STRUCTURAL DESIGN

FOR

4332 MERCER ISLAND ADDITION

Submitted to: CHU KEN+XU WEI

Date: *12/11/2024*

F.T. Engineering & Construction Management, LLC

T: (509) 822-0489 **E-mail:** F.T.Eng.cm@gmail.com

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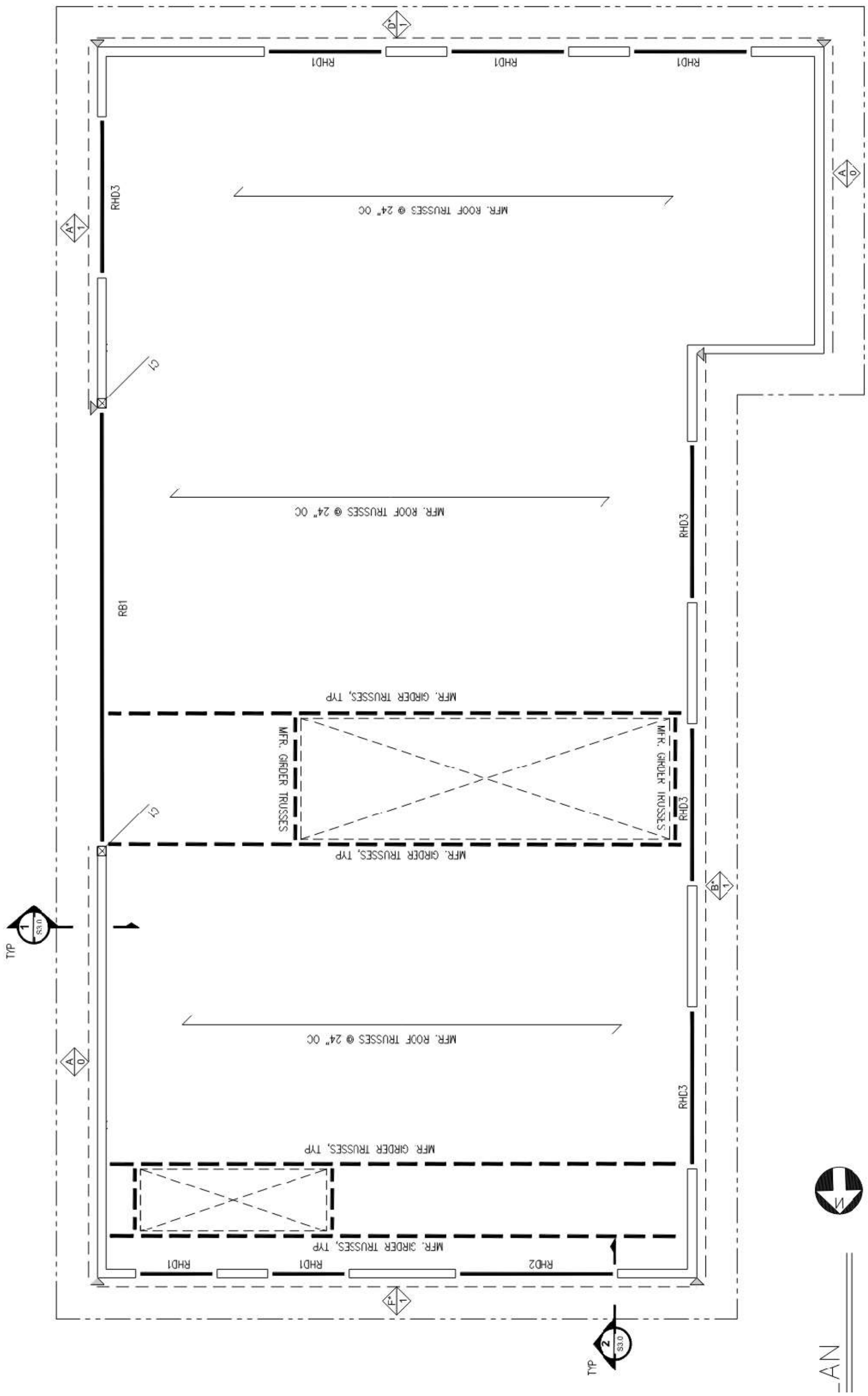
1.0 OBJECTIVE



Job Number: 2024065
Job Name: 4332 Mercer Island Addition
Location: 4332 West Mercer Way, Mercer Island, WA

Engineer: Frankie Tsui
Date: 12/11/2024
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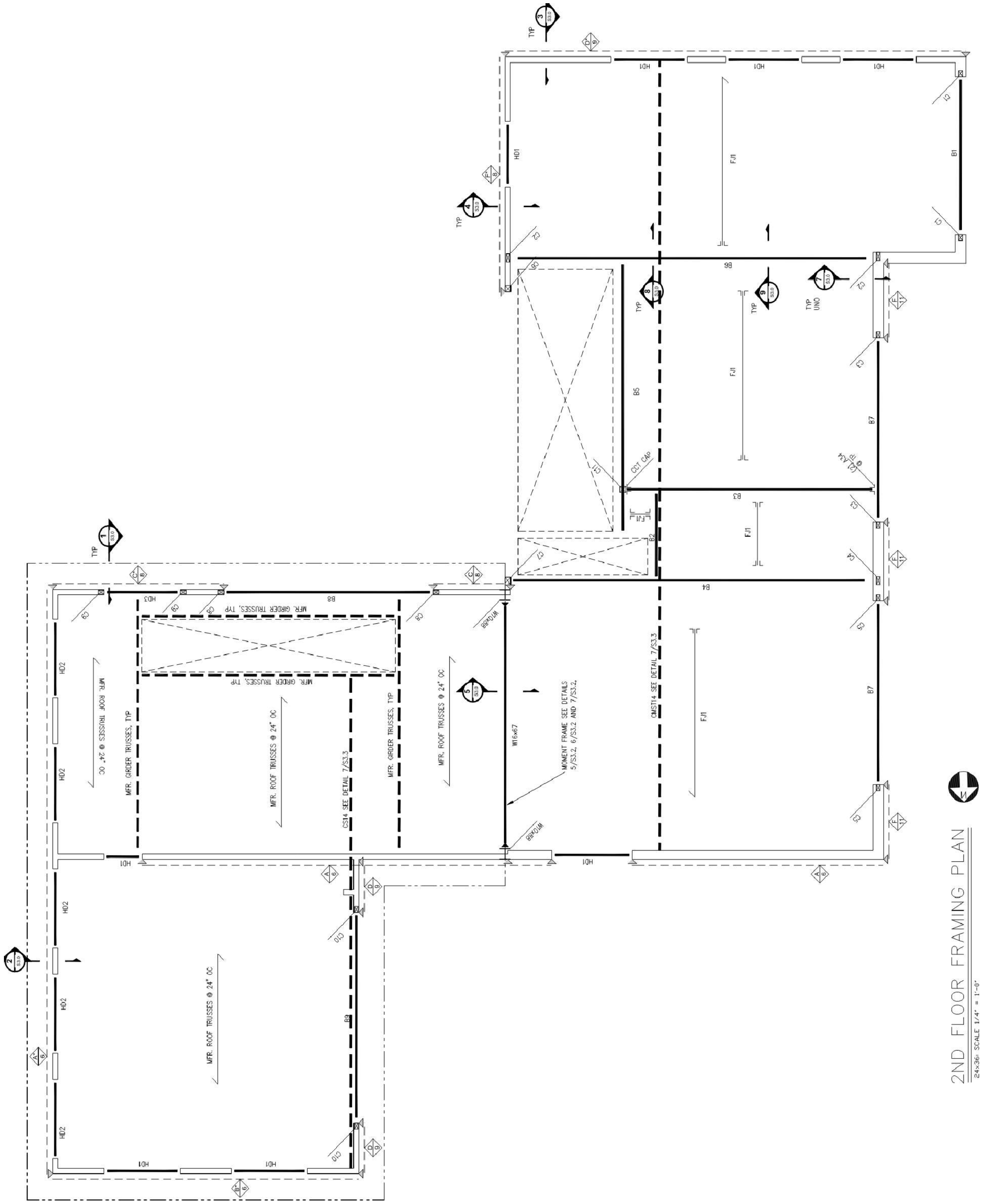


Job Number:	2024065	Engineer:	Frankie Tsui
Job Name:	4332 Mercer Island Addition	Date:	12/11/2024
Location:	4332 West Mercer Way, Mercer Island, WA	Page:	3

2.0 LOAD

Roof live Load =	20	PSF
Floor live load =	40	psf
Deck live Load =	60	psf
Snow Load, P_f =	$0.7C_eC_tI_sP_g$	
C_e =	1	
C_t =	1	
I_s =	1	
P_g =	25	
P_f =	17.5	
Use =	25	psf
Floor Dead Load =	15	psf
Roof Dead Load =	20	psf
Wind Design :		
Design Wind speed =	110	mph
Exp =	B	
Seismic Design :		
Sds =	1.244	(Shear Wall)
R =	6.5	
Ω =	2.5	
R =	3.5	(OMF)
Ω =	3	
Soil Bearing Capacity :		
Assumed Soil Bearing Capacity =	1500	psf

3.0 Garvity Framing Design

Level			
Member Name	**Results (Max UTIL %)**	**Current Solution**	**Comments**
RHD1	Passed (28% M)	1 piece(s) 4 x 8 HF No.2	
RHD2	Passed (46% M)	1 piece(s) 4 x 8 DF No.2	
RHD3	Passed (71% M+)	1 piece(s) 3 1/2" x 9" 24F-V4 DF Glulam	
RB1	Passed (79% ΔT)	1 piece(s) 5 1/2" x 18" 24F-V4 DF Glulam	
Floor: Joist	Passed (91% M)	1 piece(s) 16" TJI® 110 @ 16" OC	
HD1	Passed (80% R)	1 piece(s) 4 x 12 DF No.2	
HD2	Passed (28% M)	1 piece(s) 4 x 8 HF No.2	
HD3	Passed (75% M+)	1 piece(s) 3 1/2" x 9" 24F-V4 DF Glulam	
B1	Passed (84% M+)	1 piece(s) 3 1/2" x 13 1/2" 24F-V4 DF Glulam	
B2	Passed (93% R)	1 piece(s) 3 1/2" x 12" 24F-V4 DF Glulam	
B3	Passed (93% R)	1 piece(s) 5 1/2" x 16" 24F-V4 DF Glulam	
B4	Passed (97% R)	1 piece(s) 8 3/4" x 21" 24F-V4 DF Glulam	
B5	Passed (22% R)	1 piece(s) 5 1/2" x 16" 24F-V4 DF Glulam	
B6	Passed (99% R)	1 piece(s) 8 3/4" x 21" 24F-V4 DF Glulam	
B7	Passed (98% R)	1 piece(s) 3 1/2" x 16 1/2" 24F-V4 DF Glulam	
B8	Passed (87% R)	1 piece(s) 3 1/2" x 12" 24F-V4 DF Glulam	
B9	Passed (60% ΔT)	1 piece(s) 3 1/2" x 10 1/2" 24F-V4 DF Glulam	
6X6 Post	Passed (98% f_{cp})	1 piece(s) 6 x 6 DF No.2	
4X6 Post	Passed (100% f_{cp})	1 piece(s) 4 x 6 DF No.2	
4X8 Post	Passed (97% f_{cp})	1 piece(s) 4 x 8 DF No.2	
6X10 Post	Passed (99% f_{cp})	1 piece(s) 6 x 10 DF No.2	
4X10 Post	Passed (99% f_{cp})	1 piece(s) 4 x 10 DF No.2	

ForteWEB Software Operator	**Job Notes**
Frankie Tsui F.T. Engineering & Construction Management, LLC (509) 822-0489 f.t.eng.cm@gmail.com	

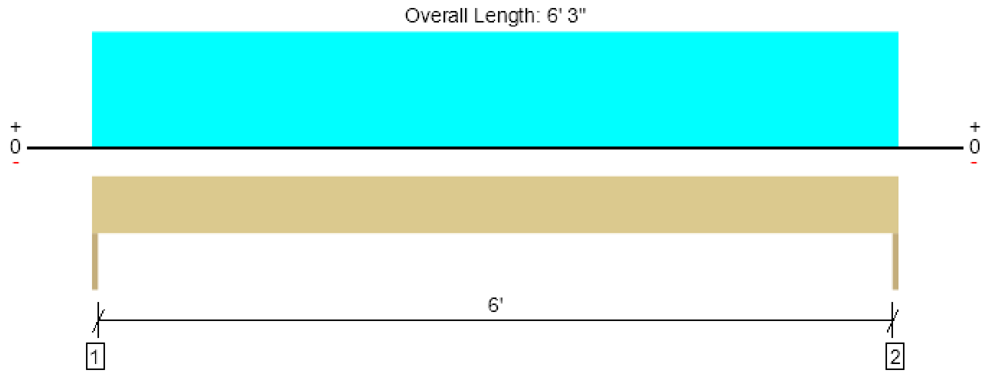
FORTEWEB

MEMBER REPORT

PASSED

Level, RHD1

1 piece(s) 4 x 8 HF No.2



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

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	583 @ 0	2126 (1.50")	Passed (27%)	--	1.0 D + 1.0 S (All Spans)
Shear (lbs)	447 @ 8 3/4"	2918	Passed (15%)	1.15	1.0 D + 1.0 S (All Spans)
Moment (Ft-lbs)	910 @ 3' 1 1/2"	3247	Passed (28%)	1.15	1.0 D + 1.0 S (All Spans)
Live Load Defl. (in)	0.024 @ 3' 1 1/2"	0.208	Passed (L/999+)	--	1.0 D + 1.0 S (All Spans)
Total Load Defl. (in)	0.044 @ 3' 1 1/2"	0.313	Passed (L/999+)	--	1.0 D + 1.0 S (All Spans)

Member Length : 6' 3"
System : Wall
Member Type : Header
Building Use : Residential
Building Code : IBC 2021
Design Methodology : ASD

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

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Snow	Factored	
1 - Trimmer - HF	1.50"	1.50"	1.50"	270	313	583	None
2 - Trimmer - HF	1.50"	1.50"	1.50"	270	313	583	None

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

•Maximum allowable bracing intervals based on applied load.

Vertical Loads	Location	Tributary Width	Dead (0.90)	Snow (1.15)	Comments
0 - Self Weight (PLF)	0 to 6' 3"	N/A	6.4	--	
1 - Uniform (PSF)	0 to 6' 3"	4'	20.0	25.0	Default Load

Weyerhaeuser Notes

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

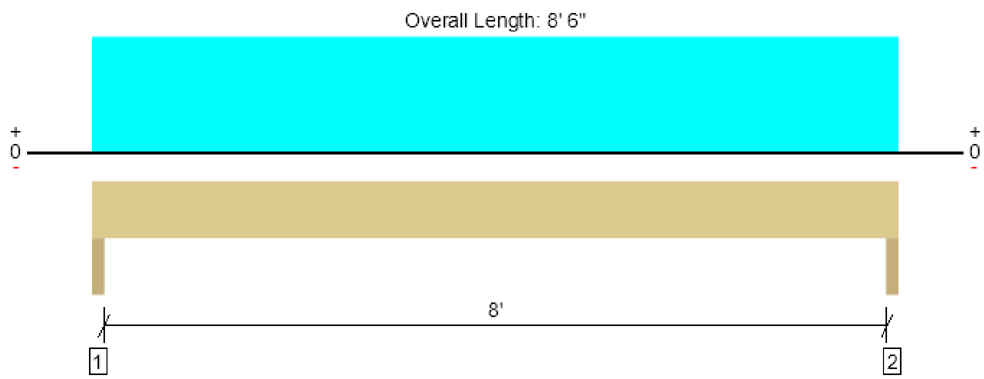
ForteWEB Software Operator	Job Notes
Frankie Tsui F.T. Engineering & Construction Management, LLC (509) 822-0489 f.t.eng.cm@gmail.com	

Weyerhaeuser

12/11/2024 4:35:41 PM UTC
ForteWEB v3.8, Engine: V8.4.1.24, Data: V8.1.6.3
File Name: 2024065 - 4332 Mercer Island Addition

Level, RHD2

1 piece(s) 4 x 8 DF No.2



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Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	792 @ 1 1/2"	6563 (3.00")	Passed (12%)	--	1.0 D + 1.0 S (All Spans)
Shear (lbs)	633 @ 10 1/4"	3502	Passed (18%)	1.15	1.0 D + 1.0 S (All Spans)
Moment (Ft-lbs)	1586 @ 4' 3"	3438	Passed (46%)	1.15	1.0 D + 1.0 S (All Spans)
Live Load Defl. (in)	0.059 @ 4' 3"	0.275	Passed (L/999+)	--	1.0 D + 1.0 S (All Spans)
Total Load Defl. (in)	0.109 @ 4' 3"	0.313	Passed (L/906)	--	1.0 D + 1.0 S (All Spans)

Member Length : 8' 6"
System : Wall
Member Type : Header
Building Use : Residential
Building Code : IBC 2021
Design Methodology : ASD

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

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Snow	Factored	
1 - Trimmer - HF	3.00"	3.00"	1.50"	367	425	792	None
2 - Trimmer - HF	3.00"	3.00"	1.50"	367	425	792	None

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	8' 6" o/c	
Bottom Edge (Lu)	8' 6" o/c	

•Maximum allowable bracing intervals based on applied load.

Vertical Loads	Location	Tributary Width	Dead (0.90)	Snow (1.15)	Comments
0 - Self Weight (PLF)	0 to 8' 6"	N/A	6.4	--	
1 - Uniform (PSF)	0 to 8' 6"	4'	20.0	25.0	Default Load

Weyerhaeuser Notes

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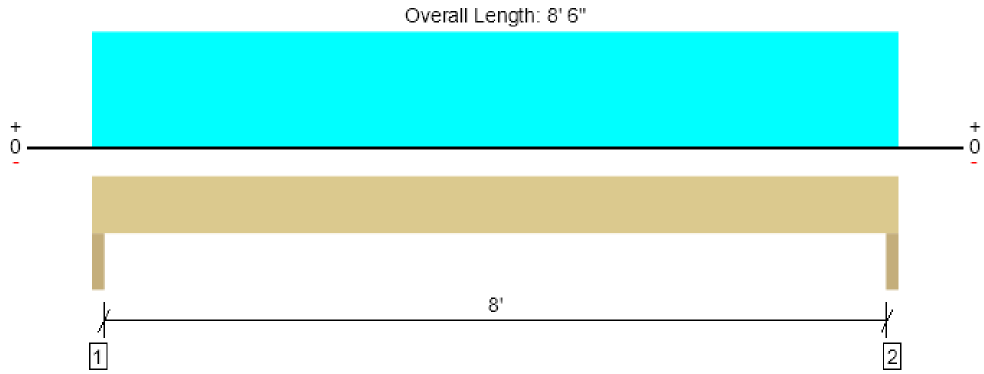
The product application, input design loads, dimensions and support information have been provided by ForteWEB Software Operator

ForteWEB Software Operator	Job Notes
Frankie Tsui F.T. Engineering & Construction Management, LLC (509) 822-0489 f.t.eng.cm@gmail.com	

Level, RHD3

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



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Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	3858 @ 1 1/2"	6825 (3.00")	Passed (57%)	--	1.0 D + 1.0 S (All Spans)
Shear (lbs)	2950 @ 1'	6400	Passed (46%)	1.15	1.0 D + 1.0 S (All Spans)
Pos Moment (Ft-lbs)	7722 @ 4' 3"	10868	Passed (71%)	1.15	1.0 D + 1.0 S (All Spans)
Live Load Defl. (in)	0.136 @ 4' 3"	0.275	Passed (L/727)	--	1.0 D + 1.0 S (All Spans)
Total Load Defl. (in)	0.247 @ 4' 3"	0.412	Passed (L/400)	--	1.0 D + 1.0 S (All Spans)

Member Length : 8' 6"
System : Wall
Member Type : Header
Building Use : Residential
Building Code : IBC 2021
Design Methodology : ASD

- Deflection criteria: LL (L/360) and TL (L/240).
- Allowed moment does not reflect the adjustment for the beam stability factor.
- Critical positive moment adjusted by a volume/size factor of 1.00 that was calculated using length L = 8' 3".
- The effects of positive or negative camber have not been accounted for when calculating deflection.
- The specified glulam is assumed to have its strong laminations at the bottom of the beam. Install with proper side up as indicated by the manufacturer.
- Applicable calculations are based on NDS.

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Snow	Factored	
1 - Trimmer - HF	3.00"	3.00"	1.70"	1733	2125	3858	None
2 - Trimmer - HF	3.00"	3.00"	1.70"	1733	2125	3858	None

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	8' 6" o/c	
Bottom Edge (Lu)	8' 6" o/c	

•Maximum allowable bracing intervals based on applied load.

Vertical Loads	Location	Tributary Width	Dead (0.90)	Snow (1.15)	Comments
0 - Self Weight (PLF)	0 to 8' 6"	N/A	7.7	--	
1 - Uniform (PSF)	0 to 8' 6"	20'	20.0	25.0	Default Load

Weyerhaeuser Notes

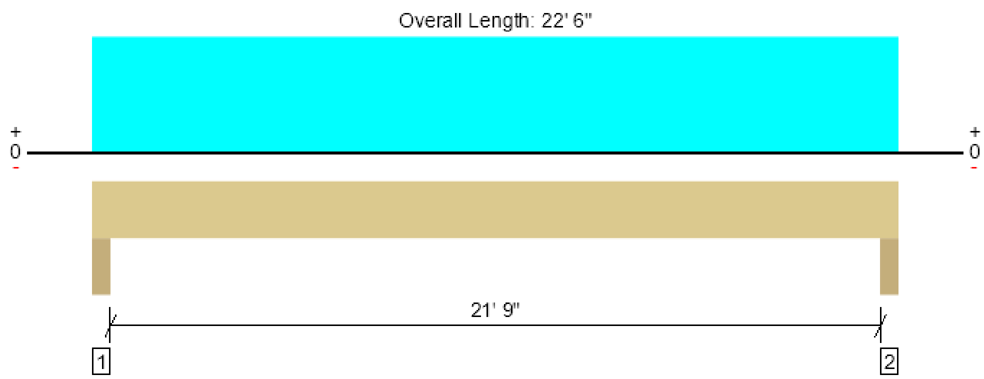
ForteWEB Software Operator	Job Notes
Frankie Tsui F.T. Engineering & Construction Management, LLC (509) 822-0489 f.t.eng.cm@gmail.com	

MEMBER REPORT

PASSED

Level, RB1

1 piece(s) 5 1/2" x 18" 24F-V4 DF Glulam



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Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	8877 @ 3"	16088 (4.50")	Passed (55%)	--	1.0 D + 1.0 S (All Spans)
Shear (lbs)	7397 @ 1' 10 1/2"	20114	Passed (37%)	1.15	1.0 D + 1.0 S (All Spans)
Pos Moment (Ft-lbs)	47738 @ 11' 3"	64832	Passed (74%)	1.15	1.0 D + 1.0 S (All Spans)
Live Load Defl. (in)	0.466 @ 11' 3"	0.733	Passed (L/567)	--	1.0 D + 1.0 S (All Spans)
Total Load Defl. (in)	0.864 @ 11' 3"	1.100	Passed (L/305)	--	1.0 D + 1.0 S (All Spans)

Member Length : 22' 6"
System : Wall
Member Type : Header
Building Use : Residential
Building Code : IBC 2021
Design Methodology : ASD

- Deflection criteria: LL (L/360) and TL (L/240).
- Allowed moment does not reflect the adjustment for the beam stability factor.
- Critical positive moment adjusted by a volume/size factor of 0.95 that was calculated using length L = 22'.
- The effects of positive or negative camber have not been accounted for when calculating deflection.
- The specified glulam is assumed to have its strong laminations at the bottom of the beam. Install with proper side up as indicated by the manufacturer.
- Applicable calculations are based on NDS.

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Snow	Factored	
1 - Trimmer - HF	4.50"	4.50"	2.48"	4096	4781	8877	None
2 - Trimmer - HF	4.50"	4.50"	2.48"	4096	4781	8877	None

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	22' 6" o/c	
Bottom Edge (Lu)	22' 6" o/c	

•Maximum allowable bracing intervals based on applied load.

Vertical Loads	Location	Tributary Width	Dead (0.90)	Snow (1.15)	Comments
0 - Self Weight (PLF)	0 to 22' 6"	N/A	24.1	--	
1 - Uniform (PSF)	0 to 22' 6"	17'	20.0	25.0	Default Load

Weyerhaeuser Notes

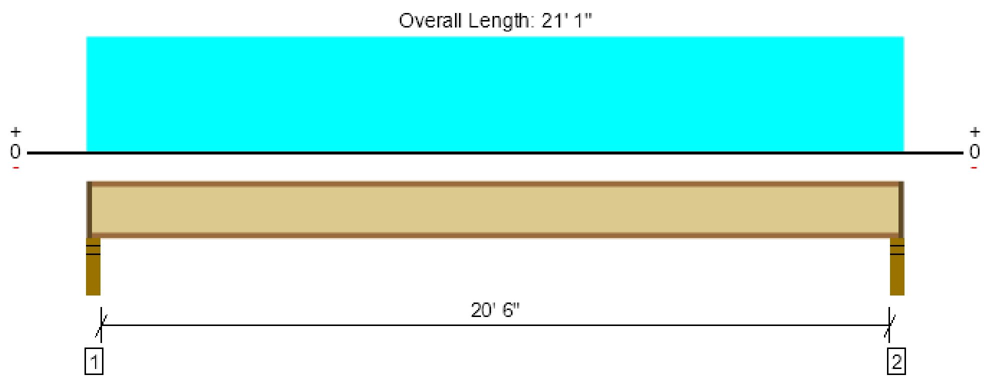
ForteWEB Software Operator	Job Notes
Frankie Tsui F.T. Engineering & Construction Management, LLC (509) 822-0489 f.t.eng.cm@gmail.com	

MEMBER REPORT

PASSED

Level, Floor: Joist

1 piece(s) 16" TJI® 110 @ 16" OC



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Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	765 @ 2 1/2"	1041 (2.25")	Passed (73%)	1.00	1.0 D + 1.0 L (All Spans)
Shear (lbs)	752 @ 3 1/2"	2145	Passed (35%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	3915 @ 10' 6 1/2"	4280	Passed (91%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.369 @ 10' 6 1/2"	0.517	Passed (L/671)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.508 @ 10' 6 1/2"	1.033	Passed (L/488)	--	1.0 D + 1.0 L (All Spans)
TJ-Pro™ Rating	40	40	Passed	--	--

Member Length : 20' 10 1/2"
System : Floor
Member Type : Joist
Building Use : Residential
Building Code : IBC 2021
Design Methodology : ASD

- Deflection criteria: LL (L/480) and TL (L/240).
- Allowed moment does not reflect the adjustment for the beam stability factor.
- A structural analysis of the deck has not been performed.
- Deflection analysis is based on composite action with a single layer of 23/32" Weyerhaeuser Edge™ Panel (24" Span Rating) that is glued and nailed down.
- Additional considerations for the TJ-Pro™ Rating include: None.

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Floor Live	Factored	
1 - Stud wall - SPF	3.50"	2.25"	1.75"	211	562	773	1 1/4" Rim Board
2 - Stud wall - SPF	3.50"	2.25"	1.75"	211	562	773	1 1/4" Rim Board

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

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	3' 3" o/c	
Bottom Edge (Lu)	20' 11" o/c	

•TJI joists are only analyzed using Maximum Allowable bracing solutions.

•Maximum allowable bracing intervals based on applied load.

Vertical Load	Location	Spacing	Dead (0.90)	Floor Live (1.00)	Comments
1 - Uniform (PSF)	0 to 21' 1"	16"	15.0	40.0	Default Load

Weyerhaeuser Notes

ForteWEB Software Operator	Job Notes
Frankie Tsui F.T. Engineering & Construction Management, LLC (509) 822-0489 f.t.eng.cm@gmail.com	

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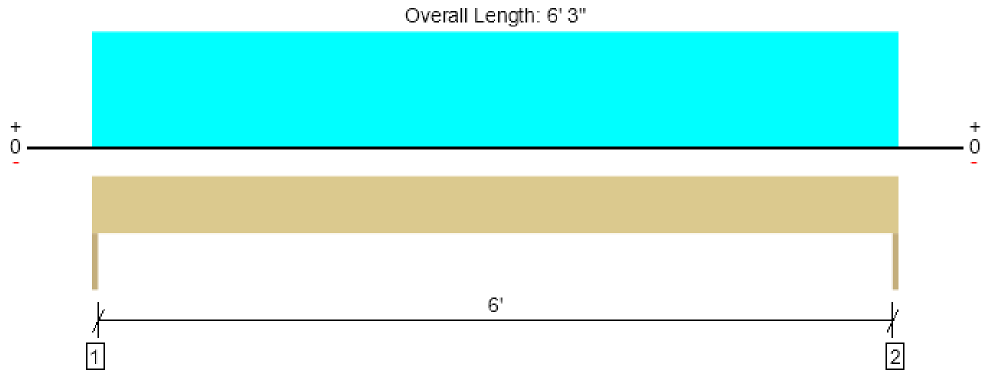
12/11/2024 4:35:41 PM UTC
ForteWEB v3.8, Engine: V8.4.1.24, Data: V8.1.6.3
File Name: 2024065 - 4332 Mercer Island Addition

Level, HD1

1 piece(s) 4 x 12 DF No.2



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Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	2641 @ 0	3281 (1.50")	Passed (80%)	--	1.0 D + 1.0 L (All Spans)
Shear (lbs)	1743 @ 1' 3/4"	4725	Passed (37%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	4126 @ 3' 1 1/2"	6091	Passed (68%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.023 @ 3' 1 1/2"	0.208	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.044 @ 3' 1 1/2"	0.313	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)

Member Length : 6' 3"
System : Wall
Member Type : Header
Building Use : Residential
Building Code : IBC 2021
Design Methodology : ASD

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

Supports	Bearing Length			Loads to Supports (lbs)				Accessories
	Total	Available	Required	Dead	Floor Live	Snow	Factored	
1 - Trimmer - HF	1.50"	1.50"	1.50"	1266	1375	313	2641	None
2 - Trimmer - HF	1.50"	1.50"	1.50"	1266	1375	313	2641	None

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

•Maximum allowable bracing intervals based on applied load.

Vertical Loads	Location	Tributary Width	Dead (0.90)	Floor Live (1.00)	Snow (1.15)	Comments
0 - Self Weight (PLF)	0 to 6' 3"	N/A	10.0	--	--	
1 - Uniform (PSF)	0 to 6' 3"	4'	20.0	-	25.0	Default Load
2 - Uniform (PSF)	0 to 6' 3"	11'	15.0	40.0	-	Floor
3 - Uniform (PLF)	0 to 6' 3"	N/A	150.0	-	-	Wall

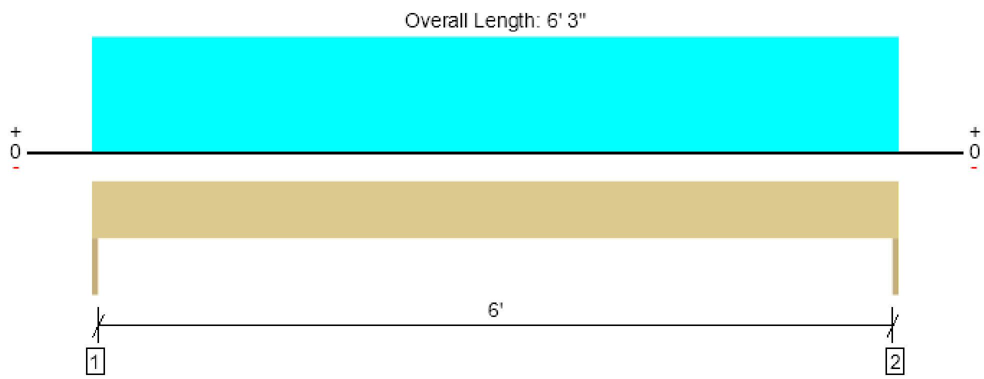
Weyerhaeuser Notes

ForteWEB Software Operator	Job Notes
Frankie Tsui F.T. Engineering & Construction Management, LLC (509) 822-0489 f.t.eng.cm@gmail.com	

Weyerhaeuser

Level, HD2

1 piece(s) 4 x 8 HF No.2



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Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	583 @ 0	2126 (1.50")	Passed (27%)	--	1.0 D + 1.0 S (All Spans)
Shear (lbs)	447 @ 8 3/4"	2918	Passed (15%)	1.15	1.0 D + 1.0 S (All Spans)
Moment (Ft-lbs)	910 @ 3' 1 1/2"	3247	Passed (28%)	1.15	1.0 D + 1.0 S (All Spans)
Live Load Defl. (in)	0.024 @ 3' 1 1/2"	0.208	Passed (L/999+)	--	1.0 D + 1.0 S (All Spans)
Total Load Defl. (in)	0.044 @ 3' 1 1/2"	0.313	Passed (L/999+)	--	1.0 D + 1.0 S (All Spans)

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

Member Length : 6' 3"
System : Wall
Member Type : Header
Building Use : Residential
Building Code : IBC 2021
Design Methodology : ASD

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Snow	Factored	
1 - Trimmer - HF	1.50"	1.50"	1.50"	270	313	583	None
2 - Trimmer - HF	1.50"	1.50"	1.50"	270	313	583	None

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

•Maximum allowable bracing intervals based on applied load.

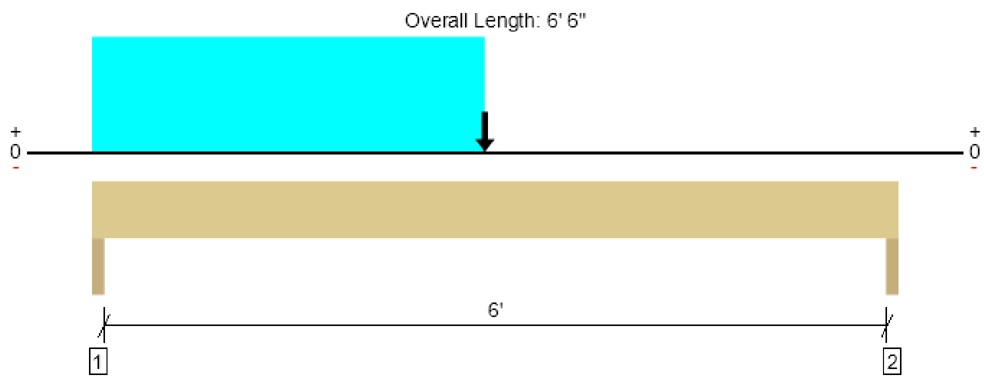
Vertical Loads	Location	Tributary Width	Dead (0.90)	Snow (1.15)	Comments
0 - Self Weight (PLF)	0 to 6' 3"	N/A	6.4	--	
1 - Uniform (PSF)	0 to 6' 3"	4'	20.0	25.0	Default Load

Weyerhaeuser Notes

ForteWEB Software Operator	Job Notes
Frankie Tsui F.T. Engineering & Construction Management, LLC (509) 822-0489 f.t.eng.cm@gmail.com	

Level, HD3

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



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

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	4410 @ 1 1/2"	6825 (3.00")	Passed (65%)	--	1.0 D + 1.0 S (All Spans)
Shear (lbs)	3367 @ 1'	6400	Passed (53%)	1.15	1.0 D + 1.0 S (All Spans)
Pos Moment (Ft-lbs)	8194 @ 3' 2"	10868	Passed (75%)	1.15	1.0 D + 1.0 S (All Spans)
Live Load Defl. (in)	0.072 @ 3' 1 13/16"	0.208	Passed (L/999+)	--	1.0 D + 1.0 S (All Spans)
Total Load Defl. (in)	0.129 @ 3' 1 13/16"	0.313	Passed (L/581)	--	1.0 D + 1.0 S (All Spans)

Member Length : 6' 6"
System : Wall
Member Type : Header
Building Use : Residential
Building Code : IBC 2021
Design Methodology : ASD

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

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Snow	Factored	
1 - Trimmer - HF	3.00"	3.00"	1.94"	1962	2448	4410	None
2 - Trimmer - HF	3.00"	3.00"	1.50"	1144	1423	2567	None

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

•Maximum allowable bracing intervals based on applied load.

Vertical Loads	Location	Tributary Width	Dead (0.90)	Snow (1.15)	Comments
0 - Self Weight (PLF)	0 to 6' 6"	N/A	7.7	--	
1 - Uniform (PSF)	0 to 3' 2"	23'	20.0	25.0	Default Load
2 - Point (lb)	3' 2"	N/A	1600	2050	Truss

ForteWEB Software Operator	Job Notes
Frankie Tsui F.T. Engineering & Construction Management, LLC (509) 822-0489 f.t.eng.cm@gmail.com	

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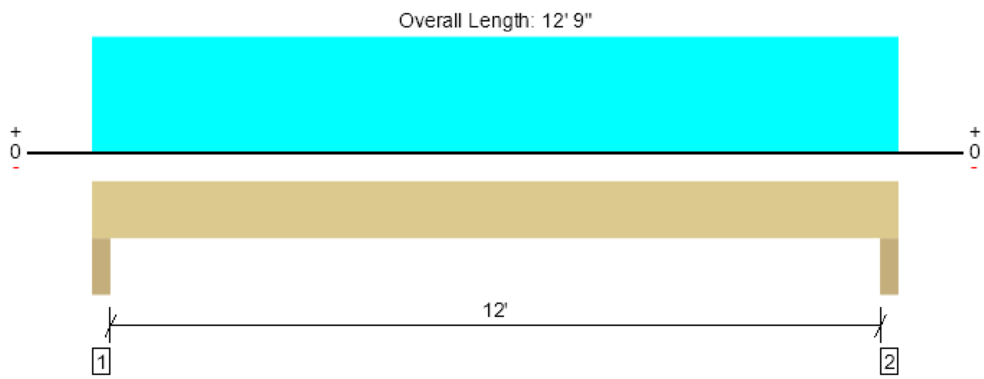
Level, B1

1 piece(s) 3 1/2" x 13 1/2" 24F-V4 DF Glulam



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

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	6958 @ 3"	10238 (4.50")	Passed (68%)	--	1.0 D + 1.0 S (All Spans)
Shear (lbs)	5321 @ 1' 6"	9600	Passed (55%)	1.15	1.0 D + 1.0 S (All Spans)
Pos Moment (Ft-lbs)	20474 @ 6' 4 1/2"	24452	Passed (84%)	1.15	1.0 D + 1.0 S (All Spans)
Live Load Defl. (in)	0.196 @ 6' 4 1/2"	0.408	Passed (L/750)	--	1.0 D + 1.0 S (All Spans)
Total Load Defl. (in)	0.428 @ 6' 4 1/2"	0.613	Passed (L/343)	--	1.0 D + 1.0 S (All Spans)

Member Length : 12' 9"
System : Wall
Member Type : Header
Building Use : Residential
Building Code : IBC 2021
Design Methodology : ASD

- Deflection criteria: LL (L/360) and TL (L/240).
- Allowed moment does not reflect the adjustment for the beam stability factor.
- Critical positive moment adjusted by a volume/size factor of 1.00 that was calculated using length L = 12' 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.

	Bearing Length			Loads to Supports (lbs)				
Supports	Total	Available	Required	Dead	Floor Live	Snow	Factored	Accessories
1 - Trimmer - SPF	4.50"	4.50"	3.06"	3771	510	3187	6958	None
2 - Trimmer - SPF	4.50"	4.50"	3.06"	3771	510	3187	6958	None

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

•Maximum allowable bracing intervals based on applied load.

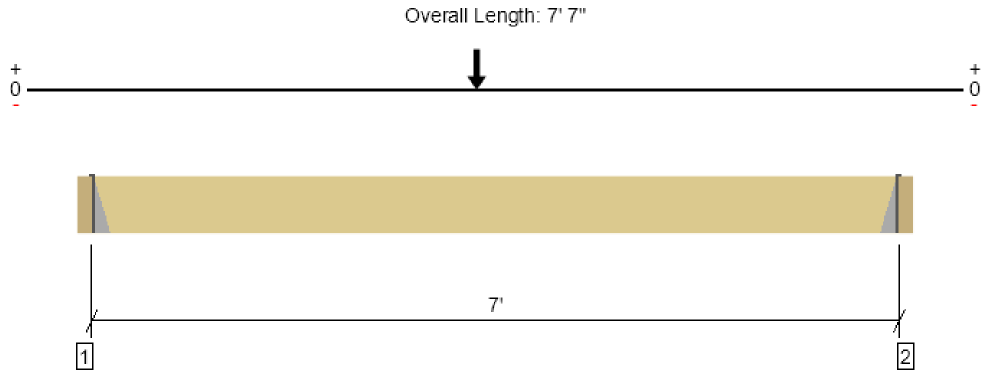
Vertical Loads	Location	Tributary Width	Dead (0.90)	Floor Live (1.00)	Snow (1.15)	Comments
0 - Self Weight (PLF)	0 to 12' 9"	N/A	11.5	--	--	
1 - Uniform (PSF)	0 to 12' 9"	20'	20.0	-	25.0	Roof
2 - Uniform (PSF)	0 to 12' 9"	2'	15.0	40.0	-	FLoor
3 - Uniform (PLF)	0 to 12' 9"	N/A	150.0	-	-	Wall

Weyerhaeuser Notes

ForteWEB Software Operator	Job Notes
Frankie Tsui F.T. Engineering & Construction Management, LLC (509) 822-0489 f.t.eng.cm@gmail.com	

Level, B2

1 piece(s) 3 1/2" x 12" 24F-V4 DF Glulam



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

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	3179 @ 3 1/2"	3413 (1.50")	Passed (93%)	--	1.0 D + 1.0 L (All Spans)
Shear (lbs)	3168 @ 1' 3 1/2"	7420	Passed (43%)	1.00	1.0 D + 1.0 L (All Spans)
Pos Moment (Ft-lbs)	10539 @ 3' 7 1/2"	16800	Passed (63%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.065 @ 3' 9 1/8"	0.233	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.082 @ 3' 9 1/8"	0.350	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)

Member Length : 7'
System : Floor
Member Type : Flush Beam
Building Use : Residential
Building Code : IBC 2021
Design Methodology : ASD

- Deflection criteria: LL (L/360) and TL (L/240).
- Allowed moment does not reflect the adjustment for the beam stability factor.
- Critical positive moment adjusted by a volume/size factor of 1.00 that was calculated using length L = 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	Roof Live	Factored	
1 - Hanger on 12" HF beam	3.50"	Hanger[1]	1.50"	664	2514	10	3179	See note [1]
2 - Hanger on 12" HF beam	3.50"	Hanger[1]	1.50"	607	2286	10	2893	See note [1]

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

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	7' o/c	
Bottom Edge (Lu)	7' 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 - Top Mount Hanger	BA3.56X H=12	3.00"	6-10d	10-10d	8-10dx1.5	
2 - Top Mount Hanger	THA426	1.75"	4-16d	6-16d	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)	Roof Live (1.25)	Comments
0 - Self Weight (PLF)	3 1/2" to 7' 3 1/2"	N/A	10.2	--	--	
1 - Point (lb)	3' 7 1/2" (Front)	N/A	1200	4800	20	Stair

- Side loads are assumed to not induce cross-grain tension.

ForteWEB Software Operator	Job Notes
Frankie Tsui F.T. Engineering & Construction Management, LLC (509) 822-0489 f.t.eng.cm@gmail.com	

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12/11/2024 4:35:41 PM UTC
ForteWEB v3.8, Engine: V8.4.1.24, Data: V8.1.6.3
File Name: 2024065 - 4332 Mercer Island Addition

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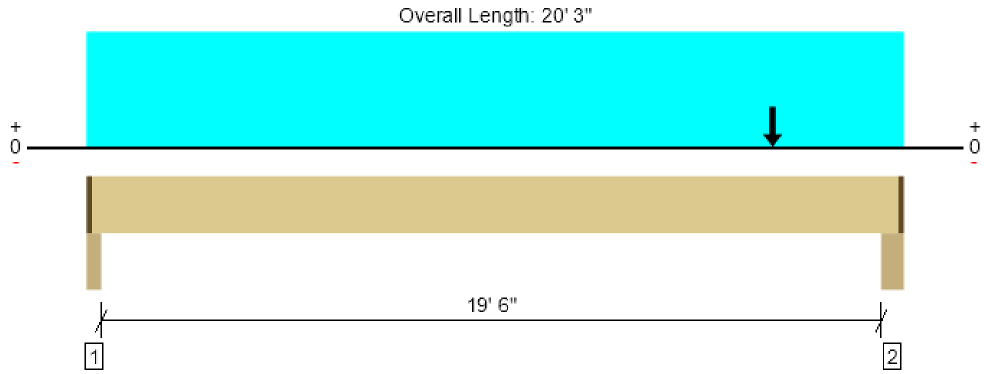
The product application, input design loads, dimensions and support information have been provided by ForteWEB Software Operator

Level, B3

1 piece(s) 5 1/2" x 16" 24F-V4 DF Glulam



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

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	7488 @ 2"	8044 (2.25")	Passed (93%)	--	1.0 D + 1.0 L (All Spans)
Shear (lbs)	8523 @ 18' 5 1/2"	15547	Passed (55%)	1.00	1.0 D + 1.0 L (All Spans)
Pos Moment (Ft-lbs)	39077 @ 10' 8"	45560	Passed (86%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.592 @ 10' 2 3/4"	0.658	Passed (L/401)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.824 @ 10' 2 1/2"	0.988	Passed (L/288)	--	1.0 D + 1.0 L (All Spans)

Member Length : 20' 1/2"
System : Floor
Member Type : Flush Beam
Building Use : Residential
Building Code : IBC 2021
Design Methodology : ASD

- Deflection criteria: LL (L/360) and TL (L/240).
- Allowed moment does not reflect the adjustment for the beam stability factor.
- Critical positive moment adjusted by a volume/size factor of 0.97 that was calculated using length L = 19' 9".
- The effects of positive or negative camber have not been accounted for when calculating deflection.
- The specified glulam is assumed to have its strong laminations at the bottom of the beam. Install with proper side up as indicated by the manufacturer.
- Applicable calculations are based on NDS.

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Floor Live	Factored	
1 - Column - HF	3.50"	2.25"	2.09"	2184	5375	7559	1 1/4" Rim Board
2 - Column - HF	5.50"	4.25"	2.72"	2642	7150	9791	1 1/4" Rim Board

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

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	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)	Comments
0 - Self Weight (PLF)	1 1/4" to 20' 1 3/4"	N/A	21.4	--	
1 - Point (lb)	17' (Front)	N/A	600	2400	Stair
2 - Uniform (PSF)	0 to 20' 3" (Front)	12' 6"	15.0	40.0	floor

• Side loads are assumed to not induce cross-grain tension.

Weyerhaeuser Notes

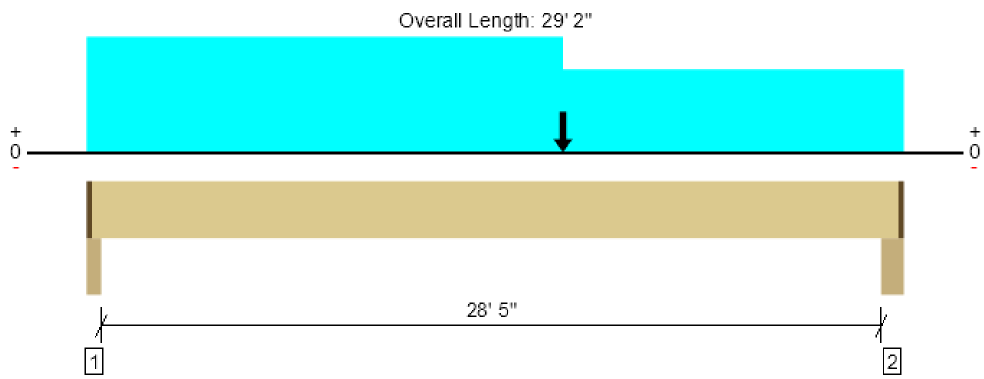
ForteWEB Software Operator	Job Notes
Frankie Tsui F.T. Engineering & Construction Management, LLC (509) 822-0489 f.t.eng.cm@gmail.com	

MEMBER REPORT

PASSED

Level, B4

1 piece(s) 8 3/4" x 21" 24F-V4 DF Glulam



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

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	12429 @ 2"	12797 (2.25")	Passed (97%)	--	1.0 D + 1.0 L (All Spans)
Shear (lbs)	10850 @ 2' 1/2"	32463	Passed (33%)	1.00	1.0 D + 1.0 L (All Spans)
Pos Moment (Ft-lbs)	94034 @ 15' 4 5/16"	111757	Passed (84%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.788 @ 14' 6 5/16"	0.956	Passed (L/437)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	1.119 @ 14' 6 1/8"	1.433	Passed (L/307)	--	1.0 D + 1.0 L (All Spans)

Member Length : 28' 11 1/2"
System : Floor
Member Type : Flush Beam
Building Use : Residential
Building Code : IBC 2021
Design Methodology : ASD

- Deflection criteria: LL (L/360) and TL (L/240).
- Allowed moment does not reflect the adjustment for the beam stability factor.
- Critical positive moment adjusted by a volume/size factor of 0.87 that was calculated using length L = 28' 8".
- The effects of positive or negative camber have not been accounted for when calculating deflection.
- The specified glulam is assumed to have its strong laminations at the bottom of the beam. Install with proper side up as indicated by the manufacturer.
- Applicable calculations are based on NDS.

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Floor Live	Factored	
1 - Column - HF	3.50"	2.25"	2.19"	3789	8720	12509	1 1/4" Rim Board
2 - Column - HF	5.50"	4.25"	2.02"	3499	8067	11566	1 1/4" Rim Board

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

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	29' o/c	
Bottom Edge (Lu)	29' o/c	

•Maximum allowable bracing intervals based on applied load.

Vertical Loads	Location (Side)	Tributary Width	Dead (0.90)	Floor Live (1.00)	Comments
0 - Self Weight (PLF)	1 1/4" to 29' 3/4"	N/A	44.7	--	
1 - Point (lb)	17' (Front)	N/A	600	2400	Stair
2 - Uniform (PSF)	0 to 29' 2" (Front)	10'	15.0	40.0	floor
3 - Uniform (PSF)	0 to 17' (Front)	4'	15.0	40.0	floor

• Side loads are assumed to not induce cross-grain tension.

Weyerhaeuser Notes

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

ForteWEB Software Operator	Job Notes
Frankie Tsui F.T. Engineering & Construction Management, LLC (509) 822-0489 f.t.eng.cm@gmail.com	

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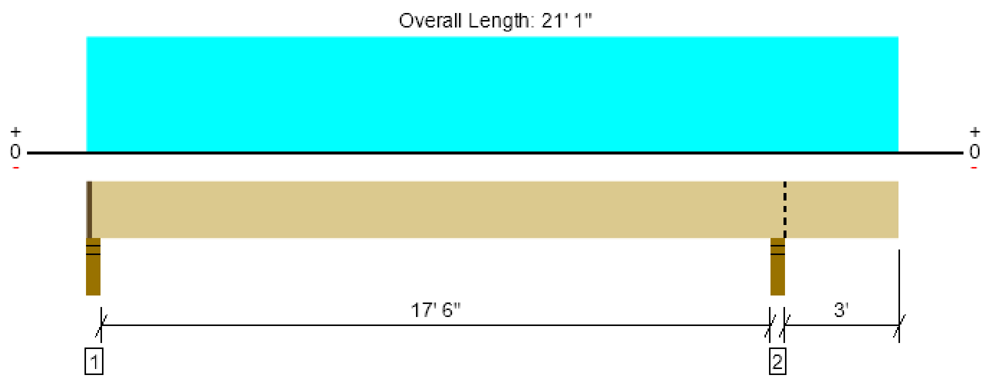
12/11/2024 4:35:41 PM UTC
ForteWEB v3.8, Engine: V8.4.1.24, Data: V8.1.6.3
File Name: 2024065 - 4332 Mercer Island Addition

Level, B5

1 piece(s) 5 1/2" x 16" 24F-V4 DF Glulam



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

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	1161 @ 2"	5259 (2.25")	Passed (22%)	--	1.0 D + 1.0 L (Alt Spans)
Shear (lbs)	1010 @ 16' 5 1/2"	15547	Passed (6%)	1.00	1.0 D + 1.0 L (All Spans)
Pos Moment (Ft-lbs)	5060 @ 8' 11 5/16"	46101	Passed (11%)	1.00	1.0 D + 1.0 L (Alt Spans)
Neg Moment (Ft-lbs)	-650 @ 17' 11 1/4"	36178	Passed (2%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.053 @ 9' 5/8"	0.444	Passed (L/999+)	--	1.0 D + 1.0 L (Alt Spans)
Total Load Defl. (in)	0.085 @ 9' 3/16"	0.889	Passed (L/999+)	--	1.0 D + 1.0 L (Alt Spans)

Member Length : 20' 11 3/4"
System : Floor
Member Type : Flush Beam
Building Use : Residential
Building Code : IBC 2021
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/size factor of 0.98 that was calculated using length L = 17' 6 5/8".
- Critical negative moment adjusted by a volume/size factor of 1.00 that was calculated using length L = 3' 8 7/16".
- 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	Factored	
1 - Stud wall - SPF	3.50"	2.25"	1.50"	449	724/-14	1173	1 1/4" Rim Board
2 - Stud wall - SPF	3.50"	3.50"	1.50"	633	985	1617	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)	21' o/c	
Bottom Edge (Lu)	21' o/c	

- Maximum allowable bracing intervals based on applied load.

Vertical Loads	Location (Side)	Tributary Width	Dead (0.90)	Floor Live (1.00)	Comments
0 - Self Weight (PLF)	1 1/4" to 21' 1"	N/A	21.4	--	
1 - Uniform (PSF)	0 to 21' 1" (Front)	2'	15.0	40.0	Default Load

- Side loads are assumed to not induce cross-grain tension.

Weyerhaeuser Notes

ForteWEB Software Operator	Job Notes
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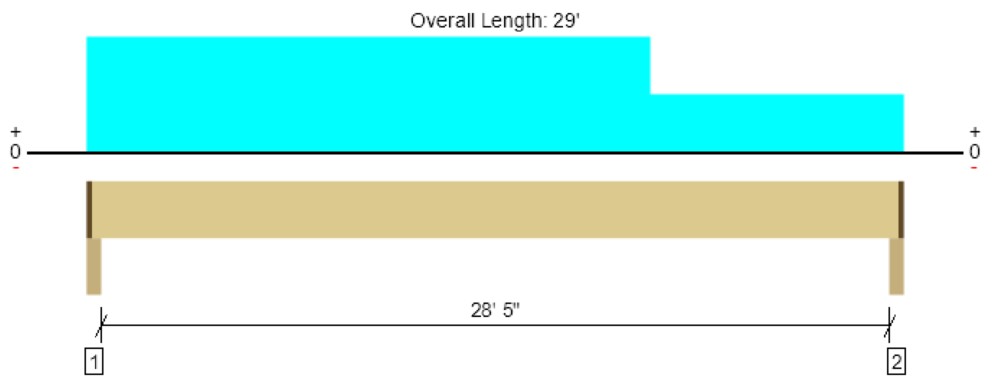
Weyerhaeuser

MEMBER REPORT

PASSED

Level, B6

1 piece(s) 8 3/4" x 21" 24F-V4 DF Glulam



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

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	12712 @ 2"	12797 (2.25")	Passed (99%)	--	1.0 D + 1.0 L (All Spans)
Shear (lbs)	10921 @ 2' 1/2"	32463	Passed (34%)	1.00	1.0 D + 1.0 L (All Spans)
Pos Moment (Ft-lbs)	86594 @ 13' 10 1/4"	111757	Passed (77%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.715 @ 14' 3 9/16"	0.956	Passed (L/481)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	1.039 @ 14' 3 11/16"	1.433	Passed (L/331)	--	1.0 D + 1.0 L (All Spans)

Member Length : 28' 9 1/2"
System : Floor
Member Type : Flush Beam
Building Use : Residential
Building Code : IBC 2021
Design Methodology : ASD

- Deflection criteria: LL (L/360) and TL (L/240).
- Allowed moment does not reflect the adjustment for the beam stability factor.
- Critical positive moment adjusted by a volume/size factor of 0.87 that was calculated using length L = 28' 8".
- The effects of positive or negative camber have not been accounted for when calculating deflection.
- The specified glulam is assumed to have its strong laminations at the bottom of the beam. Install with proper side up as indicated by the manufacturer.
- Applicable calculations are based on NDS.

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Floor Live	Factored	
1 - Column - HF	3.50"	2.25"	2.24"	3960	8844	12804	1 1/4" Rim Board
2 - Column - HF	3.50"	2.25"	1.76"	3206	6836	10042	1 1/4" Rim Board

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

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	28' 10" o/c	
Bottom Edge (Lu)	28' 10" o/c	

•Maximum allowable bracing intervals based on applied load.

Vertical Loads	Location (Side)	Tributary Width	Dead (0.90)	Floor Live (1.00)	Comments
0 - Self Weight (PLF)	1 1/4" to 28' 10 3/4"	N/A	44.7	--	
1 - Uniform (PSF)	0 to 20' (Front)	16'	15.0	40.0	floor
2 - Uniform (PSF)	20' to 29' (Front)	8'	15.0	40.0	floor

• Side loads are assumed to not induce cross-grain tension.

Weyerhaeuser Notes

ForteWEB Software Operator	Job Notes
Frankie Tsui F.T. Engineering & Construction Management, LLC (509) 822-0489 f.t.eng.cm@gmail.com	

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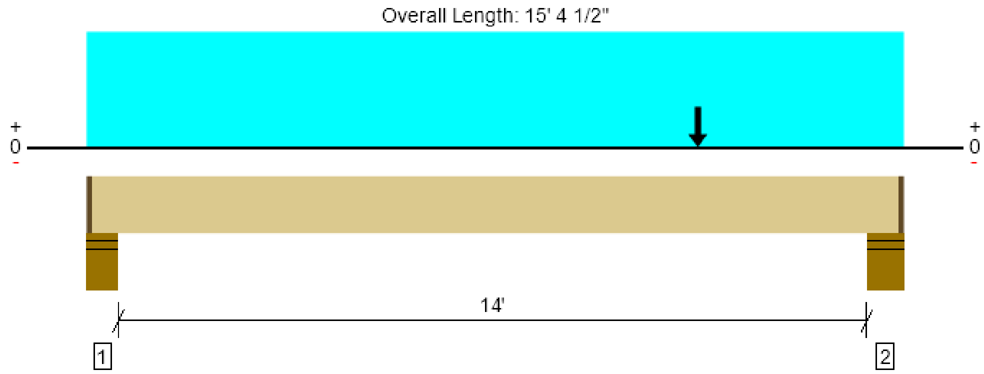
12/11/2024 4:35:41 PM UTC
ForteWEB v3.8, Engine: V8.4.1.24, Data: V8.1.6.3
File Name: 2024065 - 4332 Mercer Island Addition

Level, B7

1 piece(s) 3 1/2" x 16 1/2" 24F-V4 DF Glulam



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

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	10715 @ 14' 9"	10986 (7.75")	Passed (98%)	--	1.0 D + 0.75 L + 0.75 S (All Spans)
Shear (lbs)	8554 @ 13' 3"	10203	Passed (84%)	1.00	1.0 D + 1.0 L (All Spans)
Pos Moment (Ft-lbs)	27614 @ 11' 4 15/16"	31763	Passed (87%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.266 @ 8'	0.356	Passed (L/643)	--	1.0 D + 0.75 L + 0.75 S (All Spans)
Total Load Defl. (in)	0.479 @ 7' 11 3/8"	0.712	Passed (L/357)	--	1.0 D + 0.75 L + 0.75 S (All Spans)

Member Length : 15' 2"
System : Floor
Member Type : Flush Beam
Building Use : Residential
Building Code : IBC 2021
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/size factor of 1.00 that was calculated using length L = 14' 3".
- The effects of positive or negative camber have not been accounted for when calculating deflection.
- The specified glulam is assumed to have its strong laminations at the bottom of the beam. Install with proper side up as indicated by the manufacturer.
- Applicable calculations are based on NDS.

Supports	Bearing Length			Loads to Supports (lbs)				Accessories
	Total	Available	Required	Dead	Floor Live	Snow	Factored	
1 - Stud wall - HF	7.50"	6.25"	5.06"	3429	1864	3241	7257	1 1/4" Rim Board
2 - Stud wall - HF	9.00"	7.75"	7.56"	4673	4866	3294	10793	1 1/4" Rim Board

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

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	10' 4" o/c	
Bottom Edge (Lu)	15' 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 15' 3 1/4"	N/A	14.0	--	--	
1 - Uniform (PSF)	0 to 15' 4 1/2" (Front)	17'	20.0	-	25.0	roof
2 - Uniform (PSF)	0 to 15' 4 1/2" (Front)	2'	15.0	40.0	-	floor
3 - Point (lb)	11' 6" (Front)	N/A	2200	5500	-	B3

• Side loads are assumed to not induce cross-grain tension.

ForteWEB Software Operator	Job Notes
Frankie Tsui F.T. Engineering & Construction Management, LLC (509) 822-0489 f.t.eng.cm@gmail.com	

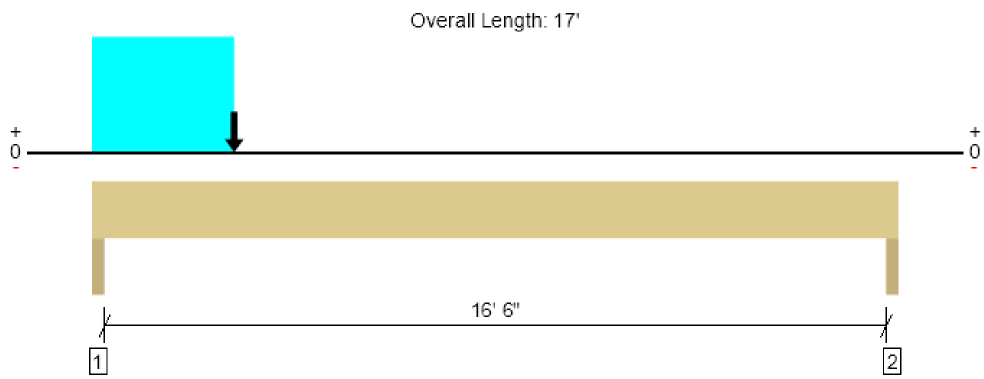
FORTEWEB

MEMBER REPORT

PASSED

Level, B8

1 piece(s) 3 1/2" x 12" 24F-V4 DF Glulam



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

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	5960 @ 1 1/2"	6825 (3.00")	Passed (87%)	--	1.0 D + 1.0 S (All Spans)
Shear (lbs)	4653 @ 1' 3"	8533	Passed (55%)	1.15	1.0 D + 1.0 S (All Spans)
Pos Moment (Ft-lbs)	12439 @ 3'	19320	Passed (64%)	1.15	1.0 D + 1.0 S (All Spans)
Live Load Defl. (in)	0.273 @ 7' 4 3/8"	0.558	Passed (L/737)	--	1.0 D + 1.0 S (All Spans)
Total Load Defl. (in)	0.507 @ 7' 4 15/16"	0.837	Passed (L/397)	--	1.0 D + 1.0 S (All Spans)

Member Length : 17'
System : Wall
Member Type : Header
Building Use : Residential
Building Code : IBC 2021
Design Methodology : ASD

- Deflection criteria: LL (L/360) and TL (L/240).
- Allowed moment does not reflect the adjustment for the beam stability factor.
- Critical positive moment adjusted by a volume/size factor of 1.00 that was calculated using length L = 16' 9".
- The effects of positive or negative camber have not been accounted for when calculating deflection.
- The specified glulam is assumed to have its strong laminations at the bottom of the beam. Install with proper side up as indicated by the manufacturer.
- Applicable calculations are based on NDS.

	Bearing Length			Loads to Supports (lbs)			
Supports	Total	Available	Required	Dead	Snow	Factored	Accessories
1 - Trimmer - SPF	3.00"	3.00"	2.62"	2679	3281	5960	None
2 - Trimmer - SPF	3.00"	3.00"	1.50"	475	494	969	None

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	17' o/c	
Bottom Edge (Lu)	17' o/c	

•Maximum allowable bracing intervals based on applied load.

Vertical Loads	Location	Tributary Width	Dead (0.90)	Snow (1.15)	Comments
0 - Self Weight (PLF)	0 to 17'	N/A	10.2	--	
1 - Point (lb)	3'	N/A	1600	2050	truss
2 - Uniform (PSF)	0 to 3'	23'	20.0	25.0	roof

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

ForteWEB Software Operator	Job Notes
Frankie Tsui F.T. Engineering & Construction Management, LLC (509) 822-0489 f.t.eng.cm@gmail.com	

Weyerhaeuser

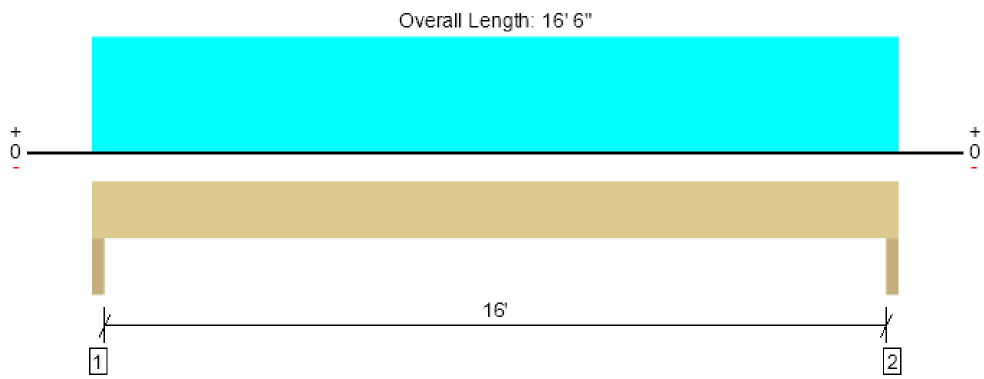
12/11/2024 4:35:41 PM UTC
ForteWEB v3.8, Engine: V8.4.1.24, Data: V8.1.6.3
File Name: 2024065 - 4332 Mercer Island Addition

MEMBER REPORT

PASSED

Level, B9

1 piece(s) 3 1/2" x 10 1/2" 24F-V4 DF Glulam



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

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	1559 @ 1 1/2"	6825 (3.00")	Passed (23%)	--	1.0 D + 1.0 S (All Spans)
Shear (lbs)	1346 @ 1' 1 1/2"	7466	Passed (18%)	1.15	1.0 D + 1.0 S (All Spans)
Pos Moment (Ft-lbs)	6236 @ 8' 3"	14792	Passed (42%)	1.15	1.0 D + 1.0 S (All Spans)
Live Load Defl. (in)	0.258 @ 8' 3"	0.542	Passed (L/755)	--	1.0 D + 1.0 S (All Spans)
Total Load Defl. (in)	0.488 @ 8' 3"	0.813	Passed (L/400)	--	1.0 D + 1.0 S (All Spans)

Member Length : 16' 6"
System : Wall
Member Type : Header
Building Use : Residential
Building Code : IBC 2021
Design Methodology : ASD

- Deflection criteria: LL (L/360) and TL (L/240).
- Allowed moment does not reflect the adjustment for the beam stability factor.
- Critical positive moment adjusted by a volume/size factor of 1.00 that was calculated using length L = 16' 3".
- The effects of positive or negative camber have not been accounted for when calculating deflection.
- The specified glulam is assumed to have its strong laminations at the bottom of the beam. Install with proper side up as indicated by the manufacturer.
- Applicable calculations are based on NDS.

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Snow	Factored	
1 - Trimmer - SPF	3.00"	3.00"	1.50"	734	825	1559	None
2 - Trimmer - SPF	3.00"	3.00"	1.50"	734	825	1559	None

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	16' 6" o/c	
Bottom Edge (Lu)	16' 6" o/c	

•Maximum allowable bracing intervals based on applied load.

Vertical Loads	Location	Tributary Width	Dead (0.90)	Snow (1.15)	Comments
0 - Self Weight (PLF)	0 to 16' 6"	N/A	8.9	--	
1 - Uniform (PSF)	0 to 16' 6"	4'	20.0	25.0	Default Load

Weyerhaeuser Notes

ForteWEB Software Operator	Job Notes
Frankie Tsui F.T. Engineering & Construction Management, LLC (509) 822-0489 f.t.eng.cm@gmail.com	

Weyerhaeuser

Level, 6X6 Post

1 piece(s) 6 x 6 DF No.2

Post Height: 10'

Drawing is Conceptual

Design Results	Actual	Allowed	Result	LDF	Load: Combination
Slenderness	22	50	Passed (44%)	--	--
Compression (lbs)	12000	15659	Passed (77%)	1.00	1.0 D + 1.0 L
Base Bearing (lbs)	12000	12251	Passed (98%)	--	1.0 D + 1.0 L
Bending/Compression	N/A	1	Passed (N/A)	--	N/A

- Input axial load eccentricity for the design is zero
- Applicable calculations are based on NDS.
- Bearing shall be on a metal plate or strap, or on other equivalently durable, rigid, homogeneous material with sufficient stiffness to distribute applied load.

Supports	Type	Material
Base	Plate	Hem Fir

Member Type : Free Standing Post
Building Code : IBC 2021
Design Methodology : ASD

Max Unbraced Length	Comments
Full Member Length	No bracing assumed.

Vertical Load	Dead (0.90)	Floor Live (1.00)	Comments
1 - Point (lb)	-	12000	Default Load

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

ForteWEB Software Operator	Job Notes
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Level, 4X6 Post

1 piece(s) 4 x 6 DF No.2

Post Height: 9' 6"

Drawing is Conceptual

Design Results	Actual	Allowed	Result	LDF	Load: Combination
Slenderness	33	50	Passed (65%)	--	--
Compression (lbs)	7800	8024	Passed (97%)	1.00	1.0 D + 1.0 L
Base Bearing (lbs)	7800	7796	Passed (100%)	--	1.0 D + 1.0 L
Bending/Compression	N/A	1	Passed (N/A)	--	N/A

- Input axial load eccentricity for the design is zero
- Applicable calculations are based on NDS.
- Bearing shall be on a metal plate or strap, or on other equivalently durable, rigid, homogeneous material with sufficient stiffness to distribute applied load.

Supports	Type	Material
Base	Plate	Hem Fir

Member Type : Free Standing Post
Building Code : IBC 2021
Design Methodology : ASD

Max Unbraced Length	Comments
Full Member Length	No bracing assumed.

Vertical Load	Dead (0.90)	Floor Live (1.00)	Comments
1 - Point (lb)	-	7800	Default Load

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

Level, 4X8 Post

1 piece(s) 4 x 8 DF No.2

Post Height: 9' 6"

Drawing is Conceptual

Design Results	Actual	Allowed	Result	LDF	Load: Combination
Slenderness	33	50	Passed (65%)	--	--
Compression (lbs)	10000	10531	Passed (95%)	1.00	1.0 D + 1.0 L
Base Bearing (lbs)	10000	10277	Passed (97%)	--	1.0 D + 1.0 L
Bending/Compression	N/A	1	Passed (N/A)	--	N/A

- Input axial load eccentricity for the design is zero
- Applicable calculations are based on NDS.
- Bearing shall be on a metal plate or strap, or on other equivalently durable, rigid, homogeneous material with sufficient stiffness to distribute applied load.

Supports	Type	Material
Base	Plate	Hem Fir

Member Type : Free Standing Post
Building Code : IBC 2021
Design Methodology : ASD

Max Unbraced Length	Comments
Full Member Length	No bracing assumed.

Vertical Load	Dead (0.90)	Floor Live (1.00)	Comments
1 - Point (lb)	-	10000	Default Load

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

Level, 6X10 Post

1 piece(s) 6 x 10 DF No.2

Post Height: 9' 6"

Drawing is Conceptual

Design Results	Actual	Allowed	Result	LDF	Load: Combination
Slenderness	21	50	Passed (41%)	--	--
Compression (lbs)	21000	25384	Passed (83%)	1.00	1.0 D + 1.0 L
Base Bearing (lbs)	21000	21161	Passed (99%)	--	1.0 D + 1.0 L
Bending/Compression	N/A	1	Passed (N/A)	--	N/A

- Input axial load eccentricity for the design is zero
- Applicable calculations are based on NDS.
- Bearing shall be on a metal plate or strap, or on other equivalently durable, rigid, homogeneous material with sufficient stiffness to distribute applied load.
- Lumber grading provisions must be extended over the length of the member per NDS 4.2.5.5.

Supports	Type	Material
Base	Plate	Hem Fir

Member Type : Free Standing Post
Building Code : IBC 2021
Design Methodology : ASD

Max Unbraced Length	Comments
Full Member Length	No bracing assumed.

Vertical Load	Dead (0.90)	Floor Live (1.00)	Comments
1 - Point (lb)	-	21000	Default Load

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

Level, 4X10 Post

1 piece(s) 4 x 10 DF No.2

Post Height: 9' 6"

Drawing is Conceptual

Design Results	Actual	Allowed	Result	LDF	Load: Combination
Slenderness	33	50	Passed (65%)	--	--
Compression (lbs)	13000	13370	Passed (97%)	1.00	1.0 D + 1.0 L
Base Bearing (lbs)	13000	13112	Passed (99%)	--	1.0 D + 1.0 L
Bending/Compression	N/A	1	Passed (N/A)	--	N/A

- Input axial load eccentricity for the design is zero
- Applicable calculations are based on NDS.
- Bearing shall be on a metal plate or strap, or on other equivalently durable, rigid, homogeneous material with sufficient stiffness to distribute applied load.

Supports	Type	Material
Base	Plate	Hem Fir

Member Type : Free Standing Post
Building Code : IBC 2021
Design Methodology : ASD

Max Unbraced Length	Comments
Full Member Length	No bracing assumed.

Vertical Load	Dead (0.90)	Floor Live (1.00)	Comments
1 - Point (lb)	-	13000	Default Load

Weyerhaeuser Notes

ForteWEB Software Operator	Job Notes
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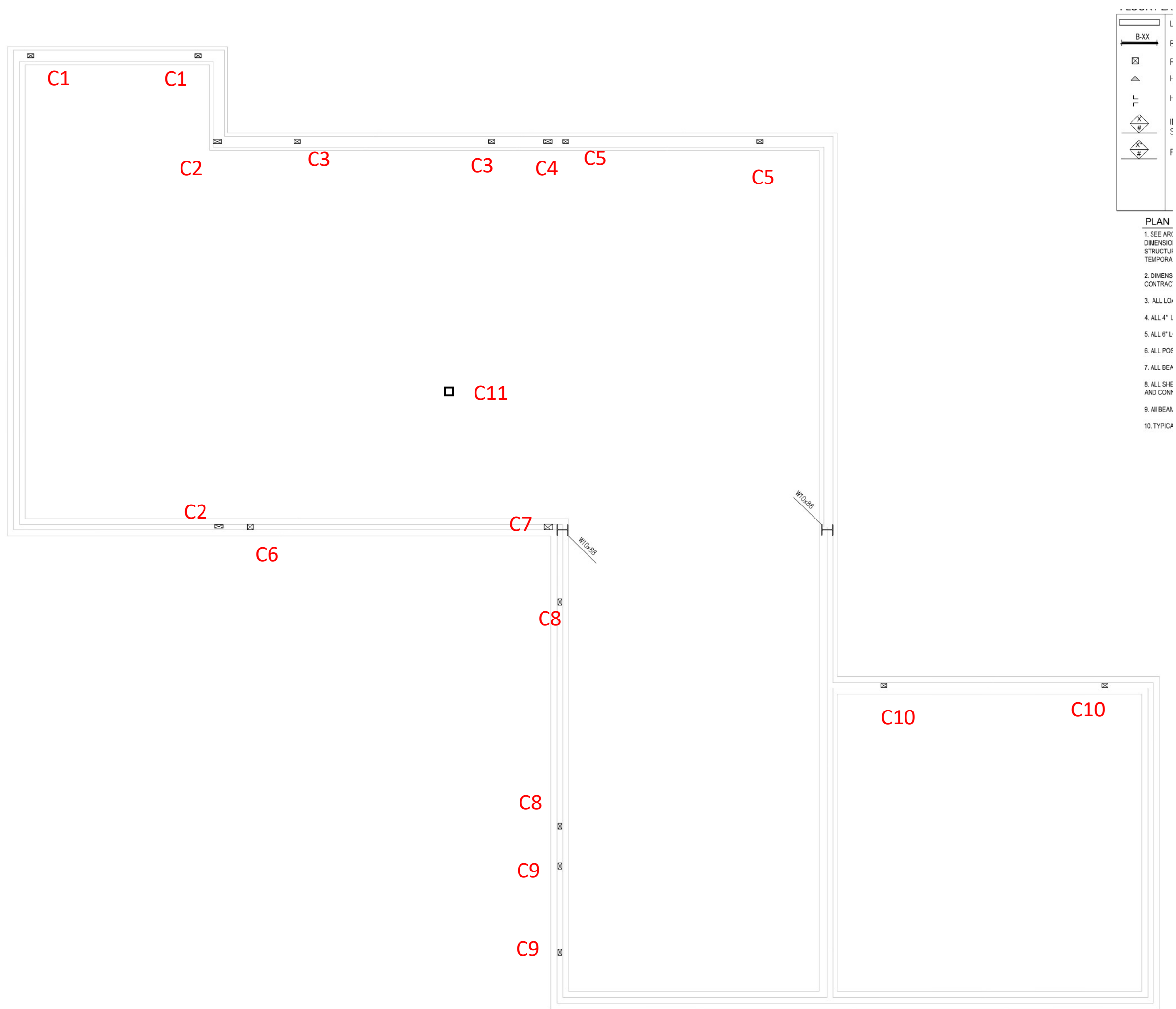
4.0 Column and Foundation Design



Member Name		Capacity (at 10 ft)
6x12 df#2		36000
6x8 df#2		17000
Post 6x6 df#2		12000
Wall: 2x6		3300
Wall: 2x4		2100

	DL	LL	Total (psf)
Roof	20	25	45
Floor	15	40	55
Wall	15		15

		Area	**Load**	
C1	Roof	125	5625	
	Floor	13	688	
	Wall	63	938	
	Other	0	0	
	Total		7250	lbs
C2	Roof	0	0	
	Floor	232	12760	
	Wall	0	0	
	Other		0	
	Total		12760	lbs
C3	Roof	0	0	
	Floor	0	0	
	Wall	0	0	
	Other		12833	Beam B7
	Total		12833	lbs
C4	Roof	0	0	
	Floor	0	0	
	Wall	0	0	
	Other		12509	Beam B4
	Total		12509	lbs
C5	Roof	123	5546	
	Floor	15	798	
	Wall	73	1088	
	Other		0	Beam B4
	Total		7431	lbs
C6	Roof	0	0	
	Floor	0	0	
	Wall	0	0	
	Other		8877	Beam RB1
	Total		8877	lbs
C7	Roof	0	0	
	Floor	0	0	
	Wall	0	0	
	Other		20443	Beam RB1+B4
	Total		20443	lbs

C8	Roof	0	0	
	Floor	0	0	
	Wall	0	0	
	Other		5960	Beam B8
	Total		5960	lbs

C9	Roof	0	0	
	Floor	0	0	
	Wall	0	0	
	Other		4410	HD3
	Total		4410	lbs

C10	Roof	0	0	
	Floor	0	0	
	Wall	0	0	
	Other		1559	B9
	Total		1559	lbs

C11	Roof	0	0	
	Floor	0	0	
	Wall	0	0	
	Other		9798	B3
	Total		9798	lbs

C12	V roof =	8750	lbs
	V 2nd =	7520	lbs
	Wdl =	420	plf
	Wdd =	525	plf
	beam span =	20	ft
	roof h =	10	ft
	2nd h =	11	ft

Max Axial Load =	13324	lbs (seismic)
Max Axial Load =	4200	lbs (DL)
Max Axial Load =	5250	lbs (LL)
OTM =	266470	lb-ft
RTM =	105000	lb-ft
Uplift =	8599	lbs (LRFD)
Max Axial =	11846	lbs (ASD 0.6D+0.7E)

Column FTG

Col	Load	Use	Col Cap	Req'd ftg	use ftg		FTG AREA
	lbs	Col size	lbs	ft2	B	D	ft2
C1	7250	4x6	7800	4.83	3.0	3.0	9.0
C2	12760	4x10	13000	8.51	3.0	4.0	12.0
C3	12833	4x10	13000	8.56	3.5	3.5	12.3
C4	12509	4x10	13000	8.34	3.5	3.5	12.3
C5	7431	4x6	7800	4.95	2.5	2.5	6.3
C6	8877	6x6	12000	5.92	2.0	4.0	8.0
C7	20443	6x10	21000	13.63	4.0	4.0	16.0
C8	5960	4x6	7800	3.97	3.0	3.0	9.0
C9	4410	4x6	7800	2.94	2.5	1.3	3.3
C10	1559	4x6	7800	2.50	1.3	2.5	3.3
C11	9798	6X6	12000	6.53	3.0	3.0	9.0
C12	11846	W10x88	na	7.90	3.0	3.0	9.0
C4+C5	19940	na	na	13.29	5.0	5.0	25.0
C7+C12	32289	na	na	21.53	6.0	6.0	36.0

Find load center for the combined FTG

		CG
C4+C5	19940	0.63
C7+C12	32289	0.63

Existing Wall FTG: Assumed 16" width

For Column effective area = 3.33 sqft assumed 12" depth

EXT.Wall FTG:

Max Dead Load =	400	plf Roof
Max Live Load =	500	plf Roof
Max Dead Load =	75	plf Floor
Max Live Load =	200	plf Floor
Wall Weight =	300	plf
FTG Wt =	300	plf
Total =	1775	
req'd ftg width =	14.2	in
Assumed the exsiting ftg =	16	in ok

Column	FTG B ft	FTG W FTG W	FTG THK in	Axial Load lbs	Bearing Stress	Factored Axial(lbs)	Mu (lb-ft/ft)	f'c
C1	3.00	3.00	12.00	7250	806	11600	1450	2500
C2	3.00	4.00	12.00	12760	1063	20416	3403	2500
C3	3.50	3.50	12.00	12833	1048	20533	2567	2500
C4	3.50	3.50	12.00	12509	1021	20014	2502	2500
C5	2.50	2.50	12.00	7431.25	1189	11890	1486	2500
C6	2.00	4.00	12.00	8877	1110	14203	3551	2500
C7	4.00	4.00	12.00	20443	1278	32709	4089	2500
C8	3.00	3.00	12.00	5960	662	9536	1192	2501
C9	2.50	1.30	8.00	4410	1357	7056	459	2500
C10	1.30	2.50	8.00	1559	480	2494	600	2500
C11	3.00	3.00	12.00	9798	1089	15677	1960	2500
C12	3.00	3.00	12.00	11846.45	1316	18954	2369	2501
C4+C5	5.00	5.00	14.00	19940.25	798	31904	3988	2502
C7+C12	6.00	6.00	14.00	32289.45	897	51663	6458	2503

bo	d	Shear ϕVc	Check	Rebar size /ft	As/ft in2	a (in)	Moment ϕMn/ft	Check
56.75	8.69	73952	OK	5	0.31	0.24	11828	OK
56.75	8.69	73952	OK	5	0.31	0.24	11828	OK
56.75	8.69	73952	OK	5	0.31	0.21	11851	OK
56.75	8.69	73952	OK	5	0.31	0.21	11851	OK
56.75	8.69	73952	OK	5	0.31	0.29	11794	OK
56.75	8.69	73952	OK	5	0.31	0.36	11745	OK
56.75	8.69	73952	OK	5	0.31	0.18	11869	OK
56.75	8.69	73967	OK	5	0.31	0.24	11828	OK
41.25	4.81	29777	OK	3	0.11	0.10	2366	OK
41.25	4.81	29777	OK	3	0.11	0.20	2342	OK
56.75	8.69	73952	OK	5	0.31	0.24	11828	OK
56.75	8.69	73967	OK	5	0.31	0.24	11828	OK
64.75	10.69	103844	OK	5	0.31	0.14	14655	OK
64.75	10.69	103865	OK	5	0.31	0.12	14672	OK

5.0 Lateral Analysis

Dead Load: (only at the timber framing Area)

Roof DL	**20.00**	**PSF**
Floor DL	**12.00**	**PSF**
Main Floor DL	**12.00**	**PSF**
IntWall	**10.00**	**PSF**
Ext Wall	**15.00**	**PSF**

Roof

Diaphragm Area:		2864.00	sq. ft.
Height of Diaphragm:		11.00	ft
	Weight of Diaphragm:	57280.00	lbs
Wall Weights Below:			
Wall Height:		10.00	ft
Concrete Wall Lengths:		0.00	lf
Int wall Wall Lengths:		240.00	lf
Ext Wall Perimeter:		194.00	lf
Concrete Wall Weight:		150.00	psf
Int Wall Weight:		10.00	psf
Ext Wall Wall Weight:		15.00	psf
	Weight of Walls Below:	26550.00	lbs
	Seismic Weight at Roof:	**83830.00**	**lbs**

2nd Floor

Diaphragm Area:		3765.00	sq. ft.
Height of Diaphragm:		11.00	ft
	Weight of Diaphragm:	45180.00	lbs
Wall Weights Below:			
Wall Height:		10.00	ft
Concrete Wall Lengths:		0.00	lf
Int wall Wall Lengths:		60.00	lf
Ext Wall Perimeter:		314.00	lf
Concrete Wall Weight:		150.00	psf
Int Wall Weight:		10.00	psf
Ext Wall Wall Weight:		15.00	psf
	Weight of Walls Below:	26550.00	lbs
	Seismic Weight at Floor:	**98280.00**	**lbs**

Base Shear: Shear Wall

V= CS*w
CS = SDS / (R/Ie)
SDS = 0.951
R(N-S) = 6.5
R(E-W) = 6.5
Cs = 0.15
V = 26.64 kips
$0.7V_E$ = 18.65 kips

Seismic Loads

Floor	Seismic Weight	Height	W*h	w*h/Σw*h	V
Roof	83.83 kips	22.00 ft	1844.26	0.63	**16.80 kips**
2nd Floor	98.28 kips	11.00 ft	1081.08	0.37	**9.85 kips**
	182.11 kips		2925.34		

	$0.7V_E$ (Kips)
Roof	11.76
2nd Floor	6.89
	18.65

Base Shear: OMF

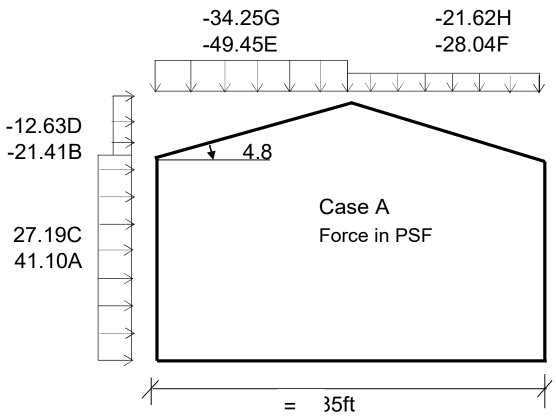
V=	CS*w
CS =	SDS / (R/Ie)
SDS =	0.951
R(N-S) =	3.5
Cs =	0.27
V =	49.48 kips
$0.7V_E$ =	34.64 kips

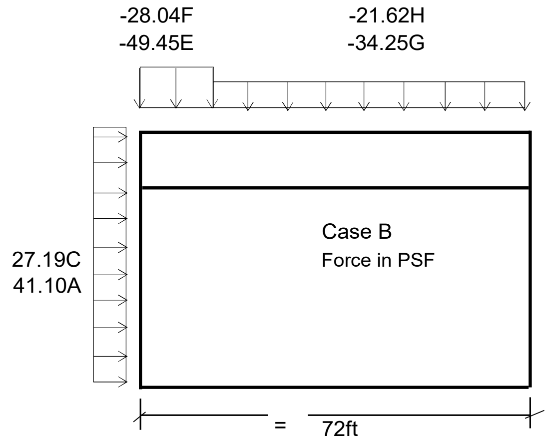
	0VE (Kips)
Roof	31.20
2nd Floor	18.29
	49.48

V Roof =	8.75
V2nd =	7.52
V =	16.27

28.4 Envelope Procedure,
MWFRS For Low-Rise Building. Part 2: Enclosed Simple Diaphragm Building (≤ 60 ft)

Roof Height h =	24	feet		
Roof Pitch =	1.00	:12	= 4.76	Degree
Building & Structure Risk Category =	II, standard		IBC T-1604.5	
Wind Speed V =	110	MPH	Fig. 26.5-1A, MRI = 700 yrs	
Topography factor K_{zt} =	1.60		26.8, Figure 26.8-1	
Exposure	C			
Height Adjustment factor λ =	1.338	Fig 28.6-1		





Plus and minus signs signify pressures acting toward and away from projected surfaces, respectively.
For Case B use Θ = 0°
Total horizontal load shall not be less than that determined by assume p_s = 0 in zones B & D

a= 10% of least horizontal dimension or 0.4h, whichever smaller, but bot less than either 4% of least horizontal dimention or 3ft.

10 % of least dimension=	7.2 ft	⇐
40 % of the eave height =	9.6 ft	
4 % of least dimension or 3 ft=	3.0 ft	

	Section	Wind pressure	Area (sqft)	Wind force (kips)
	A	41.1	154	6.33
Roof	C	27.2	286	7.78
	A	41.1	147	6.04
2nd Floor	C	27.2	569.4	15.48
			Sum W =	35.63
			0.6W =	21.38

	$0.6V_w$ (Kips)
Roof	8.46
2nd Floor	12.91
Sum	21.38

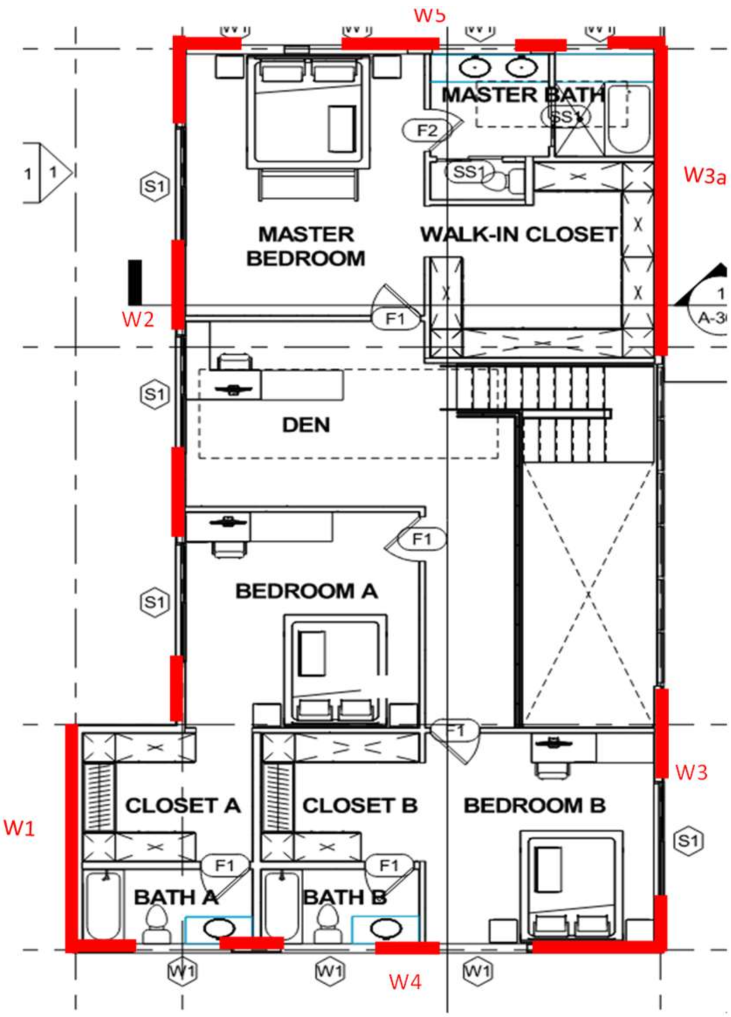
Wind Area -CASE A

	Section	Wind pressure	Area (sqft)	Wind force (kips)
	A	41.1	140	5.75
Roof	C	27.2	378.5	10.29
	A	41.1	147	6.04
2nd Floor	C	27.2	637.5	17.33
			Sum W =	39.42
			0.6W =	23.65

	$0.6V_w$ (Kips)
Roof	9.63
2nd Floor	14.02
Sum	23.65

Wind Area - CASE B

Roof



Roof

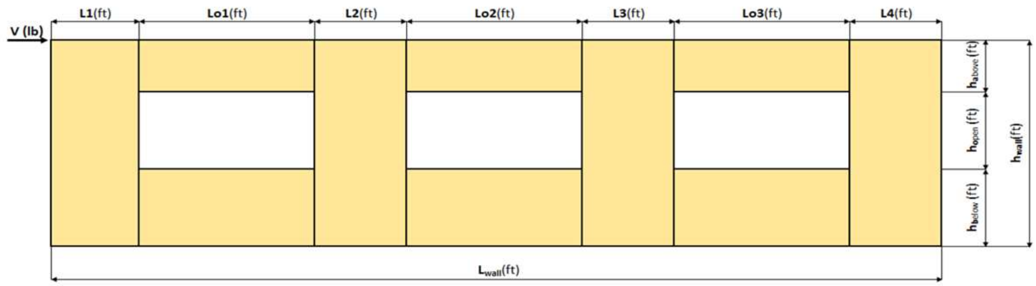
Shear, V= **11.76 kips**

Shear Line	H(ft)	Roof Shear (lbs)	Wall L (ft)	Wall V (plf)
W1	10.0	1076	15.0	**71.8**
W2	10.0	5879	22.2	**265.4**
W3	10.0	1504	9.8	**153.4**
W3a	10.0	3299	21.5	**153.4**
W4	10.0	5879	18.1	**325.7**
W5	10.0	5879	13.8	**427.6**

Wall Pier Loading (Wall Reactions are Treated as Perforated Shearwalls)

Wall	Wall Length	W(DL)	Total Tension(0.6D)	Shear Strength	HD	Shear Wall	Allowable Shear	RATIO
W1	15.00 ft	435 plf	-1240	72 plf	NA	A	230.0 plf	0.31
W2	22.15 ft	435 plf	1274	312 plf	MST37	B	380.0 plf	0.82
W3	9.80 ft	435 plf	845	199 plf	MST37	A	230.0 plf	0.87
W3a	21.50 ft	435 plf	-1271	153 plf	NA	A	230.0 plf	0.67
W4	18.05 ft	175 plf	1631	471 plf	MST37	D	560.0 plf	0.84
W5	13.75 ft	175 plf	1976	497 plf	MST37	D	560.0 plf	0.89

Wall Line: Roof W2



Shear Wall Calculation Variables

			Opening 1		Opening 2		Opening 3
V	5879 lbf						
L1	4.75 ft	h_a1	1.50 ft	h_a2	1.50 ft	h_a3	1.50 ft
L2	5.75 ft	h_o1	5.75 ft	h_o2	5.75 ft	h_o3	5.75 ft
L3	5.90 ft	h_b1	2.75 ft	h_b2	2.75 ft	h_b3	2.75 ft
L4	5.75 ft	Lo1	8.00 ft	Lo2	8.00 ft	Lo3	8.00 ft
h_{wall}	10.00 ft						
L_{wall}	46.15 ft						

Adj. Factor Method = 1.25-0.125h/bs

Wall Pier Aspect Ratio		Adj. Factor
P1=h_o/L1=	1.21	N/A
P2=h_o/L2=	1.00	N/A
P3=h_o/L3=	0.97	N/A
P4=h_o/L4=	1.00	N/A

1. Hold-down forces: H = Vh_{wall}/L_{wall} 1274 lbf

2. Unit shear above + below opening

First opening: va1 = vb1 = H/(h_a1+h_b1) =	300 plf
Second opening: va2 = vb2 = H/(h_a2+h_b2) =	300 plf
Third opening: va3 = vb3 = H/(h_a3+h_b3) =	300 plf

3. Total boundary force above + below openings

First opening: O1 = va1 x (Lo1) =	2398 lbf
Second opening: O2 = va2 x (Lo2) =	2398 lbf
Third opening: O3 = va3 x (Lo3) =	2398 lbf

4. Corner forces

F1 = O1(L1)/(L1+L2) =	1085 lbf
F2 = O1(L2)/(L1+L2) =	1313 lbf
F3 = O2(L2)/(L2+L3) =	1184 lbf
F4 = O2(L3)/(L2+L3) =	1214 lbf
F5 = O3(L3)/(L3+L4) =	1214 lbf
F6 = O3(L4)/(L3+L4) =	1184 lbf

5. Tributary length of openings

T1 = (L1*Lo1)/(L1+L2) =	3.62 ft
T2 = (L2*Lo1)/(L1+L2) =	4.38 ft
T3 = (L2*Lo2)/(L2+L3) =	3.95 ft
T4 = (L3*Lo2)/(L2+L3) =	4.05 ft
T5 = (L3*Lo3)/(L3+L4) =	4.05 ft
T6 = (L4*Lo3)/(L3+L4) =	3.95 ft

6. Unit shear beside opening

v1 = (V/L)(L1+T1)/L1 =	224 plf	
v2 = (V/L)(T2+L2+T3)/L2 =	312 plf	
v3 = (V/L)(T4+L3+T5)/L3 =	302 plf	
v4 = (V/L)(T6+L4)/L4 =	215 plf	
Check v1*L1+v2*L2+v3*L3+v4*L4=V?	5879 lbf	**OK**

7. Resistance to corner forces

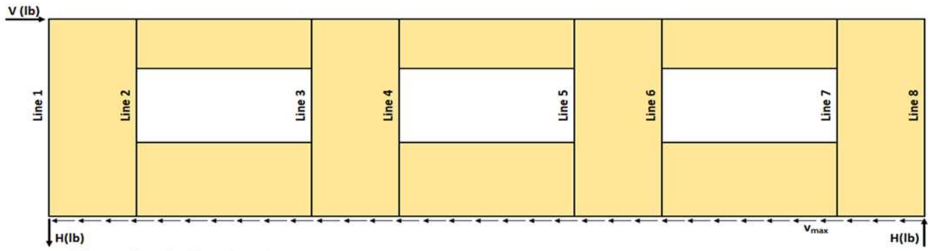
R1 = v1*L1 =	1066 lbf
R2 = v2*L2 =	1794 lbf
R3 = v3*L3 =	1784 lbf
R4 = v4*L4 =	1235 lbf

8. Difference corner force + resistance

R1-F1 =	-19 lbf
R2-F2-F3 =	-703 lbf
R3-F4-F5 =	-645 lbf
R4-F6 =	52 lbf

9. Unit shear in corner zones

vc1 = (R1-F1)/L1 =	-4 plf
vc2 = (R2-F2-F3)/L2 =	-122 plf
vc3 = (R3-F4-F5)/L3 =	-109 plf
vc4 = (R4-F6)/L4 =	9 plf



Check Summary of Shear Values for Three Openings

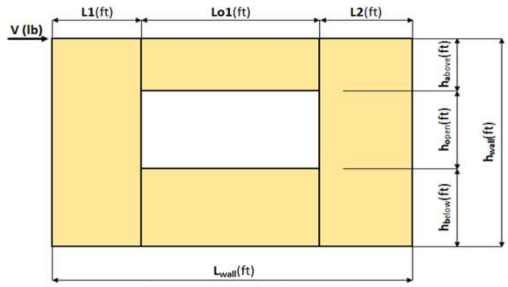
Line 1: $vc1(h_a1+h_b1)+v1(h_o1)=H$?		-17	1291	1274 lbf
Line 2: $va1(h_a1+h_b1)-vc1(h_a1+h_b1)-v1(h_o1)=0$?	1274	-17	1291	0
Line 3: $vc2(h_a1+h_b1)+v2(h_o1)-va1(h_a1+h_b1)=0$?	-520	1794	1274	0
Line 4: $va2(h_a2+h_b2)-v2(h_o2)-vc2(h_a2+h_b2)=0$?	1274	1794	-520	0
Line 5: $va2(h_a2+h_b2)-vc3(h_a2+h_b2)-v3(h_o2)=0$?	1274	-465	1738	0
Line 6: $va3(h_a3+h_b3)-v3(h_o3)-vc3(h_a3+h_b3)=0$?	1274	1738	-465	0
Line 7: $va3(h_a3+h_b3)-vc4(h_a3+h_b3)-v4(h_o3)=0$?	1274	38	1235	0
Line 8: $vc4(h_a3+h_b3)+v4(h_o3)=H$?		38	1235	1274 lbf

Design Summary*

Req. Sheathing Capacity	312 plf	**4-Term Deflection**		**3-Term Deflection**	
Req. Strap Force	1313 lbf	**4-Term Story Drift %**		**3-Term Story Drift %**	
Req. HD Force (H)	1274 lbf				
Req. Shear Wall Anchorage Force (v_{max})	127 plf				

***The Design Summary assumes that the shear wall is designed as blocked.**

Wall Line: Roof W3



Shear Wall Calculation Variables

V	1504 lbf	Opening 1		Adj. Factor Method =	1.25-0.125h/bs	
L1	3.30 ft	h_a	1.50 ft	Wall Pier Aspect Ratio		Adj. Factor
L2	6.50 ft	h_o	5.75 ft	$P1=h_o/L1=$	1.74	N/A
h_{wall}	10.00 ft	h_b	2.75 ft	$P2=h_o/L2=$	0.88	N/A
L_{wall}	17.80 ft	Lo1	8.00 ft			

1. Hold-down forces: $H = Vh_{wall}/L_{wall}$ 845 lbf

2. Unit shear above + below opening

First opening: $va1 = vb1 = H/(h_a+h_b) =$ 199 plf

3. Total boundary force above + below openings

First opening: O1 = va1 x (Lo1) = 1590 lbf

4. Corner forces

F1 = O1(L1)/(L1+L2) = 536 lbf

F2 = O1(L2)/(L1+L2) = 1055 lbf

5. Tributary length of openings

T1 = (L1*Lo1)/(L1+L2) = 2.69 ft

T2 = (L2*Lo1)/(L1+L2) = 5.31 ft

6. Unit shear beside opening

v1 = (V/L)(L1+T1)/L1 = 153 plf

v2 = (V/L)(T2+L2)/L2 = 153 plf

Check v1*L1+v2*L2=V? 1504 lbf **OK**

7. Resistance to corner forces

R1 = v1*L1 = 506 lbf

R2 = v2*L2 = 998 lbf

8. Difference corner force + resistance

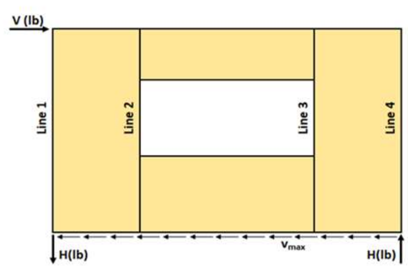
R1-F1 = -29 lbf

R2-F2 = -57 lbf

9. Unit shear in corner zones

vc1 = (R1-F1)/L1 = -9 plf

vc2 = (R2-F2)/L2 = -9 plf



Check Summary of Shear Values for One Opening

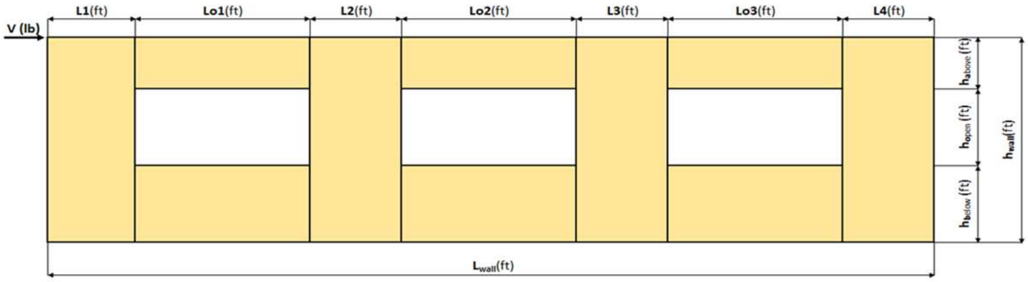
Line 1: $vc1(h_a+h_b)+v1(h_o)=H?$		-38	882	845 lbf
Line 2: $va1(h_a+h_b)-vc1(h_a+h_b)-v1(h_o)=0?$	845	-38	882	0
Line 3: $va1(h_a+h_b)-vc2(h_a+h_b)-v1(h_o)=0?$	845	-38	882	0
Line 4: $vc2(h_a+h_b)+v2(h_o)=H?$		-38	882	845 lbf

Design Summary*

Req. Sheathing Capacity	199 plf	**4-Term Deflection**		**3-Term Deflection**	
Req. Strap Force	1055 lbf	**4-Term Story Drift %**		**3-Term Story Drift %**	
Req. HD Force (H)	845 lbf				
Req. Shear Wall Anchorage Force (v_{max})	84 plf				

***The Design Summary assumes that the shear wall is designed as blocked.**

Wall Line: Roof W4



Shear Wall Calculation Variables

			Opening 1		Opening 2		Opening 3		
V	5879 lbf							Adj. Factor Method =	1.25-0.125h/bs
L1	3.75 ft	h_a1	1.50 ft	h_a2	1.50 ft	h_a3	1.50 ft		
L2	3.00 ft	h_o1	0.75 ft	h_o2	0.75 ft	h_o3	0.75 ft		
L3	3.00 ft	h_b1	7.75 ft	h_b2	7.75 ft	h_b3	7.75 ft		
L4	8.30 ft	Lo1	6.00 ft	Lo2	6.00 ft	Lo3	6.00 ft		
h_{wall}	10.00 ft								
L_{wall}	36.05 ft								

Wall Pier Aspect Ratio		Adj. Factor
$P1=h_o/L1=$	0.20	N/A
$P2=h_o/L2=$	0.25	N/A
$P3=h_o/L3=$	0.25	N/A
$P4=h_o/L4=$	0.09	N/A

1. Hold-down forces: $H = Vh_{wall}/L_{wall}$ 1631 lbf

2. Unit shear above + below opening

First opening: $va1 = vb1 = H/(h_a1+h_b1) =$	176 plf
Second opening: $va2 = vb2 = H/(h_a2+h_b2) =$	176 plf
Third opening: $va3 = vb3 = H/(h_a3+h_b3) =$	176 plf

3. Total boundary force above + below openings

First opening: O1 = va1 x (Lo1) =	1058 lbf
Second opening: O2 = va2 x (Lo2) =	1058 lbf
Third opening: O3 = va3 x (Lo3) =	1058 lbf

4. Corner forces

F1 = O1(L1)/(L1+L2) =	588 lbf
F2 = O1(L2)/(L1+L2) =	470 lbf
F3 = O2(L2)/(L2+L3) =	529 lbf
F4 = O2(L3)/(L2+L3) =	529 lbf
F5 = O3(L3)/(L3+L4) =	281 lbf
F6 = O3(L4)/(L3+L4) =	777 lbf

5. Tributary length of openings

T1 = (L1*Lo1)/(L1+L2) =	3.33 ft
T2 = (L2*Lo1)/(L1+L2) =	2.67 ft
T3 = (L2*Lo2)/(L2+L3) =	3.00 ft
T4 = (L3*Lo2)/(L2+L3) =	3.00 ft
T5 = (L3*Lo3)/(L3+L4) =	1.59 ft
T6 = (L4*Lo3)/(L3+L4) =	4.41 ft

6. Unit shear beside opening

v1 = (V/L)(L1+T1)/L1 =	308 plf
v2 = (V/L)(T2+L2+T3)/L2 =	471 plf
v3 = (V/L)(T4+L3+T5)/L3 =	413 plf
v4 = (V/L)(T6+L4)/L4 =	250 plf
Check v1*L1+v2*L2+v3*L3+v4*L4=V?	5879 lbf **OK**

7. Resistance to corner forces

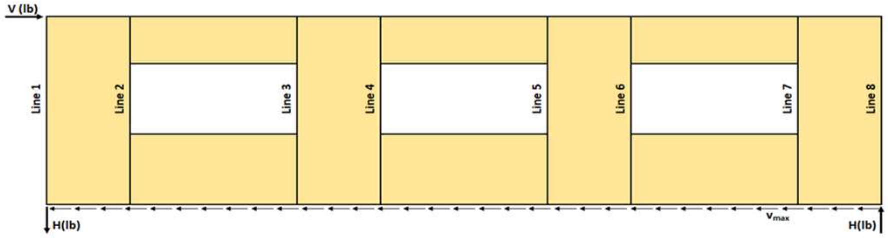
R1 = v1*L1 =	1155 lbf
R2 = v2*L2 =	1413 lbf
R3 = v3*L3 =	1238 lbf
R4 = v4*L4 =	2072 lbf

8. Difference corner force + resistance

R1-F1 =	567 lbf
R2-F2-F3 =	414 lbf
R3-F4-F5 =	429 lbf
R4-F6 =	1295 lbf

9. Unit shear in corner zones

vc1 = (R1-F1)/L1 =	151 plf
vc2 = (R2-F2-F3)/L2 =	138 plf
vc3 = (R3-F4-F5)/L3 =	143 plf
vc4 = (R4-F6)/L4 =	156 plf



Check Summary of Shear Values for Three Openings

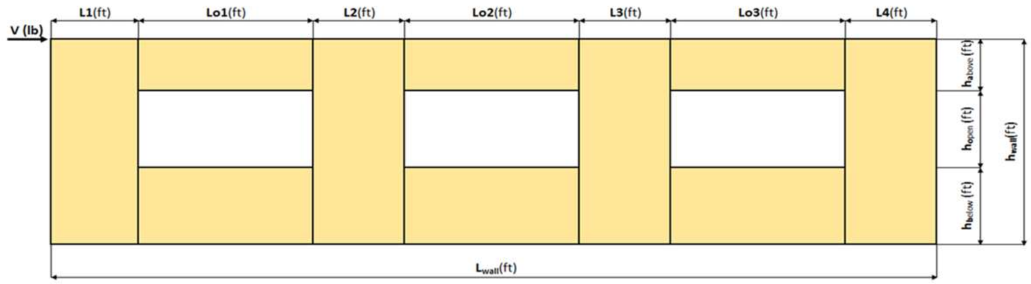
Line 1: vc1(h_a1+h_b1)+v1(h_o1)=H?		1400	231	1631 lbf
Line 2: va1(h_a1+h_b1)-vc1(h_a1+h_b1)-v1(h_o1)=0?	1631	1400	231	0
Line 3: vc2(h_a1+h_b1)+v2(h_o1)-va1(h_a1+h_b1)=0?	1277	353	1631	0
Line 4: va2(h_a2+h_b2)-v2(h_o2)-vc2(h_a2+h_b2)=0?	1631	353	1277	0
Line 5: va2(h_a2+h_b2)-vc3(h_a2+h_b2)-v3(h_o2)=0?	1631	1321	310	0
Line 6: va3(h_a3+h_b3)-v3(h_o3)-vc3(h_a3+h_b3)=0?	1631	310	1321	0
Line 7: va3(h_a3+h_b3)-vc4(h_a3+h_b3)-v4(h_o3)=0?	1631	1444	187	0
Line 8: vc4(h_a3+h_b3)+v4(h_o3)=H?		1444	187	1631 lbf

Design Summary*

Req. Sheathing Capacity	471 plf	4-Term Deflection		3-Term Deflection	
Req. Strap Force	777 lbf	4-Term Story Drift %		3-Term Story Drift %	
Req. HD Force (H)	1631 lbf				
Req. Shear Wall Anchorage Force (v_{max})	163 plf				

*The Design Summary assumes that the shear wall is designed as blocked.

Wall Line: Roof W5



Shear Wall Calculation Variables

			Opening 1		Opening 2		Opening 3
V	5879 lbf						
L1	4.00 ft	h_a1	1.50 ft	h_a2	1.50 ft	h_a3	1.50 ft
L2	5.75 ft	h_o1	0.75 ft	h_o2	0.75 ft	h_o3	0.75 ft
L3	2.00 ft	h_b1	7.75 ft	h_b2	7.75 ft	h_b3	7.75 ft
L4	2.00 ft	Lo1	8.00 ft	Lo2	4.00 ft	Lo3	4.00 ft
h_{wall}	10.00 ft						
L_{wall}	29.75 ft						

Adj. Factor Method = 1.25-0.125h/bs

Wall Pier Aspect Ratio		Adj. Factor
$P1=h_o/L1=$	0.19	N/A
$P2=h_o/L2=$	0.13	N/A
$P3=h_o/L3=$	0.38	N/A
$P4=h_o/L4=$	0.38	N/A

1. Hold-down forces: $H = Vh_{wall}/L_{wall}$ 1976 lbf

2. Unit shear above + below opening

First opening: $va1 = vb1 = H/(h_a1+h_b1) =$	214 plf
Second opening: $va2 = vb2 = H/(h_a2+h_b2) =$	214 plf
Third opening: $va3 = vb3 = H/(h_a3+h_b3) =$	214 plf

3. Total boundary force above + below openings

First opening: O1 = va1 x (Lo1) =	1709 lbf
Second opening: O2 = va2 x (Lo2) =	855 lbf
Third opening: O3 = va3 x (Lo3) =	855 lbf

4. Corner forces

F1 = O1(L1)/(L1+L2) =	701 lbf
F2 = O1(L2)/(L1+L2) =	1008 lbf
F3 = O2(L2)/(L2+L3) =	634 lbf
F4 = O2(L3)/(L2+L3) =	221 lbf
F5 = O3(L3)/(L3+L4) =	427 lbf
F6 = O3(L4)/(L3+L4) =	427 lbf

5. Tributary length of openings

T1 = (L1*Lo1)/(L1+L2) =	3.28 ft
T2 = (L2*Lo1)/(L1+L2) =	4.72 ft
T3 = (L2*Lo2)/(L2+L3) =	2.97 ft
T4 = (L3*Lo2)/(L2+L3) =	1.03 ft
T5 = (L3*Lo3)/(L3+L4) =	2.00 ft
T6 = (L4*Lo3)/(L3+L4) =	2.00 ft

6. Unit shear beside opening

v1 = (V/L)(L1+T1)/L1 =	360 plf
v2 = (V/L)(T2+L2+T3)/L2 =	462 plf
v3 = (V/L)(T4+L3+T5)/L3 =	497 plf
v4 = (V/L)(T6+L4)/L4 =	395 plf
Check v1*L1+v2*L2+v3*L3+v4*L4=V?	5879 lbf **OK**

7. Resistance to corner forces

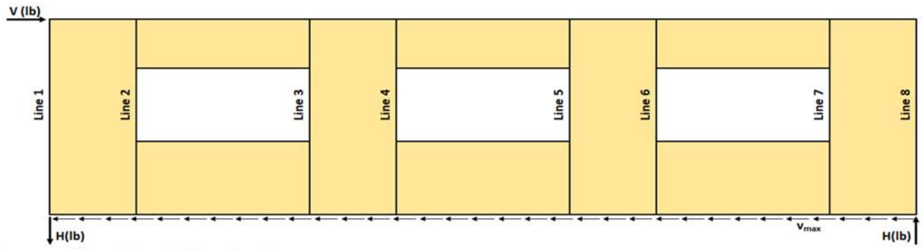
R1 = v1*L1 =	1439 lbf
R2 = v2*L2 =	2655 lbf
R3 = v3*L3 =	994 lbf
R4 = v4*L4 =	790 lbf

8. Difference corner force + resistance

R1-F1 =	738 lbf
R2-F2-F3 =	1013 lbf
R3-F4-F5 =	347 lbf
R4-F6 =	363 lbf

9. Unit shear in corner zones

vc1 = (R1-F1)/L1 =	184 plf
vc2 = (R2-F2-F3)/L2 =	176 plf
vc3 = (R3-F4-F5)/L3 =	173 plf
vc4 = (R4-F6)/L4 =	182 plf



Check Summary of Shear Values for Three Openings

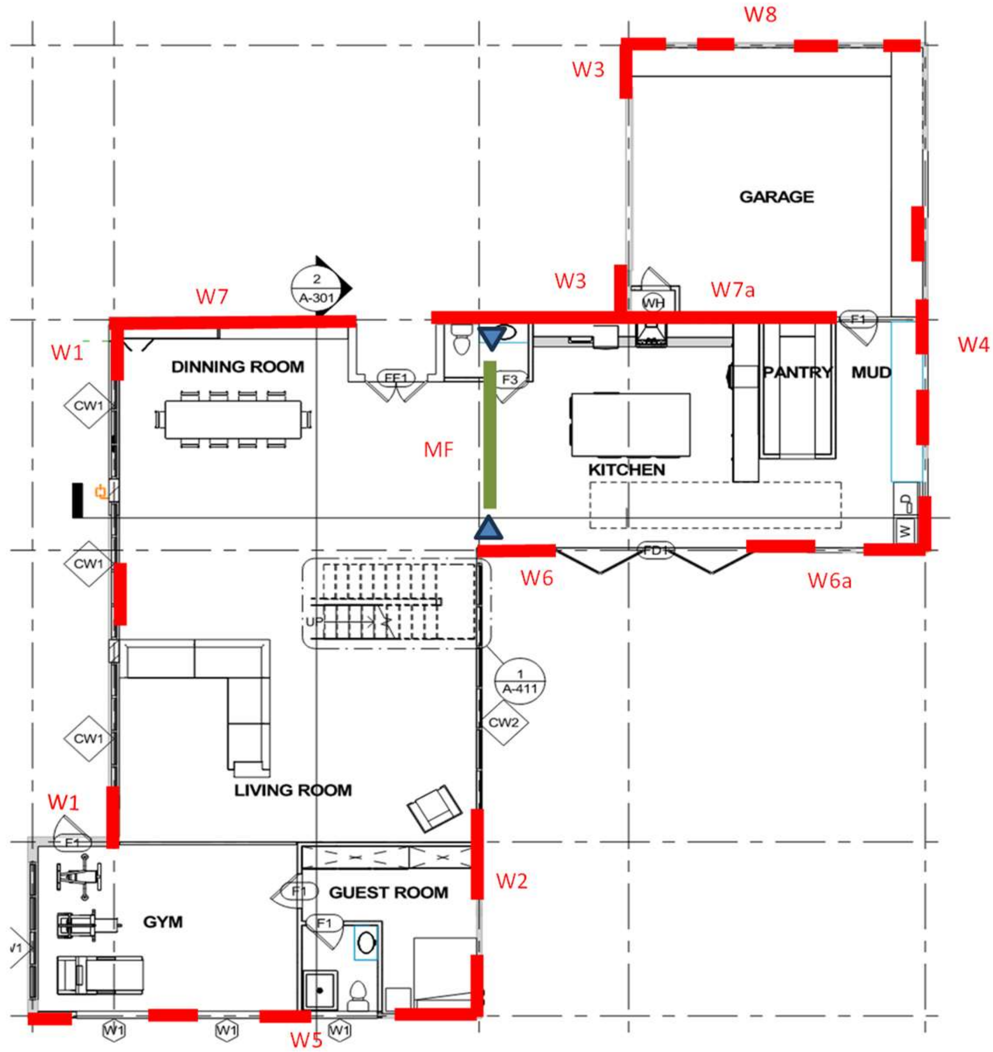
Line 1: $vc1(h_a1+h_b1)+v1(h_o1)=H?$		1706	270	1976 lbf
Line 2: $va1(h_a1+h_b1)-vc1(h_a1+h_b1)-v1(h_o1)=0?$	1976	1706	270	0
Line 3: $vc2(h_a1+h_b1)+v2(h_o1)-va1(h_a1+h_b1)=0?$	1630	346	1976	0
Line 4: $va2(h_a2+h_b2)-v2(h_o2)-vc2(h_a2+h_b2)=0?$	1976	346	1630	0
Line 5: $va2(h_a2+h_b2)-vc3(h_a2+h_b2)-v3(h_o2)=0?$	1976	1603	373	0
Line 6: $va3(h_a3+h_b3)-v3(h_o3)-vc3(h_a3+h_b3)=0?$	1976	373	1603	0
Line 7: $va3(h_a3+h_b3)-vc4(h_a3+h_b3)-v4(h_o3)=0?$	1976	1680	296	0
Line 8: $vc4(h_a3+h_b3)+v4(h_o3)=H?$		1680	296	1976 lbf

Design Summary*

Req. Sheathing Capacity	497 plf	**4-Term Deflection**		**3-Term Deflection**	
Req. Strap Force	1008 lbf	**4-Term Story Drift %**		**3-Term Story Drift %**	
Req. HD Force (H)	1976 lbf				
Req. Shear Wall Anchorage Force (v_{max})	198 plf				

***The Design Summary assumes that the shear wall is designed as blocked.**

2nd Floor



	Roof	2nd
Shear, V=	**11.76 kips**	**14.02 kips**

Shear Line	H(ft)	Roof Shear (lbs)	H(ft)	2nd Shear (lbs)	Wall L (ft)	Wall V (plf)
W1	10.0	2319	11.0	1379	5.5	**672.3**
W2	10.0	1504	11.0	4038	13.0	**426.3**
W3	10.0	0	11.0	1758	3.8	**468.8**
W4	10.0	0	11.0	2334	12.0	**194.5**
W5	10.0	5879	11.0	3361	18.1	**511.9**
W6	10.0	0	11.0	2231	6.0	**371.9**
W6a	10.0	0	11.0	2789	7.5	**371.9**
W7	10.0	2190	11.0	1360	19.0	**186.9**
W7a	10.0	3689	11.0	2291	32.0	**186.9**
W8	10.0	0	11.0	1992	12.5	**159.6**
MF	10.0	8752	11.0	7521	NA	**NA**

Wall Pier Loading (Wall Reactions are Treated as Perforated Shearwalls)

Wall	Wall Length	W(DL)	Total Tension(0.6D)	Shear Strength	HD	Shear Wall	Allowable Shear	RATIO
W1	5.50 ft	600 plf	10620	672 plf	HHDQ14	F	760.0 plf	0.88
W2	13.00 ft	600 plf	2468	470 plf	HDU5	D	560.0 plf	0.84
W3	3.75 ft	600 plf	4482	469 plf	HDU8	D	560.0 plf	0.84
W4	12.00 ft	600 plf	856	218 plf	HDU2	A	230.0 plf	0.95
W5	18.05 ft	600 plf	2819	740 plf	HDU5	F	760.0 plf	0.97
W6	6.00 ft	600 plf	3011	372 plf	HDU5	C	420.0 plf	0.89
W6a	7.50 ft	600 plf	2273	372 plf	HDU5	C	420.0 plf	0.89
W7	19.00 ft	600 plf	-212	187 plf	NA	A	230.0 plf	0.81
W7a	32.00 ft	600 plf	-2552	187 plf	NA	A	230.0 plf	0.81
W8	12.48 ft	600 plf	909	196 plf	HDU2	B	380.0 plf	0.52

Wall Line: 2nd W2

Shear Wall Calculation Variables

V	4038 lbf		Opening 1		
L1	5.00 ft	h_a	2.50 ft		
L2	8.00 ft	h_o	5.75 ft		
h_{wall}	11.00 ft	h_b	2.75 ft		
L_{wall}	18.00 ft	Lo1	5.00 ft		

Adj. Factor Method = 1.25-0.125h/bs

Wall Pier Aspect Ratio		Adj. Factor
P1=h_o/L1=	1.15	N/A
P2=h_o/L2=	0.72	N/A

1. Hold-down forces: H = Vh_{wall}/L_{wall} 2468 lbf

2. Unit shear above + below opening

First opening: va1 = vb1 = H/(h_a+h_b) =	470 plf

3. Total boundary force above + below openings

First opening: O1 = va1 x (Lo1) =	2350 lbf

4. Corner forces

F1 = O1(L1)/(L1+L2) =	904 lbf
F2 = O1(L2)/(L1+L2) =	1446 lbf

5. Tributary length of openings

T1 = (L1*Lo1)/(L1+L2) =	1.92 ft
T2 = (L2*Lo1)/(L1+L2) =	3.08 ft

6. Unit shear beside opening

v1 = (V/L)(L1+T1)/L1 =	311 plf
v2 = (V/L)(T2+L2)/L2 =	311 plf
Check v1*L1+v2*L2=V?	4038 lbf **OK**

7. Resistance to corner forces

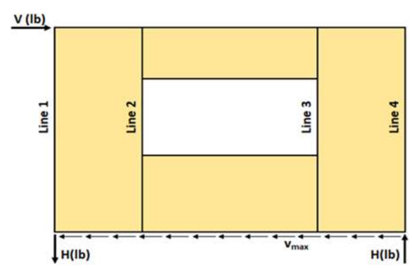
R1 = v1*L1 =	1553 lbf
R2 = v2*L2 =	2485 lbf

8. Difference corner force + resistance

R1-F1 =	649 lbf
R2-F2 =	1039 lbf

9. Unit shear in corner zones

vc1 = (R1-F1)/L1 =	130 plf
vc2 = (R2-F2)/L2 =	130 plf



Check Summary of Shear Values for One Opening

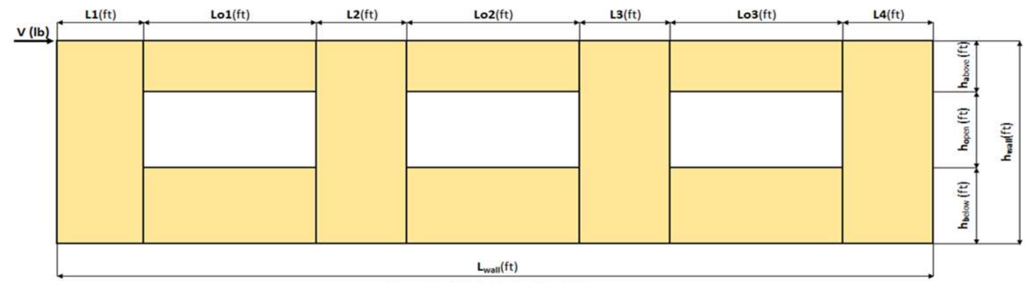
Line 1: vc1(h_a+h_b)+v1(h_o)=H?		682	1786	2468 lbf
Line 2: va1(h_a+h_b)-vc1(h_a+h_b)-v1(h_o)=0?	2468	682	1786	0
Line 3: va1(h_a+h_b)-vc2(h_a+h_b)-v1(h_o)=0?	2468	682	1786	0
Line 4: vc2(h_a+h_b)+v2(h_o)=H?		682	1786	2468 lbf

Design Summary*

Req. Sheathing Capacity	470 plf	**4-Term Deflection**		**3-Term Deflection**	
Req. Strap Force	1446 lbf	**4-Term Story Drift %**		**3-Term Story Drift %**	
Req. HD Force (H)	2468 lbf				
Req. Shear Wall Anchorage Force (v_{max})	224 plf				

***The Design Summary assumes that the shear wall is designed as blocked.**

Wall Line: 2nd W4



Shear Wall Calculation Variables

			Opening 1		Opening 2		Opening 3
V	2334 lbf						
L1	2.30 ft	h_a1	2.50 ft	h_a2	2.50 ft	h_a3	2.50 ft
L2	3.75 ft	h_o1	0.75 ft	h_o2	0.75 ft	h_o3	0.75 ft
L3	3.75 ft	h_b1	7.75 ft	h_b2	7.75 ft	h_b3	7.75 ft
L4	2.20 ft	Lo1	6.00 ft	Lo2	6.00 ft	Lo3	6.00 ft
h_{wall}	11.00 ft						
L_{wall}	30.00 ft						

Adj. Factor Method = 1.25-0.125h/bs

Wall Pier Aspect Ratio		Adj. Factor
P1=h_o/L1=	0.33	N/A
P2=h_o/L2=	0.20	N/A
P3=h_o/L3=	0.20	N/A
P4=h_o/L4=	0.34	N/A

1. Hold-down forces: H = Vh_{wall}/L_{wall} 856 lbf

2. Unit shear above + below opening

First opening: va1 = vb1 = H/(h_a1+h_b1) =	83 plf
Second opening: va2 = vb2 = H/(h_a2+h_b2) =	83 plf
Third opening: va3 = vb3 = H/(h_a3+h_b3) =	83 plf

3. Total boundary force above + below openings

First opening: O1 = va1 x (Lo1) =	501 lbf
Second opening: O2 = va2 x (Lo2) =	501 lbf
Third opening: O3 = va3 x (Lo3) =	501 lbf

4. Corner forces

F1 = O1(L1)/(L1+L2) =	190 lbf
F2 = O1(L2)/(L1+L2) =	311 lbf
F3 = O2(L2)/(L2+L3) =	250 lbf
F4 = O2(L3)/(L2+L3) =	250 lbf
F5 = O3(L3)/(L3+L4) =	316 lbf
F6 = O3(L4)/(L3+L4) =	185 lbf

5. Tributary length of openings

T1 = (L1*Lo1)/(L1+L2) =	2.28 ft
T2 = (L2*Lo1)/(L1+L2) =	3.72 ft
T3 = (L2*Lo2)/(L2+L3) =	3.00 ft
T4 = (L3*Lo2)/(L2+L3) =	3.00 ft
T5 = (L3*Lo3)/(L3+L4) =	3.78 ft
T6 = (L4*Lo3)/(L3+L4) =	2.22 ft

6. Unit shear beside opening

v1 = (V/L)(L1+T1)/L1 =	155 plf
v2 = (V/L)(T2+L2+T3)/L2 =	217 plf
v3 = (V/L)(T4+L3+T5)/L3 =	218 plf
v4 = (V/L)(T6+L4)/L4 =	156 plf
Check v1*L1+v2*L2+v3*L3+v4*L4=V?	2334 lbf **OK**

7. Resistance to corner forces

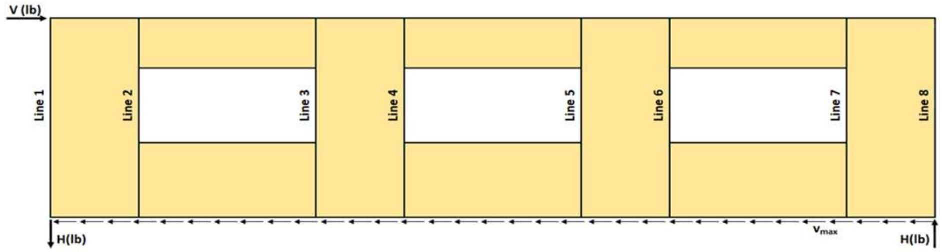
R1 = v1*L1 =	356 lbf
R2 = v2*L2 =	814 lbf
R3 = v3*L3 =	819 lbf
R4 = v4*L4 =	344 lbf

8. Difference corner force + resistance

R1-F1 =	166 lbf
R2-F2-F3 =	254 lbf
R3-F4-F5 =	253 lbf
R4-F6 =	159 lbf

9. Unit shear in corner zones

vc1 = (R1-F1)/L1 =	72 plf
vc2 = (R2-F2-F3)/L2 =	68 plf
vc3 = (R3-F4-F5)/L3 =	68 plf
vc4 = (R4-F6)/L4 =	72 plf



Check Summary of Shear Values for Three Openings

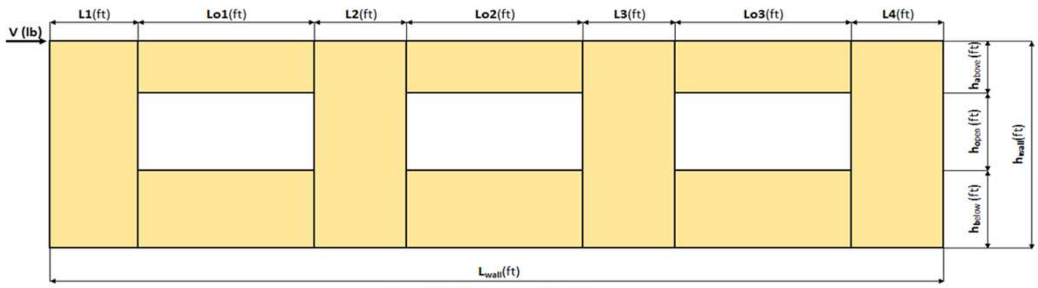
Line 1: $vc1(h_a1+h_b1)+v1(h_o1)=H$?		740	116	856 lbf
Line 2: $va1(h_a1+h_b1)-vc1(h_a1+h_b1)-v1(h_o1)=0$?	856	740	116	0
Line 3: $vc2(h_a1+h_b1)+v2(h_o1)-va1(h_a1+h_b1)=0$?	693	163	856	0
Line 4: $va2(h_a2+h_b2)-v2(h_o2)-vc2(h_a2+h_b2)=0$?	856	163	693	0
Line 5: $va2(h_a2+h_b2)-vc3(h_a2+h_b2)-v3(h_o2)=0$?	856	692	164	0
Line 6: $va3(h_a3+h_b3)-v3(h_o3)-vc3(h_a3+h_b3)=0$?	856	164	692	0
Line 7: $va3(h_a3+h_b3)-vc4(h_a3+h_b3)-v4(h_o3)=0$?	856	739	117	0
Line 8: $vc4(h_a3+h_b3)+v4(h_o3)=H$?		739	117	856 lbf

Design Summary*

Req. Sheathing Capacity	218 plf	**4-Term Deflection**		**3-Term Deflection**	
Req. Strap Force	316 lbf	**4-Term Story Drift %**		**3-Term Story Drift %**	
Req. HD Force (H)	856 lbf				
Req. Shear Wall Anchorage Force (v_{max})	78 plf				

***The Design Summary assumes that the shear wall is designed as blocked.**

Wall Line: 2nd W5



Shear Wall Calculation Variables

			Opening 1		Opening 2		Opening 3		Wall Pier Aspect Ratio		Adj. Factor
V	9240 lbf							Adj. Factor Method =			1.25-0.125h/bs
L1	3.75 ft	h_a1	2.50 ft	h_a2	2.50 ft	h_a3	2.50 ft				
L2	3.00 ft	h_o1	0.75 ft	h_o2	0.75 ft	h_o3	0.75 ft		$P1=h_o/L1=$	0.20	N/A
L3	3.00 ft	h_b1	7.75 ft	h_b2	7.75 ft	h_b3	7.75 ft		$P2=h_o/L2=$	0.25	N/A
L4	8.30 ft	Lo1	6.00 ft	Lo2	6.00 ft	Lo3	6.00 ft		$P3=h_o/L3=$	0.25	N/A
h_{wall}	11.00 ft								$P4=h_o/L4=$	0.09	N/A
L_{wall}	36.05 ft										

1. Hold-down forces: $H = Vh_{wall}/L_{wall}$ 2819 lbf

2. Unit shear above + below opening

First opening: $va1 = vb1 = H/(h_a1+h_b1)$ =	275 plf
Second opening: $va2 = vb2 = H/(h_a2+h_b2)$ =	275 plf
Third opening: $va3 = vb3 = H/(h_a3+h_b3)$ =	275 plf

3. Total boundary force above + below openings

First opening: O1 = va1 x (Lo1) =	1650 lbf
Second opening: O2 = va2 x (Lo2) =	1650 lbf
Third opening: O3 = va3 x (Lo3) =	1650 lbf

4. Corner forces

F1 = O1(L1)/(L1+L2) =	917 lbf
F2 = O1(L2)/(L1+L2) =	734 lbf
F3 = O2(L2)/(L2+L3) =	825 lbf
F4 = O2(L3)/(L2+L3) =	825 lbf
F5 = O3(L3)/(L3+L4) =	438 lbf
F6 = O3(L4)/(L3+L4) =	1212 lbf

5. Tributary length of openings

T1 = (L1*Lo1)/(L1+L2) =	3.33 ft
T2 = (L2*Lo1)/(L1+L2) =	2.67 ft
T3 = (L2*Lo2)/(L2+L3) =	3.00 ft
T4 = (L3*Lo2)/(L2+L3) =	3.00 ft
T5 = (L3*Lo3)/(L3+L4) =	1.59 ft
T6 = (L4*Lo3)/(L3+L4) =	4.41 ft

6. Unit shear beside opening

v1 = (V/L)(L1+T1)/L1 =	484 plf	
v2 = (V/L)(T2+L2+T3)/L2 =	740 plf	
v3 = (V/L)(T4+L3+T5)/L3 =	649 plf	
v4 = (V/L)(T6+L4)/L4 =	392 plf	
Check v1*L1+v2*L2+v3*L3+v4*L4=V?	9240 lbf	**OK**

7. Resistance to corner forces

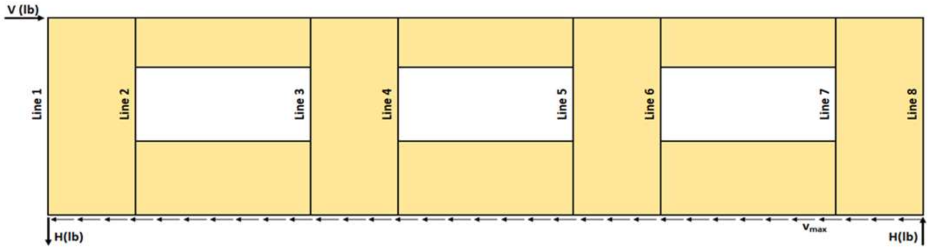
R1 = v1*L1 =	1816 lbf
R2 = v2*L2 =	2221 lbf
R3 = v3*L3 =	1946 lbf
R4 = v4*L4 =	3257 lbf

8. Difference corner force + resistance

R1-F1 =	899 lbf
R2-F2-F3 =	663 lbf
R3-F4-F5 =	683 lbf
R4-F6 =	2045 lbf

9. Unit shear in corner zones

vc1 = (R1-F1)/L1 =	240 plf
vc2 = (R2-F2-F3)/L2 =	221 plf
vc3 = (R3-F4-F5)/L3 =	228 plf
vc4 = (R4-F6)/L4 =	246 plf



Check Summary of Shear Values for Three Openings

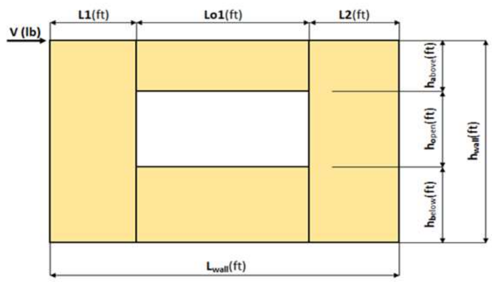
Line 1: $vc1(h_a1+h_b1)+v1(h_o1)=H$?		2456	363	2819 lbf
Line 2: $va1(h_a1+h_b1)-vc1(h_a1+h_b1)-v1(h_o1)=0$?	2819	2456	363	0
Line 3: $vc2(h_a1+h_b1)+v2(h_o1)-va1(h_a1+h_b1)=0$?	2264	555	2819	0
Line 4: $va2(h_a2+h_b2)-v2(h_o2)-vc2(h_a2+h_b2)=0$?	2819	555	2264	0
Line 5: $va2(h_a2+h_b2)-vc3(h_a2+h_b2)-v3(h_o2)=0$?	2819	2333	487	0
Line 6: $va3(h_a3+h_b3)-v3(h_o3)-vc3(h_a3+h_b3)=0$?	2819	487	2333	0
Line 7: $va3(h_a3+h_b3)-vc4(h_a3+h_b3)-v4(h_o3)=0$?	2819	2525	294	0
Line 8: $vc4(h_a3+h_b3)+v4(h_o3)=H$?		2525	294	2819 lbf

Design Summary*

Req. Sheathing Capacity	740 plf	**4-Term Deflection**		**3-Term Deflection**	
Req. Strap Force	1212 lbf	**4-Term Story Drift %**		**3-Term Story Drift %**	
Req. HD Force (H)	2819 lbf				
Req. Shear Wall Anchorage Force (v_{max})	256 plf				

***The Design Summary assumes that the shear wall is designed as blocked.**

Wall Line: 2nd W6a



Shear Wall Calculation Variables

V	2789 lbf		Opening 1	Adj. Factor Method =	1.25-0.125h/bs	
L1	3.50 ft	h_a	2.50 ft	Wall Pier Aspect Ratio		Adj. Factor
L2	4.00 ft	h_o	0.75 ft	$P1=h_o/L1=$	0.21	N/A
h_{wall}	11.00 ft	h_b	7.75 ft	$P2=h_o/L2=$	0.19	N/A
L_{wall}	13.50 ft	Lo1	6.00 ft			

1. Hold-down forces: $H = Vh_{wall}/L_{wall}$ 2273 lbf

2. Unit shear above + below opening

First opening: $va1 = vb1 = H/(h_a+h_b)$ = 222 plf

3. Total boundary force above + below openings

First opening: O1 = va1 x (Lo1) = 1330 lbf

4. Corner forces

F1 = O1(L1)/(L1+L2) = 621 lbf
F2 = O1(L2)/(L1+L2) = 709 lbf

5. Tributary length of openings

T1 = (L1*Lo1)/(L1+L2) = 2.80 ft
T2 = (L2*Lo1)/(L1+L2) = 3.20 ft

6. Unit shear beside opening

v1 = (V/L)(L1+T1)/L1 = 372 plf
v2 = (V/L)(T2+L2)/L2 = 372 plf
Check v1*L1+v2*L2=V? 2789 lbf **OK**

7. Resistance to corner forces

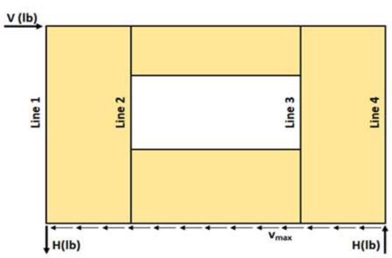
R1 = v1*L1 = 1302 lbf
R2 = v2*L2 = 1487 lbf

8. Difference corner force + resistance

R1-F1 = 681 lbf
R2-F2 = 778 lbf

9. Unit shear in corner zones

vc1 = (R1-F1)/L1 = 194 plf
vc2 = (R2-F2)/L2 = 194 plf



Check Summary of Shear Values for One Opening

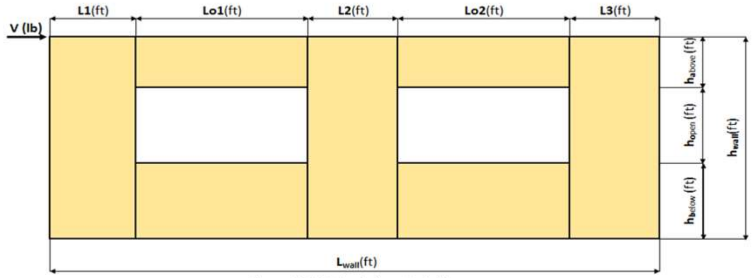
Line 1: $vc1(h_a+h_b)+v1(h_o)=H$?		1994	279	2273 lbf
Line 2: $va1(h_a+h_b)-vc1(h_a+h_b)-v1(h_o)=0$?	2273	1994	279	0
Line 3: $va1(h_a+h_b)-vc2(h_a+h_b)-v1(h_o)=0$?	2273	1994	279	0
Line 4: $vc2(h_a+h_b)+v2(h_o)=H$?		1994	279	2273 lbf

Design Summary*

Req. Sheathing Capacity	372 plf	**4-Term Deflection**		**3-Term Deflection**	
Req. Strap Force	709 lbf	**4-Term Story Drift %**		**3-Term Story Drift %**	
Req. HD Force (H)	2273 lbf				
Req. Shear Wall Anchorage Force (v_{max})	207 plf				

***The Design Summary assumes that the shear wall is designed as blocked.**

Wall Line: 2nd W8



Shear Wall Calculation Variables

			Opening 1		Opening 2
V	1992 lbf				
L1	4.20 ft	h_a1	2.50 ft	h_a2	2.50 ft
L2	4.20 ft	h_o1	0.75 ft	h_o2	0.75 ft
L3	4.20 ft	h_b1	7.75 ft	h_b2	7.75 ft
h_{wall}	11.00 ft	Lo1	5.75 ft	Lo2	5.75 ft
L_{wall}	24.10 ft				

Adj. Factor Method =		1.25-0.125h/bs
Wall Pier Aspect Ratio		Adj. Factor
$P1=h_o/L1=$	0.18	N/A
$P2=h_o/L2=$	0.18	N/A
$P3=h_o/L3=$	0.18	N/A

1. Hold-down forces: $H = Vh_{wall}/L_{wall}$ 909 lbf

2. Unit shear above + below opening

First opening: $va1 = vb1 = H/(h_a1+h_b1) =$	89 plf
Second opening: $va2 = vb2 = H/(h_a2+h_b2) =$	89 plf

3. Total boundary force above + below openings

First opening: O1 = va1 x (Lo1) =	510 lbf
Second opening: O2 = va2 x (Lo2) =	510 lbf

4. Corner forces

F1 = O1(L1)/(L1+L2) =	255 lbf
F2 = O1(L2)/(L1+L2) =	255 lbf
F3 = O2(L2)/(L2+L3) =	255 lbf
F4 = O2(L3)/(L2+L3) =	255 lbf

5. Tributary length of openings

T1 = (L1*Lo1)/(L1+L2) =	2.88 ft
T2 = (L2*Lo1)/(L1+L2) =	2.88 ft
T3 = (L2*Lo2)/(L2+L3) =	2.88 ft
T4 = (L3*Lo2)/(L2+L3) =	2.88 ft

6. Unit shear beside opening

v1 = (V/L)(L1+T1)/L1 =	139 plf
v2 = (V/L)(T2+L2+T3)/L2 =	196 plf
v3 = (V/L)(T4+L3)/L3 =	139 plf
Check v1*L1+v2*L2+v3*L3=V?	1992 lbf **OK**

7. Resistance to corner forces

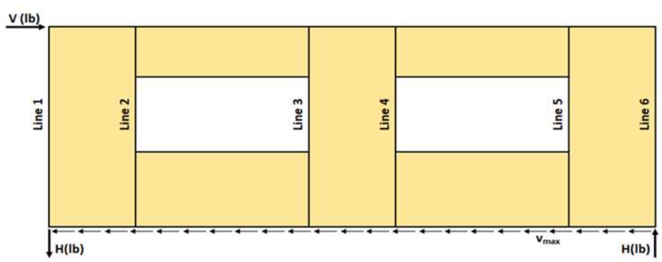
R1 = v1*L1 =	585 lbf
R2 = v2*L2 =	822 lbf
R3 = v3*L3 =	585 lbf

8. Difference corner force + resistance

R1-F1 =	330 lbf
R2-F2-F3 =	312 lbf
R3-F4 =	330 lbf

9. Unit shear in corner zones

vc1 = (R1-F1)/L1 =	79 plf
vc2 = (R2-F2-F3)/L2 =	74 plf
vc3 = (R3-F4)/L3 =	79 plf



Check Summary of Shear Values for Two Openings

Line 1: $vc1(h_a1+h_b1)+v1(h_o1)=H$?		805	104	909 lbf
Line 2: $va1(h_a1+h_b1)-vc1(h_a1+h_b1)-v1(h_o1)=0$?	909	805	104	0
Line 3: $vc2(h_a1+h_b1)+v2(h_o1)-va1(h_a1+h_b1)=0$?	762	147	909	0
Line 4: $va2(h_a2+h_b2)-v2(h_o2)-vc2(h_a2+h_b2)=0$?	909	147	762	0
Line 5: $va2(h_a2+h_b2)-vc3(h_a2+h_b2)-v3(h_o2)=0$?	909	805	104	0
Line 6: $vc3(h_a2+h_b2)+ v3(h_o2) = H$?		805	104	909 lbf

Design Summary*

Req. Sheathing Capacity	196 plf	**4-Term Deflection**		**3-Term Deflection**	
Req. Strap Force	255 lbf	**4-Term Story Drift %**		**3-Term Story Drift %**	
Req. HD Force	909 lbf				
Req. Shear Wall Anchorage Force	83 plf				

***The Design Summary assumes that the shear wall is designed as blocked.**

RISA 3d and Risa Connection Were used for the moment frame analysis

Column Base Plate:

Max Vertical Force =	18.00	kips
Max Uplift Force =	8.60	kips
Max Shear =	8.14	kips

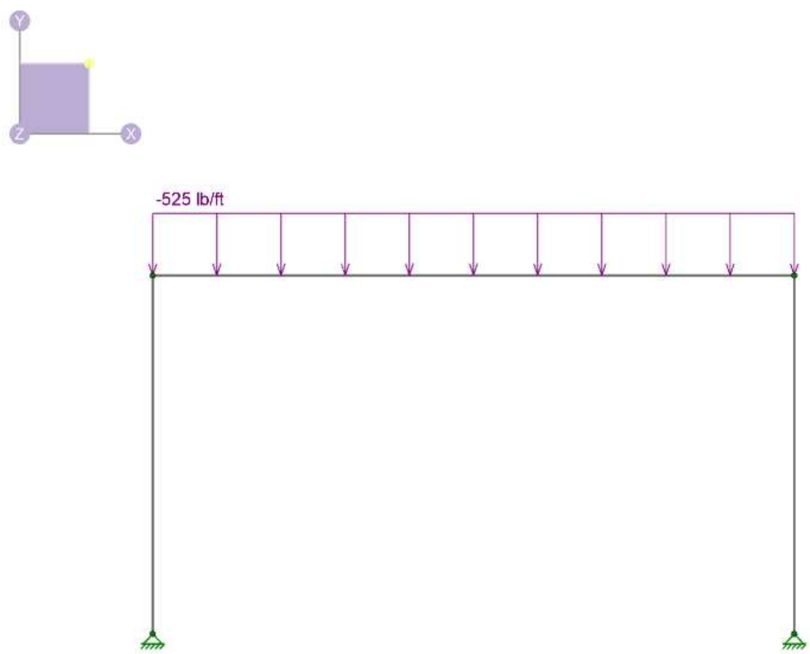
Column: W10x88

N =	17.00	in
B =	17.00	in
d =	10.00	in
bf =	10.50	in
X =	1.00	
m =	3.75	
n =	4.30	
n' =	2.56	
ʎ =	1.00	
l =	4.3	
tmin =	0.27	in

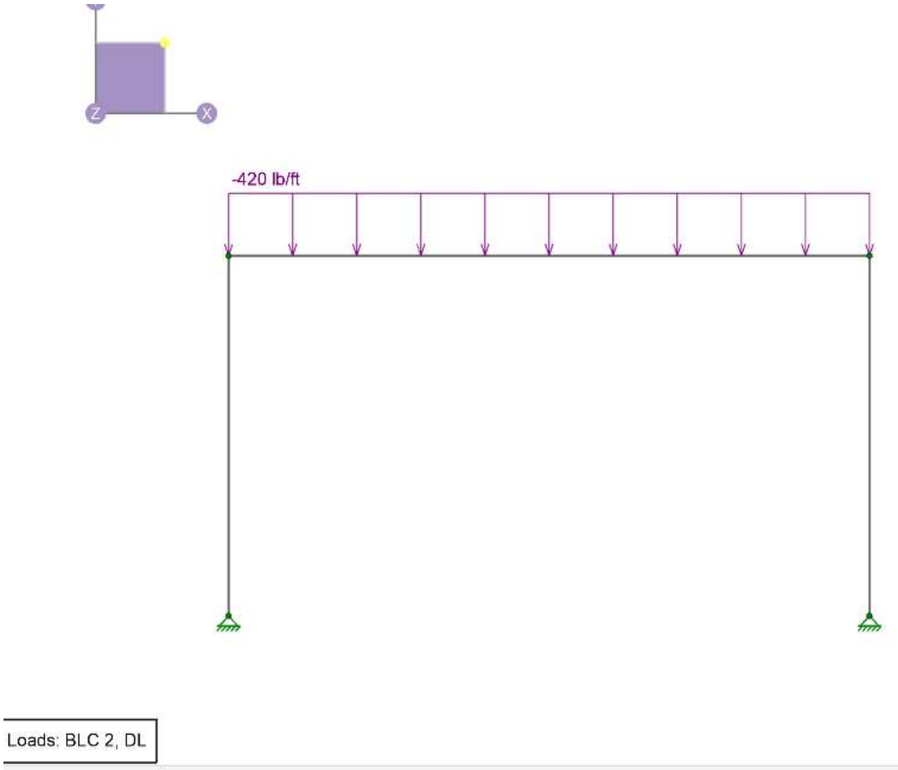
Use (4) 5/8" Dia F1554 Fy36 bolt

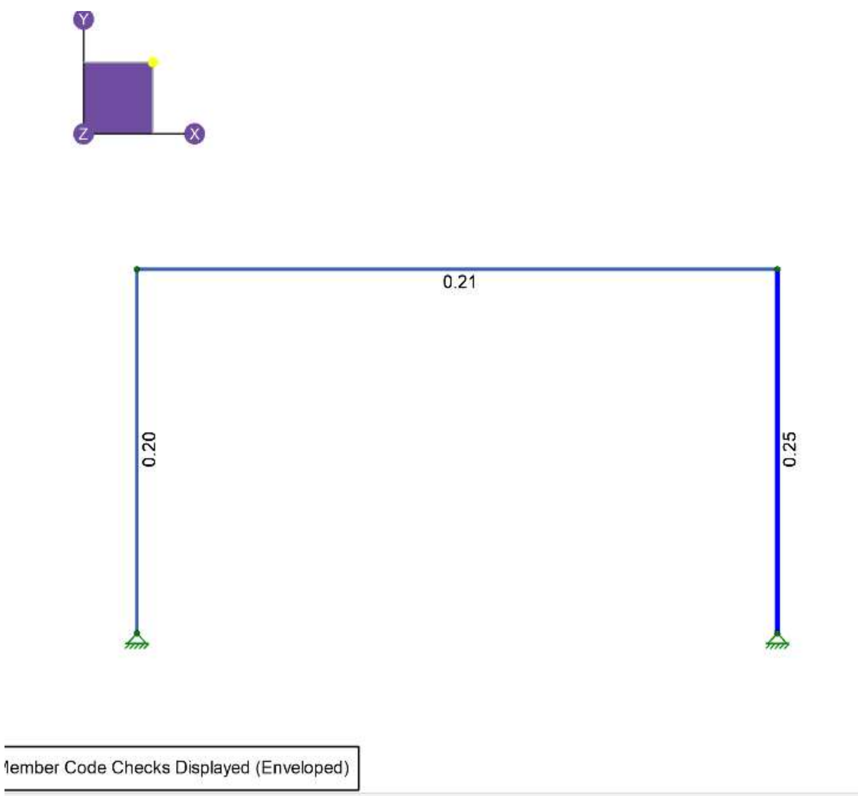
Max Shear =	8.135	kips
Uplift =	8.6	kips

Loads: BLC 1, EL



Loads: BLC 3, Live


-420 lb/ft
Loads: BLC 2, DL

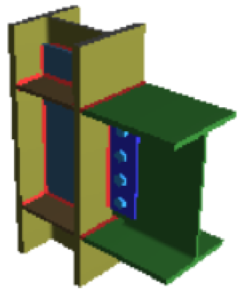

0.21
0.20
0.25
lember Code Checks Displayed (Enveloped)

Envelope AISC 15th (360-16): LRFD Steel Code Checks

Hot Rolled Steel | Cold Formed Steel | Wood | Concrete Beams | Concrete Columns | Aluminu

	Member	Shape	Code Check	Loc[ft]	LC	Shear Che...	Loc[ft]	Dir	LC
1	M1	W16X67	0.215	20	1	0.082	20	y	1
2	M3	W10X88	0.2	11	2	0.039	11	y	2
3	M5	W10X88	0.254	11	1	0.048	11	y	1

M1 I - M3: LRFD Results Report



Material Properties:

Column	W10X88	A992	F_y = 50.00 ksi	F_u = 65.00 ksi
Beam	W16X67	A992	F_y = 50.00 ksi	F_u = 65.00 ksi
Plate	P0.38x4.00x12.00	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Doubler	P0.75x8.82x23.59	A572 Gr.50	F_y = 50.00 ksi	F_u = 65.00 ksi
Transverse Stiffener	P0.50x4.10x8.82	A572 Gr.50	F_y = 50.00 ksi	F_u = 65.00 ksi

Input Data:

Shear Load	-23.68 kips	*User Input Shear Load*
Moment	-264.51 kips-ft	*User Input Moment*
Axial Load	0.13 kips	*User Input Axial Force (compression)*
Puf_c	203.01 kips	*Required Flange Force (compression)*
Puf_t	203.01 kips	*Required Flange Force (tension)*
Top Column Dist	0.00 in	*User Input Top Column Dist*
Column Force	-23.67 kips	*User Input Column Force*
Story Shear	-24.05 kips	*User Input Story Shear*

Governing LC: 3D - 4 - LC 4*: IBC 21/ASCE Strength 7

Note: Unless specified, all code references are from AISC 360-16

Limit State	Required	Available	Unity Check	Result
Geometry Restrictions at Beam				**PASS**
Check Min Bolt Spacing	**Pass**	*Condition: S_{min} >= (2+2/3) * d_{bolt}*	*(J3.3)*	
S_{min}	3.00 in	*Min bolt spacing*		
d_{bolt}	0.75 in	*Bolt diameter*		
Check Max Bolt Spacing	**Pass**	*Condition: S_{max}<= min(12.00 in, 24*t)*	*(J3.5a)*	
S_{max}	3.00 in	*Max bolt spacing*		
t	0.38 in	*Thickness of governing element (Plate)*		
Check Min Edge Distance	**Pass**	*Condition: ED_{min} >= ED_{allow}*	*(J3.4)*	
Check Max Edge Distance	**Pass**	*Condition: ED_{max} <= min (6.00 in, 12*t)*	*(J3.5)*	
Column Weld Limitations				**PASS**
Weld Max/Min Size, Length			(J2.2b)	
Check Weld Min Size	**Pass**			
D	0.25 in	*Weld size*		
D_{min}	0.19 in	*Min size allowed per Table J2.4*		
t_{min}	0.38 in	*Controlling member thickness*		
Check Weld Min Length	**Pass**	*Condition: L_{min} ≥ 4*D per J2.2b*		
D	0.25 in	*Weld size*		
L_{min}	12.00 in	*Min weld segment length*		
Beam Web Shear Yield	23.68 kips	159.62 kips	**0.15**	**PASS**
$R_n = 0.6 *F_y*A_{gv}*C_{v1}$		**ϕ = 1.00**	(G2-1)	
F_y	50.00 ksi	*Minimum yield stress of material*		
A_{gv}	5.32 in^2	*Gross area subject to shear*		

continued on next page...

M1 I - M3: LRFD Results Report (continued):

Limit State	Required	Available	Unity Check	Result
C_{v1}	1.00	*Web shear coefficient (G2-2)*		
ϕR_n	159.62 kips	*Shear yield strength*		
Plate Shear Yield	23.68 kips	97.20 kips	**0.24**	**PASS**
$R_n = 0.6 * F_y * A_{gv}$		$\phi = 1.00$	(J4-3)	
F_y	36.00 ksi	*Minimum yield stress of material*		
A_{gv}	4.50 in^2	*Gross area subject to shear*		
ϕR_n	97.20 kips	*Shear yield strength*		
Beam Web Shear Rupture	23.68 kips	115.19 kips	**0.21**	**PASS**
$R_n = 0.6 * F_u * A_{nv}$		$\phi = 0.75$	(J4-4)	
F_u	65.00 ksi	*Minimum tensile stress of material*		
A_{nv}	3.94 in^2	*Net area subject to shear*		
ϕR_n	115.19 kips	*Shear rupture strength*		
Plate Shear Rupture	23.68 kips	83.19 kips	**0.28**	**PASS**
$R_n = 0.6 * F_u * A_{nv}$		$\phi = 0.75$	(J4-4)	
F_u	58.00 ksi	*Minimum tensile stress of material*		
A_{nv}	3.19 in^2	*Net area subject to shear*		
ϕR_n	83.19 kips	*Shear rupture strength*		
Beam Block Shear	23.68 kips	116.62 kips	**0.20**	**PASS**
$R_n = [\min(0.6 * F_u * A_{nv}, 0.6 * F_y * A_{gv}) + U_{bs} * F_u * A_{nt}]$		$\phi = 0.75$	(J4-5)	
Failure mode is considered for negative shear load				
A_{gv}	4.50 in^2	*Gross area subject to shear*		
A_{nv}	3.29 in^2	*Net area subject to shear*		
U_{bs}	1.00	*Uniform tension stress factor*		
A_{nt}	0.42 in^2	*Net area subject to tension*		
F_u	65.00 ksi	*Minimum tensile stress of material*		
F_y	50.00 ksi	*Minimum yield stress of material*		
ϕR_n	116.62 kips	*Block shear strength*		
Plate Block Shear at Beam	23.68 kips	89.28 kips	**0.27**	**PASS**
$R_n = [\min(0.6 * F_u * A_{nv}, 0.6 * F_y * A_{gv}) + U_{bs} * F_u * A_{nt}]$		$\phi = 0.75$	(J4-5)	
Failure mode is considered for negative shear load				
A_{gv}	3.94 in^2	*Gross area subject to shear*		
A_{nv}	2.79 in^2	*Net area subject to shear*		
U_{bs}	1.00	*Uniform tension stress factor*		
A_{nt}	0.59 in^2	*Net area subject to tension*		
F_u	58.00 ksi	*Minimum tensile stress of material*		
F_y	36.00 ksi	*Minimum yield stress of material*		
ϕR_n	89.28 kips	*Block shear strength*		
Bolt Bearing at Beam Web	23.68 kips	71.57 kips	**0.33**	**PASS**
$R_n = 4 * R_{n\text{-spacing}}$		$\phi = 0.75$	(section J3.10)	
Failure mode is considered for negative shear load				
d	0.75 in	*Bolt diameter*		

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M1 I - M3: LRFD Results Report (continued):

Limit State	Required	Available	Unity Check	Result
F_u	65.00 ksi	*Minimum tensile stress of material*		
t	0.40 in	*Thickness of material*		
$L_{c\text{-}spacing}$	2.19 in	*Vertical distance from edges of adjacent holes*		
$R_{n\text{-}spacing}$	23.86 kips	*Strength at spaces = min($R_{n\text{-}spacing\text{-}tearout}$, $R_{n\text{-}bearing}$, $R_{n\text{-}bolt}$)*		
$R_{n\text{-}bearing}$	46.21 kips	*Bearing = 2.4*d*t*F_u*		
$R_{n\text{-}spacing\text{-}tearout}$	67.40 kips	*Tear out at spaces = 1.2*$L_{c\text{-}spacing}$*t*F_u*		
$R_{n\text{-}bolt}$	23.86 kips	*Bolt shear strength $R_{n\text{-}bolt}$=F_{nv}*A_{bolt}*		
F_{nv}	54.00 ksi	*Nominal shear stress of bolt*		
ϕR_n	71.57 kips	*Bolt bearing strength*		
Bolt Bearing at Shear Plate	23.68 kips	71.57 kips	**0.33**	**PASS**
R_n = 1*$R_{n\text{-}edge}$ + 3*$R_{n\text{-}spacing}$		**ϕ = 0.75**	(section J3.10)	
Failure mode is considered for negative shear load				
d	0.75 in	*Bolt diameter*		
F_u	58.00 ksi	*Minimum tensile stress of material*		
t	0.38 in	*Thickness of material*		
$L_{c\text{-}edge}$	1.09 in	*Vertical distance from edge of hole to edge of material*		
$L_{c\text{-}spacing}$	2.19 in	*Vertical distance from edges of adjacent holes*		
$R_{n\text{-}edge}$	23.86 kips	*Strength at edge = min($R_{n\text{-}edge\text{-}tearout}$, $R_{n\text{-}bearing}$, $R_{n\text{-}bolt}$)*		
$R_{n\text{-}spacing}$	23.86 kips	*Strength at spaces = min($R_{n\text{-}spacing\text{-}tearout}$, $R_{n\text{-}bearing}$, $R_{n\text{-}bolt}$)*		
$R_{n\text{-}bearing}$	39.15 kips	*Bearing = 2.4*d*t*F_u*		
$R_{n\text{-}edge\text{-}tearout}$	28.55 kips	*Tear out at edge = 1.2*$L_{c\text{-}edge}$*t*F_u*		
$R_{n\text{-}spacing\text{-}tearout}$	57.09 kips	*Tear out at spaces = 1.2*$L_{c\text{-}spacing}$*t*F_u*		
$R_{n\text{-}bolt}$	23.86 kips	*Bolt shear strength $R_{n\text{-}bolt}$=F_{nv}*A_{bolt}*		
F_{nv}	54.00 ksi	*Nominal shear stress of bolt*		
ϕR_n	71.57 kips	*Bolt bearing strength*		
Bolt Shear at Beam Web	23.68 kips	71.57 kips	**0.33**	**PASS**
R_n = F_{nv}*A_b*N_{bolt}*C		**ϕ = 0.75**	(J3-1)	
F_{nv}	54.00 ksi	*Shear stress N type*		
A_b	0.44 in^2	*Area of bolt*		
N_{bolt}	4	*Number of bolts*		
C	1.00	*Eccentricity coefficient*		
ϕR_n	71.57 kips	*Bolt shear rupture strength*		
Column Weld Strength	23.68 kips	133.63 kips	**0.18**	**PASS**
ϕRn = 2 * C_1 * α * 1.392 * D_{16} * L **Double Fillet** **1.392 = ϕ * 0.6 * F_{E70} * $2^{0.5}$/2 * 1/16, ϕ=0.75 (AISC 15th Eqn 8-2a)**				
C_1	1.00	*Electrode strength coefficient (AISC 15th table 8-3)*		
t	0.99 in	*Base material thickness (column)*		
α	1.00	*Base material proration factor (re-arrangement of AISC 15th Eqn 9-2)*		
D_{16}	4.00	*Weld fillet size in sixteenths of an inch*		
L	12.00 in	*Weld length*		
ϕRn	133.63 kips	*Weld strength*		

continued on next page...

M1 I - M3: LRFD Results Report (continued):

Limit State	Required	Available	Unity Check	Result
Flange Weld Strength				**PASS**
Complete Joint Penetration			(J2.6)	
Req'd Filler (Column)	E70			
Req'd Filler (Beam)	E70			
Weld Electrode (Filler)	E70			
Beam Flange Tensile Yield	203.01 kips	305.23 kips	**0.67**	**PASS**
$R_n = F_y*A_g$		**ϕ = 0.90**	(J4-1)	
F_y	50.00 ksi	*Minimum yield stress of material*		
A_g	6.78 in^2	*Gross area subject to compression*		
ϕR_n	305.23 kips	*Tensile yield strength*		
d_m	15.64 in	*Moment arm between the flange forces*		
R_{req}	203.01 kips	*Required Flange Force (tension)*		
Beam Flange Tensile Rupture	203.01 kips	330.67 kips	**0.61**	**PASS**
$R_n = F_u*A_n$		**ϕ = 0.75**	(J4-2)	
F_u	65.00 ksi	*Minimum tensile stress of material*		
A_n	6.78 in^2	*Net area subject to tension*		
ϕR_n	330.67 kips	*Tensile rupture strength*		
d_m	15.64 in	*Moment arm between the flange forces*		
R_{req}	203.01 kips	*Required Flange Force (tension)*		
Beam Flange Compression	203.01 kips	305.24 kips	**0.67**	**PASS**
$R_n = F_y * A_g$		**ϕ = 0.9**	(J4-6)	
K	0.65	*Effective length factor*		
L	0.18 ft	*Unbraced length*		
r	0.02 ft	*Radius of gyration*		
L_c	1.41 in	*Effective length, L_c=K*L*		
L_c/r	7.33	*Plate slenderness check from J4-6*		
F_y	50.00 ksi	*Capacity = Minimum Yield stress for L_c/r <= 25*		
A_g	6.78 in^2	*Gross area subject to compression*		
ϕR_n	305.24 kips	*Compressive strength*		
Column Flange Bending	203.01 kips	137.83 kips		**N/A**
$R_n = 0.5 * 6.25 * F_{yf} * t_f^2$		**ϕ = 0.90**	(J10-1)	
d_{end}	0.33 in	*Distance from concentrated force to top of column*		
F_{yf}	50.00 ksi	*Minimum yield stress of column*		
t_f	0.99 in	*Column flange thickness*		
ϕR_n	137.83 kips	*Column flange local bending*		
d_m	15.64 in	*Moment arm between the flange forces*		
R_{req}	203.01 kips	*Required Flange Force (tension)*		
Unbalanced force	65.18 kips	*Transverse stiffeners are provided*		
Column Web Yielding	203.01 kips	297.42 kips	**0.68**	**PASS**
$R_n = (2.5 * k + l_b) * F_y * t_{w\text{-}eq}$		**ϕ = 1.00**	(J10-3)	
d_{end}	0.33 in	*Distance from concentrated force to top of column*		
d_{col}	10.80 in	*Column depth*		
F_y	50.00 ksi	*Minimum yield stress of column*		
$t_{w\text{-}eq}$	1.35 in	*Equivalent web thickness for yielding. $t_{w\text{-}eq}=t_w+t_{dp}*F_{ydp}/F_y$*		

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M1 I - M3: LRFD Results Report (continued):

Limit State	Required	Available	Unity Check	Result
t_w	0.60 in	*Column web thickness*		
t_{dp}	0.75 in	*Doubler plate thickness*		
F_{ydp}	50.00 ksi	*Minimum yield stress of doubler plate*		
k	1.49 in	*Distance from outer face of the flange to the web toe of the fillet*		
l_b	0.67 in	*Length of bearing*		
ϕR_n	297.42 kips	*Column web local yielding*		
d_m	15.64 in	*Moment arm between the flange forces*		
R_{req}	203.01 kips	*Required Flange Force (worst) in bolts due to moment*		
Column Web Buckling	203.01 kips	1069.86 kips	**0.19**	**PASS**
$R_n = 12 * t_w^3 * (E * F_y)^{0.5} / h *Q_f + 12 * t_{dp}^3 * (E * F_{ydp})^{0.5} / h *Q_f$		**ϕ = 0.90**	(J10-8)	
d_{end}	0.33 in	*Distance from concentrated force to top of column*		
d_{col}	10.80 in	*Column depth*		
t_w	0.60 in	*Column web thickness*		
F_y	50.00 ksi	*Minimum yield stress of column*		
E	29000.00 ksi	*Modulus of elasticity*		
t_{dp}	0.75 in	*Doubler plate thickness*		
F_{ydp}	50.00 ksi	*Minimum yield stress of doubler plate*		
h	7.82 in	*Clear distance between flanges* $h=d-2*k_{des}$		
Q_f	1.00	*Chord stress interaction parameter*		
ϕR_n	1069.86 kips	*Column web compression buckling*		
d_m	15.64 in	*Moment arm between the flange forces*		
R_{req}	203.01 kips	*Required Flange Force (compression)*		
Column Web Crippling	203.01 kips	445.97 kips	**0.46**	**PASS**
$R_n= 0.4*t_w^2*(1+3*(l_b/d_{col})*(t_w/t_f)^{1.5})*(E*F_y*t_f/t_w)^{0.5}*Q_f + 0.4*t_{dp}^2*(1+3*(l_b/d_{col})*(t_{dp}/t_f)^{1.5})*(E*F_{ydp}*t_f/t_{dp})^{0.5} *Q_f$		**ϕ = 0.75**	(J10-5a)	
d_{end}	0.33 in	*Distance from concentrated force to top of column*		
l_b/d_{col}	0.06	*Bearing length to column depth ratio*		
d_{col}	10.80 in	*Column depth*		
t_w	0.60 in	*Column web thickness*		
t_f	0.99 in	*Column flange thickness*		
l_b	0.67 in	*Length of bearing*		
F_y	50.00 ksi	*Minimum yield stress of column*		
E	29000.00 ksi	*Modulus of elasticity*		
t_{dp}	0.75 in	*Doubler plate thickness*		
F_{ydp}	50.00 ksi	*Minimum yield stress of doubler plate*		
Q_f	1.00	*Chord stress interaction parameter*		
ϕR_n	445.97 kips	*Column web crippling capacity*		
d_m	15.64 in	*Moment arm between the flange forces*		
R_{req}	203.01 kips	*Required Flange Force (compression)*		

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M1 I - M3: LRFD Results Report (continued):

Limit State	Required	Available	Unity Check	Result
Column Panel Zone Shear	227.06 kips	176.42 kips		**N/A**
$R_n = 0.60 * F_{yc} * d_c * t_{wc}$ ($\alpha * P_r <= 0.4 * P_c$), $\alpha = 1.0$		**$\phi = 0.90$**	(J10-9)	
P_r	23.67 kips	*Axial force in the column at the connection*		
P_c	1300.00 kips	*$P_c = P_y = F_{yc} * A$*		
F_{yc}	50.00 ksi	*Minimum yield stress of column*		
A	26.00 in^2	*Column cross-sectional area*		
d_c	10.80 in	*Column depth*		
t_{wc}	0.60 in	*Column web thickness*		
t_{fc}	0.99 in	*Column flange thickness*		
b_{fc}	10.30 in	*Column flange width*		
d_b	16.30 in	*Beam depth*		
ϕR_n	176.42 kips	*Web panel zone capacity*		
R_f	203.01 kips	*Beam flange force. $R_f = M / d_m$*		
M	264.51 kips-ft	*Moment demand*		
d_m	15.64 in	*Moment arm from centerline forces*		
V_{above}	24.05 kips	*Story shear in column above connection*		
R_p	227.06 kips	*Panel zone shear demand. $R_p = R_f + V_{above}$*		
Unbalanced moment	65.98 kips-ft	*Doubler plate is provided*		
Unbalanced force	50.64 kips	*Doubler plate is provided*		
Geometry Restrictions for Trans. Stiffener				**PASS**
Check Min Thickness	**Pass**	*Condition: ts>=max(tf/2, bs/16) (J10.8)*		
tf	0.67 in	*Thickness of beam flange*		
bs	4.85 in	*Stiffener width*		
ts	0.50 in	*Stiffener thickness*		
Check Min Depth	**Pass**	*Condition: ds>=(dc-2*tcf)/2 (J10.8)*		
dc	10.80 in	*Column depth*		
tcf	0.99 in	*Column flange thickness*		
ds	8.82 in	*Stiffener depth*		
Trans. Stiffener Yield at Column Flange	65.18 kips	173.14 kips	**0.38**	**PASS**
$R_n = 2 * F_{yst} * A_{st}$		**$\phi = 0.9$**	(J4-1)	
t_s	0.50 in	*Stiffener thickness*		
b_s	4.10 in	*Stiffener width*		
clip	0.25 in	*Stiffener corner clip dimension*		
F_{yst}	50.00 ksi	*Minimum yield stress of Stiffener*		
A_{st}	1.92 in^2	*Cross-sectional area = $(b_s$ - clip$) * t_s$*		
ϕR_n	173.14 kips	*Available transverse stiffener strength*		
R_{req}	65.18 kips	*Required transverse stiffener strength*		
Trans. Stiffener Shear at Column Web	65.18 kips	222.75 kips	**0.29**	**PASS**
$R_n = 2 * 0.6 * F_{yst} * A_{gv}$		**$\phi = 1.0$**	(J4-3)	
t_s	0.50 in	*Stiffener thickness*		
d_p	8.82 in	*Stiffener depth*		
clip	0.70 in	*Stiffener corner clip dimension*		
F_{yst}	50.00 ksi	*Minimum yield stress of Stiffener*		
A_{gv}	3.71 in^2	*Cross-sectional area = $(d_p$ - 2*clip$) * t_s$*		
ϕR_n	222.75 kips	*Available transverse stiffener strength*		

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M1 I - M3: LRFD Results Report (continued):

Limit State	Required	Available	Unity Check	Result
R_{req}	65.18 kips	*Required transverse stiffener strength*		
Doubler Geometry Limitation				**PASS**
Check Fillet Encroachment	**Pass**	*Condition: $t_p >= k - t_f$ (DG-13 (4.4- 4))*		
t_p	0.75 in	*Doubler plate thickness*		
k	1.69 in	*Distance from outer face of the flange to the web toe of the fillet*		
t_f	0.99 in	*Column flange thickness*		
Check Minimum Plate Thickness	**Pass**	*Condition: $t_p >= t_{eff}$ (DG-13 (4.4-1))*		
t_p	0.75 in	*Doubler plate thickness*		
R_{req}	50.64 kips	*Required doubler plate shear force*		
$N_{doubler}$	1	*Count of doublers*		
V_{udp}	50.64 kips	*Condition: $V_{udp} = R_{req}/N_{doubler}$*		
Fy_{st}	50.00 ksi	*Yield stress of doubler plate*		
d_c	10.80 in	*Column depth*		
t_{eff}	0.17 in	*Minimum doubler plate thickness: $t_{eff} = V_{udp}/(0.9*0.6*Fy_{st}*d_c)$*		
Doubler Shear Buckling				**PASS**
Check condition : $h / t_p <= 2.24 * (E / F_y)^{0.5}$			(G2-2)	
t_p	0.75 in	*Doubler Plate thickness*		
d	10.80 in	*Depth of the column*		
K_{des}	1.49 in	*K_{des} of the column*		
h	7.82 in	*Clear distance between flanges = $d - 2 * K_{des}$*		
E	29000.00 ksi	*Modulus of elasticity*		
F_y	50.00 ksi	*Minimum Yield Stress*		
Doubler Plate Shear Yield	50.64 kips	243.00 kips	**0.21**	**PASS**
$R_n = N_{doubler} * 0.6 * F_{y\_st} * A_w * C_v$		**$\phi = 1.0$**	(G2-1)	
$N_{doubler}$	1	*Count of doublers*		
t_p	0.75 in	*Doubler plate thickness*		
F_{y_st}	50.00 ksi	*Yield stress of doubler plate*		
d_c	10.80 in	*Column depth*		
A_w	8.10 in^2	*Area of web = $t_p * d_c$ DG-13 (4.4-1)*		
C_v	1.00	*Shear coefficient*		
ϕR_n	243.00 kips	*Plate shear yield strength*		
R_{req}	50.64 kips	*Required doubler plate shear force*		
Trans. Stiffener Weld Limitations				**PASS**
Weld Min Size			(J2.2b)	
Check Weld Min Size	**Pass**			
D	0.38 in	*Weld size*		
D_{min}	0.19 in	*Min size allowed per Table J2.4*		
t_{min}	0.50 in	*Controlling member thickness*		
Check Stiffener Development	**Pass**			
D_{16}	6.00	*Weld fillet size in sixteenths of an inch*		
D_{min}	5.39	*Min weld to develop stiffener strength*		
Doubler Weld at Column Flange Limitations				**PASS**
Weld Min Size				

continued on next page...

M1 I - M3: LRFD Results Report (continued):

Limit State	Required	Available	Unity Check	Result
Check Weld Min Size (J2.2b)	**Pass**			
D	0.25 in	*Weld size*		
D_{min}	0.25 in	*Min size allowed per Table J2.4*		
t_{min}	0.75 in	*Controlling member thickness*		
Check Weld Min Size (DG-13)	**Pass**	*Condition: D ≥ W_{min} (DG-13 (4.4-7))*		
D	0.25 in	*Weld size*		
Fy_{st}	50.00 ksi	*Yield stress of doubler plate*		
t_{eff}	0.17 in	*Minimum doubler plate thickness (DG-13 (4.4-1)), see Doubler Geometry Limitation check*		
C_1	1.00	*Electrode strength coefficient (AISC 15th table 8-3)*		
F_{EXX}	70.00 ksi	*Welding electrode specified minimum strength: F_{EXX} = (70 ksi) *C_1*		
W_{min}	0.25 in	*Minimum weld size: $W_{min} = max((0.9*0.6*Fy_{st}*t_{eff}*(2^{0.5}))/(0.75*0.6*F_{EXX}), t_{eff}*(2^{0.5}))$*		
Doubler Weld at Column Web Limitations				**PASS**
Weld Min Size			(J2.2b)	
Check Weld Min Size	**Pass**			
D	0.25 in	*Weld size*		
D_{min}	0.25 in	*Min size allowed per Table J2.4*		
t_{min}	0.60 in	*Controlling member thickness*		
Trans. Stiffener Weld Strength at Column Flange	65.18 kips	192.81 kips	**0.34**	**PASS**
$\phi Rn = 1.5 * 4 * C_1 * \alpha * 1.392 * D_{16} * L$				
Double Fillet				
$1.392 = \phi * 0.6 * F_{E70} * 2^{0.5}/2 * 1/16$, ϕ=0.75 (AISC 15th Eqn 8-2a)				
C_1	1.00	*Electrode strength coefficient (AISC 15th table 8-3)*		
t	0.99 in	*Base material thickness (column)*		
α	1.00	*Base material proration factor (re-arrangement of AISC 15th Eqn 9-2)*		
D_{16}	6.00	*Weld fillet size in sixteenths of an inch*		
L	3.85 in	*Weld length*		
ϕRn	192.81 kips	*Weld strength*		
Trans. Stiffener Weld Strength at Panel Zone			**0.26**	**PASS**
$\phi Rn = 2 * C_1 * \alpha * 1.392 * D_{16} * L$				
Double Fillet				
$1.392 = \phi * 0.6 * F_{E70} * 2^{0.5}/2 * 1/16$, ϕ=0.75 (AISC 15th Eqn 8-2a)				
Weld at Doubler Plate	**Pass**			
C_1	1.00	*Electrode strength coefficient (AISC 15th table 8-3)*		
t	0.75 in	*Base material thickness (doubler)*		
α	1.00	*Base material proration factor (re-arrangement of AISC 15th Eqn 9-2)*		
D_{16}	6.00	*Weld fillet size in sixteenths of an inch*		
L	8.32 in	*Weld length*		
ϕRn	138.98 kips	*Weld strength*		
R_{req}	32.59 kips	*Required weld strength*		
UC	0.23	*Unity check*		
Weld at Column Web	**Pass**			

continued on next page...

M1 I - M3: LRFD Results Report (continued):

Limit State	Required	Available	Unity Check	Result
C_1	1.00	*Electrode strength coefficient (AISC 15th table 8-3)*		
t	0.60 in	*Base material thickness (column)*		
α	1.00	*Base material proration factor (re-arrangement of AISC 15th Eqn 9-2)*		
D_{16}	6.00	*Weld fillet size in sixteenths of an inch*		
L	7.43 in	*Weld length*		
ϕRn	124.03 kips	*Weld strength*		
R_{req}	32.59 kips	*Required weld strength*		
UC	0.26	*Unity check*		
Doubler Weld Strength at Column Flange	50.64 kips	131.32 kips	**0.39**	**PASS**
$\phi Rn = C_1 * \alpha * 1.392 * D_{16} * L$				
Single Fillet				
$1.392 = \phi * 0.6 * F_{E70} * 2^{0.5}/2 * 1/16$, ϕ=0.75 (AISC 15th Eqn 8-2a)				
C_1	1.00	*Electrode strength coefficient (AISC 15th table 8-3)*		
t	0.99 in	*Base material thickness (column)*		
α	1.00	*Base material proration factor (re-arrangement of AISC 15th Eqn 9-2)*		
D_{16}	4.00	*Weld fillet size in sixteenths of an inch*		
L	23.59 in	*Weld length*		
ϕRn	131.32 kips	*Weld strength*		

M1 I - M3: Connection Properties Report

Column/Beam Direct Weld Moment Connection

Connection	
Connection Title	M1 I - M3
Connection Type	Column/Beam Direct Weld Moment Connection
Seismic Detailing	
Seismic System	None
Connection Category	
Beam Connection	Bolted
Column Connection Type	Flange
Transverse Stiffeners	Yes
Web Doublers	Yes
Web Doublers Configuration	One Side
Loading (LRFD)	
Custom?	No
Shear Load	-23.682 kips
Axial Load	0.129 kips
Moment Load	-264.505 kips-ft
Top Column Dist	0.000 in
Column Force	-23.675 kips
Story Shear	-24.046 kips
Components	
Column Section	W10X88
Material	A992
Beam Section	W16X67
Material	A992
Hole Type	STD
Plate Section	P0.38x4.00x12.00
Material	A36
Thickness	0.375 in
Width	4.000 in
Depth	12.000 in
Hole Type	STD
Doubler Section	P0.75x8.82x23.59
Material	A572 Gr.50
Fy	50.000 ksi
Fu	65.000 ksi
E	29000.000 ksi
Thickness	0.750 in
Width	8.820 in
Depth	23.585 in
Transverse Stiffener Section	P0.50x4.10x8.82
Material	A572 Gr.50
Fy	50.000 ksi
Fu	65.000 ksi
E	29000.000 ksi
Full Depth Stiffener	Yes
Thickness	0.500 in
Min Width	4.098 in
Max Width	4.848 in
Depth	8.820 in
Column Weld	E70
Type	Double Fillet
Fillet Size	4.000 Sixteenths
Beam Bolts	3/4" Group A-N

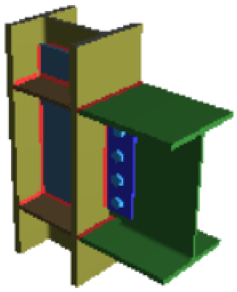
continued on next page...

M1 I - M3: Connection Properties Report (continued):

Beam Bolts	Group A-N
Diameter, in.	3/4"
Rows	1
Bolts per Row	4
Longitudinal Spacing	3.000 in
Transverse Spacing	3.000 in
Slip Critical	No
Moment Weld	E70
Type	CJP
Transverse Stiffener Weld	E70
Type	Double Fillet
Fillet Size	6.000 Sixteenths
Doubler Flange Weld	E70
Type	Fillet
Fillet Size	4.000 Sixteenths
Doubler Web Weld	E70
Type	Fillet
Fillet Size	4.000 Sixteenths
Assembly	
Column/Beam Clearance	0.500 in
Plate Vertical Position	2.000 in
Beam Bolts Edge Distance Dimensions	
Beam Bolts/Beam Edge Dist	1.500 in
Beam Bolts Horz Edge Dist	2.000 in
Beam Bolts Vert Edge Dist	1.500 in

M1 J - M5: LRFD Results Report

Column/Beam Direct Weld Moment Connection



Material Properties:

Column	W10X88	A992	F_y = 50.00 ksi	F_u = 65.00 ksi
Beam	W16X67	A992	F_y = 50.00 ksi	F_u = 65.00 ksi
Plate	P0.38x4.00x12.00	A36	F_y = 36.00 ksi	F_u = 58.00 ksi
Doubler	P0.75x8.82x23.59	A572 Gr.50	F_y = 50.00 ksi	F_u = 65.00 ksi
Transverse Stiffener	P0.50x4.10x8.82	A572 Gr.50	F_y = 50.00 ksi	F_u = 65.00 ksi

Input Data:

Note: Unless specified, all code references are from AISC 360-10

Limit State	Required	Available	Unity Check	Result

M1 J - M5: Connection Properties Report

Column/Beam Direct Weld Moment Connection

Connection	
Connection Title	M1 J - M5
Connection Type	Column/Beam Direct Weld Moment Connection
Seismic Detailing	
Seismic System	None
Connection Category	
Beam Connection	Bolted
Column Connection Type	Flange
Transverse Stiffeners	Yes
Web Doublers	Yes
Web Doublers Configuration	Both Sides
Components	
Column Section	W10X88
Material	A992
Beam Section	W16X67
Material	A992
Hole Type	STD
Plate Section	P0.38x4.00x12.00
Material	A36
Thickness	0.375 in
Width	4.000 in
Depth	12.000 in
Hole Type	STD
Doubler Section	P0.75x8.82x23.59
Material	A572 Gr.50
Fy	50.000 ksi
Fu	65.000 ksi
E	29000.000 ksi
Thickness	0.750 in
Width	8.820 in
Depth	23.585 in
Transverse Stiffener Section	P0.50x4.10x8.82
Material	A572 Gr.50
Fy	50.000 ksi
Fu	65.000 ksi
E	29000.000 ksi
Full Depth Stiffener	Yes
Thickness	0.500 in
Width	4.098 in
Depth	8.820 in
Column Weld	E70
Type	Double Fillet
Fillet Size	4.000 Sixteenths
Beam Bolts	3/4" Group A-N
Beam Bolts	Group A-N
Diameter, in.	3/4"
Rows	1
Bolts per Row	4
Longitudinal Spacing	3.000 in
Transverse Spacing	3.000 in
Slip Critical	No
Moment Weld	E70
Type	CJP

continued on next page...

M1 J - M5: Connection Properties Report (continued):

Transverse Stiffener Weld	E70
Type	Double Fillet
Fillet Size	6.000 Sixteenths
Doubler Flange Weld	E70
Type	Fillet
Fillet Size	4.000 Sixteenths
Doubler Web Weld	E70
Type	Fillet
Fillet Size	4.000 Sixteenths
Assembly	
Column/Beam Clearance	0.500 in
Plate Vertical Position	2.000 in
Beam Bolts Edge Distance Dimensions	
Beam Bolts/Beam Edge Dist	1.500 in
Beam Bolts Horz Edge Dist	2.000 in
Beam Bolts Vert Edge Dist	1.500 in