CT ENGINEERING
INC

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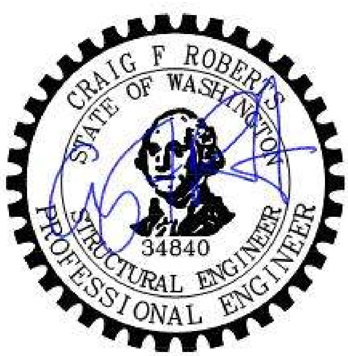
#21162

Structural Calculations For:

Piper Remodel

AT

8429 SE 33^{RD} PL
Mercer Island, WA 98040

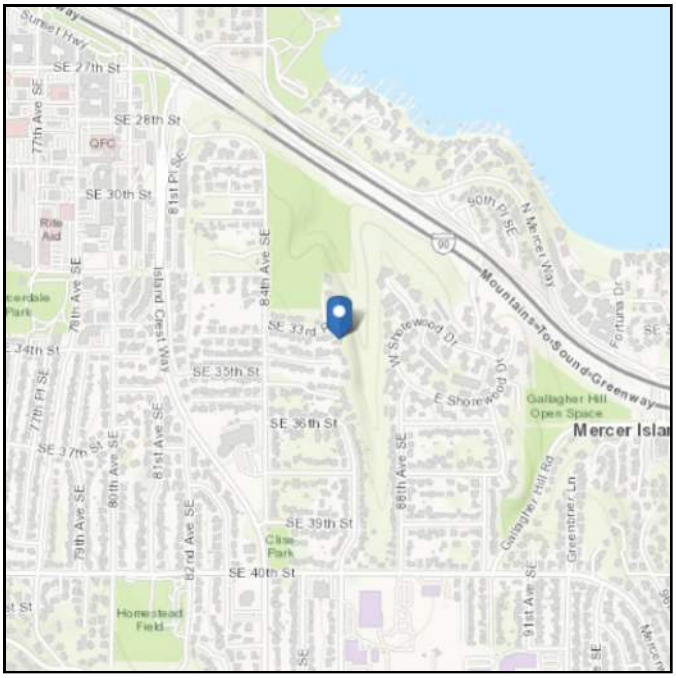


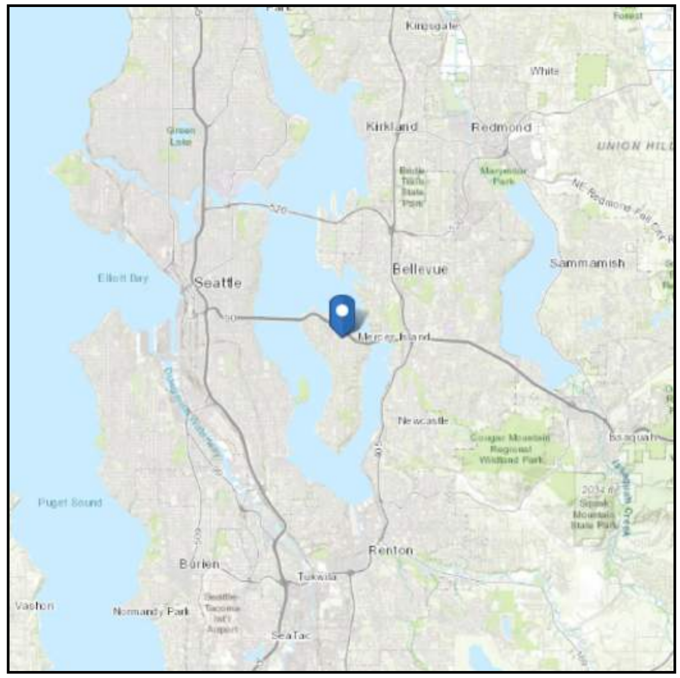
03/04/2022

Client: Form + Function Architecture
1800 Westlake Ave. N #205
Seattle, WA 98109

ASCE 7 Hazards Report

Address:		**Standard:**	ASCE/SEI 7-16	**Elevation:**	266.31 ft (NAVD 88)
No Address at This Location		**Risk Category:**	II	**Latitude:**	47.580185
		Soil Class:	D - Stiff Soil	**Longitude:**	-122.224291





Wind

Results:

Wind Speed	98 Vmph
10-year MRI	67 Vmph
25-year MRI	74 Vmph
50-year MRI	78 Vmph
100-year MRI	83 Vmph

Data Source:	ASCE/SEI 7-16, Fig. 26.5-1B and Figs. CC.2-1–CC.2-4, and Section 26.5.2
Date Accessed:	Fri Feb 25 2022

Value provided is 3-second gust wind speeds at 33 ft above ground for Exposure C Category, based on linear interpolation between contours. Wind speeds are interpolated in accordance with the 7-16 Standard. Wind speeds correspond to approximately a 7% probability of exceedance in 50 years (annual exceedance probability = 0.00143, MRI = 700 years).

Site is not in a hurricane-prone region as defined in ASCE/SEI 7-16 Section 26.2.

Seismic

Site Soil Class: D - Stiff Soil

Results:

S_S :	1.401	S_{D1} :	N/A
S_1 :	0.487	T_L :	6
F_a :	1	PGA :	0.599
F_v :	N/A	PGA_M :	0.659
S_{MS} :	1.401	F_{PGA} :	1.1
S_{M1} :	N/A	I_e :	1
S_{DS} :	0.934	C_v :	1.38

Ground motion hazard analysis may be required. See ASCE/SEI 7-16 Section 11.4.8.

Data Accessed: Fri Feb 25 2022

Date Source: USGS Seismic Design Maps

Snow

Results:

Ground Snow Load, p_g :	16 lb/ft^2
Elevation:	266.3 ft
Data Source:	
Date Accessed:	Fri Feb 25 2022

Statutory requirements of the Authority Having Jurisdiction are not included.

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CT # 21162

Project: PIPER RESIDENCE

Date: 02/28/2022

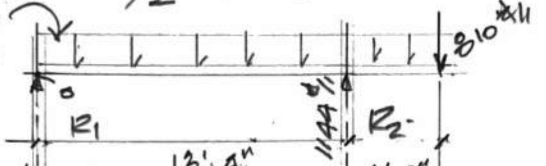
Client:

Page Number:

GRAVITY CONSIDERATIONS.

MAIN FLOOR CANTILEVER

ROOF $36/2\,(25+20) = 810$ #/1



$@M_0 = 10\ \#/FT\,(17')[17'/2] + 810\#[17'] - R_2[13'\text{-}4"]$

$\therefore R_2 = 1144\ \#/1$

$R_1 + 11.44 = 10\ \#/1\,(17') + 810$

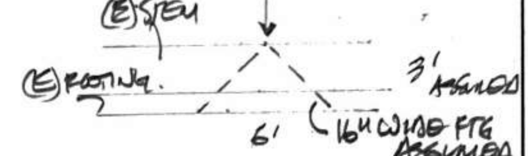
$R_1 = 980 - 1144$

$= -164$

REMAINDER OF FLOOR DL/1
$13'\text{-}4"/2\,(10\,PSF) = 66.5\ \#/1$

OPTION TO REMOVE MOST OF ROOF LOAD; IF PROVIDE GIRDER TRUSS OR BEAM.

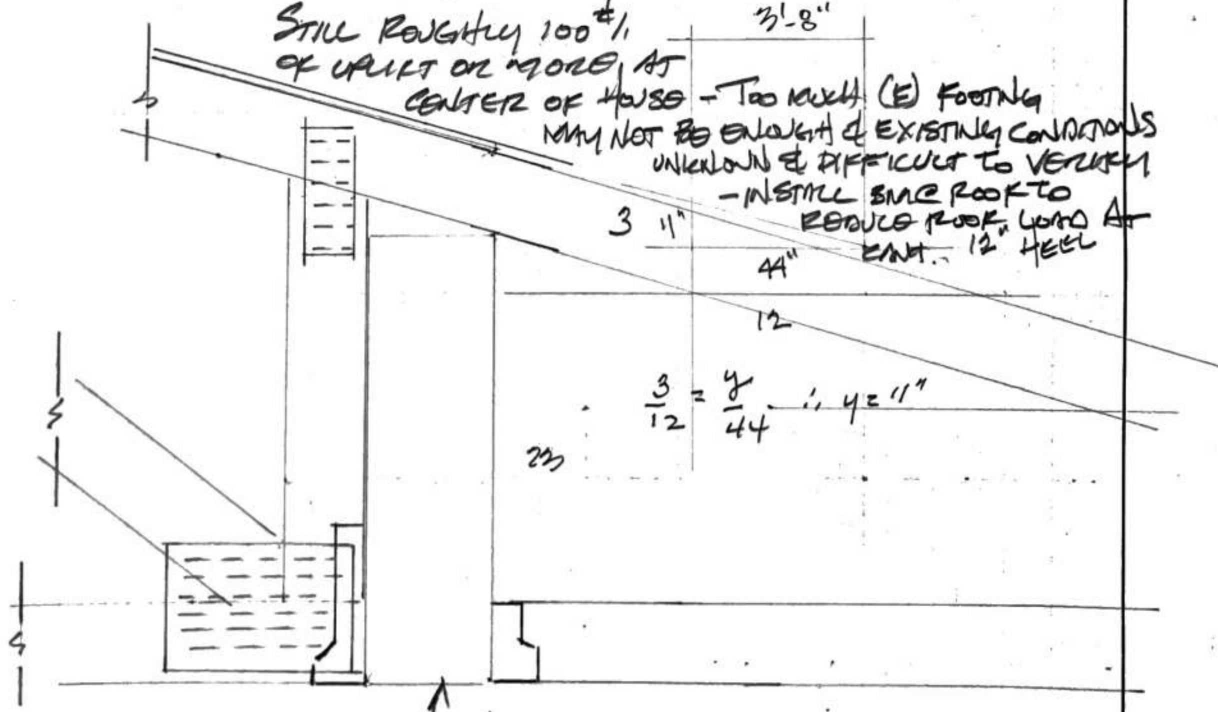
$23/2\,(810) = 9315\ \#$



$6'(16/12)(2000) = 16{,}000\ \#$

GEOMETRY.

STILL ROUGHLY 100#/1 OF UPLIFT OR MORE AT CENTER OF HOUSE - TOO MUCH (E) FOOTING MAY NOT BE ENOUGH & EXISTING CONDITIONS UNKNOWN & DIFFICULT TO VERIFY
- INSTALL SAME ROOF TO REDUCE ROOF LOAD AT CANT. 12" HEEL



$\frac{3}{12} = \frac{y}{44} \quad \therefore y = 11"$

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Client: Page Number:

GRAVITY CONSIDERATIONS (CONT.)

MAIN FLOOR CANTILEVER. (LOAD REDUCED W/(N) ROOF BM)

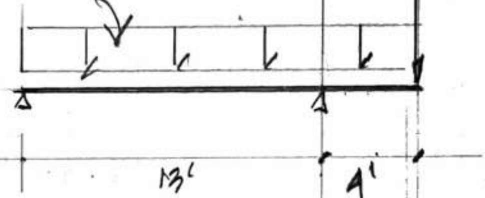
(2½ + 2')(25+20) = 113 #/, S + 90 #/, D   16/12 = 1.33

BMJ1

ATEMPT 2-2X12 @ 16" O.C.

W = 40(1.33) = 53 #/, L
20 (1.33) = 27 #/, D

(ROOF)
150 # S
120 # D



1-2X12 HF #2 SISTERED
TO EXISTING 2X12, @16" O.C.
OK - SEE FORTE WEB.

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ROOF LEVEL STEEL BEAM

SPAN = 19'

$\frac{l}{600} = \frac{19(12)}{600} = .38$

$W_{ROOF} = 35'\text{-}6''/2\,(20+25) = \underset{D}{\overset{29.58\#/\text{in}}{355\#/1}} + \underset{S}{\overset{37.08\#/\text{in}}{445\#/1}}$

$R_1 = R_2 \simeq \underset{D}{3373} + \underset{L}{4228}$

$= 7600\#$ TOTAL

SECTION IS DEFLECTION CONTROLLED.

$\Delta_D = \frac{5 \overset{29.58}{w} \overset{228''}{l^4}}{384EI} = .21$

$\Delta_L = \frac{5 \overset{37.08}{w} \overset{228''}{l^4}}{384EI} = .264 < .38$ OK.

(.47)
1/2" VERTICAL MOVEMENT
MAX @ CTR

E = 29E6 PSI
I = 170 IN⁴

W10 x 30 OK

PROVIDED COMPONENT MANUFACTURER CAN ACCOMODATE

$\Delta_D = .21''$ $\Delta_L = .38''$

NOT SURE IF SLIP TRACK @ TOP MAKES SENSE IF DOOR ATTACHES @ HEAD (HANGS)

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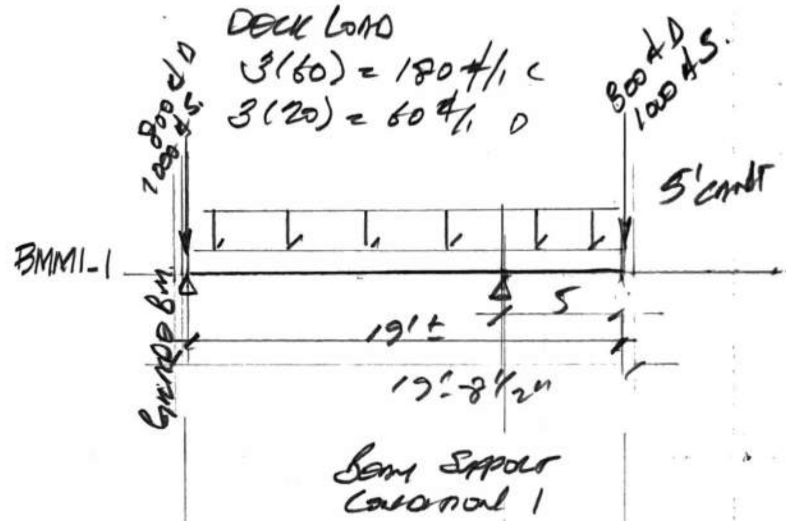
MAIN FLOOR – DECK BEAM W/ CANTILEVER

BMM1

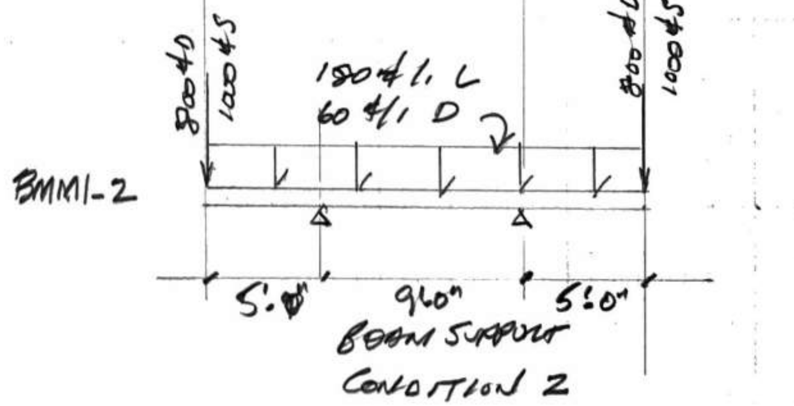
ROOF LOAD (10'x4')(20+25) = 800# D, 1000# S
(20 = D, 25 = S)

USING FORTE DESIGN SOFTWARE.

ATTEMPT 5½x12 GLB.



ATTEMPT 5½x12 GLB.



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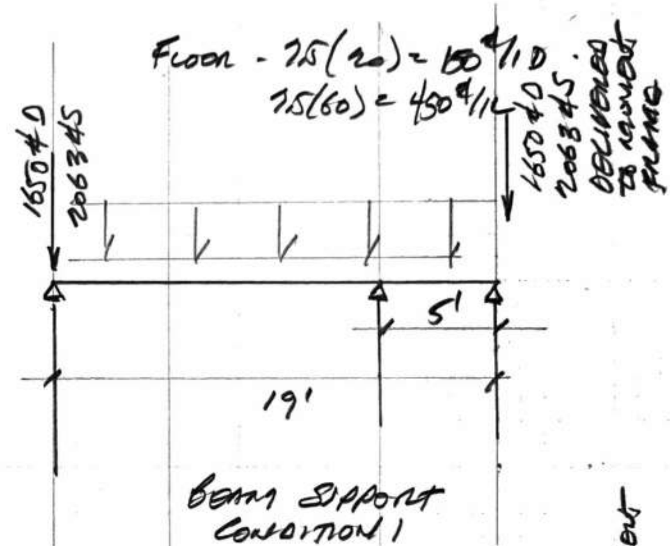
MAIN FLOOR - DECK BEAM W/ CANTILEVER

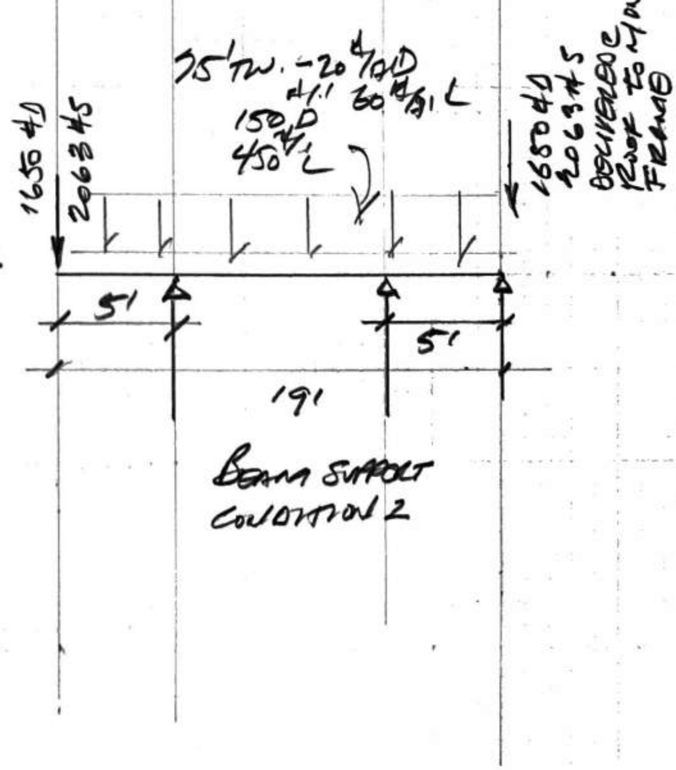
BMM2
Roof Load

7.5(11)(20+25) = 1650#, 2063#
D S

Attempt 5 1/2 x 12 GLB.   BMM2-1



Attempt 5 1/2 x 12 GLB   BMM2-2



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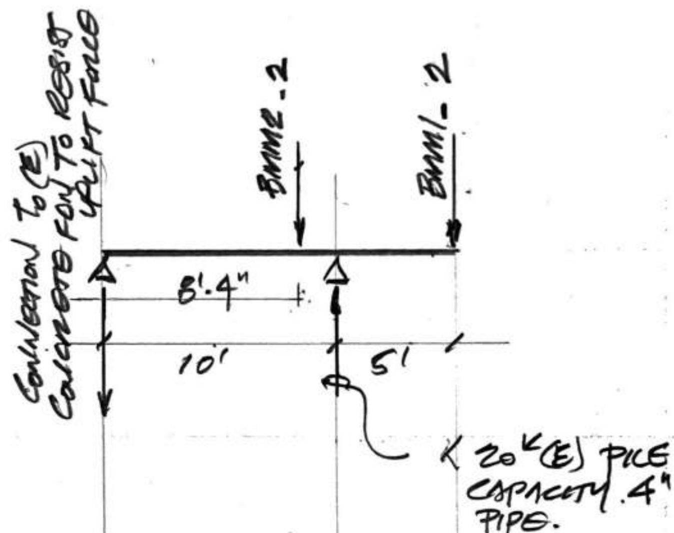
MAIN FLOOR - DECK W/ CANTILEVER.

BMM3

ATTEMPT 5½x12 GLB BMM3



ONLY CONDITION 2
OCCURS FOR THIS
BEAM.

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Project: PIPER RESIDENCE

Date: 03/04/2022

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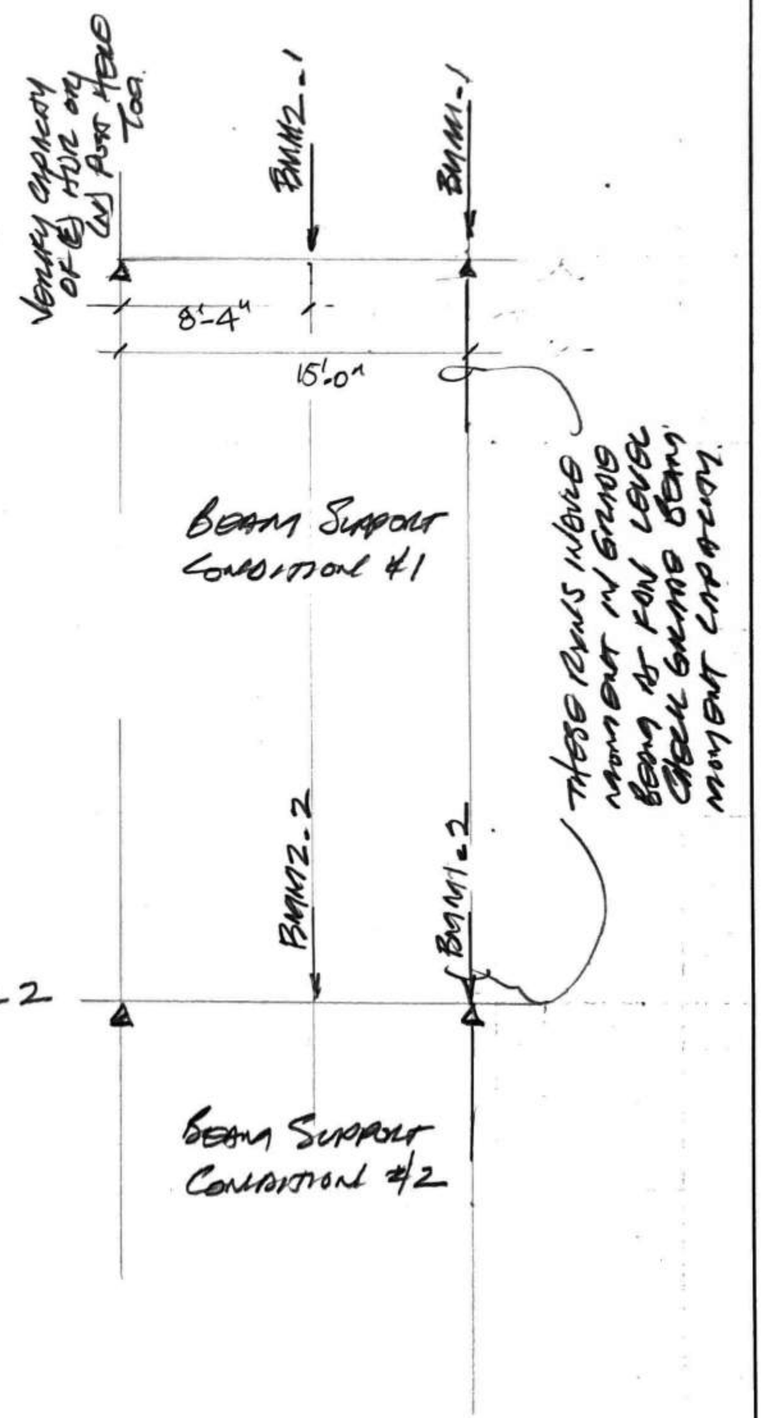
MAIN FLOOR - DECK W/ CANTILEVER.

BMM4

ATTEMPT 5½x12 GLB   BMM4-1

ATTEMPT 5½x12 GLB.   BMM4-2

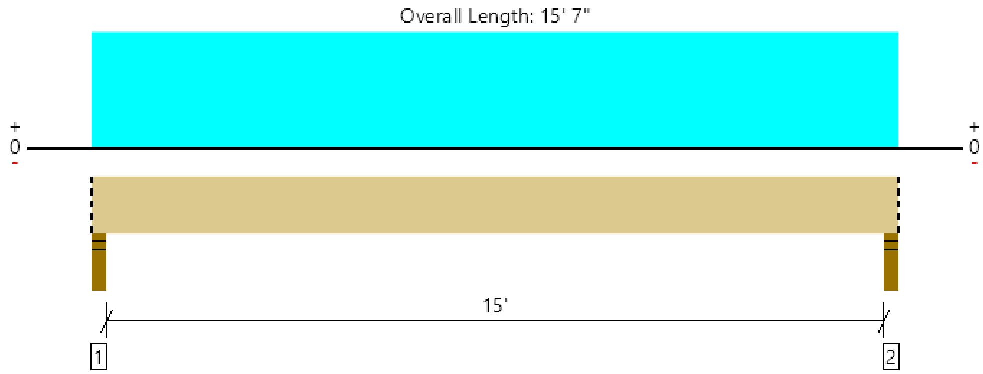


Roof			
Member Name	**Results**	**Current Solution**	**Comments**
BM1 - Roof: Drop Beam	Passed	1 piece(s) 5 1/2" x 12" 24F-V4 DF Glulam	
Main			
Member Name	**Results**	**Current Solution**	**Comments**
BMJ1 - Floor: Flush Beam	Passed	1 piece(s) 2 x 12 HF No.2	
BMM1_1 Floor: Flush Beam	Passed	1 piece(s) 5 1/2" x 12" 24F-V4 DF Glulam	
BMM1_2Floor: Flush Beam	Passed	1 piece(s) 5 1/2" x 12" 24F-V4 DF Glulam	
BMM2_1 Floor: Flush Beam	Failed	1 piece(s) 5 1/2" x 12" 24F-V4 DF Glulam	An excessive uplift of -1544 lbs at support located at 19' 11" failed this product.
BMM2_2 Floor: Flush Beam	Failed	1 piece(s) 5 1/2" x 13 1/2" 24F-V4 DF Glulam	An excessive uplift of -2718 lbs at support located at 14' 8 1/4" failed this product.
BMM3 Floor: Flush Beam	Passed	1 piece(s) 5 1/2" x 15" 24F-V4 DF Glulam	
BMM4_1 Floor: Flush Beam	Passed	1 piece(s) 8 3/4" x 12" 24F-V4 DF Glulam	

ForteWEB Software Operator	Job Notes
Benjamin J. McCann CT Engineering Inc. (206) 285-4512 bmccann@ctengineering.com	

PASSED

Roof, BM1 - Roof: Drop Beam
1 piece(s) 5 1/2" x 12" 24F-V4 DF Glulam



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

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	6261 @ 2"	12031 (3.50")	Passed (52%)	--	1.0 D + 1.0 S (All Spans)
Shear (lbs)	5223 @ 1' 3 1/2"	13409	Passed (39%)	1.15	1.0 D + 1.0 S (All Spans)
Pos Moment (Ft-lbs)	23359 @ 7' 9 1/2"	30360	Passed (77%)	1.15	1.0 D + 1.0 S (All Spans)
Live Load Defl. (in)	0.373 @ 7' 9 1/2"	0.508	Passed (L/490)	--	1.0 D + 1.0 S (All Spans)
Total Load Defl. (in)	0.686 @ 7' 9 1/2"	0.762	Passed (L/267)	--	1.0 D + 1.0 S (All Spans)

System : Roof
Member Type : Drop Beam
Building Use : Residential
Building Code : IBC 2015
Design Methodology : ASD
Member Pitch : 0/12

- Deflection criteria: LL (L/360) and TL (L/240).
- Allowed moment does not reflect the adjustment for the beam stability factor.
- Critical positive moment adjusted by a volume factor of 1.00 that was calculated using length L = 15' 3".
- The effects of positive or negative camber have not been accounted for when calculating deflection.
- The specified glulam is assumed to have its strong laminations at the bottom of the beam. Install with proper side up as indicated by the manufacturer.
- Applicable calculations are based on NDS.

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Snow	Total	
1 - Stud wall - DF	3.50"	3.50"	1.82"	2852	3409	6261	Blocking
2 - Stud wall - DF	3.50"	3.50"	1.82"	2852	3409	6261	Blocking

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

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	15' 7" o/c	
Bottom Edge (Lu)	15' 7" o/c	

- Maximum allowable bracing intervals based on applied load.

Vertical Loads	Location (Side)	Tributary Width	Dead (0.90)	Snow (1.15)	Comments
0 - Self Weight (PLF)	0 to 15' 7"	N/A	16.0	--	
1 - Uniform (PSF)	0 to 15' 7" (Front)	17' 6"	20.0	25.0	Default Load

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The product application, input design loads, dimensions and support information have been provided by BJM

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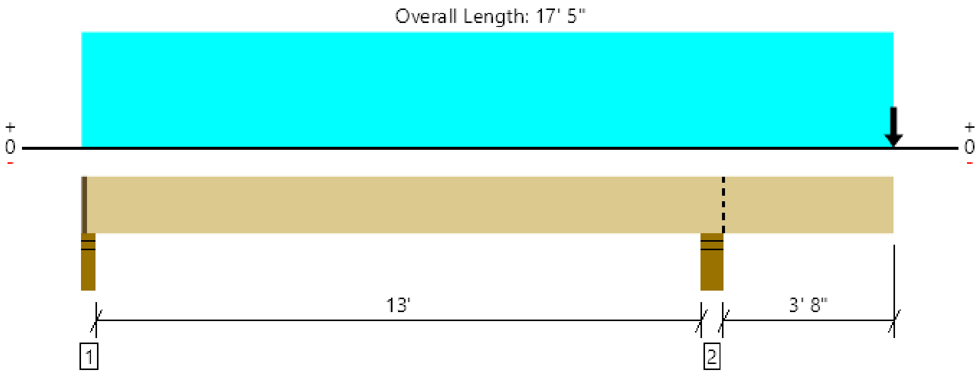
Main, BMJ1 - Floor: Flush Beam

1 piece(s) 2 x 12 HF No.2



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

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	515 @ 2"	1367 (2.25")	Passed (38%)	--	1.0 D + 1.0 L (Alt Spans)
Shear (lbs)	547 @ 12' 4 1/4"	1688	Passed (32%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	1543 @ 6' 2 5/8"	2241	Passed (69%)	1.00	1.0 D + 1.0 L (Alt Spans)
Live Load Defl. (in)	0.164 @ 6' 10 1/8"	0.267	Passed (L/978)	--	1.0 D + 1.0 L (Alt Spans)
Total Load Defl. (in)	0.146 @ 17' 5"	0.390	Passed (2L/638)	--	1.0 D + 0.75 L + 0.75 S (Alt Spans)

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

- Deflection criteria: LL (L/600) and TL (L/240).
- Overhang deflection criteria: LL (2L/600) and TL (2L/240).
- Allowed moment does not reflect the adjustment for the beam stability factor.
- Applicable calculations are based on NDS.

Supports	Bearing Length			Loads to Supports (lbs)				Accessories
	Total	Available	Required	Dead	Floor Live	Snow	Total	
1 - Stud wall - DF	3.50"	2.25"	1.50"	161	363/-25	-44	524/-69	1 1/4" Rim Board
2 - Stud wall - DF	5.50"	5.50"	1.80"	503	590	194	1287	Blocking

- Rim Board is assumed to carry all loads applied directly above it, bypassing the member being designed.
- Blocking Panels are assumed to carry no loads applied directly above them and the full load is applied to the member being designed.

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	7' 2" o/c	
Bottom Edge (Lu)	8' 2" o/c	

•Maximum allowable bracing intervals based on applied load.

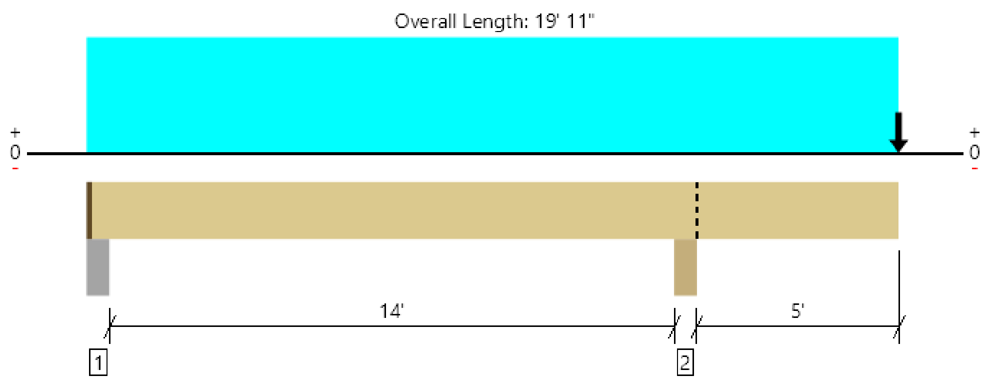
Vertical Loads	Location (Side)	Tributary Width	Dead (0.90)	Floor Live (1.00)	Snow (1.15)	Comments
0 - Self Weight (PLF)	1 1/4" to 17' 5"	N/A	4.3	--	--	
1 - Uniform (PLF)	0 to 17' 5" (Front)	N/A	27.0	53.0	-	Default Load
2 - Point (lb)	17' 5" (Front)	N/A	120	-	150	

Weyerhaeuser Notes

The product application, input design loads, dimensions and support information have been provided by BJM

ForteWEB Software Operator	Job Notes
Benjamin J. McCann CT Engineering Inc. (206) 285-4512 bmccann@ctengineering.com	

Main, BMM1_1 Floor: Flush Beam
1 piece(s) 5 1/2" x 12" 24F-V4 DF Glulam



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

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	4934 @ 14' 8 1/4"	19663 (5.50")	Passed (25%)	--	1.0 D + 0.75 L + 0.75 S (All Spans)
Shear (lbs)	2394 @ 15' 11"	13409	Passed (18%)	1.15	1.0 D + 0.75 L + 0.75 S (All Spans)
Pos Moment (Ft-lbs)	4241 @ 6' 1 1/16"	26400	Passed (16%)	1.00	1.0 D + 1.0 L (Alt Spans)
Neg Moment (Ft-lbs)	-10991 @ 14' 8 1/4"	23401	Passed (47%)	1.15	1.0 D + 0.75 L + 0.75 S (Alt Spans)
Live Load Defl. (in)	0.234 @ 19' 11"	0.261	Passed (2L/538)	--	1.0 D + 0.75 L + 0.75 S (Alt Spans)
Total Load Defl. (in)	0.387 @ 19' 11"	0.523	Passed (2L/324)	--	1.0 D + 0.75 L + 0.75 S (Alt Spans)

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

- Deflection criteria: LL (L/480) and TL (L/240).
- Overhang deflection criteria: LL (2L/480) and TL (2L/240).
- Allowed moment does not reflect the adjustment for the beam stability factor.
- Critical positive moment adjusted by a volume factor of 1.00 that was calculated using length L = 11' 6 1/8".
- Critical negative moment adjusted by a volume factor of 1.00 that was calculated using length L = 19' 7".
- The effects of positive or negative camber have not been accounted for when calculating deflection.
- The specified glulam is assumed to have its strong laminations at the bottom of the beam. Install with proper side up as indicated by the manufacturer.
- Applicable calculations are based on NDS.

Supports	Bearing Length			Loads to Supports (lbs)				Accessories
	Total	Available	Required	Dead	Floor Live	Snow	Total	
1 - Column - steel	5.50"	4.25"	1.50"	206	1352/-153	-364	1558/-517	1 1/4" Rim Board
2 - Column - DF	5.50"	5.50"	1.50"	2107	2405	1364	5876	Blocking

- Rim Board is assumed to carry all loads applied directly above it, bypassing the member being designed.
- Blocking Panels are assumed to carry no loads applied directly above them and the full load is applied to the member being designed.

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	19' 10" o/c	
Bottom Edge (Lu)	19' 10" o/c	

•Maximum allowable bracing intervals based on applied load.

Vertical Loads	Location (Side)	Tributary Width	Dead (0.90)	Floor Live (1.00)	Snow (1.15)	Comments
0 - Self Weight (PLF)	1 1/4" to 19' 11"	N/A	16.0	--	--	
1 - Uniform (PSF)	0 to 19' 11" (Front)	3'	20.0	60.0	-	Default Load
2 - Point (lb)	19' 11" (Front)	N/A	800	-	1000	

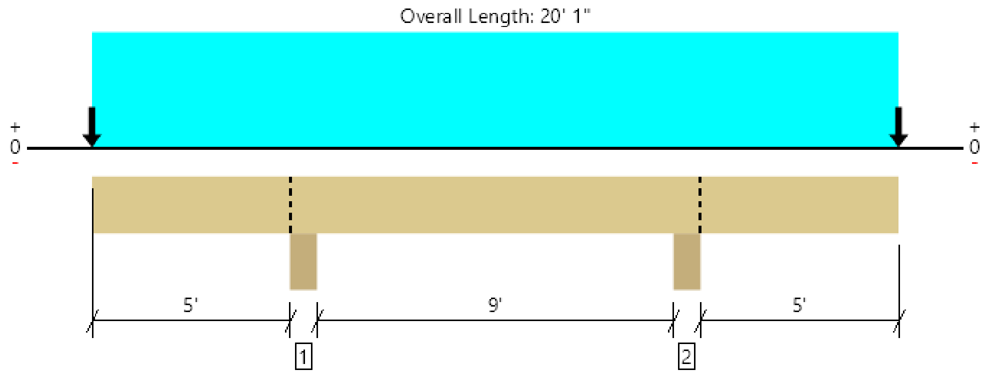
Weyerhaeuser Notes

The product application, input design loads, dimensions and support information have been provided by BJM

ForteWEB Software Operator	Job Notes
Benjamin J. McCann CT Engineering Inc. (206) 285-4512 bmccann@ctengineering.com	

Main, BMM1_2Floor: Flush Beam

1 piece(s) 5 1/2" x 12" 24F-V4 DF Glulam



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Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	4073 @ 5' 3 1/4"	22344 (6.50")	Passed (18%)	--	1.0 D + 0.75 L + 0.75 S (Adj Spans)
Shear (lbs)	2394 @ 4'	13409	Passed (18%)	1.15	1.0 D + 0.75 L + 0.75 S (All Spans)
Pos Moment (Ft-lbs)	0 @ N/A	N/A	Passed (N/A)	--	N/A
Neg Moment (Ft-lbs)	-11101 @ 5' 3 1/4"	23342	Passed (48%)	1.15	1.0 D + 0.75 L + 0.75 S (All Spans)
Live Load Defl. (in)	0.238 @ 0	0.264	Passed (2L/532)	--	1.0 D + 0.75 L + 0.75 S (Alt Spans)
Total Load Defl. (in)	0.437 @ 0	0.527	Passed (2L/290)	--	1.0 D + 0.75 L + 0.75 S (Alt Spans)

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

- Deflection criteria: LL (L/480) and TL (L/240).
- Overhang deflection criteria: LL (2L/480) and TL (2L/240).
- Allowed moment does not reflect the adjustment for the beam stability factor.
- Critical negative moment adjusted by a volume factor of 1.00 that was calculated using length L = 20' 1".
- The effects of positive or negative camber have not been accounted for when calculating deflection.
- The specified glulam is assumed to have its strong laminations at the bottom of the beam. Install with proper side up as indicated by the manufacturer.
- Applicable calculations are based on NDS.

	Bearing Length			Loads to Supports (lbs)				
Supports	Total	Available	Required	Dead	Floor Live	Snow	Total	Accessories
1 - Beam - DF	6.50"	6.50"	1.50"	1564	2070	1276	4910	Blocking
2 - Beam - DF	6.50"	6.50"	1.50"	1564	2070	1276	4910	Blocking

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

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	20' 1" o/c	
Bottom Edge (Lu)	20' 1" o/c	

- Maximum allowable bracing intervals based on applied load.

Vertical Loads	Location (Side)	Tributary Width	Dead (0.90)	Floor Live (1.00)	Snow (1.15)	Comments
0 - Self Weight (PLF)	0 to 20' 1"	N/A	16.0	--	--	
1 - Uniform (PSF)	0 to 20' 1" (Front)	3'	20.0	60.0	-	Default Load
2 - Point (lb)	0 (Front)	N/A	800	-	1000	
3 - Point (lb)	20' 1" (Front)	N/A	800	-	1000	

Weyerhaeuser Notes

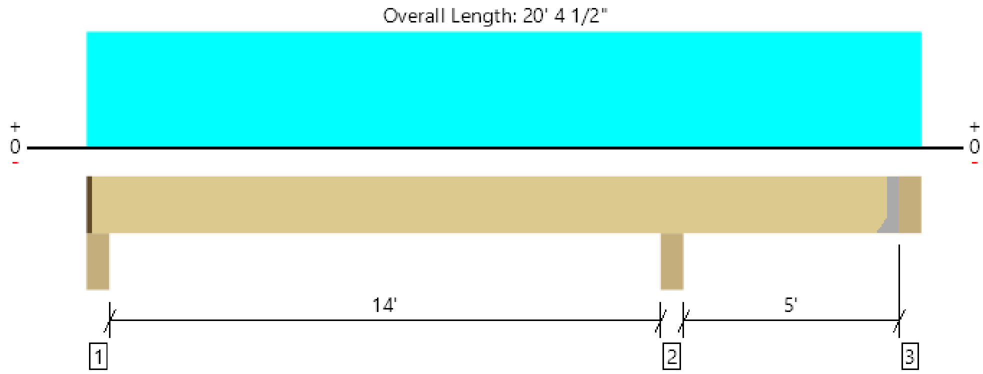
The product application, input design loads, dimensions and support information have been provided by BJM

ForteWEB Software Operator	Job Notes
Benjamin J. McCann CT Engineering Inc. (206) 285-4512 bmccann@ctengineering.com	

Main, BMM2_1 Floor: Flush Beam

1 piece(s) 5 1/2" x 12" 24F-V4 DF Glulam

An excessive uplift of -1544 lbs at support located at 19' 11" failed this product.



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

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	9213 @ 14' 8 1/4"	18906 (5.50")	Passed (49%)	--	1.0 D + 1.0 L (All Spans)
Shear (lbs)	4514 @ 13' 5 1/2"	11660	Passed (39%)	1.00	1.0 D + 1.0 L (All Spans)
Pos Moment (Ft-lbs)	10522 @ 6' 2 1/8"	26400	Passed (40%)	1.00	1.0 D + 1.0 L (Alt Spans)
Neg Moment (Ft-lbs)	-12192 @ 14' 8 1/4"	20350	Passed (60%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.171 @ 6' 10 1/8"	0.359	Passed (L/999+)	--	1.0 D + 1.0 L (Alt Spans)
Total Load Defl. (in)	0.232 @ 6' 10"	0.718	Passed (L/743)	--	1.0 D + 1.0 L (Alt Spans)

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

- Deflection criteria: LL (L/480) and TL (L/240).
- Allowed moment does not reflect the adjustment for the beam stability factor.
- Critical positive moment adjusted by a volume factor of 1.00 that was calculated using length L = 11' 8 1/4".
- Critical negative moment adjusted by a volume factor of 1.00 that was calculated using length L = 7' 11 13/16".
- The effects of positive or negative camber have not been accounted for when calculating deflection.
- The specified glulam is assumed to have its strong laminations at the bottom of the beam. Install with proper side up as indicated by the manufacturer.
- Applicable calculations are based on NDS.

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Floor Live	Total	
1 - Column - DF	5.50"	4.25"	1.50"	1016	2788	3804	1 1/4" Rim Board
2 - Beam - DF	5.50"	5.50"	2.68"	2483	6730	9213	None
3 - Hanger on 12" DF beam	5.50"	Hanger[1]	1.50"	-126	1304/-1418	1304/-1544	See note [1]

- Rim Board is assumed to carry all loads applied directly above it, bypassing the member being designed.
- At hanger supports, the Total Bearing dimension is equal to the width of the material that is supporting the hanger
- [1] See Connector grid below for additional information and/or requirements.

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	19' 10" o/c	
Bottom Edge (Lu)	19' 10" o/c	

•Maximum allowable bracing intervals based on applied load.

Connector: Simpson Strong-Tie						
Support	Model	Seat Length	Top Fasteners	Face Fasteners	Member Fasteners	Accessories
3 - Face Mount Hanger	HUCQ610-SDS	3.00"	N/A	12-SDS25212	6-SDS25212	

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

Vertical Loads	Location (Side)	Tributary Width	Dead (0.90)	Floor Live (1.00)	Comments
0 - Self Weight (PLF)	1 1/4" to 19' 11"	N/A	16.0	--	
1 - Uniform (PSF)	0 to 20' 4 1/2" (Front)	7' 6"	20.0	60.0	Default Load

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

FORTE WEB

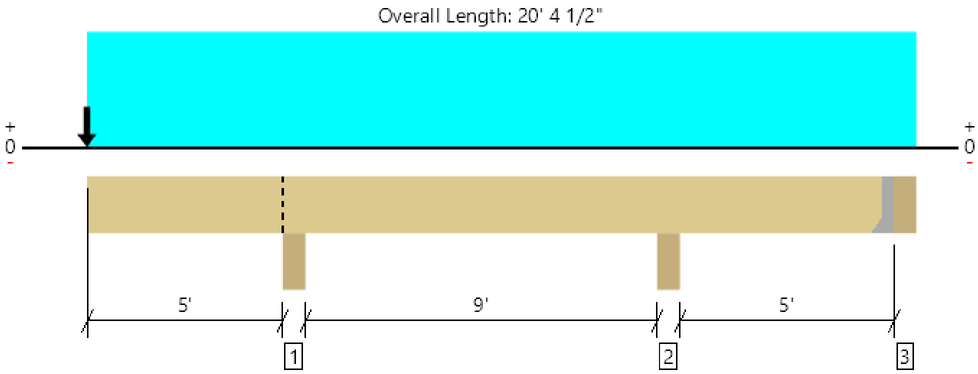
MEMBER REPORT

FAILED

Main, BMM2_2 Floor: Flush Beam

1 piece(s) 5 1/2" x 13 1/2" 24F-V4 DF Glulam

An excessive uplift of -2718 lbs at support located at 14' 8 1/4" failed this product.



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

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	11128 @ 5' 2 3/4"	18906 (5.50")	Passed (59%)	--	1.0 D + 0.75 L + 0.75 S (Adj Spans)
Shear (lbs)	5156 @ 3' 10 1/2"	15085	Passed (34%)	1.15	1.0 D + 0.75 L + 0.75 S (All Spans)
Pos Moment (Ft-lbs)	5830 @ 15' 1 3/8"	38424	Passed (15%)	1.15	1.0 D + 0.75 L + 0.75 S (Alt Spans)
Neg Moment (Ft-lbs)	-23631 @ 5' 2 3/4"	29619	Passed (80%)	1.15	1.0 D + 0.75 L + 0.75 S (All Spans)
Live Load Defl. (in)	0.242 @ 0	0.261	Passed (2L/518)	--	1.0 D + 0.75 L + 0.75 S (Alt Spans)
Total Load Defl. (in)	0.435 @ 0	0.523	Passed (2L/288)	--	1.0 D + 0.75 L + 0.75 S (Alt Spans)

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

- Deflection criteria: LL (L/480) and TL (L/240).
- Overhang deflection criteria: LL (2L/480) and TL (2L/240).
- Allowed moment does not reflect the adjustment for the beam stability factor.
- Critical positive moment adjusted by a volume factor of 1.00 that was calculated using length L = 7' 6 7/16".
- Critical negative moment adjusted by a volume factor of 1.00 that was calculated using length L = 11' 10 3/16".
- The effects of positive or negative camber have not been accounted for when calculating deflection.
- The specified glulam is assumed to have its strong laminations at the bottom of the beam. Install with proper side up as indicated by the manufacturer.
- Applicable calculations are based on NDS.

Supports	Bearing Length			Loads to Supports (lbs)				Accessories
	Total	Available	Required	Dead	Floor Live	Snow	Total	
1 - Beam - DF	5.50"	5.50"	3.24"	4701	4999	3571	13271	Blocking
2 - Beam - DF	5.50"	5.50"	1.50"	-546	4430	-2172	4430/-2718	None
3 - Hanger on 13 1/2" DF beam	5.50"	Hanger¹	1.50"	910	1657/-413	664	3231/-413	See note ¹

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

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	19' 11" o/c	
Bottom Edge (Lu)	19' 11" o/c	

•Maximum allowable bracing intervals based on applied load.

Connector: Simpson Strong-Tie						
Support	Model	Seat Length	Top Fasteners	Face Fasteners	Member Fasteners	Accessories
3 - Face Mount Hanger	HUC612	2.50"	N/A	16-16d	6-16d	

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

Vertical Loads	Location (Side)	Tributary Width	Dead (0.90)	Floor Live (1.00)	Snow (1.15)	Comments
0 - Self Weight (PLF)	0 to 19' 11"	N/A	18.0	--	--	
1 - Uniform (PSF)	0 to 20' 4 1/2" (Front)	7' 6"	20.0	60.0	-	Default Load
2 - Point (lb)	0 (Front)	N/A	1650	-	2063	

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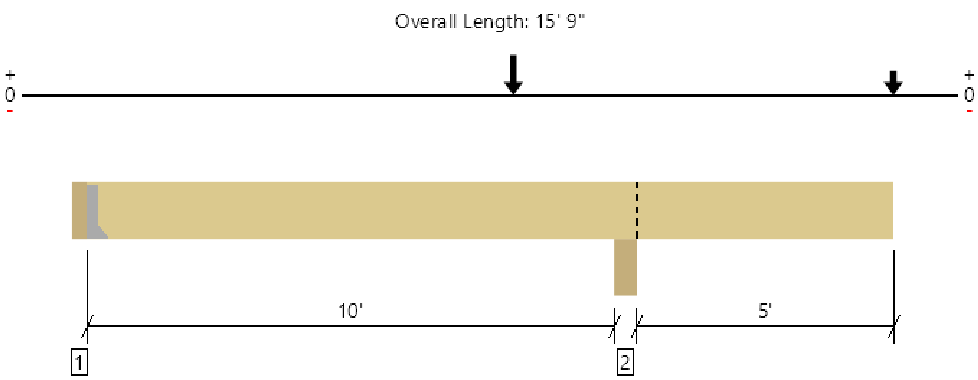
The product application, input design loads, dimensions and support information have been provided by BJM

Main, BMM3 Floor: Flush Beam

1 piece(s) 5 1/2" x 15" 24F-V4 DF Glulam



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

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	15139 @ 10' 6 1/4"	19663 (5.50")	Passed (77%)	--	1.0 D + 0.75 L + 0.75 S (All Spans)
Shear (lbs)	9583 @ 9' 1/2"	14575	Passed (66%)	1.00	1.0 D + 1.0 L (All Spans)
Pos Moment (Ft-lbs)	10213 @ 8' 4"	41250	Passed (25%)	1.00	1.0 D + 1.0 L (Alt Spans)
Neg Moment (Ft-lbs)	-19277 @ 10' 6 1/4"	31797	Passed (61%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.197 @ 15' 9"	0.261	Passed (2L/638)	--	1.0 D + 0.75 L + 0.75 S (Alt Spans)
Total Load Defl. (in)	0.255 @ 15' 9"	0.523	Passed (2L/492)	--	1.0 D + 0.75 L + 0.75 S (Alt Spans)

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

- Deflection criteria: LL (L/480) and TL (L/240).
- Overhang deflection criteria: LL (2L/480) and TL (2L/240).
- Allowed moment does not reflect the adjustment for the beam stability factor.
- Critical positive moment adjusted by a volume factor of 1.00 that was calculated using length L = 9' 2 7/8".
- Critical negative moment adjusted by a volume factor of 1.00 that was calculated using length L = 7' 2 7/8".
- -777 lbs uplift at support located at 3 1/2". Strapping or other restraint may be required.
- The effects of positive or negative camber have not been accounted for when calculating deflection.
- The specified glulam is assumed to have its strong laminations at the bottom of the beam. Install with proper side up as indicated by the manufacturer.
- Applicable calculations are based on NDS.

Supports	Bearing Length			Loads to Supports (lbs)				Accessories
	Total	Available	Required	Dead	Floor Live	Snow	Total	
1 - Hanger on 15" SPF beam	3.50"	Hanger¹	1.50"	282	1069/-1058	438/-270	1789/-1328	See note ¹
2 - Column - SPF	5.50"	5.50"	4.23"	6293	7058	4736	18087	Blocking

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

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	15' 6" o/c	
Bottom Edge (Lu)	15' 6" o/c	

- Maximum allowable bracing intervals based on applied load.

Connector: Simpson Strong-Tie						
Support	Model	Seat Length	Top Fasteners	Face Fasteners	Member Fasteners	Accessories
1 - Face Mount Hanger	HU612	2.50"	N/A	22-10dx1.5	8-10d	

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

ForteWEB Software Operator	Job Notes
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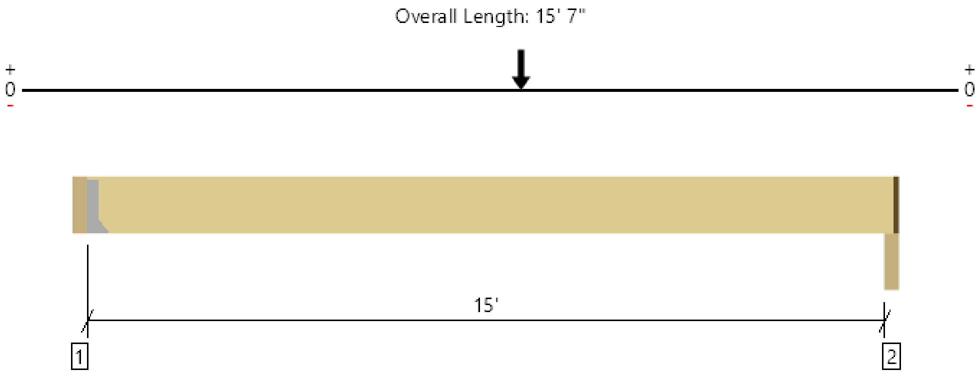
Vertical Loads	Location (Side)	Tributary Width	Dead (0.90)	Floor Live (1.00)	Snow (1.15)	Comments
0 - Self Weight (PLF)	3 1/2" to 15' 9"	N/A	20.0	--	--	
1 - Point (lb)	8' 4" (Front)	N/A	4701	4999	3571	Linked from: BMM2_2 Floor: Flush Beam, Support 1
2 - Point (lb)	15' 9" (Front)	N/A	1564	2070	1276	Linked from: BMM1_2Floor: Flush Beam, Support 1

Weyerhaeuser Notes

The product application, input design loads, dimensions and support information have been provided by BJM

Main, BMM4_1 Floor: Flush Beam
1 piece(s) 8 3/4" x 12" 24F-V4 DF Glulam



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

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	4508 @ 3 1/2"	8531 (1.50")	Passed (53%)	--	1.0 D + 1.0 L (All Spans)
Shear (lbs)	5063 @ 14' 3 1/2"	18550	Passed (27%)	1.00	1.0 D + 1.0 L (All Spans)
Pos Moment (Ft-lbs)	35424 @ 8' 4"	41141	Passed (86%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.368 @ 7' 11 3/4"	0.378	Passed (L/494)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.517 @ 7' 11 11/16"	0.756	Passed (L/351)	--	1.0 D + 1.0 L (All Spans)

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

- Deflection criteria: LL (L/480) and TL (L/240).
- Allowed moment does not reflect the adjustment for the beam stability factor.
- Critical positive moment adjusted by a volume factor of 0.98 that was calculated using length L = 15' 1 1/2".
- The effects of positive or negative camber have not been accounted for when calculating deflection.
- The specified glulam is assumed to have its strong laminations at the bottom of the beam. Install with proper side up as indicated by the manufacturer.
- Applicable calculations are based on NDS.

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Floor Live	Total	
1 - Hanger on 12" DF beam	3.50"	Hanger[1]	1.50"	1356	3152	4508	See note [1]
2 - Column - DF	3.50"	2.25"	1.50"	1515	3578	5093	1 1/4" Rim Board

- Rim Board is assumed to carry all loads applied directly above it, bypassing the member being designed.
- At hanger supports, the Total Bearing dimension is equal to the width of the material that is supporting the hanger
- [1] See Connector grid below for additional information and/or requirements.

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	15' 2" o/c	
Bottom Edge (Lu)	15' 2" o/c	

- Maximum allowable bracing intervals based on applied load.

Connector: Simpson Strong-Tie						
Support	Model	Seat Length	Top Fasteners	Face Fasteners	Member Fasteners	Accessories
1 - Face Mount Hanger	HGU9.00-SDS H=12	5.25"	N/A	36-SDS25212	24-SDS25212	

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

Vertical Loads	Location (Side)	Tributary Width	Dead (0.90)	Floor Live (1.00)	Comments
0 - Self Weight (PLF)	3 1/2" to 15' 5 3/4"	N/A	25.5	--	
1 - Point (lb)	8' 4" (Front)	N/A	2483	6730	Linked from: BMM2_1 Floor: Flush Beam, Support 2

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The product application, input design loads, dimensions and support information have been provided by BJM

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Project: PIPER RESIDENCE Date: 03/03/2022

Client: Page Number:

GRADE BEAM DESIGN

REQUIRED STRENGTH

1.4D (16-1)

1.2D + 1.6L + 0.5S (16-2)

DEMAND AT INTERIOR BEARING

$W_u = 1.4(1165\#) = 1631\#/\prime$ (16-1)

$W_u = 1.2(1165\#) + 1.6(600) + 0.5(375\#/\prime)$

$= 2545\#/\prime$ (16-2)

$M_u = W_u l_n^2/14$ POS. MOMENT END SPAN.

$W_u l_n^2/10$ NEG.

$2545\#/\prime(8.3')^2/8$

$= 21,915\#' = 21.9^{k'}$

ESTIMATE $A_s = M_u/4d = \dfrac{21.9^{k'}}{4(17.2'')} = .32^{\square''}$

MINIMUM $A_s = \dfrac{200\,b_w d}{f_y} = \dfrac{200(18'')(17.2'')}{60,000\ psi} = 1.03^{\square''}$

$A_{s\ 2\text{-}\#5} = 2(.31) = .62^{\square''}$

9.6.1.3 A_s PROVIDED ≥ 1.3 A_s REQUIRED BY ANALYSIS?

$a = \dfrac{A_s f_y}{.85 f'_c b} = \dfrac{.62^{\square''}(60)}{.85(2.5)(18)} = .973''$

$M_n = A_s f_y (d - a/2) = .62^{\square''}(60{,}000\ psi)(17.2'' - \dfrac{.973''}{2})$

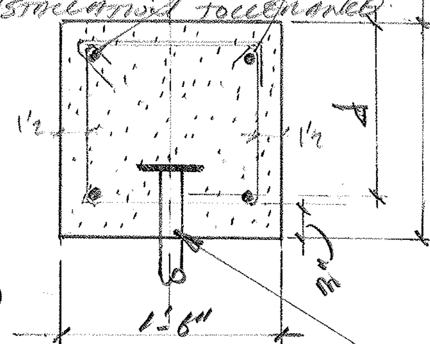
$= 621{,}742\#'' \left(\dfrac{1\ kip}{1000\#}\right)\left(\dfrac{1\ ft}{12\ in}\right)$

$= 51.8^{k'}$

$\phi M_n = 0.9(51.8^{k'}) = 46.6^{k'} \leq M_u = 21.9^{k'}$

$46.6^{k'} \leq 1.3(21.9) = 28.5^{k'}$ ∴

$18'' - 1\tfrac{1}{2}'' - 1\tfrac{1}{2}'' - 1'' - 5/8'' - 5/8'' - 3''$
$= 9.75$
$9.75/2 = 4.875''$
KEEP 18" WIDTH TO ALLOW FOR PIPE PILE INSTALLATION TOLERANCE



2-#5 TOP & BOTTOM W/ #4 @ 9" O.C. TIES.

$d = 18'' - 1/2'' - \dfrac{5/8}{2}$

$= 17.19$

$d/2 = 8.59''$

TIES @ 8" O.C. MEETS d/2 REQUIREMENT

2 OR 3" PIPE PILE @ CANTILEVERED PER PLAN.

GB 18x18

18.13.3.2

TIE SPACING =

1/2 SMALLEST ORTHOGONAL DIRECTION AND 12" ∴ 9" SPACING OK.

TABLE 9.3.1.1

$l/18.5 = \dfrac{8.3'(12''/\prime)}{18.5} = 5.38''$

18" >> 5.4" OK

MINIMUM REINF STEEL IS NOT REQUIRED IN THE GRADE BEAMS.

∴ 2-#5 T&B $f_y = 60$ ksi OK

CHECK SHEAR REINF. (ASSUME CONT. BEAM. 4 EQ SPANS)

$V_{u\,max} = .607\,w_u l = .607(2545\#/\prime)(8.3') = 12,821\#$

$= 12.8\ k$

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GRADE BEAM DESIGN (CONT.)

SHEAR REINFORCING CHECK CONT.

$$\frac{A_{v,min}}{s} = \frac{50 b_w}{f_{yt}} = \frac{50(18'')}{60{,}000 \text{ psi}} = .015 \text{ for \#4}$$

$$\frac{50(18)}{40{,}000} = .0225 \text{ for \#3}$$

9"

$\frac{1n}{1b}$ 1n2

$.015(9) = .135'' < .2^{in} \text{ OK}$ #4 BAR.

$.0225(9) = .2025$ OK

$A_{\#3} = .11 < .2025$ NG.

$$\phi V_n \geq V_u$$

$$\phi V_n = \phi(V_c + V_s)$$

$$V_c = 2\overset{1.0}{\lambda}\sqrt{\underset{2500}{f'_c}}\,\overset{18}{b_w}\overset{17.2''}{d} = 30{,}960\#$$

#4 $f_y = 60$ksi

$$V_s = \frac{A_v f_{yt}}{s} d = \frac{.2^{in}(60{,}000)}{9''}17.2'' = 22{,}933$$

$$\underset{0.7}{\phi} V_n = \underset{0.7}{\phi}(30.9^k + 22.9^k) = 33.7^k \geq 12.8^k$$

PROVIDE #4 @ 9" O.C.
CLOSED TIES (2 PART)
$f_y = 60$ksi min.

#3 $f_y = 40$ksi

$$V_s = \frac{0.11(40{,}000)}{9''}(17.2'') = 8408\# \text{ — NG.}$$

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GRADE BEAM DESIGN - CONT.　　ATTEMPT GB 16x16 W/ 2-#4 TOP & BOTTOM & #4 2 PART CLOSED TIES @ 8" O.C.

DEMAND AT INTERIOR BEARING GOVERNS SIM TO GB18x18 DESIGN PREVIOUS.

2-#5 T & B 60 GRADE W/ ~~#3~~ CLOSED 2 PART TIES @ 8" O.C.

$\therefore M_u = 21.9^{k'}$

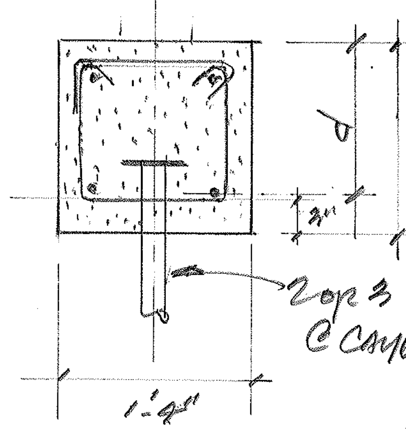
$A_s = M_u/4d = 21.9^{k'}/4(12.25) = .45^{\square''}$

$\text{MIN } A_s = \frac{200 b w d}{f_y} = .65^{\square''}$ (b_w = 16, d = 12.25, f_y = 60,000)

$A_{2\text{-}\#5} = .31(2) = .62^{\square''} < .65^{\square''}$

OK PER 9.6.1.3 PROVIDE 1.3 A_s REQ BY ANALYSIS.

$d = 16'' - 2'' - \frac{1}{2}'' - \frac{1/2''}{2} = 12.25''$



2 or 3" PIPE PILE @ CANTILEVER PER PLAN

$a = \frac{A_s f_y}{.85 f'_c b} = \frac{.62(60)}{.85(2.5)(16'')} = 1.09''$

$M_n = A_s f_y(d - a/2) = .62(60{,}000)(12.25 - \frac{1.09}{2})$
$= 435{,}426 \ \#''$
$= 36.3^{k'}$

GB 16x16

$\phi M_n = .9(36.3) = 32.6^{k'} \geq M_u = 21.9^{k'}$

$32.6^{k'} \geq 1.3 M_u = 1.3(21.9) = 28.47^{k'}$　9.6.1.3 VERIFIED

CHECK SHEAR REINFORCEMENT - #4 - 2 PART TIES @ 8" O.C.

$V_{u_{MAX}} = 12.8^k$ SIM GB18x18 DESIGN.

$Av_{MIN\ \#4} = \frac{50 b_w}{f_{yt}}(s) = .107$ (b_w = 16", f_{yt} = 60,000, s = 8")　$A_{s\ \#4} = .2^{\square''} > .107^{\square''}$

$\therefore$ ATTEMPT #4 @ 8" O.C.
→ 60 GRADE #3 BARS WOULD SAVE 45% IN STEEL FOR TIES... ~~MITCHELL~~ HARRIS SUPPLY SOLUTIONS.
* MITCHELL BLANCO
(503) 206. 2311
* ASTROM JJ, KIM, TOM
(425) 787. 9611

$\phi V_n \geq V_u$

$\phi V_n = \phi(V_c + V_s) = \phi(2\lambda\sqrt{f'_c}\, b w d + \frac{A_v f_{yt} d}{s}) =$ (ϕ = 0.7, λ = 1.0, f'_c = 2500, b_w = 16", d = 12.25", A_v = .11, f_{yt} = 60,000, d = 12.25, s = 8")

$= 0.7(19{,}600 + 10{,}106) = 20{,}794 > 12.8^{k'}$ OK
$20.8^{k'} > 12.8^{k'}$

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LATERAL ANALYSIS (WIND) (ASCE 7-16)

$V = 100$ MPH } https://doi.org/10.5066/F7NK3C76
EXPOSURE B. }
$K_{zt} = 1.3$. }

CHAPTER 28, PART 2, ENCLOSED SIMPLE DIAPHRAGM LOW-RISE BUILDINGS.

12 MRH = 20'±
13

28.5.3

$$P_S = \lambda K_{zt} P_{S30} \quad (K_{zt} = 1.3)$$

$\lambda = 1.29$ — MRH 20'
1.35 — 25'
} FIGURE 28.5-1

3:12 ROOF PITCH (14° ~ 15° OK)

		λ	K_{zt}	P_{S30}	P_S	
HORIZONTAL	A	1.29	1.3	19.9	33.4	
	B			-6.6	-11.07	
	C			13.3	22.3	WALL
	D			-3.8	-6.37	ROOF
VERTICAL	E			-19.1	-32	EDGE
	F			-12.4	-20.8	
	G			-13.3	-22.3	FIELD
	H			-9.5	-16	
OVERHANG	E_{OH}			-26.7	-44.8	OVERHANG
	G_{OH}			-20.9	-35	

3 DIFFERENT DESIGN CONSIDERATIONS.

1. P_{S30} PRESSURES.
2. P_{S30} PRESSURES w/ $P_S = 0$ IN B & D ZONES. (SEE FIGURE 28.5-1; NOTE #7)
3. P_S A&C = 16 PSF } SEE 28.5.4
   P_S B&D = 8 PSF }

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LATERAL ANALYSIS. (SEISMIC)

Google Earth

8429 SE 33RD PL. ⇒ 47.580185° lat

MERCER ISLAND, WA 98040 -122.224291° lon

288' ELEVATION

$S_{D1} = \frac{2}{3} S_{M1}$ (.8766) $= .584$

$S_{M1} = F_V S_1 = .8766$ (S_1 = 0.487, F_V = 1.8)

1.8 ∂- 11.4.8 SITE SPECIFIC GROUND MOTION
NO - EXCEPTION #2

$C_S = \frac{S_{DS}}{\left(\frac{R}{I_e}\right)} = \frac{.934}{\left(\frac{\ }{1.0}\right)}$

R VARIES (6½ ; 8 ; 3½)
SEE NEXT SHEET FOR C_S.
(PLAN SHEET W/ SUMMARY LATERAL SYSTEMS @ MAIN STORY WALLS)

A. BEARING WALL SYSTEM

15 - LIGHT FRAMED WALLS SHEATHED WITH WOOD STRUCTURAL PANELS RATED FOR SHEAR RESISTANCE — R = 6½ — Ω = 3 (2½ - FLEXIBLE DIAPHRAGM)

14.1
12.2.5.5 — C. MOMENT RESISTING FRAME SYSTEMS
1 - SPECIAL MOMENT FRAME — R = 8 — Ω = 3 (2½ FLEX DIA.)

14.1
12.2.5.6 — 4 - STEEL ORDINARY MOMENT FRAME — R = 3½ — Ω = 3 (2½ FLEX DIA.)

SEISMIC DESIGN CATEGORY D
FOOTNOTE l - 12.2.5.6 LIMITATIONS.

ORDINARY MOMENT FRAME (2 STORY)
PERMITTED BY 12.2.5.6 b.

$h_n = 35' > 18'\text{-}1''$ OK

ROOF & FLOOR DL < 35 PSF OK

EXTERIOR WALL
DL < 20 PSF OK

STUDS	1.6 PSF	2x6@16"
½" DRYWALL	12.2 PSF	
½" PLYWOOD	1.7 PSF	
	10 PSF < 20 PSF	

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LATERAL ANALYSIS (SEISMIC) CONT.

12.8.6 STORY DRIFT DETERMINATION.
Δ-DESIGN STORY DRIFT

SMF
DUAL SYSTEM W/
WOOD FRAMED
SHEAR WALLS SAME
DIRECTION

SMF - R=6½ Cd=4

OMF - R=3½ Cd=3

DEFLECTION AT LEVEL x

$$\delta_x = \frac{C_d\,\delta_{xe}}{I_e} =$$ ELASTIC TOTAL DISPLACEMENT UNDER STRENGTH LEVEL FORCES.

$I_e = 1.0$ TABLE 1.5-2
RISK CATEGORY II
(TABLE 1.5-1)

TABLE 12.12-1

$\Delta_a = .020\,h_{sx} = 4.28''$ (17'-10")

BASE-MAIN $.02(85'') = 1.7''$
MAIN-ROOF $.02(130'') = 2.6''$
$\overline{4.3.}$

12.3.4.2 $\rho = 1.3.$

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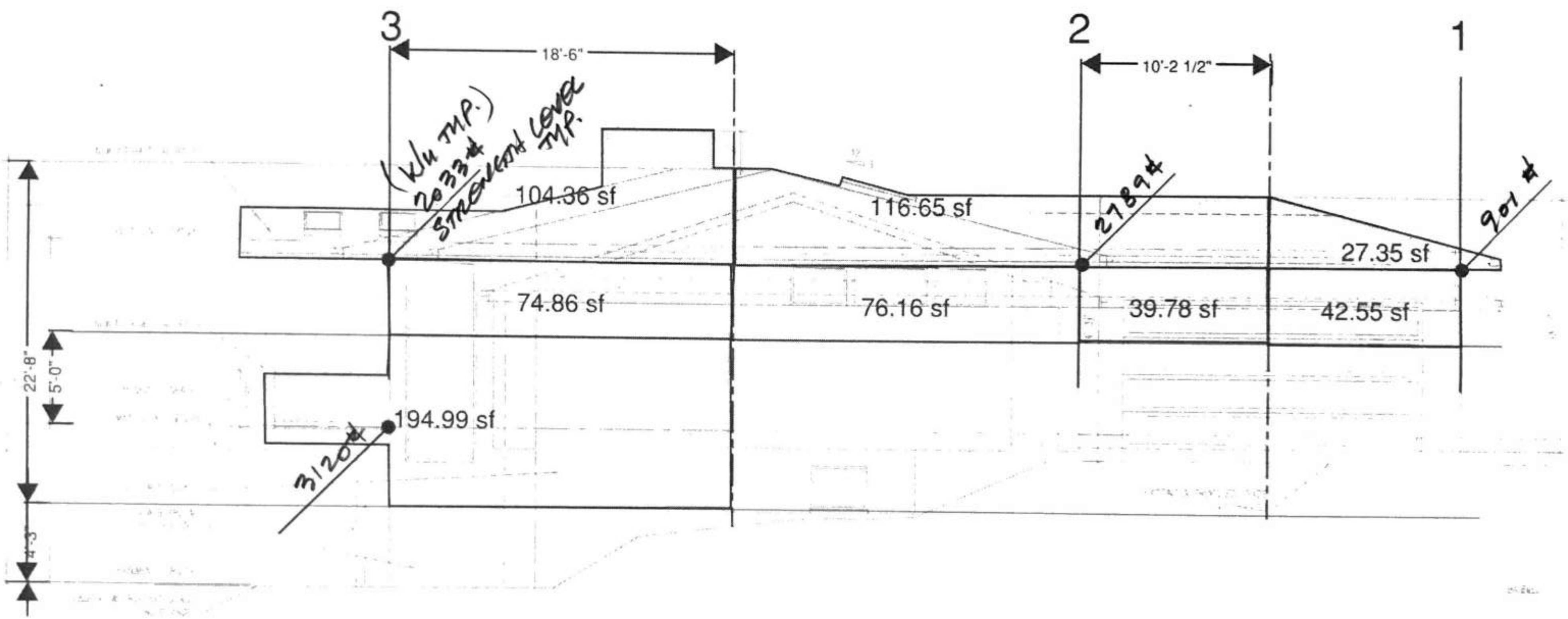
WIND INTO NORTH ELEVATION (WORST CASE WIND LOADS)



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LATERAL ANALYSIS

WIND INTO NORTH ELEVATION

3 ROOF

P_{S30} PRESSURES.

$22.3(74.9\text{ SF}) - 6.37(104.36) = 1005$

WALL-C ROOF-D

P_{S30} PRESSURES w/ $P_S = 0$ IN B & D ZONES — 28.5-1 NOTE 7

$22.3(74.9) = 1670$ #

P_{S30} A & C = 16 PSF ; B & D = 8 PSF — 28.5.4

$16(74.9) + 8(104.36) = \underline{2033}$ #. ← MIN. WIND CONTROLS DESIGN

C D

3 MAIN

$16(195) = \underline{3120}$ #

2 ROOF

$16(76.2 + 39.8) + 8(116.7) = \underline{2789}$ #

1 ROOF

$16(42.6) + 8(27.4) = \underline{901}$ #

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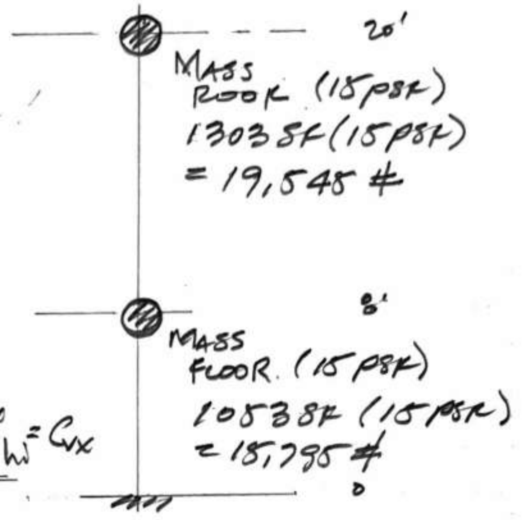
LATERAL ANALYSIS (CONT.)

SEISMIC - NORTH-SOUTH



3 - VERTICAL DISTRIBUTION OF SEISMIC FORCES.

$F_x = C_{vx} V$

$C_{vx} = \frac{W_x h_x}{\sum W_i h_i}$

	W_x	h_x	$W_x h_x$	$\frac{W_x h_x}{\sum W_i h_i} = C_{vx}$
ROOF	19,545	20'	390,900	.76
MAIN	15,795	8'	126,360	.24

$\sum W_x h_x = 517,260$

R=8 / R=6.5
SMF / WOOD FRAME SHEAR WALLS.

$C_s = \frac{.934}{6.5} = .143$ $V = .143(35,340\#) = 5053\#$

R=3½ / R=6.5
OMF / WOOD FRAMED SHEAR WALLS

$C_s = \frac{.934}{3.5} = .27$ $V = .27(35,340\#) = 9542\#$

$C_s = .143$

$F_{ROOF} = .76(5054\#) = 3841$

$F_{MAIN} = .24(5054\#) = 1213$

$C_s = .27$

$F_{ROOF} = .76(9542\#) = 7252\#$

$F_{MAIN} = .24(9542\#) = 2290\#$

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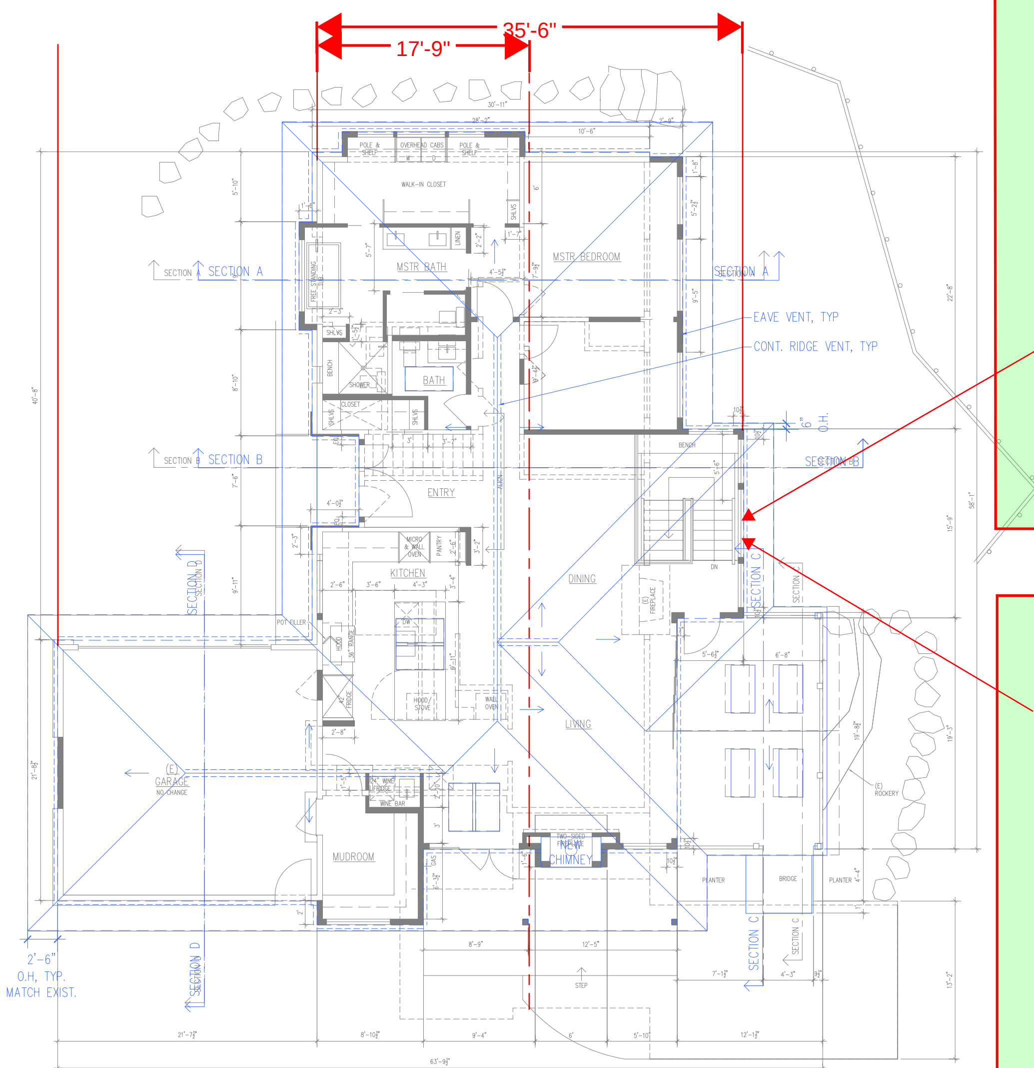
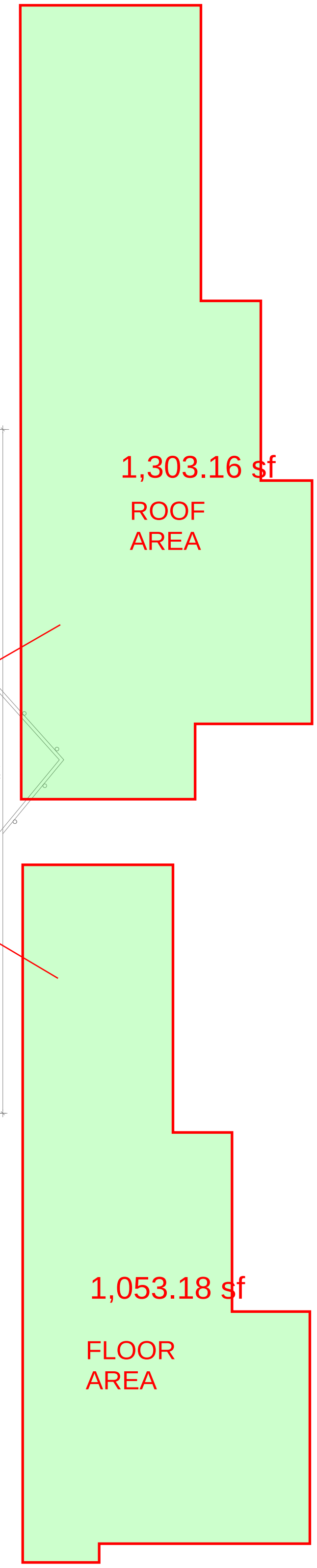


N_MAIN PLAN



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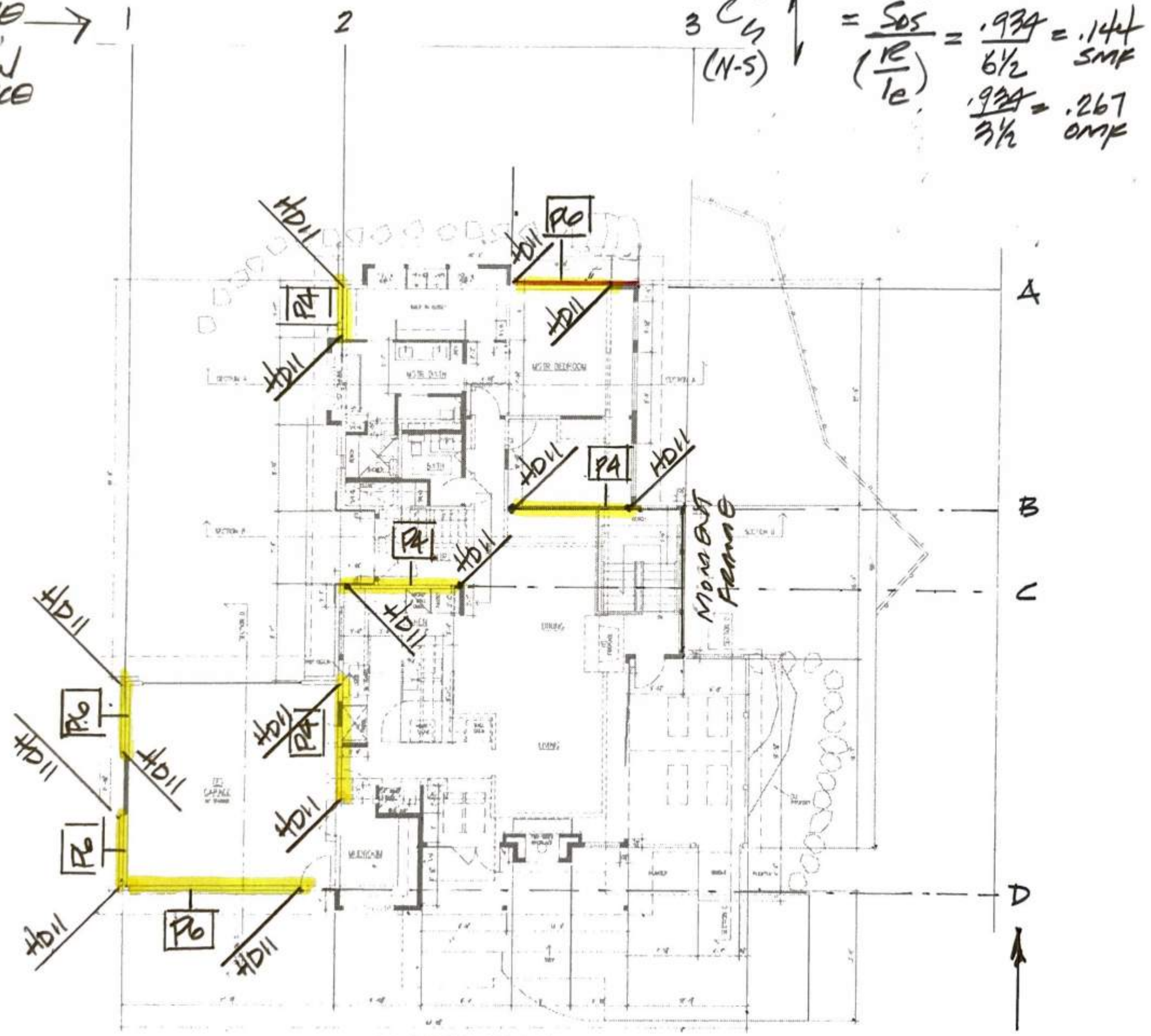
$$C_s \, (E\text{-}W) \leftrightarrow \; = \frac{S_{DS}}{\left(\frac{R}{I_e}\right)} = \frac{.934}{6\tfrac{1}{2}} = .144$$

$$3 \; C_s \, (N\text{-}S) \updownarrow \; = \frac{S_{DS}}{\left(\frac{R}{I_e}\right)} = \frac{.934}{6\tfrac{1}{2}} = .144 \text{ SMF}$$

$$\frac{.934}{3\tfrac{1}{2}} = .267 \text{ OMF}$$

GRIDS ARE FOR CALCULATION REFERENCE ONLY.



N_MAIN PLAN

GRIDS ARE FOR CALCULATION REFERENCE ONLY.

NOTE
SHEARWALLS SHOWN @ ROOF LEVEL
HARDWARE SHOWN @ MAIN FLOOR FRAMING PLAN

N

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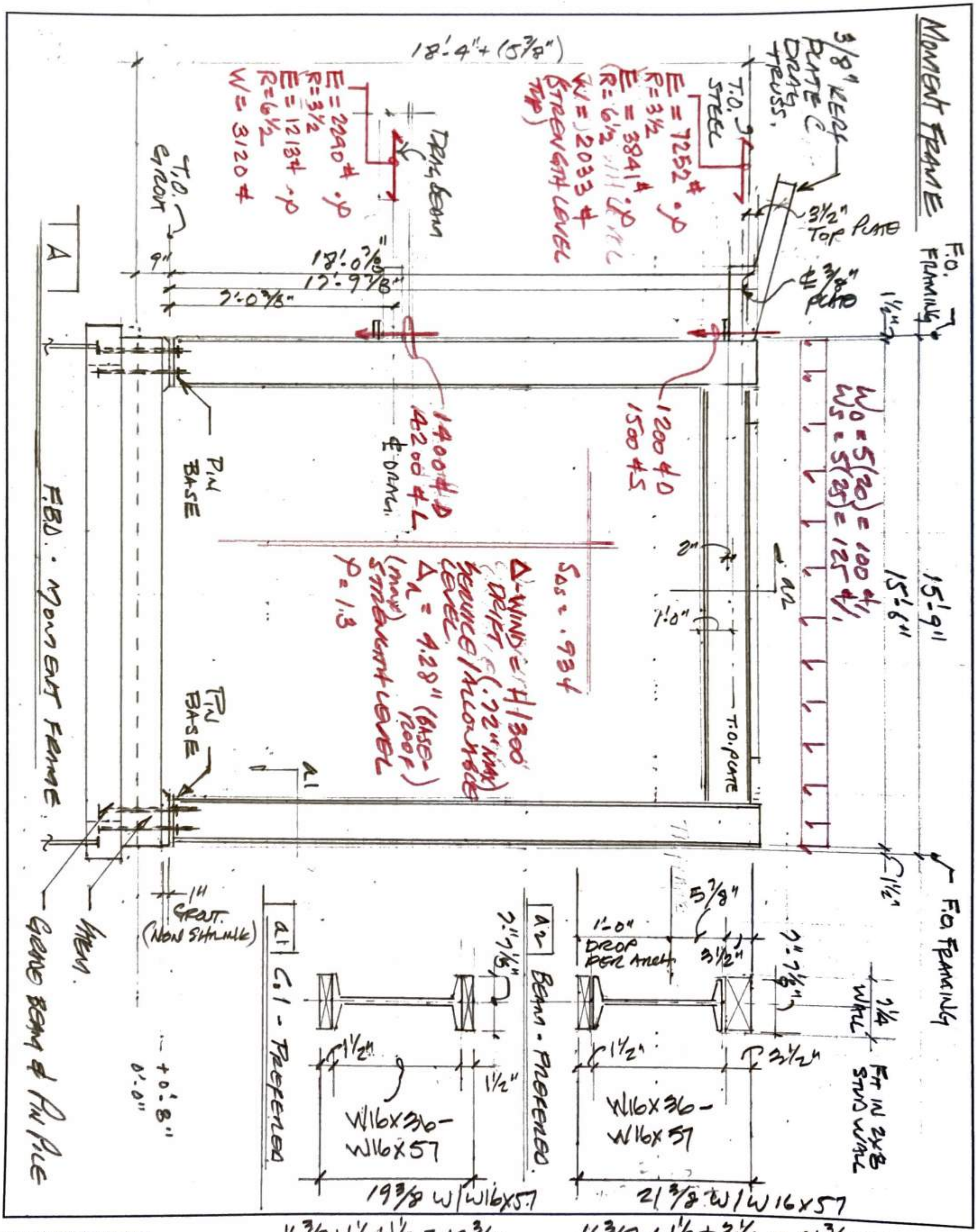

$16\frac{3}{8} + 1\frac{1}{2} + 1\frac{1}{2} = 19\frac{3}{8}$ $16\frac{3}{8} + 1\frac{1}{2} + 3\frac{1}{2} = 21\frac{3}{8}$ MAX

SIMPSON Strong-Tie

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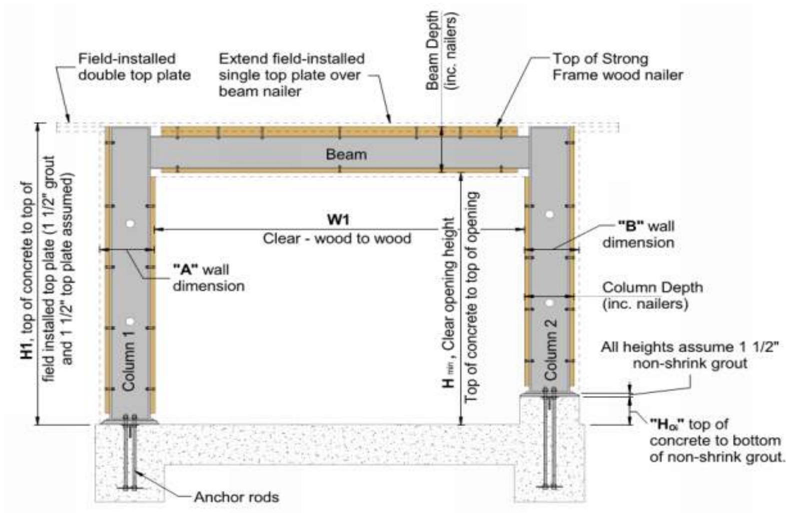
SIMPSON STRONG-FRAME®
SPECIAL MOMENT FRAME
DESIGN CALCULATION PACKAGE



Frame Model: Custom 1-story x 1-bay SMF Frame (SMF-1X)

Frame ID: MF-1
Project Location: 8429 SE 33rd PL, Mercer Island, WA, 98040
Engineer of Record:

Beam Size:	W16X45		
Column Size:	W16X45		
Link Size:	MF4-3		
W1=	150.75 in	**HO1=**	0 in
H1=	223.375 in	**HO2=**	0 in

Package Contents:

- **Design Summary**
- **SAP2000 Model Input**
- **SAP2000 beam and column design**
- **Moment Connection Design**
- **Column Base Plate Design**
- **Nailer Attachment Design**

Design Date: 3/3/2022
Design by: T.Truong
Job No.: ES-221086

Note:	**These calculations are only applicable to frames designed and manufactured by Simpson Strong-Tie Company Inc. Any other use of these calculations, or portion thereof, for purposes other than specifying frames manufactured by Simpson Strong-Tie is expressly prohibited.**

Note:	**Signed & Sealed Calculations will be provided to the Building Department as a deferred submittal upon request. Signed & Sealed Calculations will not be issued until completion of a frame order from Simpson Strong-Tie Company Inc. Stamped but Unsigned calculations are provided for the plancheck review process. Stamped and Signed Calculations are required to be on file with the building department for final permitting.**

Note:	**Simpson Strong-Tie® Strong Frame® and the Yield-Link™ structural fuse are protected under US patent number 8,001,734 B2 and must be supplied or licensed through Simpson Strong-Tie.**

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Simpson Strong-Tie
Special Moment Frame Design Summary

Frame Model: **Custom 1-story x 1-bay SMF Frame (SMF-1X)**

Design Codes/References Used:

2018 International Building Code
AISC Steel Construction Manual - 14th Edition
Simpson Strong Tie Design Procedure for SST SMF

Material Design Properties:

Beam/Column Steel:	**ASTM A992**	**F_y = 50 ksi**
Plate Steel:	**ASTM A572 Grade 50**	**F_y = 50 ksi**
		(Link, Baseplate, cap plate, stiffener and shear plate)

High Strength Bolts:

Link-Stem-to beam:	**ASTM F2280 Twisted off bolts (ASTM A449 Equivalent), Pre-tensioned**
Link-flange-to column:	**ASTM A325 (Snug-tight)**
BRP Bolt to beam:	**ASTM A325 (Snug-tight)**
Shear plate-to-column:	**ASTM A325 (Snug-tight)**
Anchor Bolts:	**ASTM F1554 Gr 36, ASTM A36, or ASTM A449 (See Base Plate design)**
Weld Electrodes:	**E70XX**

Notes:

(1) Simpson Strong-Tie Strong Frame Moment Frames are designed using Load and Resistance Factored design (LRFD) methodology for determining frame drift and strength limits. Allowable Stress Design (ASD) shear and drift are determined as V(ASD) = 0.7 X V(LRFD) and Drift(ASD) = 0.7 X Drift(LRFD) for seismic load combinations and V(ASD) = V(LRFD) / 1.6 for wind load combinations.

(2) The following calculations are provided for justifying that Simpson Strong-Tie Strong Frame Moment Frame component capacities meet or exceed the design forces and criteria provided & determined by the Designer. Simpson Strong-Tie has not confirmed and is not responsible for any of the design, engineering, calculations, or derivation of the structural forces or drifts related to the building and the designed elements. Moment frames and other lateral resisting elements placed in the same lateral resisting shear line shall have applied lateral loads proportioned based on relative stiffness.

(3) This Simpson Strong-Tie Strong Frame Moment Frame is part of the overall lateral force resisting system of the structure. Design of the building's lateral force resisting system, including the load path to transfer lateral forces from the structure to the ground, is the responsibility of the designer. The designer must specify the required components of the complete load transfer path including diaphragms, shear transfer, chords and collectors and foundations.

(4) Footing dimensions provided are the minimums required for concrete anchorage requirements only. The designer must determine required footing size and reinforcing for other design limits, such as foundation shear and bending, soil bearing shear transfer, and frame stability / overturning. Designer shall detail actual footing / grade beam size and reinforcing.

(5) Holes in base plates are over-sized for erection tolerance. Designer must evaluate effects of over-sized holes and provide plate washer with standard-size holes welded to base plate or request base plates with standard-size holes where required.

(6) Refer to Strong Frame installation detail sheets for allowable field modifications and additions. Welding shall be in accordance with AWS D1.1 and AWS D1.8 (as applicable for seismic). Welds shall be specified by the designer. Provide welding special inspection as required by the local building department.

COMMENTS:

1. Project Information:

Job Name:	8429 SE 33rd PL	Date:	3/3/2022
Project Address:	8429 SE 33rd PL, Mercer Island, WA, 98040	Engineer:	
Frame ID:	MF-1	Phone:	
SST Enginer:	**T.Truong**	E-mail:	

2. Frame Geometry: / Design Criteria:

Frame ID: **SMFX-16z16-151.75x220.375-(MF4-3)**

	ft	in
Clear Opening Width(W1)=	**12 ft**	**6.75 in**
Outside Width(W2)=	**15 ft**	**9.00 in**
Top Plate Height(H1)=	**18 ft**	**7.38 in**
Bottom Nailer Height(H1_min)=	**0 ft**	**0.00 in**
Foundation offset (HO1)=	**0 ft**	**0.00 in**
Foundation offset (HO2)=	**0 ft**	**0.00 in**
A=	**24.00 in**	OK
B=	**24.00 in**	OK

Base Fixity: **Pinned**

Custom Sizes:	
Beam=	**W16X45**
Column=	**W16X45**
Link ID=	**MF4-3**
Shear Tab	**Standard**

Bm Deflection Limits (Simply Supported):

SS_beam Limit=	**L/ 360**
Wind Drift Limit:	hx/ 300

Design Code=	**2018 IBC**
S_{DS}=	**0.934**
R_load=	**6.5**
R_frame=	**6.5**
Cd=	4.0
Omega=	**2.5**
I=	**1**
Rho=	**1.3**
Seismic Drift Limit=	**0.025 Hx**

Beam Deflection Limits:

Live Load=	**L/ 360**
Dead + Live Load=	**L/ 240**
Snow/Wind Load=	**L/ 360**

3. Loading:

3.1 Uniform Loads: (Negative value for uplift)

		Left End (ft)	RightEnd (ft)	To Rcc?
W_{DL1}=	100 plf	0.00	14.15	**Y**
W_{DL2}=	0 plf	0.00	0.00	**Y**
W_{DL3}=	0 plf	0.00	0.00	**Y**
Rain Load=	0 plf	0.00	0.00	**Y**
Wind Uplift=	0 plf	0.00	0.00	**Y**

		Left End (ft)	RightEnd (ft)	To Rcc?
W_{LR1}=	0 plf	0.00	0.00	**Y**
W_{LR2}=	0 plf	0.00	0.00	**Y**
W_{LL1}=	0 plf	0.00	0.00	**Y**
W_{LL2}=	0 plf	0.00	0.00	**Y**
Snow Load=	125 plf	0.00	14.15	**Y**
Snow Factor=	**0.2**			

3.2: Point Loads:

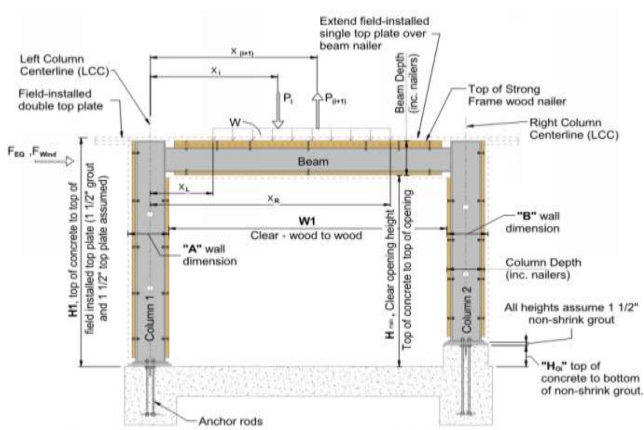
Lateral:							
V_{EQ}=	**2689**	lbs, ASD	Seismic Load	V_{WIND}=	**1220**	lbs, ASD	Wind Load
V_{EQ}=	3841	lbs, LRFD	Seismic Load	V_{WIND}=	2033	lbs, LRFD	Wind Load

Beam Point Loads ***(Negative value for uplift, wind and seismic, include Ω included as appropriate)***

	DL	LL	LR	Snow (S)	Rain (R)	Wind (W)	Seismic (Ev)		Distance	At Rcc?
P_1 (lbs)=	1200	0	0	1500	0	0	0	X_1=	0.00 ft	**N**
P_2 (lbs)=	0	0	0	0	0	0	0	X_2=	0.00 ft	**N**
P_3 (lbs)=	0	0	0	0	0	0	0	X_3=	0.00 ft	**N**
Column 1 Point Load										
P_4 (lbs)=	1400	4200	0	0	0	0	0	X_4=	7.50 ft	**N**

Column 1 Lateral Point Load (ASD)

	DL	LL	LR	Snow (S)	Rain (R)	Wind (W)	Seismic (Eh)			
P_5 (lbs)=	0	0	0	0	0	1872	849	X_5=	7.50 ft	**N**



Note: Beam to column connection not shown for clarity. Figure is for illustration purposes only.
Final locations and quantities of nailer attachments and beam or column web holes/ openings may vary.

1-STORY X 1-BAY SMF DESIGN SUMMARY:

		DCR Limit	Check
FRAME SEISMIC DRIFT DCR	0.595	1.03	OK
FRAME WIND DRIFT DCR	0.412	1.03	OK

BEAM DELECTION CHECK:

STEP 2: BEAM WITH SIMPLY SUPPORTED CONNECTIONS DEFLECTION DCR=	0.023	1.00	OK
LIVE LOAD DEFLECTION DCR=	0.002	1.00	OK
LIVE + DEAD LOAD DEFLECTION DCR=	0.014	1.00	OK
WIND DEFLECTION DCR=	0.049	1.00	OK
SNOW DEFLECTION DCR=	0.016	1.00	OK

LINK CHECK:

STEP 3: REQUIRED LINK YIELD STRENGTH/LINK AXIAL FORCE DEMAND CHECK	0.512	1.00	OK
STEP 5: LINK YIELDING STEM WIDTH AND THICKNESS	OK	-	OK
STEP 6: STEM STRAIN DEMAND CHECK/LINK STEM YIELDING LENGTH CHECK	0.840	1.00	OK
8.1: DESIGN LINK STEM TO BEAM FLANGE BOLT FOR SHEAR TRANSFER	0.954	1.00	OK
8.4a: CHECK LINK-STEM FOR BOLT BEARING	0.970	1.00	OK
8.4a: CHECK BEAM FLANGE FOR BOLT BEARING	0.858	1.00	OK
8.4b: CHECK LINK-STEM FOR TENSILE YIELDING AND TENSILE RUPTURE	0.889	1.00	OK
8.4c: CHECK LINK-STEM FOR BLOCK SHEAR	0.755	1.00	OK
9.1: DETERMINE LINK FLANGE TO COLUMN FLANGE BOLT DIAMETER	0.721	1.00	OK
9.2: DETERMINE LINK FLANGE THICKNESS	0.824	1.00	OK
9.3: CHECK LINK FLANGE THICKNESS FOR SHEAR DUE TO BOLT TENSION	0.508	1.00	OK
11.1 CHECK FRAME DRIFT WITH PR CONNECTIONS	0.595	1.03	OK

COLUMN CHECK:

LEFT COLUMN MAX DCR	0.572	1.00	OK
RIGHT COLUMN MAX DCR	0.572	1.00	OK
STEP 14: STRONG COLUMN WEAK LINK CHECK	0.563	1.00	OK
STEP 16: COLUMN PANEL ZONE CHECK	0.780	1.04	OK
18.1a CONTINUITY PLATE REQUIREMENTS BASED ON GEOMETRY AND TENSION YIELDING	OK	-	OK
18.1b FULL DEPTH STIFFENER PLATE FOR 2-SIDED MOMENT CONNECTIONS ONLY	0.000	1.00	OK
18.2a STIFFENER PLATE DOUBLE SIDE FILLET WELD TO COLUMN FLANGE	0.718	1.00	OK
18.2b STIFFENER PLATE DOUBLE SIDE FILLET WELD TO COLUMN WEB	0.482	1.00	OK
18.3a CONNECTION AWAY FROM COLUMN ENDS (SST Step 18 Table 1.1)	0.845	1.00	OK
18.3b CONNECTION AT STIFFEND COLUMN END (SST STEP 18, TABLE 1.2, CASE 1)	0.904	1.00	OK

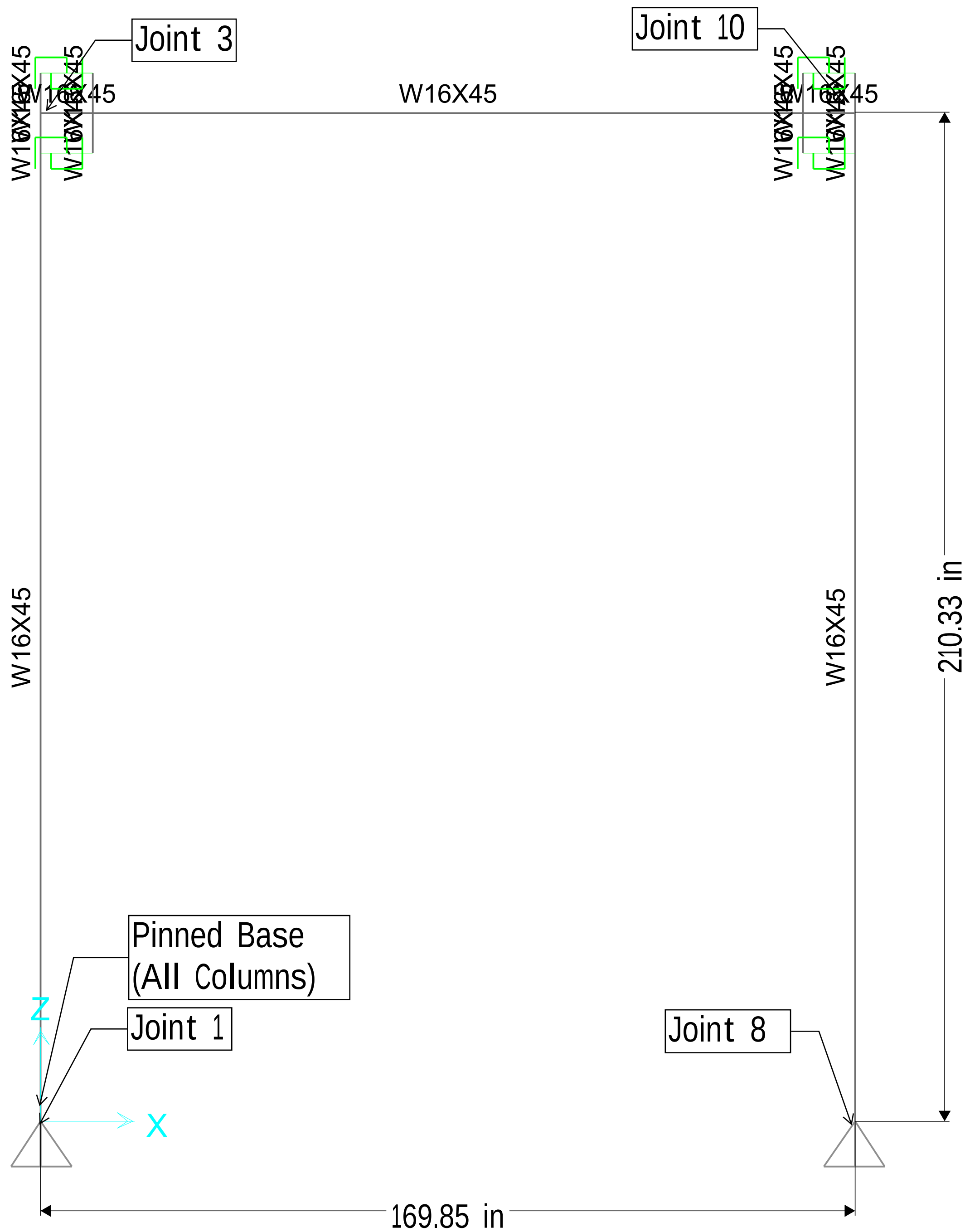
BEAM CHECK:

SIMPLY SUPPORTED BEAM MAX STRENGTH DCR	0.027	1.00	OK
PR FRAME BEAM MAX DCR	0.622	1.00	OK
BEAM FLANGE CHECK	OK	-	OK
15.1a: SHEAR PLATE BOLT SIZE	0.341	1.00	OK
SHEAR PLATE GEOMETRY CHECK	OK	-	OK
15.3a: SHEAR PLATE YIELDING	0.301	1.00	OK
15.3b: SHEAR PLATE RUPTURE	0.417	1.00	OK
15.3c: SHEAR PLATE CHECK FOR AXIAL AND MOMENT	0.331	1.00	OK
15.4: SHEAR PLATE TO COLUMN FLANGE FILLET WELD	0.938	1.00	OK
15.5: BEAM WEB BEARING X-DIR.	0.197	1.00	OK
15.5: SHEAR TAB BEARING X-DIR.	0.181	1.00	OK
15.5: BEAM WEB BEARING Y-DIR.	0.259	1.00	OK
15.5: SHEAR TAB BEARING Y-DIR.	0.243	1.00	OK
15.5: BEAM WEB BEARING X+Y	0.296	1.00	OK
15.5: SHEAR TAB BEARING X+Y	0.273	1.00	OK
15.5b: BEAM WEB BLOCKSHEAR X-DIR.	0.187	1.00	OK
15.5b: SHEAR TAB BLOCKSHEAR X-DIR,	0.172	1.00	OK
15.5b: BEAM WEB BLOCKSHEAR Y-DIR.	0.302	1.00	OK
15.5b: SHEAR TAB BLOCKSHEAR Y-DIR,	0.295	1.00	OK
15.5b: BEAM WEB BLOCKSHEAR X+Y	0.193	1.00	OK
15.5b: SHEAR TAB BLOCKSHEAR X+Y	0.177	1.00	OK
15.6: BEAM DEPTH CLEARANCE CHECK	OK	-	OK

Code= 2018 IBC
Sds= 0.934
Ω= 2.5 User input (2.5 or 3)
f_2= 0.2 Snow load factor
ρ= 1.3

Combo	Load Combinations	Load Combination Multipliers									
		D	L	Lr	S	R	W	E	Ev	Ni	
1	D	1	0	0	0	0	0	0	0	0	Load Cases
2	L	0	1	0	0	0	0	0	0	0	
3	Lr	0	0	1	0	0	0	0	0	0	
4	S	0	0	0	1	0	0	0	0	0	
5	R	0	0	0	0	1	0	0	0	0	
6	W	0	0	0	0	0	1	0	0	0	
7	E	0	0	0	0	0	0	1	1	0	
8	D	1	0	0	0	0	0	0	0	1	ASD
9	D+ L	1	1	0	0	0	0	0	0	1	
10	D + Lr	1	0	1	0	0	0	0	0	1	
11	D + S	1	0	0	1	0	0	0	0	1	
12	D + R	1	0	0	0	1	0	0	0	1	
13	D + 0.75 L+ 0.75 Lr	1	0.75	0.75	0	0	0	0	0	1	
14	D + 0.75 L+ 0.75 S	1	0.75	0	0.75	0	0	0	0	1	
15	D + 0.75 L+ 0.75 R	1	0.75	0	0	0.75	0	0	0	1	
16	D + 0.6 W	1	0	0	0	0	0.6	0	0	0	
17	D - 0.6 W	1	*0*	*0*	*0*	*0*	-0.6	0	0	0	
18	(1.0 + 0.14 S_{DS}) D + 0.7 E *ρ	1.13076	0	0	0	0	0	0.91	0.9100	0	
19	(1.0 + 0.14 S_{DS}) D - 0.7 E * ρ	1.13076	0	0	0	0	0	-0.91	-0.910	0	
20	D + 0.45 W + 0.75 L+ 0.75 Lr	1	0.75	0.75	0	0	0.45	0	0	0	
21	D - 0.45 W + 0.75 L+ 0.75 Lr	1	0.75	0.75	0	0	-0.45	0	0	0	
22	D + 0.45 W + 0.75 L+ 0.75 S	1	0.75	0	0.75	0	0.45	0	0	0	
23	D - 0.45 W + 0.75 L+ 0.75 S	1	0.75	0	0.75	0	-0.45	0	0	0	
24	D + 0.45 W + 0.75 L+ 0.75 R	1	0.75	0	0	0.75	0.45	0	0	0	
25	D - 0.45 W + 0.75 L+ 0.75 R	1	0.75	0	0	0.75	-0.45	0	0	0	
26	(1.0 + 0.105 S_{DS}) D + 0.525 E*ρ+ 0.75 L + 0.75 Lr	1.09807	0.75	0.75	0	0	0	0.683	0.683	0	
27	(1.0 + 0.105 S_{DS}) D - 0.525 E*ρ+ 0.75 L + 0.75 Lr	*1.09807*	0.75	0.75	0	0	0	-0.683	-0.683	0	
28	(1.0 + 0.105 S_{DS}) D + 0.525 E*ρ + 0.75 L + 0.75 S	1.09807	0.75	0	0.75	0	0	0.683	0.683	0	
29	(1.0 + 0.105 S_{DS}) D - 0.525 E*ρ + 0.75 L + 0.75 S	*1.09807*	0.75	0	0.75	0	0	-0.683	-0.683	0	
30	(1.0 + 0.105 S_{DS}) D + 0.525 E*ρ + 0.75 L + 0.75 R	1.09807	0.75	0	0	0.75	0	0.683	0.683	0	
31	(1.0 + 0.105 S_{DS}) D - 0.525 E*ρ + 0.75 L + 0.75 R	*1.09807*	0.75	0	0	0.75	0	-0.683	-0.683	0	
32	0.6 D + 0.6 W	0.6	0	0	0	0	0.6	0	0	0	
33	0.6 D - 0.6 W	0.6	0	0	0	0	-0.6	0	0	0	
34	(0.6 - 0.14 S_{DS}) D + 0.7 E*ρ	0.46924	0	0	0	0	0	0.91	0.9100	0	
35	(0.6 - 0.14 S_{DS}) D - 0.7 E*ρ	0.46924	0	0	0	0	0	-0.91	-0.910	0	
36	1.4 D	1.4	0	0	0	0	0	0	0	1	LRFD
37	1.2 D + 1.6 L + 0.5 Lr	1.2	1.6	0.5	0	0	0	0	0	1	
38	1.2 D + 1.6 L + 0.5 S	1.2	1.6	0	0.5	0	0	0	0	1	
39	1.2 D + 1.6 L + 0.5 R	1.2	1.6	0	0	0.5	0	0	0	1	
40	1.2 D + 1.6 Lr + 0.5 L	1.2	0.5	1.6	0	0	0	0	0	1	
41	1.2 D + 1.6 Lr + 0.5 W	1.2	0	1.6	0	0	0.5	0	0	0	
42	1.2 D + 1.6 Lr - 0.5 W	*1.2*	*0*	*1.6*	*0*	*0*	-0.5	*0*	*0*	*0*	
43	1.2 D + 1.6 S + 0.5 L	1.2	0.5	0	1.6	0	0	0	0	1	
44	1.2 D + 1.6 S + 0.5 W	1.2	0	0	1.6	0	0.5	0	0	0	
45	1.2 D + 1.6 S - 0.5 W	*1.2*	*0*	*0*	*1.6*	*0*	-0.5	*0*	*0*	*0*	
46	1.2 D + 1.6 R + 0.5 L	1.2	0.5	0	0	1.6	0	0	0	1	
47	1.2 D + 1.6 R + 0.5 W	1.2	0	0	0	1.6	0.5	0	0	0	
48	1.2 D + 1.6 R - 0.5 W	*1.2*	*0*	*0*	*0*	*1.6*	-0.5	*0*	*0*	*0*	
49	1.2 D + 1 W + 0.5 L + 0.5 Lr	1.2	0.5	0.5	0	0	1	0	0	0	
50	1.2 D - 1 W + 0.5 L + 0.5 Lr	*1.2*	*0.5*	*0.5*	*0*	*0*	-1	*0*	*0*	*0*	
51	1.2 D + 1 W + 0.5 L + 0.5 S	1.2	0.5	0	0.5	0	1	0	0	0	
52	1.2 D - 1 W + 0.5 L + 0.5 S	*1.2*	*0.5*	*0*	*0.5*	*0*	-1	*0*	*0*	*0*	
53	1.2 D + 1 W + 0.5 L + 0.5 R	1.2	0.5	0	0	0.5	1	0	0	0	
54	1.2 D - 1 W + 0.5 L + 0.5 R	*1.2*	*0.5*	*0*	*0*	*0.5*	-1	*0*	*0*	*0*	
55	(1.2 + 0.2 S_{DS})D + E*ρ + 0.5 L + f2*S	1.3868	0.5	0	0.2	0	0	1.3	1.3000	0	
56	(1.2 + 0.2 S_{DS})D - E*ρ + 0.5 L + f2* S	1.3868	0.5	0	0.2	0	0	-1.3	-1.300	0	
57	0.9 D + 1 W	0.9	0	0	0	0	1	0	0	0	
58	0.9 D - 1 W	0.9	0	0	0	0	-1	0	0	0	
59	(0.9 - 0.2 S_{DS}) D + E*ρ	0.7132	0	0	0	0	0	1.3	1.3000	0	
60	(0.9 - 0.2 S_{DS}) D - E*ρ	0.7132	0	0	0	0	0	-1.3	-1.300	0	
61	(1.2 + 0.2 S_{DS})D + ΩE + 0.5L + f2*S	1.3868	0.5	0	0.2	0	0	2.5	2.5	0	
62	(1.2 + 0.2 S_{DS})D -ΩE + 0.5L + f2*S	1.3868	0.5	0	0.2	0	0	-2.5	-2.5	0	
63	(0.9 - 0.2 S_{DS}) D + ΩE	0.7132	0	0	0	0	0	2.5	2.5	0	
64	(0.9 - 0.2 S_{DS}) D - ΩE	0.7132	0	0	0	0	0	-2.5	-2.5	0	

Frame Input


Joint 3
Joint 10
W16X45
W16X45
W16X45
W16X45
W16X45
W16X45
W16X45
W16X45
W16X45
W16X45
W16X45
210.33 in
Pinned Base
(All Columns)
Joint 1
Joint 8
Z
X
169.85 in

Joint Displacement (in)											
Joint	Combo	U1 (X)	U2 (Y)	U3 (Z)	Joint	Combo	U1 (X)	U2 (Y)	U3 (Z)	Seismic	Wind
3	LCM1	-0.001	0.000	-0.002	10	LCM1	-0.001	0.000	-0.001		
3	LCM2	-0.001	0.000	-0.001	10	LCM2	-0.001	0.000	0.000		
3	LCM3	0.000	0.000	0.000	10	LCM3	0.000	0.000	0.000		
3	LCM4	-0.001	0.000	-0.001	10	LCM4	-0.001	0.000	0.000		
3	LCM5	0.000	0.000	0.000	10	LCM5	0.000	0.000	0.000		
3	LCM6	0.676	0.000	0.002	10	LCM6	0.676	0.000	-0.002		
3	LCM7	0.828	0.000	0.003	10	LCM7	0.828	0.000	-0.003		
3	LCM8	0.034	0.000	-0.002	10	LCM8	0.034	0.000	-0.001		
3	LCM9	0.033	0.000	-0.003	10	LCM9	0.033	0.000	-0.001		
3	LCM10	0.034	0.000	-0.002	10	LCM10	0.034	0.000	-0.001		
3	LCM11	0.033	0.000	-0.003	10	LCM11	0.033	0.000	-0.001		
3	LCM12	0.034	0.000	-0.002	10	LCM12	0.034	0.000	-0.001		
3	LCM13	0.033	0.000	-0.002	10	LCM13	0.033	0.000	-0.001		
3	LCM14	0.032	0.000	-0.003	10	LCM14	0.032	0.000	-0.001		
3	LCM15	0.033	0.000	-0.002	10	LCM15	0.033	0.000	-0.001		
3	LCM16	0.404	0.000	0.000	10	LCM16	0.404	0.000	-0.002		
3	LCM17	-0.407	0.000	-0.003	10	LCM17	-0.407	0.000	0.001		
3	LCM18	0.752	0.000	0.001	10	LCM18	0.752	0.000	-0.004		
3	LCM19	-0.755	0.000	-0.005	10	LCM19	-0.755	0.000	0.002		
3	LCM20	0.302	0.000	-0.002	10	LCM20	0.302	0.000	-0.002		0.302
3	LCM21	-0.306	0.000	-0.004	10	LCM21	-0.306	0.000	0.000		-0.306
3	LCM22	0.301	0.000	-0.002	10	LCM22	0.301	0.000	-0.002		0.301
3	LCM23	-0.307	0.000	-0.005	10	LCM23	-0.307	0.000	0.000		-0.307
3	LCM24	0.302	0.000	-0.002	10	LCM24	0.302	0.000	-0.002		0.302
3	LCM25	-0.306	0.000	-0.004	10	LCM25	-0.306	0.000	0.000		-0.306
3	LCM26	0.563	0.000	-0.001	10	LCM26	0.563	0.000	-0.003		
3	LCM27	-0.568	0.000	-0.005	10	LCM27	-0.568	0.000	0.001		
3	LCM28	0.562	0.000	-0.002	10	LCM28	0.562	0.000	-0.003		
3	LCM29	-0.568	0.000	-0.006	10	LCM29	-0.568	0.000	0.001		
3	LCM30	0.563	0.000	-0.001	10	LCM30	0.563	0.000	-0.003		
3	LCM31	-0.568	0.000	-0.005	10	LCM31	-0.568	0.000	0.001		
3	LCM32	0.405	0.000	0.000	10	LCM32	0.405	0.000	-0.002		
3	LCM33	-0.406	0.000	-0.002	10	LCM33	-0.406	0.000	0.001		
3	LCM34	0.753	0.000	0.002	10	LCM34	0.753	0.000	-0.003		
3	LCM35	-0.754	0.000	-0.004	10	LCM35	-0.754	0.000	0.002		
3	LCM36	0.034	0.000	-0.002	10	LCM36	0.034	0.000	-0.001		
3	LCM37	0.032	0.000	-0.004	10	LCM37	0.032	0.000	-0.001		
3	LCM38	0.031	0.000	-0.004	10	LCM38	0.031	0.000	-0.001		
3	LCM39	0.032	0.000	-0.004	10	LCM39	0.032	0.000	-0.001		
3	LCM40	0.033	0.000	-0.003	10	LCM40	0.033	0.000	-0.001		
3	LCM41	0.337	0.000	-0.001	10	LCM41	0.336	0.000	-0.002		
3	LCM42	-0.339	0.000	-0.003	10	LCM42	-0.339	0.000	0.000		
3	LCM43	0.032	0.000	-0.005	10	LCM43	0.032	0.000	-0.002		
3	LCM44	0.335	0.000	-0.003	10	LCM44	0.335	0.000	-0.003		
3	LCM45	-0.341	0.000	-0.005	10	LCM45	-0.341	0.000	-0.001		
3	LCM46	0.033	0.000	-0.003	10	LCM46	0.033	0.000	-0.001		
3	LCM47	0.337	0.000	-0.001	10	LCM47	0.336	0.000	-0.002		
3	LCM48	-0.339	0.000	-0.003	10	LCM48	-0.339	0.000	0.000		
3	LCM49	0.674	0.000	0.000	10	LCM49	0.674	0.000	-0.003		
3	LCM50	-0.678	0.000	-0.005	10	LCM50	-0.678	0.000	0.001		
3	LCM51	0.674	0.000	-0.001	10	LCM51	0.673	0.000	-0.004		
3	LCM52	-0.679	0.000	-0.006	10	LCM52	-0.678	0.000	0.001		
3	LCM53	0.674	0.000	0.000	10	LCM53	0.674	0.000	-0.003		
3	LCM54	-0.678	0.000	-0.005	10	LCM54	-0.678	0.000	0.001		
3	LCM55	1.074	0.000	0.001	10	LCM55	1.074	0.000	-0.005	1.074	
3	LCM56	-1.079	0.000	-0.007	10	LCM56	-1.079	0.000	0.003	-1.079	
3	LCM57	0.675	0.000	0.001	10	LCM57	0.675	0.000	-0.003		
3	LCM58	-0.677	0.000	-0.004	10	LCM58	-0.677	0.000	0.002		
3	LCM59	1.076	0.000	0.003	10	LCM59	1.076	0.000	-0.004	1.076	
3	LCM60	-1.078	0.000	-0.005	10	LCM60	-1.078	0.000	0.003	-1.078	
3	LCM61	2.068	0.000	0.004	10	LCM61	2.068	0.000	-0.009		
3	LCM62	-2.073	0.000	-0.011	10	LCM62	-2.073	0.000	0.006		
3	LCM63	2.070	0.000	0.006	10	LCM63	2.070	0.000	-0.008		
3	LCM64	-2.072	0.000	-0.009	10	LCM64	-2.072	0.000	0.007		

STEP 1: Beam and Column Sizes

Note: Slightly Modified from Design Procedure, uses PR connection instead of FR connection as a start (initial condition)

Beam Size:	W16X45		
d=	16.1	in	*Beam depth (looked up value)*
bbf=	7.04	in	*Beam flange width (looked up value)*
tbf=	0.565	in	*Beam flange thickness (looked up value)*
tbw=	0.345	in	*Beam web thickness (looked up value)*
Fy_bm=	50	ksi	*Beam Yield stress (previously define value)*
Fu_bm=	65	ksi	*Beam Rupture stress (previously define value)*
Column Size:	W16X45		
dc=	16.1	in	*Column depth (looked up value)*
bcf=	7.04	in	*Column flange width (looked up value)*
tcf=	0.565	in	*Column flange thickness (looked up value)*
tcw=	0.345	in	*Column web thickness (looked up value)*
Agc=	13.3	in^2	*Column gross section (looked up value)*
Fy_col=	50	ksi	*Column Yield stress (previously defined value)*
Fu_col=	65	ksi	*Column Rupture stress (previously defined value)*

Seismic Drift Check:

H1_aver.=	223.375	in	*Average Story heigth H1_aver. (previously defined value)*
R_frame=	6.5		*Response modification coefficient R (previously defined value)*
Cd=	4		*Deflection amplication factor [if(R_frame = 6.5, 4, 5.5)]*
Allowable Drift=	0.025	Hx	*Seismic drift limit (previously defined)*
I=	1		*Importance factor (previously defined value)*
Δ_{allow}=	1.396	in	*Allow delta s = (Allowable Drift* Hcc* I)/ Cd*
Δs_PR=	**0.830**	in	*PR model delta s (looked up from SAP_Drift)*
DCR_SeismicDrift=	0.595	OK	*$=\Delta s\_PR/\Delta_{allow}$*

Wind Drift Check:

Allowable Drift= Hx/	300		*Wind drift limit (previously defined)*
Allowable Drift=	0.745	in	*Allow wind drift = Hcc/ Wind drift limit*
Δ_PR=	0.307	in	*PR model delta s (looked up from SAP_Drift)*
DCR_WindDrift=	0.412	OK	*=Δs_PR/ Allow wind drift*

SAP2000 Axial Spring Parameters:

Axial Force vs. link displacement relationship

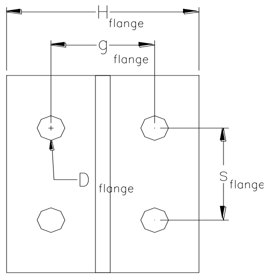
Points	**Force** kips	**Displ.** in
1	-117.000	-0.581
2	-117.000	-0.332
3	-82.500	-0.021
4	0.000	0.000
5	82.500	0.021
6	117.000	0.332
7	117.000	0.581

Force vs Link displacement (looked up from Steps 3-11)

STEP 2: Check Beam with Simply Supported Connections in SAP2000

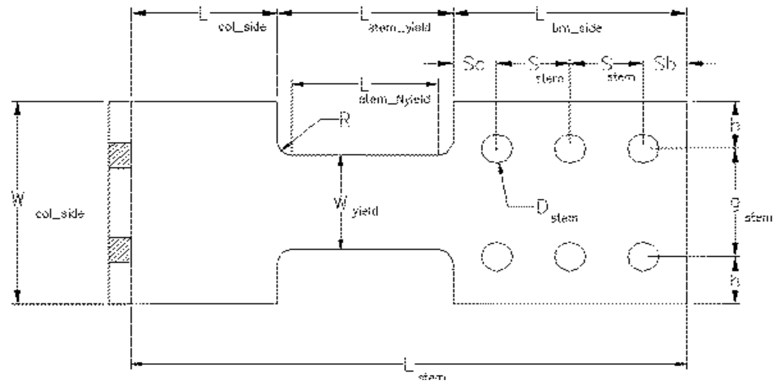
Lcc=	169.85	in	*Column center line to center line*
a=	2.75	in	*Distance from column face to center hole of beam*
Lh=	148.25	in	=Lcc - dc - 2*a
Delta_ss=	0.01	in	*SS beam deflection (looked up from SAP_BmDelta)*
Delta_allow=	0.412	in	*=Lh/ SS deflection limit*
DCR_Delta_ss=	0.023	OK	*=Delta_ss/ Delta_allow*

SMF Link Design/Check

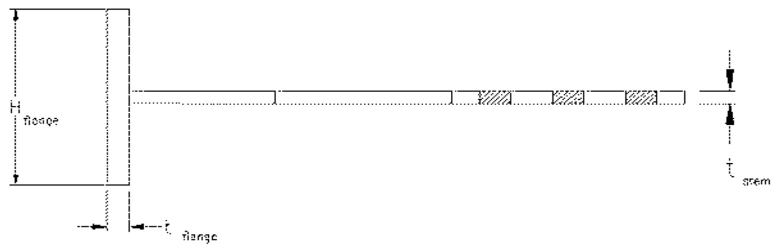


5.00 in | 7.00 in | 5.75 in

6.50 in | 3.50 in



5.75 in | 0.500 in



0.875 in

LINK ID: MF4-3

Beam= W16X45 db= 16.1 in

LINK STEM GEOMETRY:

NY Length ColSide (Lcol_side)=	5.00	in	Thickness (t_stem) =	0.50	in	
Yield Length, incl. fillets (L_stemYield) =	7.00	in	NY Width ColSide (Wcol_side)=	6.50	in	
NY Length BeamSide (Lbm_side)=	5.75	in	Central Neck Yield Width (w_stemYield)=	3.000	in	
L_stem=	17.75	in	NY Width BeamSide (Wbm_side)=	6.50	in	
Fillet Radius (r_fillet)=	0.50	in	Yielding Area (A_stemYield) =	1.500	in^2	*=t_stem *w_stemYield*

L_stem = Lcol_side + L_stemYeild + Lbm_side

LINK STEM BOLTS:

Num. Bolts (n_bolt_linkBm)=	4.00		Gauge Along Width (bolt_g_stem)=	3.50	in
Bolt Type (Bolt_Gr_linkBm)=	A490		Spacing Along Length (bolt_s_stem) =	2.75	in
Bolt Dia (boltD_linkBm)=	0.875	in	First Bolt distance to Neck (Sc)=	1.50	in
Min. Bolt length =	2.625	in	Last Bolt distance to Edge (Sb) =	1.50	in

LINK FLANGE GEOMETRY:

Thickness (t_flange)=	0.875	in
Flange Width (W_flange)=	6.50	in
Flange height (H_flange) =	5.75	in

LINK FLANGE BOLTS:

Num. Bolts (n_bolt_linkCol) =	4		Gauge Along Width (vertical) (bolt_g_flange)=	3.25	in
Bolt Type (Bolt_Gr_linkCol)=	A325		Spacing Along Length (horiz) (bolt_s_flange)=	3.50	in
Bolt Dia (boltD_linkCol)_=	0.875	in			
Min. Bolt length =	3.125	in			

LINK MATERIAL:

Fy_link =	50	ksi		
Fu_link =	65	ksi		
Material Overstrength Factor (Rt_link) =	1.2		Ry_link=	1.1

Top of Column Bracing Force

Per AISC 341-10, Section E3.4.c

A_link=	1.500	in^2	*Link Yielding Area*
Fy_link=	50	ksi	*Link Yield stress*
Ry=	1.1		*Ratio of expected yield to minimum yield*
Pye_link=	82.50	kips	*=A_link * Fy_llink*Ry*
0.02 * Pye_link=	1.65	kips	*Bracing Force (LRFD) at Top of column*
Pbrace (ASD)=	1.1550	kips	*=0.7*Bracing Force (LRFD) at Top of column*

STEP 15: DESIGN BEAM WEB TO COLUMN FLANGE CONNECTION

15.1a: SHEAR PLATE BOLT SIZE

Beam Section (bmSize) =	**W16X45**		*Value previously defined*
Beam Depth (db)=	16.10	in	*Value previously defined*
Fy_sp=	50	ksi	
Fu_sp =	65	ksi	
Fy_bm=	50	ksi	
Fu_bm =	65	ksi	
Axial Load (Pu_SST)=	5.09	kips	*Max axial force per LCM61-64 from SAP_BeamReactions, SAP output*
Vertical Load (Vu_bm)=	27.43	kips	*Value previously defined from Step 12*
Vu_bolt=	10.5	kips	*= Sqrt (Pu_SST^2 + (Vu_bm/ n_bolt_SST)^2)*
ϕbolt=	0.75		
Bolt Type (Bolt_Gr_shearTab) =	**A325-X**		*[A325-X default]*
Bolt Dia (db_sp)=	7/8	in	
No of Bolts (n_bolts_SST)=	3		
Fnv_A325N=	68	ksi	*= if (Bolt Type = "A490-X", 84, 68)*
Anb_sp=	0.60	in^2	*= Pi* (db_sp/2)^2*
Rn_stBolt_shear=	40.89	kips	*= Fnv_A325N* Anb_sb*
ϕRn_stBolt_Shear=	30.7	kips	*= Rn_stBolt_shear* ϕbolt*
DCR_shearTabl_Cbolt=	0.341	OK	*= Vu_bolt/ ϕRn_stBolt_shear*
			OK if DCR_shearTabl_Cbolt <= 1.05

15.2: SHEAR PLATE GEOMETRY

a=	2.75	in	*Value previously defined*
tsp=	**0.375**	in	*[0.375 default]*
Bolt Spacing (S_min)=	2.33	in	*= 2.6667* max (db_sp)*
Bolt Spacing (Svert)=	3.00	in	OK *looked up value*
Bolt Vert. Edge dist (Lv_min)=	1.125	in	*Minimum edge distance for bolts diameter per AISC Table J3.4*
Lv_sp=	1.5	in	OK
Plate Depth (h_sp)=	9	in	*= Lv_sp* 2 + Svert* 2*
Bolt Horz. Edge dist (Lh_min)=	1.125	in	*Minimum edge distance for bolts diameter per AISC Table J3.4*
Lh_sp=	1.75	in	OK
Plate Width (W_shearTab) =	4.5	in	*= Lh_sp + a*
Lslot_min=	1.42	in	*= db_sp + 1/8 + 0.14* Svert*
Lslot=	1.500	in	OK *OK if Lslot_min/ Lslot <= 1.03*
Shear Plate Geometry Check=	**OK**		*OK if And (Svert = OK, Lv_sp = OK, Lh_sp = OK, Lslot = OK)*

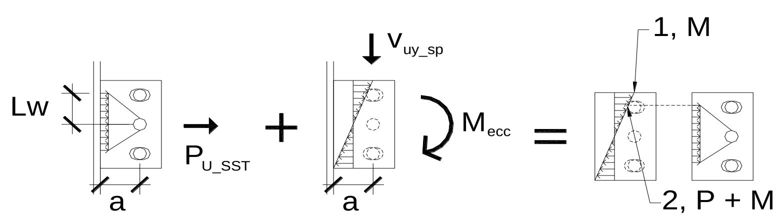
15.3a: SHEAR PLATE YIELDING

ϕyield=	0.9		
dHole_sp=	1.00	in	*= db_sp + 1/8*
Asp_Agv=	3.375	in^2	*= tsp* h_sp*
ϕVy_sp=	91.13	kips	*= ϕyield* 0.6* Asp_Agv* Fy_sp*
DCR_spYield=	0.301	OK	*= Vu_bm/ ϕVy_sp*
			OK if DCR_spYield <= DCR_allowed

15.3b: SHEAR PLATE RUPTURE

ϕrupture=	0.75		
Asp_nv=	2.25	in^2	*= hsp* tsp - dHole_sp* n_bolts_SST* tsp*
ϕVrupture_sp=	65.81	kips	*= ϕrupture* 0.6* Fu_shearTab* Asp_nv*
DCR_spRupture=	0.417	OK	*= Vu_bm/ ϕVrupture_sp*

15.3c: SHEAR PLATE CHECK FOR AXIAL AND MOMENT



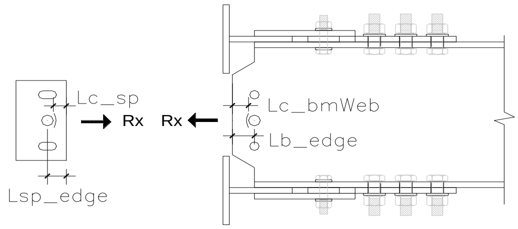
ϕb=	0.90		
a=	2.75	in	*Value previously defined*
Vuy=	27.43	kips	*= Vu_bm*
Mecc=	75.44	kips*in	*= Vuy* a*
θWhitmore=	30	deg	
Lwhitmore=	4.05	in	*= tan (θWhitmore)* a* 2 + db_sp*
Awhitmore=	1.52	in^2	*= Lwhitmore* tsp*
Ssp=	5.06	in^3	*= tsp* hsp^2/ 6*
Isp=	22.78	in^4	*= tsp* hsp^3/ 12*
fb1=	14.90	ksi	*= Mecc/ Ssp*
yb=	2.03	in	*= Lwhitmore/ 2*
fb2=	6.71	ksi	*= Mecc* yb/ Isp*
fa2=	3.35	ksi	*= Pu_SST/ Awhitmore*
ftot2=	10.06	ksi	*= fa2 + fb2*
fmax_sp=	14.90	ksi	*= max (fb1, ftot2)*
ϕb*Fy_sp=	45.00	ksi	*= ϕb* Fy_shearTab*
DCR_sp=	0.331	OK	*= fmax_sp/ (ϕb* Fy_shearTab), OK if DCR_sp <= DCR_Allowed*

15.4: SHEAR PLATE TO COLUMN FLANGE FILLET WELD

tsp=	0.375	in	*value previously defined*
tw_sp_min=	0.2344	in	*= 5/8*tsp Fillet size to develop plate, Per AISC Steel Manual 14th edition*
tw_sp=	0.2500	in	*Each Side [0.25 default] , page 10-102*
DCR_spWeld=	0.938	OK	*= tw_sp_min/ tw_sp, OK if DCR_spWeld <= DCR_Allowed*

15.5: BEAM WEB AND SHEAR TAB BEARING

CASE 1: HORIZONTAL REACTIONS



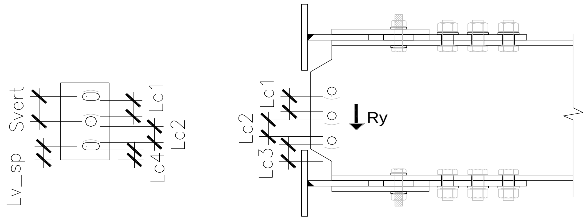
BEAM WEB:

ϕbolt=	0.75		
tbw=	0.345	in	*value previously defined*
Lb_edge=	1.75	in	*Edge distance for beam web bolt hole*
Lc_bmWeb=	1.281	in	*= Lb_edge - (db_sp + 1/16)* 0.5*
ϕRn_beamWeb1=	25.86	kips	*= ϕbolt* 1.2* Lc_bmWeb* tbw* Fu_bmWeb*
ϕRn_beamWeb2=	35.32	kips	*= ϕbolt* 2.4* db_sp* tbw* Fu_bmWeb*
ϕRn_beamWeb=	25.86	kips	*= min (ϕRn_beamWeb1, ϕRn_beamWeb2)*
DCR_bmWebX=	0.197	OK	*= Pu_SST/ ϕRn_beamWeb, OK if DCR_bmWebX <= DCR_Allowed*

SHEAR PLATE:

tsp=	0.375	in	*value previously defined*
Lsp_edge=	1.75	in	*Edge distance for shear tab bolt hole*
Lc_sp=	1.281	in	*= Lsp_edge - (db_sp + 1/16)* 0.5*
ϕRn_sp1=	28.11	kips	*= ϕbolt* 1.2* Lc_sp* tbw* Fu_bmWeb*
ϕRn_sp2=	38.39	kips	*= ϕbolt* 2.4* db_sp* tbw* Fu_bmWeb*
ϕRn_sp=	28.11	kips	*= min (ϕRn_sp1, ϕRn_sp2)*
DCR_spX=	0.181	OK	*= Pu_SST/ ϕRn_sp, OK if DCR_spX <= DCR_Allowed*

CASE 2: VERTICAL REACTIONS



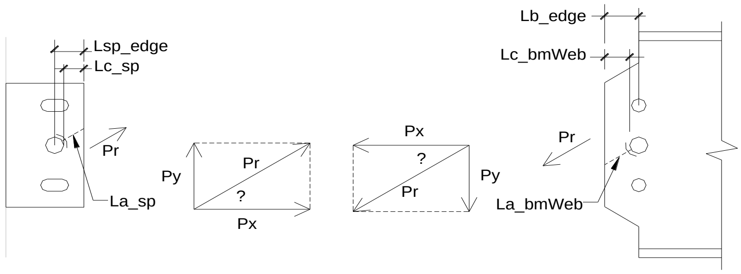
BEAM WEB:

Lc1=	2.063	in	*= Svert - (db_sp + 1/16)*
Lc2=	2.063	in	*= Lc1*
Lc3=	2.063	in	*Conservatively equal to Lc2*
ϕ*Rn_bmWebY1=	124.88	kips	*= ϕbolt* 1.2* (Lc1 + Lc2 + Lc3)* tbw* Fu_bmWeb*
ϕ*Rn_bmWebY2=	105.96	kips	*= ϕbolt* 2.4* (db_sp* n_bolts_SST)* tbw* Fu_bmWeb*
ϕ*Rn_bmWebY=	105.96	kips	*= min (ϕ*Rn_bmWebY1, ϕ*Rn_bmWebY2)*
DCR_bmWebY=	0.259	OK	*= Vu_bm/ (ϕ* Rn_bmWebY), OK if DCR_bmWebY <= DCR_Allowed*

SHEAR PLATE:

Lc4=	1.031	in	*= Lv_sp - (db_sp + 1/16)* 0.5*
ϕ*Rn_spY1=	113.12	kips	*= ϕbolt* 1.2* (Lc1 + Lc2 + Lc4)* tsp* Fu_shearTab*
ϕ*Rn_spY2=	115.17	kips	*= ϕbolt* 2.4* db_sp* n_bolts_SST* tsp* Fu_shearTab*
ϕ*Rn_spY=	113.12	kips	*= min (ϕ*Rn_spY1, ϕ*Rn_spY2)*
DCR_spY=	0.243	OK	*= Vu_bm/ (ϕ* Rn_spY), OK if DCR_spY <= DCR_Allowed*

CASE 3: COMBINED AXIAL AND VERTICAL REACTIONS



Px=	5.09	kips	*= Pu_SST*
Py=	9.14	kips	*= Vu_bm/ n_bolts_SST*
Pr=	10.46	kips	*= Sqrt (Px^2 + Py^2)*
θ=	1.063	radians	*= min(Atan (Py/ Px),1.571)*

BEAM WEB:

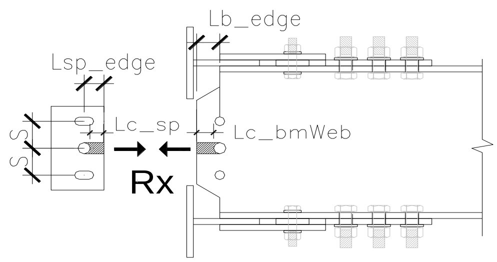
Lvg_web=	3.599	in	*= Lb_edge/ (cos θ)*
La_web=	3.162	in	*= Lvg_web - db_sp/2*
ϕ*Rn_bmWebθ1=	63.813	kips	*= ϕbolt* 1.2* La_web* tbw* Fu_bmWeb*
ϕ*Rn_bmWebθ2=	35.319	kips	*= ϕbolt* 2.4* db_sp* tbw* Fu_bmWeb*
ϕ*Rn_bmWebθ=	35.32	kips	*= min (ϕ* Rn_bmWeb θ1, ϕ* Rn_bmWeb θ2)*
DCR_bmWebθ=	0.296	OK	*= Pr/ ϕ*Rn_bmWeb θ*
			Ok if DCR_bmWebq <= DCR_Allowed

SHEAR PLATE:

Lvg_sp=	3.599	in	*= Lsp_edge/ cos(θ)*
La_sp=	3.162	in	*= Lvg_sp - db_sp/2*
ϕ*Rn_spθ1=	69.362	kips	*= ϕbolt* 1.2* La_sp* tsp* Fu_shearTab*
ϕ*Rn_spθ2=	38.391	kips	*= ϕbolt* 2.4* db_sp* tsp* Fu_shearTab*
ϕ*Rn_spθ=	38.39	kips	*= min (ϕ* Rn_sp θ1, ϕ* Rn_sp θ2)*
DCR_spθ=	0.273	OK	*= Pr/ (ϕ* Rn_sp θ)*
			OK if DCR_sp θ <= DCR_Allowed

15.5b: BEAM WEB AND SHEAR TAB BLOCKSHEAR CHECK

CASE 1: HORIZONTAL REACTIONS



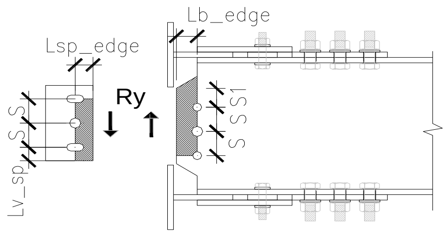
BEAM WEB:

ϕblockshear=	0.75		
Ubs=	1		
Lb_edge=	1.750	in	*value previously defined*
Lh_bmWeb=	3.500	in	*= 2* Lb_edge*
Agv_bmWebHorz=	1.208	in^2	*= Lb_edge* tbw*
Ant_bmWebHorz=	0.000	in^2	*= 0* 2* Pi* db_sp/2* 0.5* tbw*
Anv_bmWebHorz=	1.208	in^2	*= Agv_bmWebHorz*
Rn_bmWebHorz1=	47.09	kips	*= 0.6* Fu_bmWeb* Anv_bmWebHorz + Ubs* Fu_bmWeb* Ant_bmWebHorz*
Rn_bmWebHorz2=	36.23	kips	*= 0.6* Fy_bmWeb* Agv_bmWebHorz + Ubs* Fu_bmWeb* Ant_bmWebHorz*
Rn_bmWebHorz=	27.17	kips	*= ϕblocksher*min (Rn_bmWebHorz1, Rn_bmWebHorz2)*
DCR_bmWebHorz=	0.187	OK	*= Px/ Rn_bmWebHorz, OK if DCR_bmWebHorz <= DCR_Allowed*

SHEAR PLATE:

Lsp_edge=	1.750	in	*value previously defined*
Lh_sp=	3.500	in	*= 2* Lsp_edge*
Agv_spHorz=	1.313	in^2	*= Lh_sp* tsp*
Ant_spHorz=	0.0000	in^2	*= 0* 2* Pi* db_sp/2* 0.5* tsp*
Anv_spHorz=	1.313	in^2	*= Agv_bmWebHorz*
Rn_spHorz1=	51.19	kips	*= 0.6* Fu_sp* Anv_spHorz + Ubs* Fu_sp* Ant_spHorz*
Rn_spbHorz2=	39.38	kips	*= 0.6* Fy_sp* Agv_spHorz + Ubs* Fu_sp* Ant_spHorz*
ϕRn_spHorz=	29.53	kips	*= ϕblocksher* min (Rn_spHorz1, Rn_spbHorz2)*
DCR_spHorz_BS=	0.172	OK	*= Px/ Rn_spHorz, OK if DCR_spHorz <= DCR_Allowed*

CASE 2: VERTICAL REACTIONS



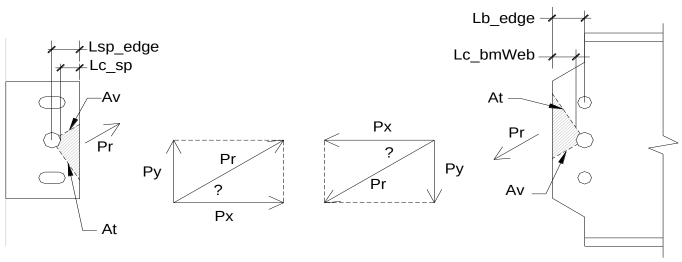
BEAM WEB:

Sbm_web=	3.00	in	*= Svert*
Lb_edge=	1.75	in	*value previously defined*
h_bmWeb=	9.000	in	*assume to be same as shear tab (conservative)*
Agv_bmWeb1=	3.105	in^2	*= tbw* h_bmWeb*
Anv_bmWeb1=	2.350	in^2	*= tbw* (h_bmWeb - 2.5* db_sp)*
Ant_bmWeb1=	0.453	in^2	*= (Lb_edge - db_sp/2)* tbw*
Rn_bmWebVert1=	121.095	kips	*= 0.6* Fu_bmWeb* Anv_bmWeb1 + Ubs*Fu_bmWeb* Ant_bmWeb1*
Rn_bmWebVert2=	122.583	kips	*= 0.6* Fy_bmWeb* Agv_bmWeb1 + Ubs*Fu_bmWeb* Ant_bmWeb1*
ϕRn_bmWebVert=	90.82	kips	*= ϕblockshear* min (Rn_bmWebVert1, Rn_bmWebVert2)*
DCR_bmWebVert_BS=	0.302	OK	*= Vu_bm/ ϕRn_bmWebVert, OK if DCR_bmWebVert_BS <= DCR_Allowed*

SHEAR PLATE:

Ssp=	3.00	in	*= sbm_web*
Lv_sp=	1.500	in	*value previously defined*
hsp=	9.000	in	*value previously defined*
Lslot=	1.500	in	*value previously defined*
Agv_sp1=	3.375	in^2	*= hsp* tsp*
Anv_sp1=	2.555	in^2	*= tsp* (hsp - 2.5* db_sp)*
Ant_sp1=	0.375	in^2	*= (Lsp_edge - Lslot/2)* tsp*
Rn_spVert1=	124.008	kips	*= 0.6* Fu_shearTab* Anv_sp1 + Ubs* Fu_shearTab* Ant_sp1*
Rn_spVert2=	125.625	kips	*= 0.6* Fy_shearTab* Agv_sp1 + Ubs* Fu_shearTab* Ant_sp1*
ϕRn_spVert=	93.01	kips	*= ϕblocksher* min (Rn_spVert1, Rn_spVert2)*
DCR_spVert_BS=	0.295	OK	*= Vu_bm/ ϕRn_spVert, OK if DCR_spVert_BS <= DCR_Allowed*

CASE 3: COMBINED AXIAL AND VERTICAL REACTIONS



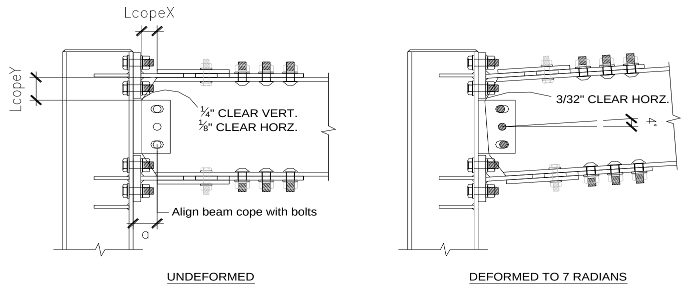
BEAM WEB:

θ=	1.063	radians	*value previously defined*
Lb_edge=	1.750	in	*value previously defined*
Lvg_web=	3.599	in	*value previously defined*
Lvn_web=	3.16	in	*= Lvg_web - db_sp/ 2*
Ltg_web=	2.00	in	*= Lb_edge/ Sin(θ)*
Ltn_web=	1.57	in	*= Ltg_web - db_sp/ 2*
Agv_web=	1.24	in^2	*= Lvg_web* tbw*
Anv_web=	1.09	in^2	*= Lvn_web* tbw*
Agt_web=	0.69	in^2	*= Ltg_web* tbw*
Ant_web=	0.54	in^2	*= Ltn_web* tbw*
Rn_bmWeb_BS1=	77.64	kips	*= 0.6* Fu_bmWeb* Agv_web + Ubs* Fu_bmWeb* Ant_web*
Rn_bmWeb_BS2=	72.35	kips	*= 0.6* Fy_bmWeb* Agv_web + Ubs* Fu_bmWeb* Ant_web*
ϕRn_bmWeb_BS=	54.26	kips	*= ϕblockshear* min (Rn_bmWeb_BS1, Rn_bmWeb_BS2)*
DCR_bmWebθ_BS=	0.193	OK	*= Pr/ fRn_bmWeb_BS, OK if DCR_bmWEB θ_BS <= DCR_Allowed*

SHEAR PLATE:

Lvg_sp=	3.599	in	*value previously defined*
Lsp_edge=	1.750	in	*value previously defined*
Lvn_sp=	3.16	in	*= Lvg_sp - db_sp/2*
Ltg_sp=	2.00	in	*= Lsp_edge/ Sin(θ)*
Ltn_sp=	1.57	in	*= Ltg_sp - db_sp/2*
Agv_sp=	1.350	in	*= Lvg_sp* tsp*
Anv_sp=	1.186	in	*= Lvn_sp* tsp*
Agt_sp=	0.751	in	*= Ltg_sp* tsp*
Ant_sp=	0.587	in	*= Ltn_sp* tsp*
Rn_sp_BS1=	84.39	kips	*= 0.6* Fu_shearTab* Anv_sp + Ubs* Fu_shearTab* Ant_sp*
Rn_sp_BS2=	78.64	kips	*= 0.6* Fy_shearTab* Agv_sp + Ubs* Fu_shearTab* Ant_sp*
ϕRn_sp_BS=	58.98	kips	*= ϕshearblock* min (Rn_sp_BS1, Rn_sp_BS2)*
DCR_spθ_BS=	0.177	OK	*= Pr/ ϕRn_sp_BS, OK if DCR_sp θ_BS <= DCR_Allowed*

15.6: DETAIL BEAM FLANGE AND WEB COPE DISTANCE



θrotation=	0.070	radians		
Lclear_vert=	0.2500	in		
Lclear_horz=	0.125	in		
Lb_edge=	1.75	in	*value previously defined*	
t_flange=	0.875	in	*Link flange thickness*	
h_flange=	5.750	in	*value previously defined*	
d=	16.10	in	*value previously defined*	
t_stem=	0.50		*value previously defined*	
a=	2.75	in	*value previously defined*	
hsp=	9.00	in	*value previously defined*	
Lgap_vert=	0.93	in	OK	*OK if Lgap_vert >= Lclear_vert*
Lgap_horz=	0.125	in	OK	*OK if Lgap_horz >= Lclear_horz*
LcopeX=	1.75	in	*Align end of beam flange cut with beam web bolt centerline*	
LcopeY=	3.0000	in	*Align vertical web cope with top of Shear Plate*	

Base Reaction (kips, in)												
Load Combos	Node	Combo	F1 (Shear)	F2	F3 (Axial)	M2	Node	Combo	F1 (Shear)	F2	F3 (Axial)	M2
1	1	LCM1	0.07	0.00	4.52	0.00	8	LCM1	-0.07	0.00	1.92	0.00
2	1	LCM2	0.00	0.00	4.20	0.00	8	LCM2	0.00	0.00	0.00	0.00
3	1	LCM3	0.00	0.00	0.00	0.00	8	LCM3	0.00	0.00	0.00	0.00
4	1	LCM4	0.06	0.00	2.38	0.00	8	LCM4	-0.06	0.00	0.88	0.00
5	1	LCM5	0.00	0.00	0.00	0.00	8	LCM5	0.00	0.00	0.00	0.00
6	1	LCM6	-3.36	0.00	-4.22	0.00	8	LCM6	-1.80	0.00	4.22	0.00
7	1	LCM7	-2.82	0.00	-5.46	0.00	8	LCM7	-2.23	0.00	5.46	0.00
8	1	LCM8	-0.02	0.00	4.28	0.00	8	LCM8	-0.17	0.00	2.15	0.00
9	1	LCM9	-0.02	0.00	8.48	0.00	8	LCM9	-0.17	0.00	2.15	0.00
10	1	LCM10	-0.02	0.00	4.28	0.00	8	LCM10	-0.17	0.00	2.15	0.00
11	1	LCM11	0.04	0.00	6.66	0.00	8	LCM11	-0.23	0.00	3.04	0.00
12	1	LCM12	-0.02	0.00	4.28	0.00	8	LCM12	-0.17	0.00	2.15	0.00
13	1	LCM13	-0.02	0.00	7.43	0.00	8	LCM13	-0.17	0.00	2.15	0.00
14	1	LCM14	0.02	0.00	9.22	0.00	8	LCM14	-0.22	0.00	2.82	0.00
15	1	LCM15	-0.02	0.00	7.43	0.00	8	LCM15	-0.17	0.00	2.15	0.00
16	1	LCM16	-1.94	0.00	1.98	0.00	8	LCM16	-1.15	0.00	4.45	0.00
17	1	LCM17	2.09	0.00	7.05	0.00	8	LCM17	1.00	0.00	-0.62	0.00
18	1	LCM18	-2.48	0.00	0.14	0.00	8	LCM18	-2.12	0.00	7.13	0.00
19	1	LCM19	2.65	0.00	10.07	0.00	8	LCM19	1.95	0.00	-2.80	0.00
20	1	LCM20	-1.44	0.00	5.77	0.00	8	LCM20	-0.88	0.00	3.81	0.00
21	1	LCM21	1.58	0.00	9.56	0.00	8	LCM21	0.73	0.00	0.02	0.00
22	1	LCM22	-1.39	0.00	7.55	0.00	8	LCM22	-0.93	0.00	4.48	0.00
23	1	LCM23	1.63	0.00	11.35	0.00	8	LCM23	0.69	0.00	0.68	0.00
24	1	LCM24	-1.44	0.00	5.77	0.00	8	LCM24	-0.88	0.00	3.81	0.00
25	1	LCM25	1.58	0.00	9.56	0.00	8	LCM25	0.73	0.00	0.02	0.00
26	1	LCM26	-1.84	0.00	4.38	0.00	8	LCM26	-1.61	0.00	5.83	0.00
27	1	LCM27	2.01	0.00	11.83	0.00	8	LCM27	1.44	0.00	-1.62	0.00
28	1	LCM28	-1.80	0.00	6.17	0.00	8	LCM28	-1.65	0.00	6.49	0.00
29	1	LCM29	2.05	0.00	13.62	0.00	8	LCM29	1.40	0.00	-0.96	0.00
30	1	LCM30	-1.84	0.00	4.38	0.00	8	LCM30	-1.61	0.00	5.83	0.00
31	1	LCM31	2.01	0.00	11.83	0.00	8	LCM31	1.44	0.00	-1.62	0.00
32	1	LCM32	-1.97	0.00	0.18	0.00	8	LCM32	-1.12	0.00	3.68	0.00
33	1	LCM33	2.06	0.00	5.24	0.00	8	LCM33	1.03	0.00	-1.38	0.00
34	1	LCM34	-2.53	0.00	-2.85	0.00	8	LCM34	-2.07	0.00	5.87	0.00
35	1	LCM35	2.60	0.00	7.09	0.00	8	LCM35	2.00	0.00	-4.07	0.00
36	1	LCM36	0.01	0.00	6.08	0.00	8	LCM36	-0.20	0.00	2.92	0.00
37	1	LCM37	-0.01	0.00	11.90	0.00	8	LCM37	-0.19	0.00	2.54	0.00
38	1	LCM38	0.02	0.00	13.09	0.00	8	LCM38	-0.22	0.00	2.98	0.00
39	1	LCM39	-0.01	0.00	11.90	0.00	8	LCM39	-0.19	0.00	2.54	0.00
40	1	LCM40	-0.01	0.00	7.28	0.00	8	LCM40	-0.19	0.00	2.54	0.00
41	1	LCM41	-1.59	0.00	3.31	0.00	8	LCM41	-0.99	0.00	4.41	0.00
42	1	LCM42	1.77	0.00	7.53	0.00	8	LCM42	0.81	0.00	0.19	0.00
43	1	LCM43	0.09	0.00	11.10	0.00	8	LCM43	-0.28	0.00	3.95	0.00
44	1	LCM44	-1.49	0.00	7.12	0.00	8	LCM44	-1.09	0.00	5.82	0.00
45	1	LCM45	1.87	0.00	11.34	0.00	8	LCM45	0.71	0.00	1.60	0.00
46	1	LCM46	-0.01	0.00	7.28	0.00	8	LCM46	-0.19	0.00	2.54	0.00
47	1	LCM47	-1.59	0.00	3.31	0.00	8	LCM47	-0.99	0.00	4.41	0.00
48	1	LCM48	1.77	0.00	7.53	0.00	8	LCM48	0.81	0.00	0.19	0.00
49	1	LCM49	-3.27	0.00	3.30	0.00	8	LCM49	-1.89	0.00	6.52	0.00
50	1	LCM50	3.45	0.00	11.74	0.00	8	LCM50	1.71	0.00	-1.92	0.00
51	1	LCM51	-3.24	0.00	4.49	0.00	8	LCM51	-1.92	0.00	6.96	0.00
52	1	LCM52	3.48	0.00	12.93	0.00	8	LCM52	1.68	0.00	-1.48	0.00
53	1	LCM53	-3.27	0.00	3.30	0.00	8	LCM53	-1.89	0.00	6.52	0.00
54	1	LCM54	3.45	0.00	11.74	0.00	8	LCM54	1.71	0.00	-1.92	0.00
55	1	LCM55	-3.55	0.00	1.74	0.00	8	LCM55	-3.02	0.00	9.93	0.00
56	1	LCM56	3.78	0.00	15.94	0.00	8	LCM56	2.79	0.00	-4.26	0.00
57	1	LCM57	-3.29	0.00	-0.16	0.00	8	LCM57	-1.86	0.00	5.94	0.00
58	1	LCM58	3.42	0.00	8.28	0.00	8	LCM58	1.73	0.00	-2.50	0.00
59	1	LCM59	-3.61	0.00	-3.88	0.00	8	LCM59	-2.96	0.00	8.46	0.00
60	1	LCM60	3.72	0.00	10.32	0.00	8	LCM60	2.85	0.00	-5.73	0.00
61	1	LCM61	-6.94	0.00	-4.81	0.00	8	LCM61	-5.70	0.00	16.48	0.00
62	1	LCM62	7.17	0.00	22.49	0.00	8	LCM62	5.47	0.00	-10.81	0.00
63	1	LCM63	-7.00	0.00	-10.43	0.00	8	LCM63	-5.64	0.00	15.01	0.00
64	1	LCM64	7.10	0.00	16.87	0.00	8	LCM64	5.53	0.00	-12.28	0.00

Note: Negative (-) values for axial load indicate tension

SMF BASE PLATE CONNECTION DESIGN (COLUMN 1)

Ref: AISC DG#1 and AISC DG#16

Column Base Plate Summary

Column = W16X45

Base Plate =	**7.125" x 17.125" x 0.625" thick**
F_{yp} =	**50 ksi ASTM A572 Gr 50**
Web to Plate Weld =	**4/16 in, 2 sides**
Flange to Plate Weld =	**4/16 in, 2 sides**
Anchor Bolts =	**6/8 in. diameter ASTM A449**

DESIGN DATA

Design Forces: Combo ID= 62 **Design Code:** IBC 2018

Left Column:

V=	7.167	kips	
P=	22.486	kips	(Tension is negative)

Base Plate:

Grade =	ASTM A572 Gr 50		Base plate material grade
F_y =	50	ksi	Base plate yield stress
Φ_b =	0.9		Resistance factor for base plate flexure
B =	7.125	in	Base plate width
N =	17.125	in	Base plate length
t =	0.625	in	Base plate thickness

Column:

d =	16.10	in	Column depth
t_w =	0.35	in	Column web thickness
h_{web} =	14.97	in	Column web depth
b_f =	7.04	in	Column flange width
t_f =	0.57	in	Exterior column flange thickness

Anchor Bolts:

d_b =	0.75	in	Anchor bolt diameter
n_b =	4		No. anchor bolts
a =	3.00	in	Anchor bolt spacing
g =	3.00	in	Anchor bolt gage
Ase=	0.442	in	= Pi* (d_b/2)^2
Grade=	A449		
Fut=	120	ksi	

Welds:

F_{EXX} =	70	ksi	Weld electrode minimum strength
Φ_{Weld} =	0.75		Resistance factor for welds
w_{web} =	0.25	in	Fillet weld size between column web and base plate
	2	sides	Number of sides for column web weld
w_{flg} =	0.25	in	Fillet weld size between column flange and base plate
	2	sides	Number of sides for column flange weld

Concrete:

f_c' =	2,500	psi	Concrete compressive stress
Φ_{brg} =	0.65		Resistance factor for concrete bearing

COMPRESSION DESIGN

Concrete Bearing:

A_1 =	122.01563	in^2	Area of base plate
A_2 =	122.01563	in^2	Area of concrete support concentric with base plate
$\sqrt{A_2/A_1}$ =	1.0		
$f_{p(max)}$ =	2.13	ksi	Maximum bearing stress
P_p =	259	kips	Nominal strength for bearing
ϕP_p =	169	kips	Axial design strength for bearing
DCR_conc=	**0.133**	OK	

Plate Yielding:

m =	0.92	in	Base plate cantilever dimension
n =	0.75	in	Base plate cantilever dimension
X =	0.847		Factor assuming $P_u = \phi P_p$
λ=	1.000		
λn' =	2.66	in	Base plate cantilever dimension
l =	2.66	in	Critical base plate cantilever dimension
ϕP_n =	151	kips	Axial design strength for plate yielding
DCR_PLyield=	**0.149**	OK	

WELD DESIGN

Minimum Weld Size:

t_w =	0.35	in	Column web thickness
w_{min} =	3/16	in	AISC Table J2.4
w_{web} =	4/16	in	Weld size provided - web to base plate
t_f =	0.57	in	Column flange thickness
w_{min} =	4/16	in	AISC Table J2.4
w_{flg} =	4/16	in	Weld size provided - flange to base plate

Weld Capacity & Length:

$\phi R_{n\text{-}para}$ =	1.39	kip/in	Weld design strength (per 1/16")
$\phi R_{n\text{-}perp}$ =	2.09	kip/in	Weld design strength (per 1/16")
$L_{weld\text{-}web}$ =	13.97	in	Web weld length (1 side)
$L_{weld\text{-}flg}$ =	6.54	in	Flange weld length (1 flange, 1 sides)

Shear Only

V=	7.167	kips
t_weld_web=	0.25	in
ϕRn_web=	155.58	kips
DCR_V=	**0.05**	OK

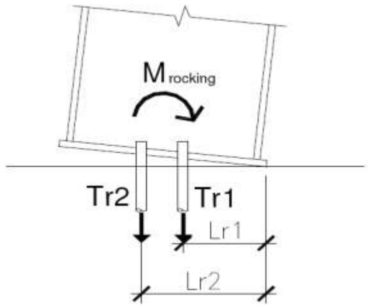
Tension Only

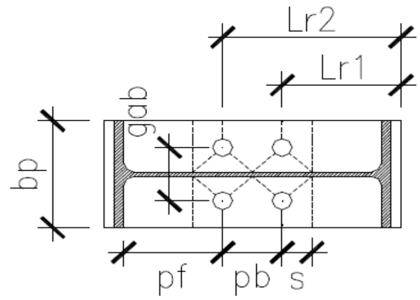
T=	0.00	kips
t_weld_flg=	0.25	in
ϕRn_flg=	109.25	kips
ϕRn=	264.84	kips
DCR_T=	**0.000**	OK

Shear+Tension

Vu_srss=	7.17	kips	
θ_srss=	0.00	degrees	
$\phi R_{n\text{-}\theta}$=	1.39	kip/in	Weld design strength (per 1/16")
ϕR_n=	301.25	kips	
DCR_V+T=	**0.024**	OK	

TENSION DESIGN





Base Plate Bending Cpacity Based on DG #16 Table 3-3

Lr1=	7.0625	in		bp=	7.125	in	Y=	100.71	in
Lr2=	10.0625	in		pb=	3.00	in	gamma_r=	1.25	
Tr1=	11.24	kips	Assume equal to 1/2 of P	gab=	3.00	in	ϕb=	0.9	
Tr2=	11.24	kips	Assume equal to 1/2 of P	pf=	5.985	in			
Mrocking=	192.5	kip-in	Rocking moment about Bp tip	s_tmp=	2.31	in			
				s=	5.98500	in			
treq=	0.243	in	Plate thickness required for tension						
tbp=	0.625	in							
DCR_bp=	**0.388**	OK							

ANCHOR BOLT DESIGN (ACI 318)

ϕv=	0.65	ϕgrout=	0.8	V=	7.167	kips	ϕT_{ab}=	159.0	kips
ϕt=	0.75	ϕseismic=	0.75	T/C=	0.00	kips	ϕV_{ab}=	49.6	kips

Shear Only DCR_abV= **0.144** OK

Tension Only DCR_abT= **0.000** OK

Shear+Tension DCR_V+T= **0.144** OK

Anchor Bolt Design per AISC DG #1 use 2 bolts for shear resistance with oversized baseplate holes

nv=	2										
Fnv=	54	ksi	0.45 * Fut	nt=	4			F'nt=	90	ksi
ϕRv=	35.8	kips	0.75* nv * Ase * Fnv	Fnt=	90	ksi	0.75 * Fut	Rnt=	119	kips
DCR_abV2=	**0.200**	OK		ϕRt=	119	kips		DCR2_V+T=	**0.000**	OK

(For tension (4 bolts) + Shear (2 bolts) check)

Check Plate Moment Yielding PRIOR to Anchor Bolts Moment Yield

Mbp_yield=	1276	kip-in	
Tab=	53.0	kips	Anchor bolt yield force
Mab_yield=	1816	kip-in	Anchor bolt yield moment
DCR_yield=	**0.703**	OK	

LEFT COLUMN DESIGN DCRs

Combo	Load Combinations	DCR ConcBearing	DCR PL_comp	DCR Weld	DCR PL_tension	DCR AnchorBolt
36	1.4 D	0.036	0.040	0.000	0.202	0.000
37	1.2 D + 1.6 L + 0.5 Lr	0.071	0.079	0.000	0.283	0.000
38	1.2 D + 1.6 L + 0.5 S	0.078	0.086	0.000	0.296	0.001
39	1.2 D + 1.6 L + 0.5 R	0.071	0.079	0.000	0.283	0.000
40	1.2 D + 1.6 Lr + 0.5 L	0.043	0.048	0.000	0.221	0.000
41	1.2 D + 1.6 Lr + 0.5 W	0.020	0.022	0.010	0.149	0.044
42	1.2 D + 1.6 Lr - 0.5 W	0.045	0.050	0.011	0.225	0.049
43	1.2 D + 1.6 S + 0.5 L	0.066	0.073	0.001	0.273	0.003
44	1.2 D + 1.6 S + 0.5 W	0.042	0.047	0.010	0.219	0.042
45	1.2 D + 1.6 S - 0.5 W	0.067	0.075	0.012	0.276	0.052
46	1.2 D + 1.6 R + 0.5 L	0.043	0.048	0.000	0.221	0.000
47	1.2 D + 1.6 R + 0.5 W	0.020	0.022	0.010	0.149	0.044
48	1.2 D + 1.6 R - 0.5 W	0.045	0.050	0.011	0.225	0.049
49	1.2 D + 1.0 W + 0.5 L + 0.5 Lr	0.020	0.022	0.021	0.149	0.091
50	1.2 D - 1.0 W + 0.5 L + 0.5 Lr	0.070	0.078	0.022	0.281	0.096
51	1.2 D + 1.0 W + 0.5 L + 0.5 S	0.027	0.030	0.021	0.174	0.090
52	1.2 D - 1.0 W + 0.5 L + 0.5 S	0.077	0.085	0.022	0.295	0.097
53	1.2 D + 1.0 W + 0.5 L + 0.5 R	0.020	0.022	0.021	0.149	0.091
54	1.2 D - 1.0 W + 0.5 L + 0.5 R	0.070	0.078	0.022	0.281	0.096
55	(1.2 + 0.2 SDS)D + E + 0.5 L + f2* S	0.010	0.012	0.023	0.108	0.099
56	(1.2 + 0.2 SDS)D - E + 0.5 L + f2* S	0.095	0.105	0.024	0.327	0.106
57	0.9 D + 1.0 W	0.000	0.000	0.021	0.032	0.092
58	0.9 D - 1.0 W	0.049	0.055	0.022	0.236	0.096
59	(0.9 - 0.2 SDS) D + E	0.000	0.000	0.023	0.161	0.101
60	(0.9 - 0.2 SDS) D - E	0.061	0.068	0.024	0.263	0.104
61	(1.2 + 0.2 SDS)D + Omega*E + 0.5L + f2*S	0.000	0.000	0.045	0.180	0.194
62	(1.2 + 0.2 SDS)D -Omega*E + 0.5L + f2*S	0.133	0.149	0.046	0.388	0.200
63	(0.9 - 0.2 SDS) D + Omega*E	0.000	0.000	0.045	0.265	0.196
64	(0.9 - 0.2 SDS) D - Omega*E	0.100	0.111	0.046	0.336	0.199
	Max=	0.133	0.149	0.046	0.388	0.200
	Check=	OK	OK	OK	OK	OK

SMF BASE PLATE CONNECTION DESIGN (COLUMN 2)

Ref: AISC DG#1 and AISC DG#16

Column Base Plate Summary

Column = W16X45

Base Plate =	**7.125" x 17.125" x 0.625" thick**
F_{yp} =	**50 ksi ASTM A572 Gr 50**
Web to Plate Weld =	**4/16 in, 2 sides**
Flange to Plate Weld =	**4/16 in, 2 sides**
Anchor Bolts =	**6/8 in. diameter ASTM A449**

DESIGN DATA

Design Forces: Combo ID= 61 **Design Code:** IBC 2018

Left Column:

V=	-5.698	kips	
P=	16.480	kips	(Tension is negative)

Base Plate:

Grade =	ASTM A572 Gr 50		Base plate material grade
F_y =	50	ksi	Base plate yield stress
Φ_b =	0.9		Resistance factor for base plate flexure
B =	7.125	in	Base plate width
N =	17.125	in	Base plate length
t =	0.625	in	Base plate thickness

Column:

d =	16.10	in	Column depth
t_w =	0.35	in	Column web thickness
h_{web} =	14.97	in	Column web depth
b_f =	7.04	in	Column flange width
t_f =	0.57	in	Exterior column flange thickness

Welds:

F_{EXX} =	70	ksi	Weld electrode minimum strength
Φ_{Weld} =	0.75		Resistance factor for welds
w_{web} =	0.25	in	Fillet weld size between column web and base plate
	2	sides	Number of sides for column web weld
w_{flg} =	0.25	in	Fillet weld size between column flange and base plate
	2	sides	Number of sides for column flange weld

Anchor Bolts:

d_b =	0.75	in	Anchor bolt diameter
n_b =	4		No. anchor bolts
a =	3.00	in	Anchor bolt spacing
g =	3.00	in	Anchor bolt gage
Ase=	0.442	in	= Pi* $(d_b/2)$^2
Grade=	A449		
Fut=	120	ksi	

Concrete:

f_c' =	2,500	psi	Concrete compressive stress
Φ_{brg} =	0.65		Resistance factor for concrete bearing

COMPRESSION DESIGN

Concrete Bearing:

A_1 =	122.01563	in^2	Area of base plate
A_2 =	122.01563	in^2	Area of concrete support concentric with base plate
$\sqrt{A_2/A_1}$ =	1.0		
$f_{p(max)}$ =	2.13	ksi	Maximum bearing stress
P_p =	259	kips	Nominal strength for bearing
ϕP_p =	169	kips	Axial design strength for bearing
DCR_conc=	**0.098**	OK	

Plate Yielding:

m =	0.92	in	Base plate cantilever dimension
n =	0.75	in	Base plate cantilever dimension
X =	0.847		Factor assuming $P_u = \phi P_p$
λ=	1.000		
λn' =	2.66	in	Base plate cantilever dimension
l =	2.66	in	Critical base plate cantilever dimension
ϕP_n =	151	kips	Axial design strength for plate yielding
DCR_PLyield=	**0.109**	OK	

WELD DESIGN

Minimum Weld Size:

t_w =	0.35	in	Column web thickness
w_{min} =	3/16	in	AISC Table J2.4
w_{web} =	4/16	in	Weld size provided - web to base plate
t_f =	0.57	in	Column flange thickness
w_{min} =	4/16	in	AISC Table J2.4
w_{flg} =	4/16	in	Weld size provided - flange to base plate

Weld Capacity & Length:

$\phi R_{n\text{-}para}$ =	1.39	kip/in	Weld design strength (per 1/16")
$\phi R_{n\text{-}perp}$ =	2.09	kip/in	Weld design strength (per 1/16")
$L_{weld\text{-}web}$ =	13.97	in	Web weld length (1 side)
$L_{weld\text{-}flg}$ =	6.54	in	Flange weld length (1 flange, 1 sides)

Shear Only

V=	5.698	kips
t_weld_web=	0.25	in
ϕRn_web=	155.58	kips
DCR_V=	**0.04**	OK

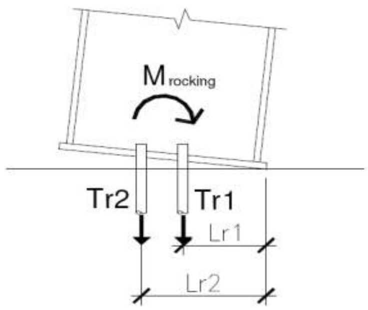
Tension Only

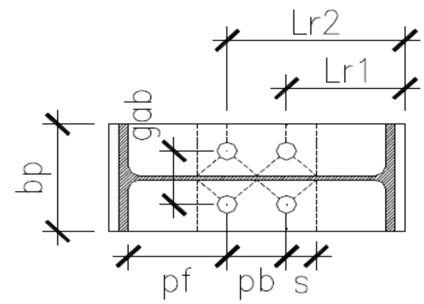
T=	0.00	kips
t_weld_flg=	0.25	in
ϕRn_flg=	109.25	kips
ϕRn=	264.84	kips
DCR_T=	**0.000**	OK

Shear+Tension

Vu_srss=	5.70	kips	
θ_srss=	0.00	degrees	
$\phi R_{n\text{-}\theta}$=	1.39	kip/in	Weld design strength (per 1/16")
ϕR_n=	301.25	kips	
DCR_V+T=	**0.019**	OK	

TENSION DESIGN





Base Plate Bending Cpacity Based on DG #16 Table 3-3

Lr1=	7.0625	in	
Lr2=	10.0625	in	
Tr1=	8.24	kips	Assume equal to 1/2 of P
Tr2=	8.24	kips	Assume equal to 1/2 of P
Mrocking=	141.1	kip-in	Rocking moment about Bp tip
treq=	0.208	in	Plate thickness required for tension
tbp=	0.625	in	
DCR_bp=	**0.333**	OK	

bp=	7.125	in
pb=	3.00	in
gab=	3.00	in
pf=	5.985	in
s_tmp=	2.31	in
s=	5.985	in

Y=	100.71	in
gamma_r=	1.25	
ϕb=	0.9	

ANCHOR BOLT DESIGN (ACI 318)

ϕv=	0.65	ϕgrout=	0.8	V=	5.698	kips	ϕT_{ab}=	159.0	kips
ϕt=	0.75	ϕseismic=	0.75	T/C=	0.00	kips	ϕV_{ab}=	49.6	kips

Shear Only
DCR_abV= **0.115** OK

Tension Only
DCR_abT= **0.000** OK

Shear+Tension
DCR_V+T= **0.115** OK

Anchor Bolt Design per AISC DG #1 use 2 bolts for shear resistance with oversized baseplate holes

nv=	2		
Fnv=	54	ksi	0.45 * Fut
ϕRv=	35.8	kips	0.75* nv * Ase * Fnv
DCR_abV2=	**0.159**	OK	

For tension (4 bolts) + Shear (2 bolts) check

nt=	4		
Fnt=	90	ksi	0.75 * Fut
ϕRt=	119	kips	

F'nt=	90	ksi
Rnt=	119	kips
DCR2_V+T=	**0.000**	OK

Check Plate Moment Yielding PRIOR to Anchor Bolts Moment Yield

Mbp_yield=	1276	kip-in	
Tab=	53.0	kips	Anchor bolt yield force
Mab_yield=	1816	kip-in	Anchor bolt yield moment
DCR_yield=	**0.703**	OK	

LEFT COLUMN DESIGN DCRs

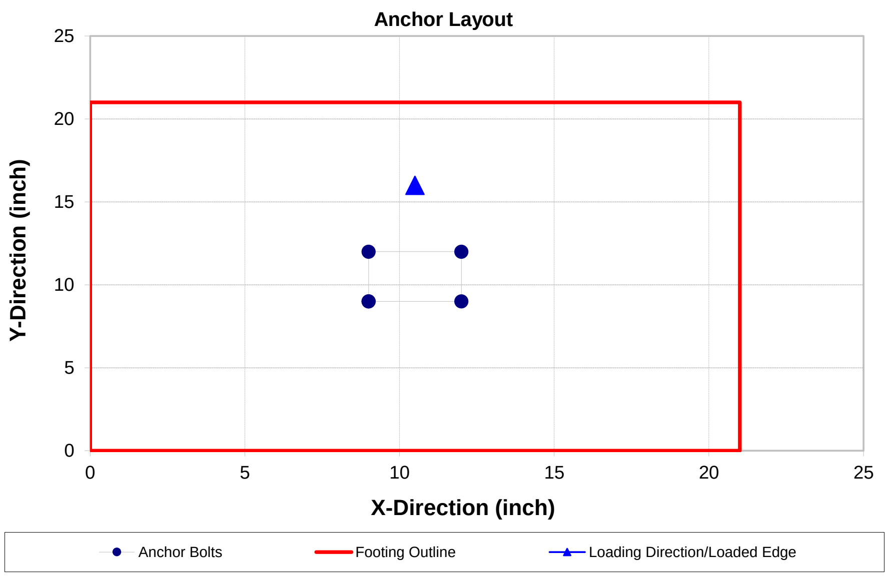
Combo	Load Combinations	DCR ConcBearing	DCR PL_comp	DCR Weld	DCR PL_tension	DCR AnchorBolt
36	1.4 D	0.017	0.019	0.001	0.140	0.006
37	1.2 D + 1.6 L + 0.5 Lr	0.015	0.017	0.001	0.130	0.005
38	1.2 D + 1.6 L + 0.5 S	0.018	0.020	0.001	0.141	0.006
39	1.2 D + 1.6 L + 0.5 R	0.015	0.017	0.001	0.130	0.005
40	1.2 D + 1.6 Lr + 0.5 L	0.015	0.017	0.001	0.130	0.005
41	1.2 D + 1.6 Lr + 0.5 W	0.026	0.029	0.006	0.172	0.028
42	1.2 D + 1.6 Lr - 0.5 W	0.001	0.001	0.005	0.036	0.023
43	1.2 D + 1.6 S + 0.5 L	0.023	0.026	0.002	0.163	0.008
44	1.2 D + 1.6 S + 0.5 W	0.035	0.038	0.007	0.198	0.030
45	1.2 D + 1.6 S - 0.5 W	0.010	0.011	0.005	0.104	0.020
46	1.2 D + 1.6 R + 0.5 L	0.015	0.017	0.001	0.130	0.005
47	1.2 D + 1.6 R + 0.5 W	0.026	0.029	0.006	0.172	0.028
48	1.2 D + 1.6 R - 0.5 W	0.001	0.001	0.005	0.036	0.023
49	1.2 D + 1.0 W + 0.5 L + 0.5 Lr	0.039	0.043	0.012	0.209	0.053
50	1.2 D - 1.0 W + 0.5 L + 0.5 Lr	0.000	0.000	0.011	0.114	0.048
51	1.2 D + 1.0 W + 0.5 L + 0.5 S	0.041	0.046	0.012	0.216	0.054
52	1.2 D - 1.0 W + 0.5 L + 0.5 S	0.000	0.000	0.011	0.100	0.047
53	1.2 D + 1.0 W + 0.5 L + 0.5 R	0.039	0.043	0.012	0.209	0.053
54	1.2 D - 1.0 W + 0.5 L + 0.5 R	0.000	0.000	0.011	0.114	0.048
55	(1.2 + 0.2 SDS)D + E + 0.5 L + f2* S	0.059	0.066	0.019	0.258	0.084
56	(1.2 + 0.2 SDS)D - E + 0.5 L + f2* S	0.000	0.000	0.018	0.169	0.078
57	0.9 D + 1.0 W	0.035	0.039	0.012	0.200	0.052
58	0.9 D - 1.0 W	0.000	0.000	0.011	0.129	0.048
59	(0.9 - 0.2 SDS) D + E	0.050	0.056	0.019	0.238	0.083
60	(0.9 - 0.2 SDS) D - E	0.000	0.000	0.022	0.196	0.080
61	(1.2 + 0.2 SDS)D + Omega*E + 0.5L + f2*S	0.098	0.109	0.037	0.333	0.159
62	(1.2 + 0.2 SDS)D -Omega*E + 0.5L + f2*S	0.000	0.000	0.041	0.269	0.153
63	(0.9 - 0.2 SDS) D + Omega*E	0.089	0.099	0.036	0.317	0.158
64	(0.9 - 0.2 SDS) D - Omega*E	0.000	0.000	0.046	0.287	0.155
	Max=	0.098	0.109	0.046	0.333	0.159
	Check=	OK	OK	OK	OK	OK

ACI 318 CHAPTER 17: ANCHOR STRENGTH (SQUARE FOOTING) (COLUMN 1 & 2)

1. Anchor Geometry

j= 8

Across (X-Dir)			Up & Down (Y-Dir)		
Columns	2		Rows	2	
Left Edge	9.00	inch	Top Edge	9.00	inch
Spacing	3.00	inch	Spacing	3.00	inch
Right Edge	9.00	inch	Bot Edge	9.00	inch
Length=	21	inch	Length=	21	inch



2. Design Parameters

Design Code: IBC 2018

Shear Direction	Perp	to Top Edge	Anchor Type:	Headed Bolt	
Grouted Pad	YES	Yes/No	Steel Material:	A449	
Code	Sec. 9.2	App C/Sec.9.2	Hex Head Type:	Heavy	
Seismic	YES	Yes/No	Fy:	90	ksi
Tens Reinf	NO	Yes/No	Fu:	120	ksi
Shear Reinf	NO	Yes/No	d_b:	0.75	in
Min Cover	2	per ACI-7	$A_{se:}$	0.334	in2
Concrete f'c=	2.5	ksi	SDC:	D	
Cracked Conc.?	YES	Yes/No	Ductile Steel:	Yes	
Concrete Depth=	10.00	in	$h_{ef:}$	6.00	in

$t_{plate\ washer}$: 0.375 in

n: 4 Anchors

3. Loading

LCM 64 Column 1 N

Nu=	12.28	kips	e'_{xV}=	0	inch	e'_{yV}=	0	inch
Vu=	5.53	kips	e'_{xN}=	0	inch	e'_{yN}=	0	inch

Tension Limit States

17.4.1 - Steel Strength of Anchor in Tension

f_{uta}=	120	ksi	Not to exceed the smaller of 1.9fy or 125,000 psi
Ase=	0.334		
Nsa=n*Ase*futa=	**160.5**	kips	Limit state for steel in tension

17.4.2 - Group Projected Area Determination

h_{ef}=	6.00	inch	Embeddment Depth			
$1.5h_{ef}$:	9.00	inch				
#Sides:	0		# of Sides with c < $1.5h_{ef}$			
$c_{a,max}$:	0.00	inch	*For 3-or-more edge effect only*			
Anchor Spacing:	0.00	inch	*For 3-or-more edge effect only*			
h'_{ef}=	6.00	inch	Modified Embedment (17.4.2.3)			
$1.5h'_{ef}$=	9.00	inch				
Left Edge=	9.00	inch		Top=	9.00	inch
S_{COL}	3.00	inch		S_{ROW}	3.00	inch
Right Edge=	9.00	inch		Bottom=	9.00	inch
A_{Nc}=	441	in2				

17.4.2 - Concrete Breakout Strength in Tension

A_{Nco}=	324	in2	Single Anchor Projected Area
A_{Nc}=	441	in2	Group Anchor Projected Area
k_c=	24		CIP Anchor Type Coefficient
N_b=	17.64	kip	(17.4.2.2b) Single Anchor Breakout Strength
N_b=	0	kip	(17.4.2.4) Alternate Single Anchor Breakout Strength
Use N_b=	18	kip	Single Anchor Breakout Strength
$\Psi_{ed,N}$ =	1.00		Edge Distance Effect
$\Psi_{c,N}$ =	1.00		Service Cracking Effect
$\Psi_{cp,N}$ =	1.00		
$\Psi_{ec,N}$ =	1.00		Eccentrically Loaded Group Effect
N_{cb} =	**24.0**	kip	Nominal Strength

17.4.3 - Pullout Strength of Anchors in Tension

$\Psi_{c,P}$ =	1.00		Service Cracking Effect
Abrg=	0.91	in^2	Used for Headed Bolt and Headed Stud Only
Abrg, plate washer=	2.25	in^2	Used for Headed Bolt Only
$e_h\ d_o$=	NA	in^2	Used for Hooks only, where eh = 4.5do
Np=	45.0	kip	
n $\Psi_{c,P}$ N_p=	**180.0**	kip	

17.4.4 - Side Face Blow-out of headed anchor in tension (not required for Post Installed anchors)

Type=	Headed		Type of head on the anchor
Abrg=	2.25	in^2	Bearing Area of the Head or Plate washer
$0.4h_{ef}$=	2.4	inch	
Critical Edge, c_{a1}=	9.0	inch	
$c_{a1}<.4h_{ef}$?	NO		
Actual Edge $c_{a1,min}$=	9.00	inch	Minimum Edge Distance Used
$c_{a2,min}$=	9.00	inch	Edge Distance Perpendicular
Prelim Nsb (single)=	108.0	kip	
perp modifier=	1.0		
Nsb (single)=	108.0	kip	
Anchor Spacing=	0.0	inch	
group modifer=	1.0		
Nsb (group)=	**NA**	kip	

4. Capacity Summary

Limit State	Φ (Seismic)	ϕ	N	ΦϕN	Failure Mode
N_{sa}	*1.00*	0.75	160.5	120.41	17.4.1, Steel Tension Failure
nN_{pn}	0.75	0.70	24.0	12.60	17.4.2, Concrete Breakout
N_{sb}	0.75	0.70	180.0	94.50	17.4.3, Anchor Pullout
N_{cb}	0.75	0.70	NA	NA	17.4.4, Side Face Blowout

Tension Capacity= **12.60 k** 17.4.2, Concrete Breakout

5. Demand-Capacity Check

Nu=	12.28 k		ϕNn=	12.60 k	
DCR Limit=	1.00		Min. W=	21	in
DCR=	0.97	OK	Min. Embed=	6.0	in

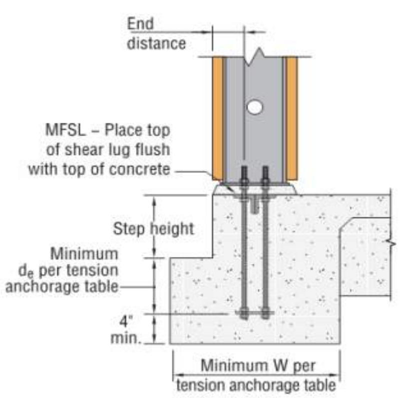
		Column 1			Column 2		
Combo	**Load Combinations**	**Nu (kips)**	**Nu/ϕNn**	**Check**	**Nu (kips)**	**Nu/ϕNn**	**Check**
36	1.4 D	0.00	0.00	OK	0.00	0.00	OK
37	1.2 D + 1.6 L + 0.5 Lr	0.00	0.00	OK	0.00	0.00	OK
38	1.2 D + 1.6 L + 0.5 S	0.00	0.00	OK	0.00	0.00	OK
39	1.2 D + 1.6 L + 0.5 R	0.00	0.00	OK	0.00	0.00	OK
40	1.2 D + 1.6 Lr + 0.5 L	0.00	0.00	OK	0.00	0.00	OK
41	1.2 D + 1.6 Lr + 0.5 W	0.00	0.00	OK	0.00	0.00	OK
42	1.2 D + 1.6 Lr - 0.5 W	0.00	0.00	OK	0.00	0.00	OK
43	1.2 D + 1.6 S + 0.5 L	0.00	0.00	OK	0.00	0.00	OK
44	1.2 D + 1.6 S + 0.5 W	0.00	0.00	OK	0.00	0.00	OK
45	1.2 D + 1.6 S - 0.5 W	0.00	0.00	OK	0.00	0.00	OK
46	1.2 D + 1.6 R + 0.5 L	0.00	0.00	OK	0.00	0.00	OK
47	1.2 D + 1.6 R + 0.5 W	0.00	0.00	OK	0.00	0.00	OK
48	1.2 D + 1.6 R - 0.5 W	0.00	0.00	OK	0.00	0.00	OK
49	1.2 D + 1.0 W + 0.5 L + 0.5 Lr	0.00	0.00	OK	0.00	0.00	OK
50	1.2 D - 1.0 W + 0.5 L + 0.5 Lr	0.00	0.00	OK	1.92	0.11	OK
51	1.2 D + 1.0 W + 0.5 L + 0.5 S	0.00	0.00	OK	0.00	0.00	OK
52	1.2 D - 1.0 W + 0.5 L + 0.5 S	0.00	0.00	OK	1.48	0.09	OK
53	1.2 D + 1.0 W + 0.5 L + 0.5 R	0.00	0.00	OK	0.00	0.00	OK
54	1.2 D - 1.0 W + 0.5 L + 0.5 R	0.00	0.00	OK	1.92	0.15	OK
55	(1.2 + 0.2 SDS)D + E + 0.5 L + f2* S	0.00	0.00	OK	0.00	0.00	OK
56	(1.2 + 0.2 SDS)D - E + 0.5 L + f2* S	0.00	0.00	OK	4.26	0.25	OK
57	0.9 D + 1.0 W	0.16	0.01	OK	0.00	0.00	OK
58	0.9 D - 1.0 W	0.00	0.00	OK	2.50	0.20	OK
59	(0.9 - 0.2 SDS) D + E	3.88	0.31	OK	0.00	0.00	OK
60	(0.9 - 0.2 SDS) D - E	0.00	0.00	OK	5.73	0.45	OK
61	(1.2 + 0.2 SDS)D + Omega*E + 0.5L + f2*S	4.81	0.38	OK	0.00	0.00	OK
62	(1.2 + 0.2 SDS)D -Omega*E + 0.5L + f2*S	0.00	0.00	OK	10.81	0.86	OK
63	(0.9 - 0.2 SDS) D + Omega*E	10.43	0.83	OK	0.00	0.00	OK
64	(0.9 - 0.2 SDS) D - Omega*E	0.00	0.00	OK	12.28	0.97	OK

6. Anchorage Solution

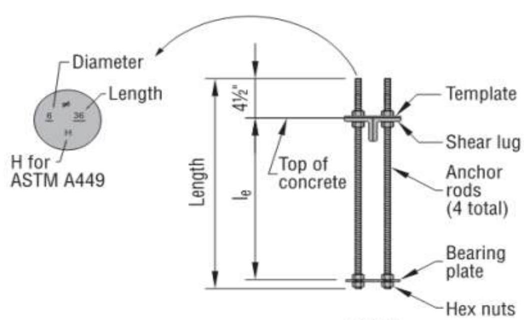
Anchorage Solution **Yes**
Foundation Type Slab On Grade
Anchor Solution Type MFSL

Step Height H_{step} = 16.00 in
Min. Embedment de = 6.00 in
le = 22.00 in $= H_{step} + de$
Grade of Rod = High Strength

Anchorage Model **MFSL-30-HS6**

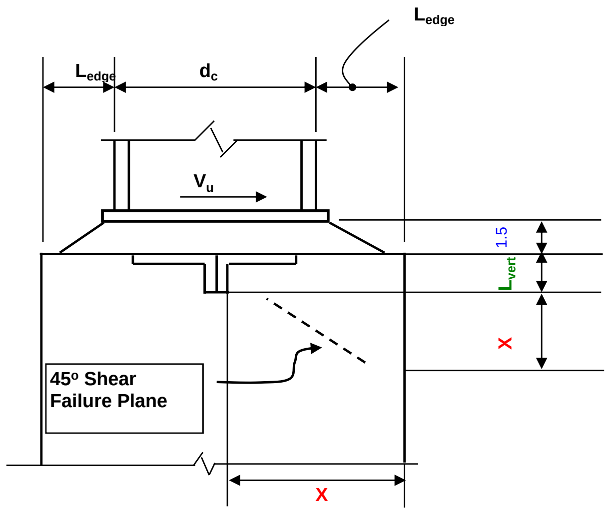


Section View
Slab on Grade

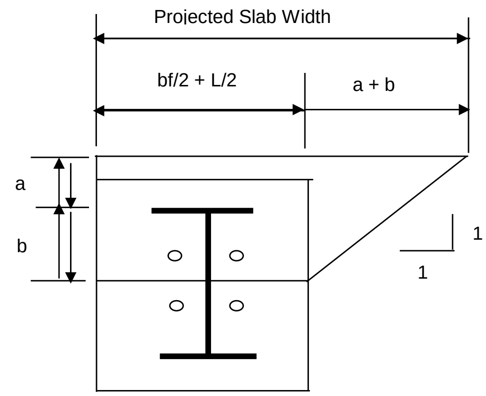


MFSL
U.S. Patent Pending

SHEAR LUG DESIGN AISC DESIGN GUIDE 1 - SOG FOUNDATION (COLUMN 1 & 2)



SHEAR LUG ELEVATION



SHEAR LUG PLAN

f'c= 2,500 psi
L_{edge}= 1.5 in

Column 1= W16X45
ϕVn= 25.39 kips
ϕVn_shearLug= 25.39 kips

Column 2= W16X45
ϕVn= 25.39 kips
ϕVn_shearLug= 25.39 kips

	Column 1				Column 2			
	Vu	ϕVn	Vu/ϕVn	DCR	Vu	ϕVn	Vu/ϕVn	DCR
Combo	kips	kips	-	Check	kips	kips	-	Check
36	0.01	25.39	0.000	OK	0.20	25.39	0.008	OK
37	0.01	25.39	0.000	OK	0.19	25.39	0.007	OK
38	0.02	25.39	0.001	OK	0.22	25.39	0.009	OK
39	0.01	25.39	0.000	OK	0.19	25.39	0.007	OK
40	0.01	25.39	0.000	OK	0.19	25.39	0.007	OK
41	1.59	25.39	0.063	OK	0.99	25.39	0.039	OK
42	1.77	25.39	0.070	OK	0.81	25.39	0.032	OK
43	0.09	25.39	0.004	OK	0.28	25.39	0.011	OK
44	1.49	25.39	0.059	OK	1.09	25.39	0.043	OK
45	1.87	25.39	0.074	OK	0.71	25.39	0.028	OK
46	0.01	25.39	0.000	OK	0.19	25.39	0.007	OK
47	1.59	25.39	0.063	OK	0.99	25.39	0.039	OK
48	1.77	25.39	0.070	OK	0.81	25.39	0.032	OK
49	3.27	25.39	0.129	OK	1.89	25.39	0.074	OK
50	3.45	25.39	0.136	OK	1.71	25.39	0.067	OK
51	3.24	25.39	0.127	OK	1.92	25.39	0.075	OK
52	3.48	25.39	0.137	OK	1.68	25.39	0.066	OK
53	3.27	25.39	0.129	OK	1.89	25.39	0.074	OK
54	3.45	25.39	0.136	OK	1.71	25.39	0.067	OK
55	3.55	25.39	0.140	OK	3.02	25.39	0.119	OK
56	3.78	25.39	0.149	OK	2.79	25.39	0.110	OK
57	3.29	25.39	0.130	OK	1.86	25.39	0.073	OK
58	3.42	25.39	0.135	OK	1.73	25.39	0.068	OK
59	3.61	25.39	0.142	OK	2.96	25.39	0.116	OK
60	3.72	25.39	0.147	OK	2.85	25.39	0.112	OK
61	6.94	25.39	0.273	OK	5.70	25.39	0.224	OK
62	7.17	25.39	0.282	OK	5.47	25.39	0.215	OK
63	7.00	25.39	0.276	OK	5.64	25.39	0.222	OK
64	7.10	25.39	0.280	OK	5.53	25.39	0.218	OK
		Max=	0.282			Max=	0.224	

Summary:

f'c= 2,500 psi
Ledge= 1.5 in
Extra Studs Reqd? No
Grade of Rod = A449

DCR Abs(Max)= 0.282
Iterate Ledge to get DCR <= 1.01 OK

Simpson Strong-Tie
5956 West Las Positas Boulevard
Pleasanton, CA 94588

Date: 3/3/2022
By: bchi

SMF BEAM TOP WOOD NAILER ATTACHMENT DESIGN

BOLT LATERAL DESIGN VALUES

Wood Data

Species: DFL
Main Member Single 4X
Countersink 3/4"

Steel Data

Steel Gauge: 0.5
F_u: 65000

Bolt Data

Bolt Size: 3/4
Angle to Grain 0

ASTM A572 Gr 50 Steel

Bolt Design Information

D	F_{yb}	l_m	R_t	K_θ	R_e	R_d		
						I_m, I_s	II	III_m, III_s, IV
0.75	45000	2.750	5.500	1.00	0.0627	4	3.6	3.2

Steel Design Information

F_u	t_{steel}
65000	0.5

Wood Design Information

t_{wood}	SG	$F_{em(\theta)}$	F_{es}
2.75	0.5	5600	89375

Design Calculations

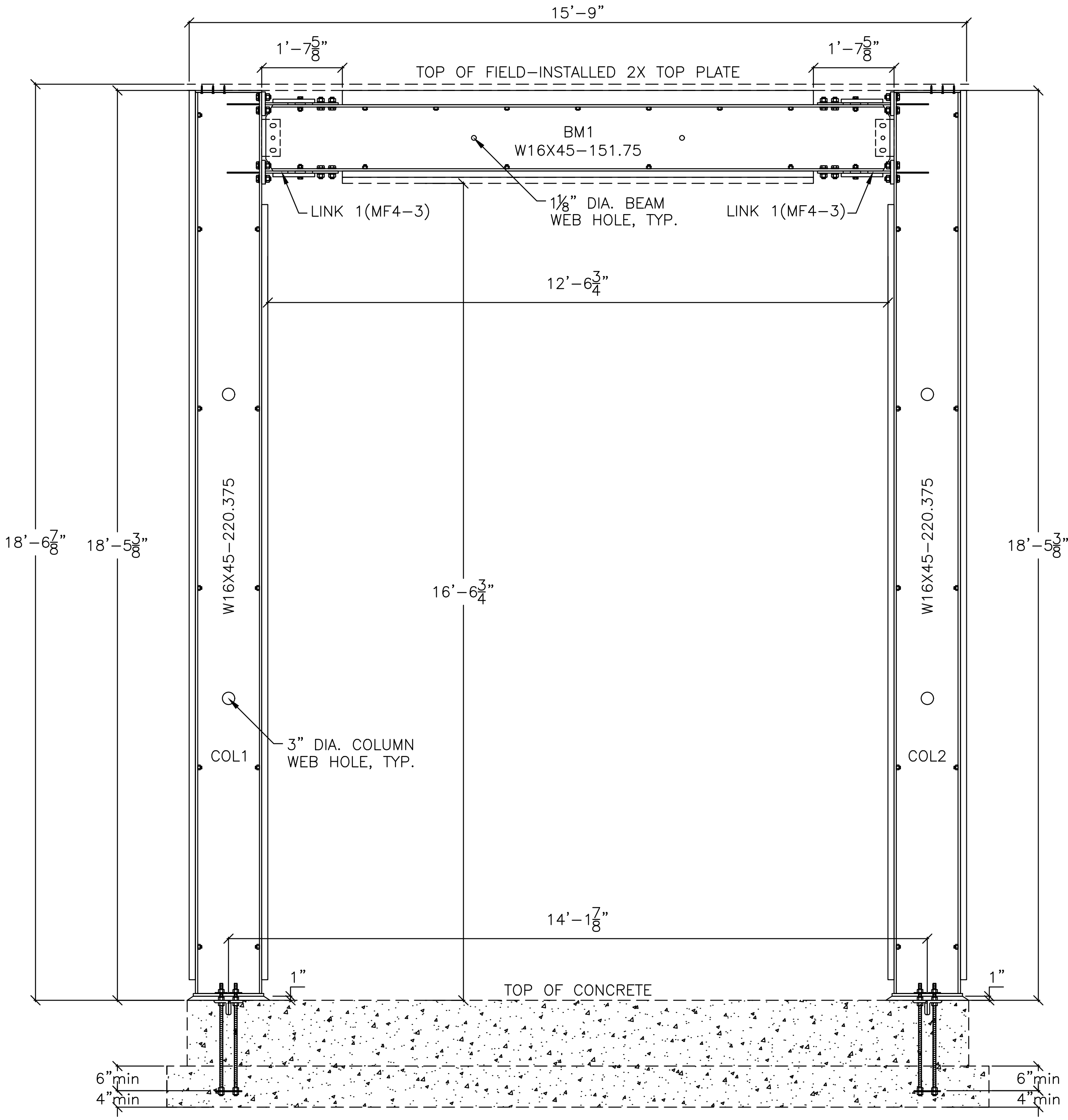
k_1	K_2	K_3	Z (I_m)	Z (I_s):	Z (II):	Z (III_m):	Z (III_s):	Z (IV):
0.180	0.604	6.667	2888	8379	1679	1938	2121	2210

	ALLOWABLE LOADS				
	100%	*115%*	*125%*	*133%*	*160%*
Single Shear:	**1679**	**1930**	**2098**	**2233**	**2686**

ASD Bolt Z_{allow} = **2686 lbs** **Per NDS: 3/4" Diameter Bolts, Single Shear, C_D = 1.60**
1.2 x ASD Bolt Z_{allow} = **3223 lbs** **1.2 Increase for Overstrength Forces per ASCE 7-10 12.4.3.3**

Seismic Lateral Load = 2,689 lbs
Wind Lateral Load = 1,220 lbs
Ωo = **2.5**
Vreq = 6,722 lbs
n_bolts_req'd = 2.09
n_bolt_min (total) = 4 3/4" dia. A307 Bolts

Link = MF4-3
Lbm = 151.75 in
Lnailer = 103.5 in
S_bolt = 48 in
Check = **OK**



Note: Footing sizes shown is based on anchorage design only. Actual footing/grade beam size and reinforcing must be determined by Designer based on project specific geometry and allowable soil pressures.

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Project: PIPER RESIDENCE

Date: 02/27/2022

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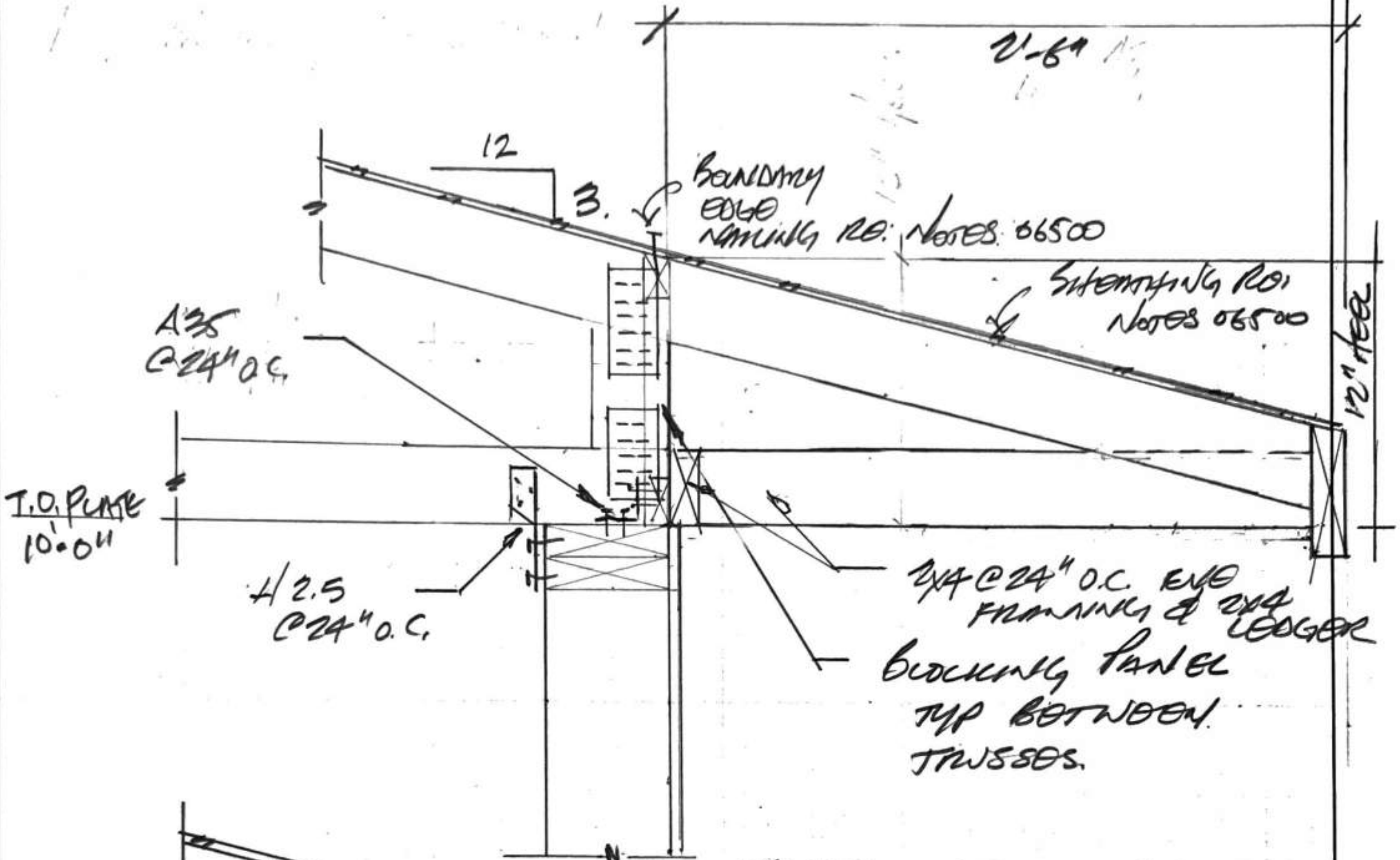
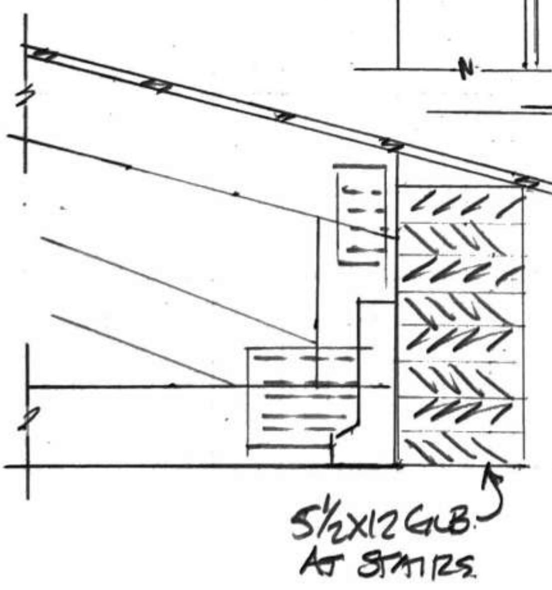
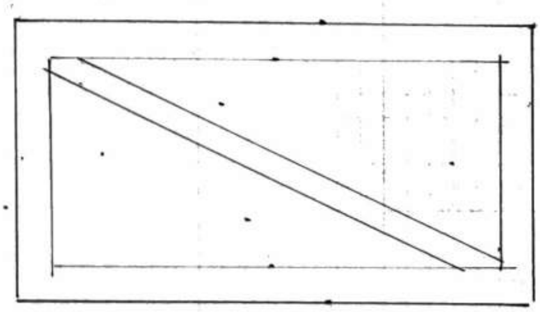


TYPICAL TRUSS @ EVE DETAIL



TYPICAL TRUSS @ 5½"x12 GLB.



BLOCKING PANEL

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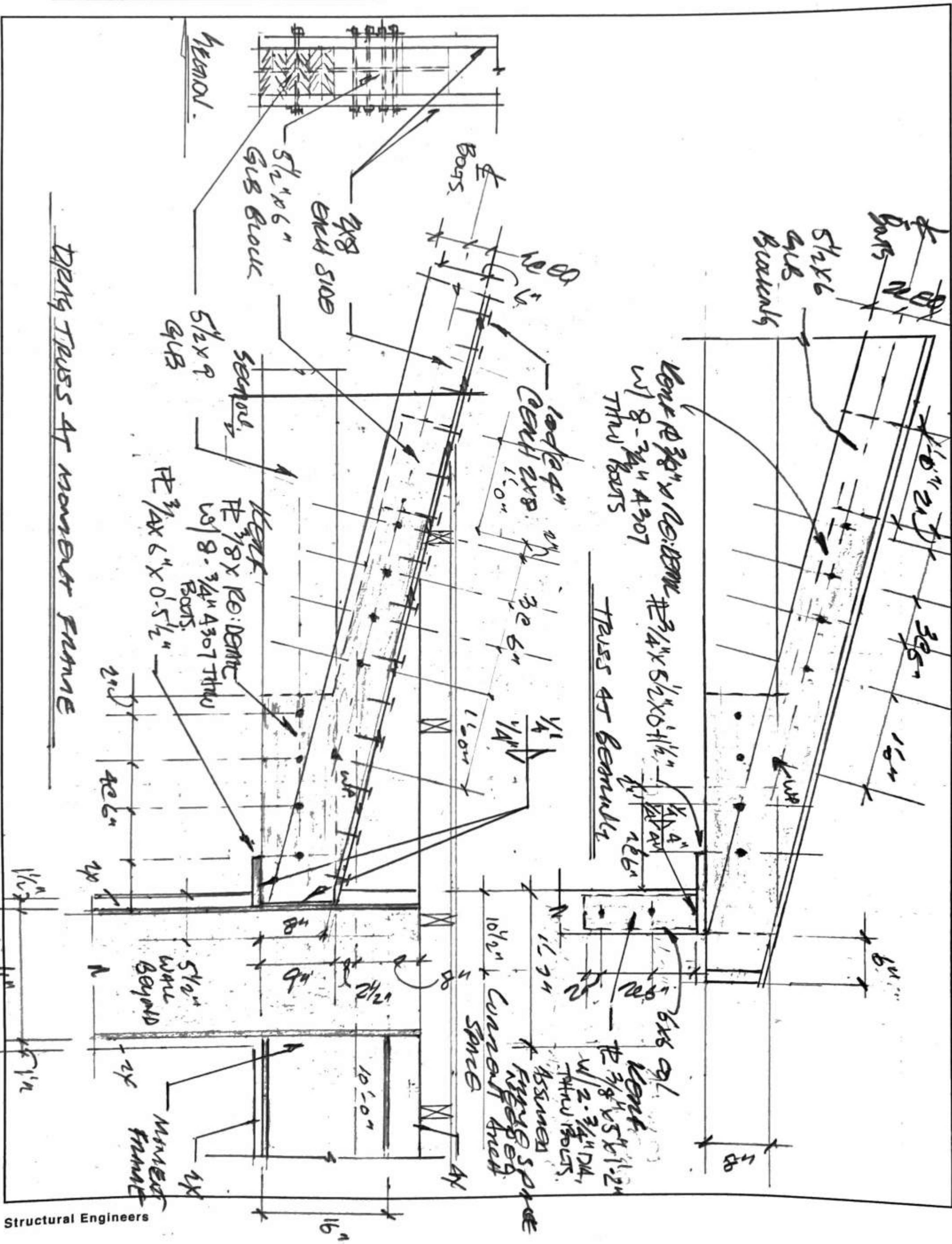

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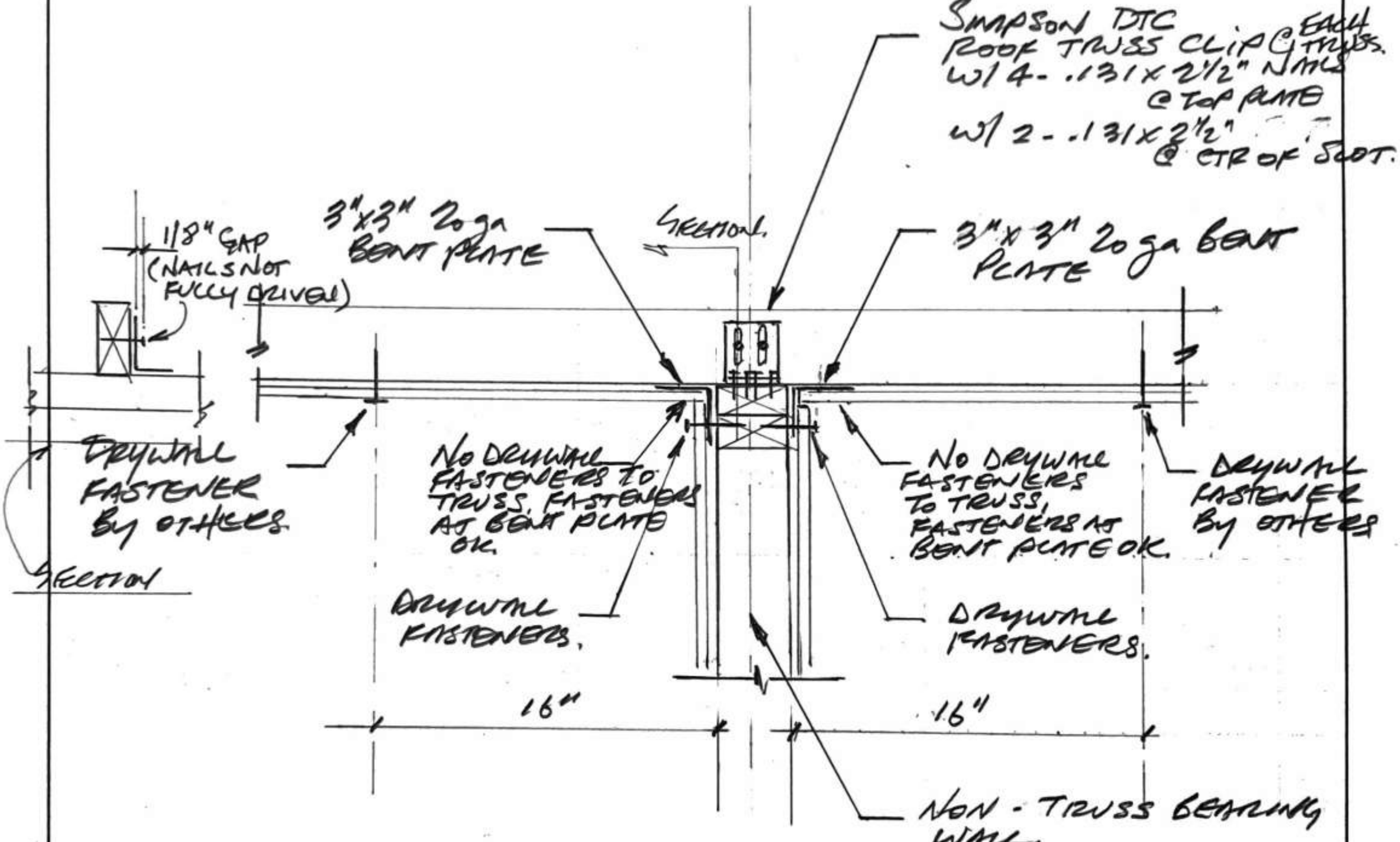
Project: ____________________ Date: 03/03/2022

Client: ____________________ Page Number: ________



TYPICAL NON-BEARING WALL TO TRUSS ATTACHMENT. - WALL TRANSVERSE TO TRUSSES.

1 1/2" = 1'-0"

CT ENGINEERING INC.

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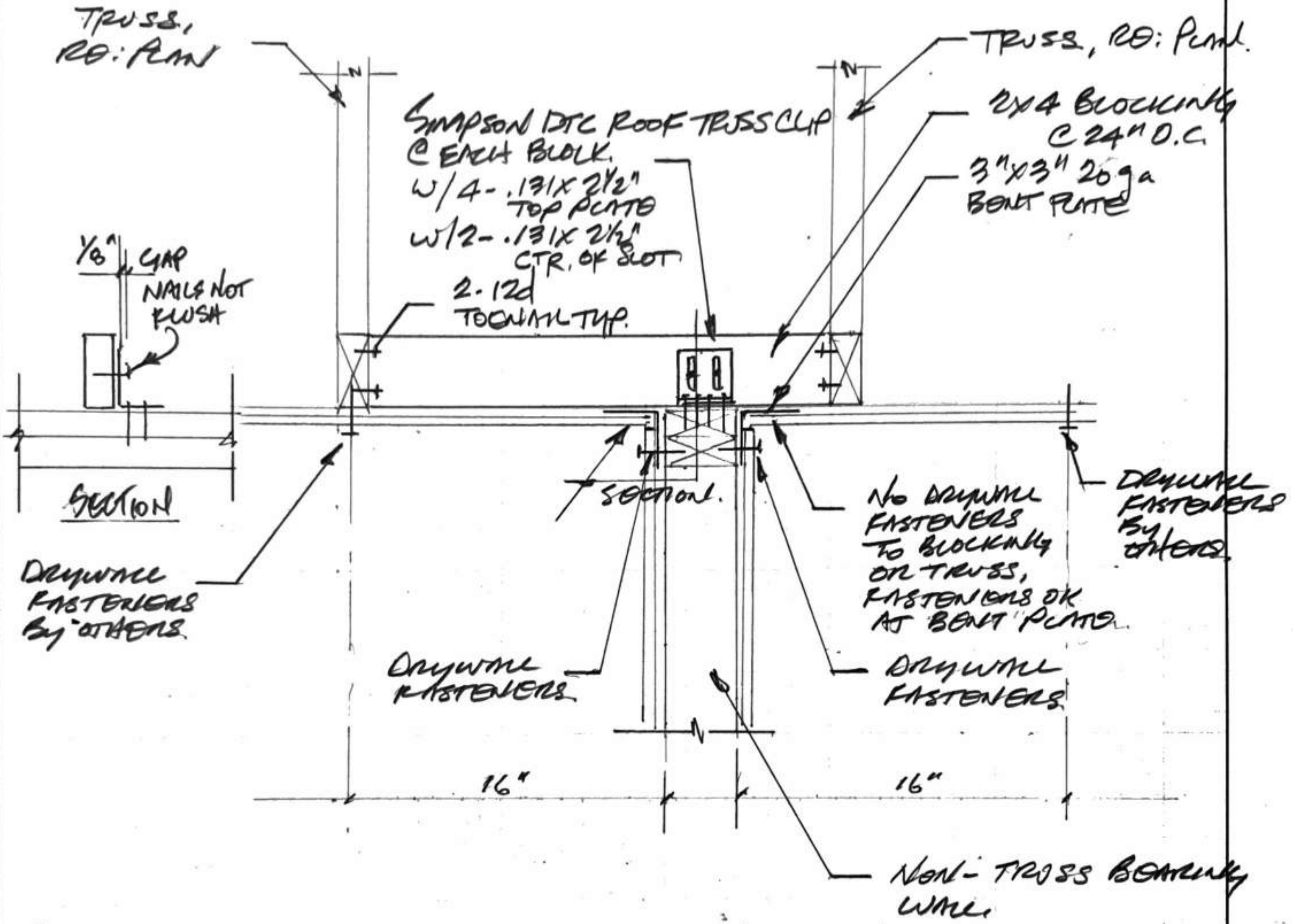

TYPICAL NON-BEARING WALL TO TRUSS ATTACHMENT.
WALL PARALLEL WITH TRUSSES.

1/2" = 1'-0"

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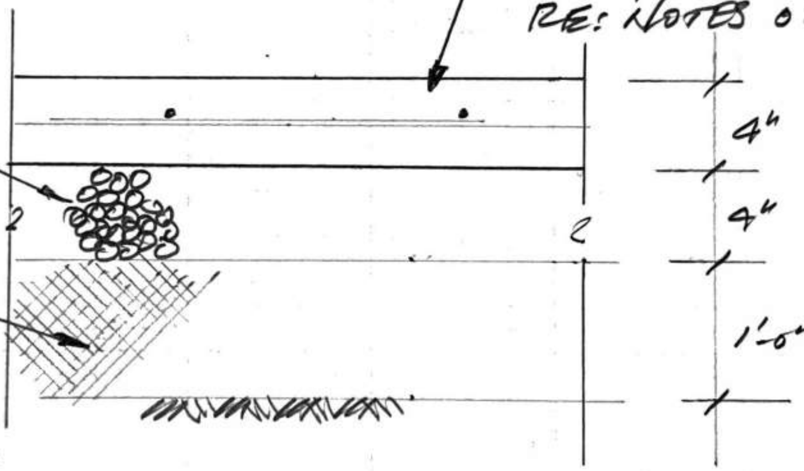
4" CONCRETE SLAB ON GRADE
w/ #4 @ 18" EACH WAY CTR.
RE: NOTES 03000

CRUSHED
ROCK OR
PEA GRAVEL



COMPACT TO
95% OF
MODIFIED
PROCTOR MAX
DRY DENSITY
PER ASTM D 1557
PER GEOTECHNICAL
REPORT BY ZIPPER
GEO.

EXTERIOR SLAB.

CT ENGINEERING INC.

CT # 21162

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Project: PIPER RESIDENCE

Date: 02/27/2022

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