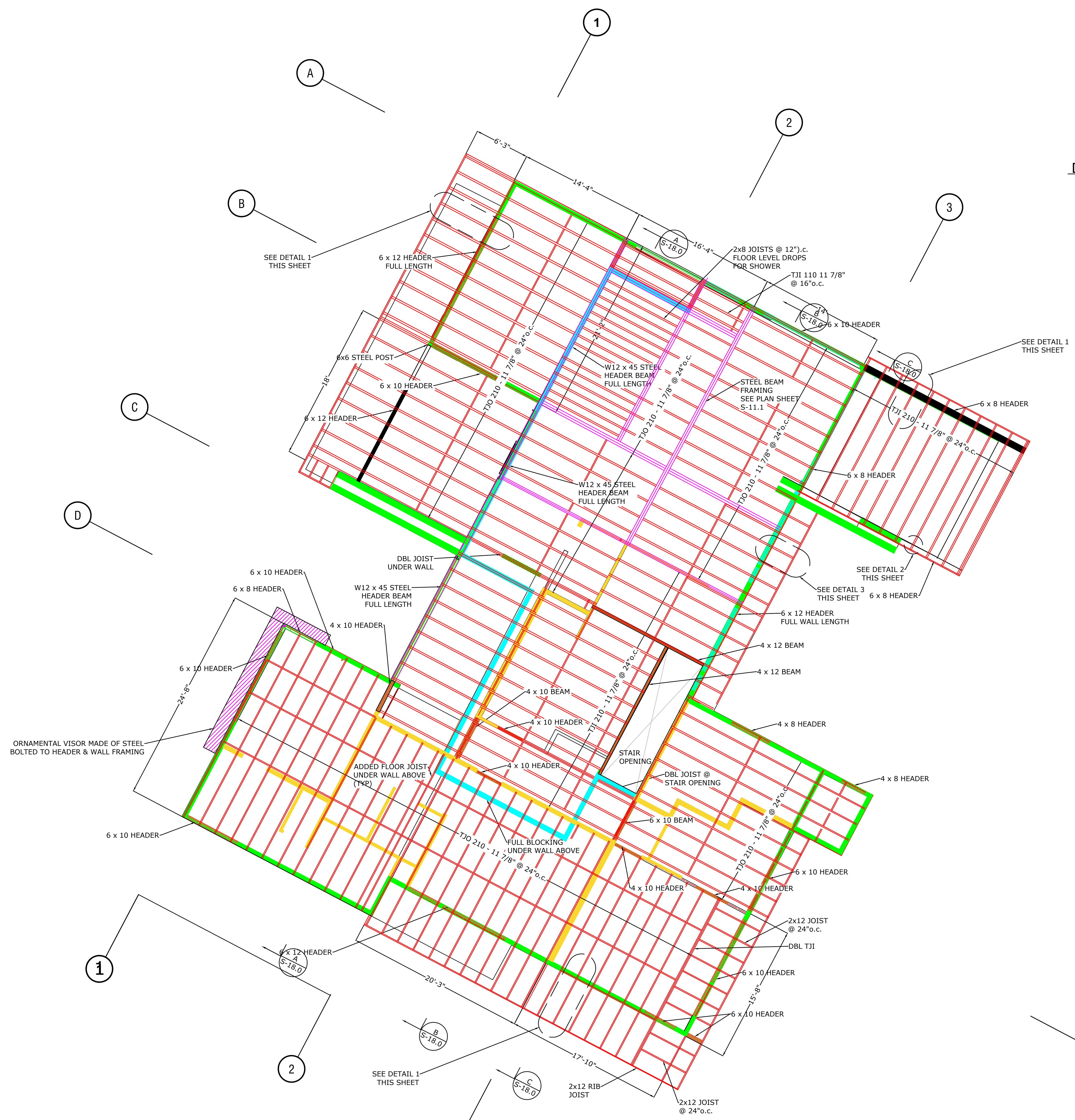
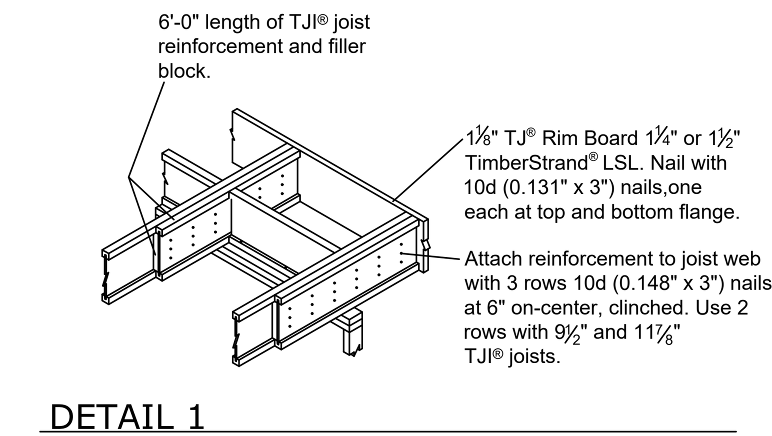
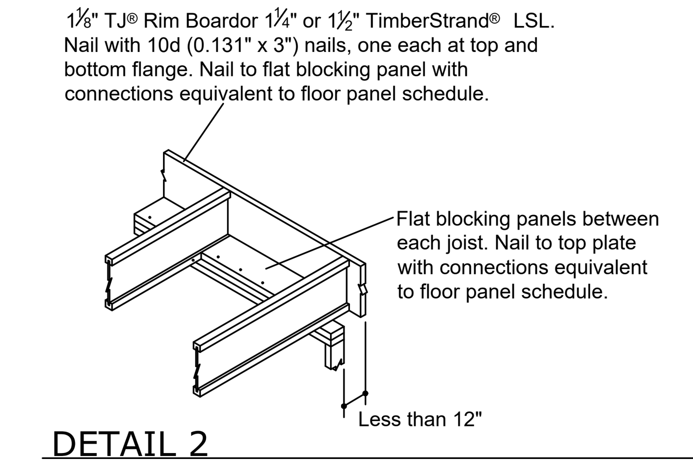


FRAMING NOTES:
- USE 1 1/4" T&G STRUCTURAL 1 PLYWOOD WITH 10d NAILS SET @ 6" CENTERS EDGES & FIELD
- SEE SHEET S-14.0 FOR TJI FRAMING DETAILS

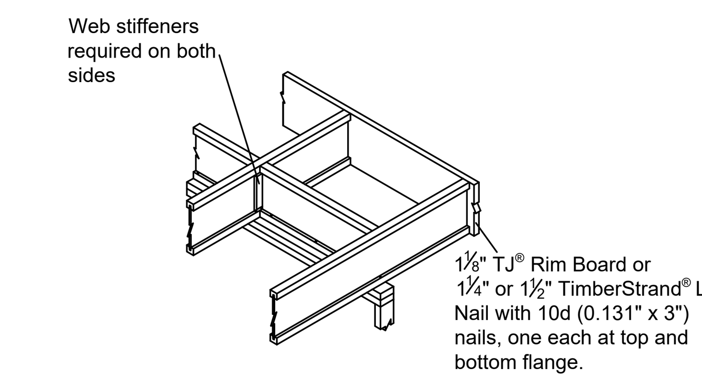
SECOND FLOOR - FLOOR FRAMING



DETAIL 1



DETAIL 2



DETAIL 3

LANZ RESIDENCE
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SHEET
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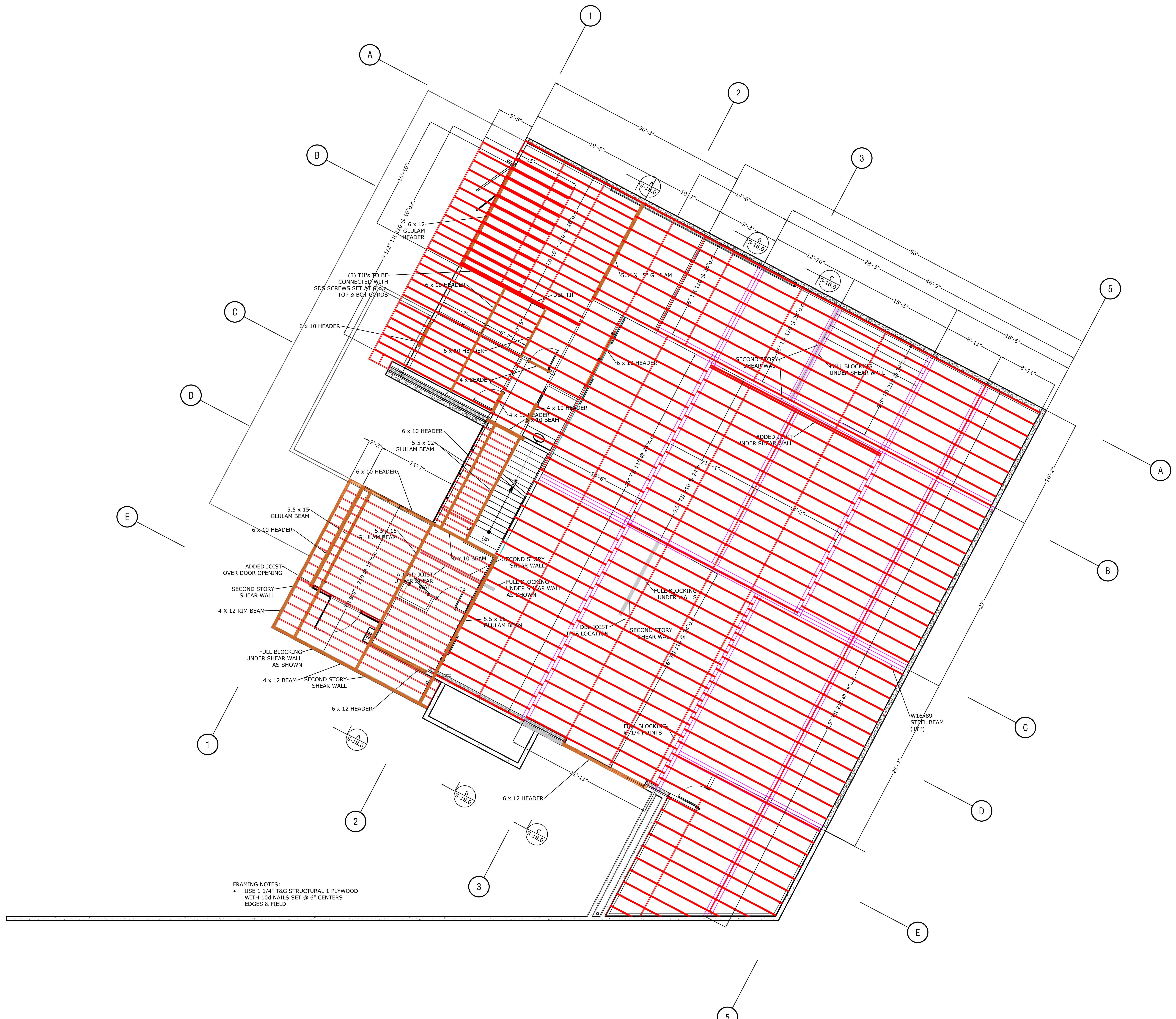

FRAMING NOTES:

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FIRST FLOOR - FLOOR FRAMING

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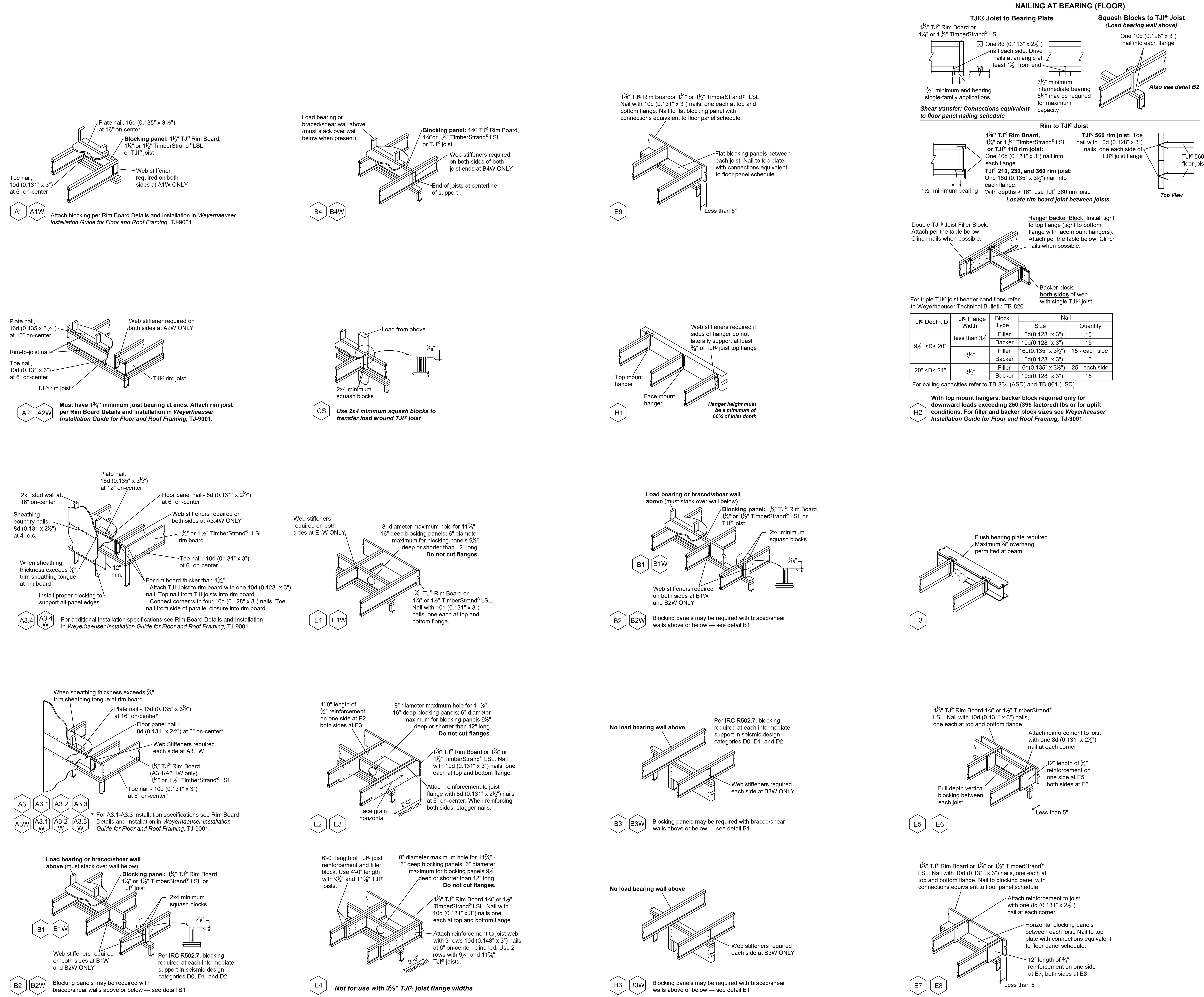

TJI® Depth, D	TJI® Flange Width	Block Type	Nail	
			Size	Quantity
9½" <D≤ 20"	less than 3½"	Filler	10d(0.128" x 3")	15
		Backer	10d(0.128" x 3")	15
	3½"	Filler	16d(0.135" x 3½")	15 - each side
		Backer	10d(0.128" x 3")	15
20" <D≤ 24"	3½"	Filler	16d(0.135" x 3½")	25 - each side
		Backer	10d(0.128" x 3")	15

For nailing capacities refer to TB-834 (ASD) and TB-861 (LSD)

H2 **With top mount hangers, backer block required only for downward loads exceeding 250 (395 factored) lbs or for uplift conditions. For filler and backer block sizes see *Weyerhaeuser Installation Guide for Floor and Roof Framing*, TJ-9001.**

TYPICAL INSTALLATION DETAILS - TJI® Joists

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SHEET
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Number	Date	By	Description
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SHEET S-14.0

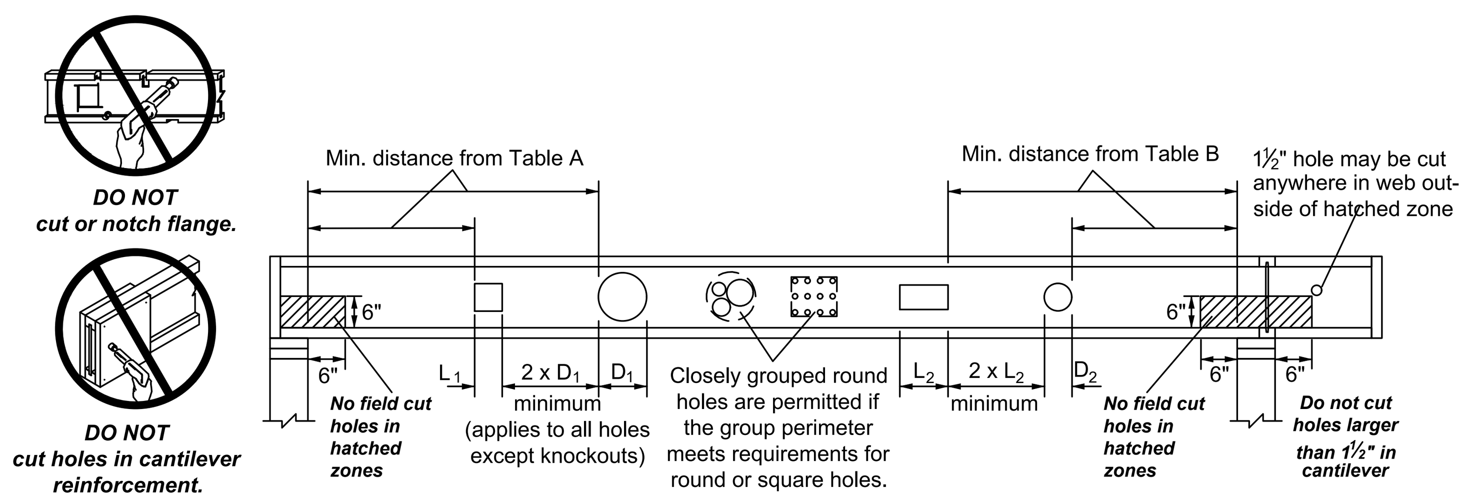

Table A - End Support

Minimum distance from edge of hole to inside face of nearest end support

JOIST DEPTH	TJI®	ROUND HOLE SIZE 2"	3"	4"	5"	6½"	7"	8⅞"	11"	13"	SQUARE OR RECTANGULAR HOLE SIZE 2"	3"	4"	5"	6½"	7"	8⅞"	11"	13"
9½"	110	1'-0"	1'-6"	2'-0"	3'-0"	5'-0"					1'-0"	1'-6"	2'-6"	3'-6"	4'-6"				
	210	1'-0"	1'-6"	2'-6"	3'-0"	5'-6"					1'-0"	2'-0"	2'-6"	4'-0"	5'-0"				
	230	1'-6"	2'-0"	2'-6"	3'-6"	5'-6"					1'-0"	2'-0"	3'-0"	4'-6"	5'-0"				
	360	1'-6"	2'-0"	3'-0"	4'-0"	6'-0"					1'-6"	2'-6"	3'-6"	5'-0"	5'-6"				
	560	1'-6"	2'-6"	3'-6"	5'-0"	7'-0"					2'-0"	3'-0"	4'-0"	5'-6"	6'-0"				
11⅞"	110	1'-0"	1'-0"	1'-6"	2'-0"	2'-6"	3'-0"	5'-6"			1'-0"	1'-6"	2'-0"	2'-6"	4'-6"	5'-0"	6'-0"		
	210	1'-0"	1'-6"	2'-0"	2'-0"	3'-0"	3'-6"	6'-0"			1'-0"	1'-6"	2'-6"	3'-0"	5'-0"	5'-6"	6'-6"		
	230	1'-0"	1'-6"	2'-0"	2'-6"	3'-0"	3'-6"	6'-6"			1'-0"	2'-0"	2'-6"	3'-6"	5'-6"	5'-6"	7'-0"		
	360	1'-6"	2'-0"	3'-0"	3'-6"	4'-6"	5'-0"	7'-0"			1'-6"	2'-6"	3'-6"	4'-6"	6'-6"	6'-6"	7'-6"		
	560	1'-6"	2'-6"	3'-0"	4'-0"	5'-6"	6'-0"	8'-0"			2'-6"	3'-6"	4'-6"	5'-6"	7'-0"	7'-6"	8'-0"		
14"	110	1'-0"	1'-0"	1'-0"	1'-0"	1'-6"	2'-0"	3'-0"	5'-6"		1'-0"	1'-0"	1'-6"	2'-0"	3'-6"	4'-0"	6'-0"	8'-0"	
	210	1'-0"	1'-0"	1'-0"	1'-6"	2'-0"	2'-6"	3'-6"	6'-0"		1'-0"	1'-0"	2'-0"	2'-6"	4'-0"	4'-6"	6'-6"	8'-6"	
	230	1'-0"	1'-0"	1'-0"	1'-6"	2'-6"	2'-6"	4'-0"	7'-0"		1'-0"	1'-0"	2'-0"	3'-0"	4'-0"	5'-0"	7'-0"	9'-0"	
	360	1'-0"	1'-0"	1'-6"	2'-6"	3'-6"	4'-0"	5'-6"	8'-0"		1'-0"	1'-6"	2'-6"	4'-0"	6'-0"	6'-6"	8'-0"	9'-6"	
	560	1'-0"	1'-0"	2'-0"	3'-0"	4'-6"	5'-0"	6'-6"	9'-0"		1'-6"	3'-0"	4'-0"	5'-0"	7'-0"	7'-6"	9'-0"	10'-0"	
16"	110	1'-0"	1'-0"	1'-0"	1'-0"	1'-0"	1'-0"	2'-0"	3'-0"	5'-0"	1'-0"	1'-0"	1'-0"	1'-6"	3'-0"	3'-0"	5'-6"	7'-6"	10'-0"
	210	1'-0"	1'-0"	1'-0"	1'-0"	1'-0"	1'-6"	2'-6"	3'-6"	6'-0"	1'-0"	1'-0"	1'-0"	2'-0"	3'-0"	3'-6"	6'-6"	8'-0"	11'-0"
	230	1'-0"	1'-0"	1'-0"	1'-0"	1'-6"	1'-6"	3'-0"	4'-0"	7'-0"	1'-0"	1'-0"	1'-0"	2'-0"	3'-6"	4'-0"	7'-0"	9'-0"	11'-0"
	360	1'-0"	1'-0"	1'-0"	1'-0"	2'-6"	2'-6"	4'-6"	6'-6"	9'-0"	1'-0"	1'-0"	1'-6"	3'-0"	5'-0"	5'-6"	9'-0"	10'-0"	11'-6"
	560	1'-0"	1'-0"	1'-0"	1'-0"	2'-6"	3'-0"	5'-0"	7'-6"	10'-0"	1'-0"	2'-0"	3'-0"	4'-6"	6'-6"	7'-0"	10'-0"	11'-0"	12'-0"

JOIST DEPTH	TJI®	4"	6"	7"	8"	10"	12"	14¾"	16¾"		4"	6"	7"	8"	10"	12"	14¾"	16¾"	
18"	360	1'-0"	1'-0"	1'-0"	2'-0"	4'-0"	5'-6"	9'-6"			1'-0"	3'-0"	4'-6"	6'-0"	10'-0"	11'-0"	13'-6"		
	560	1'-0"	1'-0"	1'-0"	2'-0"	4'-6"	7'-0"	10'-6"			2'-0"	5'-0"	6'-6"	8'-0"	11'-0"	12'-0"	14'-0"		
20"	360	1'-0"	1'-0"	1'-0"	1'-0"	2'-0"	4'-0"	7'-0"	10'-0"		1'-0"	1'-6"	3'-0"	4'-6"	8'-0"	11'-6"	13'-6"	15'-6"	
	560	1'-0"	1'-0"	1'-0"	1'-0"	2'-0"	4'-6"	8'-6"	11'-0"		1'-0"	3'-6"	5'-0"	7'-0"	10'-6"	13'-0'	14'-6"	15'-6"	

Table B - Intermediate or Cantilever Support

Minimum distance from edge of hole to inside face of nearest intermediate or cantilever support

JOIST DEPTH	TJI®	ROUND HOLE SIZE 2"	3"	4"	5"	6½"	7"	8⅞"	11"	13"	SQUARE OR RECTANGULAR HOLE SIZE 2"	3"	4"	5"	6½"	7"	8⅞"	11"	13"
9½"	110	2'-0"	2'-6"	3'-6"	4'-6"	7'-6"					1'-6"	2'-6"	3'-6"	5'-6"	6'-6"				
	210	2'-0"	2'-6"	3'-6"	5'-0"	8'-0"					2'-0"	3'-0"	4'-0"	6'-6"	7'-6"				
	230	2'-6"	3'-0"	4'-0"	5'-6"	8'-6"					2'-0"	3'-6"	4'-6"	6'-6"	7'-6"				
	360	3'-0"	4'-0"	5'-6"	6'-6"	9'-0"					3'-0"	4'-6"	5'-6"	7'-6"	8'-0"				
	560	3'-6"	5'-0"	6'-0"	7'-6"	10'-0"					4'-0"	5'-6"	6'-6"	8'-0"	9'-0"				
11⅞"	110	1'-0"	1'-0"	1'-6"	2'-6"	4'-0"	4'-6"	8'-6"			1'-0"	1'-6"	2'-6"	4'-0"	7'-0"	7'-0"	9'-6"		
	210	1'-0"	1'-0"	2'-0"	3'-0"	4'-6"	5'-0"	9'-0"			1'-0"	2'-0"	3'-0"	4'-6"	8'-0"	8'-0"	10'-0"		
	230	1'-0"	2'-0"	2'-6"	3'-6"	5'-0"	5'-6"	10'-0"			1'-0"	2'-6"	3'-6"	5'-0"	8'-6"	9'-0"	10'-6"		
	360	2'-0"	3'-0"	4'-0"	5'-6"	7'-0"	7'-6"	11'-0"			2'-0"	3'-6"	5'-0"	7'-0"	9'-6"	9'-6"	11'-0"		
	560	1'-6"	3'-0"	4'-6"	5'-6"	8'-0"	8'-6"	12'-0"			3'-0"	4'-6"	6'-0"	8'-0"	10'-6"	11'-0"	12'-0"		
14"	110	1'-0"	1'-0"	1'-0"	1'-0"	2'-0"	2'-6"	4'-6"	8'-6"		1'-0"	1'-0"	1'-0"	2'-6"	5'-0"	6'-0"	9'-0"	12'-0"	
	210	1'-0"	1'-0"	1'-0"	1'-0"	2'-6"	3'-0"	5'-6"	9'-6"		1'-0"	1'-0"	2'-0"	3'-6"	6'-0"	7'-0"	10'-0"	13'-0"	
	230	1'-0"	1'-0"	1'-0"	2'-0"	3'-6"	4'-0"	6'-0"	10'-6"		1'-0"	1'-0"	2'-6"	4'-0"	6'-6"	7'-6"	11'-0"	13'-6"	
	360	1'-0"	1'-0"	2'-0"	3'-6"	5'-6"	6'-0"	8'-6"	12'-6"		1'-0"	2'-0"	4'-0"	5'-6"	9'-0"	10'-0"	12'-0"	14'-0"	
	560	1'-0"	1'-0"	1'-6"	3'-6"	5'-6"	6'-6"	9'-6"	13'-6"		1'-0"	3'-0"	5'-0"	7'-0"	10'-0"	11'-0"	13'-6"	15'-0"	
16"	110	1'-0"	1'-0"	1'-0"	1'-0"	1'-0"	1'-0"	2'-6"	5'-0"	8'-6"	1'-0"	1'-0"	1'-0"	1'-0"	3'-6"	4-6"	8'-6"	11'-6"	15'-0"
	210	1'-0"	1'-0"	1'-0"	1'-0"	1'-0"	1'-0"	3'-6"	6'-0"	10'-0"	1'-0"	1'-0"	1'-0"	1'-6"	4'-6"	5'-6"	10'-0"	12'-6"	16'-0"
	230	1'-0"	1'-0"	1'-0"	1'-0"	1'-6"	2'-0"	4'-0"	6'-6"	11'-0"	1'-0"	1'-0"	1'-0"	2'-6"	5'-0"	6'-0"	10'-6"	13'-6"	16'-6"
	360	1'-0"	1'-0"	1'-0"	1'-0"	3'-0"	4'-0"	6'-6"	10'-0"	13'-6"	1'-0"	1'-0"	2'-0"	4'-0"	7'-6"	8'-6"	13'-0"	14'-6"	17'-0"
	560	1'-0"	1'-0"	1'-0"	1'-0"	2'-6"	3'-6"	7'-0"	11'-0"	15'-0"	1'-0"	1'-0"	3'-6"	5'-6"	9'-0"	10'-0"	14'-6"	16'-0"	18'-0"

JOIST DEPTH	TJI®	4"	6"	7"	8"	10"	12"	14¾"	16¾"		4"	6"	7"	8"	10"	12"	14¾"	16¾"	
18"	360	1'-0"	1'-0"	1'-6"	3'-0"	6'-0"	9'-0"	14'-6"			1'-0"	4'-0"	6'-6"	9'-0"	14'-6"	16'-6"	19'-0"		
	560	1'-0"	1'-0"	1'-0"	2'-0"	6'-0"	10'-0"	15'-6"			1'-0"	6'-0"	8'-6"	11'-6"	16'-6"	18'-0"	19'-6"		
20"	360	1'-0"	1'-0"	1'-0"	1'-0"	3'-0"	6'-0"	11'-0"	15'-0"		1'-0"	1'-6"	4'-0"	7'-0"	12'-6"	16'-6"	19'-0"	20'-6"	
	560	1'-0"	1'-0"	1'-0"	1'-0"	1'-6"	5'-6"	11'-6"	15'-6"		1'-0"	3'-0"	6'-0"	8'-6"	14'-0"	17'-6'	19'-0"	20'-6"	

Rectangular holes based on measurement of longest side.

- Leave ⅛" of web (minimum) at top and bottom of hole. **DO NOT cut joist flanges.**
- Tables are based on uniform load tables in current design literature.
- For simple span (5' minimum), uniformly loaded joists used in residential applications, one maximum size round hole may be located at the center of the joist span **provided that no other holes occur in the joist.**

ALLOWABLE HOLES - TJI® Joists

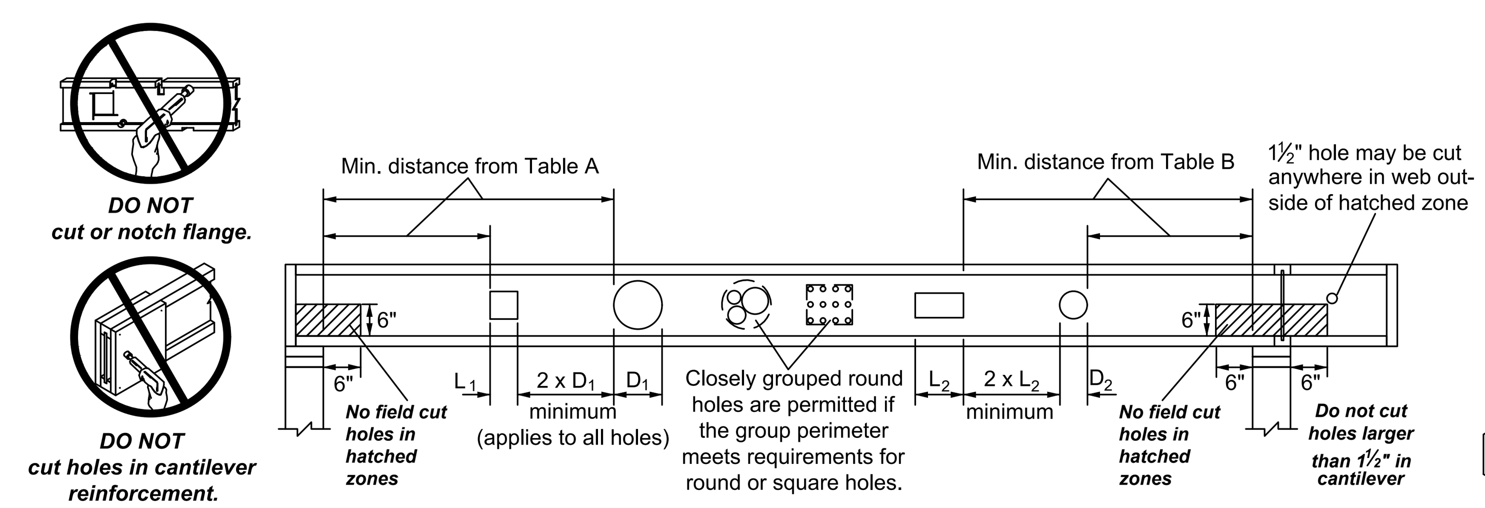

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Table A - End Support

Minimum distance from edge of hole to inside face of nearest end support

JOIST DEPTH	TJI®	ROUND HOLE SIZE 2"	3"	4"	5"	6¼"	7"	8⅝"	10¾"	12¾"	SQUARE OR RECTANGULAR HOLE SIZE 2"	3"	4"	5"	6¼"	7"	8⅝"	10¾"	12¾"
9½"	s31	1'-0"	2'-0"	2'-6"	3'-6"	5'-6"					1'-0"	1'-6"	2'-6"	4'-0"	4'-6"				
	s33	1'-6"	2'-6"	3'-0"	4'-0"	6'-0"					1'-0"	2'-0"	3'-0"	4'-6"	5'-0"				
	s47	1'-0"	1'-0"	2'-6"	4'-0"	6'-0"					1'-6"	2'-6"	3'-6"	5'-0"	5'-6"				
11⅞"	s31	1'-0"	1'-6"	1'-6"	2'-0"	3'-0"	3'-6"	6'-0"			1'-0"	1'-6"	2'-6"	3'-0"	4'-6"	5'-0"	6'-0"		
	s33	1'-0"	1'-6"	2'-6"	3'-0"	3'-6"	4'-6"	7'-0"			1'-0"	2'-0"	3'-0"	3'-6"	5'-6"	6'-0"	7'-0"		
	s47	1'-0"	1'-0"	2'-0"	3'-0"	4'-0"	4'-6"	7'-0"			2'-0"	3'-0"	3'-6"	4'-6"	6'-6"	6'-6"	7'-6"		
14"	s31	1'-0"	1'-0"	1'-0"	1'-6"	2'-0"	2'-6"	3'-0"	6'-0"		1'-0"	1'-6"	2'-0"	2'-6"	3'-6"	4'-6"	6'-0"	7'-6"	
	s33	1'-0"	1'-0"	1'-6"	2'-0"	2'-6"	3'-0"	4'-6"	8'-0"		1'-0"	1'-6"	2'-6"	3'-0"	4'-6"	5'-6"	7'-0"	8'-6"	
	s47	1'-0"	1'-0"	1'-0"	1'-6"	2'-6"	3'-6"	5'-0"	8'-6"		1'-0"	2'-0"	3'-0"	4'-0"	5'-6"	6'-6"	8'-0"	9'-6"	
16"	s31	1'-0"	1'-0"	1'-0"	1'-0"	1'-6"	1'-6"	2'-6"	3'-6"	6'-0"	1'-0"	1'-0"	1'-6"	2'-0"	3'-0"	3'-6"	6'-0"	7'-0"	9'-6"
	s33	1'-0"	1'-0"	1'-0"	1'-0"	1'-6"	2'-0"	3'-6"	5'-0"	8'-0"	1'-0"	1'-0"	1'-6"	2'-6"	4'-0"	4'-6"	7'-0"	9'-0"	10'-6"
	s47	1'-0"	1'-0"	1'-0"	1'-0"	1'-6"	2'-0"	3'-6"	5'-6"	9'-0"	1'-0"	1'-0"	2'-6"	3'-6"	4'-6"	5'-6"	8'-6"	10'-0"	11'-0"

JOIST DEPTH	TJI®	4"	5"	6¼"	7"	8⅝"	10¾"	12¾"	14¾"	16¾"	4"	5"	6¼"	7"	8⅝"	10¾"	12¾"	14¾"	16¾"
18"	s47	1'-0"	1'-0"	1'-0"	1'-0"	2'-0"	4'-0"	6'-0"	9'-6"		1'-0"	2'-6"	4'-0"	5'-0"	7-0"	10'-6"	12'-0"	13'-6"	
20"	s47	1'-0"	1'-0"	1'-0"	1'-0"	1'-6"	2'-6"	4'-6"	6'-6"	10'-0"	1'-0"	1'-0"	2'-6"	3'-6"	6'-0"	10'-0"	11'-6"	13'-0"	14'-6"

Table B - Intermediate or Cantilever Support

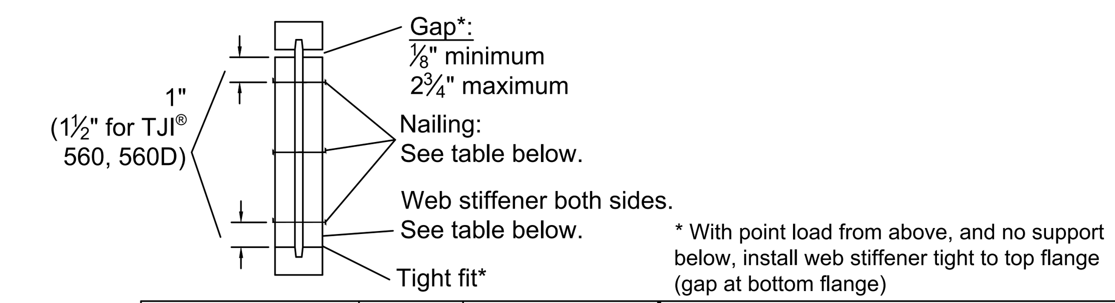
Minimum distance from edge of hole to inside face of nearest intermediate or cantilever support

JOIST DEPTH	TJI®	ROUND HOLE SIZE 2"	3"	4"	5"	6¼"	7"	8⅝"	10¾"	12¾"	SQUARE OR RECTANGULAR HOLE SIZE 2"	3"	4"	5"	6¼"	7"	8⅝"	10¾"	12¾"
9½"	s31	2'-0"	3'-0"	4'-0"	5'-0"	8'-6"					2'-0"	3'-0"	4'-0"	5'-6"	6'-6"				
	s33	2'-6"	3'-6"	5'-0"	6'-6"	9'-0"					2'-0"	3'-6"	4'-6"	6'-6"	7'-6"				
	s47	1'-6"	3'-0"	4'-6"	6'-0"	8'-6"					3'-0"	4'-6"	5'-6"	7'-6"	8'-0"				
11⅞"	s31	1'-6"	2'-0"	2'-6"	3'-6"	4'-6"	5'-0"	9'-0"			1'-6"	2'-6"	3'-6"	4'-6"	7'-0"	7'-6"	9'-0"		
	s33	2'-0"	3'-0"	3'-6"	4'-6"	5'-6"	7'-0"	10'-6"			2'-0"	3'-0"	4'-0"	5'-6"	8'-6"	9'-0"	10'-0"		
	s47	1'-0"	1'-0"	2'-0"	3'-6"	5'-6"	7'-0"	11'-0"			2'-0"	3'-6"	5'-0"	6'-6"	9'-6"	10'-0"	11'-0"		
14"	s31	1'-0"	1'-0"	1'-6"	2'-0"	3'-0"	3'-6"	5'-0"	9'-0"		1'-0"	1'-6"	2'-6"	4'-0"	5'-6"	6'-6"	9'-0"	11'-6"	
	s33	1'-0"	1'-0"	2'-0"	3'-0"	4'-0"	5'-0"	6'-6"	12'-0"		1'-0"	2'-0"	3'-6"	4'-6"	6'-6"	8'-0"	11'-0"	13'-0"	
	s47	1'-0"	1'-0"	1'-0"	2'-0"	4'-0"	5'-0"	7'-6"	12'-6"		1'-0"	2'-6"	4'-0"	6'-0"	8'-0"	10'-0"	12'-0"	13'-6"	
16"	s31	1'-0"	1'-0"	1'-0"	1'-0"	2'-0"	2'-6"	3'-6"	5'-6"	9'-6"	1'-0"	1'-0"	1'-6"	3'-0"	4'-6"	5'-6"	9'-0"	11'-0"	14'-0"
	s33	1'-0"	1'-0"	1'-0"	1'-0"	2'-6"	3'-6"	5'-0"	7'-6"	12'-6"	1'-0"	1'-0"	2'-0"	3'-6"	5'-6"	7'-0"	11'-0"	13'-6"	15'-6"
	s47	1'-0"	1'-0"	1'-0"	1'-0"	2'-0"	3'-0"	5'-6"	9'-0"	14'-0"	1'-0"	1'-6"	3'-0"	5'-0"	7'-0"	8'-6"	13'-0"	15'-0"	16'-6"

JOIST DEPTH	TJI®	4"	5"	6¼"	7"	8⅝"	10¾"	12¾"	14¾"	16¾"	4"	5"	6¼"	7"	8⅝"	10¾"	12¾"	14¾"	16¾"
18"	s47	1'-0"	1'-0"	1'-0"	1'-0"	3'-0"	6'-6"	9'-6"	14'-6"		1'-6"	3'-6"	6'-0"	7'-6"	11'-0"	15'-6"	17'-0"	18'-6"	
20"	s47	1'-0"	1'-0"	1'-0"	1'-0"	1'-6"	3'-6"	7'-0"	10'-6"	15'-0"	1'-0"	1'-0"	4'-0"	5'-6"	9'-0"	15'-0"	16'-6"	18'-0"	19'-6"

Rectangular holes based on measurement of longest side.

- Leave ⅛" of web (minimum) at top and bottom of hole. **DO NOT cut joist flanges.**
- Tables are based on uniform load tables in current design literature.
- For simple span (5' minimum), uniformly loaded joists used in residential applications, one maximum size round hole may be located at the center of the joist span **provided that no other holes occur in the joist.**



* With point load from above, and no support below, install web stiffener tight to top flange (gap at bottom flange)

TJI® Joist Series	Depth (in.)	Minimum Web Stiffener Size	Nailing Requirements: Type	Number Nails: End	Intermediate
110 210 230 & 360	All All All	⅝" x 2$\frac{5}{16}$"(1) ¾" x 2$\frac{5}{16}$"(1) ⅞" x 2$\frac{5}{16}$"(1)	8d (0.113" x 2½")	3	3
560	All	2x4 (2)	16d (0.135" x 3½")		
560D	18" 20" 22"(3) 24"(3)	2x4 (2)	16d (0.135" x 3½")	4 5 6 6	4 5 11 13

(1) PS1 or PS2 sheathing, face grain vertical
(2) Construction grade or better
(3) Web stiffeners are always required for 22" and 24" TJI® 560D Joists

WEB STIFFENER ATTACHMENT

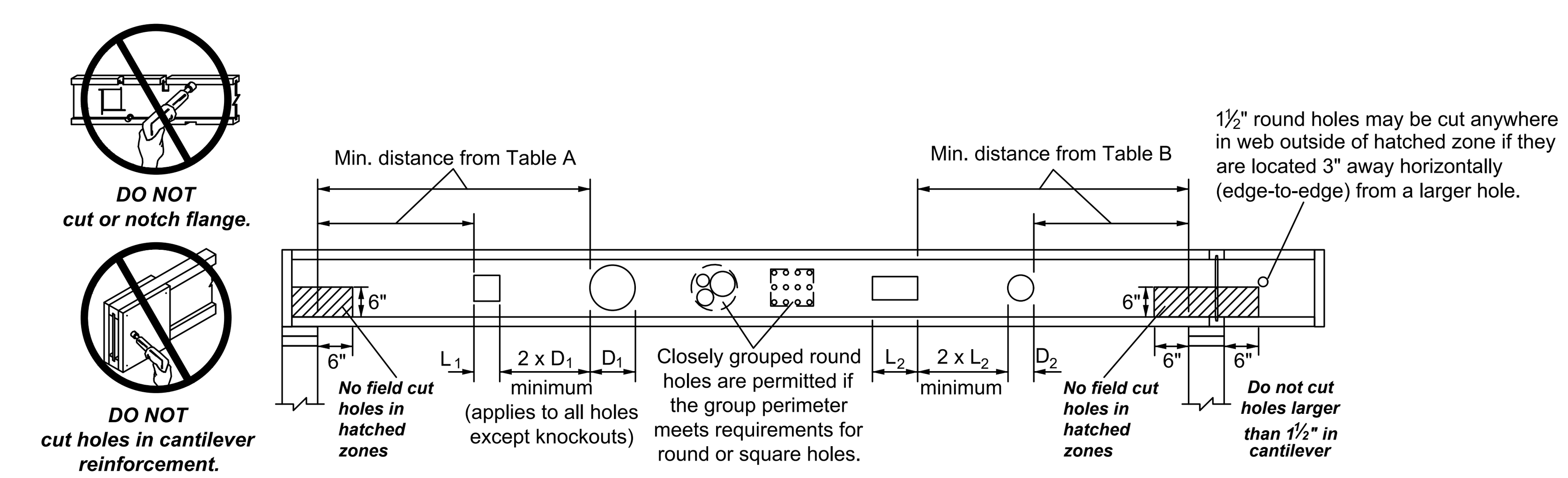


Table A - End Support

Minimum distance from edge of hole to inside face of nearest end support

JOIST DEPTH	TJI®	ROUND HOLE SIZE 4"	6"	7"	8"	10"	12"	14¾"	16¾"	18¾"	20"	SQUARE OR RECTANGULAR HOLE SIZE 4"	6"	7"	8"	10"	12"	14¾"	16¾"	18¾"	20"
18"	560D	1'-0"	1'-6"	2'-6"	3'-6"	5'-6"	7'-6"	11'-0"				3'-0"	5'-6"	6'-6"	8'-0"	10'-6"	11'-6"	13'-6"			
20"	560D	1'-0"	1'-0"	1'-6"	2'-6"	4'-6"	6'-0"	9'-0"	11'-6"			2'-6"	5'-0"	6'-0"	7'-0"	10'-0"	12'-6"	14'-0"	15'-0"		
22"	560D	1'-0"	1'-0"	1'-0"	1'-0"	3'-6"	5'-0"	7'-0"	9'-6"	12'-6"		1'-0"	3'-6"	5'-0"	6'-6"	14'-6"	15'-0"	16'-0"	16'-6"	17'-0"	
24"	560D	1'-0"	1'-0"	1'-6"	2'-0"	3'-6"	5'-0"	7'-0"	8'-6"	11'-0"	12'-6"	1'-6"	4'-0"	5'-0"	6'-6"	9'-6"	15'-0"	16'-0"	16'-6"	17'-0"	17'-0"

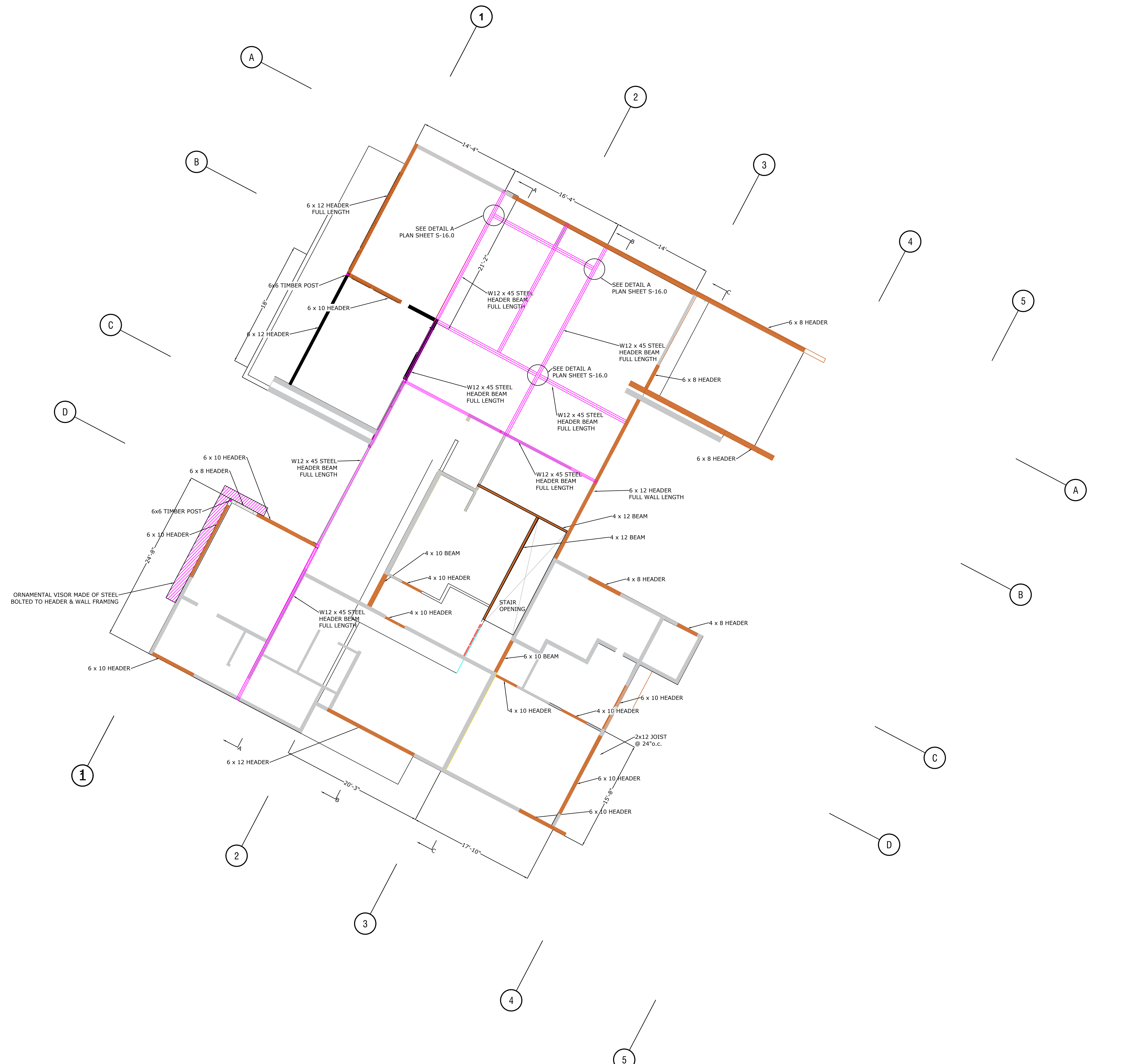
Table B - Intermediate or Cantilever Support

Minimum distance from edge of hole to inside face of nearest intermediate or cantilever support

JOIST DEPTH	TJI®	ROUND HOLE SIZE 4"	6"	7"	8"	10"	12"	14¾"	16¾"	18¾"	20"	SQUARE OR RECTANGULAR HOLE SIZE 4"	6"	7"	8"	10"	12"	14¾"	16¾"	18¾"	20"
18"	560D	1'-0"	1'-6"	2'-6"	4'-6"	7'-6"	11'-0"	16'-6"				3'-0"	7'-6"	9'-6"	11'-6"	16'-0"	17'-0"	19'-0"			
20"	560D	1'-0"	1'-0"	1'-6"	3'-0"	6'-0"	8'-6"	13'-6"	17'-0"			1'-0"	5'-6"	8'-0"	10'-0"	15'-0"	18'-0"	19'-6"	20'-6"		
22"	560D	1'-0"	2'-6"	3'-6"	4'-6"	6'-6"	8'-0"	11'-0"	14'-6"	17'-6"		3'-6"	6'-6"	8'-6"	10'-0"	19'-0"	20'-0"	21'-0"	21'-6"	22'-0"	
24"	560D	2'-6"	4'-0"	5'-0"	5'-6"	7'-0"	8'-6"	11'-0"	13'-6"	16'-0"	17'-6"	5'-0"	7'-6"	9'-0"	10'-6"	14'-0"	20'-0"	21'-0"	21'-6"	22'-0"	22'-0"

Rectangular holes based on measurement of longest side.

- Leave ⅛" of web (minimum) at top and bottom of hole. **DO NOT cut joist flanges.**
- Tables are based on uniform load tables in current design literature.
- For simple span (5' minimum), uniformly loaded joists used in residential applications, one maximum size round hole may be located at the center of the joist span **provided that no other holes occur in the joist.**



SECOND FLOOR - FLOOR FRAMING

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

Permanent Soldier Pile
& Timber Lagging
Retaining Wall

LUCIA E N G I N E E R I N G, I N C.
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
04-27-24

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Permanent Soldier Pile & Timber Lagging Retaining Wall

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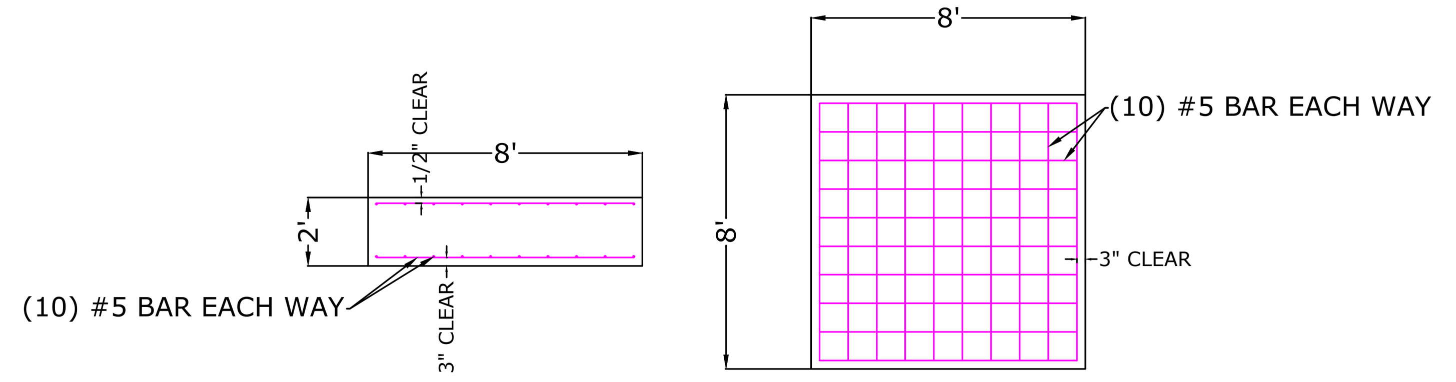
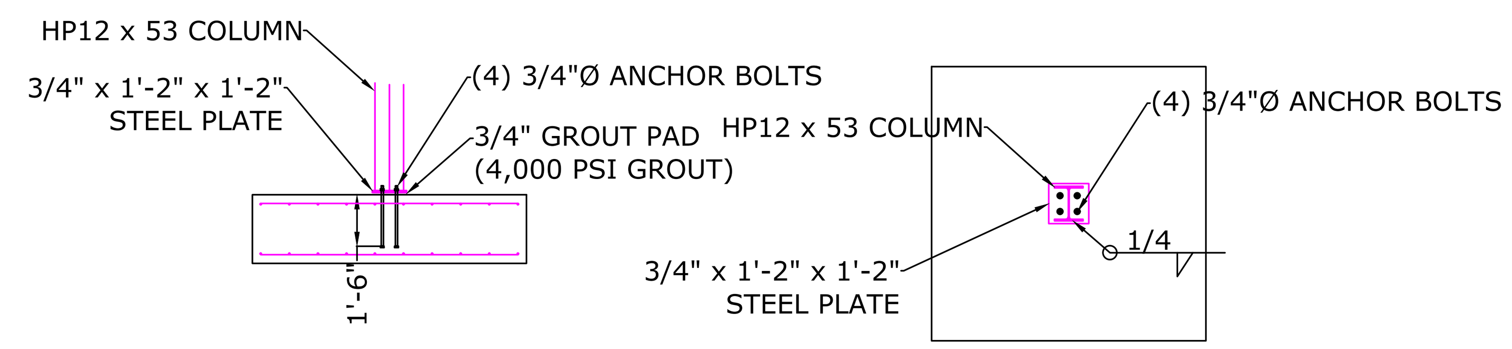
FRAMING NOTES:
- USE 3/4" STRUCTURAL 1 PLYWOOD WITH 10d NAILS SET @ 6" CENTERS EDGES & FIELD

ALTERNATES TO THE W16x89 STEEL BEAM SHOWN
W14 X 109, W18 X 97

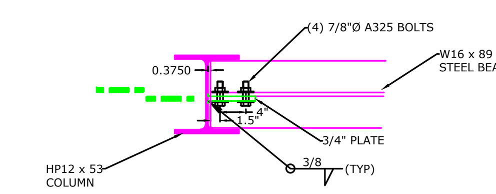
FIRST FLOOR - FLOOR FRAMING



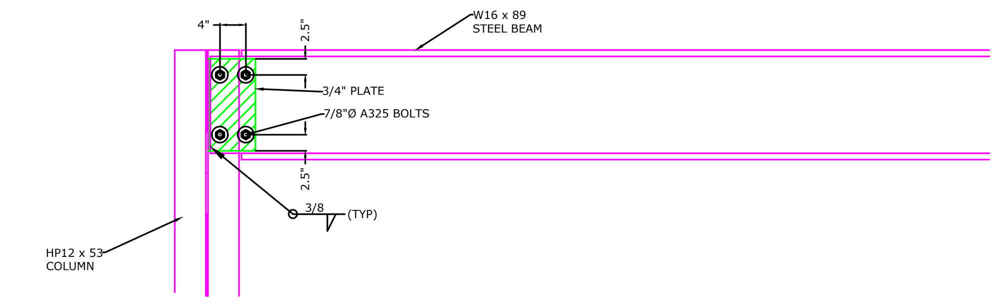
COLUMN FOOTING DETAIL



COLUMN TO FOOTING DETAIL

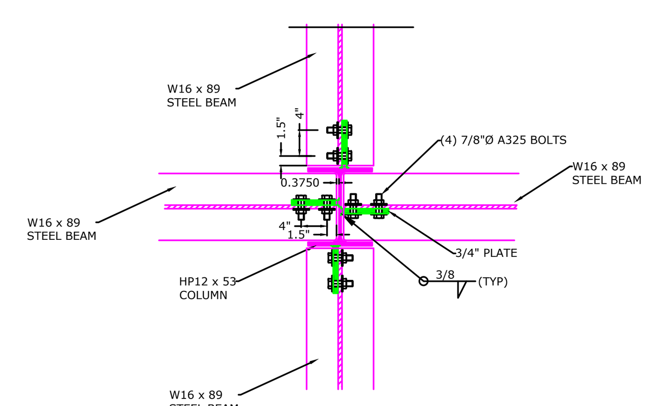


PLAN

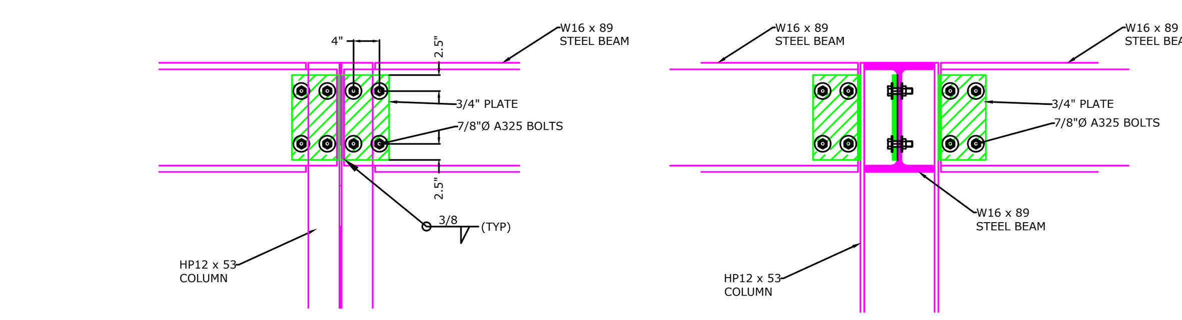


ELEVATION

DETAIL D

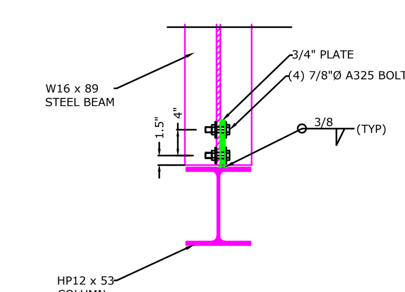


PLAN

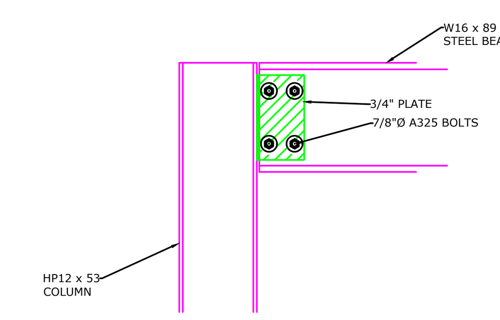


ELEVATION

DETAIL B

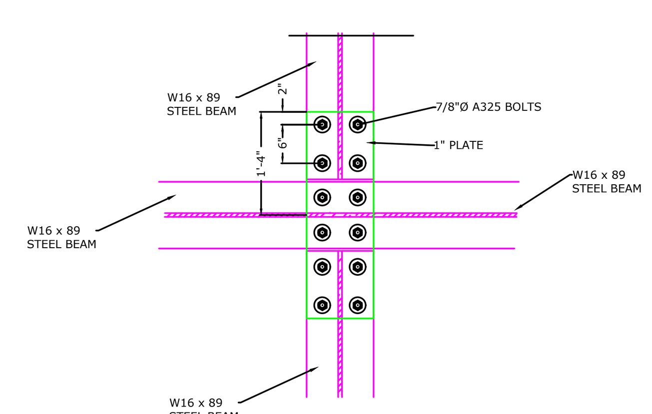


PLAN

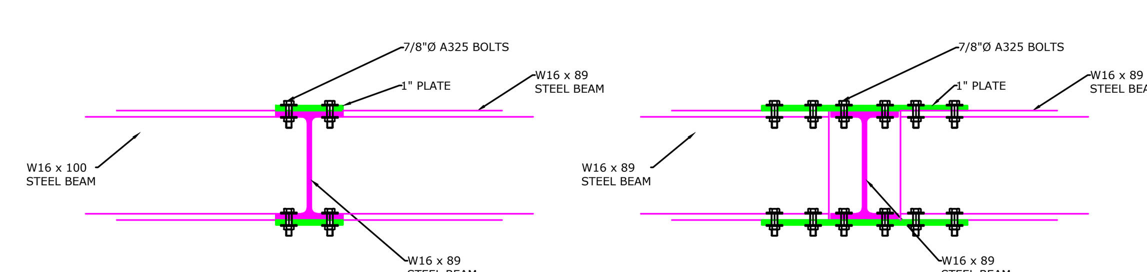


ELEVATION

DETAIL C

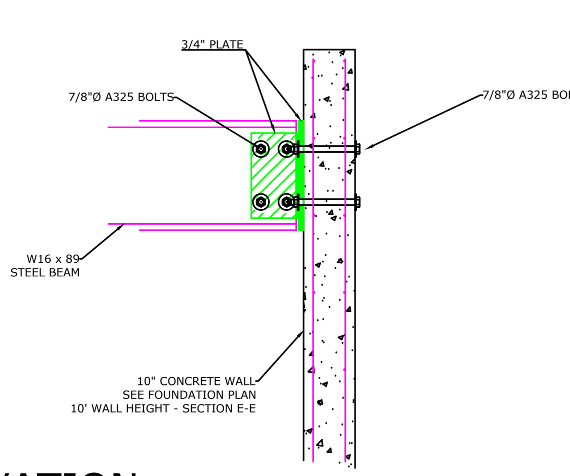


PLAN

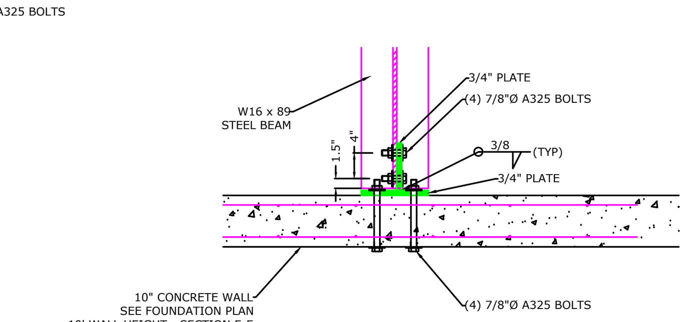


ELEVATION

DETAIL A



ELEVATION



PLAN

DETAIL E

ALTERNATES TO THE W16x89 STEEL BEAM SHOWN
W14 X 109, W18 X 97

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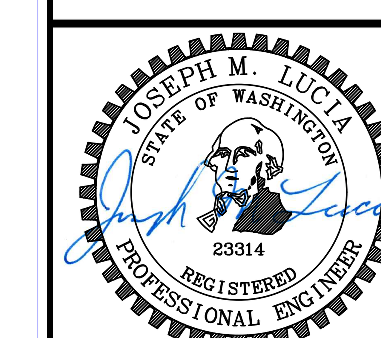

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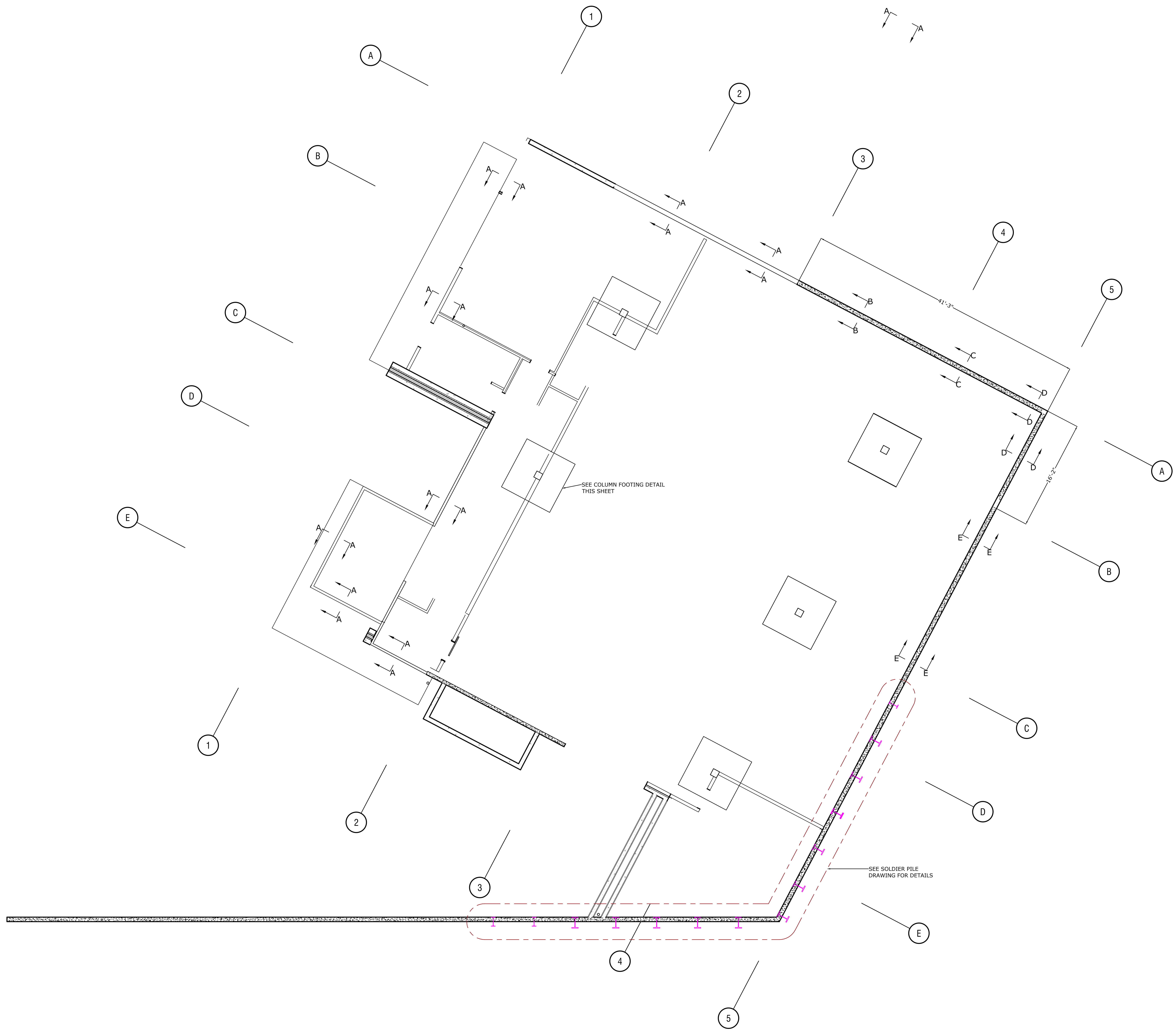

FOUNDATION PLAN

LANZ RESIDENCE
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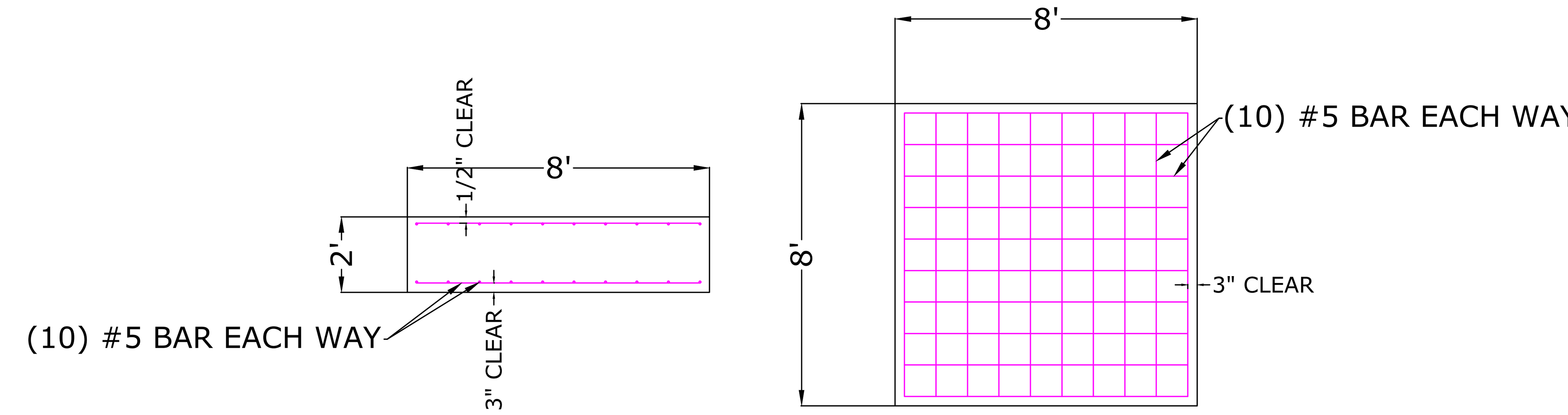
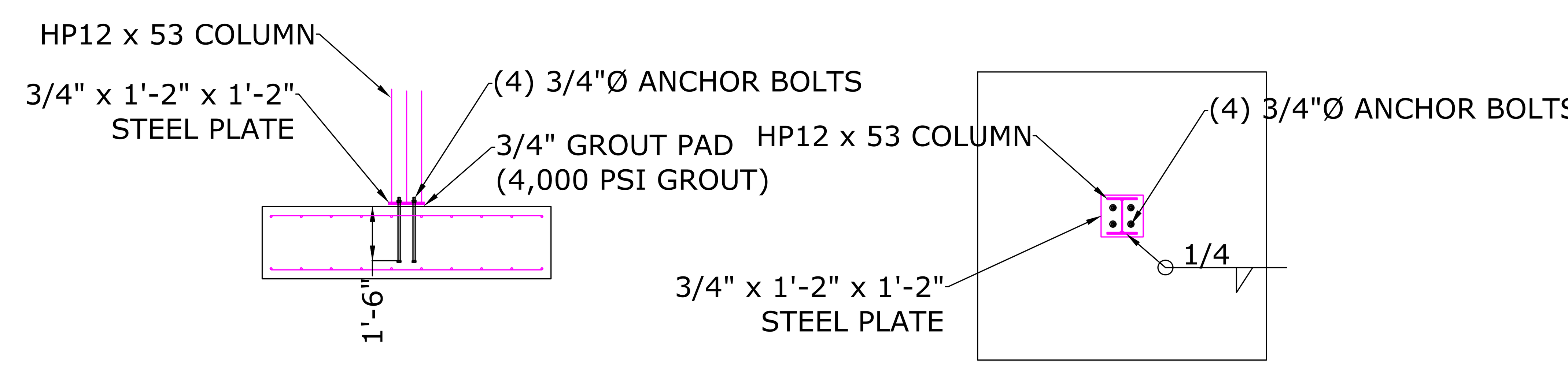
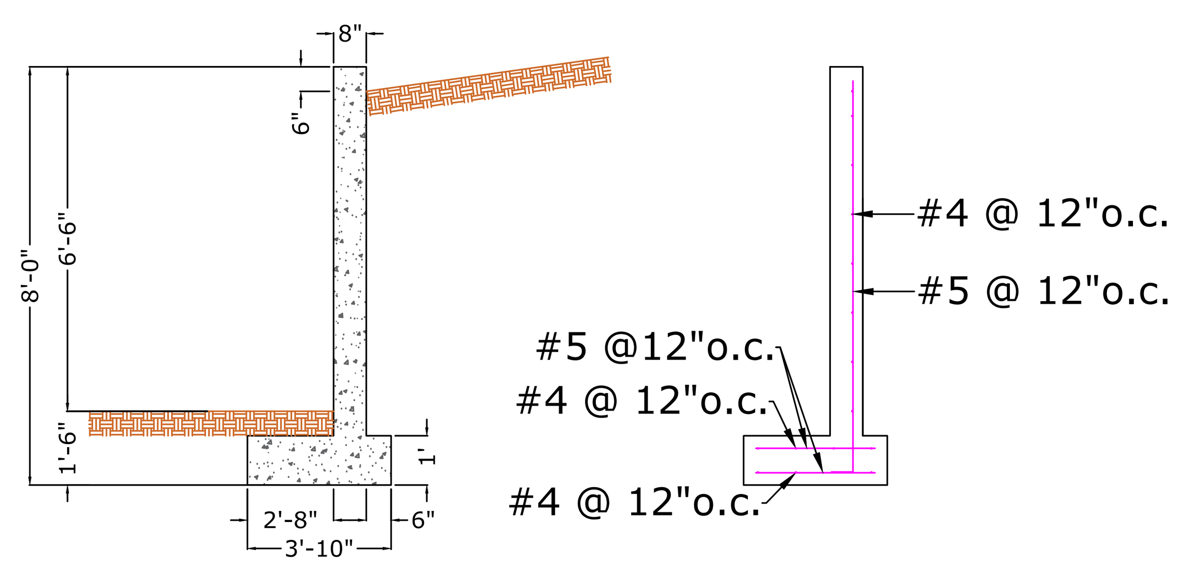


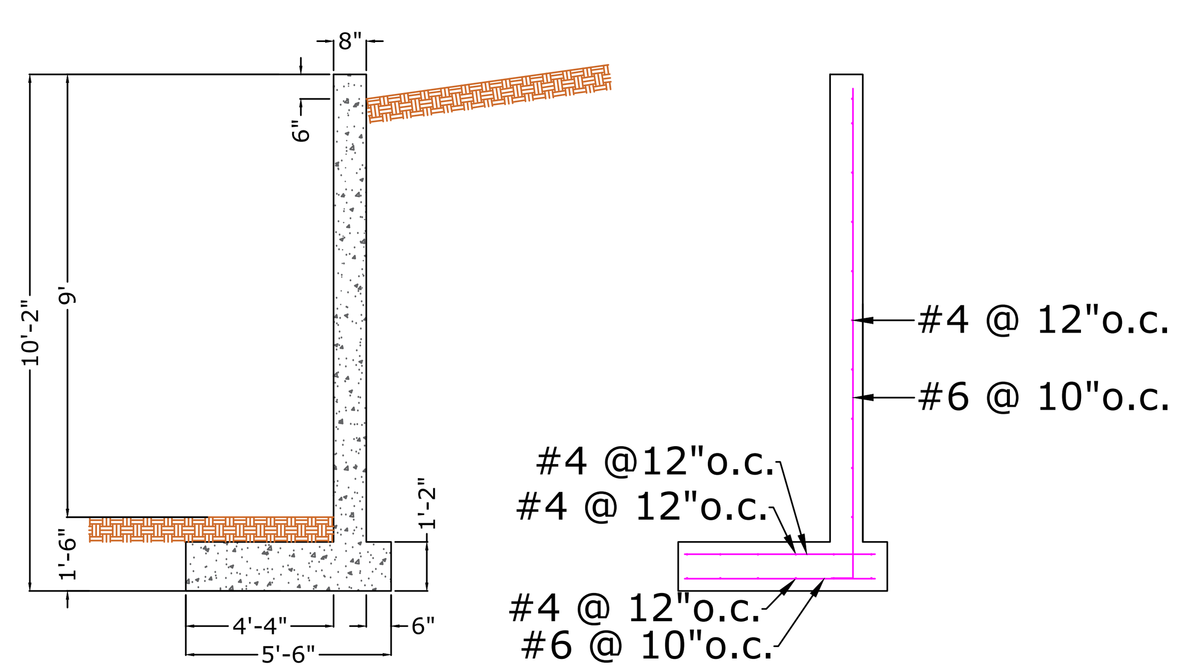
COLUMN FOOTING DETAIL



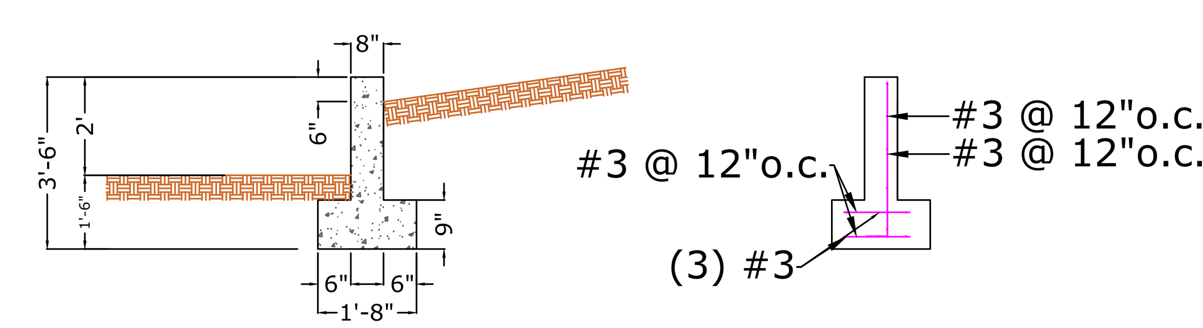
COLUMN TO FOOTING DETAIL



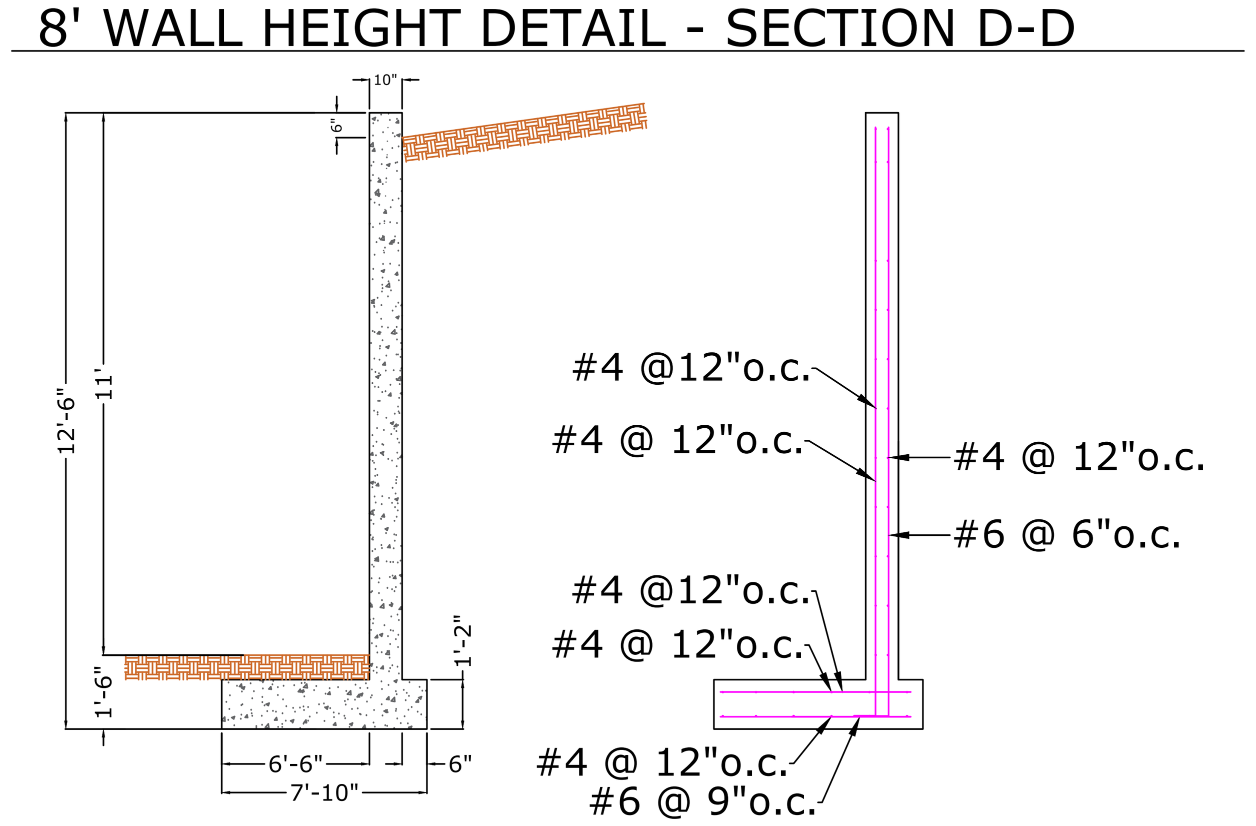
6' WALL HEIGHT DETAIL - SECTION C-C



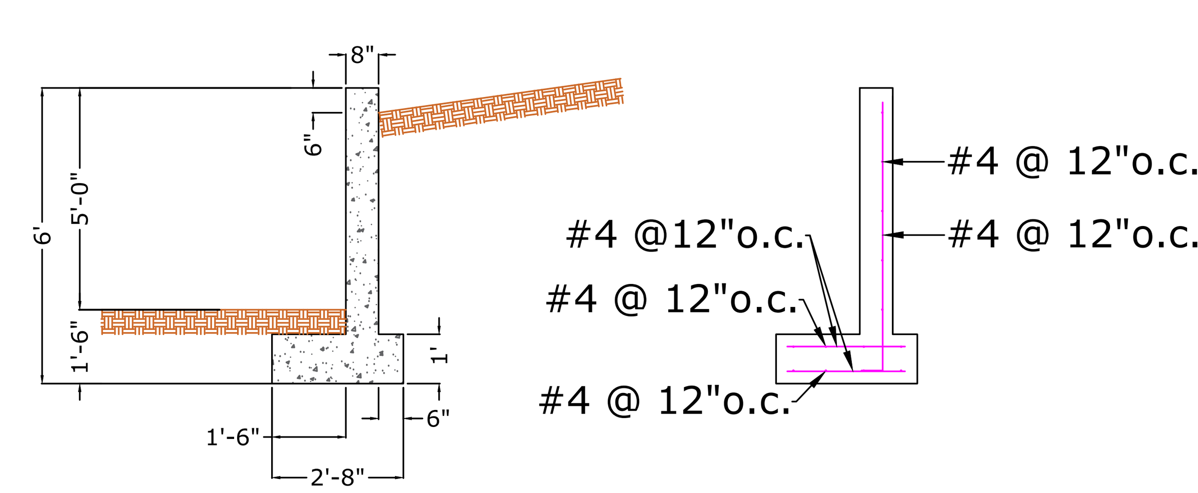
8' WALL HEIGHT DETAIL - SECTION D-D



STANDARD FOOTING DETAIL - SECTION A-A



10' WALL HEIGHT DETAIL - SECTION E-E



4' WALL HEIGHT DETAIL - SECTION B-B

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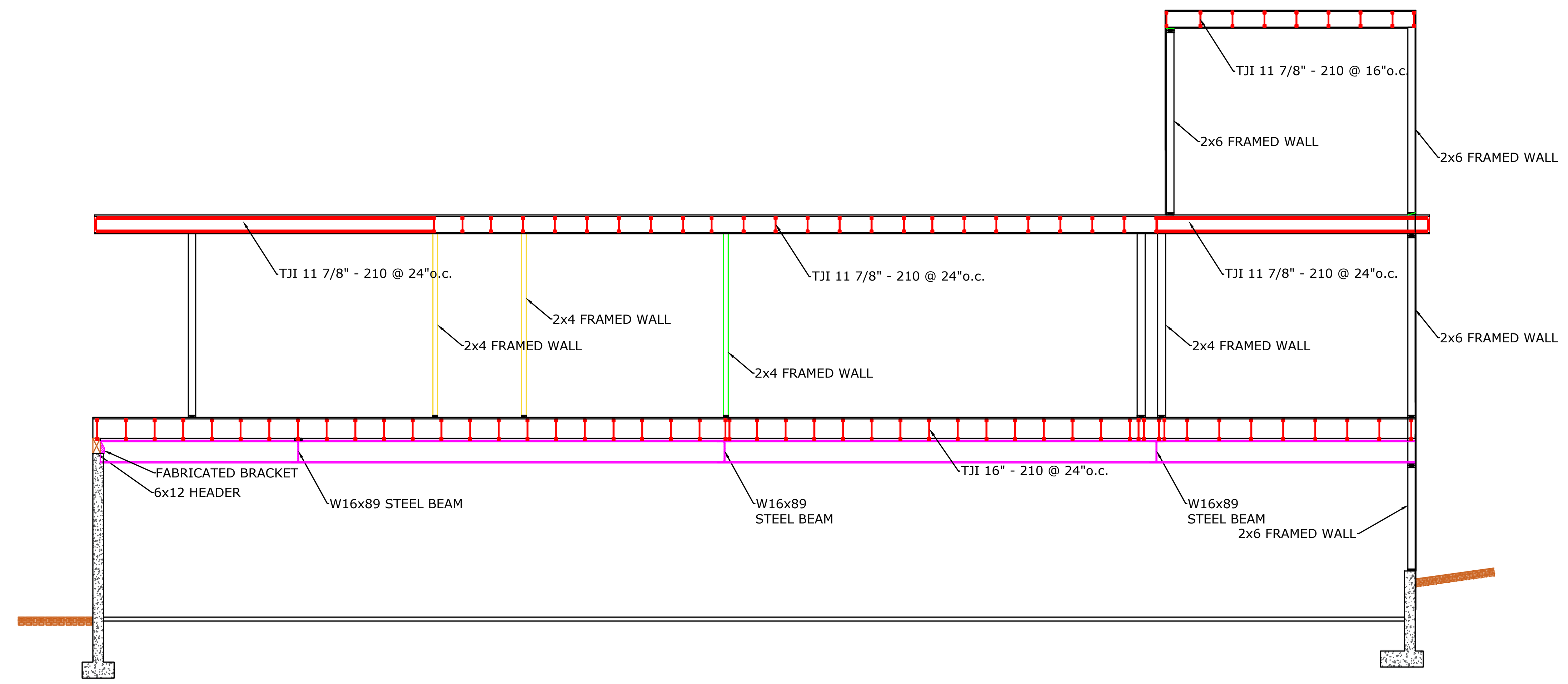
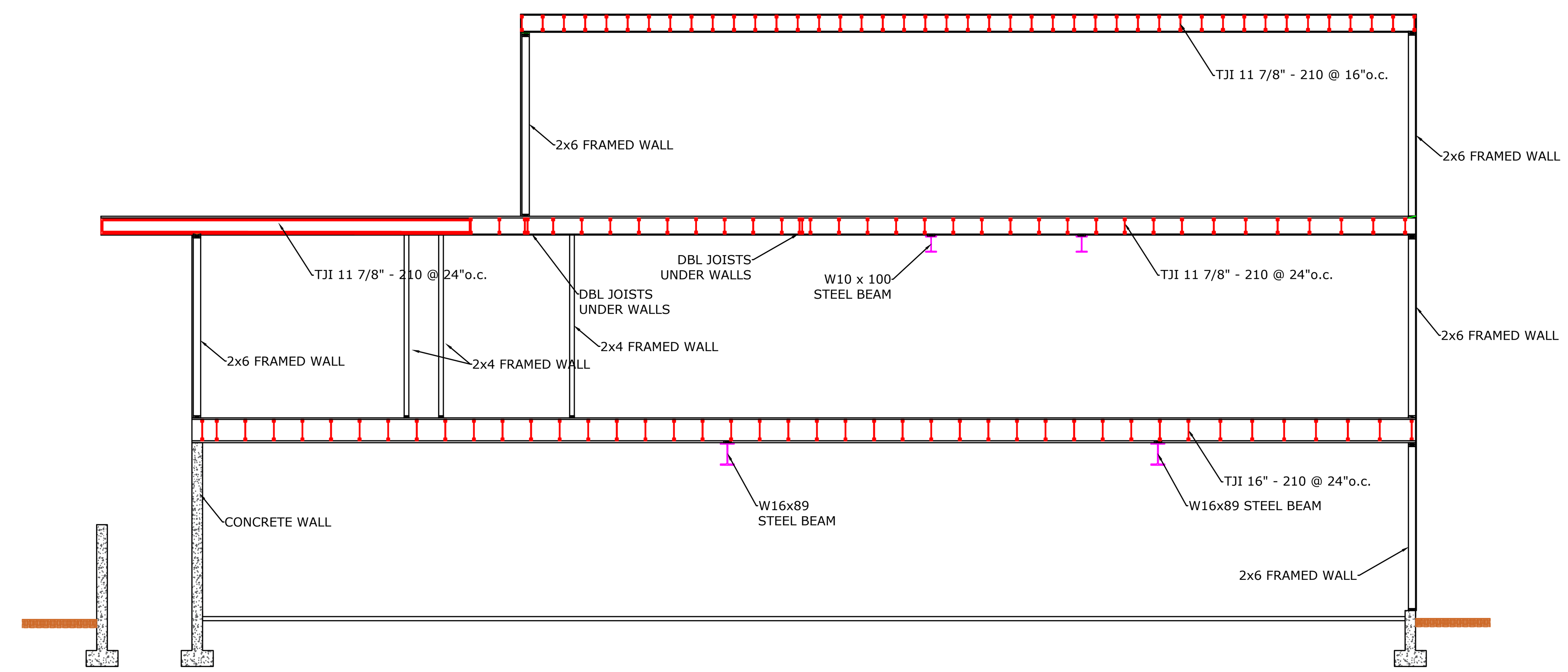
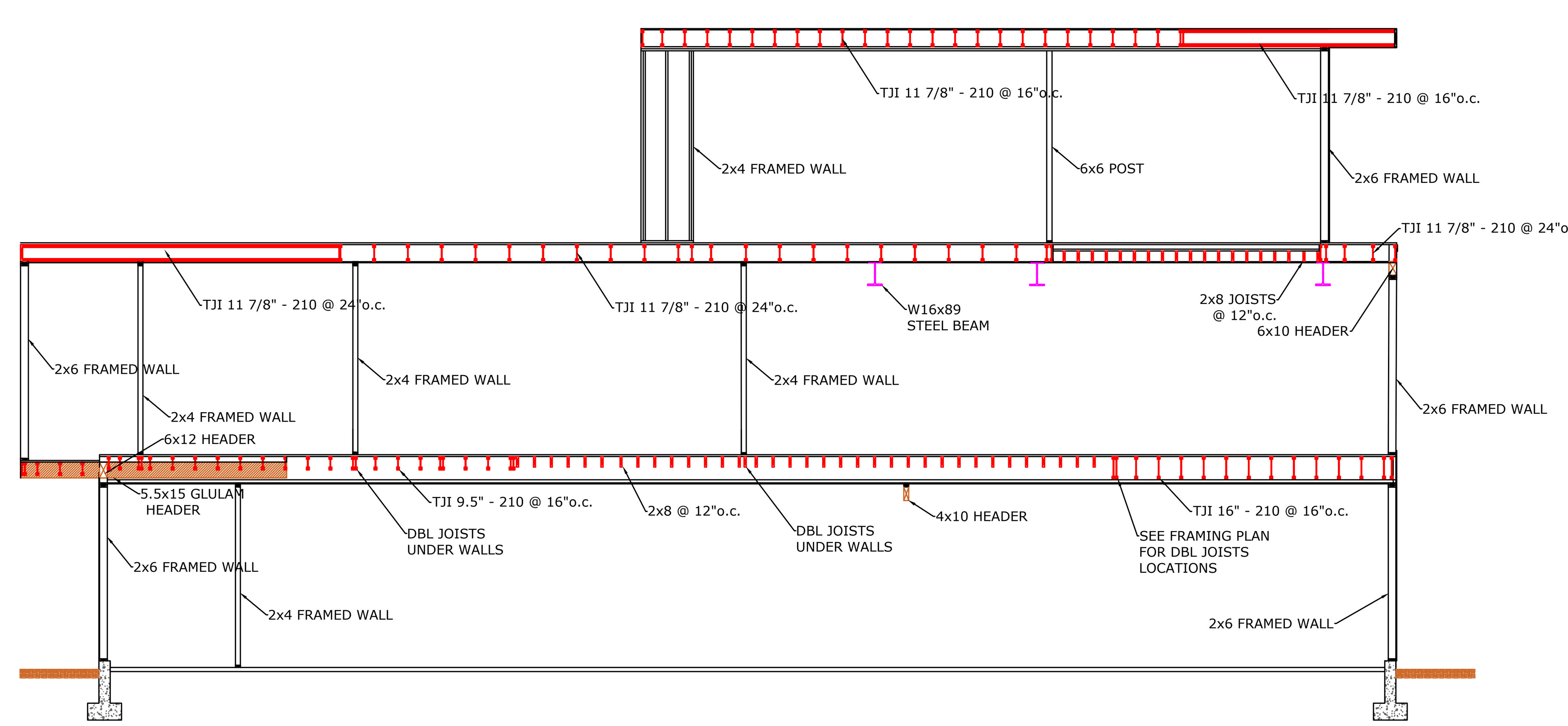


SECTION C-C



SECTION B-B



SECTION A-A

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