PRMU20240407

ENGINEERING ANALYSIS FOR: EAST TOWN CROSSING APARTMENTS PIONEER & SHAW PUYALLUP, WA BUILDING A

DESIGN CRITERIA

BUILDING CODE: 2018 INTERNATIONAL BUILDING CODE (IBC) AS AMENDED BY THE LOCAL JURISDICTION.

VERTICAL LOADS

ROOF LIVE LOAD:	25 PSF (SNOW)
ROOF DEAD LOAD:	25 PSF
RESIDENTIAL FLOOR LIVE LOAD:	40 PSF (REDUCIBLE) : 60 PSF (FOR DECKS)
STAIRWAY LANDING AREAS:	150 PSF (INCLUDING Ip=1.5)
FLOOR DEAD LOAD:	30 PSF (INCLUDES 1 $\frac{1}{2}$" GYP TOPPING)

SNOW DESIGN DATA (ASCE 7-16)

FLAT SNOW LOAD: N/A
SNOW EXPOSURE FACTOR, Ce=1.0,
SNOW IMPORTANCE FACTOR, Is=1.0,
THERMAL FACTOR, Ct=1.1

WIND DESIGN DATA (ASCE 7-16)

BASIC WIND SPEED (ASD) V= 85MPH
ULTIMATE WIND SPEED V= 110MPH
RISK CATEGORY: II EXPOSURE: B
IMPORTANCE FACTOR, Iw= 1.0
TOPOGRAPHIC FACTOR, Kzt= 1.0

SEISMIC DESIGN DATA (ASCE7-16)

SEISMIC RESPONSE SYSTEM: WOOD SHEARWALLS
EQUIVALENT LATERAL FORCE PROCEDURE (ASCE 7-16)
RISK CATEGORY: II SEISMIC IMPORTANCE FACTOR, Ie= 1.0
MAPPED SPECTRAL RESPONSE ACCELERATION: Ss=1.24, S1=0.476
DESIGN SPECTRAL RESPONSE ACCELERATION: Sds=0.831, Sd1=0.476
SITE CLASS: D SEISMIC DESIGN CATEGORY: D
SEISMIC RESPONSE COEFFICIENT: Cs= 0.091
DESIGN BASE SHEAR: 37,932#

SOIL PROPERTIES:

BEARING CAPACITY: 2,000 PSF
LATERAL CAPACITY: 250 PSF/FT

REVISION 1

COMPLETE STRUCTURAL REDESIGN

01

Calculations required to be provided by the Permittee on site for all Inspections

City of Puyallup Building REVIEWED FOR COMPLIANCE
BSnowden
07/30/2025
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CITY OF PUYALLUP STATE OF WASHINGTON

CHON K. PIERUCCIONI STATE OF WASHINGTON 42816 REGISTERED PROFESSIONAL ENGINEER

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EAST TOWN CROSSING
BUILDING 'A'
PIONEER & SHAW PUYALLUP WA

REVISIONS

01	REVIEW 1 2024.08.05

REVISIONS

ENGINEER:	CP
CHECKED BY:	CP
DATE:	2023.03.05
TITLE:	STRUCTURAL ANALYSIS
PROJECT # :	----

2nd Floor Framing			
Member Name	**Results (Max UTIL %)**	**Current Solution**	**Comments**
Floor Joist 15'-2" and Under	Passed (101% M)	1 piece(s) 2 x 12 DF No.2 @ 16" OC	
Short Stair Stringers	Passed (68% R)	1 piece(s) 4 x 12 HF No.2	
Long Short Stair Stringers	Passed (98% ΔL)	1 piece(s) 3 1/2" x 12" 24F-V4 DF Glulam	
7'-1" Landing Joists	Passed (100% R)	1 piece(s) 2 x 12 HF No.2 @ 16" OC	
8'-3" Landing Joists	Passed (88% R)	1 piece(s) 2 x 12 HF No.2 @ 12" OC	
Top Landing Beam	Passed (89% R)	1 piece(s) 5 1/2" x 13 1/2" 24F-V4 DF Glulam	
Grid 1&8 Deck Beam	Passed (63% M)	2 piece(s) 2 x 10 DF No.2	
4'-7" Deck Joist	Passed (30% R)	1 piece(s) 2 x 10 HF No.2 @ 16" OC	
Grid A Deck Beam	Passed (55% M)	2 piece(s) 2 x 10 DF No.2	
5'-7" Deck Joist	Passed (37% R)	1 piece(s) 2 x 10 HF No.2 @ 16" OC	
6' Window Header (Grids 1&8)	Passed (88% M)	1 piece(s) 4 x 8 DF No.2	
Grid 3.1 (D-D.2) Door Header	Passed (95% R)	1 piece(s) 4 x 8 DF No.2	
Grid 3.1 (D.3-D.6) Flush Beam	Passed (98% R)	2 piece(s) 1 3/4" x 11 1/4" 2.0E Microllam® LVL	Squash Blocks Required
Grid 3.6 (3-3.3) Door Header	Passed (73% R)	1 piece(s) 4 x 8 DF No.2	
Grid 3.3 (B.6-B.-8) Flush Beam	Passed (28% R)	1 piece(s) 4 x 12 DF No.2	
Grid 4.1 (B.6-C) Flush Beam	Passed (93% R)	1 piece(s) 4 x 12 DF No.2	
Grid 3.1D.3 Post	Passed (90% f_{cp})	1 piece(s) 4 x 8 DF No.2	Could not find information based on inputs for ~1.

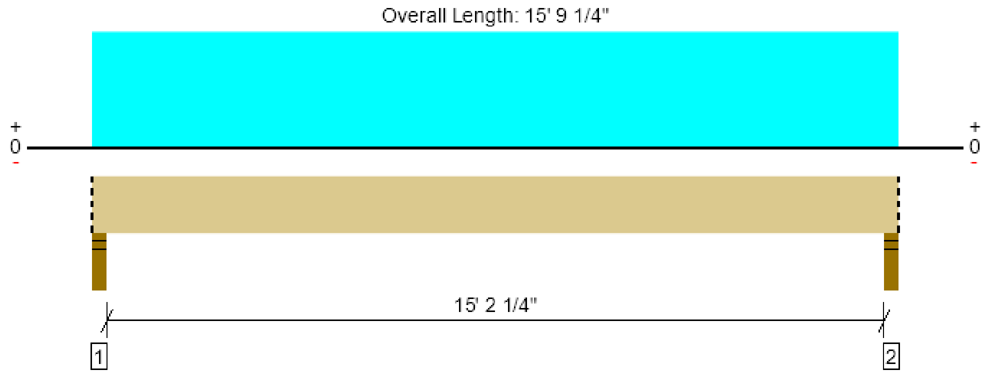
3rd Floor Framing			
Member Name	**Results (Max UTIL %)**	**Current Solution**	**Comments**
Floor Joist 15'-2" and Under	Passed (101% M)	1 piece(s) 2 x 12 DF No.2 @ 16" OC	
7'-1" Landing Joists	Passed (100% R)	1 piece(s) 2 x 12 HF No.2 @ 16" OC	
8'-3" Landing Joists	Passed (88% R)	1 piece(s) 2 x 12 HF No.2 @ 12" OC	
Top Landing Beam	Passed (82% ΔL)	1 piece(s) 5 1/2" x 12" 24F-V4 DF Glulam	
Short Stair Stringers	Passed (68% R)	1 piece(s) 4 x 12 HF No.2	
5'-3" Mid Landing Joists	Passed (75% R)	1 piece(s) 2 x 8 HF No.2 @ 16" OC	
Mid Landing Beam Inner	Passed (79% ΔL)	1 piece(s) 5 1/2" x 12" 24F-V4 DF Glulam	
Mid Landing Beam Outer	Passed (102% ΔL)	1 piece(s) 3 1/2" x 10 1/2" 24F-V4 DF Glulam	
Grid 1&8 Deck Beam	Passed (63% M)	2 piece(s) 2 x 10 DF No.2	
4'-7" Deck Joist	Passed (30% R)	1 piece(s) 2 x 10 HF No.2 @ 16" OC	
6' Window Header (Grids 1&8)	Passed (88% M)	1 piece(s) 4 x 8 DF No.2	
6' Window Header (Grid A)	Passed (70% R)	1 piece(s) 4 x 10 DF No.2	
6' Window Header (Grid B)	Passed (72% R)	1 piece(s) 4 x 10 DF No.2	
Grid A - Deck Roof Beams	Passed (68% M+)	1 piece(s) 3 1/2" x 7 1/2" 24F-V4 DF Glulam	
Grid 3.1 (D-D.2) Door Header	Passed (48% R)	1 piece(s) 4 x 8 DF No.2	
Grid 3.1 (D.3-D.6) Flush Beam	Passed (98% R)	2 piece(s) 1 3/4" x 11 1/4" 2.0E Microllam® LVL	
Grid 3.6 (3-3.3) Door Header	Passed (58% R)	1 piece(s) 4 x 8 DF No.2	

Roof Framing			
Member Name	**Results (Max UTIL %)**	**Current Solution**	**Comments**
Entry Roof Beam	Passed (100% R)	1 piece(s) 3 1/2" x 10 1/2" 24F-V4 DF Glulam	
6' Window Header (Grid 1 & 7)	Passed (60% M)	1 piece(s) 4 x 8 DF No.2	
5' Window Header (Grid H)	Passed (94% M)	1 piece(s) 4 x 8 DF No.2	

ForteWEB Software Operator	Job Notes
Chon Pieruccioni Pieruccioni Engineering (206) 949-7866 cpieru@hotmail.com	

2nd Floor Framing, Floor Joist 15'-2" and Under

1 piece(s) 2 x 12 DF No.2 @ 16" OC



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)	736 @ 2 1/2"	2126 (3.50")	Passed (35%)	--	1.0 D + 1.0 L (All Spans)
Shear (lbs)	621 @ 1' 2 3/4"	2025	Passed (31%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	2750 @ 7' 10 5/8"	2729	Passed (101%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.234 @ 7' 10 5/8"	0.512	Passed (L/787)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.410 @ 7' 10 5/8"	0.768	Passed (L/450)	--	1.0 D + 1.0 L (All Spans)
TJ-Pro™ Rating	N/A	N/A	N/A	--	N/A

Member Length : 15' 9 1/4"
System : Floor
Member Type : Joist
Building Use : Residential
Building Code : IBC 2018
Design Methodology : ASD

- Deflection criteria: LL (L/360) and TL (L/240).
- Allowed moment does not reflect the adjustment for the beam stability factor.
- A 15% increase in the moment capacity has been added to account for repetitive member usage.
- Applicable calculations are based on NDS.
- No composite action between deck and joist was considered in analysis.

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Floor Live	Factored	
1 - Stud wall - HF	3.50"	3.50"	1.50"	315	421	736	Blocking
2 - Stud wall - HF	3.50"	3.50"	1.50"	315	421	736	Blocking

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

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	6" o/c	
Bottom Edge (Lu)	15' 9" o/c	

•Maximum allowable bracing intervals based on applied load.

Vertical Load	Location (Side)	Spacing	Dead (0.90)	Floor Live (1.00)	Comments
1 - Uniform (PSF)	0 to 15' 9 1/4"	16"	30.0	40.0	Default Load

Weyerhaeuser Notes

ForteWEB Software Operator	Job Notes
Chon Pieruccioni Pieruccioni Engineering (206) 949-7866 cpieru@hotmail.com	

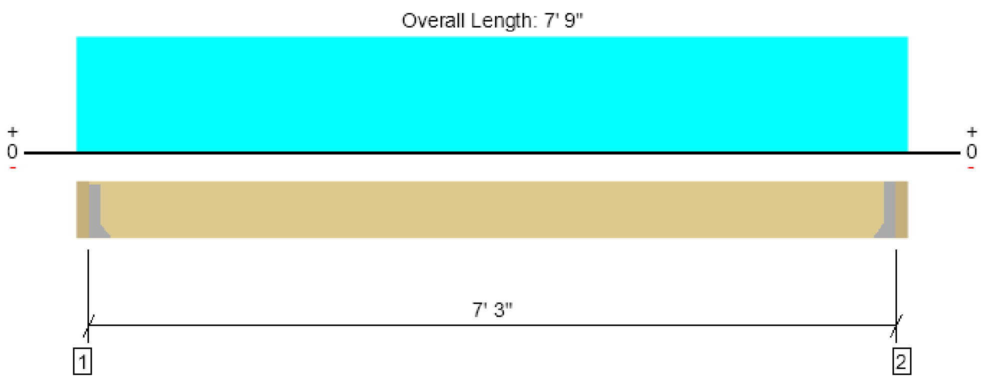
Weyerhaeuser

MEMBER REPORT

PASSED

2nd Floor Framing, Short Stair Stringers

1 piece(s) 4 x 12 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)	1450 @ 3"	2126 (1.50")	Passed (68%)	--	1.0 D + 1.0 L (All Spans)
Shear (lbs)	1075 @ 1' 2 1/4"	3938	Passed (27%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	2628 @ 3' 10 1/2"	5752	Passed (46%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.035 @ 3' 10 1/2"	0.181	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.046 @ 3' 10 1/2"	0.363	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)

Member Length : 7' 3"
System : Floor
Member Type : Flush Beam
Building Use : Residential
Building Code : IBC 2018
Design Methodology : ASD

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

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Floor Live	Factored	
1 - Hanger on 11 1/4" GLB beam	3.00"	Hanger[1]	1.50"	385	1163	1547	See note [1]
2 - Hanger on 11 1/4" GLB beam	3.00"	Hanger[1]	1.50"	385	1163	1547	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' 3" o/c	
Bottom Edge (Lu)	7' 3" o/c	

•Maximum allowable bracing intervals based on applied load.

Connector: Simpson Strong-Tie

Support	Model	Seat Length	Top Fasteners	Face Fasteners	Member Fasteners	Accessories
1 - Face Mount Hanger	LUS410	2.00"	N/A	8-10d	6-10d	
2 - Face Mount Hanger	LUS410	2.00"	N/A	8-10d	6-10d	

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

Vertical Loads	Location (Side)	Tributary Width	Dead (0.90)	Floor Live (1.00)	Comments
0 - Self Weight (PLF)	3" to 7' 6"	N/A	10.0	--	
1 - Uniform (PSF)	0 to 7' 9" (Front)	2'	45.0	150.0	Default Load

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

Weyerhaeuser Notes

ForteWEB Software Operator	Job Notes
Chon Pieruccioni Pieruccioni Engineering (206) 949-7866 cpieru@hotmail.com	

Weyerhaeuser

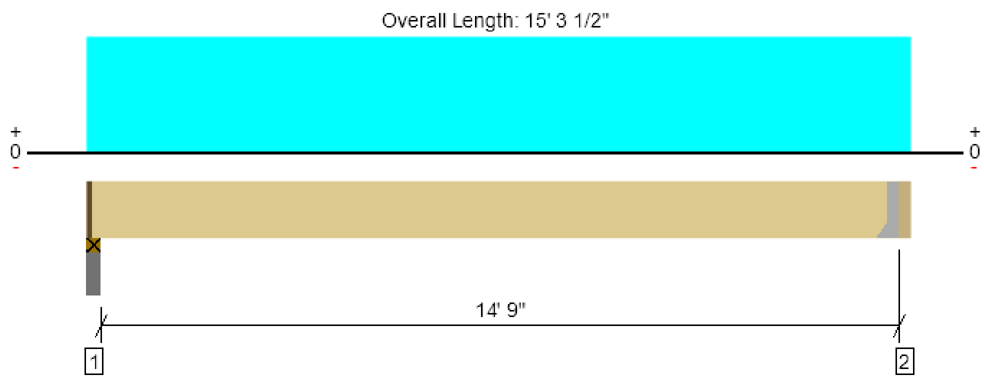
6/2/2025 6:46:29 PM UTC

ForteWEB v3.9, Engine: V8.4.3.94, Data: V8.1.7.3

File Name: East Town Crossing Building A

2nd Floor Framing, Long Short Stair Stringers

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



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Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	3002 @ 2"	3189 (2.25")	Passed (94%)	--	1.0 D + 1.0 L (All Spans)
Shear (lbs)	2576 @ 14' 1/2"	7420	Passed (35%)	1.00	1.0 D + 1.0 L (All Spans)
Pos Moment (Ft-lbs)	11069 @ 7' 7 1/4"	16800	Passed (66%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.364 @ 7' 7 1/4"	0.372	Passed (L/490)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.486 @ 7' 7 1/4"	0.744	Passed (L/367)	--	1.0 D + 1.0 L (All Spans)

Member Length : 14' 11 1/4"
System : Floor
Member Type : Flush Beam
Building Use : Residential
Building Code : IBC 2018
Design Methodology : ASD

- Deflection criteria: LL (L/480) and TL (L/240).
- Allowed moment does not reflect the adjustment for the beam stability factor.
- Volume factor of 1.00 was calculated for positive bending using length L = 14' 10 1/2".
- The effects of positive or negative camber have not been accounted for when calculating deflection.
- The specified glulam is assumed to have its strong laminations at the bottom of the beam. Install with proper side up as indicated by the manufacturer.
- Applicable calculations are based on NDS.

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Floor Live	Factored	
1 - Plate on concrete - HF	3.50"	2.25"	2.12"	761	2281	3042	1 1/4" Rim Board
2 - Hanger on 12" GLB beam	3.00"	Hanger[1]	1.50"	768	2306	3074	See note [1]

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

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	14' 11" o/c	
Bottom Edge (Lu)	14' 11" o/c	

- Maximum allowable bracing intervals based on applied load.

Connector: Simpson Strong-Tie

Support	Model	Seat Length	Top Fasteners	Face Fasteners	Member Fasteners	Accessories
2 - Face Mount Hanger	HHUS410	3.00"	N/A	30-10d	10-10d	

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

Vertical Loads	Location (Side)	Tributary Width	Dead (0.90)	Floor Live (1.00)	Comments
0 - Self Weight (PLF)	1 1/4" to 15' 1/2"	N/A	10.2	--	
1 - Uniform (PSF)	0 to 15' 3 1/2" (Front)	2'	45.0	150.0	Default Load

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

Weyerhaeuser Notes

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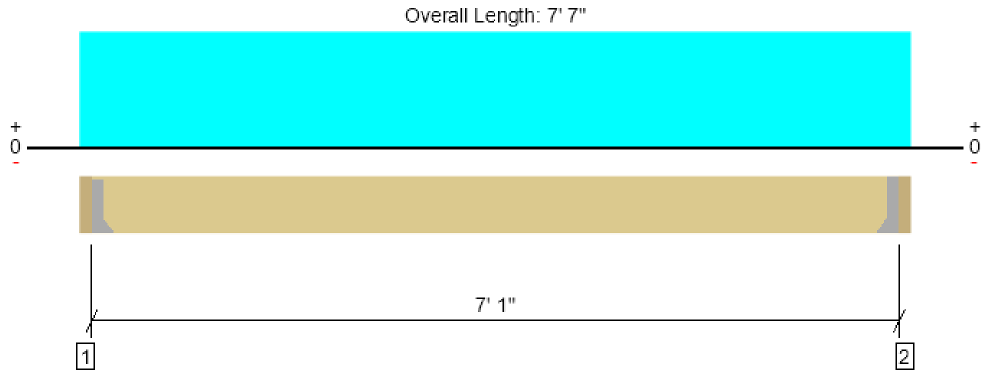
The product application, input design loads, dimensions and support information have been provided by ForteWEB Software Operator

2nd Floor Framing, 7'-1" Landing Joists

1 piece(s) 2 x 12 HF No.2 @ 16" OC



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Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	921 @ 3"	921 (1.52")	Passed (100%)	--	1.0 D + 1.0 L (All Spans)
Shear (lbs)	677 @ 1' 2 1/4"	1688	Passed (40%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	1631 @ 3' 9 1/2"	2577	Passed (63%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.049 @ 3' 9 1/2"	0.236	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.064 @ 3' 9 1/2"	0.354	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)
TJ-Pro™ Rating	N/A	N/A	N/A	--	N/A

Member Length : 7' 1"
System : Floor
Member Type : Joist
Building Use : Residential
Building Code : IBC 2018
Design Methodology : ASD

- Deflection criteria: LL (L/360) and TL (L/240).
- Allowed moment does not reflect the adjustment for the beam stability factor.
- A 15% increase in the moment capacity has been added to account for repetitive member usage.
- Applicable calculations are based on NDS.
- No composite action between deck and joist was considered in analysis.

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Floor Live	Factored	
1 - Hanger on 11 1/4" LSL beam	3.00"	Hanger[1]	1.52"	228	758	986	See note [1]
2 - Hanger on 11 1/4" LSL beam	3.00"	Hanger[1]	1.52"	228	758	986	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)	6' 10" o/c	
Bottom Edge (Lu)	7' 1" o/c	

•Maximum allowable bracing intervals based on applied load.

Connector: Simpson Strong-Tie

Support	Model	Seat Length	Top Fasteners	Face Fasteners	Member Fasteners	Accessories
1 - Face Mount Hanger	LUS28	1.75"	N/A	6-10dx1.5	4-10d	
2 - Face Mount Hanger	LUS28	1.75"	N/A	6-10dx1.5	4-10d	

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

Vertical Load	Location (Side)	Spacing	Dead (0.90)	Floor Live (1.00)	Comments
1 - Uniform (PSF)	0 to 7' 7"	16"	45.0	150.0	Default Load

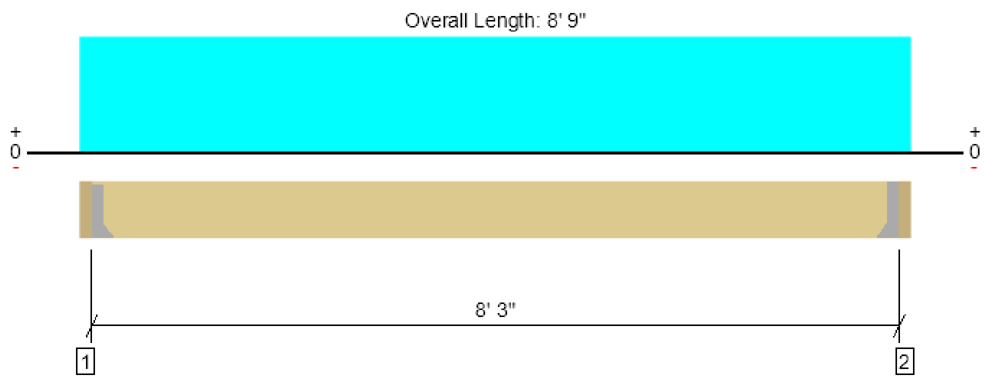
Weyerhaeuser Notes

ForteWEB Software Operator	Job Notes
Chon Pieruccioni Pieruccioni Engineering (206) 949-7866 cpieru@hotmail.com	

PASSED

2nd Floor Framing, 8'-3" Landing Joists

1 piece(s) 2 x 12 HF No.2 @ 12" OC



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Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	804 @ 3"	911 (1.50")	Passed (88%)	--	1.0 D + 1.0 L (All Spans)
Shear (lbs)	622 @ 1' 2 1/4"	1688	Passed (37%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	1659 @ 4' 4 1/2"	2577	Passed (64%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.068 @ 4' 4 1/2"	0.275	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.088 @ 4' 4 1/2"	0.412	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)
TJ-Pro™ Rating	N/A	N/A	N/A	--	N/A

Member Length : 8' 3"
System : Floor
Member Type : Joist
Building Use : Residential
Building Code : IBC 2018
Design Methodology : ASD

- Deflection criteria: LL (L/360) and TL (L/240).
- Allowed moment does not reflect the adjustment for the beam stability factor.
- A 15% increase in the moment capacity has been added to account for repetitive member usage.
- Applicable calculations are based on NDS.
- No composite action between deck and joist was considered in analysis.

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Floor Live	Factored	
1 - Hanger on 11 1/4" LSL beam	3.00"	Hanger[1]	1.50"	197	656	853	See note [1]
2 - Hanger on 11 1/4" LSL beam	3.00"	Hanger[1]	1.50"	197	656	853	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)	6' 8" o/c	
Bottom Edge (Lu)	8' 3" o/c	

•Maximum allowable bracing intervals based on applied load.

Connector: Simpson Strong-Tie

Support	Model	Seat Length	Top Fasteners	Face Fasteners	Member Fasteners	Accessories
1 - Face Mount Hanger	LUS28	1.75"	N/A	6-10dx1.5	3-10d	
2 - Face Mount Hanger	LUS28	1.75"	N/A	6-10dx1.5	3-10d	

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

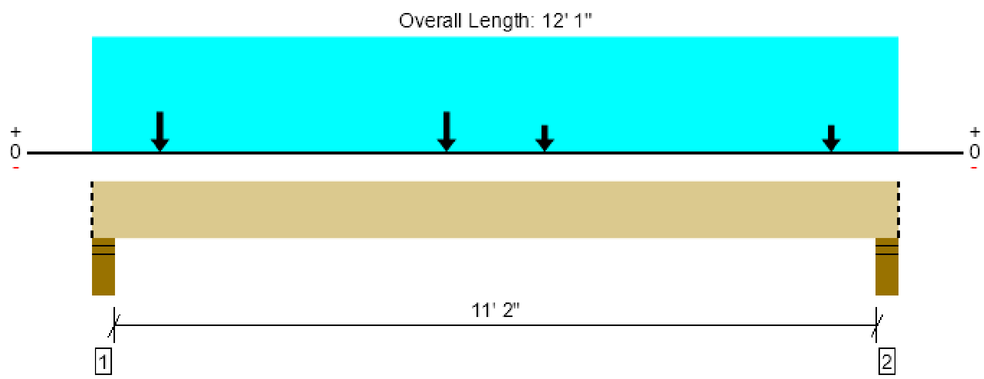
Vertical Load	Location (Side)	Spacing	Dead (0.90)	Floor Live (1.00)	Comments
1 - Uniform (PSF)	0 to 8' 9"	12"	45.0	150.0	Default Load

Weyerhaeuser Notes

ForteWEB Software Operator	Job Notes
Chon Pieruccioni Pieruccioni Engineering (206) 949-7866 cpieru@hotmail.com	

2nd Floor Framing, Top Landing Beam

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



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Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	10897 @ 4"	12251 (5.50")	Passed (89%)	--	1.0 D + 1.0 L (All Spans)
Shear (lbs)	8008 @ 1' 7"	13118	Passed (61%)	1.00	1.0 D + 1.0 L (All Spans)
Pos Moment (Ft-lbs)	28247 @ 5' 3 3/4"	33413	Passed (85%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.235 @ 5' 11 15/16"	0.285	Passed (L/582)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.313 @ 5' 11 15/16"	0.571	Passed (L/438)	--	1.0 D + 1.0 L (All Spans)

Member Length : 12' 1"
System : Floor
Member Type : Flush Beam
Building Use : Residential
Building Code : IBC 2018
Design Methodology : ASD

- Deflection criteria: LL (L/480) and TL (L/240).
- Allowed moment does not reflect the adjustment for the beam stability factor.
- Volume factor of 1.00 was calculated for positive bending using length L = 11' 5".
- 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	Factored	Accessories
1 - Stud wall - HF	5.50"	5.50"	4.89"	2700	8196	10897	Blocking
2 - Stud wall - HF	5.50"	5.50"	4.20"	2316	7049	9365	Blocking

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

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	12' 1" o/c	
Bottom Edge (Lu)	12' 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)	0 to 12' 1"	N/A	18.0	--	
1 - Uniform (PSF)	0 to 12' 1" (Front)	4' 7"	45.0	150.0	Default Load
2 - Point (lb)	6' 9 3/8" (Front)	N/A	385	1163	Linked from: Short Stair Stringers, Support 1
3 - Point (lb)	11' 7/8" (Front)	N/A	385	1163	Linked from: Short Stair Stringers, Support 1
4 - Point (lb)	1' 1/4" (Front)	N/A	768	2306	Linked from: Long Short Stair Stringers, Support 2
5 - Point (lb)	5' 3 3/4" (Front)	N/A	768	2306	Linked from: Long Short Stair Stringers, Support 2

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

ForteWEB Software Operator	Job Notes
Chon Pieruccioni Pieruccioni Engineering (206) 949-7866 cpieru@hotmail.com	

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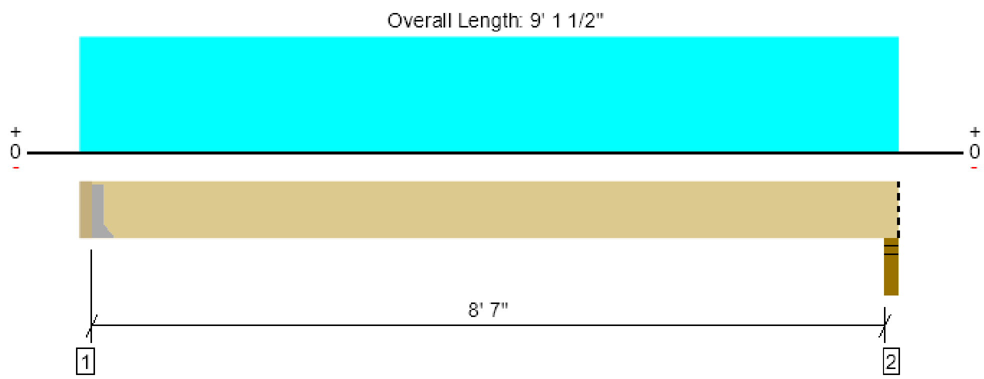
The product application, input design loads, dimensions and support information have been provided by ForteWEB Software Operator

2nd Floor Framing, Grid 1&8 Deck Beam

2 piece(s) 2 x 10 DF 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)	1027 @ 3"	2813 (1.50")	Passed (37%)	--	1.0 D + 1.0 L (All Spans)
Shear (lbs)	845 @ 1' 1/4"	3330	Passed (25%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	2235 @ 4' 7 1/4"	3529	Passed (63%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.062 @ 4' 7 1/4"	0.218	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.096 @ 4' 7 1/4"	0.435	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)

Member Length : 8' 10 1/2"
System : Floor
Member Type : Flush Beam
Building Use : Residential
Building Code : IBC 2018
Design Methodology : ASD

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

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Floor Live	Factored	
1 - Hanger on 9 1/4" HF beam	3.00"	Hanger[1]	1.50"	382	702	1084	See note [1]
2 - Stud wall - HF	3.50"	3.50"	1.50"	376	689	1066	Blocking

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

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	8' 11" o/c	
Bottom Edge (Lu)	8' 11" o/c	

- Maximum allowable bracing intervals based on applied load.

Connector: Simpson Strong-Tie

Support	Model	Seat Length	Top Fasteners	Face Fasteners	Member Fasteners	Accessories
1 - Face Mount Hanger	LUS28-2	2.00"	N/A	6-16d	3-16d	

- 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)	Comments
0 - Self Weight (PLF)	3" to 9' 1 1/2"	N/A	7.0	--	
1 - Uniform (PSF)	0 to 9' 1 1/2" (Front)	2' 6 1/2"	30.0	60.0	Default Load

- 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
Chon Pieruccioni Pieruccioni Engineering (206) 949-7866 cpieru@hotmail.com	

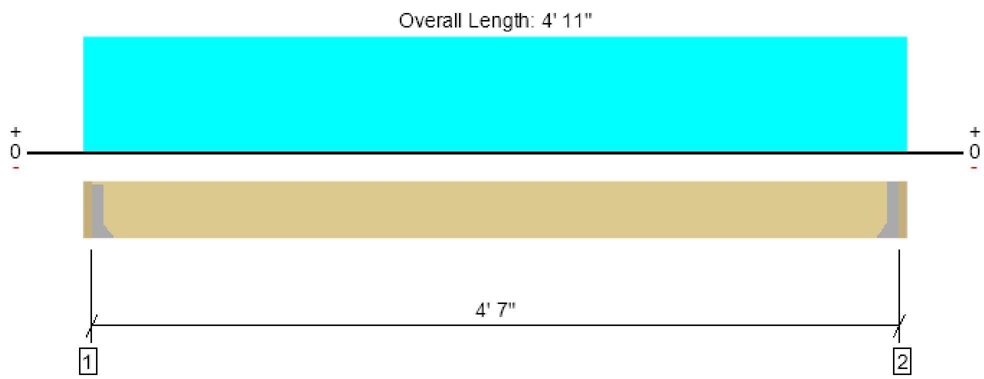
FORTEWEB

MEMBER REPORT

PASSED

3rd Floor Framing, 4'-7" Deck Joist

1 piece(s) 2 x 10 HF No.2 @ 16" OC



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)	275 @ 2"	911 (1.50")	Passed (30%)	--	1.0 D + 1.0 L (All Spans)
Shear (lbs)	183 @ 11 1/4"	1388	Passed (13%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	315 @ 2' 5 1/2"	1917	Passed (16%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.006 @ 2' 5 1/2"	0.153	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.009 @ 2' 5 1/2"	0.229	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)
TJ-Pro™ Rating	N/A	N/A	N/A	--	N/A

Member Length : 4' 7"
System : Floor
Member Type : Joist
Building Use : Residential
Building Code : IBC 2018
Design Methodology : ASD

- Deflection criteria: LL (L/360) and TL (L/240).
- Allowed moment does not reflect the adjustment for the beam stability factor.
- A 15% increase in the moment capacity has been added to account for repetitive member usage.
- Applicable calculations are based on NDS.
- No composite action between deck and joist was considered in analysis.

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Floor Live	Factored	
1 - Hanger on 9 1/4" HF beam	2.00"	Hanger[1]	1.50"	98	197	295	See note [1]
2 - Hanger on 9 1/4" HF beam	2.00"	Hanger[1]	1.50"	98	197	295	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)	4' 7" o/c	
Bottom Edge (Lu)	4' 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 - Face Mount Hanger	LU28	1.50"	N/A	8-10dx1.5	6-10dx1.5	
2 - Face Mount Hanger	LU28	1.50"	N/A	8-10dx1.5	6-10dx1.5	

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

Vertical Load	Location (Side)	Spacing	Dead (0.90)	Floor Live (1.00)	Comments
1 - Uniform (PSF)	0 to 4' 11"	16"	30.0	60.0	Default Load

Weyerhaeuser Notes

ForteWEB Software Operator	Job Notes
Chon Pieruccioni Pieruccioni Engineering (206) 949-7866 cpieru@hotmail.com	

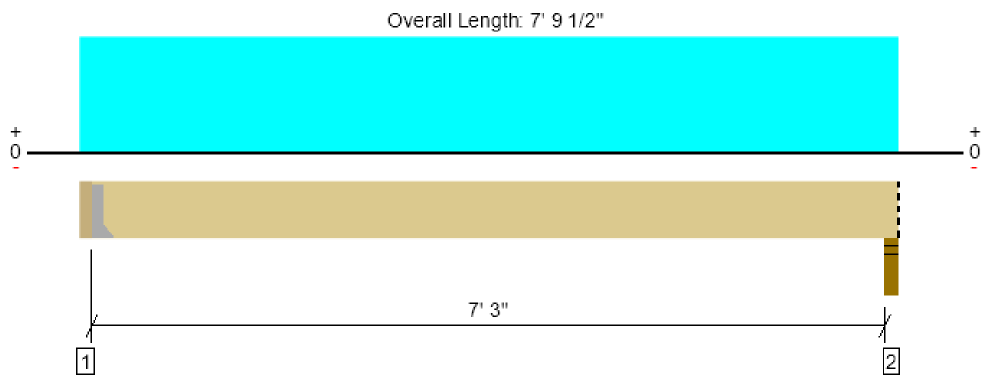
Weyerhaeuser

MEMBER REPORT

PASSED

2nd Floor Framing, Grid A Deck Beam

2 piece(s) 2 x 10 DF 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)	1049 @ 3"	2813 (1.50")	Passed (37%)	--	1.0 D + 1.0 L (All Spans)
Shear (lbs)	830 @ 1' 1/4"	3330	Passed (25%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	1934 @ 3' 11 1/4"	3529	Passed (55%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.039 @ 3' 11 1/4"	0.184	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.060 @ 3' 11 1/4"	0.369	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)

Member Length : 7' 6 1/2"
System : Floor
Member Type : Flush Beam
Building Use : Residential
Building Code : IBC 2018
Design Methodology : ASD

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

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Floor Live	Factored	
1 - Hanger on 9 1/4" HF beam	3.00"	Hanger[1]	1.50"	390	728	1119	See note [1]
2 - Stud wall - HF	3.50"	3.50"	1.50"	384	713	1097	Blocking

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

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	7' 7" o/c	
Bottom Edge (Lu)	7' 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 - Face Mount Hanger	LUS28-2	2.00"	N/A	6-16d	3-16d	

- 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)	Comments
0 - Self Weight (PLF)	3" to 7' 9 1/2"	N/A	7.0	--	
1 - Uniform (PSF)	0 to 7' 9 1/2" (Front)	3' 1"	30.0	60.0	Default Load

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

Weyerhaeuser Notes

ForteWEB Software Operator	Job Notes
Chon Pieruccioni Pieruccioni Engineering (206) 949-7866 cpieru@hotmail.com	

Weyerhaeuser

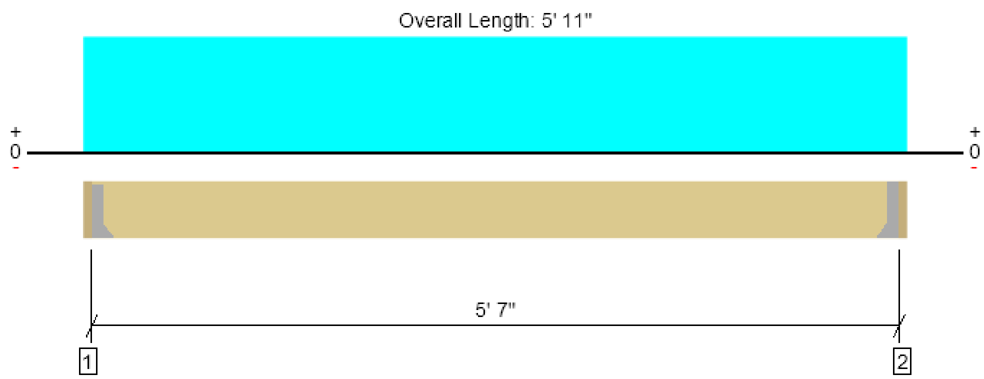
FORTEWEB

MEMBER REPORT

PASSED

2nd Floor Framing, 5'-7" Deck Joist

1 piece(s) 2 x 10 HF No.2 @ 16" OC



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)	335 @ 2"	911 (1.50")	Passed (37%)	--	1.0 D + 1.0 L (All Spans)
Shear (lbs)	243 @ 11 1/4"	1388	Passed (17%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	468 @ 2' 11 1/2"	1917	Passed (24%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.014 @ 2' 11 1/2"	0.186	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.020 @ 2' 11 1/2"	0.279	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)
TJ-Pro™ Rating	N/A	N/A	N/A	--	N/A

Member Length : 5' 7"
System : Floor
Member Type : Joist
Building Use : Residential
Building Code : IBC 2018
Design Methodology : ASD

- Deflection criteria: LL (L/360) and TL (L/240).
- Allowed moment does not reflect the adjustment for the beam stability factor.
- A 15% increase in the moment capacity has been added to account for repetitive member usage.
- Applicable calculations are based on NDS.
- No composite action between deck and joist was considered in analysis.

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Floor Live	Factored	
1 - Hanger on 9 1/4" HF beam	2.00"	Hanger[1]	1.50"	118	237	355	See note [1]
2 - Hanger on 9 1/4" HF beam	2.00"	Hanger[1]	1.50"	118	237	355	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)	5' 7" o/c	
Bottom Edge (Lu)	5' 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 - Face Mount Hanger	LU28	1.50"	N/A	8-10dx1.5	6-10dx1.5	
2 - Face Mount Hanger	LU28	1.50"	N/A	8-10dx1.5	6-10dx1.5	

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

Vertical Load	Location (Side)	Spacing	Dead (0.90)	Floor Live (1.00)	Comments
1 - Uniform (PSF)	0 to 5' 11"	16"	30.0	60.0	Default Load

Weyerhaeuser Notes

ForteWEB Software Operator	Job Notes
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Weyerhaeuser

6/2/2025 6:46:29 PM UTC

ForteWEB v3.9, Engine: V8.4.3.94, Data: V8.1.7.3

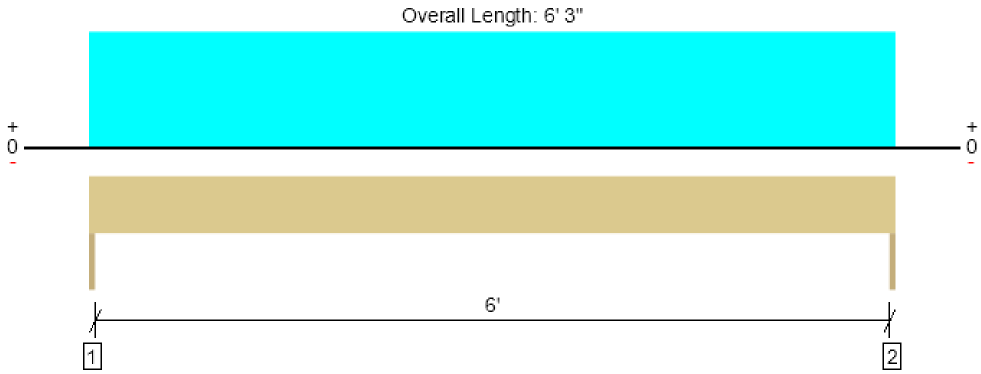
File Name: East Town Crossing Building A

MEMBER REPORT

PASSED

3rd Floor Framing, 6' Window Header (Grids 1&8)

1 piece(s) 4 x 8 DF 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)	1688 @ 0	3281 (1.50")	Passed (51%)	--	1.0 D + 1.0 L (All Spans)
Shear (lbs)	1294 @ 8 3/4"	3045	Passed (43%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	2638 @ 3' 1 1/2"	2989	Passed (88%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.047 @ 3' 1 1/2"	0.208	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.104 @ 3' 1 1/2"	0.313	Passed (L/719)	--	1.0 D + 1.0 L (All Spans)

Member Length : 6' 3"
System : Wall
Member Type : Header
Building Use : Residential
Building Code : IBC 2018
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	Factored	
1 - Trimmer - HF	1.50"	1.50"	1.50"	928	760	1688	None
2 - Trimmer - HF	1.50"	1.50"	1.50"	928	760	1688	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)	Comments
0 - Self Weight (PLF)	0 to 6' 3"	N/A	6.4	--	
1 - Uniform (PSF)	0 to 6' 3"	6' 1"	30.0	40.0	Floor
2 - Uniform (PLF)	0 to 6' 3"	N/A	108.0	--	Wall

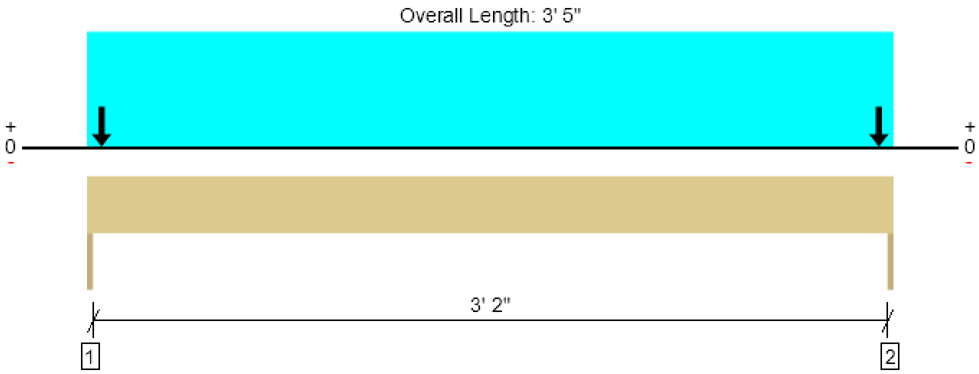
Weyerhaeuser Notes

ForteWEB Software Operator	Job Notes
Chon Pieruccioni Pieruccioni Engineering (206) 949-7866 cpieru@hotmail.com	

MEMBER REPORT **PASSED**

2nd Floor Framing, Grid 3.1 (D-D.2) Door Header

1 piece(s) 4 x 8 DF 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)	3131 @ 0	3281 (1.50")	Passed (95%)	--	1.0 D + 1.0 L (All Spans)
Shear (lbs)	897 @ 8 3/4"	3045	Passed (29%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	1337 @ 1' 8 1/2"	2989	Passed (45%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.009 @ 1' 8 1/2"	0.114	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.016 @ 1' 8 1/2"	0.171	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)

Member Length : 3' 5"
System : Wall
Member Type : Header
Building Use : Residential
Building Code : IBC 2018
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	Factored	
1 - Trimmer - HF	1.50"	1.50"	1.50"	1354	1776	3131	None
2 - Trimmer - HF	1.50"	1.50"	1.50"	1354	1776	3131	None

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	3' 5" o/c	
Bottom Edge (Lu)	3' 5" o/c	

•Maximum allowable bracing intervals based on applied load.

Vertical Loads	Location	Tributary Width	Dead (0.90)	Floor Live (1.00)	Comments
0 - Self Weight (PLF)	0 to 3' 5"	N/A	6.4	--	
1 - Uniform (PSF)	0 to 3' 5"	13'	30.0	40.0	Default Load
2 - Point (lb)	3/4"	N/A	677	888	Linked from: Grid 3.1 (D-D.2) Door Header, Support 1
3 - Point (lb)	3' 4 1/4"	N/A	677	888	Linked from: Grid 3.1 (D-D.2) Door Header, Support 2

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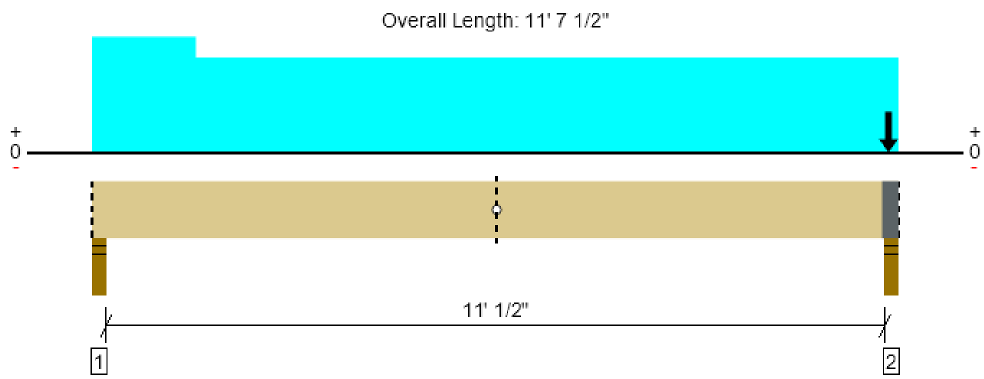
The product application, input design loads, dimensions and support information have been provided by ForteWEB Software Operator

ForteWEB Software Operator	Job Notes
Chon Pieruccioni Pieruccioni Engineering (206) 949-7866 cpieru@hotmail.com	

2nd Floor Framing, Grid 3.1 (D.3-D.6) Flush Beam

2 piece(s) 1 3/4" x 11 1/4" 2.0E Microllam® LVL



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)	4872 @ 2"	4961 (3.50")	Passed (98%)	--	1.0 D + 1.0 L (All Spans)
Shear (lbs)	3682 @ 1' 2 3/4"	7481	Passed (49%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	12764 @ 5' 9 9/16"	16137	Passed (79%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.220 @ 5' 9 11/16"	0.376	Passed (L/616)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.390 @ 5' 9 11/16"	0.565	Passed (L/347)	--	1.0 D + 1.0 L (All Spans)

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

Member Length : 11' 7 1/2"
System : Floor
Member Type : Flush Beam
Building Use : Residential
Building Code : IBC 2018
Design Methodology : ASD

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Floor Live	Factored	
1 - Stud wall - HF	3.50"	3.50"	3.44"	2126	2746	4872	Blocking
2 - Stud wall - HF	3.50"	3.50"	3.27"	4054	5228	9282	Blocking, Squash Blocks

- Squash Blocks must match bearing length and are assumed to carry all loads applied directly above them, 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)	9' 2" o/c	
Bottom Edge (Lu)	11' 8" o/c	

•Maximum allowable bracing intervals based on applied load.

Vertical Loads	Location (Side)	Tributary Width	Dead (0.90)	Floor Live (1.00)	Comments
0 - Self Weight (PLF)	0 to 11' 7 1/2"	N/A	11.5	--	
1 - Uniform (PSF)	0 to 1' 6" (Front)	13' 8"	30.0	40.0	Default Load
2 - Uniform (PSF)	1' 6" to 11' 7 1/2" (Front)	11' 2 1/2"	30.0	40.0	Default Load
3 - Point (lb)	11' 5 3/4" (Top)	N/A	2027	2614	Linked from: Grid 3.1 (D.3-D.6) Flush Beam, Support 2

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

Holes (Size)	Direction	Diameter	Vertical Offset	Location	Shear (lbs)			Moment (Ft-lbs)			Comments
					Actual	Allowed	Result	Actual	Allowed	Result	
1 - Circular (Per Lit.)	Horz	2.00"	5 5/8"	5' 10"	--	--	Passed	--	--	Passed	

- Hole locations are measured from the outside face of left support (or left cantilever end) to the centerline of the hole.
- Vertical Offset is measured from the top of the member to the centerline of the hole.

ForteWEB Software Operator	Job Notes
Chon Pieruccioni Pieruccioni Engineering (206) 949-7866 cpieru@hotmail.com	

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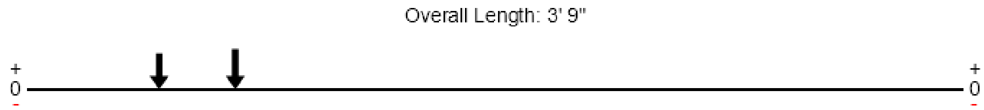
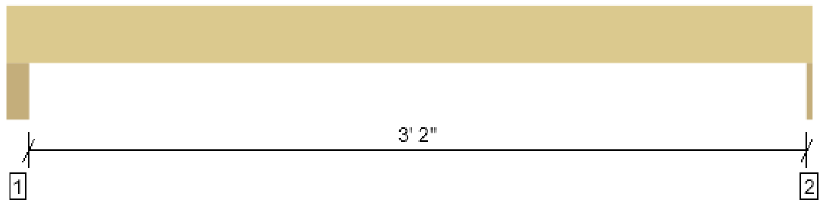
The product application, input design loads, dimensions and support information have been provided by ForteWEB Software Operator

2nd Floor Framing, Grid 3.6 (3-3.3) Door Header

1 piece(s) 4 x 8 DF 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)	8819 @ 4"	12031 (5.50")	Passed (73%)	--	1.0 D + 1.0 L (All Spans)
Shear (lbs)	1522 @ 1' 3/4"	3045	Passed (50%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	1469 @ 8"	2989	Passed (49%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.007 @ 1' 9 3/4"	0.114	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.012 @ 1' 9 13/16"	0.171	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)

Member Length : 3' 9"
System : Wall
Member Type : Header
Building Use : Residential
Building Code : IBC 2018
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	Factored	
1 - Trimmer - HF	5.50"	5.50"	4.03"	3863	4956	8819	None
2 - Trimmer - HF	1.50"	1.50"	1.50"	218	268	486	None

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	3' 9" o/c	
Bottom Edge (Lu)	3' 9" o/c	

•Maximum allowable bracing intervals based on applied load.

Vertical Loads	Location	Tributary Width	Dead (0.90)	Floor Live (1.00)	Comments
0 - Self Weight (PLF)	0 to 3' 9"	N/A	6.4	--	
1 - Point (lb)	8"	N/A	2126	2746	Linked from: Grid 3.1 (D.3-D.6) Flush Beam, Support 1
2 - Point (lb)	3 3/4"	N/A	1931	2478	Linked from: Grid 3.6 (3-3.3) Door Header, Support 1

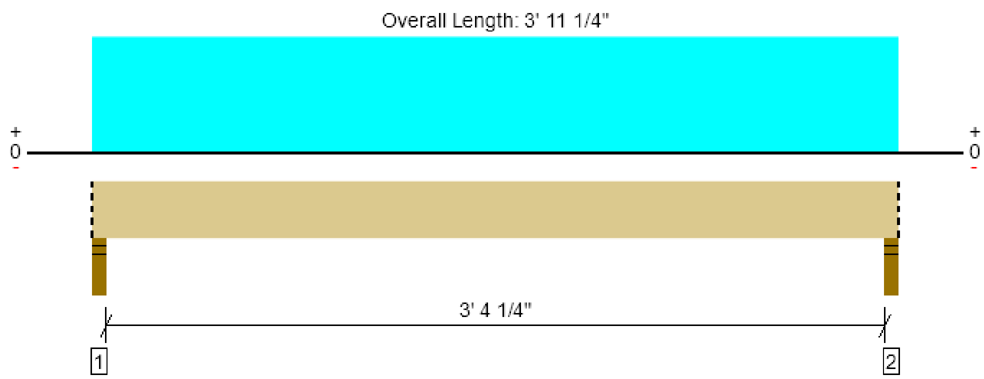
Weyerhaeuser Notes

MEMBER REPORT

PASSED

2nd Floor Framing, Grid 3.3 (B.6-B.-8) Flush Beam

1 piece(s) 4 x 12 DF 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)	1409 @ 2"	4961 (3.50")	Passed (28%)	--	1.0 D + 1.0 L (All Spans)
Shear (lbs)	529 @ 1' 2 3/4"	4725	Passed (11%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	1162 @ 1' 11 5/8"	6091	Passed (19%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.002 @ 1' 11 5/8"	0.120	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.004 @ 1' 11 5/8"	0.180	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)

Member Length : 3' 11 1/4"
System : Floor
Member Type : Flush Beam
Building Use : Residential
Building Code : IBC 2018
Design Methodology : ASD

- Deflection criteria: LL (L/360) and TL (L/240).
- Allowed moment does not reflect the adjustment for the beam stability factor.
- Software only analyzes holes in TJI® Joists, Microllam® LVL, Parallam® PSL and TimberStrand® LSL.
- Applicable calculations are based on NDS.

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Floor Live	Factored	
1 - Stud wall - HF	3.50"	3.50"	1.50"	615	794	1409	Blocking
2 - Stud wall - HF	3.50"	3.50"	1.50"	615	794	1409	Blocking

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

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

•Maximum allowable bracing intervals based on applied load.

Vertical Loads	Location (Side)	Tributary Width	Dead (0.90)	Floor Live (1.00)	Comments
0 - Self Weight (PLF)	0 to 3' 11 1/4"	N/A	10.0	--	
1 - Uniform (PSF)	0 to 3' 11 1/4" (Front)	10' 1"	30.0	40.0	Default Load

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

Weyerhaeuser Notes

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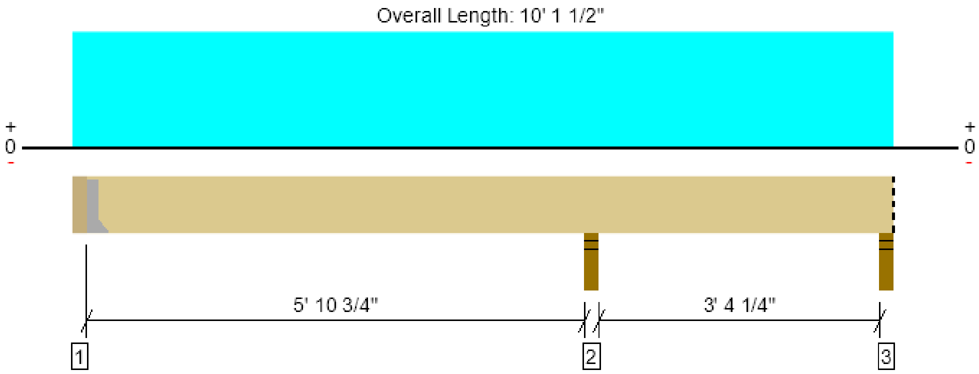
2nd Floor Framing, Grid 4.1 (B.6-C) Flush Beam

1 piece(s) 4 x 12 DF 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)	4620 @ 6' 4"	4961 (3.50")	Passed (93%)	--	1.0 D + 1.0 L (All Spans)
Shear (lbs)	1823 @ 5' 3"	4725	Passed (39%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	-2518 @ 6' 4"	6091	Passed (41%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.012 @ 3' 1 1/16"	0.201	Passed (L/999+)	--	1.0 D + 1.0 L (Alt Spans)
Total Load Defl. (in)	0.020 @ 3' 11/16"	0.302	Passed (L/999+)	--	1.0 D + 1.0 L (Alt Spans)

Member Length : 9' 10"
System : Floor
Member Type : Flush Beam
Building Use : Residential
Building Code : IBC 2018
Design Methodology : ASD

- Deflection criteria: LL (L/360) and TL (L/240).
- Allowed moment does not reflect the adjustment for the beam stability factor.
- Software only analyzes holes in TJI® Joists, Microllam® LVL, Parallam® PSL and TimberStrand® LSL.
- Applicable calculations are based on NDS.

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Floor Live	Factored	
1 - Hanger on 11 1/4" HF beam	3.50"	Hanger[1]	1.50"	865	1162	2027	See note [1]
2 - Stud wall - HF	3.50"	3.50"	3.26"	2016	2604	4620	None
3 - Stud wall - HF	3.50"	3.50"	1.50"	324	740/-322	1064	Blocking

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

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	9' 10" o/c	
Bottom Edge (Lu)	9' 10" o/c	

•Maximum allowable bracing intervals based on applied load.

Connector: Simpson Strong-Tie

Support	Model	Seat Length	Top Fasteners	Face Fasteners	Member Fasteners	Accessories
1 - Face Mount Hanger	LUS414	2.00"	N/A	10-16d	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)	Comments
0 - Self Weight (PLF)	3 1/2" to 10' 1 1/2"	N/A	10.0	--	
1 - Uniform (PSF)	0 to 10' 1 1/2" (Front)	10' 2 3/4"	30.0	40.0	Default Load

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

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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
Chon Pieruccioni Pieruccioni Engineering (206) 949-7866 cpieru@hotmail.com	

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6/2/2025 6:46:29 PM UTC
ForteWEB v3.9, Engine: V8.4.3.94, Data: V8.1.7.3
File Name: East Town Crossing Building A

MEMBER REPORT

PASSED

2nd Floor Framing, Grid 3.1D.3 Post

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

Post Height: 9'

Drawing is Conceptual

Design Results	Actual	Allowed	Result	LDF	Load: Combination
Slenderness	31	50	Passed (62%)	--	--
Compression (lbs)	9282	11601	Passed (80%)	1.00	1.0 D + 1.0 L
Base Bearing (lbs)	9282	10277	Passed (90%)	--	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	Beam	Hem Fir

Member Type : Free Standing Post
Building Code : IBC 2018
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)	4054	5228	Linked from: Grid 3.1 (D.3-D.6) Flush Beam, Support 2

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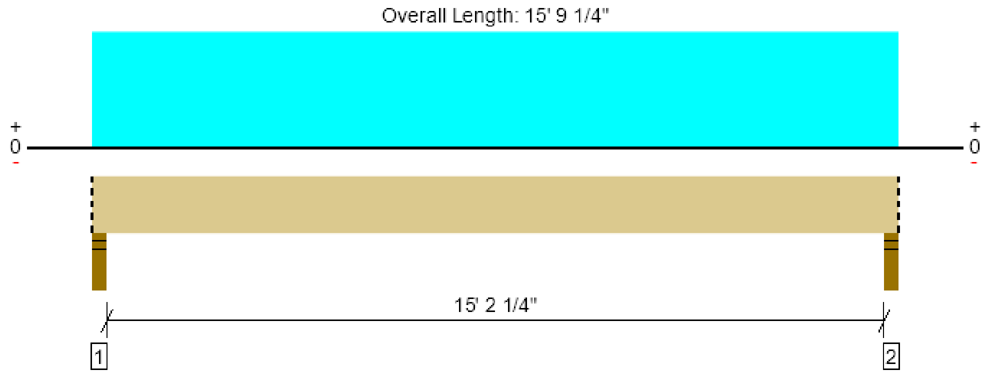
ForteWEB Software Operator	Job Notes
Chon Pieruccioni Pieruccioni Engineering (206) 949-7866 cpieru@hotmail.com	

3rd Floor Framing, Floor Joist 15'-2" and Under

1 piece(s) 2 x 12 DF No.2 @ 16" OC



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)	736 @ 2 1/2"	2126 (3.50")	Passed (35%)	--	1.0 D + 1.0 L (All Spans)
Shear (lbs)	621 @ 1' 2 3/4"	2025	Passed (31%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	2750 @ 7' 10 5/8"	2729	Passed (101%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.234 @ 7' 10 5/8"	0.512	Passed (L/787)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.410 @ 7' 10 5/8"	0.768	Passed (L/450)	--	1.0 D + 1.0 L (All Spans)
TJ-Pro™ Rating	N/A	N/A	N/A	--	N/A

Member Length : 15' 9 1/4"
System : Floor
Member Type : Joist
Building Use : Residential
Building Code : IBC 2018
Design Methodology : ASD

- Deflection criteria: LL (L/360) and TL (L/240).
- Allowed moment does not reflect the adjustment for the beam stability factor.
- A 15% increase in the moment capacity has been added to account for repetitive member usage.
- Applicable calculations are based on NDS.
- No composite action between deck and joist was considered in analysis.

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Floor Live	Factored	
1 - Stud wall - HF	3.50"	3.50"	1.50"	315	421	736	Blocking
2 - Stud wall - HF	3.50"	3.50"	1.50"	315	421	736	Blocking

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

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	6" o/c	
Bottom Edge (Lu)	15' 9" o/c	

•Maximum allowable bracing intervals based on applied load.

Vertical Load	Location (Side)	Spacing	Dead (0.90)	Floor Live (1.00)	Comments
1 - Uniform (PSF)	0 to 15' 9 1/4"	16"	30.0	40.0	Default Load

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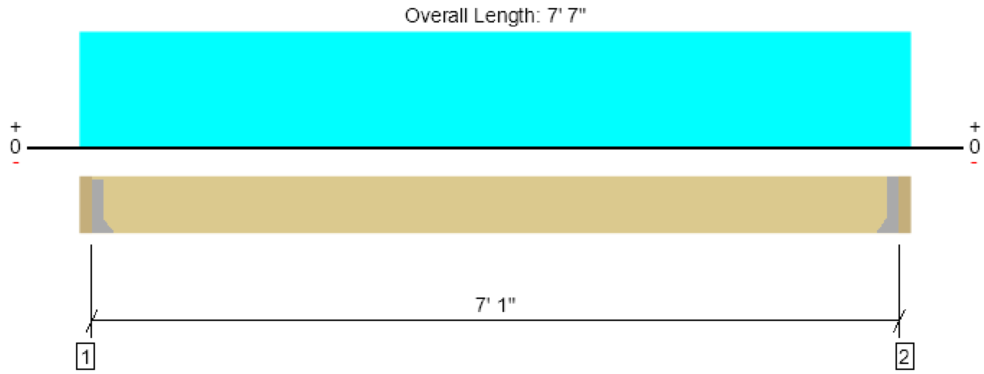
The product application, input design loads, dimensions and support information have been provided by ForteWEB Software Operator

ForteWEB Software Operator	Job Notes
Chon Pieruccioni Pieruccioni Engineering (206) 949-7866 cpieru@hotmail.com	

3rd Floor Framing, 7'-1" Landing Joists

1 piece(s) 2 x 12 HF No.2 @ 16" OC



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)	921 @ 3"	921 (1.52")	Passed (100%)	--	1.0 D + 1.0 L (All Spans)
Shear (lbs)	677 @ 1' 2 1/4"	1688	Passed (40%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	1631 @ 3' 9 1/2"	2577	Passed (63%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.049 @ 3' 9 1/2"	0.236	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.064 @ 3' 9 1/2"	0.354	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)
TJ-Pro™ Rating	N/A	N/A	N/A	--	N/A

Member Length : 7' 1"
System : Floor
Member Type : Joist
Building Use : Residential
Building Code : IBC 2018
Design Methodology : ASD

- Deflection criteria: LL (L/360) and TL (L/240).
- Allowed moment does not reflect the adjustment for the beam stability factor.
- A 15% increase in the moment capacity has been added to account for repetitive member usage.
- Applicable calculations are based on NDS.
- No composite action between deck and joist was considered in analysis.

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Floor Live	Factored	
1 - Hanger on 11 1/4" LSL beam	3.00"	Hanger¹	1.52"	228	758	986	See note ¹
2 - Hanger on 11 1/4" LSL beam	3.00"	Hanger¹	1.52"	228	758	986	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)	6' 10" o/c	
Bottom Edge (Lu)	7' 1" o/c	

•Maximum allowable bracing intervals based on applied load.

Connector: Simpson Strong-Tie

Support	Model	Seat Length	Top Fasteners	Face Fasteners	Member Fasteners	Accessories
1 - Face Mount Hanger	THA29	2.25"	N/A	16-10d	4-10d	
2 - Face Mount Hanger	THA29	2.25"	N/A	16-10d	4-10d	

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

Vertical Load	Location (Side)	Spacing	Dead (0.90)	Floor Live (1.00)	Comments
1 - Uniform (PSF)	0 to 7' 7"	16"	45.0	150.0	Default Load

Weyerhaeuser Notes

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Chon Pieruccioni Pieruccioni Engineering (206) 949-7866 cpieru@hotmail.com	

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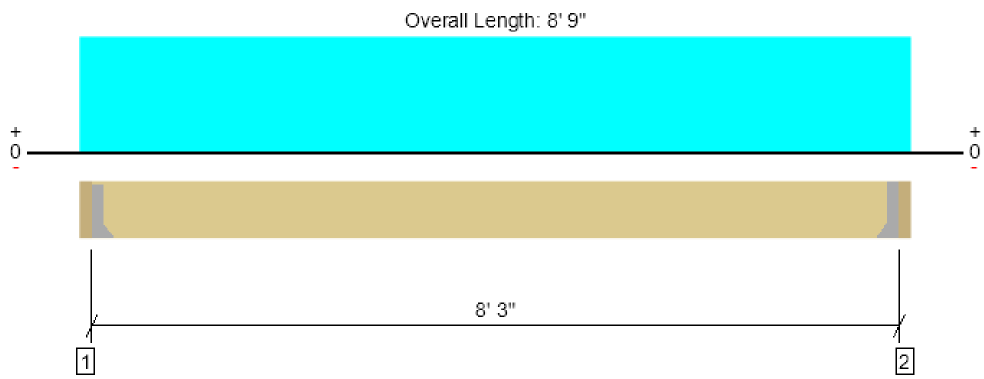
3rd Floor Framing, 8'-3" Landing Joists

1 piece(s) 2 x 12 HF No.2 @ 12" OC



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)	804 @ 3"	911 (1.50")	Passed (88%)	--	1.0 D + 1.0 L (All Spans)
Shear (lbs)	622 @ 1' 2 1/4"	1688	Passed (37%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	1659 @ 4' 4 1/2"	2577	Passed (64%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.068 @ 4' 4 1/2"	0.275	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.088 @ 4' 4 1/2"	0.412	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)
TJ-Pro™ Rating	N/A	N/A	N/A	--	N/A

Member Length : 8' 3"
System : Floor
Member Type : Joist
Building Use : Residential
Building Code : IBC 2018
Design Methodology : ASD

- Deflection criteria: LL (L/360) and TL (L/240).
- Allowed moment does not reflect the adjustment for the beam stability factor.
- A 15% increase in the moment capacity has been added to account for repetitive member usage.
- Applicable calculations are based on NDS.
- No composite action between deck and joist was considered in analysis.

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Floor Live	Factored	
1 - Hanger on 11 1/4" LSL beam	3.00"	Hanger[1]	1.50"	197	656	853	See note [1]
2 - Hanger on 11 1/4" LSL beam	3.00"	Hanger[1]	1.50"	197	656	853	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)	6' 8" o/c	
Bottom Edge (Lu)	8' 3" o/c	

•Maximum allowable bracing intervals based on applied load.

Connector: Simpson Strong-Tie

Support	Model	Seat Length	Top Fasteners	Face Fasteners	Member Fasteners	Accessories
1 - Face Mount Hanger	LUS28	1.75"	N/A	6-10dx1.5	3-10d	
2 - Face Mount Hanger	LUS28	1.75"	N/A	6-10dx1.5	3-10d	

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

Vertical Load	Location (Side)	Spacing	Dead (0.90)	Floor Live (1.00)	Comments
1 - Uniform (PSF)	0 to 8' 9"	12"	45.0	150.0	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
Chon Pieruccioni Pieruccioni Engineering (206) 949-7866 cpieru@hotmail.com	

Weyerhaeuser

6/2/2025 6:46:29 PM UTC

ForteWEB v3.9, Engine: V8.4.3.94, Data: V8.1.7.3

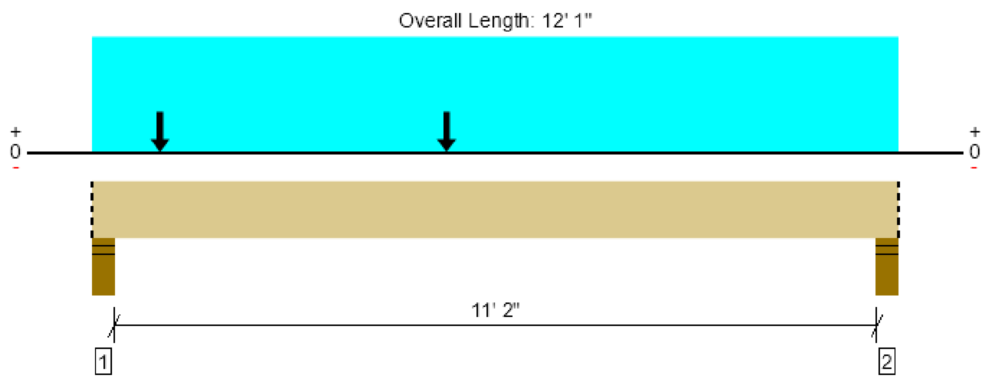
File Name: East Town Crossing Building A

MEMBER REPORT

PASSED

3rd Floor Framing, Top Landing Beam

1 piece(s) 5 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)	7824 @ 4"	12251 (5.50")	Passed (64%)	--	1.0 D + 1.0 L (All Spans)
Shear (lbs)	5861 @ 1' 5 1/2"	11660	Passed (50%)	1.00	1.0 D + 1.0 L (All Spans)
Pos Moment (Ft-lbs)	19527 @ 5' 3 3/4"	26400	Passed (74%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.235 @ 5' 11 13/16"	0.285	Passed (L/583)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.311 @ 5' 11 13/16"	0.571	Passed (L/440)	--	1.0 D + 1.0 L (All Spans)

Member Length : 12' 1"
System : Floor
Member Type : Flush Beam
Building Use : Residential
Building Code : IBC 2018
Design Methodology : ASD

- Deflection criteria: LL (L/480) and TL (L/240).
- Allowed moment does not reflect the adjustment for the beam stability factor.
- Volume factor of 1.00 was calculated for positive bending using length L = 11' 5".
- 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 - HF	5.50"	5.50"	3.51"	1922	5902	7824	Blocking
2 - Stud wall - HF	5.50"	5.50"	2.81"	1534	4731	6265	Blocking

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

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	12' 1" o/c	
Bottom Edge (Lu)	12' 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)	0 to 12' 1"	N/A	16.0	--	
1 - Uniform (PSF)	0 to 12' 1" (Front)	4' 7"	45.0	150.0	Default Load
2 - Point (lb)	1' 1/4" (Front)	N/A	385	1163	Linked from: Short Stair Stringers, Support 1
3 - Point (lb)	5' 3 3/4" (Front)	N/A	385	1163	Linked from: Short Stair Stringers, Support 1

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

Weyerhaeuser Notes

ForteWEB Software Operator	Job Notes
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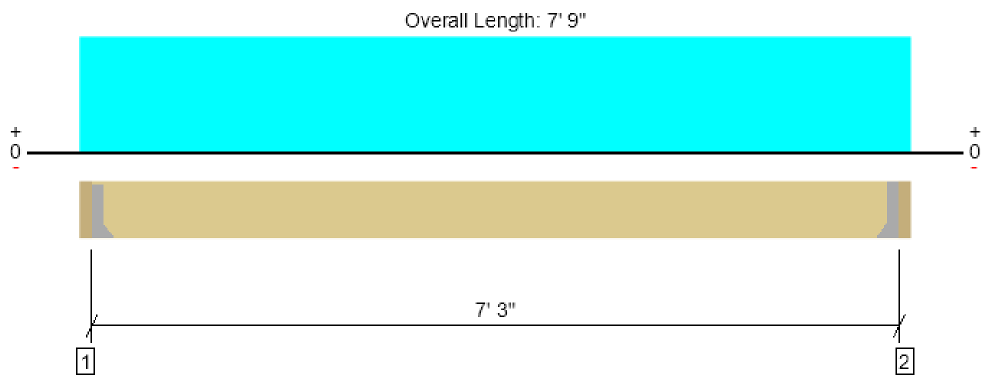
6/2/2025 6:46:29 PM UTC
ForteWEB v3.9, Engine: V8.4.3.94, Data: V8.1.7.3
File Name: East Town Crossing Building A

3rd Floor Framing, Short Stair Stringers

1 piece(s) 4 x 12 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)	1450 @ 3"	2126 (1.50")	Passed (68%)	--	1.0 D + 1.0 L (All Spans)
Shear (lbs)	1075 @ 1' 2 1/4"	3938	Passed (27%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	2628 @ 3' 10 1/2"	5752	Passed (46%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.035 @ 3' 10 1/2"	0.181	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.046 @ 3' 10 1/2"	0.363	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)

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

Member Length : 7' 3"
System : Floor
Member Type : Flush Beam
Building Use : Residential
Building Code : IBC 2018
Design Methodology : ASD

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Floor Live	Factored	
1 - Hanger on 11 1/4" GLB beam	3.00"	Hanger[1]	1.50"	385	1163	1547	See note [1]
2 - Hanger on 11 1/4" GLB beam	3.00"	Hanger[1]	1.50"	385	1163	1547	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' 3" o/c	
Bottom Edge (Lu)	7' 3" o/c	

•Maximum allowable bracing intervals based on applied load.

Connector: Simpson Strong-Tie

Support	Model	Seat Length	Top Fasteners	Face Fasteners	Member Fasteners	Accessories
1 - Face Mount Hanger	LUS410	2.00"	N/A	8-10d	6-10d	
2 - Face Mount Hanger	LUS410	2.00"	N/A	8-10d	6-10d	

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

Vertical Loads	Location (Side)	Tributary Width	Dead (0.90)	Floor Live (1.00)	Comments
0 - Self Weight (PLF)	3" to 7' 6"	N/A	10.0	--	
1 - Uniform (PSF)	0 to 7' 9" (Front)	2'	45.0	150.0	Default Load

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

Weyerhaeuser Notes

ForteWEB Software Operator	Job Notes
Chon Pieruccioni Pieruccioni Engineering (206) 949-7866 cpieru@hotmail.com	

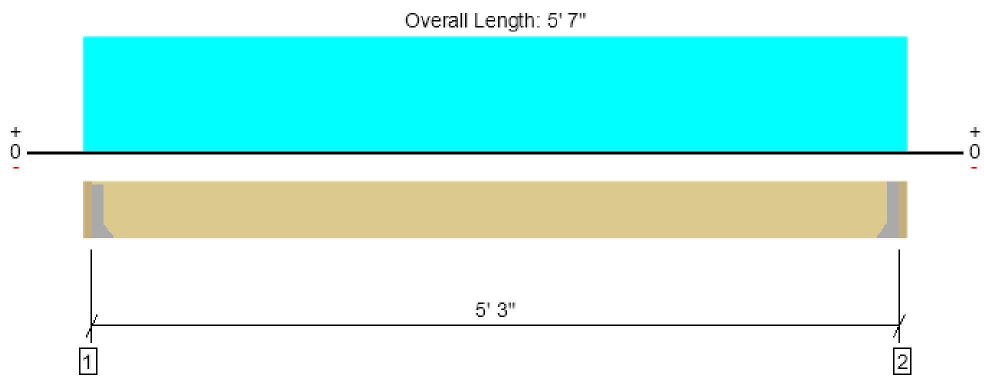
FORTEWEB

MEMBER REPORT

PASSED

3rd Floor Framing, 5'-3" Mid Landing Joists

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



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)	683 @ 2"	911 (1.50")	Passed (75%)	--	1.0 D + 1.0 L (All Spans)
Shear (lbs)	525 @ 9 1/4"	1088	Passed (48%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	896 @ 2' 9 1/2"	1284	Passed (70%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.055 @ 2' 9 1/2"	0.175	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.072 @ 2' 9 1/2"	0.262	Passed (L/878)	--	1.0 D + 1.0 L (All Spans)
TJ-Pro™ Rating	N/A	N/A	N/A	--	N/A

Member Length : 5' 3"
System : Floor
Member Type : Joist
Building Use : Residential
Building Code : IBC 2018
Design Methodology : ASD

- Deflection criteria: LL (L/360) and TL (L/240).
- Allowed moment does not reflect the adjustment for the beam stability factor.
- A 15% increase in the moment capacity has been added to account for repetitive member usage.
- Applicable calculations are based on NDS.
- No composite action between deck and joist was considered in analysis.

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Floor Live	Factored	
1 - Hanger on 7 1/4" LSL beam	2.00"	Hanger[1]	1.50"	168	558	726	See note [1]
2 - Hanger on 7 1/4" LSL beam	2.00"	Hanger[1]	1.50"	168	558	726	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)	5' 3" o/c	
Bottom Edge (Lu)	5' 3" o/c	

•Maximum allowable bracing intervals based on applied load.

Connector: Simpson Strong-Tie

Support	Model	Seat Length	Top Fasteners	Face Fasteners	Member Fasteners	Accessories
1 - Face Mount Hanger	LU26	1.50"	N/A	6-10d	4-10dx1.5	
2 - Face Mount Hanger	LU26	1.50"	N/A	6-10d	4-10dx1.5	

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

Vertical Load	Location (Side)	Spacing	Dead (0.90)	Floor Live (1.00)	Comments
1 - Uniform (PSF)	0 to 5' 7"	16"	45.0	150.0	Default Load

Weyerhaeuser Notes

ForteWEB Software Operator	Job Notes
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Weyerhaeuser

6/2/2025 6:46:29 PM UTC

ForteWEB v3.9, Engine: V8.4.3.94, Data: V8.1.7.3

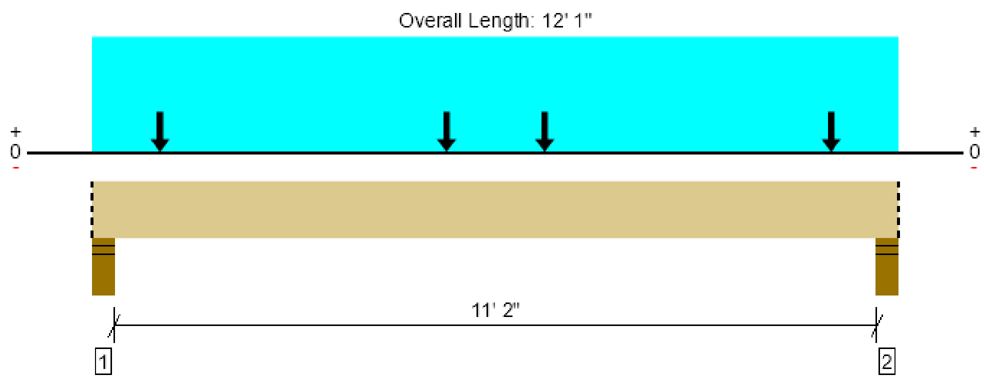
File Name: East Town Crossing Building A

MEMBER REPORT

PASSED

3rd Floor Framing, Mid Landing Beam Inner

1 piece(s) 5 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)	6828 @ 11' 9"	12251 (5.50")	Passed (56%)	--	1.0 D + 1.0 L (All Spans)
Shear (lbs)	5286 @ 1' 5 1/2"	11660	Passed (45%)	1.00	1.0 D + 1.0 L (All Spans)
Pos Moment (Ft-lbs)	18813 @ 6' 7/16"	26400	Passed (71%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.225 @ 6' 1/2"	0.285	Passed (L/609)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.300 @ 6' 1/2"	0.571	Passed (L/457)	--	1.0 D + 1.0 L (All Spans)

Member Length : 12' 1"
System : Floor
Member Type : Flush Beam
Building Use : Residential
Building Code : IBC 2018
Design Methodology : ASD

- Deflection criteria: LL (L/480) and TL (L/240).
- Allowed moment does not reflect the adjustment for the beam stability factor.
- Volume factor of 1.00 was calculated for positive bending using length L = 11' 5".
- 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 - HF	5.50"	5.50"	3.06"	1704	5118	6823	Blocking
2 - Stud wall - HF	5.50"	5.50"	3.07"	1706	5122	6828	Blocking

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

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	12' 1" o/c	
Bottom Edge (Lu)	12' 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)	0 to 12' 1"	N/A	16.0	--	
1 - Uniform (PSF)	0 to 12' 1" (Front)	3' 1"	45.0	150.0	Default Load
2 - Point (lb)	1' 1/4" (Front)	N/A	385	1163	Linked from: Short Stair Stringers, Support 1
3 - Point (lb)	5' 3 3/4" (Front)	N/A	385	1163	Linked from: Short Stair Stringers, Support 1
4 - Point (lb)	6' 9 3/8" (Front)	N/A	385	1163	Linked from: Short Stair Stringers, Support 1
5 - Point (lb)	11' 7/8" (Front)	N/A	385	1163	Linked from: Short Stair Stringers, Support 1

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

ForteWEB Software Operator	Job Notes
Chon Pieruccioni Pieruccioni Engineering (206) 949-7866 cpieru@hotmail.com	

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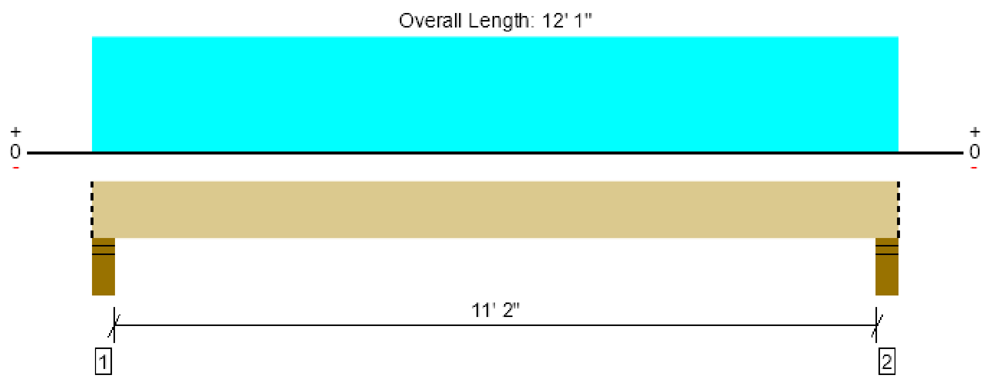
The product application, input design loads, dimensions and support information have been provided by ForteWEB Software Operator

MEMBER REPORT

PASSED

3rd Floor Framing, Mid Landing Beam Outer

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)	3687 @ 4"	7796 (5.50")	Passed (47%)	--	1.0 D + 1.0 L (All Spans)
Shear (lbs)	2873 @ 1' 4"	6493	Passed (44%)	1.00	1.0 D + 1.0 L (All Spans)
Pos Moment (Ft-lbs)	9941 @ 6' 1/2"	12863	Passed (77%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.291 @ 6' 1/2"	0.285	Passed (L/471)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.384 @ 6' 1/2"	0.571	Passed (L/357)	--	1.0 D + 1.0 L (All Spans)

Member Length : 12' 1"
System : Floor
Member Type : Flush Beam
Building Use : Residential
Building Code : IBC 2018
Design Methodology : ASD

- Deflection criteria: LL (L/480) and TL (L/240).
- Allowed moment does not reflect the adjustment for the beam stability factor.
- Volume factor of 1.00 was calculated for positive bending using length L = 11' 5".
- 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 - HF	5.50"	5.50"	2.60"	892	2794	3687	Blocking
2 - Stud wall - HF	5.50"	5.50"	2.60"	892	2794	3687	Blocking

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

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	12' 1" o/c	
Bottom Edge (Lu)	12' 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)	0 to 12' 1"	N/A	8.9	--	
1 - Uniform (PSF)	0 to 12' 1" (Front)	3' 1"	45.0	150.0	Default Load

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

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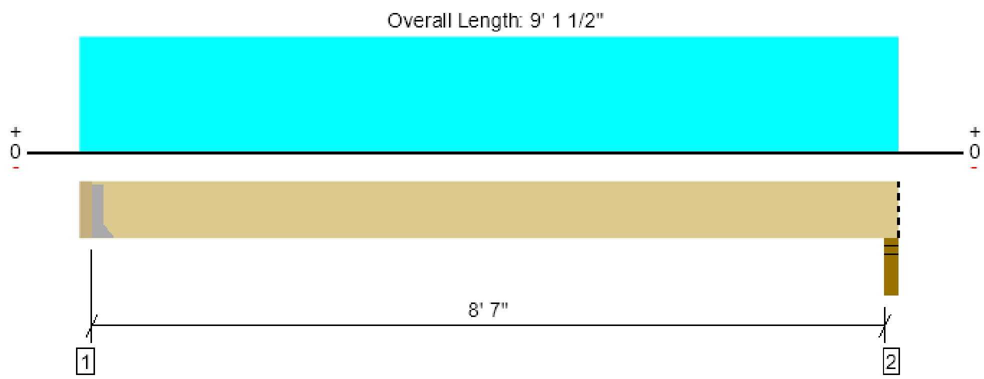
The product application, input design loads, dimensions and support information have been provided by ForteWEB Software Operator

ForteWEB Software Operator	Job Notes
Chon Pieruccioni Pieruccioni Engineering (206) 949-7866 cpieru@hotmail.com	

3rd Floor Framing, Grid 1&8 Deck Beam

2 piece(s) 2 x 10 DF 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)	1027 @ 3"	2813 (1.50")	Passed (37%)	--	1.0 D + 1.0 L (All Spans)
Shear (lbs)	845 @ 1' 1/4"	3330	Passed (25%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	2235 @ 4' 7 1/4"	3529	Passed (63%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.062 @ 4' 7 1/4"	0.218	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.096 @ 4' 7 1/4"	0.435	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)

Member Length : 8' 10 1/2"
System : Floor
Member Type : Flush Beam
Building Use : Residential
Building Code : IBC 2018
Design Methodology : ASD

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

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Floor Live	Factored	
1 - Hanger on 9 1/4" HF beam	3.00"	Hanger[1]	1.50"	382	702	1084	See note [1]
2 - Stud wall - HF	3.50"	3.50"	1.50"	376	689	1066	Blocking

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

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	8' 11" o/c	
Bottom Edge (Lu)	8' 11" o/c	

•Maximum allowable bracing intervals based on applied load.

Connector: Simpson Strong-Tie

Support	Model	Seat Length	Top Fasteners	Face Fasteners	Member Fasteners	Accessories
1 - Face Mount Hanger	LUS28-2	2.00"	N/A	6-16d	3-16d	

- 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)	Comments
0 - Self Weight (PLF)	3" to 9' 1 1/2"	N/A	7.0	--	
1 - Uniform (PSF)	0 to 9' 1 1/2" (Front)	2' 6 1/2"	30.0	60.0	Default Load

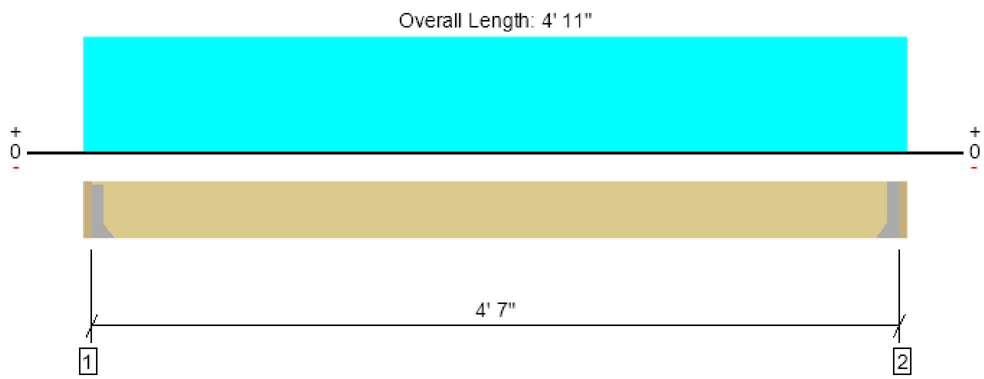
- Side loads are assumed to not induce cross-grain tension.

Weyerhaeuser Notes

ForteWEB Software Operator	Job Notes
Chon Pieruccioni Pieruccioni Engineering (206) 949-7866 cpieru@hotmail.com	

3rd Floor Framing, 4'-7" Deck Joist

1 piece(s) 2 x 10 HF No.2 @ 16" OC



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)	275 @ 2"	911 (1.50")	Passed (30%)	--	1.0 D + 1.0 L (All Spans)
Shear (lbs)	183 @ 11 1/4"	1388	Passed (13%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	315 @ 2' 5 1/2"	1917	Passed (16%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.006 @ 2' 5 1/2"	0.153	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.009 @ 2' 5 1/2"	0.229	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)
TJ-Pro™ Rating	N/A	N/A	N/A	--	N/A

Member Length : 4' 7"
System : Floor
Member Type : Joist
Building Use : Residential
Building Code : IBC 2018
Design Methodology : ASD

- Deflection criteria: LL (L/360) and TL (L/240).
- Allowed moment does not reflect the adjustment for the beam stability factor.
- A 15% increase in the moment capacity has been added to account for repetitive member usage.
- Applicable calculations are based on NDS.
- No composite action between deck and joist was considered in analysis.

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Floor Live	Factored	
1 - Hanger on 9 1/4" HF beam	2.00"	Hanger[1]	1.50"	98	197	295	See note [1]
2 - Hanger on 9 1/4" HF beam	2.00"	Hanger[1]	1.50"	98	197	295	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)	4' 7" o/c	
Bottom Edge (Lu)	4' 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 - Face Mount Hanger	LU28	1.50"	N/A	8-10dx1.5	6-10dx1.5	
2 - Face Mount Hanger	LU28	1.50"	N/A	8-10dx1.5	6-10dx1.5	

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

Vertical Load	Location (Side)	Spacing	Dead (0.90)	Floor Live (1.00)	Comments
1 - Uniform (PSF)	0 to 4' 11"	16"	30.0	60.0	Default Load

Weyerhaeuser Notes

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

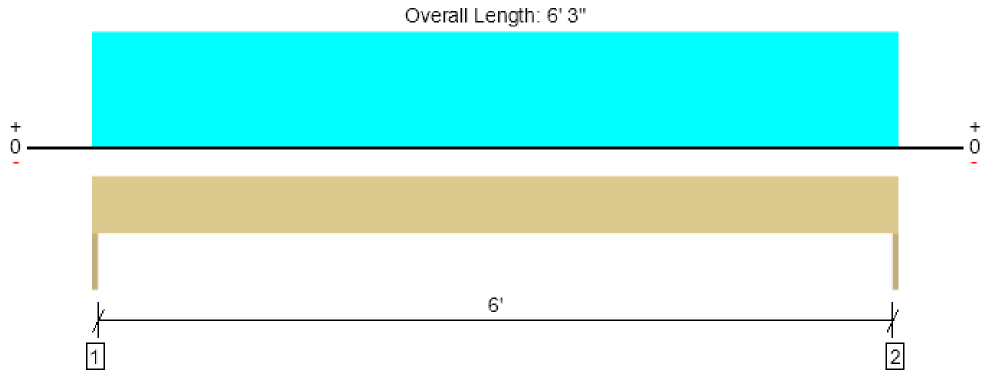
ForteWEB Software Operator	Job Notes
Chon Pieruccioni Pieruccioni Engineering (206) 949-7866 cpieru@hotmail.com	

MEMBER REPORT

PASSED

3rd Floor Framing, 6' Window Header (Grids 1&8)

1 piece(s) 4 x 8 DF 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)	1688 @ 0	3281 (1.50")	Passed (51%)	--	1.0 D + 1.0 L (All Spans)
Shear (lbs)	1294 @ 8 3/4"	3045	Passed (43%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	2638 @ 3' 1 1/2"	2989	Passed (88%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.047 @ 3' 1 1/2"	0.208	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.104 @ 3' 1 1/2"	0.313	Passed (L/719)	--	1.0 D + 1.0 L (All Spans)

Member Length : 6' 3"
System : Wall
Member Type : Header
Building Use : Residential
Building Code : IBC 2018
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	Factored	
1 - Trimmer - HF	1.50"	1.50"	1.50"	928	760	1688	None
2 - Trimmer - HF	1.50"	1.50"	1.50"	928	760	1688	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)	Comments
0 - Self Weight (PLF)	0 to 6' 3"	N/A	6.4	--	
1 - Uniform (PSF)	0 to 6' 3"	6' 1"	30.0	40.0	Floor
2 - Uniform (PLF)	0 to 6' 3"	N/A	108.0	--	Wall

Weyerhaeuser Notes

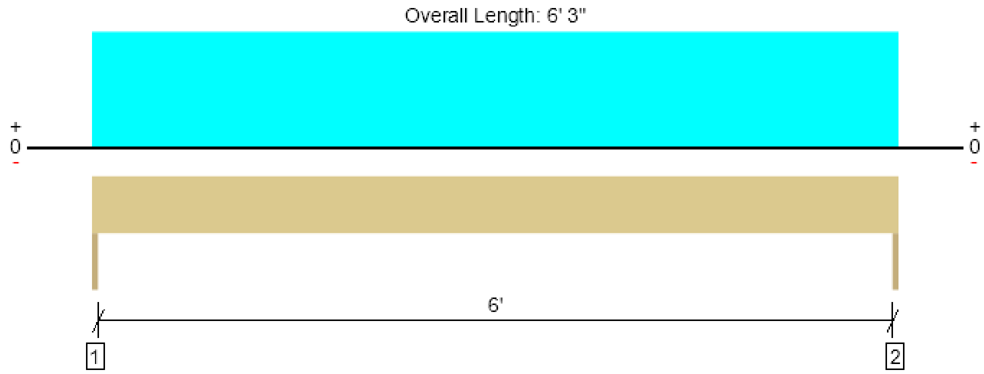
ForteWEB Software Operator	Job Notes
Chon Pieruccioni Pieruccioni Engineering (206) 949-7866 cpieru@hotmail.com	

MEMBER REPORT

PASSED

3rd Floor Framing, 6' Window Header (Grid A)

1 piece(s) 4 x 10 DF 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)	2292 @ 0	3281 (1.50")	Passed (70%)	--	1.0 D + 1.0 S (All Spans)
Shear (lbs)	1635 @ 10 3/4"	4468	Passed (37%)	1.15	1.0 D + 1.0 S (All Spans)
Moment (Ft-lbs)	3580 @ 3' 1 1/2"	5166	Passed (69%)	1.15	1.0 D + 1.0 S (All Spans)
Live Load Defl. (in)	0.034 @ 3' 1 1/2"	0.208	Passed (L/999+)	--	1.0 D + 1.0 S (All Spans)
Total Load Defl. (in)	0.068 @ 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 2018
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"	1159	1133	2292	None
2 - Trimmer - HF	1.50"	1.50"	1.50"	1159	1133	2292	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	8.2	--	
1 - Uniform (PSF)	0 to 6' 3"	14' 6"	25.0	25.0	Roof

Weyerhaeuser Notes

ForteWEB Software Operator	Job Notes
Chon Pieruccioni Pieruccioni Engineering (206) 949-7866 cpieru@hotmail.com	

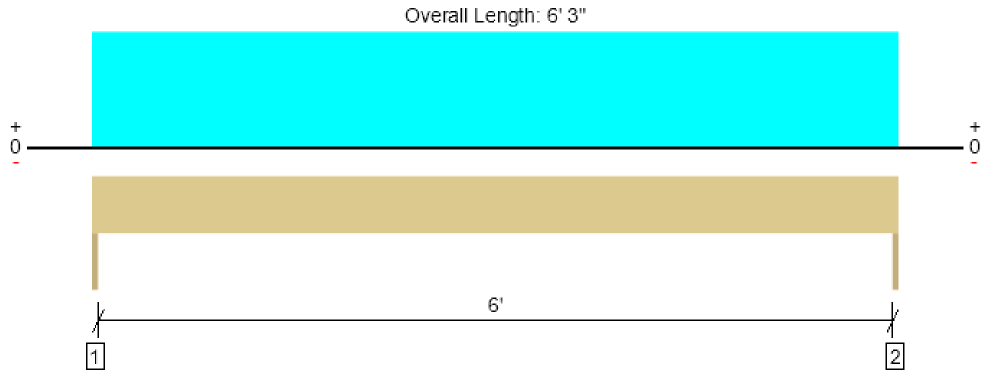
Weyerhaeuser

MEMBER REPORT

PASSED

3rd Floor Framing, 6' Window Header (Grid B)

1 piece(s) 4 x 10 DF 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)	2370 @ 0	3281 (1.50")	Passed (72%)	--	1.0 D + 1.0 S (All Spans)
Shear (lbs)	1690 @ 10 3/4"	4468	Passed (38%)	1.15	1.0 D + 1.0 S (All Spans)
Moment (Ft-lbs)	3703 @ 3' 1 1/2"	5166	Passed (72%)	1.15	1.0 D + 1.0 S (All Spans)
Live Load Defl. (in)	0.035 @ 3' 1 1/2"	0.208	Passed (L/999+)	--	1.0 D + 1.0 S (All Spans)
Total Load Defl. (in)	0.070 @ 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 2018
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"	1198	1172	2370	None
2 - Trimmer - HF	1.50"	1.50"	1.50"	1198	1172	2370	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.

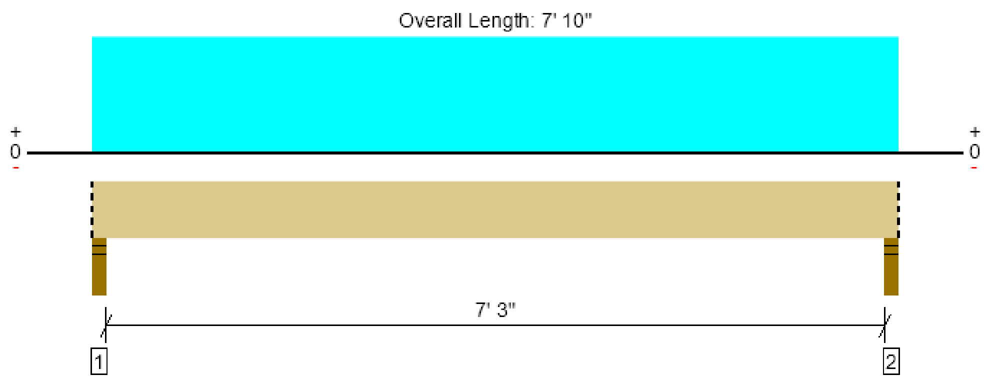
Vertical Loads	Location	Tributary Width	Dead (0.90)	Snow (1.15)	Comments
0 - Self Weight (PLF)	0 to 6' 3"	N/A	8.2	--	
1 - Uniform (PSF)	0 to 6' 3"	15'	25.0	25.0	Roof

Weyerhaeuser Notes

ForteWEB Software Operator	Job Notes
Chon Pieruccioni Pieruccioni Engineering (206) 949-7866 cpieru@hotmail.com	

3rd Floor Framing, Grid A - Deck Roof Beams

1 piece(s) 3 1/2" x 7 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)	2865 @ 2"	4961 (3.50")	Passed (58%)	--	1.0 D + 1.0 S (All Spans)
Shear (lbs)	2195 @ 11"	5333	Passed (41%)	1.15	1.0 D + 1.0 S (All Spans)
Pos Moment (Ft-lbs)	5144 @ 3' 11"	7547	Passed (68%)	1.15	1.0 D + 1.0 S (All Spans)
Live Load Defl. (in)	0.117 @ 3' 11"	0.375	Passed (L/772)	--	1.0 D + 1.0 S (All Spans)
Total Load Defl. (in)	0.235 @ 3' 11"	0.500	Passed (L/383)	--	1.0 D + 1.0 S (All Spans)

Member Length : 7' 10"
System : Roof
Member Type : Drop Beam
Building Use : Residential
Building Code : IBC 2018
Design Methodology : ASD
Member Pitch : 0.25/12

- Deflection criteria: LL (L/240) and TL (L/180).
- Allowed moment does not reflect the adjustment for the beam stability factor.
- Volume factor of 1.00 was calculated for positive bending using length L = 7' 6".
- The effects of positive or negative camber have not been accounted for when calculating deflection.
- The specified glulam is assumed to have its strong laminations at the bottom of the beam. Install with proper side up as indicated by the manufacturer.
- Applicable calculations are based on NDS.

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Snow	Factored	
1 - Stud wall - HF	3.50"	3.50"	2.02"	1445	1420	2865	Blocking
2 - Stud wall - HF	3.50"	3.50"	2.02"	1445	1420	2865	Blocking

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

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	7' 10" o/c	
Bottom Edge (Lu)	7' 10" o/c	

•Maximum allowable bracing intervals based on applied load.

Vertical Loads	Location (Side)	Tributary Width	Dead (0.90)	Snow (1.15)	Comments
0 - Self Weight (PLF)	0 to 7' 10"	N/A	6.4	--	
1 - Uniform (PSF)	0 to 7' 10" (Front)	14' 6"	25.0	25.0	Default Load

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

Weyerhaeuser Notes

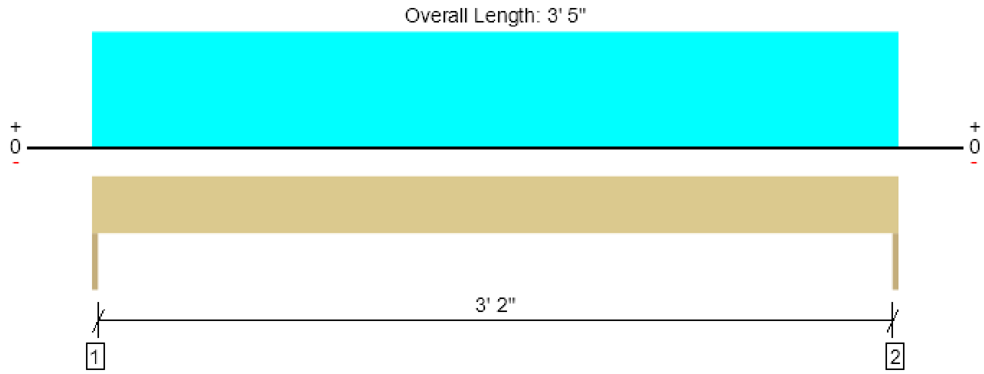
ForteWEB Software Operator	Job Notes
Chon Pieruccioni Pieruccioni Engineering (206) 949-7866 cpieru@hotmail.com	

MEMBER REPORT

PASSED

3rd Floor Framing, Grid 3.1 (D-D.2) Door Header

1 piece(s) 4 x 8 DF 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)	1566 @ 0	3281 (1.50")	Passed (48%)	--	1.0 D + 1.0 L (All Spans)
Shear (lbs)	897 @ 8 3/4"	3045	Passed (29%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	1337 @ 1' 8 1/2"	2989	Passed (45%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.009 @ 1' 8 1/2"	0.114	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.016 @ 1' 8 1/2"	0.171	Passed (L/999+)	--	1.0 D + 1.0 L (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 : 3' 5"
System : Wall
Member Type : Header
Building Use : Residential
Building Code : IBC 2018
Design Methodology : ASD

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Floor Live	Factored	
1 - Trimmer - HF	1.50"	1.50"	1.50"	677	888	1566	None
2 - Trimmer - HF	1.50"	1.50"	1.50"	677	888	1566	None

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	3' 5" o/c	
Bottom Edge (Lu)	3' 5" o/c	

•Maximum allowable bracing intervals based on applied load.

Vertical Loads	Location	Tributary Width	Dead (0.90)	Floor Live (1.00)	Comments
0 - Self Weight (PLF)	0 to 3' 5"	N/A	6.4	--	
1 - Uniform (PSF)	0 to 3' 5"	13'	30.0	40.0	Default Load

Weyerhaeuser Notes

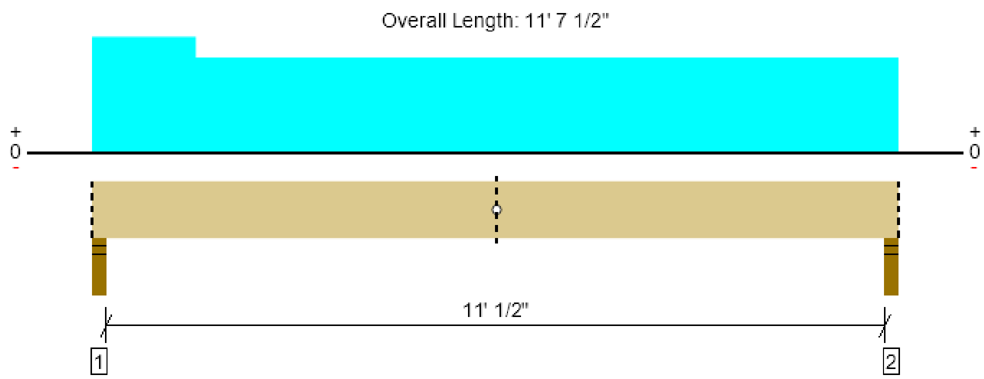
ForteWEB Software Operator	Job Notes
Chon Pieruccioni Pieruccioni Engineering (206) 949-7866 cpieru@hotmail.com	

MEMBER REPORT

PASSED

3rd Floor Framing, Grid 3.1 (D.3-D.6) Flush Beam

2 piece(s) 1 3/4" x 11 1/4" 2.0E Microllam® LVL



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)	4872 @ 2"	4961 (3.50")	Passed (98%)	--	1.0 D + 1.0 L (All Spans)
Shear (lbs)	3682 @ 1' 2 3/4"	7481	Passed (49%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	12764 @ 5' 9 9/16"	16137	Passed (79%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.220 @ 5' 9 11/16"	0.376	Passed (L/616)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.390 @ 5' 9 11/16"	0.565	Passed (L/347)	--	1.0 D + 1.0 L (All Spans)

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

Member Length : 11' 7 1/2"
System : Floor
Member Type : Flush Beam
Building Use : Residential
Building Code : IBC 2018
Design Methodology : ASD

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Floor Live	Factored	
1 - Stud wall - HF	3.50"	3.50"	3.44"	2126	2746	4872	Blocking
2 - Stud wall - HF	3.50"	3.50"	3.27"	2027	2614	4641	Blocking

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

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	9' 2" o/c	
Bottom Edge (Lu)	11' 8" o/c	

•Maximum allowable bracing intervals based on applied load.

Vertical Loads	Location (Side)	Tributary Width	Dead (0.90)	Floor Live (1.00)	Comments
0 - Self Weight (PLF)	0 to 11' 7 1/2"	N/A	11.5	--	
1 - Uniform (PSF)	0 to 1' 6" (Front)	13' 8"	30.0	40.0	Default Load
2 - Uniform (PSF)	1' 6" to 11' 7 1/2" (Front)	11' 2 1/2"	30.0	40.0	Default Load

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

Holes (Size)	Direction	Diameter	Vertical Offset	Location	Shear (lbs)			Moment (Ft-lbs)			Comments
					Actual	Allowed	Result	Actual	Allowed	Result	
1 - Circular (Per Lit.)	Horz	2.00"	5 5/8"	5' 10"	--	--	Passed	--	--	Passed	

- Hole locations are measured from the outside face of left support (or left cantilever end) to the centerline of the hole.
- Vertical Offset is measured from the top of the member to the centerline of the hole.

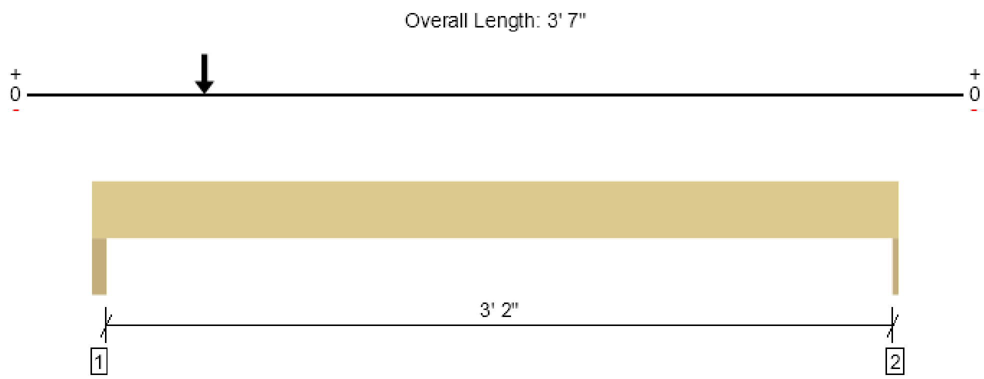
Weyerhaeuser Notes

ForteWEB Software Operator	Job Notes
Chon Pieruccioni Pieruccioni Engineering (206) 949-7866 cpieru@hotmail.com	

Weyerhaeuser

3rd Floor Framing, Grid 3.6 (3-3.3) Door Header

1 piece(s) 4 x 8 DF 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)	4409 @ 2"	7656 (3.50")	Passed (58%)	--	1.0 D + 1.0 L (All Spans)
Shear (lbs)	1522 @ 10 3/4"	3045	Passed (50%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	1469 @ 6"	2989	Passed (49%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.007 @ 1' 7 3/4"	0.114	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.012 @ 1' 7 13/16"	0.171	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)

Member Length : 3' 7"
System : Wall
Member Type : Header
Building Use : Residential
Building Code : IBC 2018
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	Factored	
1 - Trimmer - HF	3.50"	3.50"	2.02"	1931	2478	4409	None
2 - Trimmer - HF	1.50"	1.50"	1.50"	218	268	486	None

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	3' 7" o/c	
Bottom Edge (Lu)	3' 7" o/c	

•Maximum allowable bracing intervals based on applied load.

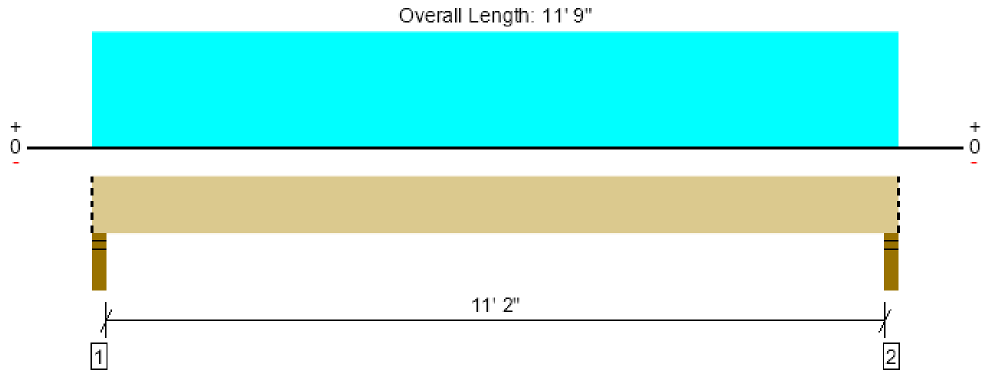
Vertical Loads	Location	Tributary Width	Dead (0.90)	Floor Live (1.00)	Comments
0 - Self Weight (PLF)	0 to 3' 7"	N/A	6.4	--	
1 - Point (lb)	6"	N/A	2126	2746	Linked from: Grid 3.1 (D.3-D.6) Flush Beam, Support 1

Weyerhaeuser Notes

ForteWEB Software Operator	Job Notes
Chon Pieruccioni Pieruccioni Engineering (206) 949-7866 cpieru@hotmail.com	

Roof Framing, Entry Roof Beam

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)	4974 @ 2"	4961 (3.50")	Passed (100%)	--	1.0 D + 1.0 S (All Spans)
Shear (lbs)	3986 @ 1' 2"	7466	Passed (53%)	1.15	1.0 D + 1.0 S (All Spans)
Pos Moment (Ft-lbs)	13794 @ 5' 10 1/2"	14792	Passed (93%)	1.15	1.0 D + 1.0 S (All Spans)
Live Load Defl. (in)	0.263 @ 5' 10 1/2"	0.571	Passed (L/520)	--	1.0 D + 1.0 S (All Spans)
Total Load Defl. (in)	0.533 @ 5' 10 1/2"	0.761	Passed (L/257)	--	1.0 D + 1.0 S (All Spans)

Member Length : 11' 9"
System : Roof
Member Type : Drop Beam
Building Use : Residential
Building Code : IBC 2018
Design Methodology : ASD
Member Pitch : 0.25/12

- Deflection criteria: LL (L/240) and TL (L/180).
- Allowed moment does not reflect the adjustment for the beam stability factor.
- Volume factor of 1.00 was calculated for positive bending using length L = 11' 5".
- The effects of positive or negative camber have not been accounted for when calculating deflection.
- The specified glulam is assumed to have its strong laminations at the bottom of the beam. Install with proper side up as indicated by the manufacturer.
- Applicable calculations are based on NDS.

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Snow	Factored	
1 - Stud wall - HF	3.50"	3.50"	3.50"	2514	2460	4974	Blocking
2 - Stud wall - HF	3.50"	3.50"	3.50"	2514	2460	4974	Blocking

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

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

•Maximum allowable bracing intervals based on applied load.

Vertical Loads	Location (Side)	Tributary Width	Dead (0.90)	Snow (1.15)	Comments
0 - Self Weight (PLF)	0 to 11' 9"	N/A	8.9	--	
1 - Uniform (PSF)	0 to 11' 9" (Front)	16' 9"	25.0	25.0	Default Load

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

Weyerhaeuser Notes

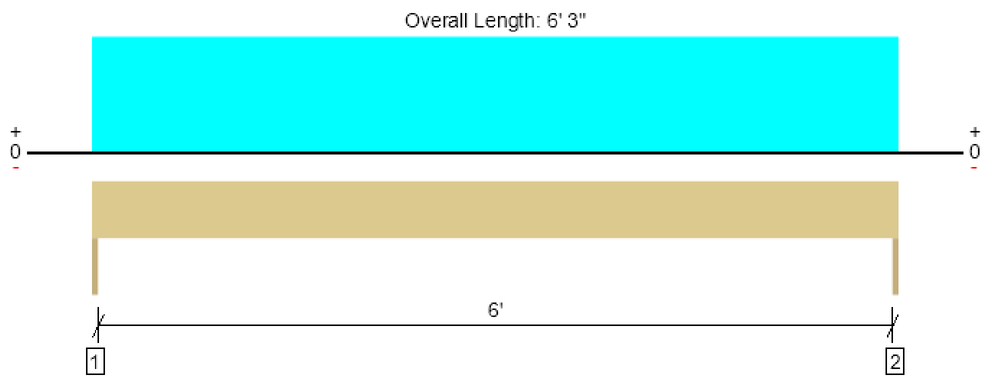
ForteWEB Software Operator	Job Notes
Chon Pieruccioni Pieruccioni Engineering (206) 949-7866 cpieru@hotmail.com	

MEMBER REPORT

PASSED

Roof Framing, 6' Window Header (Grid 1 & 7)

1 piece(s) 4 x 8 DF 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)	1322 @ 0	3281 (1.50")	Passed (40%)	--	1.0 D + 1.0 S (All Spans)
Shear (lbs)	1014 @ 8 3/4"	3502	Passed (29%)	1.15	1.0 D + 1.0 S (All Spans)
Moment (Ft-lbs)	2066 @ 3' 1 1/2"	3438	Passed (60%)	1.15	1.0 D + 1.0 S (All Spans)
Live Load Defl. (in)	0.040 @ 3' 1 1/2"	0.208	Passed (L/999+)	--	1.0 D + 1.0 S (All Spans)
Total Load Defl. (in)	0.082 @ 3' 1 1/2"	0.313	Passed (L/918)	--	1.0 D + 1.0 S (All Spans)

Member Length : 6' 3"
System : Wall
Member Type : Header
Building Use : Residential
Building Code : IBC 2018
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"	671	651	1322	None
2 - Trimmer - HF	1.50"	1.50"	1.50"	671	651	1322	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.

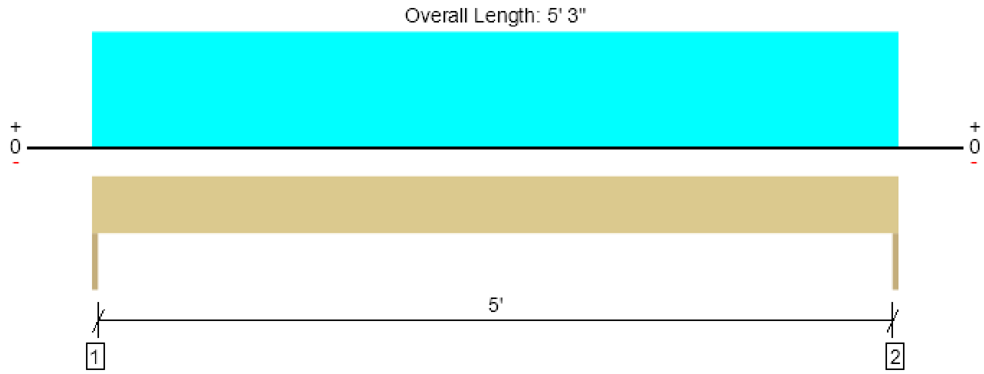
Vertical Loads	Location	Tributary Width	Dead (0.90)	Snow (1.15)	Comments
0 - Self Weight (PLF)	0 to 6' 3"	N/A	6.4	--	
1 - Uniform (PSF)	0 to 6' 3"	8' 4"	25.0	25.0	Default Load

Weyerhaeuser Notes

ForteWEB Software Operator	Job Notes
Chon Pieruccioni Pieruccioni Engineering (206) 949-7866 cpieru@hotmail.com	

Roof Framing, 5' Window Header (Grid H)

1 piece(s) 4 x 8 DF 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)	2470 @ 0	3281 (1.50")	Passed (75%)	--	1.0 D + 1.0 S (All Spans)
Shear (lbs)	1784 @ 8 3/4"	3502	Passed (51%)	1.15	1.0 D + 1.0 S (All Spans)
Moment (Ft-lbs)	3242 @ 2' 7 1/2"	3438	Passed (94%)	1.15	1.0 D + 1.0 S (All Spans)
Live Load Defl. (in)	0.045 @ 2' 7 1/2"	0.175	Passed (L/999+)	--	1.0 D + 1.0 S (All Spans)
Total Load Defl. (in)	0.090 @ 2' 7 1/2"	0.262	Passed (L/697)	--	1.0 D + 1.0 S (All Spans)

Member Length : 5' 3"
System : Wall
Member Type : Header
Building Use : Residential
Building Code : IBC 2018
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"	1244	1226	2470	None
2 - Trimmer - HF	1.50"	1.50"	1.50"	1244	1226	2470	None

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	5' 3" o/c	
Bottom Edge (Lu)	5' 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 5' 3"	N/A	6.4	--	
1 - Uniform (PSF)	0 to 5' 3"	18' 8 1/4"	25.0	25.0	Default Load

Weyerhaeuser Notes

ForteWEB Software Operator	Job Notes
Chon Pieruccioni Pieruccioni Engineering (206) 949-7866 cpieru@hotmail.com	

Weyerhaeuser

SPREAD FOOTING DESIGN

GEOMETRY

Footing Length (X-dir)	2.50	ft	
Footing Width (Z-dir)	2.50	ft	
Footing Thickness	8.0	in	OK
Soil Cover	0.00	ft	
Column Length (X-dir)	6.0	in	
Column Width (Z-dir)	6.0	in	
Offset (X-dir)	0.00	in	OK
Offset (Z-dir)	0.00	in	OK
Base Plate (L x W)	6.0 x 6.0	in	

SOIL PRESSURES (D+L)

Gross Allow. Soil Pressure	2.0	ksf
Soil Pressure at Corner 1	1.5	ksf
Soil Pressure at Corner 2	1.5	ksf
Soil Pressure at Corner 3	1.5	ksf
Soil Pressure at Corner 4	1.5	ksf
Bearing Pressure Ratio	0.77	OK
Ftg. Area in Contact with Soil	100.0	%
X-eccentricity / Ftg. Length	0.00	OK
Z-eccentricity / Ftg. Width	0.00	OK

APPLIED LOADS

	Dead	Live	RLive	Snow	Wind	Seismic	
Axial Force P	4.1	5.2	0.0	0.0	0.0	0.0	kip
Moment about X Mx ..	0.0	0.0	0.0	0.0	0.0	0.0	k-ft
Moment about Z Mz ..	0.0	0.0	0.0	0.0	0.0	0.0	k-ft
Shear Force Vx	0.0	0.0	0.0	0.0	0.0	0.0	kip
Shear Force Vz	0.0	0.0	0.0	0.0	0.0	0.0	kip

OVERTURNING CALCULATIONS (Comb: 0.6D+0.6W)

- Overturning about X-X

- Moment Mx = 0.6 * 0.0 + 0.6 * 0.0 = 0.0 k-ft

- Shear Force Vz = 0.6 * 0.0 + 0.6 * 0.0 = 0.0 kip

Arm = 0.00 + 8.0 / 12 = 0.67 ft Moment = 0.0 * 0.67 = 0.0 k-ft

- Passive Force = 0.0 kip Arm = 0.27 ft Moment = 0.0 k-ft

- Overturning moment X-X = 0.0 + 0.0 = 0.0 k-ft

- Resisting about X-X

- Footing weight = *0.6 * W * L * Thick * Density =* 0.6 * 2.50 * 2.50 * 8.0 / 12 * 0.15 = 0.4 kip

Arm = *W / 2 =* 2.50 / 2 = 1.25 ft Moment = 0.4 * 1.25 = 0.5 k-ft

- Pedestal weight = *0.6 * W * L * H * Density =* 0.6 * 6.0 / 12 * 6.0 / 12 * 0.0 * 0.15 = 0.0 kip

Arm = *W / 2 - Offset =* 2.50 / 2 - 0.0 / 12 = 1.25 ft Moment = 0.0 * 1.25 = 0.0 k-ft

- Soil cover = *0.6 * W * L * SC * Density =* 0.6 * (2.50 * 2.50 - 6.0 / 12 * 6.0 / 12) * 0.0 * 110 = 0.0 kip

Arm = *W / 2 =* 2.50 / 2 = 1.25 ft Moment = 0.0 * 1.25 = 0.0 k-ft

- Buoyancy = *0.6 * W * L * γ * (SC + Thick - WT) =* 0.6 * 2.50 * 2.50 * 62 * (0.67) = -0.2 kip

Arm = *W / 2 =* 2.50 / 2 = 1.25 ft Moment = 0.2 * 1.25 = -0.2 k-ft

- Axial force P = 0.6 * 4.1 + 0.6 * 0.0 = 2.5 kip

Arm = *W / 2 - Offset =* 2.50 / 2 - 0.0 / 12 = 1.25 ft Moment = 2.5 * 1.25 = 3.1 k-ft

- Resisting moment X-X = 0.5 + 0.0 + 0.0 + 3.1 + -0.2 = 3.3 k-ft

- Overturning safety factor X-X = $\dfrac{\textit{Resisting moment}}{\textit{Overturning moment}} = \dfrac{3.3}{0.0}$ = 33.49 > 1.50 OK

- Overturning about Z-Z

- Moment Mz = 0.6 * 0.0 + 0.6 * 0.0 = 0.0 k-ft

- Shear Force Vx = 0.6 * 0.0 + 0.6 * 0.0 = 0.0 kip

Arm = 0.00 + 8.0 / 12 = 0.67 ft Moment = 0.0 * 0.67 = 0.0 k-ft

- Passive Force = 0.0 kip Arm = 0.27 ft Moment = 0.0 k-ft

- Overturning moment Z-Z = 0.0 + 0.0 = 0.0 k-ft

- Resisting about Z-Z

- Footing weight = *0.6 * W * L * Thick * Density =* 0.6 * 2.50 * 2.50 * 8.0 / 12 * 0.15 = 0.4 kip

Arm = *L / 2 =* 2.50 / 2 = 1.25 ft Moment = 0.4 * 1.25 = 0.5 k-ft

- Pedestal weight = *0.6 * W * L * H * Density =* 0.6 * 6.0 / 12 * 6.0 / 12 * 0.0 * 0.15 = 0.0 kip

Arm = *L / 2 - Offset =* 2.50 / 2 - 0.0 / 12 = 1.25 ft Moment = 0.0 * 1.25 = 0.0 k-ft

- Soil cover = *0.6 * W * L * SC * Density =* 0.6 * (2.50 * 2.50 - 6.0 / 12 * 6.0 / 12) * 0.0 * 110 = 0.0 kip

Arm = *L / 2 =* 2.50 / 2 = 1.25 ft Moment = 0.0 * 1.25 = 0.0 k-ft

- Buoyancy = *0.6 * W * L * γ * (SC + Thick - WT) =* 0.6 * 2.50 * 2.50 * 62 * (0.67) = -0.2 kip

Arm = *L / 2 =* 2.50 / 2 = 1.25 ft Moment = 0.2 * 1.25 = -0.2 k-ft

- Axial force P = 0.6 * 4.1 + 0.6 * 0.0 = 2.5 kip

Arm = *L / 2 - Offset =* 2.50 / 2 - 0.0 / 12 = 1.25 ft Moment = 2.5 * 1.25 = 3.1 k-ft

- Resisting moment Z-Z = 0.5 + 0.0 + 0.0 + 3.1 + -0.2 = 3.3 k-ft

- Overturning safety factor Z-Z = $\frac{\textit{Resisting moment}}{\textit{Overturning moment}} = \frac{3.3}{0.0}$ = 33.49 > 1.50 OK

SOIL BEARING PRESSURES (Comb: D+L)

Overturning moment X-X = 0.0 + 0.0 = 0.0 k-ft

Resisting moment X-X = 0.8 + 0.0 + 0.0 + -0.3 + 11.6 = 12.1 k-ft

Overturning moment Z-Z = 0.0 + 0.0 = 0.0 k-ft

Resisting moment Z-Z = 0.8 + 0.0 + 0.0 + -0.3 + 11.6 = 12.1 k-ft

Resisting force = *Footing + Pedestal + Soil - Buoyancy + P =* 0.6 + 0.0 + 0.0 - 0.3 + 9.3 = 9.7 kip

X-coordinate of resultant from maximum bearing corner:

$$Xp = \frac{\textit{Z-Resisting moment - Z-Overturning moment}}{\textit{Resisting force}} = \frac{12.1 - 0.0}{9.7} = 1.25 \text{ ft}$$

Z-coordinate of resultant from maximum bearing corner:

$$Zp = \frac{\textit{X-Resisting moment - X-Overturning moment}}{\textit{Resisting force}} = \frac{12.1 - 0.0}{9.7} = 1.25 \text{ ft}$$

X-ecc = *Length / 2 - Xp =*2.50 / 2 - 1.25 = 0.00 ft

Z-ecc = *Width / 2 - Zp =* 2.50 / 2 - 1.25 = 0.00 ft

Area = *Width * Length =* 2.50 * 2.50 = 6.3 ft²

Sx = *Length * Width² / 6 =* 2.50 * 2.50² / 6 = 2.6 ft³

Sz = *Width * Length² / 6 =* 2.50 * 2.50² / 6 = 2.6 ft³

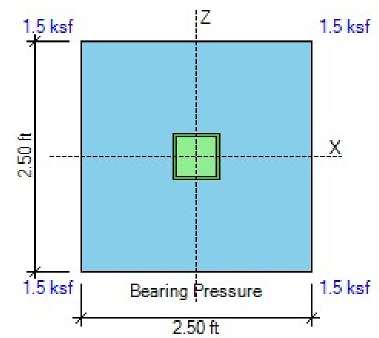
- Footing is in full bearing. Soil pressures are as follows:

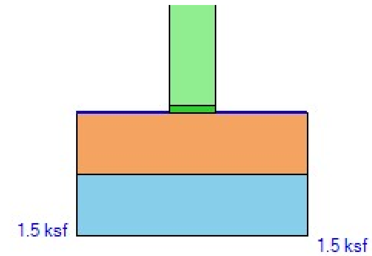
P1 = *P * (1/ A + Z-ecc / Sx + X-ecc / Sz) =* 9.7 * (1 / 6.3 + 0.00 / 2.6 + 0.00 / 2.6) = 1.55 ksf

P2 = *P * (1/ A - Z-ecc / Sx + X-ecc / Sz) =* 9.7 * (1 / 6.3 - 0.00 / 2.6 + 0.00 / 2.6) = 1.55 ksf

P3 = *P * (1/ A - Z-ecc / Sx - X-ecc / Sz) =* 9.7 * (1 / 6.3 - 0.00 / 2.6 - 0.00 / 2.6) = 1.55 ksf

P4 = *P * (1/ A + Z-ecc / Sx - X-ecc / Sz) =* 9.7 * (1 / 6.3 + 0.00 / 2.6 - 0.00 / 2.6) = 1.55 ksf





SLIDING CALCULATIONS (Comb: 0.6D+0.6W)

Internal friction angle = 28.0 deg

Passive coefficient *kp = 4.33* *(per Coulomb)*

Pressure at mid-depth = *kp * Density * (Cover + Thick / 2) =* 4.33 * 110 * (0.00 + 8.0 / 12 / 2) = 0.16 ksf

X-Passive force = *Pressure * Thick * Width =* 0.16 * 8.0 / 12 * 2.50 = 0.3 kip

Z-Passive force = *Pressure * Thick * Length =* 0.16 * 8.0 / 12 * 2.50 = 0.3 kip

Friction force = *Resisting force * Friction coeff. =* Max (0, 2.7 * 0.35) = 0.9 kip

Use 100% of Passive + 100% of Friction for sliding resistance

- Sliding safety factor X-X = $\dfrac{\text{X-Passive force + Friction}}{\text{X-Horizontal load}} = \dfrac{1.00 * 0.3 + 1.00 * 0.9}{0.0}$ = 12.02 > 1.50 OK

- Sliding safety factor Z-Z = $\dfrac{\text{Z-Passive force + Friction}}{\text{Z-Horizontal load}} = \dfrac{1.00 * 0.3 + 1.00 * 0.9}{0.0}$ = 12.02 > 1.50 OK

UPLIFT CALCULATIONS (Comb: 0.6D+0.6W)

- Uplift safety factor = $\dfrac{\text{Pedestal + Footing + Cover - Buoyancy}}{\text{Uplift load}} = \dfrac{0.0 + 0.4 + 0.0 - 0.2}{0.0}$ = 99.99 > 1.00 OK

ONE-WAY SHEAR CALCULATIONS (Comb: 1.2D+1.6L+0.5S)

Concrete f'c = 2.5 ksi Steel fy = 40.0 ksi Soil density = 110 pcf

Use Plain Concrete Shear Strength

ϕVcx = *4/3 * ϕ * √(f'c) * Width * t / 1000 =* 4/3 * 0.60 * √(2500) * 2.5 * 12 * 8.0 / 1000 = 9.6 kip ACI 14.5.5.1

ϕVcz = *4/3 * ϕ * √(f'c) * Length * t / 1000 =* 4/3 * 0.60 * √(2500) * 2.5 * 12 * 8.0 / 1000 = 9.6 kip

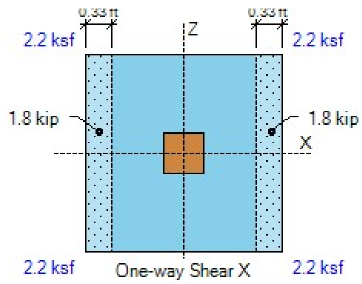
- Shear forces calculated as the volume of the bearing pressures under the effective areas:

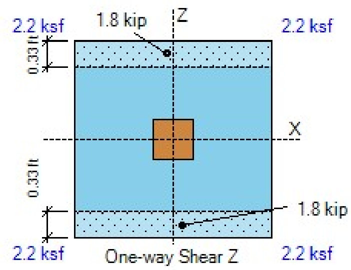
One-way shear Vux (- Side) = 1.8 kip < 9.6 kip OK

One-way shear Vux (+ Side) = 1.8 kip < 9.6 kip OK

One-way shear Vuz (- Side) = 1.8 kip < 9.6 kip OK

One-way shear Vuz (+ Side) = 1.8 kip < 9.6 kip OK





FLEXURE CALCULATIONS (Comb: 1.2D+1.6L+0.5S)

Plain ϕMnx = *5 * ϕ * √(f'c) * L * Thick² / 6* = 5 * 0.60 * √(2500) * 2.50 * 8.0² / 6 / 1000 = 1.1 k-ft ACI Eq. (14.5.2.1a)

Plain ϕMnz = *5 * ϕ * √(f'c) * W * Thick² / 6* = 5 * 0.60 * √(2500) * 2.50 * 8.0² / 6 / 1000 = 1.1 k-ft

- Top Bars

No Top Reinforcement Provided at the Footing

Use Plain Concrete Flexural Strength at Top

- Top moments calculated as the overburden minus the bearing pressures times the lever arm:

Top moment -Mux (- Side) = 0.0 k-ft < 4.0 k-ft OK

Top moment -Mux (+ Side) = 0.0 k-ft < 4.0 k-ft OK

Top moment -Muz (- Side) = 0.0 k-ft < 4.0 k-ft OK

Top moment -Muz (+ Side) = 0.0 k-ft < 4.0 k-ft OK

- Bottom Bars

No Bottom Reinforcement Provided at the Footing

Use Plain Concrete Flexural Strength at Bottom

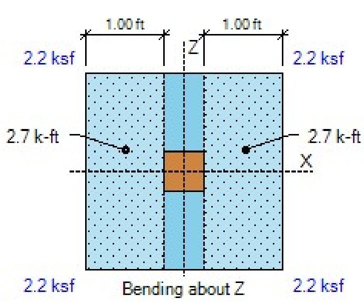
- Bottom moments calculated as the bearing minus the overburden pressures times the lever arm:

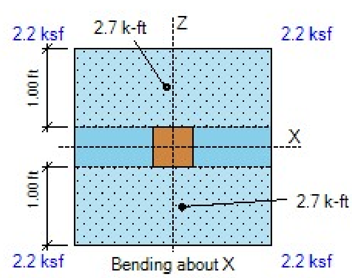
Bottom moment Mux (- Side) = 2.7 k-ft < 4.0 k-ft OK ratio = 0.66

Bottom moment Mux (+ Side) = 2.7 k-ft < 4.0 k-ft OK ratio = 0.66

Bottom moment Muz (- Side) = 2.7 k-ft < 4.0 k-ft OK ratio = 0.66

Bottom moment Muz (+ Side) = 2.7 k-ft < 4.0 k-ft OK ratio = 0.66





LOAD TRANSFER CALCULATIONS (Comb: 1.2D+1.6L+0.5S)

Area *A1 = col L * col W =* 6.0 * 6.0 = 36.0 in²

Sx = *col W * col L² / 6 =* 6.0 * 6.0² / 6 = 36.0 in³

Sz = *col L * col W² / 6 =* 6.0 * 6.0² / 6 = 36.0 in³

Bearing *Pbu = P / A1 + Mz / Sx + Mx / Sz =* 13.2 / 36.0 + 0.0 * 12 / 36.0 + 0.0 * 12 / 36.0 = 0.4 ksi

Min edge = *Min (L / 2 - X-offset - col L / 2, W / 2 - Z-offset - col W / 2)*

Min edge = Min (2.50 * 12 / 2 - 0.0 - 6.0 / 2, 2.50 * 12 / 2 - 0.0 - 6.0 / 2 = 12.0 in

Area *A2 = Min [L * W, (col L + 2 * Min edge) * (col W + 2 * Min edge)]* ACI R22.8.3.2

A2 = Min [2.50 * 12 * 2.5 * 12, (6.0 + 2 * 12.0) * (6.0 + 2 * 12.0)] = 900.0 in²

Footing *ϕPnc = ϕ * 0.85 * f'c * Min [2, √(A2 / A1)] =* 0.65 * 0.85 * 2.5 * Min [2, √(900.0 / 36.0)] = 2.8 ksi

Footing *ϕPns = ϕ * As * Fy / A1 =* 0.0 ksi ACI 22.8.3.2

Footing bearing *ϕPn = ϕPnc + ϕPns =* 2.8 + 0.0 = 2.8 ksi > 0.4 psi OK

Hooked *Ldh = Max (8 db, 6, 0.02 * fy / (f'c)½ * Confining * Location * Concrete * db * ratio)* ACI 25.4.3

Ldh = Max (8 db, 6, 0.02 * 60.0 * 1000 / (2500)½ * 1.0 * 0.7 * 0.0 * 0.75 * 0.06) = 6.0 in

Ld provided = *Dowel length =* 3.00 * 12 = 36.0 in > 12.0 in OK

Ldh provided = *Footing thickness - Cover =* 8.00 - 3.0 = 5.0 in < 6.0 in NG

PUNCHING SHEAR CALCULATIONS (Comb: 1.2D+1.6L+0.5Lr)

X-Edge = *Length / 2 - Offset - Col / 2 =* 2.50 * 12 / 2 - 0.0 - 6.0 / 2 = 12.0 in αsx = 10

Z-Edge = *Width / 2 - Offset - Col / 2 =* 2.50 * 12 / 2 - 0.0 - 6.0 / 2 = 12.0 in αsz = 10

αs = *αsx + αsz =* 10 + 10 = 20 Col type = Corner β = *L / W =* 6.0 / 6.0 = 1.00 ACI 22.6.5.2

Perimeter *bo = αsz / 10 * (L + d / 2 + X-Edge) + αsx / 10 * (W + d / 2 + Z-Edge)* ACI 22.6.4.2

bo = 10 / 10 * (6.0 + 8.0 / 2 + 12.0) + 10 / 10 * (6.0 + 8.0 / 2 + 12.0) = 44.0 in

Area *Abo = (L + d / 2 + X-Edge) * (W + d / 2 + Z-Edge) =* (6.0 + 8.0 / 2 + 12.0) * (6.0 + 8.0 / 2 + 12.0) = 484.0 in²

Use Plain Concrete Shear Strength

ϕVc = *ϕ * Min (1 + 2 / β, 2) * 4/3 * √(f'c)* ACI 14.5.5.1

ϕVc = 0.60 * Min (1 + 2 / 1.00, 2) * 4/3 √(2500) = 80.0 psi

Punching force *F = P + Overburden * Abo - Bearing*

F = 13.2 + 0.07 * 484.0 / 144 - 3.0 = 10.5 kip

b1 = *L + d / 2 + X-Edge =* 6.0 + 8.0 / 2 + 12.0 = 22.0 in b2 = *W + d / 2 + Z-Edge =* 6.0 + 8.0 / 2 + 12.0 = 22.0 in

$$\gamma vx \text{ factor} = 1 - \frac{1}{1 + (2/3)\sqrt{(b2/b1)}} = 1 - \frac{1}{1 + (2/3)\sqrt{(22.0/22.0)}} = 0.40$$

ACI Eq. (8.4.4.2.2)

ACI Eq. (8.4.2.3.2)

$$\gamma vz \text{ factor} = 1 - \frac{1}{1 + (2/3)\sqrt{(b1/b2)}} = 1 - \frac{1}{1 + (2/3)\sqrt{(22.0/22.0)}} = 0.40$$

X2z = *b1² / 2 / (b1 + b2) =* 22.0² / 2 / (22.0 + 22.0) = 5.5 in X2x = *b2² / 2 / (b2 + b1) =* 5.5 in

Jcz = *b1 * d³ / 12 + b1³ * d / 12 + b1 * d * (b1 / 2 - X2z)² + b2 * d * X2z²* ACI R8.4.4.2.3

Jcz = 22.0 * 8.0³ / 12 + 22.0³ * 8.0 / 12 + 22.0 * 8.0 * (22.0 / 2 * 5.5)² + 22.0 * 8.0 * 5.5² = 18685 in⁴

Jcx = *b2 * d³ / 12 + b2³ * d / 12 + b2 * d * (b2 / 2 - X2x)² + b1 * d * X2x²* ACI R8.4.4.2.3

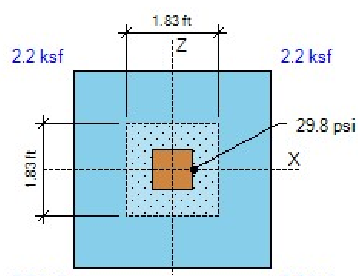
Jcz = 22.0 * 8.0³ / 12 + 22.0³ * 8.0 / 12 + 22.0 * 8.0 * (22.0 / 2 * 5.5)² + 22.0 * 8.0 * 5.5² = 18685 in⁴

Stress due to P = *F / (bo * d) * 1000 =* 10.5 / (44.0 * 8.0) * 1000 = 29.8 psi

Stress due to Mx = *γvx * X-OTM * X2x / Jcx =* 0.40 * 0.0 * 12 * 5.5 / 18685 * 1000 = 0.0 psi

Stress due to Mz = *γvz * Z-OTM * X2z / Jcz =* 0.40 * 0.0 * 12 * 5.5 / 18685 * 1000 = 0.0 psi

Punching stress = *P-stress + Mx-stress + Mz-stress =* 29.8 + 0.0 + 0.0 = 29.8 psi < 80.0 psi OK



Punching Shear

LOAD TRANSFER CALCULATIONS (Comb: 1.2D+1.6L+0.5S)

Area *A1 = col L * col W* = 6.0 * 6.0 = 36.0 in²

Sx = *col W * col L² / 6* = 6.0 * 6.0² / 6 = 36.0 in³

Sz = *col L * col W² / 6* = 6.0 * 6.0² / 6 = 36.0 in³

Bearing *Pbu = P / A1 + Mz / Sx + Mx / Sz* = 13.2 / 36.0 + 0.0 * 12 / 36.0 + 0.0 * 12 / 36.0 = 0.4 ksi

Min edge = *Min (L / 2 - X-offset - col L / 2, W / 2 - Z-offset - col W / 2)*

Min edge = Min (2.50 * 12 / 2 - 0.0 - 6.0 / 2, 2.50 * 12 / 2 - 0.0 - 6.0 / 2 = 12.0 in

Area *A2 = Min [L * W, (col L + 2 * Min edge) * (col W + 2 * Min edge)]* ACI R22.8.3.2

A2 = Min [2.50 * 12 * 2.5 * 12, (6.0 + 2 * 12.0) * (6.0 + 2 * 12.0)] = 900.0 in²

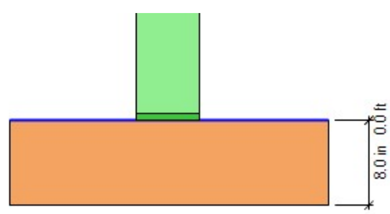
Footing *ϕPnc = ϕ * 0.85 * f'c * Min [2, √(A2 / A1)]* = 0.65 * 0.85 * 2.5 * Min [2, √(900.0 / 36.0)] = 2.8 ksi

Footing *ϕPns = ϕ * As * Fy / A1* = 0.0 ksi ACI 22.8.3.2

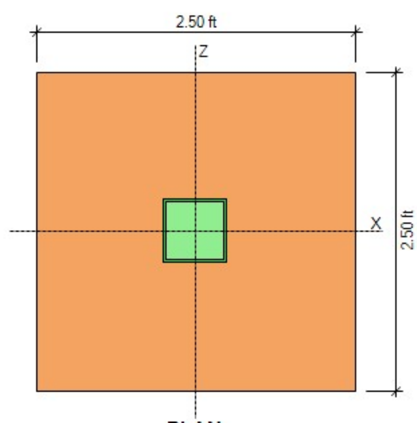
Footing bearing *ϕPn = ϕPnc + ϕPns* = 2.8 + 0.0 = 2.8 ksi > 0.4 psi OK

DESIGN CODES

Concrete Design	ACI 318-14
Load Combinations	ASCE 7-10/16



ELEVATION



PLAN

ASDIP Foundation 5.6.0.4 **SPREAD FOOTING DESIGN** www.asdipsoft.com

GEOMETRY			
Footing Length (X-dir)	2.50	ft	
Footing Width (Z-dir)	2.50	ft	
Footing Thickness	8.0	in	OK
Soil Cover	0.00	ft	
Column Length (X-dir)	6.0	in	
Column Width (Z-dir)	6.0	in	
Offset (X-dir)	0.00	in	OK
Offset (Z-dir)	0.00	in	OK
Base Plate (L x W)	6.0 x 6.0	in	

SOIL PRESSURES (D+L)			
Gross Allow. Soil Pressure	2.0	ksf	
Soil Pressure at Corner 1	1.5	ksf	
Soil Pressure at Corner 2	1.5	ksf	
Soil Pressure at Corner 3	1.5	ksf	
Soil Pressure at Corner 4	1.5	ksf	
Bearing Pressure Ratio	0.74		OK
Ftg. Area in Contact with Soil	100.0	%	
X-eccentricity / Ftg. Length	0.00		OK
Z-eccentricity / Ftg. Width	0.00		OK

APPLIED LOADS

	Dead	Live	RLive	Snow	Wind	Seismic	
Axial Force P	3.9	5.0	0.0	0.0	0.0	0.0	kip
Moment about X Mx ..	0.0	0.0	0.0	0.0	0.0	0.0	k-ft
Moment about Z Mz ..	0.0	0.0	0.0	0.0	0.0	0.0	k-ft
Shear Force Vx	0.0	0.0	0.0	0.0	0.0	0.0	kip
Shear Force Vz	0.0	0.0	0.0	0.0	0.0	0.0	kip

OVERTURNING CALCULATIONS (Comb: 0.6D+0.6W)

- Overturning about X-X
- Moment Mx = 0.6 * 0.0 + 0.6 * 0.0 = 0.0 k-ft
- Shear Force Vz = 0.6 * 0.0 + 0.6 * 0.0 = 0.0 kip
  Arm = 0.00 + 8.0 / 12 = 0.67 ft   Moment = 0.0 * 0.67 = 0.0 k-ft
- Passive Force = 0.0 kip   Arm = 0.27 ft   Moment = 0.0 k-ft
- Overturning moment X-X = 0.0 + 0.0 = 0.0 k-ft

- Resisting about X-X
- Footing weight = *0.6 * W * L * Thick * Density =* 0.6 * 2.50 * 2.50 * 8.0 / 12 * 0.15 = 0.4 kip
  Arm = *W / 2 =* 2.50 / 2 = 1.25 ft   Moment = 0.4 * 1.25 = 0.5 k-ft
- Pedestal weight = *0.6 * W * L * H * Density =* 0.6 * 6.0 / 12 * 6.0 / 12 * 0.0 * 0.15 = 0.0 kip
  Arm = *W / 2 - Offset =* 2.50 / 2 - 0.0 / 12 = 1.25 ft   Moment = 0.0 * 1.25 = 0.0 k-ft
- Soil cover = *0.6 * W * L * SC * Density =* 0.6 * (2.50 * 2.50 - 6.0 / 12 * 6.0 / 12) * 0.0 * 110 = 0.0 kip
  Arm = *W / 2 =* 2.50 / 2 = 1.25 ft   Moment = 0.0 * 1.25 = 0.0 k-ft
- Buoyancy = *0.6 * W * L * γ * (SC + Thick - WT) =* 0.6 * 2.50 * 2.50 * 62 * (0.67) = -0.2 kip
  Arm = *W / 2 =* 2.50 / 2 = 1.25 ft   Moment = 0.2 * 1.25 = -0.2 k-ft
- Axial force P = 0.6 * 3.9 + 0.6 * 0.0 = 2.3 kip
  Arm = *W / 2 - Offset =* 2.50 / 2 - 0.0 / 12 = 1.25 ft   Moment = 2.3 * 1.25 = 2.9 k-ft
- Resisting moment X-X = 0.5 + 0.0 + 0.0 + 2.9 + -0.2 = 3.2 k-ft

- Overturning safety factor X-X = $\dfrac{\textit{Resisting moment}}{\textit{Overturning moment}} = \dfrac{3.2}{0.0}$ = 31.99 > 1.50 OK

- Overturning about Z-Z

- Moment Mz = 0.6 * 0.0 + 0.6 * 0.0 = 0.0 k-ft

- Shear Force Vx = 0.6 * 0.0 + 0.6 * 0.0 = 0.0 kip

  Arm = 0.00 + 8.0 / 12 = 0.67 ft   Moment = 0.0 * 0.67 = 0.0 k-ft

- Passive Force = 0.0 kip   Arm = 0.27 ft   Moment = 0.0 k-ft

- Overturning moment Z-Z = 0.0 + 0.0 = 0.0 k-ft

- Resisting about Z-Z

- Footing weight = *0.6 * W * L * Thick * Density =* 0.6 * 2.50 * 2.50 * 8.0 / 12 * 0.15 = 0.4 kip

  Arm = *L / 2 =* 2.50 / 2 = 1.25 ft   Moment = 0.4 * 1.25 = 0.5 k-ft

- Pedestal weight = *0.6 * W * L * H * Density =* 0.6 * 6.0 / 12 * 6.0 / 12 * 0.0 * 0.15 = 0.0 kip

  Arm = *L / 2 - Offset =* 2.50 / 2 - 0.0 / 12 = 1.25 ft   Moment = 0.0 * 1.25 = 0.0 k-ft

- Soil cover = *0.6 * W * L * SC * Density =* 0.6 * (2.50 * 2.50 - 6.0 / 12 * 6.0 / 12) * 0.0 * 110 = 0.0 kip

  Arm = *L / 2 =* 2.50 / 2 = 1.25 ft   Moment = 0.0 * 1.25 = 0.0 k-ft

- Buoyancy = *0.6 * W * L * γ * (SC + Thick - WT) =* 0.6 * 2.50 * 2.50 * 62 * (0.67) = -0.2 kip

  Arm = *L / 2 =* 2.50 / 2 = 1.25 ft   Moment = 0.2 * 1.25 = -0.2 k-ft

- Axial force P = 0.6 * 3.9 + 0.6 * 0.0 = 2.3 kip

  Arm = *L / 2 - Offset =* 2.50 / 2 - 0.0 / 12 = 1.25 ft   Moment = 2.3 * 1.25 = 2.9 k-ft

- Resisting moment Z-Z = 0.5 + 0.0 + 0.0 + 2.9 + -0.2 = 3.2 k-ft

- Overturning safety factor Z-Z = $\dfrac{\textit{Resisting moment}}{\textit{Overturning moment}} = \dfrac{3.2}{0.0}$ = 31.99 > 1.50 OK

SOIL BEARING PRESSURES (Comb: D+L)

Overturning moment X-X = 0.0 + 0.0 = 0.0 k-ft

Resisting moment X-X = 0.8 + 0.0 + 0.0 + -0.3 + 11.1 = 11.6 k-ft

Overturning moment Z-Z = 0.0 + 0.0 = 0.0 k-ft

Resisting moment Z-Z = 0.8 + 0.0 + 0.0 + -0.3 + 11.1 = 11.6 k-ft

Resisting force = *Footing + Pedestal + Soil - Buoyancy + P =* 0.6 + 0.0 + 0.0 - 0.3 + 8.9 = 9.3 kip

X-coordinate of resultant from maximum bearing corner:

$$Xp = \frac{\textit{Z-Resisting moment - Z-Overturning moment}}{\textit{Resisting force}} = \frac{11.6 - 0.0}{9.3} = 1.25 \text{ ft}$$

Z-coordinate of resultant from maximum bearing corner:

$$Zp = \frac{\textit{X-Resisting moment - X-Overturning moment}}{\textit{Resisting force}} = \frac{11.6 - 0.0}{9.3} = 1.25 \text{ ft}$$

X-ecc = *Length / 2 - Xp =* 2.50 / 2 - 1.25 = 0.00 ft

Z-ecc = *Width / 2 - Zp =* 2.50 / 2 - 1.25 = 0.00 ft

Area = *Width * Length =* 2.50 * 2.50 = 6.3 ft²

Sx = *Length * Width² / 6 =* 2.50 * 2.50² / 6 = 2.6 ft³

Sz = *Width * Length² / 6 =* 2.50 * 2.50² / 6 = 2.6 ft³

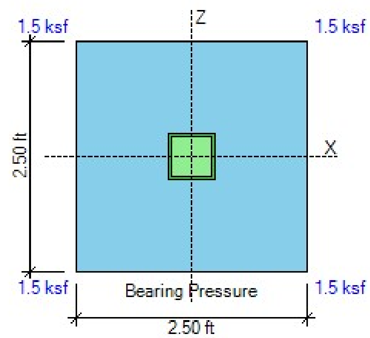
- Footing is in full bearing. Soil pressures are as follows:

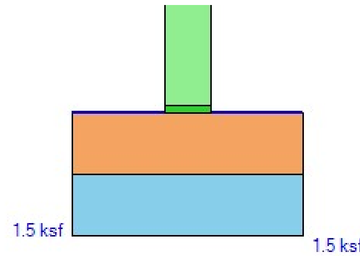
P1 = *P * (1/ A + Z-ecc / Sx + X-ecc / Sz) =* 9.3 * (1 / 6.3 + 0.00 / 2.6 + 0.00 / 2.6) = 1.48 ksf

P2 = *P * (1/ A - Z-ecc / Sx + X-ecc / Sz) =* 9.3 * (1 / 6.3 - 0.00 / 2.6 + 0.00 / 2.6) = 1.48 ksf

P3 = *P * (1/ A - Z-ecc / Sx - X-ecc / Sz) =* 9.3 * (1 / 6.3 - 0.00 / 2.6 - 0.00 / 2.6) = 1.48 ksf

P4 = *P * (1/ A + Z-ecc / Sx - X-ecc / Sz) =* 9.3 * (1 / 6.3 + 0.00 / 2.6 - 0.00 / 2.6) = 1.48 ksf





SLIDING CALCULATIONS (Comb: 0.6D+0.6W)

Internal friction angle = 28.0 deg

Passive coefficient *kp = 4.33* *(per Coulomb)*

Pressure at mid-depth = *kp * Density * (Cover + Thick / 2) =* 4.33 * 110 * (0.00 + 8.0 / 12 / 2) = 0.16 ksf

X-Passive force = *Pressure * Thick * Width =* 0.16 * 8.0 / 12 * 2.50 = 0.3 kip

Z-Passive force = *Pressure * Thick * Length =* 0.16 * 8.0 / 12 * 2.50 = 0.3 kip

Friction force = *Resisting force * Friction coeff. =* Max (0, 2.6 * 0.35) = 0.9 kip

Use 100% of Passive + 100% of Friction for sliding resistance

- Sliding safety factor X-X = $\dfrac{\text{X-Passive force + Friction}}{\text{X-Horizontal load}} = \dfrac{1.00 * 0.3 + 1.00 * 0.9}{0.0}$ = 11.60 > 1.50 OK

- Sliding safety factor Z-Z = $\dfrac{\text{Z-Passive force + Friction}}{\text{Z-Horizontal load}} = \dfrac{1.00 * 0.3 + 1.00 * 0.9}{0.0}$ = 11.60 > 1.50 OK

UPLIFT CALCULATIONS (Comb: 0.6D+0.6W)

- Uplift safety factor = $\dfrac{\text{Pedestal + Footing + Cover - Buoyancy}}{\text{Uplift load}} = \dfrac{0.0 + 0.4 + 0.0 - 0.2}{0.0}$ = 99.99 > 1.00 OK

ONE-WAY SHEAR CALCULATIONS (Comb: 1.2D+1.6L+0.5S)

Concrete f'c = 2.5 ksi | Steel fy = 40.0 ksi | Soil density = 110 pcf

Use Plain Concrete Shear Strength

ϕVcx = *4/3 * ϕ * √(f'c) * Width * t / 1000 =* 4/3 * 0.60 * √(2500) * 2.5 * 12 * 8.0 / 1000 = 9.6 kip ACI 14.5.5.1

ϕVcz = *4/3 * ϕ * √(f'c) * Length * t / 1000 =* 4/3 * 0.60 * √(2500) * 2.5 * 12 * 8.0 / 1000 = 9.6 kip

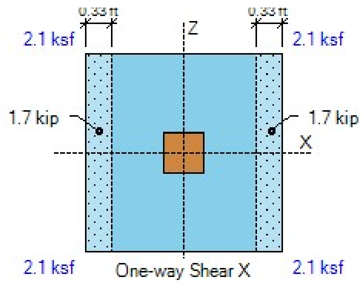
- Shear forces calculated as the volume of the bearing pressures under the effective areas:

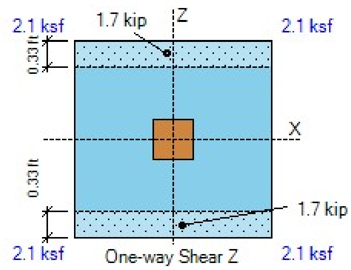
One-way shear Vux (- Side) = 1.7 kip < 9.6 kip OK

One-way shear Vux (+ Side) = 1.7 kip < 9.6 kip OK

One-way shear Vuz (- Side) = 1.7 kip < 9.6 kip OK

One-way shear Vuz (+ Side) = 1.7 kip < 9.6 kip OK





FLEXURE CALCULATIONS (Comb: 1.2D+1.6L+0.5S)

Plain ϕMnx = *5 * ϕ * √(f'c) * L * Thick² / 6* = 5 * 0.60 * √(2500) * 2.50 * 8.0² / 6 / 1000 = 1.1 k-ft ACI Eq. (14.5.2.1a)

Plain ϕMnz = *5 * ϕ * √(f'c) * W * Thick² / 6* = 5 * 0.60 * √(2500) * 2.50 * 8.0² / 6 / 1000 = 1.1 k-ft

- Top Bars

No Top Reinforcement Provided at the Footing

Use Plain Concrete Flexural Strength at Top

- Top moments calculated as the overburden minus the bearing pressures times the lever arm:

Top moment -Mux (- Side) = 0.0 k-ft < 4.0 k-ft OK

Top moment -Mux (+ Side) = 0.0 k-ft < 4.0 k-ft OK

Top moment -Muz (- Side) = 0.0 k-ft < 4.0 k-ft OK

Top moment -Muz (+ Side) = 0.0 k-ft < 4.0 k-ft OK

- Bottom Bars

No Bottom Reinforcement Provided at the Footing

Use Plain Concrete Flexural Strength at Bottom

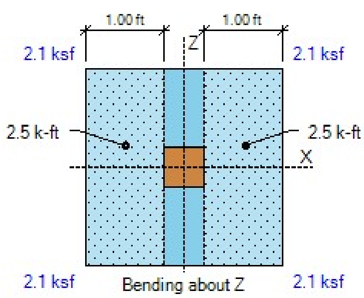
- Bottom moments calculated as the bearing minus the overburden pressures times the lever arm:

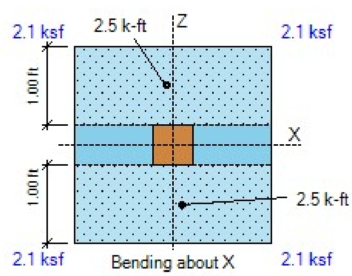
Bottom moment Mux (- Side) = 2.5 k-ft < 4.0 k-ft OK ratio = 0.64

Bottom moment Mux (+ Side) = 2.5 k-ft < 4.0 k-ft OK ratio = 0.64

Bottom moment Muz (- Side) = 2.5 k-ft < 4.0 k-ft OK ratio = 0.64

Bottom moment Muz (+ Side) = 2.5 k-ft < 4.0 k-ft OK ratio = 0.64





LOAD TRANSFER CALCULATIONS (Comb: 1.2D+1.6L+0.5S)

Area *A1 = col L * col W =* 6.0 * 6.0 = 36.0 in²

Sx = *col W * col L² / 6 =* 6.0 * 6.0² / 6 = 36.0 in³

Sz = *col L * col W² / 6 =* 6.0 * 6.0² / 6 = 36.0 in³

Bearing *Pbu = P / A1 + Mz / Sx + Mx / Sz =* 12.7 / 36.0 + 0.0 * 12 / 36.0 + 0.0 * 12 / 36.0 = 0.4 ksi

Min edge = *Min (L / 2 - X-offset - col L / 2, W / 2 - Z-offset - col W / 2)*

Min edge = Min (2.50 * 12 / 2 - 0.0 - 6.0 / 2, 2.50 * 12 / 2 - 0.0 - 6.0 / 2 = 12.0 in

Area *A2 = Min [L * W, (col L + 2 * Min edge) * (col W + 2 * Min edge)]* ACI R22.8.3.2

A2 = Min [2.50 * 12 * 2.5 * 12, (6.0 + 2 * 12.0) * (6.0 + 2 * 12.0)] = 900.0 in²

Footing *ϕPnc = ϕ * 0.85 * f'c * Min [2, √(A2 / A1)] =* 0.65 * 0.85 * 2.5 * Min [2, √(900.0 / 36.0)] = 2.8 ksi

Footing *ϕPns = ϕ * As * Fy / A1 =* 0.0 ksi ACI 22.8.3.2

Footing bearing *ϕPn = ϕPnc + ϕPns =* 2.8 + 0.0 = 2.8 ksi > 0.4 psi OK

Hooked *Ldh = Max (8 db, 6, 0.02 * fy / (f'c)½ * Confining * Location * Concrete * db * ratio)* ACI 25.4.3

Ldh = Max (8 db, 6, 0.02 * 60.0 * 1000 / (2500)½ * 1.0 * 0.7 * 0.0 * 0.75 * 0.06) = 6.0 in

Ld provided = *Dowel length =* 3.00 * 12 = 36.0 in > 12.0 in OK

Ldh provided = *Footing thickness - Cover =* 8.00 - 3.0 = 5.0 in < 6.0 in NG

PUNCHING SHEAR CALCULATIONS (Comb: 1.2D+1.6L+0.5Lr)

X-Edge = *Length / 2 - Offset - Col / 2 =* 2.50 * 12 / 2 - 0.0 - 6.0 / 2 = 12.0 in $\alpha sx = 10$

Z-Edge = *Width / 2 - Offset - Col / 2 =* 2.50 * 12 / 2 - 0.0 - 6.0 / 2 = 12.0 in $\alpha sz = 10$

αs = *αsx + αsz =* 10 + 10 = 20 Col type = Corner β = *L / W =* 6.0 / 6.0 = 1.00 ACI 22.6.5.2

Perimeter *bo = αsz / 10 * (L + d / 2 + X-Edge) + αsx / 10 * (W + d / 2 + Z-Edge)* ACI 22.6.4.2

bo = 10 / 10 * (6.0 + 8.0 / 2 + 12.0) + 10 / 10 * (6.0 + 8.0 / 2 + 12.0) = 44.0 in

Area *Abo = (L + d / 2 + X-Edge) * (W + d / 2 + Z-Edge) =* (6.0 + 8.0 / 2 + 12.0) * (6.0 + 8.0 / 2 + 12.0) = 484.0 in²

Use Plain Concrete Shear Strength

ϕVc = *ϕ * Min (1 + 2 / β, 2) * 4/3 * √(f'c)* ACI 14.5.5.1

ϕVc = 0.60 * Min (1 + 2 / 1.00, 2) * 4/3 √(2500) = 80.0 psi

Punching force *F = P + Overburden * Abo - Bearing*

F = 12.7 + 0.07 * 484.0 / 144 - 2.9 = 10.1 kip

b1 = *L + d / 2 + X-Edge =* 6.0 + 8.0 / 2 + 12.0 = 22.0 in b2 = *W + d / 2 + Z-Edge =* 6.0 + 8.0 / 2 + 12.0 = 22.0 in

$$\gamma vx \text{ factor} = 1 - \frac{1}{1 + (2/3)\sqrt{b2 / b1}} = 1 - \frac{1}{1 + (2/3)\sqrt{22.0 / 22.0}} = 0.40$$

ACI Eq. (8.4.4.2.2)

ACI Eq. (8.4.2.3.2)

$$\gamma vz \text{ factor} = 1 - \frac{1}{1 + (2/3)\sqrt{b1 / b2}} = 1 - \frac{1}{1 + (2/3)\sqrt{22.0 / 22.0}} = 0.40$$

X2z = *b1² / 2 / (b1 + b2) =* 22.0² / 2 / (22.0 + 22.0) = 5.5 in X2x = *b2² / 2 / (b2 + b1) =* 5.5 in

Jcz = *b1 * d³ / 12 + b1³ * d / 12 + b1 * d * (b1 / 2 - X2z)² + b2 * d * X2z²* ACI R8.4.4.2.3

Jcz = 22.0 * 8.0³ / 12 + 22.0³ * 8.0 / 12 + 22.0 * 8.0 * (22.0 / 2 * 5.5)² + 22.0 * 8.0 * 5.5² = 18685 in⁴

Jcx = *b2 * d³ / 12 + b2³ * d / 12 + b2 * d * (b2 / 2 - X2x)² + b1 * d * X2x²* ACI R8.4.4.2.3

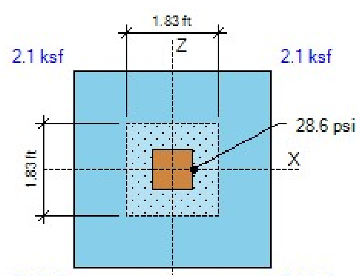
Jcz = 22.0 * 8.0³ / 12 + 22.0³ * 8.0 / 12 + 22.0 * 8.0 * (22.0 / 2 * 5.5)² + 22.0 * 8.0 * 5.5² = 18685 in⁴

Stress due to P = *F / (bo * d) * 1000 =* 10.1 / (44.0 * 8.0) * 1000 = 28.6 psi

Stress due to Mx = *γvx * X-OTM * X2x / Jcx =* 0.40 * 0.0 * 12 * 5.5 / 18685 * 1000 = 0.0 psi

Stress due to Mz = *γvz * Z-OTM * X2z / Jcz =* 0.40 * 0.0 * 12 * 5.5 / 18685 * 1000 = 0.0 psi

Punching stress = *P-stress + Mx-stress + Mz-stress =* 28.6 + 0.0 + 0.0 = 28.6 psi < 80.0 psi OK



Punching Shear

LOAD TRANSFER CALCULATIONS (Comb: 1.2D+1.6L+0.5S)

Area *A1 = col L * col W* = 6.0 * 6.0 = 36.0 in²

Sx = *col W * col L² / 6* = 6.0 * 6.0² / 6 = 36.0 in³

Sz = *col L * col W² / 6* = 6.0 * 6.0² / 6 = 36.0 in³

Bearing *Pbu = P / A1 + Mz / Sx + Mx / Sz* = 12.7 / 36.0 + 0.0 * 12 / 36.0 + 0.0 * 12 / 36.0 = 0.4 ksi

Min edge = *Min (L / 2 - X-offset - col L / 2, W / 2 - Z-offset - col W / 2)*

Min edge = Min (2.50 * 12 / 2 - 0.0 - 6.0 / 2, 2.50 * 12 / 2 - 0.0 - 6.0 / 2 = 12.0 in

Area *A2 = Min [L * W, (col L + 2 * Min edge) * (col W + 2 * Min edge)]* ACI R22.8.3.2

A2 = Min [2.50 * 12 * 2.5 * 12, (6.0 + 2 * 12.0) * (6.0 + 2 * 12.0)] = 900.0 in²

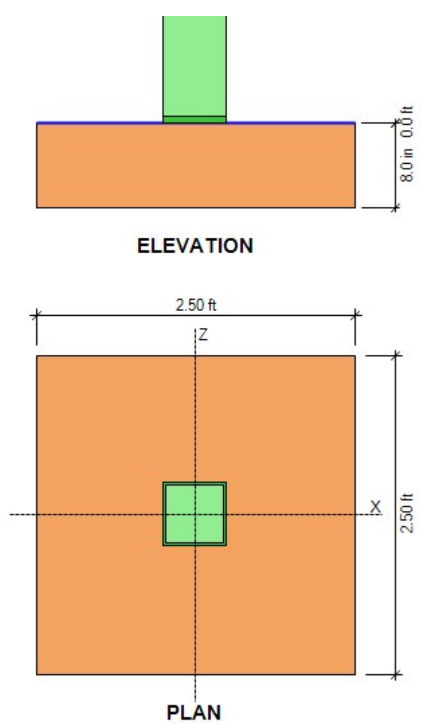
Footing *ϕPnc = ϕ * 0.85 * f'c * Min [2, √(A2 / A1)]* = 0.65 * 0.85 * 2.5 * Min [2, √(900.0 / 36.0)] = 2.8 ksi

Footing *ϕPns = ϕ * As * Fy / A1* = 0.0 ksi ACI 22.8.3.2

Footing bearing *ϕPn = ϕPnc + ϕPns* = 2.8 + 0.0 = 2.8 ksi > 0.4 psi OK

DESIGN CODES

Concrete Design	ACI 318-14
Load Combinations	ASCE 7-10/16



ASDIP Foundation 5.6.0.4 **SPREAD FOOTING DESIGN** www.asdipsoft.com

GEOMETRY

Footing Length (X-dir)	3.50	ft	
Footing Width (Z-dir)	3.50	ft	
Footing Thickness	8.0	in	OK
Soil Cover	0.00	ft	
Column Length (X-dir)	6.0	in	
Column Width (Z-dir)	6.0	in	
Offset (X-dir)	0.00	in	OK
Offset (Z-dir)	0.00	in	OK
Base Plate (L x W)	6.0 x 6.0	in	

SOIL PRESSURES (D+L)

Gross Allow. Soil Pressure	2.0	ksf	
Soil Pressure at Corner 1	1.6	ksf	
Soil Pressure at Corner 2	1.6	ksf	
Soil Pressure at Corner 3	1.6	ksf	
Soil Pressure at Corner 4	1.6	ksf	
Bearing Pressure Ratio	0.79		OK
Ftg. Area in Contact with Soil	100.0	%	
X-eccentricity / Ftg. Length	0.00		OK
Z-eccentricity / Ftg. Width	0.00		OK

APPLIED LOADS

	Dead	Live	RLive	Snow	Wind	Seismic	
Axial Force P	4.6	14.1	0.0	0.0	0.0	0.0	kip
Moment about X Mx ..	0.0	0.0	0.0	0.0	0.0	0.0	k-ft
Moment about Z Mz ..	0.0	0.0	0.0	0.0	0.0	0.0	k-ft
Shear Force Vx	0.0	0.0	0.0	0.0	0.0	0.0	kip
Shear Force Vz	0.0	0.0	0.0	0.0	0.0	0.0	kip

OVERTURNING CALCULATIONS (Comb: 0.6D+0.6W)

- Overturning about X-X

- Moment Mx = 0.6 * 0.0 + 0.6 * 0.0 = 0.0 k-ft
- Shear Force Vz = 0.6 * 0.0 + 0.6 * 0.0 = 0.0 kip
  Arm = 0.00 + 8.0 / 12 = 0.67 ft   Moment = 0.0 * 0.67 = 0.0 k-ft
- Passive Force = 0.0 kip   Arm = 0.27 ft   Moment = 0.0 k-ft
- Overturning moment X-X = 0.0 + 0.0 = 0.0 k-ft

- Resisting about X-X

- Footing weight = *0.6 * W * L * Thick * Density =* 0.6 * 3.50 * 3.50 * 8.0 / 12 * 0.15 = 0.7 kip
  Arm = *W / 2 =* 3.50 / 2 = 1.75 ft   Moment = 0.7 * 1.75 = 1.3 k-ft
- Pedestal weight = *0.6 * W * L * H * Density =* 0.6 * 6.0 / 12 * 6.0 / 12 * 0.0 * 0.15 = 0.0 kip
  Arm = *W / 2 - Offset =* 3.50 / 2 - 0.0 / 12 = 1.75 ft   Moment = 0.0 * 1.75 = 0.0 k-ft
- Soil cover = *0.6 * W * L * SC * Density =* 0.6 * (3.50 * 3.50 - 6.0 / 12 * 6.0 / 12) * 0.0 * 110 = 0.0 kip
  Arm = *W / 2 =* 3.50 / 2 = 1.75 ft   Moment = 0.0 * 1.75 = 0.0 k-ft
- Buoyancy = *0.6 * W * L * γ * (SC + Thick - WT) =* 0.6 * 3.50 * 3.50 * 62 * (0.67) = -0.3 kip
  Arm = *W / 2 =* 3.50 / 2 = 1.75 ft   Moment = 0.3 * 1.75 = -0.5 k-ft
- Axial force P = 0.6 * 4.6 + 0.6 * 0.0 = 2.8 kip
  Arm = *W / 2 - Offset =* 3.50 / 2 - 0.0 / 12 = 1.75 ft   Moment = 2.8 * 1.75 = 4.8 k-ft
- Resisting moment X-X = 1.3 + 0.0 + 0.0 + 4.8 + -0.5 = 5.6 k-ft

- Overturning safety factor X-X = $\dfrac{\textit{Resisting moment}}{\textit{Overturning moment}} = \dfrac{5.6}{0.0}$ = 55.81 > 1.50 OK

- Overturning about Z-Z

- Moment Mz = 0.6 * 0.0 + 0.6 * 0.0 = 0.0 k-ft

- Shear Force Vx = 0.6 * 0.0 + 0.6 * 0.0 = 0.0 kip

  Arm = 0.00 + 8.0 / 12 = 0.67 ft   Moment = 0.0 * 0.67 = 0.0 k-ft

- Passive Force = 0.0 kip   Arm = 0.27 ft   Moment = 0.0 k-ft

- Overturning moment Z-Z = 0.0 + 0.0 = 0.0 k-ft

- Resisting about Z-Z

- Footing weight = *0.6 * W * L * Thick * Density =* 0.6 * 3.50 * 3.50 * 8.0 / 12 * 0.15 = 0.7 kip

  Arm = *L / 2 =* 3.50 / 2 = 1.75 ft   Moment = 0.7 * 1.75 = 1.3 k-ft

- Pedestal weight = *0.6 * W * L * H * Density =* 0.6 * 6.0 / 12 * 6.0 / 12 * 0.0 * 0.15 = 0.0 kip

  Arm = *L / 2 - Offset =* 3.50 / 2 - 0.0 / 12 = 1.75 ft   Moment = 0.0 * 1.75 = 0.0 k-ft

- Soil cover = *0.6 * W * L * SC * Density =* 0.6 * (3.50 * 3.50 - 6.0 / 12 * 6.0 / 12) * 0.0 * 110 = 0.0 kip

  Arm = *L / 2 =* 3.50 / 2 = 1.75 ft   Moment = 0.0 * 1.75 = 0.0 k-ft

- Buoyancy = *0.6 * W * L * γ * (SC + Thick - WT) =* 0.6 * 3.50 * 3.50 * 62 * (0.67) = -0.3 kip

  Arm = *L / 2 =* 3.50 / 2 = 1.75 ft   Moment = 0.3 * 1.75 = -0.5 k-ft

- Axial force P = 0.6 * 4.6 + 0.6 * 0.0 = 2.8 kip

  Arm = *L / 2 - Offset =* 3.50 / 2 - 0.0 / 12 = 1.75 ft   Moment = 2.8 * 1.75 = 4.8 k-ft

- Resisting moment Z-Z = 1.3 + 0.0 + 0.0 + 4.8 + -0.5 = 5.6 k-ft

- Overturning safety factor Z-Z = $\dfrac{\text{Resisting moment}}{\text{Overturning moment}} = \dfrac{5.6}{0.0}$ = 55.81 > 1.50 OK

SOIL BEARING PRESSURES (Comb: D+L)

Overturning moment X-X = 0.0 + 0.0 = 0.0 k-ft

Resisting moment X-X = 2.1 + 0.0 + 0.0 + -0.9 + 32.7 = 34.0 k-ft

Overturning moment Z-Z = 0.0 + 0.0 = 0.0 k-ft

Resisting moment Z-Z = 2.1 + 0.0 + 0.0 + -0.9 + 32.7 = 34.0 k-ft

Resisting force = *Footing + Pedestal + Soil - Buoyancy + P =* 1.2 + 0.0 + 0.0 - 0.5 + 18.7 = 19.4 kip

X-coordinate of resultant from maximum bearing corner:

Xp = $\dfrac{\text{Z-Resisting moment - Z-Overturning moment}}{\text{Resisting force}} = \dfrac{34.0 - 0.0}{19.4}$ = 1.75 ft

Z-coordinate of resultant from maximum bearing corner:

Zp = $\dfrac{\text{X-Resisting moment - X-Overturning moment}}{\text{Resisting force}} = \dfrac{34.0 - 0.0}{19.4}$ = 1.75 ft

X-ecc = *Length / 2 - Xp =* 3.50 / 2 - 1.75 = 0.00 ft

Z-ecc = *Width / 2 - Zp =* 3.50 / 2 - 1.75 = 0.00 ft

Area = *Width * Length =* 3.50 * 3.50 = 12.3 ft²

Sx = *Length * Width² / 6 =* 3.50 * 3.50² / 6 = 7.1 ft³

Sz = *Width * Length² / 6 =* 3.50 * 3.50² / 6 = 7.1 ft³

- Footing is in full bearing. Soil pressures are as follows:

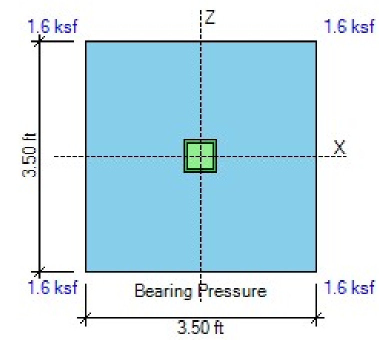
P1 = *P * (1/ A + Z-ecc / Sx + X-ecc / Sz) =* 19.4 * (1 / 12.3 + 0.00 / 7.1 + 0.00 / 7.1) = 1.58 ksf

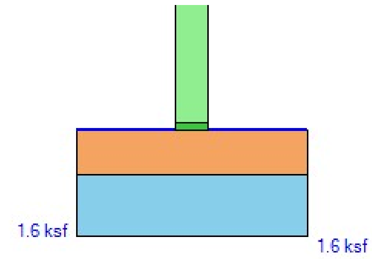
P2 = *P * (1/ A - Z-ecc / Sx + X-ecc / Sz) =* 19.4 * (1 / 12.3 - 0.00 / 7.1 + 0.00 / 7.1) = 1.58 ksf

P3 = *P * (1/ A - Z-ecc / Sx - X-ecc / Sz) =* 19.4 * (1 / 12.3 - 0.00 / 7.1 - 0.00 / 7.1) = 1.58 ksf

P4 = *P * (1/ A + Z-ecc / Sx - X-ecc / Sz) =* 19.4 * (1 / 12.3 + 0.00 / 7.1 - 0.00 / 7.1) = 1.58 ksf

SPREAD FOOTING DESIGN





SLIDING CALCULATIONS (Comb: 0.6D+0.6W)

Internal friction angle = 28.0 deg

Passive coefficient *kp = 4.33* *(per Coulomb)*

Pressure at mid-depth = *kp * Density * (Cover + Thick / 2) =* 4.33 * 110 * (0.00 + 8.0 / 12 / 2) = 0.16 ksf

X-Passive force = *Pressure * Thick * Width =* 0.16 * 8.0 / 12 * 3.50 = 0.4 kip

Z-Passive force = *Pressure * Thick * Length =* 0.16 * 8.0 / 12 * 3.50 = 0.4 kip

Friction force = *Resisting force * Friction coeff. =* Max (0, 3.2 * 0.35) = 1.1 kip

Use 100% of Passive + 100% of Friction for sliding resistance

- Sliding safety factor X-X = $\dfrac{\textit{X-Passive force + Friction}}{\textit{X-Horizontal load}} = \dfrac{1.00 * 0.4 + 1.00 * 1.1}{0.0}$ = 14.86 > 1.50 OK

- Sliding safety factor Z-Z = $\dfrac{\textit{Z-Passive force + Friction}}{\textit{Z-Horizontal load}} = \dfrac{1.00 * 0.4 + 1.00 * 1.1}{0.0}$ = 14.86 > 1.50 OK

UPLIFT CALCULATIONS (Comb: 0.6D+0.6W)

- Uplift safety factor = $\dfrac{\textit{Pedestal + Footing + Cover - Buoyancy}}{\textit{Uplift load}} = \dfrac{0.0 + 0.7 + 0.0 - 0.3}{0.0}$ = 99.99 > 1.00 OK

ONE-WAY SHEAR CALCULATIONS (Comb: 1.2D+1.6L+0.5S)

Concrete f'c = 2.5 ksi | Steel fy = 40.0 ksi | Soil density = 110 pcf

d Top X-dir = *Thick - Cover - X-diameter / 2 =* 8.0 - 2.0 - 0.8 / 2 = 5.6 in

d Top Z-dir = *Thick - Cover - X-diameter - Z-diameter / 2 =* 8.0 - 2.0 - 0.8 - 0.8 / 2 = 4.9 in

d Bot X-dir = *Thick - Cover - X-diameter / 2 =* 8.0 - 3.0 - 0.5 / 2 = 4.8 in

d Bot Z-dir = *Thick - Cover - X-diameter - Z-diameter / 2 =* 8.0 - 3.0 - 0.5 - 0.5 / 2 = 4.3 in

*ϕVcx = 2 * ϕ * √(f'c) * Width * d / 1000 =*2 * 0.75 * √(2500) * 3.5 * 12 * 4.8 / 1000 = 15.0 kip   ACI Eq. (22.5.5.1)

*ϕVcz = 2 * ϕ * √(f'c) * Length * d / 1000 =*2 * 0.75 * √(2500) * 3.5 * 12 * 4.3 / 1000 = 13.4 kip

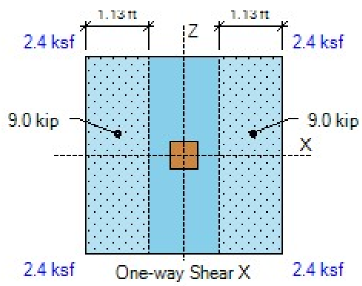
- Shear forces calculated as the volume of the bearing pressures under the effective areas:

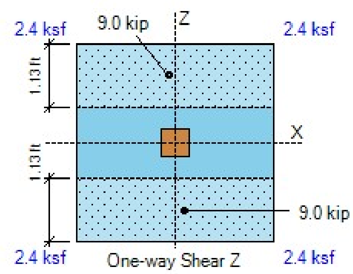
One-way shear Vux (- Side) = 9.0 kip < 15.0 kip OK

One-way shear Vux (+ Side) = 9.0 kip < 15.0 kip OK

One-way shear Vuz (- Side) = 9.0 kip < 13.4 kip OK

One-way shear Vuz (+ Side) = 9.0 kip < 13.4 kip OK





FLEXURE CALCULATIONS (Comb: 1.2D+1.6L+0.5S)

Plain ϕMnx = *5 * ϕ * √(f'c) * L * Thick² / 6* = 5 * 0.60 * √(2500) * 3.50 * 8.0^2 / 6 / 1000 = 1.5 k-ft — ACI Eq. (14.5.2.1a)

Plain ϕMnz = *5 * ϕ * √(f'c) * W * Thick² / 6* = 5 * 0.60 * √(2500) * 3.50 * 8.0^2 / 6 / 1000 = 1.5 k-ft

- Top Bars

No Top Reinforcement Provided at the Footing

Use Plain Concrete Flexural Strength at Top

- Top moments calculated as the overburden minus the bearing pressures times the lever arm:

Top moment -Mux (- Side) = 0.0 k-ft < 5.6 k-ft OK

Top moment -Mux (+ Side) = 0.0 k-ft < 5.6 k-ft OK

Top moment -Muz (- Side) = 0.0 k-ft < 5.6 k-ft OK

Top moment -Muz (+ Side) = 0.0 k-ft < 5.6 k-ft OK

- Bottom Bars

Use 5 #4 Z-Bars — *ρ = As / b d* = 1.0 / (3.50 * 12 * 4.3) = 0.0056 — q = 0.0056 * 40 / 2.5 = 0.090

Use 5 #4 X-Bars — *ρ = As / b d* = 1.0 / (3.50 * 12 * 4.8) = 0.0050 — q = 0.0050 * 40 / 2.5 = 0.080

β = L / W = 3.50 / 3.50 = 1.00 — *γs = 2 * β / (β + 1)* = 2 * 1.00 / (1.00 + 1) = 1.00 — ACI 13.3.3.3

Bending strength *ϕMn = ϕ * b * d² * f'c * q * (1 - 0.59 * q)* — ACI 22.2.2

ϕMnx = 0.90 * 3.50 * 12 * 4.3^2 * 2.5 * 0.090 * (1 - 0.59 * 0.090) = 12.1 k-ft

ϕMnz = 0.90 * 3.50 * 12 * 4.8^2 * 2.5 * 0.080 / 1.00 * (1 - 0.59 * 0.080 / 1.00) = 13.6 k-ft

- Bottom moments calculated as the bearing minus the overburden pressures times the lever arm:

Bottom moment Mux (- Side) = 9.0 k-ft < 12.1 k-ft OK ratio = 0.75

Bottom moment Mux (+ Side) = 9.1 k-ft < 12.1 k-ft OK ratio = 0.75

Bottom moment Muz (- Side) = 9.0 k-ft < 13.6 k-ft OK ratio = 0.67

Bottom moment Muz (+ Side) = 9.1 k-ft < 13.6 k-ft OK ratio = 0.67

X-As min = *0.0018 * Width * Thick* = 0.0018 * 3.50 * 12 * 8.0 = 0.6 in² < 1.0 in² OK — ACI 8.6.1.1

Z-As min = *0.0018 * Length * Thick* = 0.0018 * 3.50 * 12 * 8.0 = 0.6 in² < 1.0 in² OK — ACI 8.6.1.1

X-As max for 0.005 tension strain = 3.20 in² > 1.00 in² OK — ACI 21.2.2

Z-As max for 0.005 tension strain = 3.20 in² > 1.00 in² OK — ACI 21.2.2

X-Cover factor = *Min (2.5, (Cover + db /2, Spacing / 2) / db)* = Min (2.5, (3.0 + 0.50 / 2, 9.0 / 2) / 0.50) = 2.5

Straight *X-Ld = Max (12.0, 3 / 40 * fy / (f'c)½ * Grade * Size * Casting / Cover * db * ratio)* — ACI Eq. (25.4.2.3a)

X-Ld = Max (12.0, 3 / 40 * 40.0 * 1000 / (2500)½ * 1.0 * 0.8 * 1.0 / 2.5 * 0.50 * 0.67) = 12.0 in

Hooked *X-Ldh = Max (8 db, 6, 0.02 * fy / (f'c)½ * Confining * Location * Concrete * db * ratio) =* — ACI 25.4.3

X-Ldh = Max (8 db, 6, 0.02 * 40.0 * 1000 / (2500)½ * 1.0 * 0.7 * 0.0 * 0.50 * 0.67) = 6.0 in

-X Ld provided = *(Length - Col) / 2 + Offset - Cover =* 3.50 * 12 / 2 + 0.0 - 6.0 / 2 - 2.5 = 15.5 in > 12.0 in OK

+X Ld provided = *(Length - Col) / 2 - Offset - Cover =* 3.50 * 12 / 2 - 0.0 - 6.0 / 2 - 2.5 = 15.5 in > 12.0 in OK

Z-Cover factor = *Min (2.5, (Cover + db /2, Spacing / 2) / db) =* Min (2.5, (3.0 + 0.50 / 2, 9.0 / 2) / 0.50) = 2.5

Straight *Z-Ld = Max (12.0, 3 / 40 * fy / (f'c)½ * Grade * Size * Casting / Cover * db * ratio)* ACI Eq. (25.4.2.3a)

Z-Ld = Max (12.0, 3 / 40 * 40.0 * 1000 / (2500)½ * 1.0 * 0.8 * 1.0 / 2.5 * 0.50 * 0.67) = 12.0 in

Hooked *Z-Ldh = Max (8 db, 6, 0.02 * fy / (f'c)½ * Confining * Location * Concrete * db * ratio) =* ACI 25.4.3

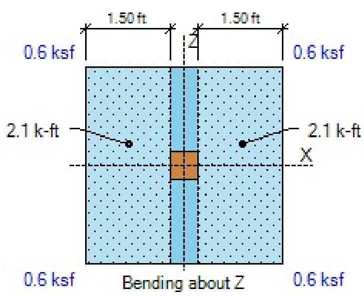
Z-Ldh = Max (8 db, 6, 0.02 * 40.0 * 1000 / (2500)½ * 1.0 * 0.7 * 0.0 * 0.50 * 0.75) = 6.0 in

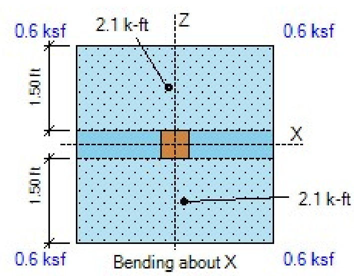
-Z Ld provided = *(Width - Col) / 2 + Offset - Cover =* 3.50 * 12 / 2 + 0.0 - 6.0 / 2 - 2.5 = 15.5 in > 12.0 in OK

+Z Ld provided =*(Width - Col) / 2 - Offset - Cover =* 3.50 * 12 / 2 - 0.0 - 6.0 / 2 - 2.5 = 15.5 in > 12.0 in OK

X-bar spacing = 9.0 in < Min (3 * t, 18.0) = 18.0 in OK ACI 7.7.2.3

Z-bar spacing = 9.0 in < Min (3 * t, 18.0) = 18.0 in OK





LOAD TRANSFER CALCULATIONS (Comb: 1.2D+1.6L+0.5S)

Area *A1 = col L * col W =* 6.0 * 6.0 = 36.0 in²

Sx = *col W * col L² / 6 =* 6.0 * 6.0² / 6 = 36.0 in³

Sz = *col L * col W² / 6 =* 6.0 * 6.0² / 6 = 36.0 in³

Bearing *Pbu = P / A1 + Mz / Sx + Mx / Sz =* 28.1 / 36.0 + 0.0 * 12 / 36.0 + 0.0 * 12 / 36.0 = 0.8 ksi

Min edge = *Min (L / 2 - X-offset - col L / 2, W / 2 - Z-offset - col W / 2)*

Min edge = Min (3.50 * 12 / 2 - 0.0 - 6.0 / 2, 3.50 * 12 / 2 - 0.0 - 6.0 / 2 = 18.0 in

Area *A2 = Min [L * W, (col L + 2 * Min edge) * (col W + 2 * Min edge)]* ACI R22.8.3.2

A2 = Min [3.50 * 12 * 3.5 * 12, (6.0 + 2 * 18.0) * (6.0 + 2 * 18.0)] = 1764.0 in²

Footing *ϕPnc = ϕ * 0.85 * f'c * Min [2, √(A2 / A1)] =* 0.65 * 0.85 * 2.5 * Min [2, √(1764.0 / 36.0)] = 2.8 ksi

Footing *ϕPns = ϕ * As * Fy / A1 =* 0.0 ksi ACI 22.8.3.2

Footing bearing *ϕPn = ϕPnc + ϕPns =* 2.8 + 0.0 = 2.8 ksi > 0.8 psi OK

Hooked *Ldh = Max (8 db, 6, 0.02 * fy / (f'c)½ * Confining * Location * Concrete * db * ratio)* ACI 25.4.3

Ldh = Max (8 db, 6, 0.02 * 60.0 * 1000 / (2500)½ * 1.0 * 0.7 * 0.0 * 0.75 * 0.14) = 6.0 in

Ld provided = *Dowel length =* 3.00 * 12 = 36.0 in > 24.3 in OK

Ldh provided = *Footing thickness - Cover =* 8.00 - 3.0 = 5.0 in < 6.0 in NG

PUNCHING SHEAR CALCULATIONS (Comb: 1.2D+1.6L+0.5Lr)

X-Edge = *d / 2 =* 4.5 / 2 = 2.3 in αsx = 20

Z-Edge = *d / 2 =* 4.5 / 2 = 2.3 in αsz = 20

αs = *αsx + αsz =* 20 + 20 = 40 Col type = Interior β = *L / W =* 6.0 / 6.0 = 1.00 ACI 22.6.5.2

Perimeter *bo = αsz / 10 * (L + d / 2 + X-Edge) + αsx / 10 * (W + d / 2 + Z-Edge)* ACI 22.6.4.2

bo = 20 / 10 * (6.0 + 4.5 / 2 + 2.3) + 20 / 10 * (6.0 + 4.5 / 2 + 2.3) = 42.0 in

Area *Abo = (L + d / 2 + X-Edge) * (W + d / 2 + Z-Edge) =* (6.0 + 4.5 / 2 + 2.3) * (6.0 + 4.5 / 2 + 2.3) = 110.3 in²

ϕVc = *ϕ * Min (2 + 4 / β, αs * d / bo + 2, 4) * √(f'c)* ACI 22.6.5.2

ϕVc = 0.75 * Min (2 + 4 / 1.00, 40 * 4.5 / 42.0 + 2, 4) * √(2500) = 150.0 psi

Punching force *F = P + Overburden * Abo - Bearing*

F = 28.1 + 0.07 * 110.3 / 144 - 1.8 = 26.3 kip

b1 = *L + d / 2 + X-Edge =* 6.0 + 4.5 / 2 + 2.3 = 10.5 in b2 = *W + d / 2 + Z-Edge =* 6.0 + 4.5 / 2 + 2.3 = 10.5 in

γvx factor = $1 - \dfrac{1}{1 + (2/3)\sqrt{b2 / b1}} = 1 - \dfrac{1}{1 + (2/3)\sqrt{10.5 / 10.5}} = 0.40$ ACI Eq. (8.4.4.2.2)

γvz factor = $1 - \dfrac{1}{1 + (2/3)\sqrt{b1 / b2}} = 1 - \dfrac{1}{1 + (2/3)\sqrt{10.5 / 10.5}} = 0.40$ ACI Eq. (8.4.2.3.2)

X2z = *b1 / 2 =* 10.5 / 2 = 5.3 in X2x = *b2 / 2 =* 10.5 / 2 = 5.3 in

Jcz = *b1 * d³ / 6 + b1³ * d / 6 + b1² * b2 * d / 2* ACI R8.4.4.2.3

Jcz = 10.5 * 4.5³ / 6 + 10.5³ * 4.5 / 6 + 10.5² * 10.5 * 4.5 / 2 = 3632 in⁴

Jcx = *b2 * d³ / 6 + b2³ * d / 6 + b2² * b1 * d / 2* ACI R8.4.4.2.3

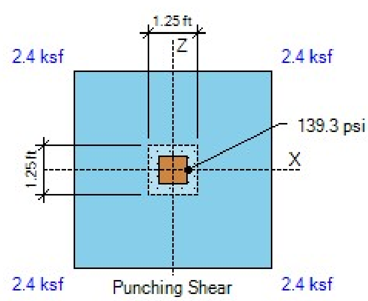
Jcx = 10.5 * 4.5³ / 6 + 10.5³ * 4.5 / 6 + 10.5² * 10.5 * 4.5 / 2 = 3632 in⁴

Stress due to P = *F / (bo * d) * 1000 =* 26.3 / (42.0 * 4.5) * 1000 = 139.3 psi

Stress due to Mx = *γvx * X-OTM * X2x / Jcx =* 0.40 * 0.0 * 12 * 5.3 / 3632 * 1000 = 0.0 psi

Stress due to Mz = *γvz * Z-OTM * X2z / Jcz =* 0.40 * 0.0 * 12 * 5.3 / 3632 * 1000 = 0.0 psi

Punching stress = *P-stress + Mx-stress + Mz-stress =* 139.3 + 0.0 + 0.0 = 139.3 psi < 150.0 psi OK



LOAD TRANSFER CALCULATIONS (Comb: 1.2D+1.6L+0.5S)

Area $A1 = col\ L * col\ W$ = 6.0 * 6.0 = 36.0 in²

Sx = $col\ W * col\ L^2 / 6$ = 6.0 * 6.0² / 6 = 36.0 in³

Sz = $col\ L * col\ W^2 / 6$ = 6.0 * 6.0² / 6 = 36.0 in³

Bearing $Pbu = P / A1 + Mz / Sx + Mx / Sz$ = 28.1 / 36.0 + 0.0 * 12 / 36.0 + 0.0 * 12 / 36.0 = 0.8 ksi

Min edge = *Min (L / 2 - X-offset - col L / 2, W / 2 - Z-offset - col W / 2)*

Min edge = Min (3.50 * 12 / 2 - 0.0 - 6.0 / 2, 3.50 * 12 / 2 - 0.0 - 6.0 / 2 = 18.0 in

Area *A2 = Min [L * W, (col L + 2 * Min edge) * (col W + 2 * Min edge)]* ACI R22.8.3.2

A2 = Min [3.50 * 12 * 3.5 * 12, (6.0 + 2 * 18.0) * (6.0 + 2 * 18.0)] = 1764.0 in²

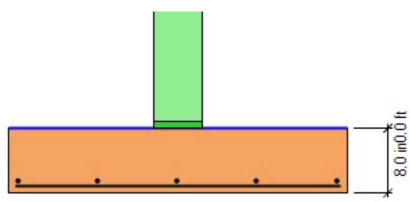
Footing $\phi Pnc = \phi * 0.85 * f'c * Min [2, \sqrt{A2 / A1}]$ = 0.65 * 0.85 * 2.5 * Min [2, √(1764.0 / 36.0)] = 2.8 ksi

Footing $\phi Pns = \phi * As * Fy / A1$ = 0.0 ksi ACI 22.8.3.2

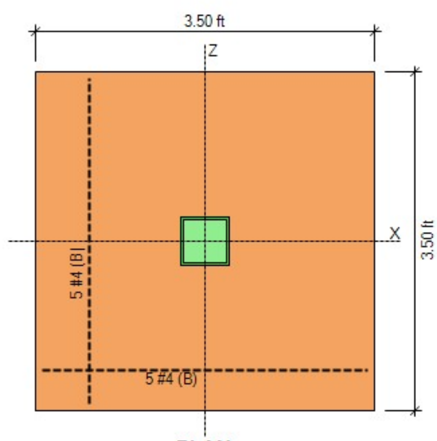
Footing bearing $\phi Pn = \phi Pnc + \phi Pns$ = 2.8 + 0.0 = 2.8 ksi > 0.8 psi OK

DESIGN CODES

Concrete Design	ACI 318-14
Load Combinations	ASCE 7-10/16



ELEVATION



PLAN

SPREAD FOOTING DESIGN

GEOMETRY

Footing Length (X-dir)	2.50	ft	
Footing Width (Z-dir)	2.50	ft	
Footing Thickness	8.0	in	OK
Soil Cover	0.00	ft	
Column Length (X-dir)	6.0	in	
Column Width (Z-dir)	6.0	in	
Offset (X-dir)	0.00	in	OK
Offset (Z-dir)	0.00	in	OK
Base Plate (L x W)	6.0 x 6.0	in	

SOIL PRESSURES (D+L)

Gross Allow. Soil Pressure	2.0	ksf	
Soil Pressure at Corner 1	1.5	ksf	
Soil Pressure at Corner 2	1.5	ksf	
Soil Pressure at Corner 3	1.5	ksf	
Soil Pressure at Corner 4	1.5	ksf	
Bearing Pressure Ratio	0.77		OK
Ftg. Area in Contact with Soil	100.0	%	
X-eccentricity / Ftg. Length	0.00		OK
Z-eccentricity / Ftg. Width	0.00		OK

APPLIED LOADS

	Dead	Live	RLive	Snow	Wind	Seismic	
Axial Force P	4.1	5.2	0.0	0.0	0.0	0.0	kip
Moment about X Mx ..	0.0	0.0	0.0	0.0	0.0	0.0	k-ft
Moment about Z Mz ..	0.0	0.0	0.0	0.0	0.0	0.0	k-ft
Shear Force Vx	0.0	0.0	0.0	0.0	0.0	0.0	kip
Shear Force Vz	0.0	0.0	0.0	0.0	0.0	0.0	kip

OVERTURNING CALCULATIONS (Comb: 0.6D+0.6W)

- Overturning about X-X

- Moment Mx = 0.6 * 0.0 + 0.6 * 0.0 = 0.0 k-ft
- Shear Force Vz = 0.6 * 0.0 + 0.6 * 0.0 = 0.0 kip
  Arm = 0.00 + 8.0 / 12 = 0.67 ft   Moment = 0.0 * 0.67 = 0.0 k-ft
- Passive Force = 0.0 kip   Arm = 0.27 ft   Moment = 0.0 k-ft
- Overturning moment X-X = 0.0 + 0.0 = 0.0 k-ft

- Resisting about X-X

- Footing weight = *0.6 * W * L * Thick * Density =* 0.6 * 2.50 * 2.50 * 8.0 / 12 * 0.15 = 0.4 kip
  Arm = *W / 2 =* 2.50 / 2 = 1.25 ft   Moment = 0.4 * 1.25 = 0.5 k-ft
- Pedestal weight = *0.6 * W * L * H * Density =* 0.6 * 6.0 / 12 * 6.0 / 12 * 0.0 * 0.15 = 0.0 kip
  Arm = *W / 2 - Offset =* 2.50 / 2 - 0.0 / 12 = 1.25 ft   Moment = 0.0 * 1.25 = 0.0 k-ft
- Soil cover = *0.6 * W * L * SC * Density =* 0.6 * (2.50 * 2.50 - 6.0 / 12 * 6.0 / 12) * 0.0 * 110 = 0.0 kip
  Arm = *W / 2 =* 2.50 / 2 = 1.25 ft   Moment = 0.0 * 1.25 = 0.0 k-ft
- Buoyancy = *0.6 * W * L * γ * (SC + Thick - WT) =* 0.6 * 2.50 * 2.50 * 62 * (0.67) = -0.2 kip
  Arm = *W / 2 =* 2.50 / 2 = 1.25 ft   Moment = 0.2 * 1.25 = -0.2 k-ft
- Axial force P = 0.6 * 4.1 + 0.6 * 0.0 = 2.5 kip
  Arm = *W / 2 - Offset =* 2.50 / 2 - 0.0 / 12 = 1.25 ft   Moment = 2.5 * 1.25 = 3.1 k-ft
- Resisting moment X-X = 0.5 + 0.0 + 0.0 + 3.1 + -0.2 = 3.3 k-ft

- Overturning safety factor X-X = $\dfrac{\textit{Resisting moment}}{\textit{Overturning moment}} = \dfrac{3.3}{0.0}$ = 33.49 > 1.50 OK

- Overturning about Z-Z

- Moment Mz = 0.6 * 0.0 + 0.6 * 0.0 = 0.0 k-ft

- Shear Force Vx = 0.6 * 0.0 + 0.6 * 0.0 = 0.0 kip

Arm = 0.00 + 8.0 / 12 = 0.67 ft Moment = 0.0 * 0.67 = 0.0 k-ft

- Passive Force = 0.0 kip Arm = 0.27 ft Moment = 0.0 k-ft

- Overturning moment Z-Z = 0.0 + 0.0 = 0.0 k-ft

- Resisting about Z-Z

- Footing weight = *0.6 * W * L * Thick * Density =* 0.6 * 2.50 * 2.50 * 8.0 / 12 * 0.15 = 0.4 kip

Arm = *L / 2 =* 2.50 / 2 = 1.25 ft Moment = 0.4 * 1.25 = 0.5 k-ft

- Pedestal weight = *0.6 * W * L * H * Density =* 0.6 * 6.0 / 12 * 6.0 / 12 * 0.0 * 0.15 = 0.0 kip

Arm = *L / 2 - Offset =* 2.50 / 2 - 0.0 / 12 = 1.25 ft Moment = 0.0 * 1.25 = 0.0 k-ft

- Soil cover = *0.6 * W * L * SC * Density =* 0.6 * (2.50 * 2.50 - 6.0 / 12 * 6.0 / 12) * 0.0 * 110 = 0.0 kip

Arm = *L / 2 =* 2.50 / 2 = 1.25 ft Moment = 0.0 * 1.25 = 0.0 k-ft

- Buoyancy = *0.6 * W * L * γ * (SC + Thick - WT) =* 0.6 * 2.50 * 2.50 * 62 * (0.67) = -0.2 kip

Arm = *L / 2 =* 2.50 / 2 = 1.25 ft Moment = 0.2 * 1.25 = -0.2 k-ft

- Axial force P = 0.6 * 4.1 + 0.6 * 0.0 = 2.5 kip

Arm = *L / 2 - Offset =* 2.50 / 2 - 0.0 / 12 = 1.25 ft Moment = 2.5 * 1.25 = 3.1 k-ft

- Resisting moment Z-Z = 0.5 + 0.0 + 0.0 + 3.1 + -0.2 = 3.3 k-ft

- Overturning safety factor Z-Z = $\dfrac{\textit{Resisting moment}}{\textit{Overturning moment}} = \dfrac{3.3}{0.0}$ = 33.49 > 1.50 OK

SOIL BEARING PRESSURES (Comb: D+L)

Overturning moment X-X = 0.0 + 0.0 = 0.0 k-ft

Resisting moment X-X = 0.8 + 0.0 + 0.0 + -0.3 + 11.6 = 12.1 k-ft

Overturning moment Z-Z = 0.0 + 0.0 = 0.0 k-ft

Resisting moment Z-Z = 0.8 + 0.0 + 0.0 + -0.3 + 11.6 = 12.1 k-ft

Resisting force = *Footing + Pedestal + Soil - Buoyancy + P =* 0.6 + 0.0 + 0.0 - 0.3 + 9.3 = 9.7 kip

X-coordinate of resultant from maximum bearing corner:

Xp = $\dfrac{\textit{Z-Resisting moment - Z-Overturning moment}}{\textit{Resisting force}} = \dfrac{12.1 - 0.0}{9.7}$ = 1.25 ft

Z-coordinate of resultant from maximum bearing corner:

Zp = $\dfrac{\textit{X-Resisting moment - X-Overturning moment}}{\textit{Resisting force}} = \dfrac{12.1 - 0.0}{9.7}$ = 1.25 ft

X-ecc = *Length / 2 - Xp =* 2.50 / 2 - 1.25 = 0.00 ft

Z-ecc = *Width / 2 - Zp =* 2.50 / 2 - 1.25 = 0.00 ft

Area = *Width * Length =* 2.50 * 2.50 = 6.3 ft²

Sx = *Length * Width² / 6 =* 2.50 * 2.50² / 6 = 2.6 ft³

Sz = *Width * Length² / 6 =* 2.50 * 2.50² / 6 = 2.6 ft³

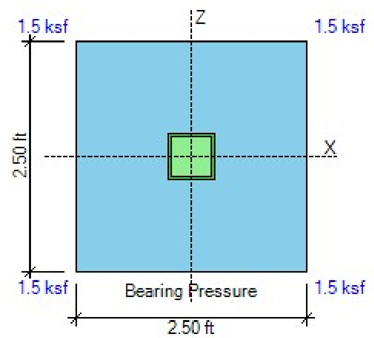
- Footing is in full bearing. Soil pressures are as follows:

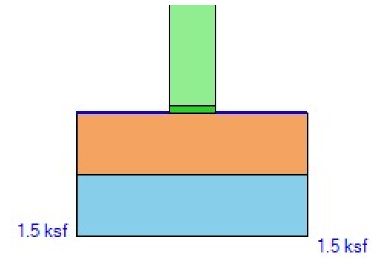
P1 = *P * (1/ A + Z-ecc / Sx + X-ecc / Sz) =* 9.7 * (1 / 6.3 + 0.00 / 2.6 + 0.00 / 2.6) = 1.55 ksf

P2 = *P * (1/ A - Z-ecc / Sx + X-ecc / Sz) =* 9.7 * (1 / 6.3 - 0.00 / 2.6 + 0.00 / 2.6) = 1.55 ksf

P3 = *P * (1/ A - Z-ecc / Sx - X-ecc / Sz) =* 9.7 * (1 / 6.3 - 0.00 / 2.6 - 0.00 / 2.6) = 1.55 ksf

P4 = *P * (1/ A + Z-ecc / Sx - X-ecc / Sz) =* 9.7 * (1 / 6.3 + 0.00 / 2.6 - 0.00 / 2.6) = 1.55 ksf





SLIDING CALCULATIONS (Comb: 0.6D+0.6W)

Internal friction angle = 28.0 deg

Passive coefficient *kp = 4.33* *(per Coulomb)*

Pressure at mid-depth = *kp * Density * (Cover + Thick / 2) =* 4.33 * 110 * (0.00 + 8.0 / 12 / 2) = 0.16 ksf

X-Passive force = *Pressure * Thick * Width =* 0.16 * 8.0 / 12 * 2.50 = 0.3 kip

Z-Passive force = *Pressure * Thick * Length =* 0.16 * 8.0 / 12 * 2.50 = 0.3 kip

Friction force = *Resisting force * Friction coeff. =* Max (0, 2.7 * 0.35) = 0.9 kip

Use 100% of Passive + 100% of Friction for sliding resistance

- Sliding safety factor X-X = $\dfrac{\textit{X-Passive force + Friction}}{\textit{X-Horizontal load}} = \dfrac{1.00 * 0.3 + 1.00 * 0.9}{0.0}$ = 12.02 > 1.50 OK

- Sliding safety factor Z-Z = $\dfrac{\textit{Z-Passive force + Friction}}{\textit{Z-Horizontal load}} = \dfrac{1.00 * 0.3 + 1.00 * 0.9}{0.0}$ = 12.02 > 1.50 OK

UPLIFT CALCULATIONS (Comb: 0.6D+0.6W)

- Uplift safety factor = $\dfrac{\textit{Pedestal + Footing + Cover - Buoyancy}}{\textit{Uplift load}} = \dfrac{0.0 + 0.4 + 0.0 - 0.2}{0.0}$ = 99.99 > 1.00 OK

ONE-WAY SHEAR CALCULATIONS (Comb: 1.2D+1.6L+0.5S)

Concrete f'c = 2.5 ksi Steel fy = 40.0 ksi Soil density = 110 pcf

Use Plain Concrete Shear Strength

ϕVcx = *4/3 * ϕ * √(f'c) * Width * t / 1000 =* 4/3 * 0.60 * √(2500) * 2.5 * 12 * 8.0 / 1000 = 9.6 kip ACI 14.5.5.1

ϕVcz = *4/3 * ϕ * √(f'c) * Length * t / 1000 =* 4/3 * 0.60 * √(2500) * 2.5 * 12 * 8.0 / 1000 = 9.6 kip

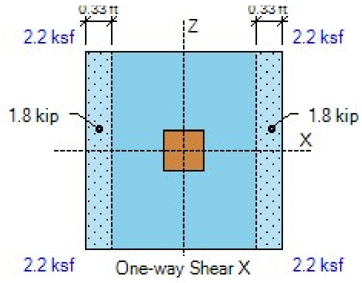
- Shear forces calculated as the volume of the bearing pressures under the effective areas:

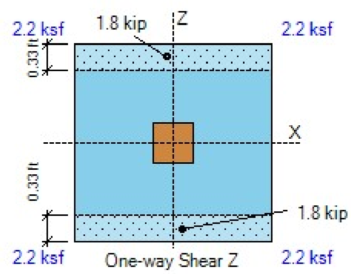
One-way shear Vux (- Side) = 1.8 kip < 9.6 kip OK

One-way shear Vux (+ Side) = 1.8 kip < 9.6 kip OK

One-way shear Vuz (- Side) = 1.8 kip < 9.6 kip OK

One-way shear Vuz (+ Side) = 1.8 kip < 9.6 kip OK





FLEXURE CALCULATIONS (Comb: 1.2D+1.6L+0.5S)

Plain ϕMnx =*5 * ϕ * √(f'c) * L * Thick² / 6* =5 * 0.60 * √(2500) * 2.50 * 8.0² / 6 / 1000 = 1.1 k-ft ACI Eq. (14.5.2.1a)

Plain ϕMnz =*5 * ϕ * √(f'c) * W * Thick² / 6* =5 * 0.60 * √(2500) * 2.50 * 8.0² / 6 / 1000 = 1.1 k-ft

- Top Bars

No Top Reinforcement Provided at the Footing

Use Plain Concrete Flexural Strength at Top

- Top moments calculated as the overburden minus the bearing pressures times the lever arm:

Top moment -Mux (- Side) = 0.0 k-ft < 4.0 k-ft OK

Top moment -Mux (+ Side) = 0.0 k-ft < 4.0 k-ft OK

Top moment -Muz (- Side) = 0.0 k-ft < 4.0 k-ft OK

Top moment -Muz (+ Side) = 0.0 k-ft < 4.0 k-ft OK

- Bottom Bars

No Bottom Reinforcement Provided at the Footing

Use Plain Concrete Flexural Strength at Bottom

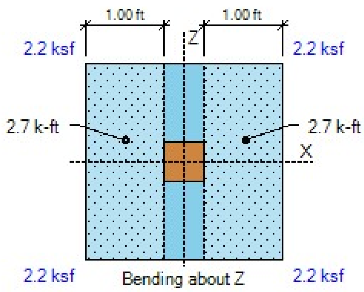
- Bottom moments calculated as the bearing minus the overburden pressures times the lever arm:

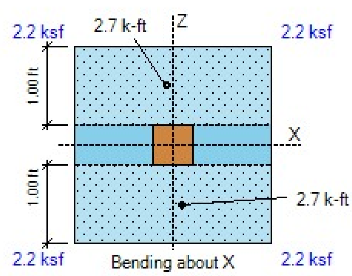
Bottom moment Mux (- Side) = 2.7 k-ft < 4.0 k-ft OK ratio = 0.66

Bottom moment Mux (+ Side) = 2.7 k-ft < 4.0 k-ft OK ratio = 0.66

Bottom moment Muz (- Side) = 2.7 k-ft < 4.0 k-ft OK ratio = 0.66

Bottom moment Muz (+ Side) = 2.7 k-ft < 4.0 k-ft OK ratio = 0.66





LOAD TRANSFER CALCULATIONS (Comb: 1.2D+1.6L+0.5S)

Area *A1 = col L * col W =* 6.0 * 6.0 = 36.0 in²

Sx = *col W * col L² / 6 =* 6.0 * 6.0² / 6 = 36.0 in³

Sz = *col L * col W² / 6 =* 6.0 * 6.0² / 6 = 36.0 in³

Bearing *Pbu = P / A1 + Mz / Sx + Mx / Sz =* 13.2 / 36.0 + 0.0 * 12 / 36.0 + 0.0 * 12 / 36.0 = 0.4 ksi

Min edge = *Min (L / 2 - X-offset - col L / 2, W / 2 - Z-offset - col W / 2)*

Min edge = Min (2.50 * 12 / 2 - 0.0 - 6.0 / 2, 2.50 * 12 / 2 - 0.0 - 6.0 / 2 = 12.0 in

Area *A2 = Min [L * W, (col L + 2 * Min edge) * (col W + 2 * Min edge)]* ACI R22.8.3.2

A2 = Min [2.50 * 12 * 2.5 * 12, (6.0 + 2 * 12.0) * (6.0 + 2 * 12.0)] = 900.0 in²

Footing *ϕPnc = ϕ * 0.85 * f'c * Min [2, √(A2 / A1)] =* 0.65 * 0.85 * 2.5 * Min [2, √(900.0 / 36.0)] = 2.8 ksi

Footing *ϕPns = ϕ * As * Fy / A1 =* 0.0 ksi ACI 22.8.3.2

Footing bearing *ϕPn = ϕPnc + ϕPns =* 2.8 + 0.0 = 2.8 ksi > 0.4 psi OK

Hooked *Ldh = Max (8 db, 6, 0.02 * fy / $(f'c)^{1/2}$ * Confining * Location * Concrete * db * ratio)* ACI 25.4.3

Ldh = Max (8 db, 6, 0.02 * 60.0 * 1000 / $(2500)^{1/2}$ * 1.0 * 0.7 * 0.0 * 0.75 * 0.06) = 6.0 in

Ld provided = *Dowel length =* 3.00 * 12 = 36.0 in > 12.0 in OK

Ldh provided = *Footing thickness - Cover =* 8.00 - 3.0 = 5.0 in < 6.0 in NG

PUNCHING SHEAR CALCULATIONS (Comb: 1.2D+1.6L+0.5Lr)

X-Edge = *Length / 2 - Offset - Col / 2 =* 2.50 * 12 / 2 - 0.0 - 6.0 / 2 = 12.0 in αsx = 10

Z-Edge = *Width / 2 - Offset - Col / 2 =* 2.50 * 12 / 2 - 0.0 - 6.0 / 2 = 12.0 in αsz = 10

αs = *αsx + αsz* = 10 + 10 = 20 Col type = Corner β = *L / W* = 6.0 / 6.0 = 1.00 ACI 22.6.5.2

Perimeter *bo = αsz / 10 * (L + d / 2 + X-Edge) + αsx / 10 * (W + d / 2 + Z-Edge)* ACI 22.6.4.2

bo = 10 / 10 * (6.0 + 8.0 / 2 + 12.0) + 10 / 10 * (6.0 + 8.0 / 2 + 12.0) = 44.0 in

Area *Abo = (L + d / 2 + X-Edge) * (W + d / 2 + Z-Edge) =* (6.0 + 8.0 / 2 + 12.0) * (6.0 + 8.0 / 2 + 12.0) = 484.0 in^2

Use Plain Concrete Shear Strength

ϕVc = *ϕ * Min (1 + 2 / β, 2) * 4/3 * √(f'c)* ACI 14.5.5.1

ϕVc = 0.60 * Min (1 + 2 / 1.00, 2) * 4/3 √(2500) = 80.0 psi

Punching force *F = P + Overburden * Abo - Bearing*

F = 13.2 + 0.07 * 484.0 / 144 - 3.0 = 10.5 kip

b1 = *L + d / 2 + X-Edge =* 6.0 + 8.0 / 2 + 12.0 = 22.0 in b2 = *W + d / 2 + Z-Edge =* 6.0 + 8.0 / 2 + 12.0 = 22.0 in

γvx factor = $1 - \dfrac{1}{1 + (2/3)\sqrt{(b2 / b1)}} = 1 - \dfrac{1}{1 + (2/3)\sqrt{(22.0 / 22.0)}} = 0.40$ ACI Eq. (8.4.4.2.2)

γvz factor = $1 - \dfrac{1}{1 + (2/3)\sqrt{(b1 / b2)}} = 1 - \dfrac{1}{1 + (2/3)\sqrt{(22.0 / 22.0)}} = 0.40$ ACI Eq. (8.4.2.3.2)

X2z = *$b1^2$ / 2 / (b1 + b2) =* 22.0^2 / 2 / (22.0 + 22.0) = 5.5 in X2x = *$b2^2$ / 2 / (b2 + b1) =* 5.5 in

Jcz = *b1 * d^3 / 12 + $b1^3$ * d / 12 + b1 * d * $(b1 / 2 - X2z)^2$ + b2 * d * $X2z^2$* ACI R8.4.4.2.3

Jcz = 22.0 * 8.0^3 / 12 + 22.0^3 * 8.0 / 12 + 22.0 * 8.0 * $(22.0 / 2 * 5.5)^2$ + 22.0 * 8.0 * 5.5^2 = 18685 in^4

Jcx = *b2 * d^3 / 12 + $b2^3$ * d / 12 + b2 * d * $(b2 / 2 - X2x)^2$ + b1 * d * $X2x^2$* ACI R8.4.4.2.3

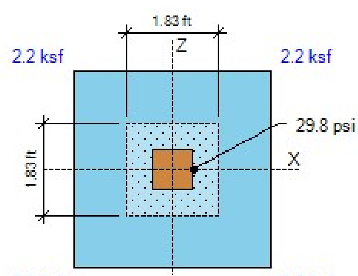
Jcz = 22.0 * 8.0^3 / 12 + 22.0^3 * 8.0 / 12 + 22.0 * 8.0 * $(22.0 / 2 * 5.5)^2$ + 22.0 * 8.0 * 5.5^2 = 18685 in^4

Stress due to P = *F / (bo * d) * 1000 =* 10.5 / (44.0 * 8.0) * 1000 = 29.8 psi

Stress due to Mx = *γvx * X-OTM * X2x / Jcx =* 0.40 * 0.0 * 12 * 5.5 / 18685 * 1000 = 0.0 psi

Stress due to Mz = *γvz * Z-OTM * X2z / Jcz =* 0.40 * 0.0 * 12 * 5.5 / 18685 * 1000 = 0.0 psi

Punching stress = *P-stress + Mx-stress + Mz-stress =* 29.8 + 0.0 + 0.0 = 29.8 psi < 80.0 psi OK



Punching Shear

LOAD TRANSFER CALCULATIONS (Comb: 1.2D+1.6L+0.5S)

Area *A1 = col L * col W* = 6.0 * 6.0 = 36.0 in²

Sx = *col W * col L² / 6* = 6.0 * 6.0² / 6 = 36.0 in³

Sz = *col L * col W² / 6* = 6.0 * 6.0² / 6 = 36.0 in³

Bearing *Pbu = P / A1 + Mz / Sx + Mx / Sz* = 13.2 / 36.0 + 0.0 * 12 / 36.0 + 0.0 * 12 / 36.0 = 0.4 ksi

Min edge = *Min (L / 2 - X-offset - col L / 2, W / 2 - Z-offset - col W / 2)*

Min edge = Min (2.50 * 12 / 2 - 0.0 - 6.0 / 2, 2.50 * 12 / 2 - 0.0 - 6.0 / 2 = 12.0 in

Area *A2 = Min [L * W, (col L + 2 * Min edge) * (col W + 2 * Min edge)]* ACI R22.8.3.2

A2 = Min [2.50 * 12 * 2.5 * 12, (6.0 + 2 * 12.0) * (6.0 + 2 * 12.0)] = 900.0 in²

Footing *ϕPnc = ϕ * 0.85 * f'c * Min [2, √(A2 / A1)]* = 0.65 * 0.85 * 2.5 * Min [2, √(900.0 / 36.0)] = 2.8 ksi

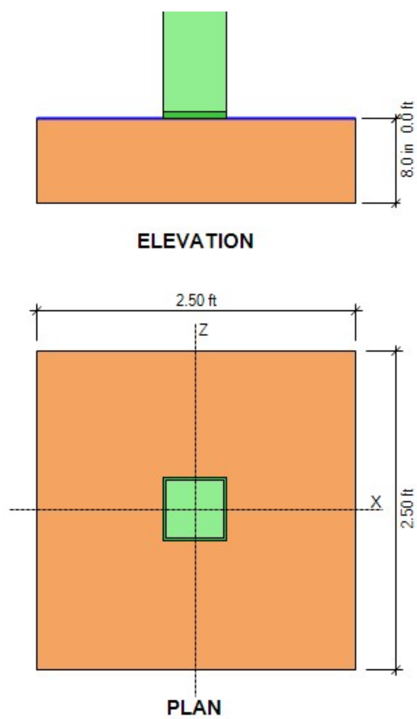
Footing *ϕPns = ϕ * As * Fy / A1* = 0.0 ksi ACI 22.8.3.2

Footing bearing *ϕPn = ϕPnc + ϕPns* = 2.8 + 0.0 = 2.8 ksi > 0.4 psi OK

DESIGN CODES

Concrete Design .... ACI 318-14

Load Combinations .... ASCE 7-10/16



SPREAD FOOTING DESIGN

GEOMETRY

Footing Length (X-dir)	2.50	ft	
Footing Width (Z-dir)	2.50	ft	
Footing Thickness	8.0	in	OK
Soil Cover	0.00	ft	
Column Length (X-dir)	6.0	in	
Column Width (Z-dir)	6.0	in	
Offset (X-dir)	0.00	in	OK
Offset (Z-dir)	0.00	in	OK
Base Plate (L x W)	6.0 x 6.0	in	

SOIL PRESSURES (D+L)

Gross Allow. Soil Pressure	2.0	ksf	
Soil Pressure at Corner 1	1.5	ksf	
Soil Pressure at Corner 2	1.5	ksf	
Soil Pressure at Corner 3	1.5	ksf	
Soil Pressure at Corner 4	1.5	ksf	
Bearing Pressure Ratio	0.77		OK
Ftg. Area in Contact with Soil	100.0	%	
X-eccentricity / Ftg. Length	0.00		OK
Z-eccentricity / Ftg. Width	0.00		OK

APPLIED LOADS

	Dead	Live	RLive	Snow	Wind	Seismic	
Axial Force P	4.1	5.2	0.0	0.0	0.0	0.0	kip
Moment about X Mx ..	0.0	0.0	0.0	0.0	0.0	0.0	k-ft
Moment about Z Mz ..	0.0	0.0	0.0	0.0	0.0	0.0	k-ft
Shear Force Vx	0.0	0.0	0.0	0.0	0.0	0.0	kip
Shear Force Vz	0.0	0.0	0.0	0.0	0.0	0.0	kip

OVERTURNING CALCULATIONS (Comb: 0.6D+0.6W)

- Overturning about X-X

- Moment Mx = 0.6 * 0.0 + 0.6 * 0.0 = 0.0 k-ft

- Shear Force Vz = 0.6 * 0.0 + 0.6 * 0.0 = 0.0 kip

Arm = 0.00 + 8.0 / 12 = 0.67 ft   Moment = 0.0 * 0.67 = 0.0 k-ft

- Passive Force = 0.0 kip   Arm = 0.27 ft   Moment = 0.0 k-ft

- Overturning moment X-X = 0.0 + 0.0 = 0.0 k-ft

- Resisting about X-X

- Footing weight = *0.6 * W * L * Thick * Density =* 0.6 * 2.50 * 2.50 * 8.0 / 12 * 0.15 = 0.4 kip

Arm = *W / 2 =* 2.50 / 2 = 1.25 ft   Moment = 0.4 * 1.25 = 0.5 k-ft

- Pedestal weight = *0.6 * W * L * H * Density =* 0.6 * 6.0 / 12 * 6.0 / 12 * 0.0 * 0.15 = 0.0 kip

Arm = *W / 2 - Offset =* 2.50 / 2 - 0.0 / 12 = 1.25 ft   Moment = 0.0 * 1.25 = 0.0 k-ft

- Soil cover = *0.6 * W * L * SC * Density =* 0.6 * (2.50 * 2.50 - 6.0 / 12 * 6.0 / 12) * 0.0 * 110 = 0.0 kip

Arm = *W / 2 =* 2.50 / 2 = 1.25 ft   Moment = 0.0 * 1.25 = 0.0 k-ft

- Buoyancy = *0.6 * W * L * γ * (SC + Thick - WT) =* 0.6 * 2.50 * 2.50 * 62 * (0.67) = -0.2 kip

Arm = *W / 2 =* 2.50 / 2 = 1.25 ft   Moment = 0.2 * 1.25 = -0.2 k-ft

- Axial force P = 0.6 * 4.1 + 0.6 * 0.0 = 2.5 kip

Arm = *W / 2 - Offset =* 2.50 / 2 - 0.0 / 12 = 1.25 ft   Moment = 2.5 * 1.25 = 3.1 k-ft

- Resisting moment X-X = 0.5 + 0.0 + 0.0 + 3.1 + -0.2 = 3.3 k-ft

- Overturning safety factor X-X = $\frac{\textit{Resisting moment}}{\textit{Overturning moment}} = \frac{3.3}{0.0}$ = 33.49 > 1.50 OK

- Overturning about Z-Z

- Moment Mz = 0.6 * 0.0 + 0.6 * 0.0 = 0.0 k-ft

- Shear Force Vx = 0.6 * 0.0 + 0.6 * 0.0 = 0.0 kip

  Arm = 0.00 + 8.0 / 12 = 0.67 ft Moment = 0.0 * 0.67 = 0.0 k-ft

- Passive Force = 0.0 kip Arm = 0.27 ft Moment = 0.0 k-ft

- Overturning moment Z-Z = 0.0 + 0.0 = 0.0 k-ft

- Resisting about Z-Z

- Footing weight = *0.6 * W * L * Thick * Density =* 0.6 * 2.50 * 2.50 * 8.0 / 12 * 0.15 = 0.4 kip

  Arm = *L / 2 =* 2.50 / 2 = 1.25 ft Moment = 0.4 * 1.25 = 0.5 k-ft

- Pedestal weight = *0.6 * W * L * H * Density =* 0.6 * 6.0 / 12 * 6.0 / 12 * 0.0 * 0.15 = 0.0 kip

  Arm = *L / 2 - Offset =* 2.50 / 2 - 0.0 / 12 = 1.25 ft Moment = 0.0 * 1.25 = 0.0 k-ft

- Soil cover = *0.6 * W * L * SC * Density =* 0.6 * (2.50 * 2.50 - 6.0 / 12 * 6.0 / 12) * 0.0 * 110 = 0.0 kip

  Arm = *L / 2 =* 2.50 / 2 = 1.25 ft Moment = 0.0 * 1.25 = 0.0 k-ft

- Buoyancy = *0.6 * W * L * γ * (SC + Thick - WT) =* 0.6 * 2.50 * 2.50 * 62 * (0.67) = -0.2 kip

  Arm = *L / 2 =* 2.50 / 2 = 1.25 ft Moment = 0.2 * 1.25 = -0.2 k-ft

- Axial force P = 0.6 * 4.1 + 0.6 * 0.0 = 2.5 kip

  Arm = *L / 2 - Offset =* 2.50 / 2 - 0.0 / 12 = 1.25 ft Moment = 2.5 * 1.25 = 3.1 k-ft

- Resisting moment Z-Z = 0.5 + 0.0 + 0.0 + 3.1 + -0.2 = 3.3 k-ft

- Overturning safety factor Z-Z = $\dfrac{\textit{Resisting moment}}{\textit{Overturning moment}} = \dfrac{3.3}{0.0}$ = 33.49 > 1.50 OK

SOIL BEARING PRESSURES (Comb: D+L)

Overturning moment X-X = 0.0 + 0.0 = 0.0 k-ft

Resisting moment X-X = 0.8 + 0.0 + 0.0 + -0.3 + 11.6 = 12.1 k-ft

Overturning moment Z-Z = 0.0 + 0.0 = 0.0 k-ft

Resisting moment Z-Z = 0.8 + 0.0 + 0.0 + -0.3 + 11.6 = 12.1 k-ft

Resisting force = *Footing + Pedestal + Soil - Buoyancy + P =* 0.6 + 0.0 + 0.0 - 0.3 + 9.3 = 9.7 kip

X-coordinate of resultant from maximum bearing corner:

Xp = $\dfrac{\textit{Z-Resisting moment - Z-Overturning moment}}{\textit{Resisting force}} = \dfrac{12.1 - 0.0}{9.7}$ = 1.25 ft

Z-coordinate of resultant from maximum bearing corner:

Zp = $\dfrac{\textit{X-Resisting moment - X-Overturning moment}}{\textit{Resisting force}} = \dfrac{12.1 - 0.0}{9.7}$ = 1.25 ft

X-ecc = *Length / 2 - Xp =* 2.50 / 2 - 1.25 = 0.00 ft

Z-ecc = *Width / 2 - Zp =* 2.50 / 2 - 1.25 = 0.00 ft

Area = *Width * Length =* 2.50 * 2.50 = 6.3 ft²

Sx = *Length * Width² / 6 =* 2.50 * 2.50² / 6 = 2.6 ft³

Sz = *Width * Length² / 6 =* 2.50 * 2.50² / 6 = 2.6 ft³

- Footing is in full bearing. Soil pressures are as follows:

P1 = *P * (1/ A + Z-ecc / Sx + X-ecc / Sz) =* 9.7 * (1 / 6.3 + 0.00 / 2.6 + 0.00 / 2.6) = 1.55 ksf

P2 = *P * (1/ A - Z-ecc / Sx + X-ecc / Sz) =* 9.7 * (1 / 6.3 - 0.00 / 2.6 + 0.00 / 2.6) = 1.55 ksf

P3 = *P * (1/ A - Z-ecc / Sx - X-ecc / Sz) =* 9.7 * (1 / 6.3 - 0.00 / 2.6 - 0.00 / 2.6) = 1.55 ksf

P4 = *P * (1/ A + Z-ecc / Sx - X-ecc / Sz) =* 9.7 * (1 / 6.3 + 0.00 / 2.6 - 0.00 / 2.6) = 1.55 ksf

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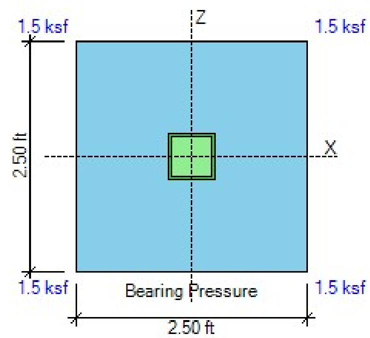
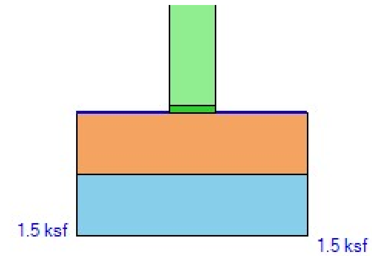




SLIDING CALCULATIONS (Comb: 0.6D+0.6W)

Internal friction angle = 28.0 deg

Passive coefficient *kp = 4.33* *(per Coulomb)*

Pressure at mid-depth = *kp * Density * (Cover + Thick / 2) =* 4.33 * 110 * (0.00 + 8.0 / 12 / 2) = 0.16 ksf

X-Passive force = *Pressure * Thick * Width =* 0.16 * 8.0 / 12 * 2.50 = 0.3 kip

Z-Passive force = *Pressure * Thick * Length =* 0.16 * 8.0 / 12 * 2.50 = 0.3 kip

Friction force = *Resisting force * Friction coeff. =* Max (0, 2.7 * 0.35) = 0.9 kip

Use 100% of Passive + 100% of Friction for sliding resistance

- Sliding safety factor X-X = $\dfrac{\textit{X-Passive force + Friction}}{\textit{X-Horizontal load}} = \dfrac{1.00 * 0.3 + 1.00 * 0.9}{0.0}$ = 12.02 > 1.50 OK

- Sliding safety factor Z-Z = $\dfrac{\textit{Z-Passive force + Friction}}{\textit{Z-Horizontal load}} = \dfrac{1.00 * 0.3 + 1.00 * 0.9}{0.0}$ = 12.02 > 1.50 OK

UPLIFT CALCULATIONS (Comb: 0.6D+0.6W)

- Uplift safety factor = $\dfrac{\textit{Pedestal + Footing + Cover - Buoyancy}}{\textit{Uplift load}} = \dfrac{0.0 + 0.4 + 0.0 - 0.2}{0.0}$ = 99.99 > 1.00 OK

ONE-WAY SHEAR CALCULATIONS (Comb: 1.2D+1.6L+0.5S)

Concrete f'c = 2.5 ksi   Steel fy = 40.0 ksi   Soil density = 110 pcf

Use Plain Concrete Shear Strength

ϕVcx = *4/3 * ϕ * √(f'c) * Width * t / 1000* = 4/3 * 0.60 * √(2500) * 2.5 * 12 * 8.0 / 1000 = 9.6 kip   ACI 14.5.5.1

ϕVcz = *4/3 * ϕ * √(f'c) * Length * t / 1000* = 4/3 * 0.60 * √(2500) * 2.5 * 12 * 8.0 / 1000 = 9.6 kip

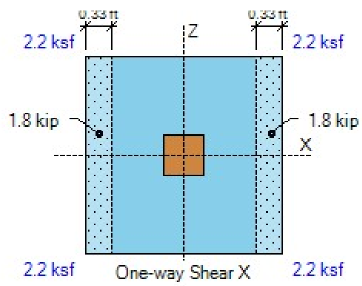
- Shear forces calculated as the volume of the bearing pressures under the effective areas:

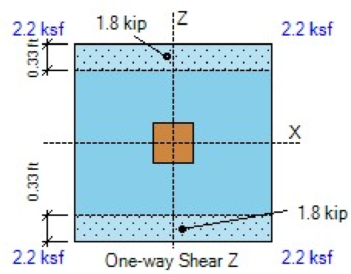
One-way shear Vux (- Side) = 1.8 kip < 9.6 kip OK

One-way shear Vux (+ Side) = 1.8 kip < 9.6 kip OK

One-way shear Vuz (- Side) = 1.8 kip < 9.6 kip OK

One-way shear Vuz (+ Side) = 1.8 kip < 9.6 kip OK





FLEXURE CALCULATIONS (Comb: 1.2D+1.6L+0.5S)

Plain ϕMnx $= 5 * \phi * \sqrt{f'c} * L * Thick^2 / 6 = 5 * 0.60 * \sqrt{2500} * 2.50 * 8.0^2 / 6 / 1000 = 1.1$ k-ft ACI Eq. (14.5.2.1a)

Plain ϕMnz $= 5 * \phi * \sqrt{f'c} * W * Thick^2 / 6 = 5 * 0.60 * \sqrt{2500} * 2.50 * 8.0^2 / 6 / 1000 = 1.1$ k-ft

- Top Bars

No Top Reinforcement Provided at the Footing

Use Plain Concrete Flexural Strength at Top

- Top moments calculated as the overburden minus the bearing pressures times the lever arm:

Top moment -Mux (- Side) = 0.0 k-ft < 4.0 k-ft OK

Top moment -Mux (+ Side) = 0.0 k-ft < 4.0 k-ft OK

Top moment -Muz (- Side) = 0.0 k-ft < 4.0 k-ft OK

Top moment -Muz (+ Side) = 0.0 k-ft < 4.0 k-ft OK

- Bottom Bars

No Bottom Reinforcement Provided at the Footing

Use Plain Concrete Flexural Strength at Bottom

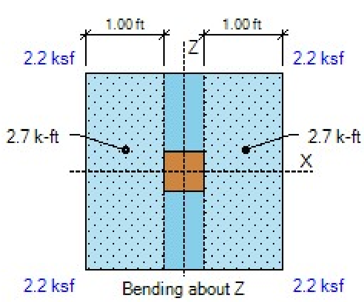
- Bottom moments calculated as the bearing minus the overburden pressures times the lever arm:

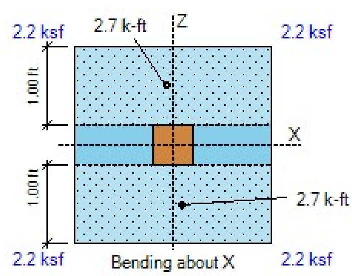
Bottom moment Mux (- Side) = 2.7 k-ft < 4.0 k-ft OK ratio = 0.66

Bottom moment Mux (+ Side) = 2.7 k-ft < 4.0 k-ft OK ratio = 0.66

Bottom moment Muz (- Side) = 2.7 k-ft < 4.0 k-ft OK ratio = 0.66

Bottom moment Muz (+ Side) = 2.7 k-ft < 4.0 k-ft OK ratio = 0.66





LOAD TRANSFER CALCULATIONS (Comb: 1.2D+1.6L+0.5S)

Area *A1 = col L * col W* = 6.0 * 6.0 = 36.0 in²

Sx = *col W * col L² / 6* = 6.0 * 6.0² / 6 = 36.0 in³

Sz = *col L * col W² / 6* = 6.0 * 6.0² / 6 = 36.0 in³

Bearing *Pbu = P / A1 + Mz / Sx + Mx / Sz* = 13.2 / 36.0 + 0.0 * 12 / 36.0 + 0.0 * 12 / 36.0 = 0.4 ksi

Min edge = *Min (L / 2 - X-offset - col L / 2, W / 2 - Z-offset - col W / 2)*

Min edge = Min (2.50 * 12 / 2 - 0.0 - 6.0 / 2, 2.50 * 12 / 2 - 0.0 - 6.0 / 2 = 12.0 in

Area *A2 = Min [L * W, (col L + 2 * Min edge) * (col W + 2 * Min edge)]* ACI R22.8.3.2

A2 = Min [2.50 * 12 * 2.5 * 12, (6.0 + 2 * 12.0) * (6.0 + 2 * 12.0)] = 900.0 in²

Footing *ϕPnc = ϕ * 0.85 * f'c * Min [2, √(A2 / A1)]* = 0.65 * 0.85 * 2.5 * Min [2, √(900.0 / 36.0)] = 2.8 ksi

Footing *ϕPns = ϕ * As * Fy / A1* = 0.0 ksi ACI 22.8.3.2

Footing bearing *ϕPn = ϕPnc + ϕPns* = 2.8 + 0.0 = 2.8 ksi > 0.4 psi OK

Hooked *Ldh = Max (8 db, 6, 0.02 * fy / (f'c)½ * Confining * Location * Concrete * db * ratio)* ACI 25.4.3

Ldh = Max (8 db, 6, 0.02 * 60.0 * 1000 / (2500)½ * 1.0 * 0.7 * 0.0 * 0.75 * 0.06) = 6.0 in

Ld provided = *Dowel length =* 3.00 * 12 = 36.0 in > 12.0 in OK

Ldh provided = *Footing thickness - Cover =* 8.00 - 3.0 = 5.0 in < 6.0 in NG

PUNCHING SHEAR CALCULATIONS (Comb: 1.2D+1.6L+0.5Lr)

X-Edge = *Length / 2 - Offset - Col / 2 =* 2.50 * 12 / 2 - 0.0 - 6.0 / 2 = 12.0 in αsx = 10

Z-Edge = *Width / 2 - Offset - Col / 2 =* 2.50 * 12 / 2 - 0.0 - 6.0 / 2 = 12.0 in αsz = 10

αs = *αsx + αsz =* 10 + 10 = 20 Col type = Corner β = *L / W =* 6.0 / 6.0 = 1.00 ACI 22.6.5.2

Perimeter *bo = αsz / 10 * (L + d / 2 + X-Edge) + αsx / 10 * (W + d / 2 + Z-Edge)* ACI 22.6.4.2

bo = 10 / 10 * (6.0 + 8.0 / 2 + 12.0) + 10 / 10 * (6.0 + 8.0 / 2 + 12.0) = 44.0 in

Area *Abo = (L + d / 2 + X-Edge) * (W + d / 2 + Z-Edge) =* (6.0 + 8.0 / 2 + 12.0) * (6.0 + 8.0 / 2 + 12.0) = 484.0 in²

Use Plain Concrete Shear Strength

ϕVc = *ϕ * Min (1 + 2 / β, 2) * 4/3 * √(f'c)* ACI 14.5.5.1

ϕVc = 0.60 * Min (1 + 2 / 1.00, 2) * 4/3 √(2500) = 80.0 psi

Punching force *F = P + Overburden * Abo - Bearing*

F = 13.2 + 0.07 * 484.0 / 144 - 3.0 = 10.5 kip

b1 = *L + d / 2 + X-Edge =* 6.0 + 8.0 / 2 + 12.0 = 22.0 in b2 = *W + d / 2 + Z-Edge =* 6.0 + 8.0 / 2 + 12.0 = 22.0 in

$$\gamma vx \text{ factor} = 1 - \frac{1}{1 + (2/3)\sqrt{b2 / b1}} = 1 - \frac{1}{1 + (2/3)\sqrt{22.0 / 22.0}} = 0.40$$ ACI Eq. (8.4.4.2.2)

$$\gamma vz \text{ factor} = 1 - \frac{1}{1 + (2/3)\sqrt{b1 / b2}} = 1 - \frac{1}{1 + (2/3)\sqrt{22.0 / 22.0}} = 0.40$$ ACI Eq. (8.4.2.3.2)

X2z = *b1² / 2 / (b1 + b2) =* 22.0² / 2 / (22.0 + 22.0) = 5.5 in X2x = *b2² / 2 / (b2 + b1) =* 5.5 in

Jcz = *b1 * d³ / 12 + b1³ * d / 12 + b1 * d * (b1 / 2 - X2z)² + b2 * d * X2z²* ACI R8.4.4.2.3

Jcz = 22.0 * 8.0³ / 12 + 22.0³ * 8.0 / 12 + 22.0 * 8.0 * (22.0 / 2 * 5.5)² + 22.0 * 8.0 * 5.5² = 18685 in⁴

Jcx = *b2 * d³ / 12 + b2³ * d / 12 + b2 * d * (b2 / 2 - X2x)² + b1 * d * X2x²* ACI R8.4.4.2.3

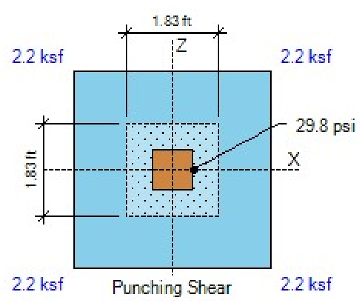
Jcz = 22.0 * 8.0³ / 12 + 22.0³ * 8.0 / 12 + 22.0 * 8.0 * (22.0 / 2 * 5.5)² + 22.0 * 8.0 * 5.5² = 18685 in⁴

Stress due to P = *F / (bo * d) * 1000 =* 10.5 / (44.0 * 8.0) * 1000 = 29.8 psi

Stress due to Mx = *γvx * X-OTM * X2x / Jcx =* 0.40 * 0.0 * 12 * 5.5 / 18685 * 1000 = 0.0 psi

Stress due to Mz = *γvz * Z-OTM * X2z / Jcz =* 0.40 * 0.0 * 12 * 5.5 / 18685 * 1000 = 0.0 psi

Punching stress = *P-stress + Mx-stress + Mz-stress =* 29.8 + 0.0 + 0.0 = 29.8 psi < 80.0 psi OK



LOAD TRANSFER CALCULATIONS (Comb: 1.2D+1.6L+0.5S)

Area *A1 = col L * col W* = 6.0 * 6.0 = 36.0 in²

Sx = *col W * col L² / 6* = 6.0 * 6.0² / 6 = 36.0 in³

Sz = *col L * col W² / 6* = 6.0 * 6.0² / 6 = 36.0 in³

Bearing *Pbu = P / A1 + Mz / Sx + Mx / Sz* = 13.2 / 36.0 + 0.0 * 12 / 36.0 + 0.0 * 12 / 36.0 = 0.4 ksi

Min edge = *Min (L / 2 - X-offset - col L / 2, W / 2 - Z-offset - col W / 2)*

Min edge = Min (2.50 * 12 / 2 - 0.0 - 6.0 / 2, 2.50 * 12 / 2 - 0.0 - 6.0 / 2 = 12.0 in

Area *A2 = Min [L * W, (col L + 2 * Min edge) * (col W + 2 * Min edge)]* ACI R22.8.3.2

A2 = Min [2.50 * 12 * 2.5 * 12, (6.0 + 2 * 12.0) * (6.0 + 2 * 12.0)] = 900.0 in²

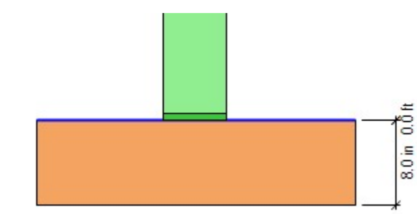
Footing *ϕPnc = ϕ * 0.85 * f'c * Min [2, √(A2 / A1)]* = 0.65 * 0.85 * 2.5 * Min [2, √(900.0 / 36.0)] = 2.8 ksi

Footing *ϕPns = ϕ * As * Fy / A1* = 0.0 ksi ACI 22.8.3.2

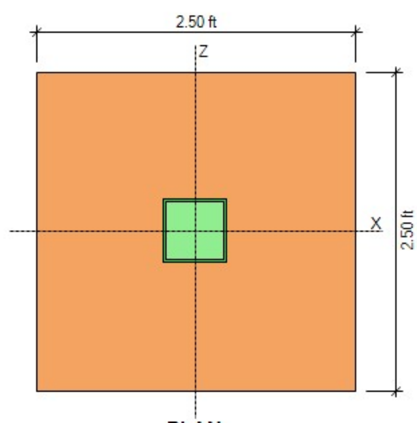
Footing bearing *ϕPn = ϕPnc + ϕPns* = 2.8 + 0.0 = 2.8 ksi > 0.4 psi OK

DESIGN CODES

Concrete Design	ACI 318-14
Load Combinations	ASCE 7-10/16



ELEVATION



PLAN

SPREAD FOOTING DESIGN

GEOMETRY

Footing Length (X-dir)	3.00	ft	
Footing Width (Z-dir)	3.00	ft	
Footing Thickness	8.0	in	OK
Soil Cover	0.00	ft	
Column Length (X-dir)	6.0	in	
Column Width (Z-dir)	6.0	in	
Offset (X-dir)	0.00	in	OK
Offset (Z-dir)	0.00	in	OK
Base Plate (L x W)	6.0 x 6.0	in	

SOIL PRESSURES (D+L)

Gross Allow. Soil Pressure	2.0	ksf
Soil Pressure at Corner 1	1.8	ksf
Soil Pressure at Corner 2	1.8	ksf
Soil Pressure at Corner 3	1.8	ksf
Soil Pressure at Corner 4	1.8	ksf
Bearing Pressure Ratio	0.90	OK
Ftg. Area in Contact with Soil	100.0	%
X-eccentricity / Ftg. Length	0.00	OK
Z-eccentricity / Ftg. Width	0.00	OK

APPLIED LOADS

	Dead	Live	RLive	Snow	Wind	Seismic	
Axial Force P	3.9	11.8	0.0	0.0	0.0	0.0	kip
Moment about X Mx ..	0.0	0.0	0.0	0.0	0.0	0.0	k-ft
Moment about Z Mz ..	0.0	0.0	0.0	0.0	0.0	0.0	k-ft
Shear Force Vx	0.0	0.0	0.0	0.0	0.0	0.0	kip
Shear Force Vz	0.0	0.0	0.0	0.0	0.0	0.0	kip

OVERTURNING CALCULATIONS (Comb: 0.6D+0.6W)

- Overturning about X-X

- Moment Mx = 0.6 * 0.0 + 0.6 * 0.0 = 0.0 k-ft
- Shear Force Vz = 0.6 * 0.0 + 0.6 * 0.0 = 0.0 kip
  Arm = 0.00 + 8.0 / 12 = 0.67 ft   Moment = 0.0 * 0.67 = 0.0 k-ft
- Passive Force = 0.0 kip   Arm = 0.27 ft   Moment = 0.0 k-ft
- Overturning moment X-X = 0.0 + 0.0 = 0.0 k-ft

- Resisting about X-X

- Footing weight = *0.6 * W * L * Thick * Density =* 0.6 * 3.00 * 3.00 * 8.0 / 12 * 0.15 = 0.5 kip
  Arm = *W / 2 =* 3.00 / 2 = 1.50 ft   Moment = 0.5 * 1.50 = 0.8 k-ft
- Pedestal weight = *0.6 * W * L * H * Density =* 0.6 * 6.0 / 12 * 6.0 / 12 * 0.0 * 0.15 = 0.0 kip
  Arm = *W / 2 - Offset =* 3.00 / 2 - 0.0 / 12 = 1.50 ft   Moment = 0.0 * 1.50 = 0.0 k-ft
- Soil cover = *0.6 * W * L * SC * Density =* 0.6 * (3.00 * 3.00 - 6.0 / 12 * 6.0 / 12) * 0.0 * 110 = 0.0 kip
  Arm = *W / 2 =* 3.00 / 2 = 1.50 ft   Moment = 0.0 * 1.50 = 0.0 k-ft
- Buoyancy = *0.6 * W * L * γ * (SC + Thick - WT) =* 0.6 * 3.00 * 3.00 * 62 * (0.67) = -0.2 kip
  Arm = *W / 2 =* 3.00 / 2 = 1.50 ft   Moment = 0.2 * 1.50 = -0.3 k-ft
- Axial force P = 0.6 * 3.9 + 0.6 * 0.0 = 2.3 kip
  Arm = *W / 2 - Offset =* 3.00 / 2 - 0.0 / 12 = 1.50 ft   Moment = 2.3 * 1.50 = 3.5 k-ft
- Resisting moment X-X = 0.8 + 0.0 + 0.0 + 3.5 + -0.3 = 4.0 k-ft

- Overturning safety factor X-X = $\frac{\textit{Resisting moment}}{\textit{Overturning moment}} = \frac{4.0}{0.0}$ = 39.83 > 1.50 OK

- Overturning about Z-Z

- Moment Mz = 0.6 * 0.0 + 0.6 * 0.0 = 0.0 k-ft
- Shear Force Vx = 0.6 * 0.0 + 0.6 * 0.0 = 0.0 kip
  Arm = 0.00 + 8.0 / 12 = 0.67 ft   Moment = 0.0 * 0.67 = 0.0 k-ft
- Passive Force = 0.0 kip   Arm = 0.27 ft   Moment = 0.0 k-ft
- Overturning moment Z-Z = 0.0 + 0.0 = 0.0 k-ft

- Resisting about Z-Z

- Footing weight = *0.6 * W * L * Thick * Density =* 0.6 * 3.00 * 3.00 * 8.0 / 12 * 0.15 = 0.5 kip
  Arm = *L / 2 =* 3.00 / 2 = 1.50 ft   Moment = 0.5 * 1.50 = 0.8 k-ft
- Pedestal weight = *0.6 * W * L * H * Density =* 0.6 * 6.0 / 12 * 6.0 / 12 * 0.0 * 0.15 = 0.0 kip
  Arm = *L / 2 - Offset =* 3.00 / 2 - 0.0 / 12 = 1.50 ft   Moment = 0.0 * 1.50 = 0.0 k-ft
- Soil cover = *0.6 * W * L * SC * Density =* 0.6 * (3.00 * 3.00 - 6.0 / 12 * 6.0 / 12) * 0.0 * 110 = 0.0 kip
  Arm = *L / 2 =* 3.00 / 2 = 1.50 ft   Moment = 0.0 * 1.50 = 0.0 k-ft
- Buoyancy = *0.6 * W * L * γ * (SC + Thick - WT) =* 0.6 * 3.00 * 3.00 * 62 * (0.67) = -0.2 kip
  Arm = *L / 2 =* 3.00 / 2 = 1.50 ft   Moment = 0.2 * 1.50 = -0.3 k-ft
- Axial force P = 0.6 * 3.9 + 0.6 * 0.0 = 2.3 kip
  Arm = *L / 2 - Offset =* 3.00 / 2 - 0.0 / 12 = 1.50 ft   Moment = 2.3 * 1.50 = 3.5 k-ft
- Resisting moment Z-Z = 0.8 + 0.0 + 0.0 + 3.5 + -0.3 = 4.0 k-ft

- Overturning safety factor Z-Z = $\dfrac{\textit{Resisting moment}}{\textit{Overturning moment}} = \dfrac{4.0}{0.0}$ = 39.83 > 1.50 OK

SOIL BEARING PRESSURES (Comb: D+L)

Overturning moment X-X = 0.0 + 0.0 = 0.0 k-ft

Resisting moment X-X = 1.4 + 0.0 + 0.0 + -0.6 + 23.6 = 24.3 k-ft

Overturning moment Z-Z = 0.0 + 0.0 = 0.0 k-ft

Resisting moment Z-Z = 1.4 + 0.0 + 0.0 + -0.6 + 23.6 = 24.3 k-ft

Resisting force = *Footing + Pedestal + Soil - Buoyancy + P =* 0.9 + 0.0 + 0.0 - 0.4 + 15.7 = 16.2 kip

X-coordinate of resultant from maximum bearing corner:

$$Xp = \frac{\textit{Z-Resisting moment - Z-Overturning moment}}{\textit{Resisting force}} = \frac{24.3 - 0.0}{16.2} = 1.50 \text{ ft}$$

Z-coordinate of resultant from maximum bearing corner:

$$Zp = \frac{\textit{X-Resisting moment - X-Overturning moment}}{\textit{Resisting force}} = \frac{24.3 - 0.0}{16.2} = 1.50 \text{ ft}$$

X-ecc = *Length / 2 - Xp =* 3.00 / 2 - 1.50 = 0.00 ft

Z-ecc = *Width / 2 - Zp =* 3.00 / 2 - 1.50 = 0.00 ft

Area = *Width * Length =* 3.00 * 3.00 = 9.0 ft²

Sx = *Length * Width² / 6 =* 3.00 * 3.00² / 6 = 4.5 ft³

Sz = *Width * Length² / 6 =* 3.00 * 3.00² / 6 = 4.5 ft³

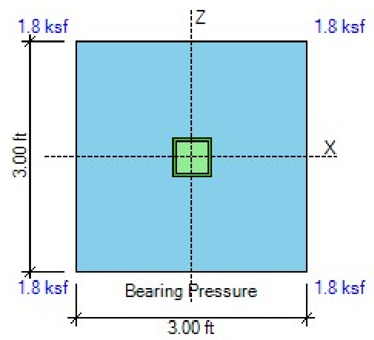
- Footing is in full bearing. Soil pressures are as follows:

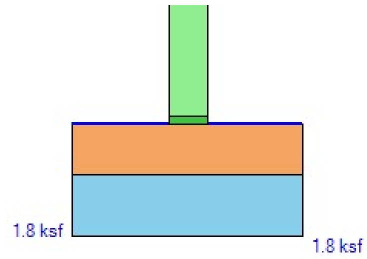
P1 = *P * (1/ A + Z-ecc / Sx + X-ecc / Sz) =* 16.2 * (1 / 9.0 + 0.00 / 4.5 + 0.00 / 4.5) = 1.80 ksf

P2 = *P * (1/ A - Z-ecc / Sx + X-ecc / Sz) =* 16.2 * (1 / 9.0 - 0.00 / 4.5 + 0.00 / 4.5) = 1.80 ksf

P3 = *P * (1/ A - Z-ecc / Sx - X-ecc / Sz) =* 16.2 * (1 / 9.0 - 0.00 / 4.5 - 0.00 / 4.5) = 1.80 ksf

P4 = *P * (1/ A + Z-ecc / Sx - X-ecc / Sz) =* 16.2 * (1 / 9.0 + 0.00 / 4.5 - 0.00 / 4.5) = 1.80 ksf





SLIDING CALCULATIONS (Comb: 0.6D+0.6W)

Internal friction angle = 28.0 deg

Passive coefficient *kp = 4.33* *(per Coulomb)*

Pressure at mid-depth = *kp * Density * (Cover + Thick / 2) =* 4.33 * 110 * (0.00 + 8.0 / 12 / 2) = 0.16 ksf

X-Passive force = *Pressure * Thick * Width =* 0.16 * 8.0 / 12 * 3.00 = 0.3 kip

Z-Passive force = *Pressure * Thick * Length =* 0.16 * 8.0 / 12 * 3.00 = 0.3 kip

Friction force = *Resisting force * Friction coeff. =* Max (0, 2.7 * 0.35) = 0.9 kip

Use 100% of Passive + 100% of Friction for sliding resistance

- Sliding safety factor X-X = $\dfrac{\textit{X-Passive force + Friction}}{\textit{X-Horizontal load}} = \dfrac{1.00 * 0.3 + 1.00 * 0.9}{0.0}$ = 12.47 > 1.50 OK

- Sliding safety factor Z-Z = $\dfrac{\textit{Z-Passive force + Friction}}{\textit{Z-Horizontal load}} = \dfrac{1.00 * 0.3 + 1.00 * 0.9}{0.0}$ = 12.47 > 1.50 OK

UPLIFT CALCULATIONS (Comb: 0.6D+0.6W)

- Uplift safety factor = $\dfrac{\textit{Pedestal + Footing + Cover - Buoyancy}}{\textit{Uplift load}} = \dfrac{0.0 + 0.5 + 0.0 - 0.2}{0.0}$ = 99.99 > 1.00 OK

ONE-WAY SHEAR CALCULATIONS (Comb: 1.2D+1.6L+0.5S)

Concrete f'c = 2.5 ksi | Steel fy = 40.0 ksi | Soil density = 110 pcf

d Top X-dir = *Thick - Cover - X-diameter / 2 =* 8.0 - 2.0 - 0.8 / 2 = 5.6 in

d Top Z-dir = *Thick - Cover - X-diameter - Z-diameter / 2 =* 8.0 - 2.0 - 0.8 - 0.8 / 2 = 4.9 in

d Bot X-dir = *Thick - Cover - X-diameter / 2 =* 8.0 - 3.0 - 0.5 / 2 = 4.8 in

d Bot Z-dir = *Thick - Cover - X-diameter - Z-diameter / 2 =* 8.0 - 3.0 - 0.5 - 0.5 / 2 = 4.3 in

*ϕVcx = 2 * ϕ * √(f'c) * Width * d / 1000 =*2 * 0.75 * √(2500) * 3.0 * 12 * 4.8 / 1000 = 12.8 kip ACI Eq. (22.5.5.1)

*ϕVcz = 2 * ϕ * √(f'c) * Length * d / 1000 =*2 * 0.75 * √(2500) * 3.0 * 12 * 4.3 / 1000 = 11.5 kip

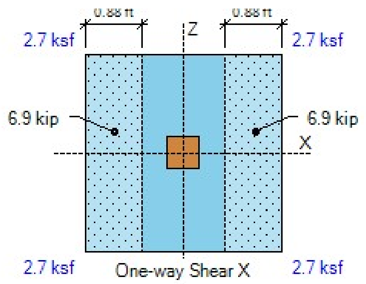
- Shear forces calculated as the volume of the bearing pressures under the effective areas:

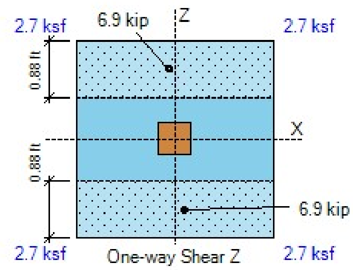
One-way shear Vux (- Side) = 6.9 kip < 12.8 kip OK

One-way shear Vux (+ Side) = 6.9 kip < 12.8 kip OK

One-way shear Vuz (- Side) = 6.9 kip < 11.5 kip OK

One-way shear Vuz (+ Side) = 6.9 kip < 11.5 kip OK





FLEXURE CALCULATIONS (Comb: 1.2D+1.6L+0.5S)

Plain ϕMnx = *5 * ϕ * √(f'c) * L * Thick^2 / 6* = 5 * 0.60 * √(2500) * 3.00 * 8.0^2 / 6 / 1000 = 1.3 k-ft ACI Eq. (14.5.2.1a)

Plain ϕMnz = *5 * ϕ * √(f'c) * W * Thick^2 / 6* = 5 * 0.60 * √(2500) * 3.00 * 8.0^2 / 6 / 1000 = 1.3 k-ft

- Top Bars

No Top Reinforcement Provided at the Footing

Use Plain Concrete Flexural Strength at Top

- Top moments calculated as the overburden minus the bearing pressures times the lever arm:

Top moment -Mux (- Side) = 0.0 k-ft < 4.8 k-ft OK

Top moment -Mux (+ Side) = 0.0 k-ft < 4.8 k-ft OK

Top moment -Muz (- Side) = 0.0 k-ft < 4.8 k-ft OK

Top moment -Muz (+ Side) = 0.0 k-ft < 4.8 k-ft OK

- Bottom Bars

Use 5 #4 Z-Bars *ρ = As / b d* = 1.0 / (3.00 * 12 * 4.3) = 0.0065 q = 0.0065 * 40 / 2.5 = 0.105

Use 5 #4 X-Bars *ρ = As / b d* = 1.0 / (3.00 * 12 * 4.8) = 0.0058 q = 0.0058 * 40 / 2.5 = 0.094

β = L / W = 3.00 / 3.00 = 1.00 *γs = 2 * β / (β + 1)* = 2 * 1.00 / (1.00 + 1) = 1.00 ACI 13.3.3.3

Bending strength *ϕMn = ϕ * b * d^2 * f'c * q * (1 - 0.59 * q)* ACI 22.2.2

ϕMnx = 0.90 * 3.00 * 12 * 4.3^2 * 2.5 * 0.105 * (1 - 0.59 * 0.105) = 12.0 k-ft

ϕMnz = 0.90 * 3.00 * 12 * 4.8^2 * 2.5 * 0.094 / 1.00 * (1 - 0.59 * 0.094 / 1.00) = 13.5 k-ft

- Bottom moments calculated as the bearing minus the overburden pressures times the lever arm:

Bottom moment Mux (- Side) = 6.2 k-ft < 12.0 k-ft OK ratio = 0.51

Bottom moment Mux (+ Side) = 6.2 k-ft < 12.0 k-ft OK ratio = 0.51

Bottom moment Muz (- Side) = 6.2 k-ft < 13.5 k-ft OK ratio = 0.46

Bottom moment Muz (+ Side) = 6.2 k-ft < 13.5 k-ft OK ratio = 0.46

X-As min = *0.0018 * Width * Thick* = 0.0018 * 3.00 * 12 * 8.0 = 0.5 in^2 < 1.0 in^2 OK ACI 8.6.1.1

Z-As min = *0.0018 * Length * Thick* = 0.0018 * 3.00 * 12 * 8.0 = 0.5 in^2 < 1.0 in^2 OK ACI 8.6.1.1

X-As max for 0.005 tension strain = 2.74 in^2 > 1.00 in^2 OK ACI 21.2.2

Z-As max for 0.005 tension strain = 2.74 in^2 > 1.00 in^2 OK ACI 21.2.2

X-Cover factor = *Min (2.5, (Cover + db /2, Spacing / 2) / db)* = Min (2.5, (3.0 + 0.50 / 2, 7.5 / 2) / 0.50) = 2.5

Straight *X-Ld = Max (12.0, 3 / 40 * fy / (f'c)½ * Grade * Size * Casting / Cover * db * ratio)* ACI Eq. (25.4.2.3a)

X-Ld = Max (12.0, 3 / 40 * 40.0 * 1000 / (2500)½ * 1.0 * 0.8 * 1.0 / 2.5 * 0.50 * 0.46) = 12.0 in

Hooked *X-Ldh = Max (8 db, 6, 0.02 * fy / (f'c)½ * Confining * Location * Concrete * db * ratio) =* ACI 25.4.3

X-Ldh = Max (8 db, 6, 0.02 * 40.0 * 1000 / (2500)½ * 1.0 * 0.7 * 0.0 * 0.50 * 0.46) = 6.0 in

-X Ld provided = *(Length - Col) / 2 + Offset - Cover* = 3.00 * 12 / 2 + 0.0 - 6.0 / 2 - 2.5 = 12.5 in > 12.0 in OK

+X Ld provided = *(Length - Col) / 2 - Offset - Cover* = 3.00 * 12 / 2 - 0.0 - 6.0 / 2 - 2.5 = 12.5 in > 12.0 in OK

Z-Cover factor = *Min (2.5, (Cover + db /2, Spacing / 2) / db) =* Min (2.5, (3.0 + 0.50 / 2, 7.5 / 2) / 0.50) = 2.5

Straight *Z-Ld = Max (12.0, 3 / 40 * fy / (f'c)½ * Grade * Size * Casting / Cover * db * ratio)* ACI Eq. (25.4.2.3a)

Z-Ld = Max (12.0, 3 / 40 * 40.0 * 1000 / (2500)½ * 1.0 * 0.8 * 1.0 / 2.5 * 0.50 * 0.46) = 12.0 in

Hooked *Z-Ldh = Max (8 db, 6, 0.02 * fy / (f'c)½ * Confining * Location * Concrete * db * ratio) =* ACI 25.4.3

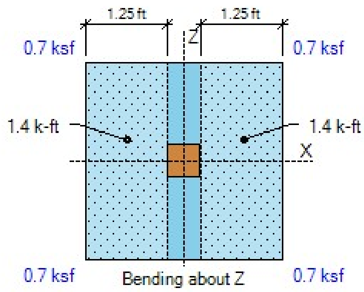
Z-Ldh = Max (8 db, 6, 0.02 * 40.0 * 1000 / (2500)½ * 1.0 * 0.7 * 0.0 * 0.50 * 0.51) = 6.0 in

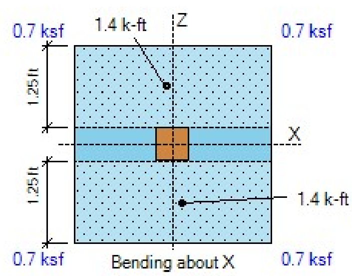
-Z Ld provided = *(Width - Col) / 2 + Offset - Cover =* 3.00 * 12 / 2 + 0.0 - 6.0 / 2 - 2.5 = 12.5 in > 12.0 in OK

+Z Ld provided =*(Width - Col) / 2 - Offset - Cover =* 3.00 * 12 / 2 - 0.0 - 6.0 / 2 - 2.5 = 12.5 in > 12.0 in OK

X-bar spacing = 7.5 in < Min (3 * t, 18.0) = 18.0 in OK ACI 7.7.2.3

Z-bar spacing = 7.5 in < Min (3 * t, 18.0) = 18.0 in OK





LOAD TRANSFER CALCULATIONS (Comb: 1.2D+1.6L+0.5S)

Area *A1 = col L * col W =* 6.0 * 6.0 = 36.0 in²

Sx = *col W * col L² / 6 =* 6.0 * 6.0² / 6 = 36.0 in³

Sz = *col L * col W² / 6 =* 6.0 * 6.0² / 6 = 36.0 in³

Bearing *Pbu = P / A1 + Mz / Sx + Mx / Sz =* 23.6 / 36.0 + 0.0 * 12 / 36.0 + 0.0 * 12 / 36.0 = 0.7 ksi

Min edge = *Min (L / 2 - X-offset - col L / 2, W / 2 - Z-offset - col W / 2)*

Min edge = Min (3.00 * 12 / 2 - 0.0 - 6.0 / 2, 3.00 * 12 / 2 - 0.0 - 6.0 / 2 = 15.0 in

Area *A2 = Min [L * W, (col L + 2 * Min edge) * (col W + 2 * Min edge)]* ACI R22.8.3.2

A2 = Min [3.00 * 12 * 3.0 * 12, (6.0 + 2 * 15.0) * (6.0 + 2 * 15.0)] = 1296.0 in²

Footing *ϕPnc = ϕ * 0.85 * f'c * Min [2, √(A2 / A1)] =* 0.65 * 0.85 * 2.5 * Min [2, √(1296.0 / 36.0)] = 2.8 ksi

Footing *ϕPns = ϕ * As * Fy / A1 =* 0.0 ksi ACI 22.8.3.2

Footing bearing *ϕPn = ϕPnc + ϕPns =* 2.8 + 0.0 = 2.8 ksi > 0.7 psi OK

Hooked *Ldh = Max (8 db, 6, 0.02 * fy / (f'c)½ * Confining * Location * Concrete * db * ratio)* ACI 25.4.3

Ldh = Max (8 db, 6, 0.02 * 60.0 * 1000 / (2500)½ * 1.0 * 0.7 * 0.0 * 0.75 * 0.11) = 6.0 in

Ld provided = *Dowel length =* 3.00 * 12 = 36.0 in > 20.4 in OK

Ldh provided = *Footing thickness - Cover =* 8.00 - 3.0 = 5.0 in < 6.0 in NG

PUNCHING SHEAR CALCULATIONS (Comb: 1.2D+1.6L+0.5Lr)

X-Edge = *d / 2 =* 4.5 / 2 = 2.3 in αsx = 20

Z-Edge = *d / 2 =* 4.5 / 2 = 2.3 in αsz = 20

αs = *αsx + αsz =* 20 + 20 = 40 Col type = Interior β = *L / W =* 6.0 / 6.0 = 1.00 ACI 22.6.5.2

Perimeter *bo = αsz / 10 * (L + d / 2 + X-Edge) + αsx / 10 * (W + d / 2 + Z-Edge)* ACI 22.6.4.2

bo = 20 / 10 * (6.0 + 4.5 / 2 + 2.3) + 20 / 10 * (6.0 + 4.5 / 2 + 2.3) = 42.0 in

Area *Abo = (L + d / 2 + X-Edge) * (W + d / 2 + Z-Edge) =* (6.0 + 4.5 / 2 + 2.3) * (6.0 + 4.5 / 2 + 2.3) = 110.3 in²

ϕVc = *ϕ * Min (2 + 4 / β, αs * d / bo + 2, 4) * √(f'c)* ACI 22.6.5.2

ϕVc = 0.75 * Min (2 + 4 / 1.00, 40 * 4.5 / 42.0 + 2, 4) * √(2500) = 150.0 psi

Punching force *F = P + Overburden * Abo - Bearing*

F = 23.6 + 0.07 * 110.3 / 144 - 2.1 = 21.6 kip

b1 = *L + d / 2 + X-Edge =* 6.0 + 4.5 / 2 + 2.3 = 10.5 in b2 = *W + d / 2 + Z-Edge =* 6.0 + 4.5 / 2 + 2.3 = 10.5 in

γvx factor = $1 - \dfrac{1}{1 + (2/3)\sqrt{b2 / b1}} = 1 - \dfrac{1}{1 + (2/3)\sqrt{10.5 / 10.5}} = 0.40$ ACI Eq. (8.4.4.2.2)

γvz factor = $1 - \dfrac{1}{1 + (2/3)\sqrt{b1 / b2}} = 1 - \dfrac{1}{1 + (2/3)\sqrt{10.5 / 10.5}} = 0.40$ ACI Eq. (8.4.2.3.2)

X2z = *b1 / 2 =* 10.5 / 2 = 5.3 in X2x = *b2 / 2 =* 10.5 / 2 = 5.3 in

Jcz = $b1 \cdot d^3 / 6 + b1^3 \cdot d / 6 + b1^2 \cdot b2 \cdot d / 2$ ACI R8.4.4.2.3

Jcz = $10.5 \cdot 4.5^3 / 6 + 10.5^3 \cdot 4.5 / 6 + 10.5^2 \cdot 10.5 \cdot 4.5 / 2 = 3632\ in^4$

Jcx = $b2 \cdot d^3 / 6 + b2^3 \cdot d / 6 + b2^2 \cdot b1 \cdot d / 2$ ACI R8.4.4.2.3

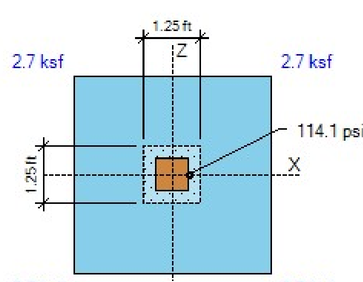
Jcx = $10.5 \cdot 4.5^3 / 6 + 10.5^3 \cdot 4.5 / 6 + 10.5^2 \cdot 10.5 \cdot 4.5 / 2 = 3632\ in^4$

Stress due to P = *F / (bo * d) * 1000 =* 21.6 / (42.0 * 4.5) * 1000 = 114.1 psi

Stress due to Mx = *γvx * X-OTM * X2x / Jcx =* 0.40 * 0.0 * 12 * 5.3 / 3632 * 1000 = 0.0 psi

Stress due to Mz = *γvz * Z-OTM * X2z / Jcz =* 0.40 * 0.0 * 12 * 5.3 / 3632 * 1000 = 0.0 psi

Punching stress = *P-stress + Mx-stress + Mz-stress =* 114.1 + 0.0 + 0.0 = 114.1 psi < 150.0 psi OK



Punching Shear

LOAD TRANSFER CALCULATIONS (Comb: 1.2D+1.6L+0.5S)

Area *A1 = col L * col W* = 6.0 * 6.0 = 36.0 in²

Sx = *col W * col L² / 6* = 6.0 * 6.0² / 6 = 36.0 in³

Sz = *col L * col W² / 6* = 6.0 * 6.0² / 6 = 36.0 in³

Bearing *Pbu = P / A1 + Mz / Sx + Mx / Sz* = 23.6 / 36.0 + 0.0 * 12 / 36.0 + 0.0 * 12 / 36.0 = 0.7 ksi

Min edge = *Min (L / 2 - X-offset - col L / 2, W / 2 - Z-offset - col W / 2)*

Min edge = Min (3.00 * 12 / 2 - 0.0 - 6.0 / 2, 3.00 * 12 / 2 - 0.0 - 6.0 / 2 = 15.0 in

Area *A2 = Min [L * W, (col L + 2 * Min edge) * (col W + 2 * Min edge)]* ACI R22.8.3.2

A2 = Min [3.00 * 12 * 3.0 * 12, (6.0 + 2 * 15.0) * (6.0 + 2 * 15.0)] = 1296.0 in²

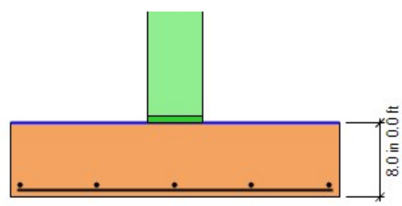
Footing *ϕPnc = ϕ * 0.85 * f'c * Min [2, √(A2 / A1)]* = 0.65 * 0.85 * 2.5 * Min [2, √(1296.0 / 36.0)] = 2.8 ksi

Footing *ϕPns = ϕ * As * Fy / A1* = 0.0 ksi ACI 22.8.3.2

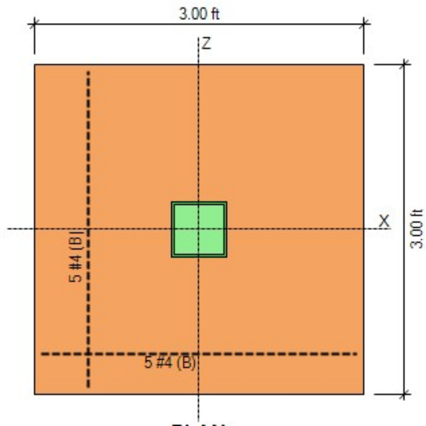
Footing bearing *ϕPn = ϕPnc + ϕPns* = 2.8 + 0.0 = 2.8 ksi > 0.7 psi OK

DESIGN CODES

Concrete Design	ACI 318-14
Load Combinations	ASCE 7-10/16



ELEVATION



PLAN

SPREAD FOOTING DESIGN

GEOMETRY

Footing Length (X-dir)	1.50	ft	
Footing Width (Z-dir)	2.60	ft	
Footing Thickness	8.0	in	OK
Soil Cover	0.00	ft	
Column Length (X-dir)	6.0	in	
Column Width (Z-dir)	6.0	in	
Offset (X-dir)	0.00	in	OK
Offset (Z-dir)	0.00	in	OK
Base Plate (L x W)	6.0 x 6.0	in	

SOIL PRESSURES (D+L)

Gross Allow. Soil Pressure	2.0	ksf	
Soil Pressure at Corner 1	2.0	ksf	
Soil Pressure at Corner 2	2.0	ksf	
Soil Pressure at Corner 3	2.0	ksf	
Soil Pressure at Corner 4	2.0	ksf	
Bearing Pressure Ratio	0.99		OK
Ftg. Area in Contact with Soil	100.0	%	
X-eccentricity / Ftg. Length	0.00		OK
Z-eccentricity / Ftg. Width	0.00		OK

APPLIED LOADS

	Dead	Live	RLive	Snow	Wind	Seismic	
Axial Force P	3.0	4.5	0.0	0.0	0.0	0.0	kip
Moment about X Mx ..	0.0	0.0	0.0	0.0	0.0	0.0	k-ft
Moment about Z Mz ..	0.0	0.0	0.0	0.0	0.0	0.0	k-ft
Shear Force Vx	0.0	0.0	0.0	0.0	0.0	0.0	kip
Shear Force Vz	0.0	0.0	0.0	0.0	0.0	0.0	kip

OVERTURNING CALCULATIONS (Comb: 0.6D+0.6W)

- Overturning about X-X

- Moment Mx = 0.6 * 0.0 + 0.6 * 0.0 = 0.0 k-ft

- Shear Force Vz = 0.6 * 0.0 + 0.6 * 0.0 = 0.0 kip

Arm = 0.00 + 8.0 / 12 = 0.67 ft — Moment = 0.0 * 0.67 = 0.0 k-ft

- Passive Force = 0.0 kip — Arm = 0.27 ft — Moment = 0.0 k-ft

- Overturning moment X-X = 0.0 + 0.0 = 0.0 k-ft

- Resisting about X-X

- Footing weight = *0.6 * W * L * Thick * Density =* 0.6 * 2.60 * 1.50 * 8.0 / 12 * 0.15 = 0.2 kip

Arm = *W / 2 =* 2.60 / 2 = 1.30 ft — Moment = 0.2 * 1.30 = 0.3 k-ft

- Pedestal weight = *0.6 * W * L * H * Density =* 0.6 * 6.0 / 12 * 6.0 / 12 * 0.0 * 0.15 = 0.0 kip

Arm = *W / 2 - Offset =* 2.60 / 2 - 0.0 / 12 = 1.30 ft — Moment = 0.0 * 1.30 = 0.0 k-ft

- Soil cover = *0.6 * W * L * SC * Density =* 0.6 * (2.60 * 1.50 - 6.0 / 12 * 6.0 / 12) * 0.0 * 110 = 0.0 kip

Arm = *W / 2 =* 2.60 / 2 = 1.30 ft — Moment = 0.0 * 1.30 = 0.0 k-ft

- Buoyancy = *0.6 * W * L * γ * (SC + Thick - WT) =* 0.6 * 2.60 * 1.50 * 62 * (0.67) = -0.1 kip

Arm = *W / 2 =* 2.60 / 2 = 1.30 ft — Moment = 0.1 * 1.30 = -0.1 k-ft

- Axial force P = 0.6 * 3.0 + 0.6 * 0.0 = 1.8 kip

Arm = *W / 2 - Offset =* 2.60 / 2 - 0.0 / 12 = 1.30 ft — Moment = 1.8 * 1.30 = 2.3 k-ft

- Resisting moment X-X = 0.3 + 0.0 + 0.0 + 2.3 + -0.1 = 2.5 k-ft

- Overturning safety factor X-X = $\dfrac{\textit{Resisting moment}}{\textit{Overturning moment}} = \dfrac{2.5}{0.0}$ = 25.18 > 1.50 OK

- Overturning about Z-Z

- Moment Mz = 0.6 * 0.0 + 0.6 * 0.0 = 0.0 k-ft
- Shear Force Vx = 0.6 * 0.0 + 0.6 * 0.0 = 0.0 kip
  Arm = 0.00 + 8.0 / 12 = 0.67 ft   Moment = 0.0 * 0.67 = 0.0 k-ft
- Passive Force = 0.0 kip   Arm = 0.27 ft   Moment = 0.0 k-ft
- Overturning moment Z-Z = 0.0 + 0.0 = 0.0 k-ft

- Resisting about Z-Z

- Footing weight = *0.6 * W * L * Thick * Density =* 0.6 * 2.60 * 1.50 * 8.0 / 12 * 0.15 = 0.2 kip
  Arm = *L / 2 =* 1.50 / 2 = 0.75 ft   Moment = 0.2 * 0.75 = 0.2 k-ft
- Pedestal weight = *0.6 * W * L * H * Density =* 0.6 * 6.0 / 12 * 6.0 / 12 * 0.0 * 0.15 = 0.0 kip
  Arm = *L / 2 - Offset =* 1.50 / 2 - 0.0 / 12 = 0.75 ft   Moment = 0.0 * 0.75 = 0.0 k-ft
- Soil cover = *0.6 * W * L * SC * Density =* 0.6 * (2.60 * 1.50 - 6.0 / 12 * 6.0 / 12) * 0.0 * 110 = 0.0 kip
  Arm = *L / 2 =* 1.50 / 2 = 0.75 ft   Moment = 0.0 * 0.75 = 0.0 k-ft
- Buoyancy = *0.6 * W * L * γ * (SC + Thick - WT) =* 0.6 * 2.60 * 1.50 * 62 * (0.67) = -0.1 kip
  Arm = *L / 2 =* 1.50 / 2 = 0.75 ft   Moment = 0.1 * 0.75 = -0.1 k-ft
- Axial force P = 0.6 * 3.0 + 0.6 * 0.0 = 1.8 kip
  Arm = *L / 2 - Offset =* 1.50 / 2 - 0.0 / 12 = 0.75 ft   Moment = 1.8 * 0.75 = 1.4 k-ft
- Resisting moment Z-Z = 0.2 + 0.0 + 0.0 + 1.4 + -0.1 = 1.5 k-ft

- Overturning safety factor Z-Z = $\dfrac{\textit{Resisting moment}}{\textit{Overturning moment}} = \dfrac{1.5}{0.0}$ = 14.52 > 1.50 OK

SOIL BEARING PRESSURES (Comb: D+L)

Overturning moment X-X = 0.0 + 0.0 = 0.0 k-ft

Resisting moment X-X = 0.5 + 0.0 + 0.0 + -0.2 + 9.8 = 10.0 k-ft

Overturning moment Z-Z = 0.0 + 0.0 = 0.0 k-ft

Resisting moment Z-Z = 0.3 + 0.0 + 0.0 + -0.1 + 5.6 = 5.8 k-ft

Resisting force = *Footing + Pedestal + Soil - Buoyancy + P =* 0.4 + 0.0 + 0.0 - 0.2 + 7.5 = 7.7 kip

X-coordinate of resultant from maximum bearing corner:

$$Xp = \frac{\textit{Z-Resisting moment - Z-Overturning moment}}{\textit{Resisting force}} = \frac{5.8 - 0.0}{7.7} = 0.75 \text{ ft}$$

Z-coordinate of resultant from maximum bearing corner:

$$Zp = \frac{\textit{X-Resisting moment - X-Overturning moment}}{\textit{Resisting force}} = \frac{10.0 - 0.0}{7.7} = 1.30 \text{ ft}$$

X-ecc = *Length / 2 - Xp =*1.50 / 2 - 0.75 = 0.00 ft

Z-ecc = *Width / 2 - Zp =* 2.60 / 2 - 1.30 = 0.00 ft

Area = *Width * Length =* 2.60 * 1.50 = 3.9 ft²

Sx = *Length * Width² / 6 =* 1.50 * 2.60² / 6 = 1.7 ft³

Sz = *Width * Length² / 6 =* 2.60 * 1.50² / 6 = 1.0 ft³

- Footing is in full bearing. Soil pressures are as follows:

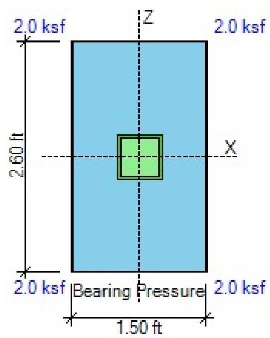
P1 = *P * (1/ A + Z-ecc / Sx + X-ecc / Sz) =* 7.7 * (1 / 3.9 + 0.00 / 1.7 + 0.00 / 1.0) = 1.98 ksf

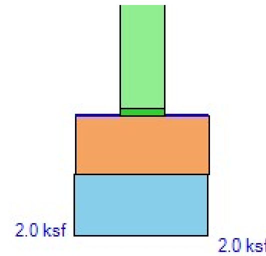
P2 = *P * (1/ A - Z-ecc / Sx + X-ecc / Sz) =* 7.7 * (1 / 3.9 - 0.00 / 1.7 + 0.00 / 1.0) = 1.98 ksf

P3 = *P * (1/ A - Z-ecc / Sx - X-ecc / Sz) =* 7.7 * (1 / 3.9 - 0.00 / 1.7 - 0.00 / 1.0) = 1.98 ksf

P4 = *P * (1/ A + Z-ecc / Sx - X-ecc / Sz) =* 7.7 * (1 / 3.9 + 0.00 / 1.7 - 0.00 / 1.0) = 1.98 ksf

SPREAD FOOTING DESIGN





SLIDING CALCULATIONS (Comb: 0.6D+0.6W)

Internal friction angle = 28.0 deg

Passive coefficient *kp = 4.33* *(per Coulomb)*

Pressure at mid-depth = *kp * Density * (Cover + Thick / 2) =* 4.33 * 110 * (0.00 + 8.0 / 12 / 2) = 0.16 ksf

X-Passive force = *Pressure * Thick * Width =* 0.16 * 8.0 / 12 * 2.60 = 0.3 kip

Z-Passive force = *Pressure * Thick * Length =* 0.16 * 8.0 / 12 * 1.50 = 0.2 kip

Friction force = *Resisting force * Friction coeff. =* Max (0, 1.9 * 0.35) = 0.7 kip

Use 100% of Passive + 100% of Friction for sliding resistance

$$\text{- Sliding safety factor X-X} = \frac{\textit{X-Passive force + Friction}}{\textit{X-Horizontal load}} = \frac{1.00 * 0.3 + 1.00 * 0.7}{0.0} = 9.53 \quad > 1.50 \quad \text{OK}$$

$$\text{- Sliding safety factor Z-Z} = \frac{\textit{Z-Passive force + Friction}}{\textit{Z-Horizontal load}} = \frac{1.00 * 0.2 + 1.00 * 0.7}{0.0} = 8.36 \quad > 1.50 \quad \text{OK}$$

UPLIFT CALCULATIONS (Comb: 0.6D+0.6W)

$$\text{- Uplift safety factor} = \frac{\textit{Pedestal + Footing + Cover - Buoyancy}}{\textit{Uplift load}} = \frac{0.0 + 0.2 + 0.0 - 0.1}{0.0} = 99.99 \quad > 1.00 \quad \text{OK}$$

ONE-WAY SHEAR CALCULATIONS (Comb: 1.2D+1.6L+0.5S)

Concrete f'c = 2.5 ksi Steel fy = 40.0 ksi Soil density = 110 pcf

Use Plain Concrete Shear Strength

ϕVcx = *4/3 * ϕ * √(f'c) * Width * t / 1000* = 4/3 * 0.60 * √(2500) * 2.6 * 12 * 8.0 / 1000 = 10.0 kip ACI 14.5.5.1

ϕVcz = *4/3 * ϕ * √(f'c) * Length * t / 1000* = 4/3 * 0.60 * √(2500) * 1.5 * 12 * 8.0 / 1000 = 5.8 kip

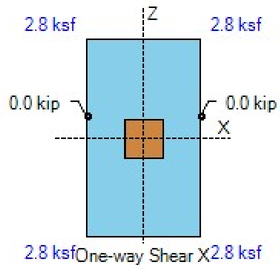
- Shear forces calculated as the volume of the bearing pressures under the effective areas:

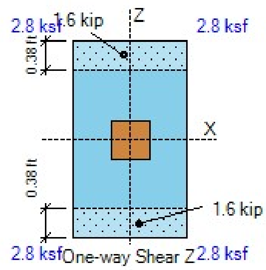
One-way shear Vux (- Side) = 0.0 kip < 10.0 kip OK

One-way shear Vux (+ Side) = 0.0 kip < 10.0 kip OK

One-way shear Vuz (- Side) = 1.6 kip < 5.8 kip OK

One-way shear Vuz (+ Side) = 1.6 kip < 5.8 kip OK





FLEXURE CALCULATIONS (Comb: 1.2D+1.6L+0.5S)

Plain ϕMnx = *5 * ϕ * √(f'c) * L * Thick² / 6* = 5 * 0.60 * √(2500) * 1.50 * 8.0² / 6 / 1000 = 0.6 k-ft ACI Eq. (14.5.2.1a)

Plain ϕMnz = *5 * ϕ * √(f'c) * W * Thick² / 6* = 5 * 0.60 * √(2500) * 2.60 * 8.0² / 6 / 1000 = 1.1 k-ft

- Top Bars

No Top Reinforcement Provided at the Footing

Use Plain Concrete Flexural Strength at Top

- Top moments calculated as the overburden minus the bearing pressures times the lever arm:

Top moment -Mux (- Side) = 0.0 k-ft < 2.4 k-ft OK

Top moment -Mux (+ Side) = 0.0 k-ft < 2.4 k-ft OK

Top moment -Muz (- Side) = 0.0 k-ft < 4.2 k-ft OK

Top moment -Muz (+ Side) = 0.0 k-ft < 4.2 k-ft OK

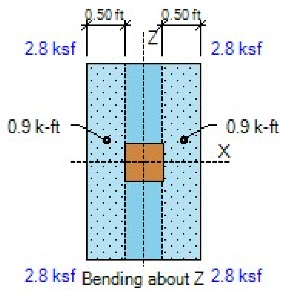
- Bottom Bars

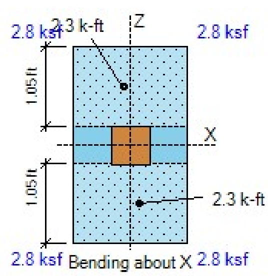
No Bottom Reinforcement Provided at the Footing

Use Plain Concrete Flexural Strength at Bottom

- Bottom moments calculated as the bearing minus the overburden pressures times the lever arm:

Bottom moment Mux (- Side) = 2.3 k-ft	< 2.4 k-ft	OK	ratio = 0.96
Bottom moment Mux (+ Side) = 2.3 k-ft	< 2.4 k-ft	OK	ratio = 0.96
Bottom moment Muz (- Side) = 0.9 k-ft	< 4.2 k-ft	OK	ratio = 0.22
Bottom moment Muz (+ Side) = 0.9 k-ft	< 4.2 k-ft	OK	ratio = 0.22





LOAD TRANSFER CALCULATIONS (Comb: 1.2D+1.6L+0.5S)

Area *A1 = col L * col W =* 6.0 * 6.0 = 36.0 in²

Sx = *col W * col L² / 6 =* 6.0 * 6.0² / 6 = 36.0 in³

Sz = *col L * col W² / 6 =* 6.0 * 6.0² / 6 = 36.0 in³

Bearing *Pbu = P / A1 + Mz / Sx + Mx / Sz =* 10.8 / 36.0 + 0.0 * 12 / 36.0 + 0.0 * 12 / 36.0 = 0.3 ksi

Min edge = *Min (L / 2 - X-offset - col L / 2, W / 2 - Z-offset - col W / 2)*

Min edge = Min (1.50 * 12 / 2 - 0.0 - 6.0 / 2, 2.60 * 12 / 2 - 0.0 - 6.0 / 2 = 6.0 in

Area *A2 = Min [L * W, (col L + 2 * Min edge) * (col W + 2 * Min edge)]* ACI R22.8.3.2

A2 = Min [1.50 * 12 * 2.6 * 12, (6.0 + 2 * 6.0) * (6.0 + 2 * 6.0)] = 324.0 in²

Footing *ϕPnc = ϕ * 0.85 * f'c * Min [2, √(A2 / A1)] =* 0.65 * 0.85 * 2.5 * Min [2, √(324.0 / 36.0)] = 2.8 ksi

Footing *ϕPns = ϕ * As * Fy / A1 =* 0.0 ksi ACI 22.8.3.2

Footing bearing *ϕPn = ϕPnc + ϕPns =* 2.8 + 0.0 = 2.8 ksi > 0.3 psi OK

Hooked *L_{dh} = Max (8 db, 6, 0.02 * fy / (f'c)½ * Confining * Location * Concrete * db * ratio)* — ACI 25.4.3

L_{dh} = Max (8 db, 6, 0.02 * 60.0 * 1000 / (2500)½ * 1.0 * 0.7 * 0.0 * 0.75 * 0.05) = 6.0 in

Ld provided = *Dowel length* = 3.00 * 12 = 36.0 in   > 12.0 in OK

Ldh provided = *Footing thickness - Cover* = 8.00 - 3.0 = 5.0 in   < 6.0 in NG

PUNCHING SHEAR CALCULATIONS (Comb: 1.2D+1.6L+0.5Lr)

X-Edge = *Length / 2 - Offset - Col / 2* = 1.50 * 12 / 2 - 0.0 - 6.0 / 2 = 6.0 in α_{sx} = 10

Z-Edge = *Width / 2 - Offset - Col / 2* = 2.60 * 12 / 2 - 0.0 - 6.0 / 2 = 12.6 in α_{sz} = 10

α_s = *α_{sx} + α_{sz}* = 10 + 10 = 20   Col type = Corner β = *L / W* = 6.0 / 6.0 = 1.00 — ACI 22.6.5.2

Perimeter *$b_o = \alpha_{sz} / 10 * (L + d / 2 + X\text{-}Edge) + \alpha_{sx} / 10 * (W + d / 2 + Z\text{-}Edge)$* — ACI 22.6.4.2

b_o = 10 / 10 * (6.0 + 8.0 / 2 + 6.0) + 10 / 10 * (6.0 + 8.0 / 2 + 12.6) = 38.6 in

Area *$A_{bo} = (L + d / 2 + X\text{-}Edge) * (W + d / 2 + Z\text{-}Edge)$* = (6.0 + 8.0 / 2 + 6.0) * (6.0 + 8.0 / 2 + 12.6) = 361.6 in²

Use Plain Concrete Shear Strength

ϕV_c = *ϕ * Min (1 + 2 / β, 2) * 4/3 * $\sqrt{f'c}$* — ACI 14.5.5.1

ϕV_c = 0.60 * Min (1 + 2 / 1.00, 2) * 4/3 $\sqrt{2500}$ = 80.0 psi

Punching force *F = P + Overburden * A_{bo} - Bearing*

F = 10.8 + 0.07 * 361.6 / 144 - 3.9 = 7.1 kip

b_1 = *L + d / 2 + X-Edge* = 6.0 + 8.0 / 2 + 6.0 = 16.0 in b_2 = *W + d / 2 + Z-Edge* = 6.0 + 8.0 / 2 + 12.6 = 22.6 in

$$\gamma_{vx} \text{ factor} = 1 - \frac{1}{1 + (2/3)\sqrt{b_2 / b_1}} = 1 - \frac{1}{1 + (2/3)\sqrt{22.6 / 16.0}} = 0.44$$

ACI Eq. (8.4.4.2.2)

ACI Eq. (8.4.2.3.2)

$$\gamma_{vz} \text{ factor} = 1 - \frac{1}{1 + (2/3)\sqrt{b_1 / b_2}} = 1 - \frac{1}{1 + (2/3)\sqrt{16.0 / 22.6}} = 0.36$$

X_{2z} = *$b_1^2 / 2 / (b_1 + b_2)$* = 16.0² / 2 / (16.0 + 22.6) = 3.3 in X_{2x} = *$b_2^2 / 2 / (b_2 + b_1)$* = 6.6 in

J_{cz} = *$b_1 * d^3 / 12 + b_1^3 * d / 12 + b_1 * d * (b_1 / 2 - X_{2z})^2 + b_2 * d * X_{2z}^2$* — ACI R8.4.4.2.3

J_{cz} = 16.0 * 8.0³ / 12 + 16.0³ * 8.0 / 12 + 16.0 * 8.0 * (16.0 / 2 * 3.3)² + 22.6 * 8.0 * 3.3² = 8210 in⁴

J_{cx} = *$b_2 * d^3 / 12 + b_2^3 * d / 12 + b_2 * d * (b_2 / 2 - X_{2x})^2 + b_1 * d * X_{2x}^2$* — ACI R8.4.4.2.3

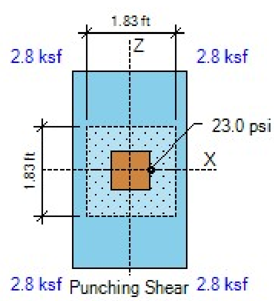
J_{cz} = 22.6 * 8.0³ / 12 + 22.6³ * 8.0 / 12 + 22.6 * 8.0 * (22.6 / 2 * 6.6)² + 16.0 * 8.0 * 6.6² = 18229 in⁴

Stress due to P = *F / (b_o * d) * 1000* = 7.1 / (38.6 * 8.0) * 1000 = 23.0 psi

Stress due to Mx = *γ_{vx} * X-OTM * X_{2x} / J_{cx}* = 0.44 * 0.0 * 12 * 6.6 / 18229 * 1000 = 0.0 psi

Stress due to Mz = *γ_{vz} * Z-OTM * X_{2z} / J_{cz}* = 0.44 * 0.0 * 12 * 3.3 / 8210 * 1000 = 0.0 psi

Punching stress = *P-stress + Mx-stress + Mz-stress* = 23.0 + 0.0 + 0.0 = 23.0 psi   < 80.0 psi OK



LOAD TRANSFER CALCULATIONS (Comb: 1.2D+1.6L+0.5S)

Area $A1 = col\ L * col\ W$ = 6.0 * 6.0 = 36.0 in²

Sx = $col\ W * col\ L^2 / 6$ = 6.0 * 6.0² / 6 = 36.0 in³

Sz = $col\ L * col\ W^2 / 6$ = 6.0 * 6.0² / 6 = 36.0 in³

Bearing $Pbu = P / A1 + Mz / Sx + Mx / Sz$ = 10.8 / 36.0 + 0.0 * 12 / 36.0 + 0.0 * 12 / 36.0 = 0.3 ksi

Min edge = *Min (L / 2 - X-offset - col L / 2, W / 2 - Z-offset - col W / 2)*

Min edge = Min (1.50 * 12 / 2 - 0.0 - 6.0 / 2, 2.60 * 12 / 2 - 0.0 - 6.0 / 2 = 6.0 in

Area *A2 = Min [L * W, (col L + 2 * Min edge) * (col W + 2 * Min edge)]* ACI R22.8.3.2

A2 = Min [1.50 * 12 * 2.6 * 12, (6.0 + 2 * 6.0) * (6.0 + 2 * 6.0)] = 324.0 in²

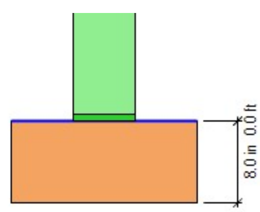
Footing $\phi Pnc = \phi * 0.85 * f'c * Min\ [2, \sqrt{(A2 / A1)}]$ = 0.65 * 0.85 * 2.5 * Min [2, √(324.0 / 36.0)] = 2.8 ksi

Footing $\phi Pns = \phi * As * Fy / A1$ = 0.0 ksi ACI 22.8.3.2

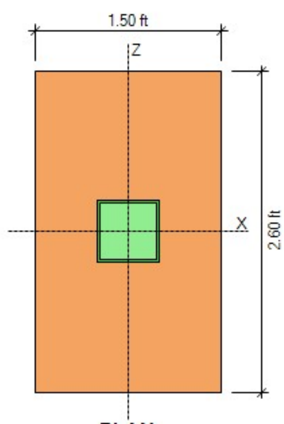
Footing bearing $\phi Pn = \phi Pnc + \phi Pns$ = 2.8 + 0.0 = 2.8 ksi > 0.3 psi OK

DESIGN CODES

Concrete Design	ACI 318-14
Load Combinations	ASCE 7-10/16



ELEVATION



PLAN

SPREAD FOOTING DESIGN

GEOMETRY

Footing Length (X-dir)	2.00	ft	
Footing Width (Z-dir)	2.60	ft	
Footing Thickness	8.0	in	OK
Soil Cover	0.00	ft	
Column Length (X-dir)	6.0	in	
Column Width (Z-dir)	6.0	in	
Offset (X-dir)	0.00	in	OK
Offset (Z-dir)	0.00	in	OK
Base Plate (L x W)	6.0 x 6.0	in	

SOIL PRESSURES (D+L)

Gross Allow. Soil Pressure	2.0	ksf
Soil Pressure at Corner 1	2.0	ksf
Soil Pressure at Corner 2	2.0	ksf
Soil Pressure at Corner 3	2.0	ksf
Soil Pressure at Corner 4	2.0	ksf
Bearing Pressure Ratio	0.99	OK
Ftg. Area in Contact with Soil	100.0	%
X-eccentricity / Ftg. Length	0.00	OK
Z-eccentricity / Ftg. Width	0.00	OK

APPLIED LOADS

	Dead	Live	RLive	Snow	Wind	Seismic	
Axial Force P	4.5	5.5	0.0	0.0	0.0	0.0	kip
Moment about X Mx ..	0.0	0.0	0.0	0.0	0.0	0.0	k-ft
Moment about Z Mz ..	0.0	0.0	0.0	0.0	0.0	0.0	k-ft
Shear Force Vx	0.0	0.0	0.0	0.0	0.0	0.0	kip
Shear Force Vz	0.0	0.0	0.0	0.0	0.0	0.0	kip

OVERTURNING CALCULATIONS (Comb: 0.6D+0.6W)

- Overturning about X-X

- Moment Mx = 0.6 * 0.0 + 0.6 * 0.0 = 0.0 k-ft
- Shear Force Vz = 0.6 * 0.0 + 0.6 * 0.0 = 0.0 kip
  Arm = 0.00 + 8.0 / 12 = 0.67 ft   Moment = 0.0 * 0.67 = 0.0 k-ft
- Passive Force = 0.0 kip   Arm = 0.27 ft   Moment = 0.0 k-ft
- Overturning moment X-X = 0.0 + 0.0 = 0.0 k-ft

- Resisting about X-X

- Footing weight = *0.6 * W * L * Thick * Density =* 0.6 * 2.60 * 2.00 * 8.0 / 12 * 0.15 = 0.3 kip
  Arm = *W / 2 =* 2.60 / 2 = 1.30 ft   Moment = 0.3 * 1.30 = 0.4 k-ft
- Pedestal weight = *0.6 * W * L * H * Density =* 0.6 * 6.0 / 12 * 6.0 / 12 * 0.0 * 0.15 = 0.0 kip
  Arm = *W / 2 - Offset =* 2.60 / 2 - 0.0 / 12 = 1.30 ft   Moment = 0.0 * 1.30 = 0.0 k-ft
- Soil cover = *0.6 * W * L * SC * Density =* 0.6 * (2.60 * 2.00 - 6.0 / 12 * 6.0 / 12) * 0.0 * 110 = 0.0 kip
  Arm = *W / 2 =* 2.60 / 2 = 1.30 ft   Moment = 0.0 * 1.30 = 0.0 k-ft
- Buoyancy = *0.6 * W * L * γ * (SC + Thick - WT) =* 0.6 * 2.60 * 2.00 * 62 * (0.67) = -0.1 kip
  Arm = *W / 2 =* 2.60 / 2 = 1.30 ft   Moment = 0.1 * 1.30 = -0.2 k-ft
- Axial force P = 0.6 * 4.5 + 0.6 * 0.0 = 2.7 kip
  Arm = *W / 2 - Offset =* 2.60 / 2 - 0.0 / 12 = 1.30 ft   Moment = 2.7 * 1.30 = 3.5 k-ft
- Resisting moment X-X = 0.4 + 0.0 + 0.0 + 3.5 + -0.2 = 3.7 k-ft
- Overturning safety factor X-X = $\dfrac{\textit{Resisting moment}}{\textit{Overturning moment}} = \dfrac{3.7}{0.0}$ = 37.47 > 1.50 OK

- Overturning about Z-Z

- Moment Mz = 0.6 * 0.0 + 0.6 * 0.0 = 0.0 k-ft
- Shear Force Vx = 0.6 * 0.0 + 0.6 * 0.0 = 0.0 kip
  Arm = 0.00 + 8.0 / 12 = 0.67 ft   Moment = 0.0 * 0.67 = 0.0 k-ft
- Passive Force = 0.0 kip   Arm = 0.27 ft   Moment = 0.0 k-ft
- Overturning moment Z-Z = 0.0 + 0.0 = 0.0 k-ft

- Resisting about Z-Z

- Footing weight = *0.6 * W * L * Thick * Density =* 0.6 * 2.60 * 2.00 * 8.0 / 12 * 0.15 = 0.3 kip
  Arm = *L / 2 =* 2.00 / 2 = 1.00 ft   Moment = 0.3 * 1.00 = 0.3 k-ft
- Pedestal weight = *0.6 * W * L * H * Density =* 0.6 * 6.0 / 12 * 6.0 / 12 * 0.0 * 0.15 = 0.0 kip
  Arm = *L / 2 - Offset =* 2.00 / 2 - 0.0 / 12 = 1.00 ft   Moment = 0.0 * 1.00 = 0.0 k-ft
- Soil cover = *0.6 * W * L * SC * Density =* 0.6 * (2.60 * 2.00 - 6.0 / 12 * 6.0 / 12) * 0.0 * 110 = 0.0 kip
  Arm = *L / 2 =* 2.00 / 2 = 1.00 ft   Moment = 0.0 * 1.00 = 0.0 k-ft
- Buoyancy = *0.6 * W * L * γ * (SC + Thick - WT) =* 0.6 * 2.60 * 2.00 * 62 * (0.67) = -0.1 kip
  Arm = *L / 2 =* 2.00 / 2 = 1.00 ft   Moment = 0.1 * 1.00 = -0.1 k-ft
- Axial force P = 0.6 * 4.5 + 0.6 * 0.0 = 2.7 kip
  Arm = *L / 2 - Offset =* 2.00 / 2 - 0.0 / 12 = 1.00 ft   Moment = 2.7 * 1.00 = 2.7 k-ft
- Resisting moment Z-Z = 0.3 + 0.0 + 0.0 + 2.7 + -0.1 = 2.9 k-ft

- Overturning safety factor Z-Z = $\dfrac{\textit{Resisting moment}}{\textit{Overturning moment}} = \dfrac{2.9}{0.0}$ = 28.82 > 1.50 OK

SOIL BEARING PRESSURES (Comb: D+L)

Overturning moment X-X = 0.0 + 0.0 = 0.0 k-ft

Resisting moment X-X = 0.7 + 0.0 + 0.0 + -0.3 + 13.0 = 13.4 k-ft

Overturning moment Z-Z = 0.0 + 0.0 = 0.0 k-ft

Resisting moment Z-Z = 0.5 + 0.0 + 0.0 + -0.2 + 10.0 = 10.3 k-ft

Resisting force = *Footing + Pedestal + Soil - Buoyancy + P =* 0.5 + 0.0 + 0.0 - 0.2 + 10.0 = 10.3 kip

X-coordinate of resultant from maximum bearing corner:

$$Xp = \frac{\textit{Z-Resisting moment - Z-Overturning moment}}{\textit{Resisting force}} = \frac{10.3 - 0.0}{10.3} = 1.00 \text{ ft}$$

Z-coordinate of resultant from maximum bearing corner:

$$Zp = \frac{\textit{X-Resisting moment - X-Overturning moment}}{\textit{Resisting force}} = \frac{13.4 - 0.0}{10.3} = 1.30 \text{ ft}$$

X-ecc = *Length / 2 - Xp =*2.00 / 2 - 1.00 = 0.00 ft

Z-ecc = *Width / 2 - Zp =* 2.60 / 2 - 1.30 = 0.00 ft

Area = *Width * Length =* 2.60 * 2.00 = 5.2 ft²

Sx = *Length * Width² / 6 =* 2.00 * 2.60² / 6 = 2.3 ft³

Sz = *Width * Length² / 6 =* 2.60 * 2.00² / 6 = 1.7 ft³

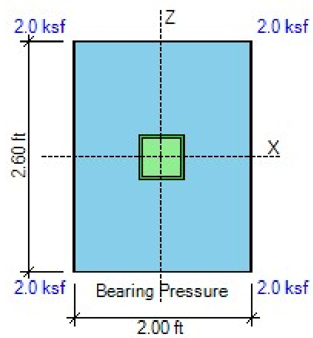
- Footing is in full bearing. Soil pressures are as follows:

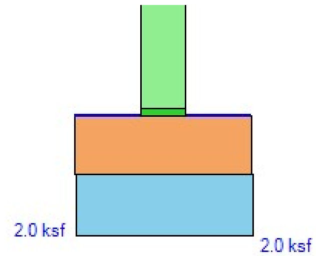
P1 = *P * (1/ A + Z-ecc / Sx + X-ecc / Sz) =* 10.3 * (1 / 5.2 + 0.00 / 2.3 + 0.00 / 1.7) = 1.98 ksf

P2 = *P * (1/ A - Z-ecc / Sx + X-ecc / Sz) =* 10.3 * (1 / 5.2 - 0.00 / 2.3 + 0.00 / 1.7) = 1.98 ksf

P3 = *P * (1/ A - Z-ecc / Sx - X-ecc / Sz) =* 10.3 * (1 / 5.2 - 0.00 / 2.3 - 0.00 / 1.7) = 1.98 ksf

P4 = *P * (1/ A + Z-ecc / Sx - X-ecc / Sz) =* 10.3 * (1 / 5.2 + 0.00 / 2.3 - 0.00 / 1.7) = 1.98 ksf





SLIDING CALCULATIONS (Comb: 0.6D+0.6W)

Internal friction angle = 28.0 deg

Passive coefficient *kp = 4.33* *(per Coulomb)*

Pressure at mid-depth = *kp * Density * (Cover + Thick / 2) =* 4.33 * 110 * (0.00 + 8.0 / 12 / 2) = 0.16 ksf

X-Passive force = *Pressure * Thick * Width =* 0.16 * 8.0 / 12 * 2.60 = 0.3 kip

Z-Passive force = *Pressure * Thick * Length =* 0.16 * 8.0 / 12 * 2.00 = 0.2 kip

Friction force = *Resisting force * Friction coeff. =* Max (0, 2.9 * 0.35) = 1.0 kip

Use 100% of Passive + 100% of Friction for sliding resistance

- Sliding safety factor X-X = $\frac{\text{X-Passive force + Friction}}{\text{X-Horizontal load}} = \frac{1.00 * 0.3 + 1.00 * 1.0}{0.0}$ = 12.84 > 1.50 OK

- Sliding safety factor Z-Z = $\frac{\text{Z-Passive force + Friction}}{\text{Z-Horizontal load}} = \frac{1.00 * 0.2 + 1.00 * 1.0}{0.0}$ = 12.20 > 1.50 OK

UPLIFT CALCULATIONS (Comb: 0.6D+0.6W)

- Uplift safety factor = $\frac{\text{Pedestal + Footing + Cover - Buoyancy}}{\text{Uplift load}} = \frac{0.0 + 0.3 + 0.0 - 0.1}{0.0}$ = 99.99 > 1.00 OK

ONE-WAY SHEAR CALCULATIONS (Comb: 1.2D+1.6L+0.5S)

Concrete f'c = 2.5 ksi Steel fy = 40.0 ksi Soil density = 110 pcf

Use Plain Concrete Shear Strength

ϕVcx = *4/3 * ϕ * √(f'c) * Width * t / 1000 =* 4/3 * 0.60 * √(2500) * 2.6 * 12 * 8.0 / 1000 = 10.0 kip ACI 14.5.5.1

ϕVcz = *4/3 * ϕ * √(f'c) * Length * t / 1000 =* 4/3 * 0.60 * √(2500) * 2.0 * 12 * 8.0 / 1000 = 7.7 kip

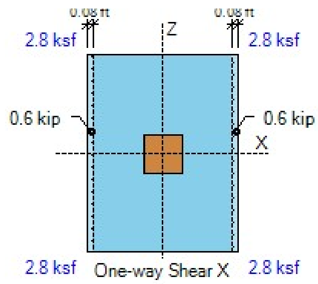
- Shear forces calculated as the volume of the bearing pressures under the effective areas:

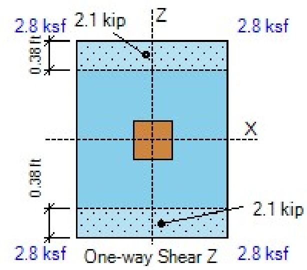
One-way shear Vux (- Side) = 0.6 kip < 10.0 kip OK

One-way shear Vux (+ Side) = 0.6 kip < 10.0 kip OK

One-way shear Vuz (- Side) = 2.1 kip < 7.7 kip OK

One-way shear Vuz (+ Side) = 2.1 kip < 7.7 kip OK





FLEXURE CALCULATIONS (Comb: 1.2D+1.6L+0.5S)

Plain ϕMnx =*5 * ϕ * √(f'c) * L * Thick² / 6* =5 * 0.60 * √(2500) * 2.00 * 8.0² / 6 / 1000 = 0.9 k-ft ACI Eq. (14.5.2.1a)

Plain ϕMnz =*5 * ϕ * √(f'c) * W * Thick² / 6* =5 * 0.60 * √(2500) * 2.60 * 8.0² / 6 / 1000 = 1.1 k-ft

- Top Bars

No Top Reinforcement Provided at the Footing

Use Plain Concrete Flexural Strength at Top

- Top moments calculated as the overburden minus the bearing pressures times the lever arm:

Top moment -Mux (- Side) = 0.0 k-ft < 3.2 k-ft OK

Top moment -Mux (+ Side) = 0.0 k-ft < 3.2 k-ft OK

Top moment -Muz (- Side) = 0.0 k-ft < 4.2 k-ft OK

Top moment -Muz (+ Side) = 0.0 k-ft < 4.2 k-ft OK

- Bottom Bars

No Bottom Reinforcement Provided at the Footing

Use Plain Concrete Flexural Strength at Bottom

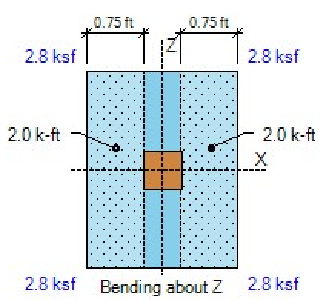
- Bottom moments calculated as the bearing minus the overburden pressures times the lever arm:

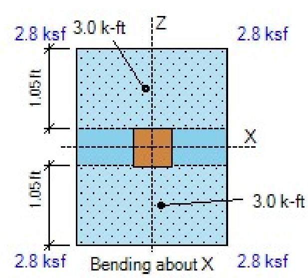
Bottom moment Mux (- Side) = 3.0 k-ft < 3.2 k-ft OK ratio = 0.94

Bottom moment Mux (+ Side) = 3.0 k-ft < 3.2 k-ft OK ratio = 0.94

Bottom moment Muz (- Side) = 2.0 k-ft < 4.2 k-ft OK ratio = 0.48

Bottom moment Muz (+ Side) = 2.0 k-ft < 4.2 k-ft OK ratio = 0.48





LOAD TRANSFER CALCULATIONS (Comb: 1.2D+1.6L+0.5S)

Area *A1 = col L * col W =* 6.0 * 6.0 = 36.0 in²

Sx = *col W * col L² / 6 =* 6.0 * 6.0² / 6 = 36.0 in³

Sz = *col L * col W² / 6 =* 6.0 * 6.0² / 6 = 36.0 in³

Bearing *Pbu = P / A1 + Mz / Sx + Mx / Sz =* 14.2 / 36.0 + 0.0 * 12 / 36.0 + 0.0 * 12 / 36.0 = 0.4 ksi

Min edge = *Min (L / 2 - X-offset - col L / 2, W / 2 - Z-offset - col W / 2)*

Min edge = Min (2.00 * 12 / 2 - 0.0 - 6.0 / 2, 2.60 * 12 / 2 - 0.0 - 6.0 / 2 = 9.0 in

Area *A2 = Min [L * W, (col L + 2 * Min edge) * (col W + 2 * Min edge)]* ACI R22.8.3.2

A2 = Min [2.00 * 12 * 2.6 * 12, (6.0 + 2 * 9.0) * (6.0 + 2 * 9.0)] = 576.0 in²

Footing *ϕPnc = ϕ * 0.85 * f'c * Min [2, √(A2 / A1)] =* 0.65 * 0.85 * 2.5 * Min [2, √(576.0 / 36.0)] = 2.8 ksi

Footing *ϕPns = ϕ * As * Fy / A1 =* 0.0 ksi ACI 22.8.3.2

Footing bearing *ϕPn = ϕPnc + ϕPns =* 2.8 + 0.0 = 2.8 ksi > 0.4 psi OK

Hooked *Ldh = Max (8 db, 6, 0.02 * fy / (f'c)½ * Confining * Location * Concrete * db * ratio)* ACI 25.4.3

Ldh = Max (8 db, 6, 0.02 * 60.0 * 1000 / (2500)½ * 1.0 * 0.7 * 0.0 * 0.75 * 0.07) = 6.0 in

Ld provided = *Dowel length =* 3.00 * 12 = 36.0 in > 12.3 in OK

Ldh provided = *Footing thickness - Cover =* 8.00 - 3.0 = 5.0 in < 6.0 in NG

PUNCHING SHEAR CALCULATIONS (Comb: 1.2D+1.6L+0.5Lr)

X-Edge = *Length / 2 - Offset - Col / 2 =* 2.00 * 12 / 2 - 0.0 - 6.0 / 2 = 9.0 in αsx = 10

Z-Edge = *Width / 2 - Offset - Col / 2 =* 2.60 * 12 / 2 - 0.0 - 6.0 / 2 = 12.6 in αsz = 10

αs = *αsx + αsz =* 10 + 10 = 20 Col type = Corner β = *L / W =* 6.0 / 6.0 = 1.00 ACI 22.6.5.2

Perimeter *bo = αsz / 10 * (L + d / 2 + X-Edge) + αsx / 10 * (W + d / 2 + Z-Edge)* ACI 22.6.4.2

bo = 10 / 10 * (6.0 + 8.0 / 2 + 9.0) + 10 / 10 * (6.0 + 8.0 / 2 + 12.6) = 41.6 in

Area *Abo = (L + d / 2 + X-Edge) * (W + d / 2 + Z-Edge) =* (6.0 + 8.0 / 2 + 9.0) * (6.0 + 8.0 / 2 + 12.6) = 429.4 in^2

Use Plain Concrete Shear Strength

ϕVc = *ϕ * Min (1 + 2 / β, 2) * 4/3 * √(f'c)* ACI 14.5.5.1

ϕVc = 0.60 * Min (1 + 2 / 1.00, 2) * 4/3 √(2500) = 80.0 psi

Punching force *F = P + Overburden * Abo - Bearing*

F = 14.2 + 0.07 * 429.4 / 144 - 3.8 = 10.6 kip

b1 = *L + d / 2 + X-Edge =* 6.0 + 8.0 / 2 + 9.0 = 19.0 in b2 = *W + d / 2 + Z-Edge =* 6.0 + 8.0 / 2 + 12.6 = 22.6 in

γvx factor = $1 - \dfrac{1}{1 + (2/3)\sqrt{b2 / b1}} = 1 - \dfrac{1}{1 + (2/3)\sqrt{22.6 / 19.0}} = 0.42$ ACI Eq. (8.4.4.2.2)

γvz factor = $1 - \dfrac{1}{1 + (2/3)\sqrt{b1 / b2}} = 1 - \dfrac{1}{1 + (2/3)\sqrt{19.0 / 22.6}} = 0.38$ ACI Eq. (8.4.2.3.2)

X2z = $b1^2 / 2 / (b1 + b2) =$ $19.0^2 / 2 / (19.0 + 22.6) = 4.3$ in X2x = $b2^2 / 2 / (b2 + b1) =$ 6.1 in

Jcz = $b1 * d^3 / 12 + b1^3 * d / 12 + b1 * d * (b1 / 2 - X2z)^2 + b2 * d * X2z^2$ ACI R8.4.4.2.3

Jcz = $19.0 * 8.0^3 / 12 + 19.0^3 * 8.0 / 12 + 19.0 * 8.0 * (19.0 / 2 * 4.3)^2 + 22.6 * 8.0 * 4.3^2 = 12836$ in^4

Jcx = $b2 * d^3 / 12 + b2^3 * d / 12 + b2 * d * (b2 / 2 - X2x)^2 + b1 * d * X2x^2$ ACI R8.4.4.2.3

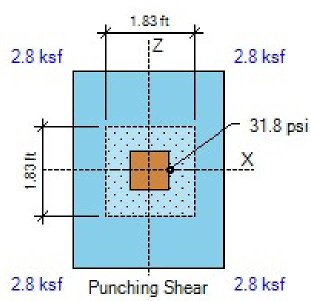
Jcz = $22.6 * 8.0^3 / 12 + 22.6^3 * 8.0 / 12 + 22.6 * 8.0 * (22.6 / 2 * 6.1)^2 + 19.0 * 8.0 * 6.1^2 = 19204$ in^4

Stress due to P = *F / (bo * d) * 1000 =* 10.6 / (41.6 * 8.0) * 1000 = 31.8 psi

Stress due to Mx = *γvx * X-OTM * X2x / Jcx =* 0.42 * 0.0 * 12 * 6.1 / 19204 * 1000 = 0.0 psi

Stress due to Mz = *γvz * Z-OTM * X2z / Jcz =* 0.42 * 0.0 * 12 * 4.3 / 12836 * 1000 = 0.0 psi

Punching stress = *P-stress + Mx-stress + Mz-stress =* 31.8 + 0.0 + 0.0 = 31.8 psi < 80.0 psi OK



SPREAD FOOTING DESIGN

LOAD TRANSFER CALCULATIONS (Comb: 1.2D+1.6L+0.5S)

Area *A1 = col L * col W* = 6.0 * 6.0 = 36.0 in²

Sx = *col W * col L² / 6* = 6.0 * 6.0² / 6 = 36.0 in³

Sz = *col L * col W² / 6* = 6.0 * 6.0² / 6 = 36.0 in³

Bearing *Pbu = P / A1 + Mz / Sx + Mx / Sz* = 14.2 / 36.0 + 0.0 * 12 / 36.0 + 0.0 * 12 / 36.0 = 0.4 ksi

Min edge = *Min (L / 2 - X-offset - col L / 2, W / 2 - Z-offset - col W / 2)*

Min edge = Min (2.00 * 12 / 2 - 0.0 - 6.0 / 2, 2.60 * 12 / 2 - 0.0 - 6.0 / 2 = 9.0 in

Area *A2 = Min [L * W, (col L + 2 * Min edge) * (col W + 2 * Min edge)]* ACI R22.8.3.2

A2 = Min [2.00 * 12 * 2.6 * 12, (6.0 + 2 * 9.0) * (6.0 + 2 * 9.0)] = 576.0 in²

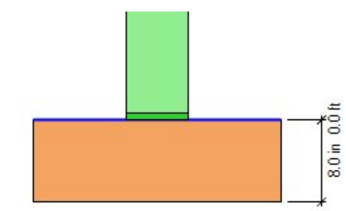
Footing *ϕPnc = ϕ * 0.85 * f'c * Min [2, √(A2 / A1)]* = 0.65 * 0.85 * 2.5 * Min [2, √(576.0 / 36.0)] = 2.8 ksi

Footing *ϕPns = ϕ * As * Fy / A1* = 0.0 ksi ACI 22.8.3.2

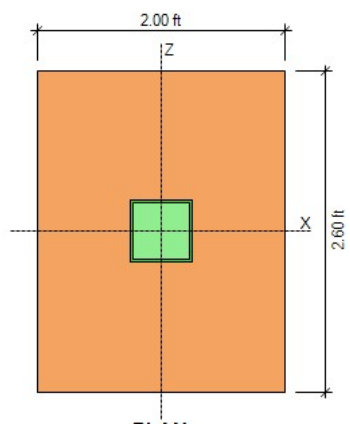
Footing bearing *ϕPn = ϕPnc + ϕPns* = 2.8 + 0.0 = 2.8 ksi > 0.4 psi OK

DESIGN CODES

Concrete Design	ACI 318-14
Load Combinations	ASCE 7-10/16



ELEVATION



PLAN

WIND $V_{ASD} = 85$ MPH $V_{ULT} = 110$ MPH EXP. B $K_{ZT} = 1.0$ SLOPE $= 0° - 34°$
$h = 36'$ $\lambda = 1.06$

ZONE A = 12.9 PSF x 1.06 = 13.7 PSF 16.0 PSF MIN

ZONE B = 8.8 PSF x 1.06 = 9.3 PSF

ZONE C = 10.2 PSF x 1.06 = 10.8 PSF 16.0 PSF MIN

ZONE D = 7.0 PSF x 1.06 = 7.4 PSF 8.0 PSF MIN

SEISMIC $S_{DS} = 0.831$ $R = 6.5$ $I_e = 1.0$

$C_S = (0.831/(6.5/1.0))/1.4 = 0.091$

$W_{ROOF} = (35\text{ PSF} \times 2{,}632\text{ SF}) = 92{,}120\#$ $h = 9'$ $h_R = 29'$
$W_{LEVEL3} = (40\text{ PSF} \times 4{,}224\text{ SF}) = 168{,}960\#$ $h = 9'$ $h_3 = 20'$
$W_{LEVEL2} = (40\text{ PSF} \times 3{,}894\text{ SF}) = 155{,}760\#$ $h = 9'$ $h_2 = 10'$
416,840#

$V_S = 416{,}840\# \times 0.091 = 37{,}932\#$

7,608,280

$$F_{ROOF} = \left[\frac{(92{,}120\# \times 29')}{(92{,}120\# \times 29') + (168{,}960\# \times 20') + (155{,}760\# \times 10')}\right] \times 37{,}932\# = 13{,}319\#$$

$$F_{LEVEL3} = \left[\frac{(168{,}960\# \times 20')}{(92{,}120\# \times 29') + (168{,}960\# \times 20') + (155{,}760\# \times 10')}\right] \times 37{,}932\# = 16{,}847\#$$

$$F_{LEVEL2} = \left[\frac{(155{,}760\# \times 10')}{(92{,}120\# \times 29') + (168{,}960\# \times 20') + (155{,}760\# \times 10')}\right] \times 37{,}931\# = 7{,}766\#$$

TOPS

GRIDS A-B

$F_{2W} = (16.0PSF \times 133SF) + (9.3PSF \times 110SF) + (8.0PSF \times 30SF) = 3,391^{\#}$

$F_{2E} = 16,347^{\#} \times (1,092SF / 4,224SF) = 4,355^{\#}$

$F_{1W} = 3,391^{\#} + (16.0PSF \times 158SF) = 5,919^{\#}$

$F_{1E} = 4,335^{\#} + 7,766^{\#} \times (931SF / 3,894SF) = 6,212^{\#}$

GRID C

$F_{3W} = (16.0PSF \times 151SF) + (8.0PSF \times 36SF) = 2,704^{\#}$

$F_{3E} = 13,319^{\#} \times (1,323SF / 2,632SF) = 6,695^{\#}$

$F_{2W} = 2,704^{\#} + (16.0PSF \times 313SF) = 7,712^{\#}$

$F_{2E} = 6,695^{\#} + 16,347^{\#} \times (2,098SF / 4,224SF) = 15,063^{\#}$

$F_{1W} = 7,712^{\#} + (16.0PSF \times 324SF) = 12,896^{\#}$

$F_{1E} = 15,063^{\#} + 7,766^{\#} \times (2,078SF / 3,894SF) = 19,207^{\#}$

GRIDS G-H

$F_{3W} = (16.0PSF \times 169SF) + (9.3PSF \times 55SF) + (8.0PSF \times 23SF) = 3,400^{\#}$

$F_{3E} = 13,319^{\#} \times (1,309SF / 2,632SF) = 6,624^{\#}$

$F_{2W} = 3,400^{\#} + (16.0PSF \times 194SF) = 6,504^{\#}$

$F_{2E} = 6,624^{\#} + 16,847^{\#} \times (1,034SF / 4,224SF) = 10,748^{\#}$

$F_{1W} = 6,504^{\#} + (16.0PSF \times 194SF) = 9,608^{\#}$

$F_{1E} = 10,748^{\#} + 7,766^{\#} \times (885SF / 3,894SF) = 12,513^{\#}$

GRID 1-2

$F_{3W} = (16.0\,PSF \times 189\,SF) = 3{,}024\#$

$F_{3E} = 13{,}319\# \times (657\,SF / 2{,}632\,SF) = 3{,}705\#$

$F_{2W} = 3{,}024\# + (16.0\,PSF \times 176\,SF) = 5{,}840\#$

$F_{2E} = 3{,}705\# + 16{,}847\# \times (917\,SF / 4{,}224\,SF) = 7{,}362\#$

$F_{1W} = 5{,}840\# + (16.0\,PSF \times 176\,SF) = 8{,}656\#$

$F_{1E} = 7{,}362\# + 7{,}766\# \times (867\,SF / 3{,}894\,SF) = 9{,}091\#$

GRIDS 4-5

$F_{3W} = (16.0\,PSF \times 253\,SF) = 4{,}048\#$

$F_{3E} = 13{,}319\# \times (1{,}307\,SF / 2{,}632\,SF) = 6{,}614\#$

$F_{2W} = 4{,}048\# + (16.0\,PSF \times 355\,SF) = 9{,}728\#$

$F_{2E} = 6{,}614\# + 16{,}847\# \times (2{,}087\,SF / 4{,}224\,SF) = 14{,}938\#$

$F_{1W} = 9{,}728\# + (16.0\,PSF \times 353\,SF) = 15{,}376\#$

$F_{1E} = 14{,}938\# + 7{,}766\# \times (1{,}872\,SF / 3{,}894\,SF) = 18{,}671\#$

GRIDS 7-8

$F_{3W} = (16.0\,PSF \times 181\,SF) = 2{,}896\#$

$F_{3E} = 13{,}319\# \times (668\,SF / 2{,}632\,SF) = 3{,}380\#$

$F_{2W} = 2{,}896\# + (16.0\,PSF \times 203\,SF) = 6{,}144\#$

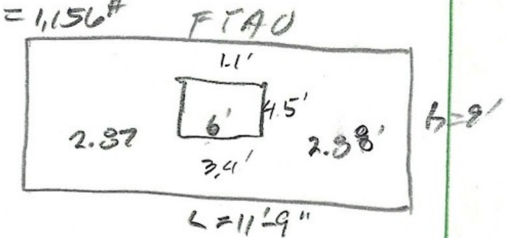
$F_{2E} = 3{,}380\# + 16{,}847\# \times (1{,}220\,SF / 4{,}224\,SF) = 8{,}246\#$

$F_{1W} = 6{,}144\# + (16.0\,PSF \times 201\,SF) = 9{,}360\#$

$F_{1E} = 8{,}246\# + 7{,}766\# \times (1{,}154\,SF / 3{,}894\,SF) = 10{,}548\#$

11'-9" FTAO

GRID A (LEVEL 2) $F_E = 4{,}355^{\#} \times 11.75'/44.75' = 1{,}156^{\#}$



$V_E = 201$ PLF

USE W1 $V_{E\,ALLOW} = 242$ PLF

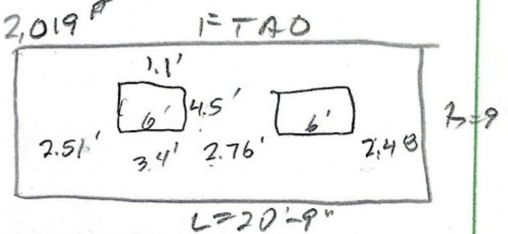
HOLD DOWNS

$T_E = 985^{\#} \times 1.25 - \frac{1}{2}(20\,PSF \times 9' \times 5.88') - \frac{1}{2}(12\,PSF \times 4.5' \times 5.88') = 418^{\#}$

$\boxed{\text{USE MST37 w/ 2 STUDS}}$ $T_{E\,ALLOW} = 2{,}140^{\#} \times 1.4/1.6 = 1{,}873^{\#}$

20'-9" FTAO

GRID B (LEVEL 2) $F_E = 4{,}355^{\#} \times 20.75'/44.75' = 2{,}019^{\#}$



$V_E = 336$ PLF

USE W2 $V_{E\,ALLOW} = 353$ PLF

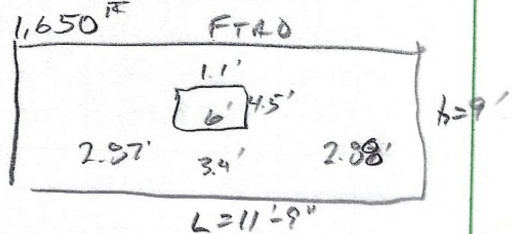
HOLD DOWNS

$T_E = 920^{\#} \times 1.25 - \frac{1}{2}(20\,PSF \times 15' \times 10.38') - \frac{1}{2}(12\,PSF \times 4.5' \times 10.38') = -687^{\#}$

SO NO HD'S REQ'D

L=11'-9"

GRID A (LEVEL 1) $F_E = 6{,}212^{\#} \times 11.75'/44.25' = 1{,}650^{\#}$



$V_E = 287$ PLF

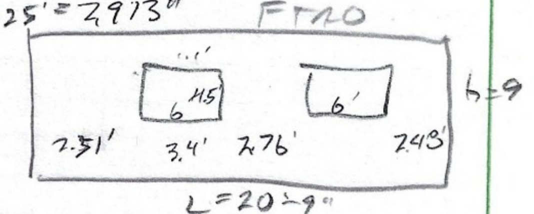
USE W2 $V_{E\,ALLOW} = 353$ PLF

HOLD DOWNS

$T_E = 1{,}264^{\#} \times 1.25 + 418^{\#} - \frac{1}{2}(30\,PSF \times 0.67' \times 5.88') - \frac{1}{2}(12\,PSF \times 9' \times 5.88') = 1{,}622^{\#}$

$\boxed{\text{USE HDU2-SDS2.5 w/ 2 STUDS}}$ $T_{E\,ALLOW} = 2{,}215^{\#} \times 1.4/1.6 = 1{,}938^{\#}$

20'-9" FTAO GRID B (LEVEL 1) $F_E = 6{,}212^{\#} \times 20.75'/44.25' = 2{,}913^{\#}$



$V_E = 484$ PLF

USE W4 $V_{E\,ALLOW} = 595$ PLF

HOLD DOWNS

$T_E = 1{,}327^{\#} \times 1.25 - 687^{\#} - \frac{1}{2}(30\,PSF \times 0.67 \times 10.38') - \frac{1}{2}(12\,PSF \times 9' \times 10.38') = 307^{\#}$

$\boxed{\text{USE HDU2-SDS2.5 w/ 2 STUDS}}$ $T_{E\,ALLOW} = 2{,}215^{\#} \times 1.4/1.6 = 1{,}938^{\#}$

GRID C (LEVEL 3) FE= 6,695#

PERF. WALL

$A_0/A_W = 4\%$
$A_{FH}/A_W = 95\%$
$C_0 = 0.9$
L=61'-6"
h=9'
L=64'-8"

VE = 6,695#/61.5' = 109PLF

USE W1 VEALLOW = 242PLF x 0.9 = 218PLF

HOLD DOWNS

$TE = \left(\frac{6,695\# \times 9'}{0.9 \times 61.5'}\right) \times 1.25 - \frac{1}{2}(20PSF \times 17.6' \times 32.33') - \frac{1}{2}(12PSF \times 4.5' \times 32.33') = -5,474\#$

SO NO HD'S REQ'D

GRID C (LEVEL 2) FE= 15,063# 3 SEGMENTS

L=6'-6" h=9'
L=13'-4"
L=18'-7"
LT = 38'-5"

VE = 15,063#/38.42' = 392PLF

USE W3 VEALLOW = 456PLF

HOLD DOWNS

L=6'-6" TE= 392PLF x 9' x 1.25 - ½(20PSF x 17.67' x 3.25') - ½(12PSF x 13.5' x 3.25') = 3,572#

USE MST60 w/ 2 STUDS TEALLOW = 5,405# x 1.4/1.6 = 4,729#

L=13'-4" TE= 392PLF x 9' x 1.25 - ½(20PSF x 29' x 6.67') - ½(40PSF x 3.67' x 6.67') - ½(12PSF x 13.5' x 6.67') = 1,513#

USE MST37 w/ 2 STUDS TEALLOW = 2,140# x 1.4/1.6 = 1,873#

L=18'-7" TE= 392PLF x 9' x 1.25 - ½(20PSF x 28' x 9.29') - ½(8PSF x 13.5' x 9.29') = 1,307#

USE (2) HDU2-SDS2.5 w/ 2 STUDS TEALLOW = 2,215# x 1.4/1.6 = 1,938#

GRID C (LEVEL 1) FE= 19,207# 3 SEGMENTS

L=6'-6" h=9'
L=13'-4"
L=18'-7"
LT = 38'-5"

VE = 19,207#/38.42' = 500PLF

USE W4 VEALLOW = 595PLF

HOLD DOWNS

L=6'-6" TE= 500PLF x 9' x 1.25 + 3,572# - ½(12PSF x 9' x 3.25') = 9,021#

USE HDU14-SDS2.5 w/ 4 STUDS TEALLOW = 12,425# x 1.4/1.6 = 10,872#

L=13'-4" TE= 500PLF x 9' x 1.25 + 1,513# - ½(40PSF x 3.67' x 6.67') - ½(12PSF x 9' x 6.67') = 6,298#

USE HDU11-SDS2.5 w/ 3 STUDS TEALLOW = 8,030# x 1.4/1.6 = 7,026#

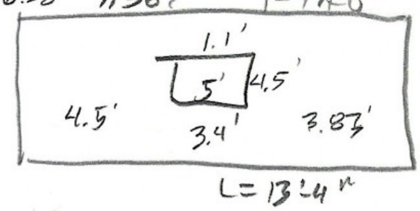
L=18'-7" TE= 500PLF x 9' x 1.25 + 1,307# - ½(8PSF x 9' x 9.29') = 6,598#

USE HDU11-SDS2.5 w/ 4 STUDS TEALLOW = 8,030# x 1.4/1.6 = 7,026#

GRID 6 (LEVEL 3) FE = 6,624# x 13.33'/56.33' = 1,567# FTAO

VE = 235 PLF

USE (W1) VE ALLOW = 242 PLF



HOLD DOWNS

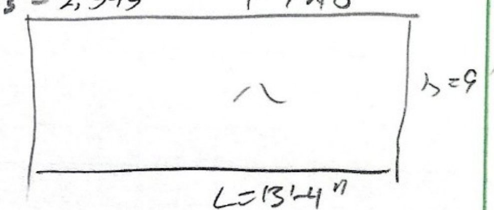
TE = 1,058# x 1.25 − (20 PSF x 17.5' x 6.67') − ½(12 PSF x 4.5' x 6.67') = −25#

∴ NO HD'S REQ'D

GRID 6 (LEVEL 2) FE = 10,748# x 13.33'/56.33' = 2,545# FTAO

VE = 382 PLF

USE (W3) VE ALLOW = 456 PLF



HOLD DOWNS

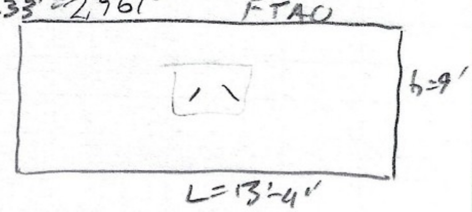
TE = 1,718# x 1.25 − 25# − ½(30 PSF x 0.67' x 6.67') − ½(12 PSF x 9' x 6.67') = 1,695#

$\boxed{\text{USE MST37 W/2 STUDS}}$ TE ALLOW = 2,140# x 1.4/1.6 = 1,873#

GRID 6 (LEVEL 1) FE = 12,513# x 13.33'/56.33' = 2,961# FTAO

VE = 444 PLF

USE (W3) VE ALLOW = 456 PLF



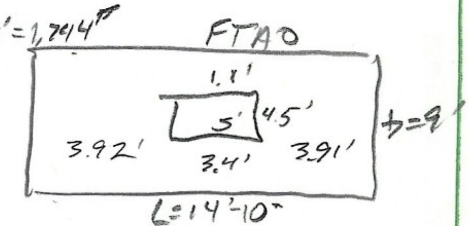
HOLD DOWNS

TE = 1,999# x 1.25 + 1,695# − ½(30 PSF x 0.67 x 6.67') − ½(12 PSF x 9' x 6.67') = 3,767#

$\boxed{\text{USE HDU5-SDS2.5 W/ 2 STUDS}}$ TE = 4,340# x 1.4/1.6 = 3,798#

14'-10" FTAO

GRID H (LEVEL 3) FE = 6,624# x 14.83'/56.33' = 1,744#



VE = 235PLF

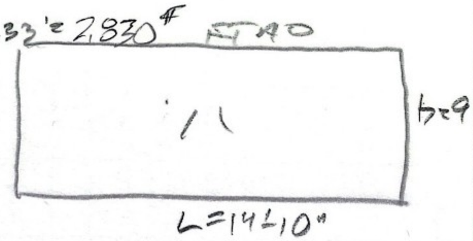
USE $\boxed{W1}$ VEALLOW

HOLD DOWNS

TE = 1,058# x1.25 - ½(20PSF x 18.67' x 7.42') - ½(12PSF x 4.5' x 2.42') = -263#

SO NO HD'S REQ'D

GRID H (LEVEL 2) FE = 10,748# x 14.83'/56.33' = 2,830# FTAO



VE = 332PLF

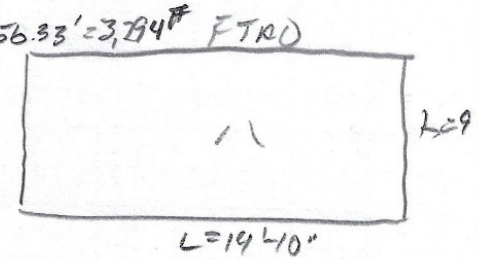
USE $\boxed{W3}$ VEALLOW = 450PLF

HOLD DOWNS

TE = 1,717# x1.25 - 263# - ½(30PSF x 0.67' x 7.42') - ½(12PSF x 9' x 7.42') = 1,408#

$\boxed{\text{USE MST37 w/ 2 STUDS}}$ TEALLOW = 2,140# x 1.4/1.6 = 1,873#

GRID H (LEVEL 1) FE = 12,513# x 14.83'/56.33' = 3,294# FTAO



VE = 444PLF

USE $\boxed{W3}$ VEALLOW = 450PLF

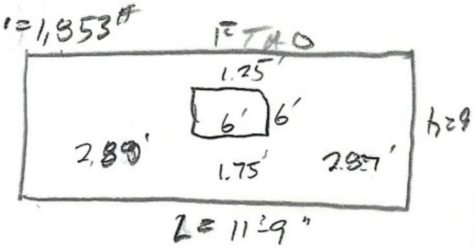
HOLD DOWNS

TE = 1,999# x1.25 + 1,408# - ½(30PSF x 0.67' x 7.42') - ½(12PSF x 9' x 7.42') = 3,931#

$\boxed{\text{USE HDU5-SDS2.5 w/ 2 STUDS}}$ TEALLOW = 4,340# x 1.4/1.6 = 3,798#

11'-9" FTAO
GRID G

GRID 1 (LEVEL 3) $F_E = 3{,}705^{\#} \times 11.75'/23.5' = 1{,}853^{\#}$



$V_E = 473$ PLF

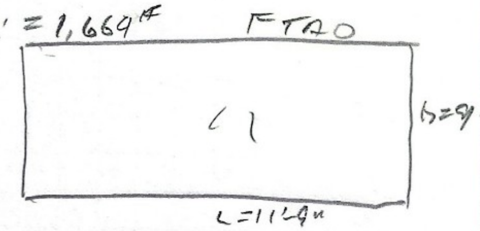
USE W4 $V_{E\,ALLOW} = 595$ PLF

HOLD DOWNS

$T_E = 1{,}419^{\#} \times 1.25 - \tfrac{1}{2}(20\text{PSF} \times 1' \times 5.88') - \tfrac{1}{2}(12\text{PSF} \times 4.5' \times 5.88') = 1{,}556^{\#}$

USE MST37 W/2 STUDS $T_{E\,ALLOW} = 2{,}140^{\#} \times 1.4/1.6 = 1{,}873^{\#}$

GRID 1 (LEVEL 2) $F_E = 7{,}362^{\#} \times 11.75'/52' = 1{,}664^{\#}$



$V_E = 425$ PLF

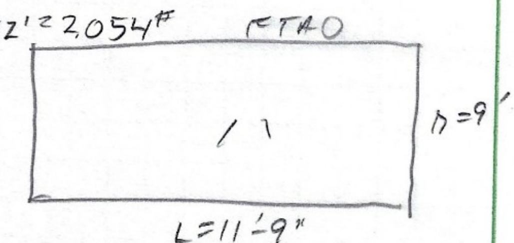
USE W4 $V_{E\,ALLOW} = 595$ PLF

HOLD DOWNS

$T_E = 1{,}275^{\#} \times 1.25 + 1{,}556^{\#} - \tfrac{1}{2}(30\text{PSF} \times 7.5' \times 5.88') - \tfrac{1}{2}(12\text{PSF} \times 9' \times 5.88') = 1{,}953^{\#}$

USE MST48 W/2 STUDS $T_{E\,ALLOW} = 3{,}425^{\#} \times 1.4/1.6 = 2{,}997^{\#}$

GRID 1 (LEVEL 1) $F_E = 9{,}091^{\#} \times 11.75'/52' = 2{,}054^{\#}$



$V_E = 524$ PLF

USE W4 $V_{E\,ALLOW} = 595$ PLF

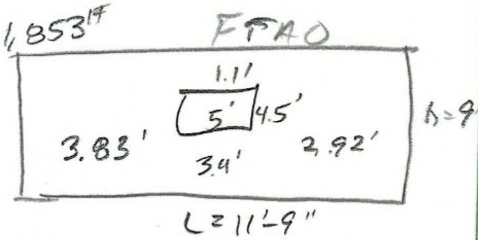
HOLD DOWNS

$T_E = 1{,}573^{\#} \times 1.25 + 1{,}953^{\#} - \tfrac{1}{2}(30\text{PSF} \times 7.5' \times 5.88') - \tfrac{1}{2}(12\text{PSF} \times 9' \times 5.88') = 2{,}940^{\#}$

USE HDU5-SDS2.5 W/2 STUDS $T_{E\,ALLOW} = 4{,}340^{\#} \times 1.4/1.6 = 3{,}798^{\#}$

FTAO
11'-9"
GRID C-D

GRID 1.2 (LEVEL 3) $FE = 3{,}705^{\#} \times 11.75' / 23.5' = 1{,}853^{\#}$



VE = 315 PLF

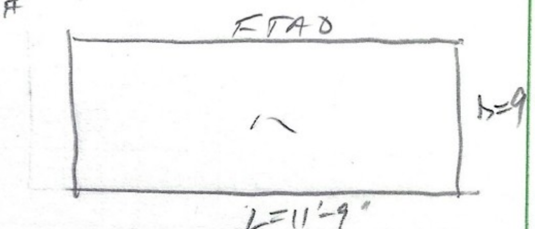
USE W2 VE ALLOW = 353 PLF

HOLD DOWNS

$TE = 1{,}419^{\#} \times 1.25 - \tfrac{1}{2}(20\,PSF \times 1' \times 5.88') - \tfrac{1}{2}(12\,PSF \times 4.5' \times 5.88') = 1{,}556^{\#}$

USE MST37 W/ 2 STUDS | $TE_{ALLOW} = 2{,}140^{\#} \times 1.4 / 1.6 = 1{,}873^{\#}$

GRID 1.2 (LEVEL 2) $FE = 11.75' / 52' = 1{,}664^{\#}$



VE = 283 PLF

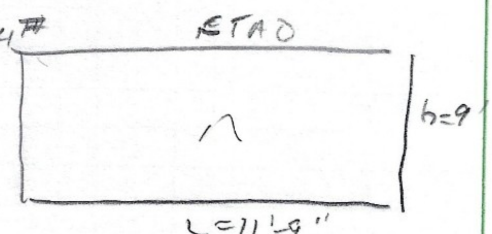
USE W2 VE ALLOW = 353 PLF

HOLD DOWNS

$TE = 1{,}275^{\#} \times 1.25 + 1{,}556^{\#} - \tfrac{1}{2}(30\,PSF \times 6.5' \times 5.88') - \tfrac{1}{2}(12\,PSF \times 9' \times 5.88') = 2{,}259^{\#}$

USE MST48 W/ 2 STUDS | $TE_{ALLOW} = 3{,}425^{\#} \times 1.4 / 1.6 = 2{,}997^{\#}$

GRID 1.2 (LEVEL 1) $FE = 11.75' / 52' = 2{,}054^{\#}$



VE = 350 PLF

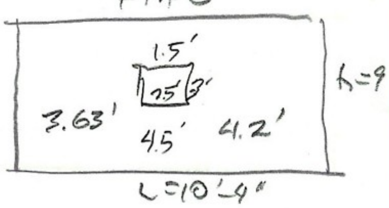
USE W2 VE ALLOW = 353 PLF

HOLD DOWNS

$TE = 1{,}573^{\#} \times 1.25 + 2{,}259^{\#} - \tfrac{1}{2}(30\,PSF \times 6.5' \times 5.88') - \tfrac{1}{2}(12\,PSF \times 9' \times 5.88') = 3{,}334^{\#}$

USE HDU5-SDS2.5 W/ 2 STUDS | $TE_{ALLOW} = 4{,}340^{\#} \times 1.4 / 1.6 = 3{,}798^{\#}$

10'-4" FTAO GRID 2.2 (LEVEL 2) FE = 7,362# x 10.33'/52' = 1,462#



VE = 212 PLF

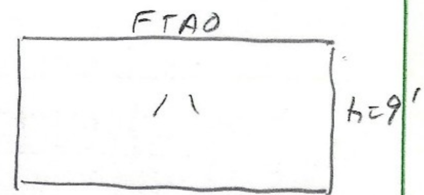
USE W1 VE ALLOW [redacted] 242 PLF

HOLD DOWNS

TE = 1,274# x 1.25 - 1/2 (20 PSF x 1' x 5.16') - 1/2 (12 PSF x 4.5' x 5.16') = 1,402#

USE MST37 W/2 STUDS   TE ALLOW = 2,140# x 1.4/1.6 = 1,873#

GRID 2.2 (LEVEL 1) FE = 9,091# x 10.33'/52' = 1,806#



VE = 762 PLF

USE W2 VE ALLOW = 553 PLF

HOLD DOWNS

TE = 1,573# x 1.25 + 1,402# - 1/2 (30 PSF x 5.1' x 5.16') - 1/2 (12 PSF x 9' x 5.16') = 2,694#

USE HDU4-SDS2.5 W/2 STUDS   TE ALLOW = 3,285# x 1.4/1.6 = 2,874#

3.2' FTAO GRID 2 (LEVEL 2) - FE = 7,362# x 13.16'/52' = 2,571#   2 SEGMENTS L=3'-7" h=9'
L=3'-7"
LT=7'-2"

VE = 2,571#/7.16' = 359 PLF

USE W3   VE ALLOW = 456 PLF x (1.25 - 0.125 x 9'/3.58') = 427 PLF

HOLD DOWNS

TE = 359 PLF x 9' x 1.25 - 1/2 (20 PSF x 1' x 1.79') - 1/2 (12 PSF x 4.5' x 1.79') = 3,973#

USE MST60 W/2 STUDS   TE ALLOW = 5,405# x 1.4/1.6 = 4,729#

GRID 2 (LEVEL 1)   FE = 9,091# x 13.16'/52' = 3,175#   2 SEGMENTS L=3'-7" h=9'
L=3'-7"
LT=7'-2"

VE = 3,175#/7.16' = 443 PLF

USE W4   VE ALLOW = 595 PLF x (1.25 - 0.125 x 9'/3.58') = 557 PLF

HOLD DOWNS

TE = 443 PLF x 9' x 1.25 + 3,973# - 1/2 (30 PSF x 5.9' x 1.79') - 1/2 (12 PSF x 9' x 1.79') = 8,702#

USE HDU14-SDS2.5 W/4 STUDS   TE ALLOW = 12,425# x 1.4/1.6 = 10,872#

GRID 485 (LEVEL 3) FE = 6,614# x 29'-4"/58'-8" = 3,307#   1 SEGMENT L = 29'-4"
h = 9'

VE = 3,307#/29.33' = 113 plf

USE W1 VE ALLOW = 242 plf

HOLD DOWNS

TE = 113 plf x 9' x 1.25 - 1/2(20 psf x 1' x 14.67') - 1/2(12 psf x 4.5' x 14.67') = 726#

$\boxed{\text{USE MST37 w/ 2 STUDS}}$ TE ALLOW = 2,140# x 1.4/1.6 = 1,873#

GRID 485 (LEVEL 2) FE = 14,938# x 29'-4"/84'-9" = 5,169#   1 SEG. L = 29'-4"
h = 9'

VE = 5,169#/29.33' = 176 plf

USE W1 VE ALLOW = 242 plf

HOLD DOWNS

TE = 176 plf x 9' x 1.25 + 726# - 1/2(30 psf x 7.1' x 14.67') - 1/2(12 psf x 9' x 14.67') = 354#

$\boxed{\text{USE MST37 w/ 2 STUDS}}$ TE ALLOW = 2,140# x 1.4/1.6 = 1,873#

GRID 485 (LEVEL 1) FE = 18,671# x 29'-4"/84'-9" = 6,493#   1 SEG L = 29'-4"
h = 9'

VE = 6,493#/29.33' = 221 plf

USE W1 VE ALLOW = 242 plf

HOLD DOWNS

TE = 221 plf x 9' x 1.25 + 354# - 1/2(30 psf x 7.1' x 14.67') - 1/2(12 psf x 9' x 14.67') = 490#

$\boxed{\text{USE HDU2-SDS2.5 w/ 2 STUDS}}$ TE ALLOW = 2,215# x 1.4/1.6 = 1,938#

L=26'-1" GRID 5 (LEVEL 2) FE = 14,938# x 26'-1" / 84'-9" = 4,597# 1 SEG. L=26'-1" h=9'

V_E = 4,597# / 26.08' = 176 PLF

USE W1 VEallow = 242 PLF

HOLD DOWNS

TE = 176 PLF x 9' x 1.25 - 1/2 (20 PSF x 1' x 13.04') - 1/2 (8 PSF x 4.5' x 13.04') = 1,618#

USE (2) HDU2-SDS2.5 w/ 2 STUDS TEallow = 2,215# x 1.4 / 1.6 = 1,938#

GRID 5 (LEVEL 1) FE = 18,671# x 26'-1" / 84'-9" = 5,773# 1 SEG. L=26'-1" h=9'

VE = 5,773# / 26.08' = 221 PLF

USE W1 VEallow = 242 PLF

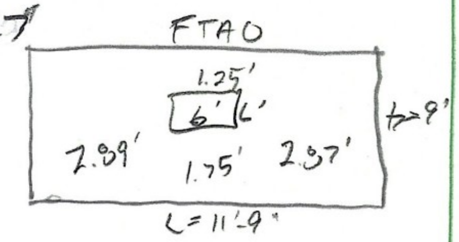
HOLD DOWNS

TE = 221 PLF x 9' x 1.25 + 1,618# - 1/2 (30 PSF x 5.1' x 13.04') - 1/2 (8 PSF x 9' x 13.04') = 2,691#

USE HDU4-SDS2.5 w/ 2 STUDS TEallow = 3,285# x 1.4 / 1.6 = 2,874#

11'-9" FTAO GRID 6

GRID 7 (LEVEL 3) $F_E = 3{,}380^{\#} \times 11.75'/23' = 1727^{\#}$



$V_E = 441\,PLF$

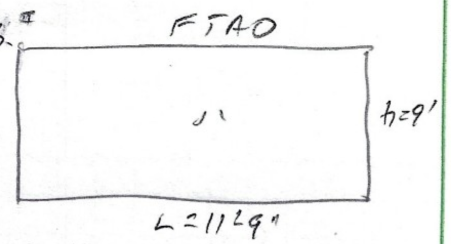
USE W3 $V_{E\,ALLOW} = 450\,PLF$

HOLD DOWNS

$T_E = 1{,}323^{\#} \times 1.25 - \tfrac{1}{2}(20PSF \times 1' \times 5.99') - \tfrac{1}{2}(12PSF \times 4.5' \times 5.98') = 1{,}436^{\#}$

USE MST37 W/ 2 STUDS $T_{E\,ALLOW} = 2{,}140^{\#} \times 1.4/1.6 = 1{,}873^{\#}$

GRID 7 (LEVEL 2) $F_E = 8{,}426^{\#} \times 11.75'/52 = 1{,}903^{\#}$



$V_E = 486\,PLF$

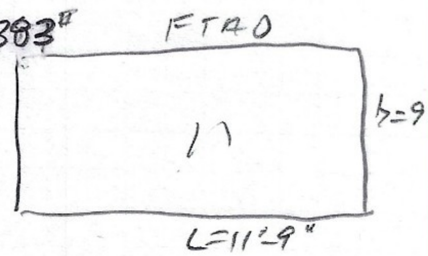
USE W4 $V_{E\,ALLOW} = 595\,PLF$

HOLD DOWNS

$T_E = 1{,}458^{\#} \times 1.25 + 1{,}436^{\#} - \tfrac{1}{2}(30PSF \times 7' \times 5.99') - \tfrac{1}{2}(12PSF \times 9' \times 5.98') = 2{,}324^{\#}$

USE MST48 W/ 2 STUDS $T_{E\,ALLOW} = 3{,}425^{\#} \times 1.4/1.6 = 2{,}997^{\#}$

GRID 7 (LEVEL 1) $F_E = 10{,}548^{\#} \times 11.75'/52' = 2{,}383^{\#}$



$V_E = 608\,PLF$

USE W5 $V_{E\,ALLOW} = 716\,PLF$

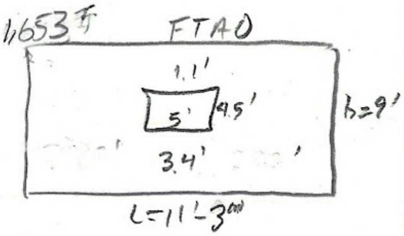
HOLD DOWNS

$T_E = 1{,}825^{\#} \times 1.75 + 2{,}324^{\#} - \tfrac{1}{2}(30PSF \times 7' \times 5.99') - \tfrac{1}{2}(12PSF \times 9' \times 5.99') = 3{,}690^{\#}$

USE HDU5-SDS2.5 W/ 2 STUDS $T_{E\,ALLOW} = 4{,}340^{\#} \times 1.4/1.6 = 3{,}798^{\#}$

TOPS

11'-3" FTAO
GRID C-D



GRID 6.8 (LEVEL 3) FE=3,380# x 11'-3"/23' = 1,653#

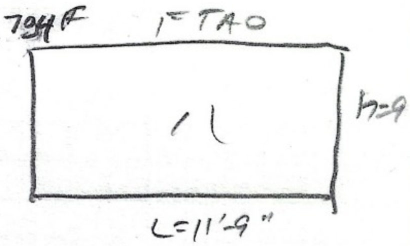
VE= 294 PLF

USE W2 VEallow=353 PLF

HOLD DOWNS

TE= 1,322# x 1.25 - 1/2(20psf x 1 x 5.12) - 1/2(12psf x 4.5' x 5.12') = 1,463#

USE MST37 w/ 2 STUDS   TEallow = 2,140# x 1.4/1.6 = 1,873#



GRID 6.8 (LEVEL 2) FE= 8,246# x 11.25'/52' = 1,784#

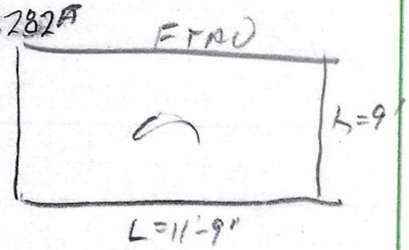
VE= 317 PLF

USE W2 VEallow=353 PLF

HOLD DOWNS

TE= 1,427# x 1.25 + 1,463# - 1/2(30psf x 6.5' x 5.12') - 1/2(12psf x 9' x 5.12') = 2,471#

USE MST48 w/ 2 STUDS   TEallow = 3,425# x 1.4/1.6 = 2,997#



GRID 6.8 (LEVEL 1) FE= 10,548# x 11.25'/52' = 2,282#

VE= 406 PLF

USE W3 VEallow = 456 PLF

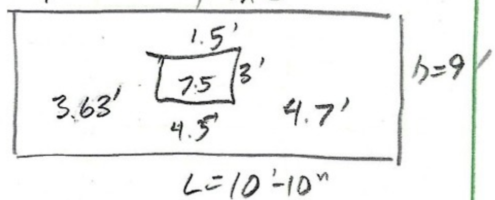
HOLD DOWNS

TE= 1,826# x 1.25 + 2,471# - 1/2(30psf x 6.5' x 5.12') - 1/2(12psf x 9' x 5.12') = 3,978#

USE HDU8-SDS2.5 w/ 2 STUDS   TEallow = 5,820# x 1.4/1.6 = 5,093#

TOPS

GRID 7.5 (LEVEL 2) FE = 8,246# x 10.83'/52' = 1,717# FTAO



VE = 238 PLF

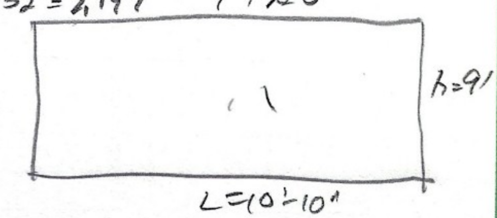
USE W1 VE allow = 242 PLF

HOLD DOWNS

TE = 1,427# x 1.25 - 1/2(20 PSF x 1' x 5.42') - 1/2(12 PSF x 4.5' x 5.42') = 1,437#

USE MST37 w/ 2 STUDS TE allow = 2,140# x 1.4/1.6 = 1,873#

GRID 7.5 (LEVEL 1) FE = 10,549# x 10.83'/52' = 2,197# FTAO



VE = 304 PLF

USE W2 VE allow = 353 PLF

HOLD DOWNS

TE = 1,826# x 1.25 + 1,437# - 1/2(30 PSF x 5.1' x 5.42') - 1/2(12 PSF x 9' x 5.42') = 3,012#

USE HDU5-SDS2.5 w/ 2 STUDS TE allow = 4,340# x 1.4/1.6 = 3,798#

GRID 8 (LEVEL 2) FE = 8,246# x 13.16'/52' = 2,880# 2 SEG L=3'-7", L=3'-7", LT=7'-2" h=9'

VE = 2,880#/7.16' = 402 PLF

USE W3 VE allow = 456 PLF x (1.25 - 0.125 x 9'/3.58') = 427 PLF

HOLD DOWNS

TE = 402 PLF x 9' x 1.25 - 1/2(20 PSF x 1' x 1.79') - 1/2(12 PSF x 4.5' x 1.79') = 4,459#

USE MST60 w/ 2 STUDS TE allow = 5,405# x 1.4/1.6 = 4,729#

GRID 8 (LEVEL 1) FE = 10,548# x 13.16'/52' = 3,694# 2 SEG L=3'-7", L=3'-7", LT=7'-2" h=9'

VE = 3,694#/7.16' = 514 PLF

USE W4 VE allow = 595 PLF x (1.25 - 0.125 x 9'/3.58') = 557 PLF

HOLD DOWNS

TE = 514 PLF x 9' x 1.25 + 4,459# - 1/2(30 PSF x 5.8' x 1.79') - 1/2(12 PSF x 9' x 1.79') = 9,995#

USE HDU14-SDS2.5 w/ 4 STUDS TE allow = 12,425# x 1.4/1.6 = 10,872#

APA

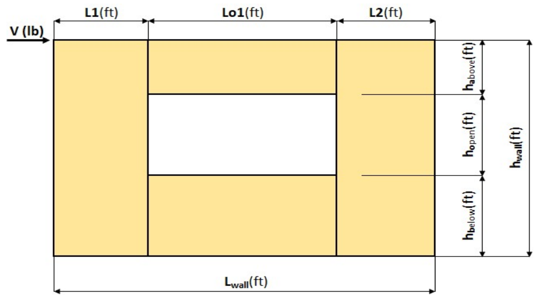
Force Transfer Around Openings Calculator

ONE OPENING

The force transfer around openings (FTAO) method of shear wall analysis is an approach that aims to reinforce the wall such that it performs as if there was no opening. This approach lends certain advantages over segmented shear walls: more versatility, because it allows for narrower wall segments while still meeting the height-to-width ratios and, often, fewer required hold-downs.

Project Information

Code:		**Date:** 6/2/2025
Designer:	Chon Pieruccioni, PE	
Client:		
Project:	East Town Crossing - Building A	
Wall Line:	Grid A - 11'-9" Segment - L2E	



Shear Wall Calculation Variables

V	1156 lbf		Opening 1	Adj. Factor Method =	1.25-0.125h/bs	
L1	2.87 ft	h_a	1.10 ft	Wall Pier Aspect Ratio		Adj. Factor
L2	2.88 ft	h_o	4.50 ft	P1=h_o/L1=	1.57	N/A
h_{wall}	9.00 ft	h_b	3.40 ft	P2=h_o/L2=	1.56	N/A
L_{wall}	11.75 ft	Lo1	6.00 ft			

1. Hold-down forces: H = Vh_{wall}/L_{wall} — 885 lbf

2. Unit shear above + below opening

First opening: va1 = vb1 = H/(h_a+h_b) =	197 plf

3. Total boundary force above + below openings

First opening: O1 = va1 x (Lo1) =	1181 lbf

4. Corner forces

F1 = O1(L1)/(L1+L2) =	589 lbf
F2 = O1(L2)/(L1+L2) =	591 lbf

5. Tributary length of openings

T1 = (L1*Lo1)/(L1+L2) =	2.99 ft
T2 = (L2*Lo1)/(L1+L2) =	3.01 ft

6. Unit shear beside opening

v1 = (V/L)(L1+T1)/L1 =	201 plf	
v2 = (V/L)(T2+L2)/L2 =	201 plf	
Check v1*L1+v2*L2=V?	1156 lbf	**OK**

7. Resistance to corner forces

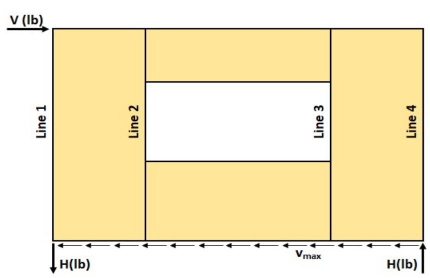
R1 = v1*L1 =	577 lbf
R2 = v2*L2 =	579 lbf

8. Difference corner force + resistance

R1-F1 =	-12 lbf
R2-F2 =	-12 lbf

9. Unit shear in corner zones

vc1 = (R1-F1)/L1 =	-4 plf
vc2 = (R2-F2)/L2 =	-4 plf



Check Summary of Shear Values for One Opening

Line 1: vc1(h_a+h_b)+v1(h_o)=H?		-19	905	885 lbf
Line 2: va1(h_a+h_b)-vc1(h_a+h_b)-v1(h_o)=0?	885	-19	905	0
Line 3: va1(h_a+h_b)-vc2(h_a+h_b)-v1(h_o)=0?	885	-19	905	0
Line 4: vc2(h_a+h_b)+v2(h_o)=H?		-19	905	885 lbf

Design Summary*

Req. Sheathing Capacity	201 plf	**4-Term Deflection**		**3-Term Deflection**	
Req. Strap Force	591 lbf	**4-Term Story Drift %**		**3-Term Story Drift %**	
Req. HD Force (H)	885 lbf				
Req. Shear Wall Anchorage Force (v_{max})	98 plf				

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

Project Information

Code:		**Date:**	6/2/2025
Designer:	Chon Pieruccioni, PE		
Client:			
Project:	East Town Crossing - Building A		
Wall Line:	Grid A - 11'-9" Segment - L2E		

Shear Wall Deflection Calculation Variables

Unfactored Shear Load $V_{unfactored}$: 1156 (lbf)

Sheathing Type: 7/16 OSB
Grade: APA Rated Sheathing

G_t Override:
G_a Override:

Wood End Post Values:
Species: HF#2
E: 1.30E+06 (psi)

Enter individual post sizes below.

C_d: 4.00

Nail Type: 8d common (penny weight)

	Pier 1	Pier 2	
Nail Spacing:	6	6	(in.)
HD Capacity:	2140	2140	(lbf)
HD Deflection:	0.088	0.088	(in.)

Four-Term Equation Deflection Check

$$\Delta = \frac{8vh^3}{EAb} + \frac{vh}{Gt} + 0.75he_n + d_a\frac{h}{b} \qquad \text{(Equation 23-2)}$$

Sheathing Type: 7/16 OSB APA Rated Sheathing
Nail Type: 8d common

	Pier 1-L	Pier 1-R	Pier 2-L	Pier 2-R	
$v_{unfactored}$:	201	201	201	201	(plf)
E:	1.30E+06	1.30E+06	1.30E+06	1.30E+06	(psi)
h:	9.00	5.60	5.60	9.00	(ft)
Qty:					
Stud Size:					
A Override:					(in.^{2})
A:					(in.^{2})
G_t:	83,500	83,500	83,500	83,500	(lbf/in.)
Nail Spacing:	6	6	6	6	(in.)
V_n:	101	101	101	101	(plf)
e_n:	0.0050	0.0050	0.0050	0.0050	(in.)
b:	2.87	2.87	2.88	2.88	(ft)
HD Capacity:	2140	2140	2140	2140	(lbf)
HD Defl:	0.088	0.088	0.088	0.088	(in.)

Check Total Deflection of Wall System

Pier 1 (left)				Pier 1 (right)			
Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 4 HD-1	Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 4 HD-2
	0.022	0.034	0.233		0.013	0.021	0.090
		Sum				Sum	
Pier 2 (left)				**Pier 2 (right)**			
Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 4 HD-1	Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 4 HD-2
	0.013	0.021	0.090		0.022	0.034	0.233
		Sum				Sum	

Total Defl.	
	(in.) %drift

Project Information

Code:		**Date:**	6/2/2025
Designer:	Chon Pieruccioni, PE		
Client:			
Project:	East Town Crossing - Building A		
Wall Line:	Grid A - 11'-9" Segment - L2E		

Shear Wall Deflection Calculation Variables

Unfactored Shear Load $V_{unfactored}$: 1156 (lbf)

Wood End Post Values:

Sheathing Type:	7/16 OSB	Species:	HF#2	Nail Type:	8d common	(penny weight)
Grade:	APA Rated Sheathing	E:	1.30E+06 (psi)			

					Pier 1	Pier 2	
G_t Override:				Nail Spacing:	6	6	(in.)
G_a Override:		C_d:	4.00	HD Capacity:	2140	2140	(lbf)
				HD Deflection:	0.088	0.088	(in.)

Three-Term Equation Deflection Check

$$\delta_{sw} = \frac{8vh^3}{EAb} + \frac{vh}{1000G_a} + \frac{h\Delta_a}{b} \qquad (4.3\text{-}1)$$

	Pier 1-L	Pier 1-R	Pier 2-L	Pier 2-R	
$v_{unfactored}$:	201	201	201	201	(plf)
E:	1.30E+06	1.30E+06	1.30E+06	1.30E+06	(psi)
h:	9.00	5.60	5.60	9.00	(ft)
Qty:					
Stud Size:					
A Override:					(in.^{2})
A:					(in.^{2})
G_a:	15.0	15.0	15.0	15.0	(kips/in.)
b:	2.87	2.87	2.88	2.88	(ft)
HD Capacity:	2140	2140	2140	2140	(lbf)
HD Defl:	0.088	0.088	0.088	0.088	(in.)

Sheathing Type: 7/16 OSB APA Rated Sheathing

Nail Type: 8d common

Check Total Deflection of Wall System

Pier 1 (left)			Pier 1 (right)		
Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 1 Bending	Term 2 Shear	Term 3 Fastener
	0.121	0.233		0.075	0.090
	Sum			Sum	
Pier 2 (left)			**Pier 2 (right)**		
Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 1 Bending	Term 2 Shear	Term 3 Fastener
	0.075	0.090		0.121	0.233
	Sum			Sum	

Total Defl.	
	(in.)
	%drift

Comment: The 3-term equation is calibrated to be approximately equal to 4-term equation at 1.4*ASD capacity.

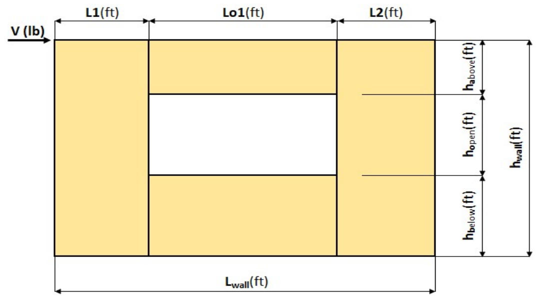
Force Transfer Around Openings Calculator

ONE OPENING

The force transfer around openings (FTAO) method of shear wall analysis is an approach that aims to reinforce the wall such that it performs as if there was no opening. This approach lends certain advantages over segmented shear walls: more versatility, because it allows for narrower wall segments while still meeting the height-to-width ratios and, often, fewer required hold-downs.

Project Information

Code:		**Date:**	6/2/2025
Designer:	Chon Pieruccioni, PE		
Client:			
Project:	East Town Crossing - Building A		
Wall Line:	Grid A - 11'-9" Segment - L1E		



Shear Wall Calculation Variables

V	1650 lbf	Opening 1			
L1	2.87 ft	h_a	1.10 ft		
L2	2.88 ft	h_o	4.50 ft		
h_{wall}	9.00 ft	h_b	3.40 ft		
L_{wall}	11.75 ft	Lo1	6.00 ft		

Adj. Factor Method =		1.25-0.125h/bs
Wall Pier Aspect Ratio		Adj. Factor
$P1=h_o/L1=$	1.57	N/A
$P2=h_o/L2=$	1.56	N/A

1. Hold-down forces: $H = Vh_{wall}/L_{wall}$ 1264 lbf

2. Unit shear above + below opening

First opening: $va1 = vb1 = H/(h_a+h_b) =$ 281 plf

3. Total boundary force above + below openings

First opening: O1 = va1 x (Lo1) = 1685 lbf

4. Corner forces

F1 = O1(L1)/(L1+L2) = 841 lbf
F2 = O1(L2)/(L1+L2) = 844 lbf

5. Tributary length of openings

T1 = (L1*Lo1)/(L1+L2) = 2.99 ft
T2 = (L2*Lo1)/(L1+L2) = 3.01 ft

6. Unit shear beside opening

v1 = (V/L)(L1+T1)/L1 = 287 plf
v2 = (V/L)(T2+L2)/L2 = 287 plf
Check v1*L1+v2*L2=V? 1650 lbf **OK**

7. Resistance to corner forces

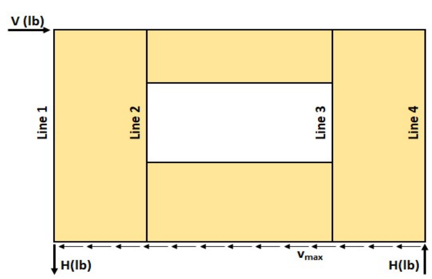
R1 = v1*L1 = 824 lbf
R2 = v2*L2 = 826 lbf

8. Difference corner force + resistance

R1-F1 = -18 lbf
R2-F2 = -18 lbf

9. Unit shear in corner zones

vc1 = (R1-F1)/L1 = -6 plf
vc2 = (R2-F2)/L2 = -6 plf



Check Summary of Shear Values for One Opening

Line 1: $vc1(h_a+h_b)+v1(h_o)=H$?		-27	1291	1264 lbf
Line 2: $va1(h_a+h_b)-vc1(h_a+h_b)-v1(h_o)=0$?	1264	-27	1291	0
Line 3: $va1(h_a+h_b)-vc2(h_a+h_b)-v1(h_o)=0$?	1264	-27	1291	0
Line 4: $vc2(h_a+h_b)+v2(h_o)=H$?		-27	1291	1264 lbf

Design Summary*

Req. Sheathing Capacity	287 plf	**4-Term Deflection**		**3-Term Deflection**	
Req. Strap Force	844 lbf	**4-Term Story Drift %**		**3-Term Story Drift %**	
Req. HD Force (H)	1264 lbf				
Req. Shear Wall Anchorage Force (v_{max})	140 plf				

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

Project Information

Code:		**Date:**	6/2/2025
Designer:	Chon Pieruccioni, PE		
Client:			
Project:	East Town Crossing - Building A		
Wall Line:	Grid A - 11'-9" Segment - L1E		

Shear Wall Deflection Calculation Variables

Unfactored Shear Load $V_{unfactored}$: 1650 (lbf)

Sheathing Type: 7/16 OSB
Grade: APA Rated Sheathing

G_t Override:
G_a Override:

Wood End Post Values:
Species: HF#2
E: 1.30E+06 (psi)

Enter individual post sizes below.

C_d: 4.00

Nail Type: 8d common (penny weight)

	Pier 1	Pier 2	
Nail Spacing:	4	4	(in.)
HD Capacity:	2140	2140	(lbf)
HD Deflection:	0.088	0.088	(in.)

Four-Term Equation Deflection Check

$$\Delta = \frac{8vh^3}{EAb} + \frac{vh}{Gt} + 0.75he_n + d_a\frac{h}{b} \qquad \text{(Equation 23-2)}$$

Sheathing Type: 7/16 OSB APA Rated Sheathing
Nail Type: 8d common

	Pier 1-L	Pier 1-R	Pier 2-L	Pier 2-R	
$v_{unfactored}$:	287	287	287	287	(plf)
E:	1.30E+06	1.30E+06	1.30E+06	1.30E+06	(psi)
h:	9.00	5.60	5.60	9.00	(ft)
Qty:					
Stud Size:					
A Override:					(in.^{2})
A:					(in.^{2})
G_t:	83,500	83,500	83,500	83,500	(lbf/in.)
Nail Spacing:	4	4	4	4	(in.)
V_n:	96	96	96	96	(plf)
e_n:	0.0043	0.0043	0.0043	0.0043	(in.)
b:	2.87	2.87	2.88	2.88	(ft)
HD Capacity:	2140	2140	2140	2140	(lbf)
HD Defl:	0.088	0.088	0.088	0.088	(in.)

Check Total Deflection of Wall System

Pier 1 (left)				Pier 1 (right)			
Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 4 HD-1	Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 4 HD-2
	0.031	0.029	0.333		0.019	0.018	0.129
		Sum				Sum	
Pier 2 (left)				**Pier 2 (right)**			
Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 4 HD-1	Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 4 HD-2
	0.019	0.018	0.128		0.031	0.029	0.332
		Sum				Sum	

Total Defl.	
	(in.) %drift

Project Information

Code:		**Date:**	6/2/2025
Designer:	Chon Pieruccioni, PE		
Client:			
Project:	East Town Crossing - Building A		
Wall Line:	Grid A - 11'-9" Segment - L1E		

Shear Wall Deflection Calculation Variables

Unfactored Shear Load $V_{unfactored}$: 1650 (lbf)

Sheathing Type:	7/16 OSB	Species:	HF#2	Nail Type:	8d common (penny weight)
Grade:	APA Rated Sheathing	E:	1.30E+06 (psi)		

Wood End Post Values:

					Pier 1	Pier 2	
G_t Override:				Nail Spacing:	4	4	(in.)
G_a Override:		C_d:	4.00	HD Capacity:	2140	2140	(lbf)
				HD Deflection:	0.088	0.088	(in.)

Three-Term Equation Deflection Check

$$\delta_{sw} = \frac{8vh^3}{EAb} + \frac{vh}{1000G_a} + \frac{h\Delta_a}{b} \quad (4.3\text{-}1)$$

	Pier 1-L	Pier 1-R	Pier 2-L	Pier 2-R	
$v_{unfactored}$:	287	287	287	287	(plf)
E:	1.30E+06	1.30E+06	1.30E+06	1.30E+06	(psi)
h:	9.00	5.60	5.60	9.00	(ft)
Qty:					
Stud Size:					
A Override:					(in.^{2})
A:					(in.^{2})
G_a:	22.0	22.0	22.0	22.0	(kips/in.)
b:	2.87	2.87	2.88	2.88	(ft)
HD Capacity:	2140	2140	2140	2140	(lbf)
HD Defl:	0.088	0.088	0.088	0.088	(in.)

Sheathing Type: 7/16 OSB APA Rated Sheathing

Nail Type: 8d common

Check Total Deflection of Wall System

Pier 1 (left)			Pier 1 (right)		
Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 1 Bending	Term 2 Shear	Term 3 Fastener
	0.117	0.333		0.073	0.129
	Sum			Sum	
Pier 2 (left)			**Pier 2 (right)**		
Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 1 Bending	Term 2 Shear	Term 3 Fastener
	0.073	0.128		0.117	0.332
	Sum			Sum	

Total Defl.	
	(in.)
	%drift

Comment: The 3-term equation is calibrated to be approximately equal to 4-term equation at 1.4*ASD capacity.

APA

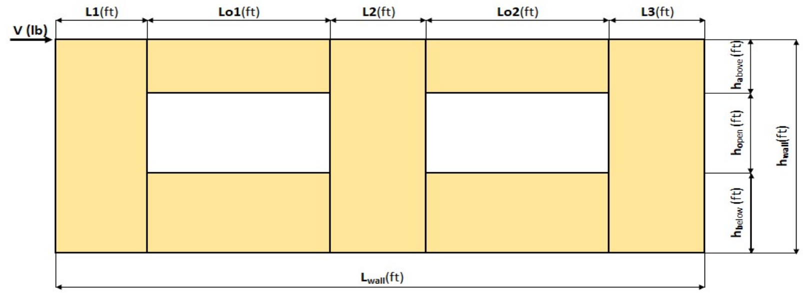
Force Transfer Around Openings Calculator

TWO OPENINGS

The force transfer around openings (FTAO) method of shear wall analysis is an approach that aims to reinforce the wall such that it performs as if there was no opening. This approach lends certain advantages over segmented shear walls: more versatility, because it allows for narrower wall segments while still meeting the height-to-width ratios and, often, fewer required hold-downs.

Project Information

Code:		**Date:**	6/2/2025
Designer:	Chon Pieruccioni, PE		
Client:			
Project:	East Town Crossing - Building A		
Wall Line:	Grid B - 20'-9" Segment - L2E		



Shear Wall Calculation Variables

V	2019 lbf	Opening 1		Opening 2		Adj. Factor Method =	1.25-0.125h/bs
L1	2.51 ft	h_a1	1.10 ft	h_a2	1.10 ft	Wall Pier Aspect Ratio	Adj. Factor
L2	2.76 ft	h_o1	4.50 ft	h_o2	4.50 ft	$P1=h_o/L1=$ 1.79	N/A
L3	2.48 ft	h_b1	3.40 ft	h_b2	3.40 ft	$P2=h_o/L2=$ 1.63	N/A
h_{wall}	9.00 ft	Lo1	6.00 ft	Lo2	6.00 ft	$P3=h_o/L3=$ 1.81	N/A
L_{wall}	19.75 ft						

1. Hold-down forces: $H = Vh_{wall}/L_{wall}$ 920 lbf

2. Unit shear above + below opening

First opening: $va1 = vb1 = H/(h_a1+h_b1) =$	204 plf
Second opening: $va2 = vb2 = H/(h_a2+h_b2) =$	204 plf

3. Total boundary force above + below openings

First opening: O1 = va1 x (Lo1) =	1227 lbf
Second opening: O2 = va2 x (Lo2) =	1227 lbf

4. Corner forces

F1 = O1(L1)/(L1+L2) =	584 lbf
F2 = O1(L2)/(L1+L2) =	642 lbf
F3 = O2(L2)/(L2+L3) =	646 lbf
F4 = O2(L3)/(L2+L3) =	581 lbf

5. Tributary length of openings

T1 = (L1*Lo1)/(L1+L2) =	2.86 ft
T2 = (L2*Lo1)/(L1+L2) =	3.14 ft
T3 = (L2*Lo2)/(L2+L3) =	3.16 ft
T4 = (L3*Lo2)/(L2+L3) =	2.84 ft

6. Unit shear beside opening

v1 = (V/L)(L1+T1)/L1 =	219 plf	
v2 = (V/L)(T2+L2+T3)/L2 =	336 plf	
v3 = (V/L)(T4+L3)/L3 =	219 plf	
Check v1*L1+v2*L2+v3*L3=V?	2019 lbf	**OK**

7. Resistance to corner forces

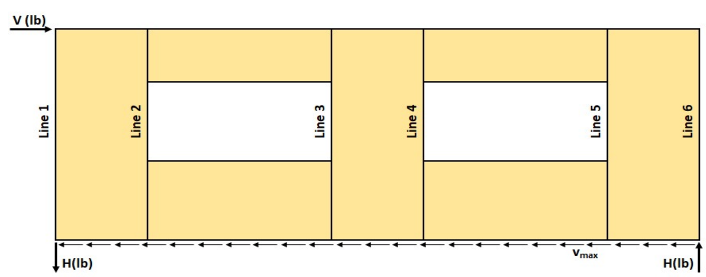
R1 = v1*L1 =	549 lbf
R2 = v2*L2 =	926 lbf
R3 = v3*L3 =	544 lbf

8. Difference corner force + resistance

R1-F1 =	-36 lbf
R2-F2-F3 =	-362 lbf
R3-F4 =	-37 lbf

9. Unit shear in corner zones

vc1 = (R1-F1)/L1 =	-14 plf
vc2 = (R2-F2-F3)/L2 =	-131 plf
vc3 = (R3-F4)/L3 =	-15 plf



Check Summary of Shear Values for Two Openings

Line 1: $vc1(h_a1+h_b1)+v1(h_o1)=H$?		-64	984	920 lbf
Line 2: $va1(h_a1+h_b1)-vc1(h_a1+h_b1)-v1(h_o1)=0$?	920	-64	984	0
Line 3: $vc2(h_a1+h_b1)+v2(h_o1)-va1(h_a1+h_b1)=0$?	-590	1511	920	0
Line 4: $va2(h_a2+h_b2)-v2(h_o2)-vc2(h_a2+h_b2)=0$?	920	1511	-590	0
Line 5: $va2(h_a2+h_b2)-vc3(h_a2+h_b2)-v3(h_o2)=0$?	920	-67	987	0
Line 6: $vc3(h_a2+h_b2)+ v3(h_o2) = H$?		-67	987	920 lbf

Design Summary*

Req. Sheathing Capacity	336 plf	**4-Term Deflection**		**3-Term Deflection**	
Req. Strap Force	646 lbf	**4-Term Story Drift %**		**3-Term Story Drift %**	
Req. HD Force	920 lbf				
Req. Shear Wall Anchorage Force	102 plf				

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

Project Information

Code:		**Date:**	6/2/2025
Designer:	Chon Pieruccioni, PE		
Client:			
Project:	East Town Crossing - Building A		
Wall Line:	Grid B - 20'-9" Segment - L2E		

Shear Wall Deflection Calculation Variables

Unfactored Shear Load $V_{unfactored}$: 2019 (lbf)

Sheathing Type: 7/16 OSB
Grade: APA Rated Sheathing

G_t Override:
G_a Override:

Wood End Post Values:
Species: HF#2
E: 1.30E+06 (psi)

Enter individual post sizes below.

C_d: 4.00

Nail Type: 8d common (penny weight)

	Pier 1	Pier 3	
Nail Spacing:	4	4	(in.)
HD Capacity:	2140	2140	(lbf)
HD Deflection:	0.088	0.088	(in.)

Four-Term Equation Deflection Check

$$\Delta = \frac{8vh^3}{EAb} + \frac{vh}{Gt} + 0.75he_n + d_a\frac{h}{b} \quad \text{(Equation 23-2)}$$

	Pier 1-L	Pier 1-R	Pier 2-L	Pier 2-R	Pier 3-L	Pier 3-R	
$v_{unfactored}$:	219	219	336	336	219	219	(plf)
E:	1.30E+06	1.30E+06	1.30E+06	1.30E+06	1.30E+06	1.30E+06	(psi)
h:	9.00	5.60	5.60	5.60	5.60	9.00	(ft)
Qty:							
Stud Size:							
A Override:							(in.^{2})
A:							(in.^{2})
G_t:	83,500	83,500	83,500	83,500	83,500	83,500	(lbf/in.)
Nail Spacing:	4	4	4	4	4	4	(in.)
V_n:	73	73	112	112	73	73	(plf)
e_n:	0.0019	0.0019	0.0070	0.0070	0.0019	0.0019	(in.)
b:	2.51	2.51	2.76	2.76	2.48	2.48	(ft)
HD Capacity:	2140	2140	2140	2140	2140	2140	(lbf)
HD Defl:	0.088	0.088	0.088	0.088	0.088	0.088	(in.)

Sheathing Type: 7/16 OSB APA Rated Sheathing
Nail Type: 8d common

Check Total Deflection of Wall System

Pier 1 (left)				Pier 1 (right)			
Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 4 HD-1	Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 4 HD-2
	0.024	0.013	0.290		0.015	0.008	0.112
		Sum				Sum	
Pier 2 (left)				**Pier 2 (right)**			
Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 4 HD-1	Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 4 HD-2
	0.023	0.029	0.157		0.023	0.029	0.157
		Sum				Sum	
Pier 3 (left)				**Pier 3 (right)**			
Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 4 HD-1	Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 4 HD-2
	0.015	0.008	0.114		0.024	0.013	0.295
		Sum				Sum	

Total Defl.	
	(in.)
	%drift

Project Information

Code:		**Date:** 6/2/2025
Designer:	Chon Pieruccioni, PE	
Client:		
Project:	East Town Crossing - Building A	
Wall Line:	Grid B - 20'-9" Segment - L2E	

Shear Wall Deflection Calculation Variables

Unfactored Shear Load $V_{unfactored}$: 2019 (lbf)

Wood End Post Values:

Sheathing Type:	7/16 OSB	Species:	HF#2	Nail Type:	8d common (penny weight)
Grade:	APA Rated Sheathing	E:	1.30E+06 (psi)		

					Pier 1	Pier 3	
G_t Override:				Nail Spacing:	4	4	(in.)
G_a Override:		C_d:	4.00	HD Capacity:	2140	2140	(lbf)
				HD Deflection:	0.088	0.088	(in.)

Three-Term Equation Deflection Check

$$\delta_{sw} = \frac{8vh^3}{EAb} + \frac{vh}{1000G_a} + \frac{h\Delta_a}{b} \qquad (4.3\text{-}1)$$

	Pier 1-L	Pier 1-R	Pier 2-L	Pier 2-R	Pier 3-L	Pier 3-R	
$v_{unfactored}$:	219	219	336	336	219	219	(plf)
E:	1.30E+06	1.30E+06	1.30E+06	1.30E+06	1.30E+06	1.30E+06	(psi)
h:	9.00	5.60	5.60	5.60	5.60	9.00	(ft)
Qty:							
Stud Size:							
A Override:							(in.^{2})
A:							(in.^{2})
G_a:	22.0	22.0	22.0	22.0	22.0	22.0	(kips/in.)
b:	2.51	2.51	2.76	2.76	2.48	2.48	(ft)
HD Capacity:	2140	2140	2140	2140	2140	2140	(lbf)
HD Defl:	0.088	0.088	0.088	0.088	0.088	0.088	(in.)

Sheathing Type: 7/16 OSB APA Rated Sheathing

Nail Type: 8d common

Check Total Deflection of Wall System

Pier 1 (left)			Pier 1 (right)		
Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 1 Bending	Term 2 Shear	Term 3 Fastener
	0.089	0.290		0.056	0.112
	Sum			Sum	
Pier 2 (left)			**Pier 2 (right)**		
Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 1 Bending	Term 2 Shear	Term 3 Fastener
	0.085	0.157		0.085	0.157
	Sum			Sum	
Pier 3 (left)			**Pier 3 (right)**		
Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 1 Bending	Term 2 Shear	Term 3 Fastener
	0.056	0.114		0.090	0.295
	Sum			Sum	

Total Defl.	
	(in.)
	%drift

Comment: The 3-term equation is calibrated to be approximately equal to 4-term equation at 1.4*ASD capacity.

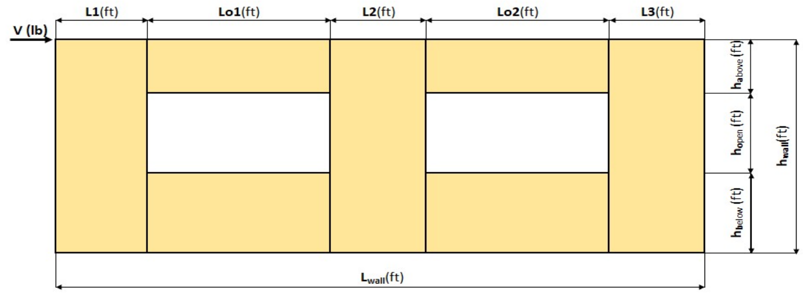
Force Transfer Around Openings Calculator

TWO OPENINGS

The force transfer around openings (FTAO) method of shear wall analysis is an approach that aims to reinforce the wall such that it performs as if there was no opening. This approach lends certain advantages over segmented shear walls: more versatility, because it allows for narrower wall segments while still meeting the height-to-width ratios and, often, fewer required hold-downs.

Project Information

Code:		**Date:**	6/2/2025
Designer:	Chon Pieruccioni, PE		
Client:			
Project:	East Town Crossing - Building A		
Wall Line:	Grid B - 20'-9" Segment - L1E		



Shear Wall Calculation Variables

			Opening 1		Opening 2		
V	2913 lbf						
L1	2.51 ft	h_a1	1.10 ft	h_a2	1.10 ft		
L2	2.76 ft	h_o1	4.50 ft	h_o2	4.50 ft		
L3	2.48 ft	h_b1	3.40 ft	h_b2	3.40 ft		
h_{wall}	9.00 ft	Lo1	6.00 ft	Lo2	6.00 ft		
L_{wall}	19.75 ft						

Adj. Factor Method =		1.25-0.125h/bs
Wall Pier Aspect Ratio		Adj. Factor
$P1=h_o/L1=$	1.79	N/A
$P2=h_o/L2=$	1.63	N/A
$P3=h_o/L3=$	1.81	N/A

1. Hold-down forces: $H = Vh_{wall}/L_{wall}$ 1327 lbf

2. Unit shear above + below opening

First opening: $va1 = vb1 = H/(h_a1+h_b1) =$	295 plf
Second opening: $va2 = vb2 = H/(h_a2+h_b2) =$	295 plf

3. Total boundary force above + below openings

First opening: O1 = va1 x (Lo1) =	1770 lbf
Second opening: O2 = va2 x (Lo2) =	1770 lbf

4. Corner forces

F1 = O1(L1)/(L1+L2) =	843 lbf
F2 = O1(L2)/(L1+L2) =	927 lbf
F3 = O2(L2)/(L2+L3) =	932 lbf
F4 = O2(L3)/(L2+L3) =	838 lbf

5. Tributary length of openings

T1 = (L1*Lo1)/(L1+L2) =	2.86 ft
T2 = (L2*Lo1)/(L1+L2) =	3.14 ft
T3 = (L2*Lo2)/(L2+L3) =	3.16 ft
T4 = (L3*Lo2)/(L2+L3) =	2.84 ft

6. Unit shear beside opening

v1 = (V/L)(L1+T1)/L1 =	315 plf	
v2 = (V/L)(T2+L2+T3)/L2 =	484 plf	
v3 = (V/L)(T4+L3)/L3 =	316 plf	
Check v1*L1+v2*L2+v3*L3=V?	2913 lbf	**OK**

7. Resistance to corner forces

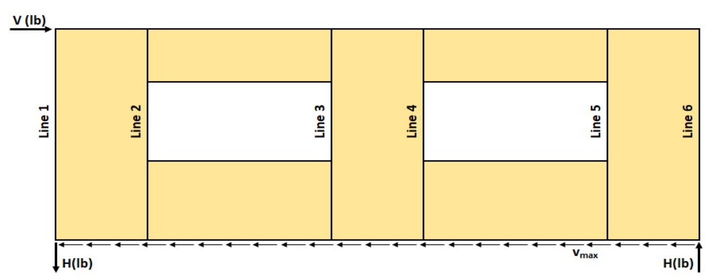
R1 = v1*L1 =	792 lbf
R2 = v2*L2 =	1337 lbf
R3 = v3*L3 =	785 lbf

8. Difference corner force + resistance

R1-F1 =	-51 lbf
R2-F2-F3 =	-523 lbf
R3-F4 =	-53 lbf

9. Unit shear in corner zones

vc1 = (R1-F1)/L1 =	-20 plf
vc2 = (R2-F2-F3)/L2 =	-189 plf
vc3 = (R3-F4)/L3 =	-21 plf



Check Summary of Shear Values for Two Openings

Line 1: $vc1(h_a1+h_b1)+v1(h_o1)=H?$		-92	1419	1327 lbf
Line 2: $va1(h_a1+h_b1)-vc1(h_a1+h_b1)-v1(h_o1)=0?$	1327	-92	1419	0
Line 3: $vc2(h_a1+h_b1)+v2(h_o1)-va1(h_a1+h_b1)=0?$	-852	2179	1327	0
Line 4: $va2(h_a2+h_b2)-v2(h_o2)-vc2(h_a2+h_b2)=0?$	1327	2179	-852	0
Line 5: $va2(h_a2+h_b2)-vc3(h_a2+h_b2)-v3(h_o2)=0?$	1327	-96	1424	0
Line 6: $vc3(h_a2+h_b2)+ v3(h_o2) = H?$		-96	1424	1327 lbf

Design Summary*

Req. Sheathing Capacity	484 plf	**4-Term Deflection**		**3-Term Deflection**	
Req. Strap Force	932 lbf	**4-Term Story Drift %**		**3-Term Story Drift %**	
Req. HD Force	1327 lbf				
Req. Shear Wall Anchorage Force	147 plf				

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

Project Information

Code:		**Date:**	6/2/2025
Designer:	Chon Pieruccioni, PE		
Client:			
Project:	East Town Crossing - Building A		
Wall Line:	Grid B - 20'-9" Segment - L1E		

Shear Wall Deflection Calculation Variables

Unfactored Shear Load $V_{unfactored}$: 2913 (lbf)

Sheathing Type: 7/16 OSB
Grade: APA Rated Sheathing

G_t Override:
G_a Override:

Wood End Post Values:
Species: HF#2
E: 1.30E+06 (psi)

Enter individual post sizes below.

C_d: 4.00

Nail Type: 8d common (penny weight)

	Pier 1	Pier 3	
Nail Spacing:	2	2	(in.)
HD Capacity:	2140	2140	(lbf)
HD Deflection:	0.088	0.088	(in.)

Four-Term Equation Deflection Check

$$\Delta = \frac{8vh^3}{EAb} + \frac{vh}{Gt} + 0.75he_n + d_a\frac{h}{b} \qquad \text{(Equation 23-2)}$$

	Pier 1-L	Pier 1-R	Pier 2-L	Pier 2-R	Pier 3-L	Pier 3-R	
$v_{unfactored}$:	315	315	484	484	316	316	(plf)
E:	1.30E+06	1.30E+06	1.30E+06	1.30E+06	1.30E+06	1.30E+06	(psi)
h:	9.00	5.60	5.60	5.60	5.60	9.00	(ft)
Qty:							
Stud Size:							
A Override:							(in.^{2})
A:							(in.^{2})
G_t:	83,500	83,500	83,500	83,500	83,500	83,500	(lbf/in.)
Nail Spacing:	2	2	2	2	2	2	(in.)
V_n:	53	53	81	81	53	53	(plf)
e_n:	0.0007	0.0007	0.0026	0.0026	0.0007	0.0007	(in.)
b:	2.51	2.51	2.76	2.76	2.48	2.48	(ft)
HD Capacity:	2140	2140	2140	2140	2140	2140	(lbf)
HD Defl:	0.088	0.088	0.088	0.088	0.088	0.088	(in.)

Sheathing Type: 7/16 OSB APA Rated Sheathing
Nail Type: 8d common

Check Total Deflection of Wall System

Pier 1 (left)				Pier 1 (right)			
Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 4 HD-1	Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 4 HD-2
	0.034	0.005	0.419		0.021	0.003	0.162
		Sum				Sum	
Pier 2 (left)				**Pier 2 (right)**			
Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 4 HD-1	Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 4 HD-2
	0.032	0.011	0.226		0.032	0.011	0.226
		Sum				Sum	
Pier 3 (left)				**Pier 3 (right)**			
Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 4 HD-1	Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 4 HD-2
	0.021	0.003	0.165		0.034	0.005	0.425
		Sum				Sum	

Total Defl.	
	(in.)
	%drift

Project Information

Code:		**Date:**	6/2/2025
Designer:	Chon Pieruccioni, PE		
Client:			
Project:	East Town Crossing - Building A		
Wall Line:	Grid B - 20'-9" Segment - L1E		

Shear Wall Deflection Calculation Variables

Unfactored Shear Load $V_{unfactored}$: 2913 (lbf)

Wood End Post Values:

Sheathing Type:	7/16 OSB	Species:	HF#2	Nail Type:	8d common	(penny weight)	
Grade:	APA Rated Sheathing	E:	1.30E+06 (psi)				
					Pier 1	Pier 3	
G_t Override:				Nail Spacing:	2	2	(in.)
G_a Override:		C_d:	4.00	HD Capacity:	2140	2140	(lbf)
				HD Deflection:	0.088	0.088	(in.)

Three-Term Equation Deflection Check

$$\delta_{sw} = \frac{8vh^3}{EAb} + \frac{vh}{1000G_a} + \frac{h\Delta_a}{b} \quad (4.3\text{-}1)$$

	Pier 1-L	Pier 1-R	Pier 2-L	Pier 2-R	Pier 3-L	Pier 3-R	
$v_{unfactored}$:	315	315	484	484	316	316	(plf)
E:	1.30E+06	1.30E+06	1.30E+06	1.30E+06	1.30E+06	1.30E+06	(psi)
h:	9.00	5.60	5.60	5.60	5.60	9.00	(ft)
Qty:							
Stud Size:							
A Override:							(in.^{2})
A:							(in.^{2})
G_a:	42.0	42.0	42.0	42.0	42.0	42.0	(kips/in.)
b:	2.51	2.51	2.76	2.76	2.48	2.48	(ft)
HD Capacity:	2140	2140	2140	2140	2140	2140	(lbf)
HD Defl:	0.088	0.088	0.088	0.088	0.088	0.088	(in.)

Sheathing Type: 7/16 OSB APA Rated Sheathing

Nail Type: 8d common

Check Total Deflection of Wall System

Pier 1 (left)			Pier 1 (right)		
Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 1 Bending	Term 2 Shear	Term 3 Fastener
	0.068	0.419		0.042	0.162
	Sum			Sum	
Pier 2 (left)			**Pier 2 (right)**		
Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 1 Bending	Term 2 Shear	Term 3 Fastener
	0.065	0.226		0.065	0.226
	Sum			Sum	
Pier 3 (left)			**Pier 3 (right)**		
Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 1 Bending	Term 2 Shear	Term 3 Fastener
	0.042	0.165		0.068	0.425
	Sum			Sum	

Total Defl.	
	(in.)
	%drift

Comment: The 3-term equation is calibrated to be approximately equal to 4-term equation at 1.4*ASD capacity.

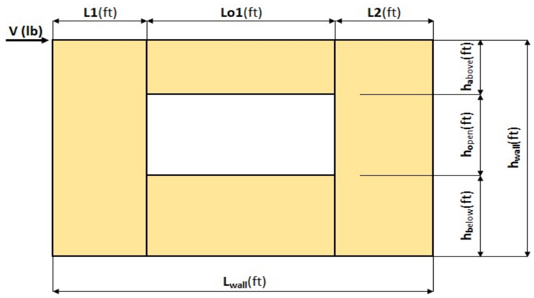
Force Transfer Around Openings Calculator

ONE OPENING

The force transfer around openings (FTAO) method of shear wall analysis is an approach that aims to reinforce the wall such that it performs as if there was no opening. This approach lends certain advantages over segmented shear walls: more versatility, because it allows for narrower wall segments while still meeting the height-to-width ratios and, often, fewer required hold-downs.

Project Information

Code:		**Date:**	6/2/2025
Designer:	Chon Pieruccioni, PE		
Client:			
Project:	East Town Crossing - Building A		
Wall Line:	Grid G - 13'-4" Segment - L3E		



Shear Wall Calculation Variables

V	1567 lbf	Opening 1		Adj. Factor Method =	1.25-0.125h/bs	
L1	4.50 ft	h_a	1.10 ft	Wall Pier Aspect Ratio		Adj. Factor
L2	3.83 ft	h_o	4.50 ft	$P1=h_o/L1=$	1.00	N/A
h_{wall}	9.00 ft	h_b	3.40 ft	$P2=h_o/L2=$	1.17	N/A
L_{wall}	13.33 ft	Lo1	5.00 ft			

1. Hold-down forces: $H = Vh_{wall}/L_{wall}$ — 1058 lbf

2. Unit shear above + below opening

First opening: $va1 = vb1 = H/(h_a+h_b) =$	235 plf

3. Total boundary force above + below openings

First opening: O1 = va1 x (Lo1) =	1176 lbf

4. Corner forces

F1 = O1(L1)/(L1+L2) =	635 lbf
F2 = O1(L2)/(L1+L2) =	540 lbf

5. Tributary length of openings

T1 = (L1*Lo1)/(L1+L2) =	2.70 ft
T2 = (L2*Lo1)/(L1+L2) =	2.30 ft

6. Unit shear beside opening

v1 = (V/L)(L1+T1)/L1 =	188 plf	
v2 = (V/L)(T2+L2)/L2 =	188 plf	
Check v1*L1+v2*L2=V?	1567 lbf	**OK**

7. Resistance to corner forces

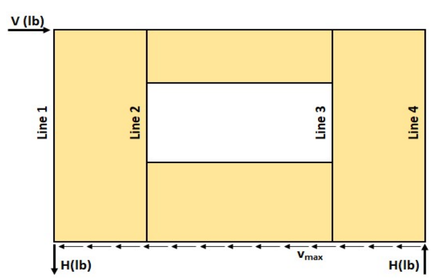
R1 = v1*L1 =	847 lbf
R2 = v2*L2 =	720 lbf

8. Difference corner force + resistance

R1-F1 =	211 lbf
R2-F2 =	180 lbf

9. Unit shear in corner zones

vc1 = (R1-F1)/L1 =	47 plf
vc2 = (R2-F2)/L2 =	47 plf



Check Summary of Shear Values for One Opening

Line 1: $vc1(h_a+h_b)+v1(h_o)=H?$		211	847	1058 lbf
Line 2: $va1(h_a+h_b)-vc1(h_a+h_b)-v1(h_o)=0?$	1058	211	847	0
Line 3: $va1(h_a+h_b)-vc2(h_a+h_b)-v1(h_o)=0?$	1058	211	847	0
Line 4: $vc2(h_a+h_b)+v2(h_o)=H?$		211	847	1058 lbf

Design Summary*

Req. Sheathing Capacity	235 plf	**4-Term Deflection**		**3-Term Deflection**	
Req. Strap Force	635 lbf	**4-Term Story Drift %**		**3-Term Story Drift %**	
Req. HD Force (H)	1058 lbf				
Req. Shear Wall Anchorage Force (v_{max})	118 plf				

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

Project Information

Code:		**Date:**	6/2/2025
Designer:	Chon Pieruccioni, PE		
Client:			
Project:	East Town Crossing - Building A		
Wall Line:	Grid G - 13'-4" Segment - L3E		

Shear Wall Deflection Calculation Variables

Unfactored Shear Load $V_{unfactored}$: 1567 (lbf)

Sheathing Type: 7/16 OSB
Grade: APA Rated Sheathing

G_t Override:
G_a Override:

Wood End Post Values:
Species: HF#2
E: 1.30E+06 (psi)

Enter individual post sizes below.

C_d: 4.00

Nail Type: 8d common (penny weight)

	Pier 1	Pier 2	
Nail Spacing:	6	6	(in.)
HD Capacity:	2140	2140	(lbf)
HD Deflection:	0.088	0.088	(in.)

Four-Term Equation Deflection Check

$$\Delta = \frac{8vh^3}{EAb} + \frac{vh}{Gt} + 0.75he_n + d_a\frac{h}{b} \qquad \text{(Equation 23-2)}$$

Sheathing Type: 7/16 OSB APA Rated Sheathing
Nail Type: 8d common

	Pier 1-L	Pier 1-R	Pier 2-L	Pier 2-R	
$v_{unfactored}$:	188	188	188	188	(plf)
E:	1.30E+06	1.30E+06	1.30E+06	1.30E+06	(psi)
h:	9.00	5.60	5.60	9.00	(ft)
Qty:					
Stud Size:					
A Override:					(in.^{2})
A:					(in.^{2})
G_t:	83,500	83,500	83,500	83,500	(lbf/in.)
Nail Spacing:	6	6	6	6	(in.)
V_n:	94	94	94	94	(plf)
e_n:	0.0041	0.0041	0.0041	0.0041	(in.)
b:	4.50	4.50	3.83	3.83	(ft)
HD Capacity:	2140	2140	2140	2140	(lbf)
HD Defl:	0.088	0.088	0.088	0.088	(in.)

Check Total Deflection of Wall System

Pier 1 (left)				Pier 1 (right)			
Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 4 HD-1	Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 4 HD-2
	0.020	0.028	0.139		0.013	0.017	0.054
		Sum				Sum	
Pier 2 (left)				**Pier 2 (right)**			
Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 4 HD-1	Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 4 HD-2
	0.013	0.017	0.063		0.020	0.028	0.164
		Sum				Sum	

Total Defl.	
	(in.) %drift

Project Information

Code:		**Date:**	6/2/2025
Designer:	Chon Pieruccioni, PE		
Client:			
Project:	East Town Crossing - Building A		
Wall Line:	Grid G - 13'-4" Segment - L3E		

Shear Wall Deflection Calculation Variables

Unfactored Shear Load $V_{unfactored}$: 1567 (lbf)

Wood End Post Values:

Sheathing Type:	7/16 OSB	Species:	HF#2	Nail Type:	8d common (penny weight)
Grade:	APA Rated Sheathing	E:	1.30E+06 (psi)		

				Pier 1	Pier 2	
G_t Override:			Nail Spacing:	6	6	(in.)
G_a Override:		C_d: 4.00	HD Capacity:	2140	2140	(lbf)
			HD Deflection:	0.088	0.088	(in.)

Three-Term Equation Deflection Check

$$\delta_{sw} = \frac{8vh^3}{EAb} + \frac{vh}{1000G_a} + \frac{h\Delta_a}{b} \qquad (4.3\text{-}1)$$

	Pier 1-L	Pier 1-R	Pier 2-L	Pier 2-R	
$v_{unfactored}$:	188	188	188	188	(plf)
E:	1.30E+06	1.30E+06	1.30E+06	1.30E+06	(psi)
h:	9.00	5.60	5.60	9.00	(ft)
Qty:					
Stud Size:					
A Override:					(in.^{2})
A:					(in.^{2})
G_a:	15.0	15.0	15.0	15.0	(kips/in.)
b:	4.50	4.50	3.83	3.83	(ft)
HD Capacity:	2140	2140	2140	2140	(lbf)
HD Defl:	0.088	0.088	0.088	0.088	(in.)

Sheathing Type: 7/16 OSB APA Rated Sheathing

Nail Type: 8d common

Check Total Deflection of Wall System

Pier 1 (left)			Pier 1 (right)		
Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 1 Bending	Term 2 Shear	Term 3 Fastener
	0.113	0.139		0.070	0.054
	Sum			Sum	
Pier 2 (left)			**Pier 2 (right)**		
Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 1 Bending	Term 2 Shear	Term 3 Fastener
	0.070	0.063		0.113	0.164
	Sum			Sum	

Total Defl.	
	(in.)
	%drift

Comment: The 3-term equation is calibrated to be approximately equal to 4-term equation at 1.4*ASD capacity.

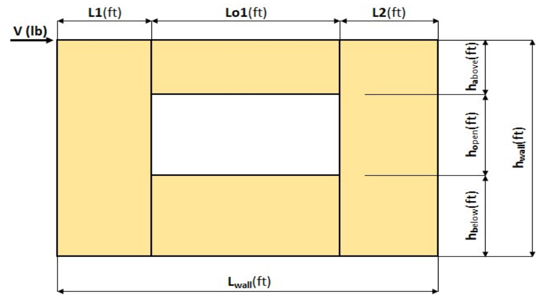
Force Transfer Around Openings Calculator

ONE OPENING

The force transfer around openings (FTAO) method of shear wall analysis is an approach that aims to reinforce the wall such that it performs as if there was no opening. This approach lends certain advantages over segmented shear walls: more versatility, because it allows for narrower wall segments while still meeting the height-to-width ratios and, often, fewer required hold-downs.

Project Information

Code:		**Date:**	6/2/2025
Designer:	Chon Pieruccioni, PE		
Client:			
Project:	East Town Crossing - Building A		
Wall Line:	Grid G - 13'-4" Segment - L2E		



Shear Wall Calculation Variables

V	2545 lbf		Opening 1		Adj. Factor Method =	1.25-0.125h/bs
L1	4.50 ft	h_a	1.10 ft		Wall Pier Aspect Ratio	Adj. Factor
L2	3.83 ft	h_o	4.50 ft		$P1=h_o/L1=$ 1.00	N/A
h_{wall}	9.00 ft	h_b	3.40 ft		$P2=h_o/L2=$ 1.17	N/A
L_{wall}	13.33 ft	Lo1	5.00 ft			

1. Hold-down forces: $H = Vh_{wall}/L_{wall}$ 1718 lbf

2. Unit shear above + below opening

First opening: $va1 = vb1 = H/(h_a+h_b) =$	382 plf

3. Total boundary force above + below openings

First opening: O1 = va1 x (Lo1) =	1909 lbf

4. Corner forces

F1 = O1(L1)/(L1+L2) =	1031 lbf
F2 = O1(L2)/(L1+L2) =	878 lbf

5. Tributary length of openings

T1 = (L1*Lo1)/(L1+L2) =	2.70 ft
T2 = (L2*Lo1)/(L1+L2) =	2.30 ft

6. Unit shear beside opening

v1 = (V/L)(L1+T1)/L1 =	306 plf	
v2 = (V/L)(T2+L2)/L2 =	306 plf	
Check v1*L1+v2*L2=V?	2545 lbf	**OK**

7. Resistance to corner forces

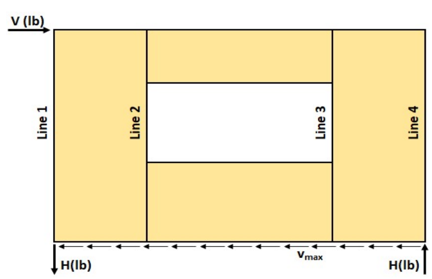
R1 = v1*L1 =	1375 lbf
R2 = v2*L2 =	1170 lbf

8. Difference corner force + resistance

R1-F1 =	343 lbf
R2-F2 =	292 lbf

9. Unit shear in corner zones

vc1 = (R1-F1)/L1 =	76 plf
vc2 = (R2-F2)/L2 =	76 plf



Check Summary of Shear Values for One Opening

Line 1: $vc1(h_a+h_b)+v1(h_o)=H?$		343	1375	1718 lbf
Line 2: $va1(h_a+h_b)-vc1(h_a+h_b)-v1(h_o)=0?$	1718	343	1375	0
Line 3: $va1(h_a+h_b)-vc2(h_a+h_b)-v1(h_o)=0?$	1718	343	1375	0
Line 4: $vc2(h_a+h_b)+v2(h_o)=H?$		343	1375	1718 lbf

Design Summary*

Req. Sheathing Capacity	382 plf	**4-Term Deflection**		**3-Term Deflection**	
Req. Strap Force	1031 lbf	**4-Term Story Drift %**		**3-Term Story Drift %**	
Req. HD Force (H)	1718 lbf				
Req. Shear Wall Anchorage Force (v_{max})	191 plf				

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

Project Information

Code:		**Date:**	6/2/2025
Designer:	Chon Pieruccioni, PE		
Client:			
Project:	East Town Crossing - Building A		
Wall Line:	Grid G - 13'-4" Segment - L2E		

Shear Wall Deflection Calculation Variables

Unfactored Shear Load $V_{unfactored}$: 2545 (lbf)

Sheathing Type: 7/16 OSB
Grade: APA Rated Sheathing

G_t Override:
G_a Override:

Wood End Post Values:
Species: HF#2
E: 1.30E+06 (psi)

Enter individual post sizes below.

C_d: 4.00

Nail Type: 8d common (penny weight)

	Pier 1	Pier 2	
Nail Spacing:	3	3	(in.)
HD Capacity:	2140	2140	(lbf)
HD Deflection:	0.088	0.088	(in.)

Four-Term Equation Deflection Check

$$\Delta = \frac{8vh^3}{EAb} + \frac{vh}{Gt} + 0.75he_n + d_a\frac{h}{b} \qquad \text{(Equation 23-2)}$$

Sheathing Type: 7/16 OSB APA Rated Sheathing
Nail Type: 8d common

	Pier 1-L	Pier 1-R	Pier 2-L	Pier 2-R	
$v_{unfactored}$:	306	306	306	306	(plf)
E:	1.30E+06	1.30E+06	1.30E+06	1.30E+06	(psi)
h:	9.00	5.60	5.60	9.00	(ft)
Qty:					
Stud Size:					
A Override:					(in.^{2})
A:					(in.^{2})
G_t:	83,500	83,500	83,500	83,500	(lbf/in.)
Nail Spacing:	3	3	3	3	(in.)
V_n:	76	76	76	76	(plf)
e_n:	0.0022	0.0022	0.0022	0.0022	(in.)
b:	4.50	4.50	3.83	3.83	(ft)
HD Capacity:	2140	2140	2140	2140	(lbf)
HD Defl:	0.088	0.088	0.088	0.088	(in.)

Check Total Deflection of Wall System

Pier 1 (left)				Pier 1 (right)			
Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 4 HD-1	Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 4 HD-2
	0.033	0.015	0.226		0.020	0.009	0.088
		Sum				Sum	
Pier 2 (left)				**Pier 2 (right)**			
Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 4 HD-1	Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 4 HD-2
	0.020	0.009	0.103		0.033	0.015	0.266
		Sum				Sum	

Total Defl.	
	(in.)
	%drift

Project Information

Code:		**Date:**	6/2/2025
Designer:	Chon Pieruccioni, PE		
Client:			
Project:	East Town Crossing - Building A		
Wall Line:	Grid G - 13'-4" Segment - L2E		

Shear Wall Deflection Calculation Variables

Unfactored Shear Load $V_{unfactored}$: 2545 (lbf)

Wood End Post Values:

Sheathing Type:	7/16 OSB	Species:	HF#2	Nail Type:	8d common (penny weight)
Grade:	APA Rated Sheathing	E:	1.30E+06 (psi)		

				Pier 1	Pier 2	
G_t Override:			Nail Spacing:	3	3	(in.)
G_a Override:		C_d: 4.00	HD Capacity:	2140	2140	(lbf)
			HD Deflection:	0.088	0.088	(in.)

Three-Term Equation Deflection Check

$$\delta_{sw} = \frac{8vh^3}{EAb} + \frac{vh}{1000G_a} + \frac{h\Delta_a}{b} \qquad (4.3\text{-}1)$$

Sheathing Type: 7/16 OSB APA Rated Sheathing

Nail Type: 8d common

	Pier 1-L	Pier 1-R	Pier 2-L	Pier 2-R	
$v_{unfactored}$:	306	306	306	306	(plf)
E:	1.30E+06	1.30E+06	1.30E+06	1.30E+06	(psi)
h:	9.00	5.60	5.60	9.00	(ft)
Qty:					
Stud Size:					
A Override:					(in.^{2})
A:					(in.^{2})
G_a:	28.0	28.0	28.0	28.0	(kips/in.)
b:	4.50	4.50	3.83	3.83	(ft)
HD Capacity:	2140	2140	2140	2140	(lbf)
HD Defl:	0.088	0.088	0.088	0.088	(in.)

Check Total Deflection of Wall System

Pier 1 (left)			Pier 1 (right)		
Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 1 Bending	Term 2 Shear	Term 3 Fastener
	0.098	0.226		0.061	0.088
	Sum			Sum	
Pier 2 (left)			**Pier 2 (right)**		
Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 1 Bending	Term 2 Shear	Term 3 Fastener
	0.061	0.103		0.098	0.266
	Sum			Sum	

Total Defl.	
	(in.)
	%drift

Comment: The 3-term equation is calibrated to be approximately equal to 4-term equation at 1.4*ASD capacity.

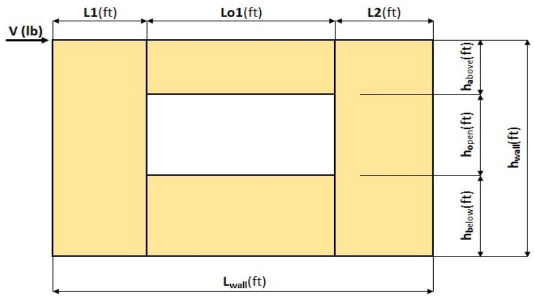
Force Transfer Around Openings Calculator

ONE OPENING

The force transfer around openings (FTAO) method of shear wall analysis is an approach that aims to reinforce the wall such that it performs as if there was no opening. This approach lends certain advantages over segmented shear walls: more versatility, because it allows for narrower wall segments while still meeting the height-to-width ratios and, often, fewer required hold-downs.

Project Information

Code:		**Date:**	6/2/2025
Designer:	Chon Pieruccioni, PE		
Client:			
Project:	East Town Crossing - Building A		
Wall Line:	Grid G - 13'-4" Segment - L1E		



Shear Wall Calculation Variables

V	2961 lbf		Opening 1	Adj. Factor Method =	1.25-0.125h/bs	
L1	4.50 ft	h_a	1.10 ft	Wall Pier Aspect Ratio		Adj. Factor
L2	3.83 ft	h_o	4.50 ft	$P1=h_o/L1=$	1.00	N/A
h_{wall}	9.00 ft	h_b	3.40 ft	$P2=h_o/L2=$	1.17	N/A
L_{wall}	13.33 ft	Lo1	5.00 ft			

1. Hold-down forces: $H = Vh_{wall}/L_{wall}$ 1999 lbf

2. Unit shear above + below opening

First opening: $va1 = vb1 = H/(h_a+h_b) =$ 444 plf

3. Total boundary force above + below openings

First opening: O1 = va1 x (Lo1) = 2221 lbf

4. Corner forces

F1 = O1(L1)/(L1+L2) = 1200 lbf

F2 = O1(L2)/(L1+L2) = 1021 lbf

5. Tributary length of openings

T1 = (L1*Lo1)/(L1+L2) = 2.70 ft

T2 = (L2*Lo1)/(L1+L2) = 2.30 ft

6. Unit shear beside opening

v1 = (V/L)(L1+T1)/L1 = 355 plf

v2 = (V/L)(T2+L2)/L2 = 355 plf

Check v1*L1+v2*L2=V? 2961 lbf **OK**

7. Resistance to corner forces

R1 = v1*L1 = 1600 lbf

R2 = v2*L2 = 1361 lbf

8. Difference corner force + resistance

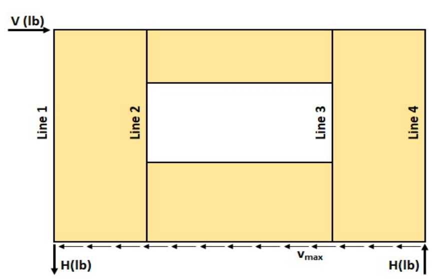
R1-F1 = 400 lbf

R2-F2 = 340 lbf

9. Unit shear in corner zones

vc1 = (R1-F1)/L1 = 89 plf

vc2 = (R2-F2)/L2 = 89 plf



Check Summary of Shear Values for One Opening

Line 1: $vc1(h_a+h_b)+v1(h_o)=H$?		400	1600	1999 lbf
Line 2: $va1(h_a+h_b)-vc1(h_a+h_b)-v1(h_o)=0$?	1999	400	1600	0
Line 3: $va1(h_a+h_b)-vc2(h_a+h_b)-v1(h_o)=0$?	1999	400	1600	0
Line 4: $vc2(h_a+h_b)+v2(h_o)=H$?		400	1600	1999 lbf

Design Summary*

Req. Sheathing Capacity	444 plf	**4-Term Deflection**		**3-Term Deflection**	
Req. Strap Force	1200 lbf	**4-Term Story Drift %**		**3-Term Story Drift %**	
Req. HD Force (H)	1999 lbf				
Req. Shear Wall Anchorage Force (v_{max})	222 plf				

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

Project Information

Code:		**Date:**	6/2/2025
Designer:	Chon Pieruccioni, PE		
Client:			
Project:	East Town Crossing - Building A		
Wall Line:	Grid G - 13'-4" Segment - L1E		

Shear Wall Deflection Calculation Variables

Unfactored Shear Load $V_{unfactored}$: 2961 (lbf)

Sheathing Type: 7/16 OSB
Grade: APA Rated Sheathing

G_t Override:
G_a Override:

Wood End Post Values:
Species: HF#2
E: 1.30E+06 (psi)

Enter individual post sizes below.

C_d: 4.00

Nail Type: 8d common (penny weight)

	Pier 1	Pier 2	
Nail Spacing:	3	3	(in.)
HD Capacity:	2140	2140	(lbf)
HD Deflection:	0.088	0.088	(in.)

Four-Term Equation Deflection Check

$$\Delta = \frac{8vh^3}{EAb} + \frac{vh}{Gt} + 0.75he_n + d_a\frac{h}{b} \qquad \text{(Equation 23-2)}$$

Sheathing Type: 7/16 OSB APA Rated Sheathing
Nail Type: 8d common

	Pier 1-L	Pier 1-R	Pier 2-L	Pier 2-R	
$v_{unfactored}$:	355	355	355	355	(plf)
E:	1.30E+06	1.30E+06	1.30E+06	1.30E+06	(psi)
h:	9.00	5.60	5.60	9.00	(ft)
Qty:					
Stud Size:					
A Override:					(in.^{2})
A:					(in.^{2})
G_t:	83,500	83,500	83,500	83,500	(lbf/in.)
Nail Spacing:	3	3	3	3	(in.)
V_n:	89	89	89	89	(plf)
e_n:	0.0035	0.0035	0.0035	0.0035	(in.)
b:	4.50	4.50	3.83	3.83	(ft)
HD Capacity:	2140	2140	2140	2140	(lbf)
HD Defl:	0.088	0.088	0.088	0.088	(in.)

Check Total Deflection of Wall System

Pier 1 (left)				Pier 1 (right)			
Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 4 HD-1	Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 4 HD-2
	0.038	0.023	0.263		0.024	0.015	0.102
		Sum				Sum	
Pier 2 (left)				**Pier 2 (right)**			
Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 4 HD-1	Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 4 HD-2
	0.024	0.015	0.120		0.038	0.023	0.309
		Sum				Sum	

Total Defl.	
	(in.) %drift

Project Information

Code:		**Date:**	6/2/2025
Designer:	Chon Pieruccioni, PE		
Client:			
Project:	East Town Crossing - Building A		
Wall Line:	Grid G - 13'-4" Segment - L1E		

Shear Wall Deflection Calculation Variables

Unfactored Shear Load $V_{unfactored}$: 2961 (lbf)

Sheathing Type: 7/16 OSB
Grade: APA Rated Sheathing

Wood End Post Values:
Species: HF#2
E: 1.30E+06 (psi)

Nail Type: 8d common (penny weight)

G_t Override:
G_a Override:

C_d: 4.00

	Pier 1	Pier 2	
Nail Spacing:	3	3	(in.)
HD Capacity:	2140	2140	(lbf)
HD Deflection:	0.088	0.088	(in.)

Three-Term Equation Deflection Check

$$\delta_{sw} = \frac{8vh^3}{EAb} + \frac{vh}{1000G_a} + \frac{h\Delta_a}{b} \qquad (4.3\text{-}1)$$

Sheathing Type: 7/16 OSB APA Rated Sheathing
Nail Type: 8d common

	Pier 1-L	Pier 1-R	Pier 2-L	Pier 2-R	
$v_{unfactored}$:	355	355	355	355	(plf)
E:	1.30E+06	1.30E+06	1.30E+06	1.30E+06	(psi)
h:	9.00	5.60	5.60	9.00	(ft)
Qty:					
Stud Size:					
A Override:					(in.^{2})
A:					(in.^{2})
G_a:	28.0	28.0	28.0	28.0	(kips/in.)
b:	4.50	4.50	3.83	3.83	(ft)
HD Capacity:	2140	2140	2140	2140	(lbf)
HD Defl:	0.088	0.088	0.088	0.088	(in.)

Check Total Deflection of Wall System

Pier 1 (left)			Pier 1 (right)		
Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 1 Bending	Term 2 Shear	Term 3 Fastener
	0.114	0.263		0.071	0.102
	Sum			Sum	
Pier 2 (left)			**Pier 2 (right)**		
Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 1 Bending	Term 2 Shear	Term 3 Fastener
	0.071	0.120		0.114	0.309
	Sum			Sum	

Total Defl.	
	(in.)
	%drift

Comment: The 3-term equation is calibrated to be approximately equal to 4-term equation at 1.4*ASD capacity.

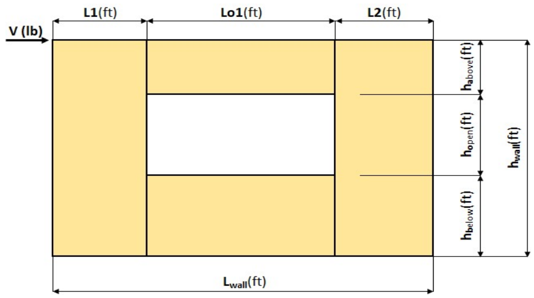
Force Transfer Around Openings Calculator

ONE OPENING

The force transfer around openings (FTAO) method of shear wall analysis is an approach that aims to reinforce the wall such that it performs as if there was no opening. This approach lends certain advantages over segmented shear walls: more versatility, because it allows for narrower wall segments while still meeting the height-to-width ratios and, often, fewer required hold-downs.

Project Information

Code:		**Date:** 6/2/2025
Designer:	Chon Pieruccioni, PE	
Client:		
Project:	East Town Crossing - Building A	
Wall Line:	Grid H - 14'-10" Segment - L3E	



Shear Wall Calculation Variables

V	1744 lbf		Opening 1		
L1	4.92 ft	h_a	1.10 ft		
L2	4.91 ft	h_o	4.50 ft		
h_{wall}	9.00 ft	h_b	3.40 ft		
L_{wall}	14.83 ft	Lo1	5.00 ft		

Adj. Factor Method =	1.25-0.125h/bs
Wall Pier Aspect Ratio	Adj. Factor
$P1=h_o/L1=$ 0.91	N/A
$P2=h_o/L2=$ 0.92	N/A

1. Hold-down forces: $H = Vh_{wall}/L_{wall}$ 1058 lbf

2. Unit shear above + below opening

First opening: $va1 = vb1 = H/(h_a+h_b) =$ 235 plf

3. Total boundary force above + below openings

First opening: O1 = va1 x (Lo1) = 1176 lbf

4. Corner forces

F1 = O1(L1)/(L1+L2) = 589 lbf

F2 = O1(L2)/(L1+L2) = 587 lbf

5. Tributary length of openings

T1 = (L1*Lo1)/(L1+L2) = 2.50 ft

T2 = (L2*Lo1)/(L1+L2) = 2.50 ft

6. Unit shear beside opening

v1 = (V/L)(L1+T1)/L1 = 177 plf

v2 = (V/L)(T2+L2)/L2 = 177 plf

Check v1*L1+v2*L2=V? 1744 lbf **OK**

7. Resistance to corner forces

R1 = v1*L1 = 873 lbf

R2 = v2*L2 = 871 lbf

8. Difference corner force + resistance

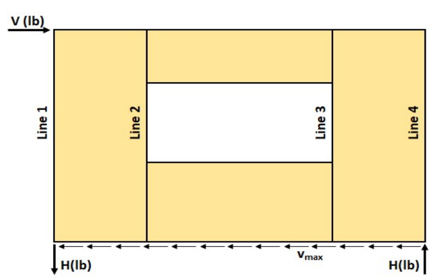
R1-F1 = 284 lbf

R2-F2 = 284 lbf

9. Unit shear in corner zones

vc1 = (R1-F1)/L1 = 58 plf

vc2 = (R2-F2)/L2 = 58 plf



Check Summary of Shear Values for One Opening

Line 1: $vc1(h_a+h_b)+v1(h_o)=H$?		260	798	1058 lbf
Line 2: $va1(h_a+h_b)-vc1(h_a+h_b)-v1(h_o)=0$?	1058	260	798	0
Line 3: $va1(h_a+h_b)-vc2(h_a+h_b)-v1(h_o)=0$?	1058	260	798	0
Line 4: $vc2(h_a+h_b)+v2(h_o)=H$?		260	798	1058 lbf

Design Summary*

Req. Sheathing Capacity	235 plf	**4-Term Deflection**		**3-Term Deflection**	
Req. Strap Force	589 lbf	**4-Term Story Drift %**		**3-Term Story Drift %**	
Req. HD Force (H)	1058 lbf				
Req. Shear Wall Anchorage Force (v_{max})	118 plf				

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

Project Information

Code:		**Date:**	6/2/2025
Designer:	Chon Pieruccioni, PE		
Client:			
Project:	East Town Crossing - Building A		
Wall Line:	Grid H - 14'-10" Segment - L3E		

Shear Wall Deflection Calculation Variables

Unfactored Shear Load $V_{unfactored}$: 1744 (lbf)

Sheathing Type: 7/16 OSB
Grade: APA Rated Sheathing

G_t Override:
G_a Override:

Wood End Post Values:
Species: HF#2
E: 1.30E+06 (psi)

Enter individual post sizes below.

C_d: 4.00

Nail Type: 8d common (penny weight)

	Pier 1	Pier 2	
Nail Spacing:	6	6	(in.)
HD Capacity:	2140	2140	(lbf)
HD Deflection:	0.088	0.088	(in.)

Four-Term Equation Deflection Check

$$\Delta = \frac{8vh^3}{EAb} + \frac{vh}{Gt} + 0.75he_n + d_a\frac{h}{b} \qquad \text{(Equation 23-2)}$$

	Pier 1-L	Pier 1-R	Pier 2-L	Pier 2-R	
$v_{unfactored}$:	177	177	177	177	(plf)
E:	1.30E+06	1.30E+06	1.30E+06	1.30E+06	(psi)
h:	9.00	5.60	5.60	9.00	(ft)
Qty:					
Stud Size:					
A Override:					(in.^{2})
A:					(in.^{2})
G_t:	83,500	83,500	83,500	83,500	(lbf/in.)
Nail Spacing:	6	6	6	6	(in.)
V_n:	89	89	89	89	(plf)
e_n:	0.0035	0.0035	0.0035	0.0035	(in.)
b:	4.92	4.92	4.91	4.91	(ft)
HD Capacity:	2140	2140	2140	2140	(lbf)
HD Defl:	0.088	0.088	0.088	0.088	(in.)

Sheathing Type: 7/16 OSB APA Rated Sheathing
Nail Type: 8d common

Check Total Deflection of Wall System

Pier 1 (left)				Pier 1 (right)			
Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 4 HD-1	Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 4 HD-2
	0.019	0.023	0.120		0.012	0.015	0.047
		Sum				Sum	
Pier 2 (left)				**Pier 2 (right)**			
Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 4 HD-1	Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 4 HD-2
	0.012	0.015	0.047		0.019	0.023	0.120
		Sum				Sum	

Total Defl.	
	(in.) %drift

Project Information

Code:		**Date:**	6/2/2025
Designer:	Chon Pieruccioni, PE		
Client:			
Project:	East Town Crossing - Building A		
Wall Line:	Grid H - 14'-10" Segment - L3E		

Shear Wall Deflection Calculation Variables

Unfactored Shear Load $V_{unfactored}$: 1744 (lbf)

Sheathing Type: 7/16 OSB
Grade: APA Rated Sheathing

Wood End Post Values:
Species: HF#2
E: 1.30E+06 (psi)

Nail Type: 8d common (penny weight)

G_t Override:
G_a Override:

C_d: 4.00

	Pier 1	Pier 2	
Nail Spacing:	6	6	(in.)
HD Capacity:	2140	2140	(lbf)
HD Deflection:	0.088	0.088	(in.)

Three-Term Equation Deflection Check

$$\delta_{sw} = \frac{8vh^3}{EAb} + \frac{vh}{1000G_a} + \frac{h\Delta_a}{b} \qquad (4.3\text{-}1)$$

	Pier 1-L	Pier 1-R	Pier 2-L	Pier 2-R	
$v_{unfactored}$:	177	177	177	177	(plf)
E:	1.30E+06	1.30E+06	1.30E+06	1.30E+06	(psi)
h:	9.00	5.60	5.60	9.00	(ft)
Qty:					
Stud Size:					
A Override:					(in.^{2})
A:					(in.^{2})
G_a:	15.0	15.0	15.0	15.0	(kips/in.)
b:	4.92	4.92	4.91	4.91	(ft)
HD Capacity:	2140	2140	2140	2140	(lbf)
HD Defl:	0.088	0.088	0.088	0.088	(in.)

Sheathing Type: 7/16 OSB APA Rated Sheathing
Nail Type: 8d common

Check Total Deflection of Wall System

Pier 1 (left)			Pier 1 (right)		
Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 1 Bending	Term 2 Shear	Term 3 Fastener
	0.106	0.120		0.066	0.047
	Sum			Sum	
Pier 2 (left)			**Pier 2 (right)**		
Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 1 Bending	Term 2 Shear	Term 3 Fastener
	0.066	0.047		0.106	0.120
	Sum			Sum	

Total Defl.	
	(in.)
	%drift

Comment: The 3-term equation is calibrated to be approximately equal to 4-term equation at 1.4*ASD capacity.

APA

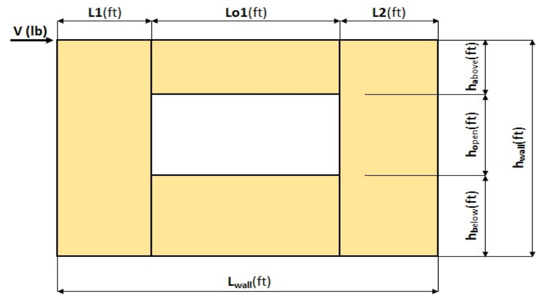
Force Transfer Around Openings Calculator

ONE OPENING

The force transfer around openings (FTAO) method of shear wall analysis is an approach that aims to reinforce the wall such that it performs as if there was no opening. This approach lends certain advantages over segmented shear walls: more versatility, because it allows for narrower wall segments while still meeting the height-to-width ratios and, often, fewer required hold-downs.

Project Information

Code:		**Date:**	6/2/2025
Designer:	Chon Pieruccioni, PE		
Client:			
Project:	East Town Crossing - Building A		
Wall Line:	Grid H - 14'-10" Segment - L2E		



Shear Wall Calculation Variables

V	2830 lbf		Opening 1	
L1	4.92 ft	h_a	1.10 ft	
L2	4.91 ft	h_o	4.50 ft	
h_{wall}	9.00 ft	h_b	3.40 ft	
L_{wall}	14.83 ft	Lo1	5.00 ft	

Adj. Factor Method =		1.25-0.125h/bs
Wall Pier Aspect Ratio		Adj. Factor
$P1=h_o/L1=$	0.91	N/A
$P2=h_o/L2=$	0.92	N/A

1. Hold-down forces: $H = Vh_{wall}/L_{wall}$ 1717 lbf

2. Unit shear above + below opening

First opening: $va1 = vb1 = H/(h_a+h_b) =$	382 plf

3. Total boundary force above + below openings

First opening: O1 = va1 x (Lo1) =	1908 lbf

4. Corner forces

F1 = O1(L1)/(L1+L2) =	955 lbf
F2 = O1(L2)/(L1+L2) =	953 lbf

5. Tributary length of openings

T1 = (L1*Lo1)/(L1+L2) =	2.50 ft
T2 = (L2*Lo1)/(L1+L2) =	2.50 ft

6. Unit shear beside opening

v1 = (V/L)(L1+T1)/L1 =	288 plf	
v2 = (V/L)(T2+L2)/L2 =	288 plf	
Check v1*L1+v2*L2=V?	2830 lbf	**OK**

7. Resistance to corner forces

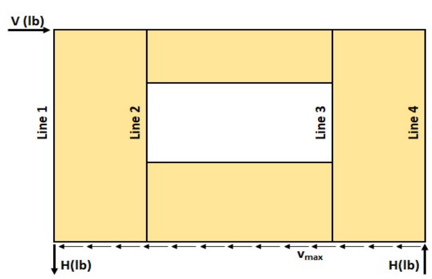
R1 = v1*L1 =	1416 lbf
R2 = v2*L2 =	1414 lbf

8. Difference corner force + resistance

R1-F1 =	461 lbf
R2-F2 =	460 lbf

9. Unit shear in corner zones

vc1 = (R1-F1)/L1 =	94 plf
vc2 = (R2-F2)/L2 =	94 plf



Check Summary of Shear Values for One Opening

Line 1: $vc1(h_a+h_b)+v1(h_o)=H?$		422	1296	1717 lbf
Line 2: $va1(h_a+h_b)-vc1(h_a+h_b)-v1(h_o)=0?$	1717	422	1296	0
Line 3: $va1(h_a+h_b)-vc2(h_a+h_b)-v1(h_o)=0?$	1717	422	1296	0
Line 4: $vc2(h_a+h_b)+v2(h_o)=H?$		422	1296	1717 lbf

Design Summary*

Req. Sheathing Capacity	382 plf	**4-Term Deflection**		**3-Term Deflection**	
Req. Strap Force	955 lbf	**4-Term Story Drift %**		**3-Term Story Drift %**	
Req. HD Force (H)	1717 lbf				
Req. Shear Wall Anchorage Force (v_{max})	191 plf				

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

Project Information

Code:		**Date:**	6/2/2025
Designer:	Chon Pieruccioni, PE		
Client:			
Project:	East Town Crossing - Building A		
Wall Line:	Grid H - 14'-10" Segment - L2E		

Shear Wall Deflection Calculation Variables

Unfactored Shear Load $V_{unfactored}$: 2830 (lbf)

Sheathing Type: 7/16 OSB
Grade: APA Rated Sheathing

G_t Override:
G_a Override:

Wood End Post Values:
Species: HF#2
E: 1.30E+06 (psi)

Enter individual post sizes below.

C_d: 4.00

Nail Type: 8d common (penny weight)

	Pier 1	Pier 2	
Nail Spacing:	3	3	(in.)
HD Capacity:	2140	2140	(lbf)
HD Deflection:	0.088	0.088	(in.)

Four-Term Equation Deflection Check

$$\Delta = \frac{8vh^3}{EAb} + \frac{vh}{Gt} + 0.75he_n + d_a\frac{h}{b} \qquad \text{(Equation 23-2)}$$

Sheathing Type: 7/16 OSB APA Rated Sheathing
Nail Type: 8d common

	Pier 1-L	Pier 1-R	Pier 2-L	Pier 2-R	
$v_{unfactored}$:	288	288	288	288	(plf)
E:	1.30E+06	1.30E+06	1.30E+06	1.30E+06	(psi)
h:	9.00	5.60	5.60	9.00	(ft)
Qty:					
Stud Size:					
A Override:					(in.^{2})
A:					(in.^{2})
G_t:	83,500	83,500	83,500	83,500	(lbf/in.)
Nail Spacing:	3	3	3	3	(in.)
V_n:	72	72	72	72	(plf)
e_n:	0.0018	0.0018	0.0018	0.0018	(in.)
b:	4.92	4.92	4.91	4.91	(ft)
HD Capacity:	2140	2140	2140	2140	(lbf)
HD Defl:	0.088	0.088	0.088	0.088	(in.)

Check Total Deflection of Wall System

Pier 1 (left)				Pier 1 (right)			
Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 4 HD-1	Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 4 HD-2
	0.031	0.012	0.195		0.019	0.008	0.075
		Sum				Sum	
Pier 2 (left)				**Pier 2 (right)**			
Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 4 HD-1	Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 4 HD-2
	0.019	0.008	0.076		0.031	0.012	0.195
		Sum				Sum	

Total Defl.	
	(in.)
	%drift

Project Information

Code:		**Date:**	6/2/2025
Designer:	Chon Pieruccioni, PE		
Client:			
Project:	East Town Crossing - Building A		
Wall Line:	Grid H - 14'-10" Segment - L2E		

Shear Wall Deflection Calculation Variables

Unfactored Shear Load $V_{unfactored}$: 2830 (lbf)

Wood End Post Values:

Sheathing Type:	7/16 OSB	Species:	HF#2	Nail Type:	8d common	(penny weight)	
Grade:	APA Rated Sheathing	E:	1.30E+06 (psi)				
					Pier 1	Pier 2	
G_t Override:				Nail Spacing:	3	3	(in.)
G_a Override:		C_d:	4.00	HD Capacity:	2140	2140	(lbf)
				HD Deflection:	0.088	0.088	(in.)

Three-Term Equation Deflection Check

$$\delta_{sw} = \frac{8vh^3}{EAb} + \frac{vh}{1000G_a} + \frac{h\Delta_a}{b} \qquad (4.3\text{-}1)$$

Sheathing Type: 7/16 OSB APA Rated Sheathing

Nail Type: 8d common

	Pier 1-L	Pier 1-R	Pier 2-L	Pier 2-R	
$v_{unfactored}$:	288	288	288	288	(plf)
E:	1.30E+06	1.30E+06	1.30E+06	1.30E+06	(psi)
h:	9.00	5.60	5.60	9.00	(ft)
Qty:					
Stud Size:					
A Override:					(in.^{2})
A:					(in.^{2})
G_a:	28.0	28.0	28.0	28.0	(kips/in.)
b:	4.92	4.92	4.91	4.91	(ft)
HD Capacity:	2140	2140	2140	2140	(lbf)
HD Defl:	0.088	0.088	0.088	0.088	(in.)

Check Total Deflection of Wall System

Pier 1 (left)			Pier 1 (right)		
Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 1 Bending	Term 2 Shear	Term 3 Fastener
	0.093	0.195		0.058	0.075
	Sum			Sum	
Pier 2 (left)			**Pier 2 (right)**		
Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 1 Bending	Term 2 Shear	Term 3 Fastener
	0.058	0.076		0.093	0.195
	Sum			Sum	

Total Defl.	
	(in.)
	%drift

Comment: The 3-term equation is calibrated to be approximately equal to 4-term equation at 1.4*ASD capacity.

APA

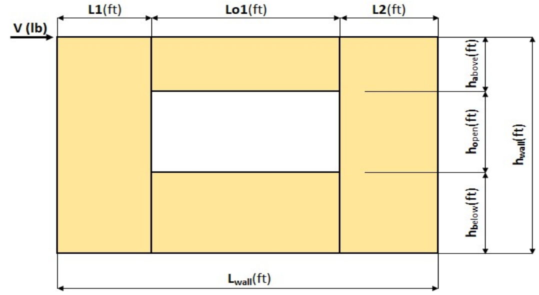
Force Transfer Around Openings Calculator

ONE OPENING

The force transfer around openings (FTAO) method of shear wall analysis is an approach that aims to reinforce the wall such that it performs as if there was no opening. This approach lends certain advantages over segmented shear walls: more versatility, because it allows for narrower wall segments while still meeting the height-to-width ratios and, often, fewer required hold-downs.

Project Information

Code:		**Date:**	6/2/2025
Designer:	Chon Pieruccioni, PE		
Client:			
Project:	East Town Crossing - Building A		
Wall Line:	Grid H - 14'-10" Segment - L1E		



Shear Wall Calculation Variables

V	3294 lbf		Opening 1		
L1	4.92 ft	h_a	1.10 ft		
L2	4.91 ft	h_o	4.50 ft		
h_{wall}	9.00 ft	h_b	3.40 ft		
L_{wall}	14.83 ft	Lo1	5.00 ft		

Adj. Factor Method =	1.25-0.125h/bs	
Wall Pier Aspect Ratio		Adj. Factor
$P1=h_o/L1=$	0.91	N/A
$P2=h_o/L2=$	0.92	N/A

1. Hold-down forces: $H = Vh_{wall}/L_{wall}$ 1999 lbf

2. Unit shear above + below opening

First opening: $va1 = vb1 = H/(h_a+h_b) =$ 444 plf

3. Total boundary force above + below openings

First opening: O1 = va1 x (Lo1) = 2221 lbf

4. Corner forces

F1 = O1(L1)/(L1+L2) = 1112 lbf

F2 = O1(L2)/(L1+L2) = 1109 lbf

5. Tributary length of openings

T1 = (L1*Lo1)/(L1+L2) = 2.50 ft

T2 = (L2*Lo1)/(L1+L2) = 2.50 ft

6. Unit shear beside opening

v1 = (V/L)(L1+T1)/L1 = 335 plf

v2 = (V/L)(T2+L2)/L2 = 335 plf

Check v1*L1+v2*L2=V? 3294 lbf **OK**

7. Resistance to corner forces

R1 = v1*L1 = 1649 lbf

R2 = v2*L2 = 1645 lbf

8. Difference corner force + resistance

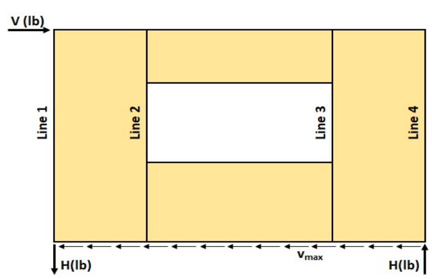
R1-F1 = 537 lbf

R2-F2 = 536 lbf

9. Unit shear in corner zones

vc1 = (R1-F1)/L1 = 109 plf

vc2 = (R2-F2)/L2 = 109 plf



Check Summary of Shear Values for One Opening

Line 1: $vc1(h_a+h_b)+v1(h_o)=H$?		491	1508	1999 lbf
Line 2: $va1(h_a+h_b)-vc1(h_a+h_b)-v1(h_o)=0$?	1999	491	1508	0
Line 3: $va1(h_a+h_b)-vc2(h_a+h_b)-v1(h_o)=0$?	1999	491	1508	0
Line 4: $vc2(h_a+h_b)+v2(h_o)=H$?		491	1508	1999 lbf

Design Summary*

Req. Sheathing Capacity	444 plf	**4-Term Deflection**		**3-Term Deflection**	
Req. Strap Force	1112 lbf	**4-Term Story Drift %**		**3-Term Story Drift %**	
Req. HD Force (H)	1999 lbf				
Req. Shear Wall Anchorage Force (v_{max})	222 plf				

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

Project Information

Code:		**Date:** 6/2/2025
Designer:	Chon Pieruccioni, PE	
Client:		
Project:	East Town Crossing - Building A	
Wall Line:	Grid H - 14'-10" Segment - L1E	

Shear Wall Deflection Calculation Variables

Unfactored Shear Load $V_{unfactored}$: 3294 (lbf)

		Wood End Post Values:		Nail Type:	8d common (penny weight)
Sheathing Type:	7/16 OSB	Species:	HF#2		
Grade:	APA Rated Sheathing	E:	1.30E+06 (psi)		

	Pier 1	Pier 2	
Nail Spacing:	3	3	(in.)
HD Capacity:	2140	2140	(lbf)
HD Deflection:	0.088	0.088	(in.)

Enter individual post sizes below.

G_t Override:

G_a Override:

C_d: 4.00

Four-Term Equation Deflection Check

$$\Delta = \frac{8vh^3}{EAb} + \frac{vh}{Gt} + 0.75he_n + d_a\frac{h}{b} \qquad \text{(Equation 23-2)}$$

Sheathing Type: 7/16 OSB APA Rated Sheathing

Nail Type: 8d common

	Pier 1-L	Pier 1-R	Pier 2-L	Pier 2-R	
$v_{unfactored}$:	335	335	335	335	(plf)
E:	1.30E+06	1.30E+06	1.30E+06	1.30E+06	(psi)
h:	9.00	5.60	5.60	9.00	(ft)
Qty:					
Stud Size:					
A Override:					(in.^{2})
A:					(in.^{2})
G_t:	83,500	83,500	83,500	83,500	(lbf/in.)
Nail Spacing:	3	3	3	3	(in.)
V_n:	84	84	84	84	(plf)
e_n:	0.0029	0.0029	0.0029	0.0029	(in.)
b:	4.92	4.92	4.91	4.91	(ft)
HD Capacity:	2140	2140	2140	2140	(lbf)
HD Defl:	0.088	0.088	0.088	0.088	(in.)

Check Total Deflection of Wall System

Pier 1 (left)				Pier 1 (right)			
Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 4 HD-1	Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 4 HD-2
	0.036	0.020	0.227		0.022	0.012	0.088
		Sum				Sum	
Pier 2 (left)				**Pier 2 (right)**			
Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 4 HD-1	Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 4 HD-2
	0.022	0.012	0.088		0.036	0.020	0.227
		Sum				Sum	

Total Defl.	
	(in.)
	%drift

Project Information

Code:		**Date:**	6/2/2025
Designer:	Chon Pieruccioni, PE		
Client:			
Project:	East Town Crossing - Building A		
Wall Line:	Grid H - 14'-10" Segment - L1E		

Shear Wall Deflection Calculation Variables

Unfactored Shear Load $V_{unfactored}$: 3294 (lbf)

Wood End Post Values:

Sheathing Type: 7/16 OSB
Grade: APA Rated Sheathing

Species: HF#2
E: 1.30E+06 (psi)

Nail Type: 8d common (penny weight)

G_t Override:
G_a Override:

C_d: 4.00

	Pier 1	Pier 2	
Nail Spacing:	3	3	(in.)
HD Capacity:	2140	2140	(lbf)
HD Deflection:	0.088	0.088	(in.)

Three-Term Equation Deflection Check

$$\delta_{sw} = \frac{8vh^3}{EAb} + \frac{vh}{1000G_a} + \frac{h\Delta_a}{b} \qquad (4.3\text{-}1)$$

	Pier 1-L	Pier 1-R	Pier 2-L	Pier 2-R	
$v_{unfactored}$:	335	335	335	335	(plf)
E:	1.30E+06	1.30E+06	1.30E+06	1.30E+06	(psi)
h:	9.00	5.60	5.60	9.00	(ft)
Qty:					
Stud Size:					
A Override:					(in.^{2})
A:					(in.^{2})
G_a:	28.0	28.0	28.0	28.0	(kips/in.)
b:	4.92	4.92	4.91	4.91	(ft)
HD Capacity:	2140	2140	2140	2140	(lbf)
HD Defl:	0.088	0.088	0.088	0.088	(in.)

Sheathing Type: 7/16 OSB APA Rated Sheathing

Nail Type: 8d common

Check Total Deflection of Wall System

Pier 1 (left)			Pier 1 (right)		
Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 1 Bending	Term 2 Shear	Term 3 Fastener
	0.108	0.227		0.067	0.088
	Sum			Sum	
Pier 2 (left)			**Pier 2 (right)**		
Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 1 Bending	Term 2 Shear	Term 3 Fastener
	0.067	0.088		0.108	0.227
	Sum			Sum	

Total Defl.	
	(in.)
	%drift

Comment: The 3-term equation is calibrated to be approximately equal to 4-term equation at 1.4*ASD capacity.

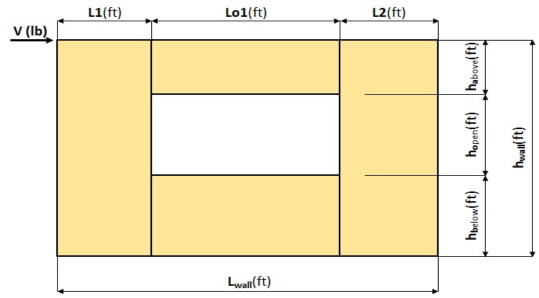
Force Transfer Around Openings Calculator

ONE OPENING

The force transfer around openings (FTAO) method of shear wall analysis is an approach that aims to reinforce the wall such that it performs as if there was no opening. This approach lends certain advantages over segmented shear walls: more versatility, because it allows for narrower wall segments while still meeting the height-to-width ratios and, often, fewer required hold-downs.

Project Information

Code:		**Date:**	6/2/2025
Designer:	Chon Pieruccioni, PE		
Client:			
Project:	East Town Crossing - Building A		
Wall Line:	Grid 1 - 11'-9" Segment - L3E		



Shear Wall Calculation Variables

V	1853 lbf		Opening 1		
L1	2.88 ft	h_a	1.25 ft		
L2	2.87 ft	h_o	6.00 ft		
h_{wall}	9.00 ft	h_b	1.75 ft		
L_{wall}	11.75 ft	Lo1	6.00 ft		

Adj. Factor Method = 1.25-0.125h/bs

Wall Pier Aspect Ratio		Adj. Factor
P1=h_o/L1=	2.08	0.990
P2=h_o/L2=	2.09	0.989

1. Hold-down forces: H = Vh_{wall}/L_{wall} 1419 lbf

2. Unit shear above + below opening

First opening: va1 = vb1 = H/(h_a+h_b) = 473 plf

3. Total boundary force above + below openings

First opening: O1 = va1 x (Lo1) = 2839 lbf

4. Corner forces

F1 = O1(L1)/(L1+L2) = 1422 lbf

F2 = O1(L2)/(L1+L2) = 1417 lbf

5. Tributary length of openings

T1 = (L1*Lo1)/(L1+L2) = 3.01 ft

T2 = (L2*Lo1)/(L1+L2) = 2.99 ft

6. Unit shear beside opening

v1 = (V/L)(L1+T1)/L1 = 322 plf

v2 = (V/L)(T2+L2)/L2 = 322 plf

Check v1*L1+v2*L2=V? 1853 lbf **OK**

7. Resistance to corner forces

R1 = v1*L1 = 928 lbf

R2 = v2*L2 = 925 lbf

8. Difference corner force + resistance

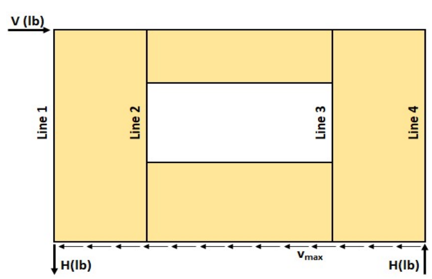
R1-F1 = -494 lbf

R2-F2 = -492 lbf

9. Unit shear in corner zones

vc1 = (R1-F1)/L1 = -171 plf

vc2 = (R2-F2)/L2 = -171 plf



Check Summary of Shear Values for One Opening

Line 1: vc1(h_a+h_b)+v1(h_o)=H?		-514	1934	1419 lbf
Line 2: va1(h_a+h_b)-vc1(h_a+h_b)-v1(h_o)=0?	1419	-514	1934	0
Line 3: va1(h_a+h_b)-vc2(h_a+h_b)-v1(h_o)=0?	1419	-514	1934	0
Line 4: vc2(h_a+h_b)+v2(h_o)=H?		-514	1934	1419 lbf

Design Summary*

Req. Sheathing Capacity	473 plf	**4-Term Deflection**		**3-Term Deflection**	
Req. Strap Force	1422 lbf	**4-Term Story Drift %**		**3-Term Story Drift %**	
Req. HD Force (H)	1419 lbf				
Req. Shear Wall Anchorage Force (v_{max})	158 plf				

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

Project Information

Code:		**Date:**	6/2/2025
Designer:	Chon Pieruccioni, PE		
Client:			
Project:	East Town Crossing - Building A		
Wall Line:	Grid 1 - 11'-9" Segment - L3E		

Shear Wall Deflection Calculation Variables

Unfactored Shear Load $V_{unfactored}$: 1853 (lbf)

Sheathing Type: 7/16 OSB
Grade: APA Rated Sheathing

G_t Override:
G_a Override:

Wood End Post Values:
Species: HF#2
E: 1.30E+06 (psi)

Enter individual post sizes below.

C_d: 4.00

Nail Type: 8d common (penny weight)

	Pier 1	Pier 2	
Nail Spacing:	2	2	(in.)
HD Capacity:	2140	2140	(lbf)
HD Deflection:	0.088	0.088	(in.)

Four-Term Equation Deflection Check

$$\Delta = \frac{8vh^3}{EAb} + \frac{vh}{Gt} + 0.75he_n + d_a\frac{h}{b} \qquad \text{(Equation 23-2)}$$

Sheathing Type: 7/16 OSB APA Rated Sheathing
Nail Type: 8d common

	Pier 1-L	Pier 1-R	Pier 2-L	Pier 2-R	
$v_{unfactored}$:	322	322	322	322	(plf)
E:	1.30E+06	1.30E+06	1.30E+06	1.30E+06	(psi)
h:	9.00	7.25	7.25	9.00	(ft)
Qty:					
Stud Size:					
A Override:					(in.^{2})
A:					(in.^{2})
G_t:	83,500	83,500	83,500	83,500	(lbf/in.)
Nail Spacing:	2	2	2	2	(in.)
V_n:	54	54	54	54	(plf)
e_n:	0.0008	0.0008	0.0008	0.0008	(in.)
b:	2.88	2.88	2.87	2.87	(ft)
HD Capacity:	2140	2140	2140	2140	(lbf)
HD Defl:	0.088	0.088	0.088	0.088	(in.)

Check Total Deflection of Wall System

Pier 1 (left)				Pier 1 (right)			
Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 4 HD-1	Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 4 HD-2
	0.035	0.005	0.373		0.028	0.004	0.242
		Sum				Sum	
Pier 2 (left)				**Pier 2 (right)**			
Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 4 HD-1	Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 4 HD-2
	0.028	0.004	0.243		0.035	0.005	0.374
		Sum				Sum	

Total Defl.	
	(in.) %drift

Project Information

Code:		**Date:**	6/2/2025
Designer:	Chon Pieruccioni, PE		
Client:			
Project:	East Town Crossing - Building A		
Wall Line:	Grid 1 - 11'-9" Segment - L3E		

Shear Wall Deflection Calculation Variables

Unfactored Shear Load $V_{unfactored}$: 1853 (lbf)

Sheathing Type: 7/16 OSB
Grade: APA Rated Sheathing

Wood End Post Values:
Species: HF#2
E: 1.30E+06 (psi)

Nail Type: 8d common (penny weight)

G_t Override:
G_a Override:

C_d: 4.00

	Pier 1	Pier 2	
Nail Spacing:	2	2	(in.)
HD Capacity:	2140	2140	(lbf)
HD Deflection:	0.088	0.088	(in.)

Three-Term Equation Deflection Check

$$\delta_{sw} = \frac{8vh^3}{EAb} + \frac{vh}{1000G_a} + \frac{h\Delta_a}{b} \qquad (4.3\text{-}1)$$

	Pier 1-L	Pier 1-R	Pier 2-L	Pier 2-R	
$v_{unfactored}$:	322	322	322	322	(plf)
E:	1.30E+06	1.30E+06	1.30E+06	1.30E+06	(psi)
h:	9.00	7.25	7.25	9.00	(ft)
Qty:					
Stud Size:					
A Override:					(in.^{2})
A:					(in.^{2})
G_a:	42.0	42.0	42.0	42.0	(kips/in.)
b:	2.88	2.88	2.87	2.87	(ft)
HD Capacity:	2140	2140	2140	2140	(lbf)
HD Defl:	0.088	0.088	0.088	0.088	(in.)

Sheathing Type: 7/16 OSB APA Rated Sheathing
Nail Type: 8d common

Check Total Deflection of Wall System

Pier 1 (left)			Pier 1 (right)		
Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 1 Bending	Term 2 Shear	Term 3 Fastener
	0.069	0.373		0.056	0.242
	Sum			Sum	
Pier 2 (left)			**Pier 2 (right)**		
Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 1 Bending	Term 2 Shear	Term 3 Fastener
	0.056	0.243		0.069	0.374
	Sum			Sum	

Total Defl.	
	(in.)
	%drift

Comment: The 3-term equation is calibrated to be approximately equal to 4-term equation at 1.4*ASD capacity.

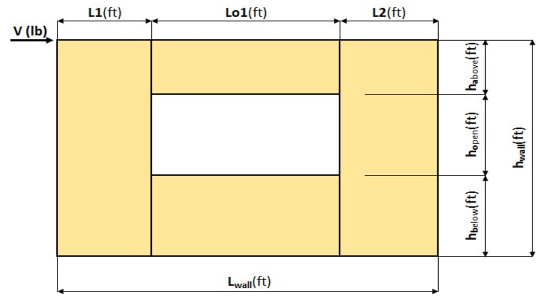
Force Transfer Around Openings Calculator

ONE OPENING

The force transfer around openings (FTAO) method of shear wall analysis is an approach that aims to reinforce the wall such that it performs as if there was no opening. This approach lends certain advantages over segmented shear walls: more versatility, because it allows for narrower wall segments while still meeting the height-to-width ratios and, often, fewer required hold-downs.

Project Information

Code:		**Date:**	6/2/2025
Designer:	Chon Pieruccioni, PE		
Client:			
Project:	East Town Crossing - Building A		
Wall Line:	Grid 1 - 11'-9" Segment - L2E		



Shear Wall Calculation Variables

V	1664 lbf		Opening 1		
L1	2.88 ft	h_a	1.25 ft		
L2	2.87 ft	h_o	6.00 ft		
h_{wall}	9.00 ft	h_b	1.75 ft		
L_{wall}	11.75 ft	Lo1	6.00 ft		

Adj. Factor Method =		1.25-0.125h/bs
Wall Pier Aspect Ratio		Adj. Factor
$P1=h_o/L1=$	2.08	0.990
$P2=h_o/L2=$	2.09	0.989

1. Hold-down forces: $H = Vh_{wall}/L_{wall}$ 1275 lbf

2. Unit shear above + below opening

First opening: $va1 = vb1 = H/(h_a+h_b) =$ 425 plf

3. Total boundary force above + below openings

First opening: O1 = va1 x (Lo1) = 2549 lbf

4. Corner forces

F1 = O1(L1)/(L1+L2) = 1277 lbf

F2 = O1(L2)/(L1+L2) = 1272 lbf

5. Tributary length of openings

T1 = (L1*Lo1)/(L1+L2) = 3.01 ft

T2 = (L2*Lo1)/(L1+L2) = 2.99 ft

6. Unit shear beside opening

v1 = (V/L)(L1+T1)/L1 = 289 plf

v2 = (V/L)(T2+L2)/L2 = 289 plf

Check v1*L1+v2*L2=V? 1664 lbf **OK**

7. Resistance to corner forces

R1 = v1*L1 = 833 lbf

R2 = v2*L2 = 831 lbf

8. Difference corner force + resistance

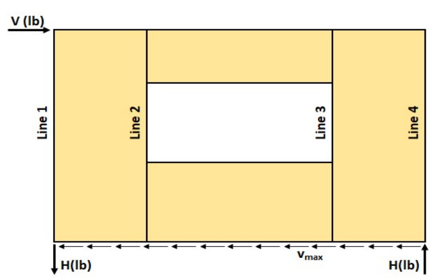
R1-F1 = -443 lbf

R2-F2 = -442 lbf

9. Unit shear in corner zones

vc1 = (R1-F1)/L1 = -154 plf

vc2 = (R2-F2)/L2 = -154 plf



Check Summary of Shear Values for One Opening

Line 1: $vc1(h_a+h_b)+v1(h_o)=H?$		-462	1736	1275 lbf
Line 2: $va1(h_a+h_b)-vc1(h_a+h_b)-v1(h_o)=0?$	1275	-462	1736	0
Line 3: $va1(h_a+h_b)-vc2(h_a+h_b)-v1(h_o)=0?$	1275	-462	1736	0
Line 4: $vc2(h_a+h_b)+v2(h_o)=H?$		-462	1736	1275 lbf

Design Summary*

Req. Sheathing Capacity	425 plf	**4-Term Deflection**		**3-Term Deflection**	
Req. Strap Force	1277 lbf	**4-Term Story Drift %**		**3-Term Story Drift %**	
Req. HD Force (H)	1275 lbf				
Req. Shear Wall Anchorage Force (v_{max})	142 plf				

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

Project Information

Code:		**Date:**	6/2/2025
Designer:	Chon Pieruccioni, PE		
Client:			
Project:	East Town Crossing - Building A		
Wall Line:	Grid 1 - 11'-9" Segment - L2E		

Shear Wall Deflection Calculation Variables

Unfactored Shear Load $V_{unfactored}$: 1664 (lbf)

Sheathing Type: 7/16 OSB
Grade: APA Rated Sheathing

G_t Override:
G_a Override:

Wood End Post Values:
Species: HF#2
E: 1.30E+06 (psi)

Enter individual post sizes below.

C_d: 4.00

Nail Type: 8d common (penny weight)

	Pier 1	Pier 2	
Nail Spacing:	2	2	(in.)
HD Capacity:	2140	2140	(lbf)
HD Deflection:	0.088	0.088	(in.)

Four-Term Equation Deflection Check

$$\Delta = \frac{8vh^3}{EAb} + \frac{vh}{Gt} + 0.75he_n + d_a\frac{h}{b} \qquad \text{(Equation 23-2)}$$

Sheathing Type: 7/16 OSB APA Rated Sheathing
Nail Type: 8d common

	Pier 1-L	Pier 1-R	Pier 2-L	Pier 2-R	
$v_{unfactored}$:	289	289	289	289	(plf)
E:	1.30E+06	1.30E+06	1.30E+06	1.30E+06	(psi)
h:	9.00	7.25	7.25	9.00	(ft)
Qty:					
Stud Size:					
A Override:					(in.^{2})
A:					(in.^{2})
G_t:	83,500	83,500	83,500	83,500	(lbf/in.)
Nail Spacing:	2	2	2	2	(in.)
V_n:	48	48	48	48	(plf)
e_n:	0.0006	0.0006	0.0006	0.0006	(in.)
b:	2.88	2.88	2.87	2.87	(ft)
HD Capacity:	2140	2140	2140	2140	(lbf)
HD Defl:	0.088	0.088	0.088	0.088	(in.)

Check Total Deflection of Wall System

Pier 1 (left)				Pier 1 (right)			
Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 4 HD-1	Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 4 HD-2
	0.031	0.004	0.335		0.025	0.003	0.217
		Sum				Sum	
Pier 2 (left)				**Pier 2 (right)**			
Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 4 HD-1	Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 4 HD-2
	0.025	0.003	0.218		0.031	0.004	0.336
		Sum				Sum	

Total Defl.	
	(in.)
	%drift

Project Information

Code:		**Date:**	6/2/2025
Designer:	Chon Pieruccioni, PE		
Client:			
Project:	East Town Crossing - Building A		
Wall Line:	Grid 1 - 11'-9" Segment - L2E		

Shear Wall Deflection Calculation Variables

Unfactored Shear Load $V_{unfactored}$: 1664 (lbf)

Sheathing Type: 7/16 OSB
Grade: APA Rated Sheathing

Wood End Post Values:
Species: HF#2
E: 1.30E+06 (psi)

Nail Type: 8d common (penny weight)

G_t Override:
G_a Override:

C_d: 4.00

	Pier 1	Pier 2	
Nail Spacing:	2	2	(in.)
HD Capacity:	2140	2140	(lbf)
HD Deflection:	0.088	0.088	(in.)

Three-Term Equation Deflection Check

$$\delta_{sw} = \frac{8vh^3}{EAb} + \frac{vh}{1000G_a} + \frac{h\Delta_a}{b} \qquad (4.3\text{-}1)$$

	Pier 1-L	Pier 1-R	Pier 2-L	Pier 2-R	
$v_{unfactored}$:	289	289	289	289	(plf)
E:	1.30E+06	1.30E+06	1.30E+06	1.30E+06	(psi)
h:	9.00	7.25	7.25	9.00	(ft)
Qty:					
Stud Size:					
A Override:					(in.^{2})
A:					(in.^{2})
G_a:	42.0	42.0	42.0	42.0	(kips/in.)
b:	2.88	2.88	2.87	2.87	(ft)
HD Capacity:	2140	2140	2140	2140	(lbf)
HD Defl:	0.088	0.088	0.088	0.088	(in.)

Sheathing Type: 7/16 OSB APA Rated Sheathing
Nail Type: 8d common

Check Total Deflection of Wall System

Pier 1 (left)			Pier 1 (right)		
Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 1 Bending	Term 2 Shear	Term 3 Fastener
	0.062	0.335		0.050	0.217
	Sum			Sum	
Pier 2 (left)			**Pier 2 (right)**		
Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 1 Bending	Term 2 Shear	Term 3 Fastener
	0.050	0.218		0.062	0.336
	Sum			Sum	

Total Defl.	
	(in.)
	%drift

Comment: The 3-term equation is calibrated to be approximately equal to 4-term equation at 1.4*ASD capacity.

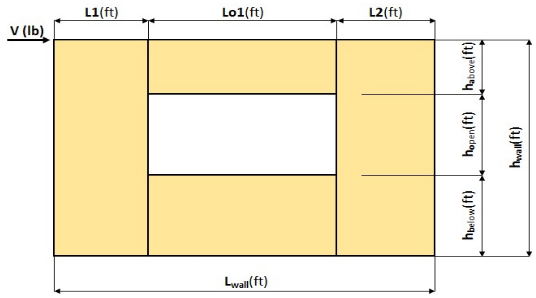
Force Transfer Around Openings Calculator

ONE OPENING

The force transfer around openings (FTAO) method of shear wall analysis is an approach that aims to reinforce the wall such that it performs as if there was no opening. This approach lends certain advantages over segmented shear walls: more versatility, because it allows for narrower wall segments while still meeting the height-to-width ratios and, often, fewer required hold-downs.

Project Information

Code:		**Date:** 6/2/2025
Designer:	Chon Pieruccioni, PE	
Client:		
Project:	East Town Crossing - Building A	
Wall Line:	Grid 1 - 11'-9" Segment - L1E	



Shear Wall Calculation Variables

V	2054 lbf	Opening 1		Adj. Factor Method =	1.25-0.125h/bs	
L1	2.88 ft	h_a	1.25 ft	Wall Pier Aspect Ratio		Adj. Factor
L2	2.87 ft	h_o	6.00 ft	$P1=h_o/L1=$	2.08	0.990
h_{wall}	9.00 ft	h_b	1.75 ft	$P2=h_o/L2=$	2.09	0.989
L_{wall}	11.75 ft	Lo1	6.00 ft			

1. Hold-down forces: $H = Vh_{wall}/L_{wall}$ 1573 lbf

2. Unit shear above + below opening

First opening: $va1 = vb1 = H/(h_a+h_b) =$ 524 plf

3. Total boundary force above + below openings

First opening: O1 = va1 x (Lo1) = 3147 lbf

4. Corner forces

F1 = O1(L1)/(L1+L2) = 1576 lbf

F2 = O1(L2)/(L1+L2) = 1571 lbf

5. Tributary length of openings

T1 = (L1*Lo1)/(L1+L2) = 3.01 ft

T2 = (L2*Lo1)/(L1+L2) = 2.99 ft

6. Unit shear beside opening

v1 = (V/L)(L1+T1)/L1 = 357 plf

v2 = (V/L)(T2+L2)/L2 = 357 plf

Check v1*L1+v2*L2=V? 2054 lbf **OK**

7. Resistance to corner forces

R1 = v1*L1 = 1029 lbf

R2 = v2*L2 = 1025 lbf

8. Difference corner force + resistance

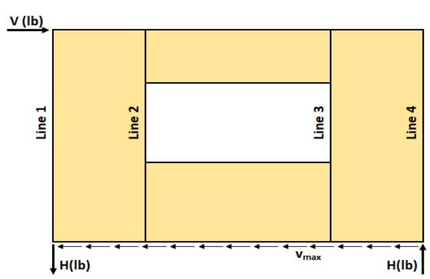
R1-F1 = -547 lbf

R2-F2 = -545 lbf

9. Unit shear in corner zones

vc1 = (R1-F1)/L1 = -190 plf

vc2 = (R2-F2)/L2 = -190 plf



Check Summary of Shear Values for One Opening

Line 1: $vc1(h_a+h_b)+v1(h_o)=H?$		-570	2143	1573 lbf
Line 2: $va1(h_a+h_b)-vc1(h_a+h_b)-v1(h_o)=0?$	1573	-570	2143	0
Line 3: $va1(h_a+h_b)-vc2(h_a+h_b)-v1(h_o)=0?$	1573	-570	2143	0
Line 4: $vc2(h_a+h_b)+v2(h_o)=H?$		-570	2143	1573 lbf

Design Summary*

Req. Sheathing Capacity	524 plf	**4-Term Deflection**		**3-Term Deflection**	
Req. Strap Force	1576 lbf	**4-Term Story Drift %**		**3-Term Story Drift %**	
Req. HD Force (H)	1573 lbf				
Req. Shear Wall Anchorage Force (v_{max})	175 plf				

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

Project Information

Code:		**Date:**	6/2/2025
Designer:	Chon Pieruccioni, PE		
Client:			
Project:	East Town Crossing - Building A		
Wall Line:	Grid 1 - 11'-9" Segment - L1E		

Shear Wall Deflection Calculation Variables

Unfactored Shear Load $V_{unfactored}$: 2054 (lbf)

Sheathing Type: 7/16 OSB
Grade: APA Rated Sheathing

G_t Override:
G_a Override:

Wood End Post Values:
Species: HF#2
E: 1.30E+06 (psi)

Enter individual post sizes below.

C_d: 4.00

Nail Type: 8d common (penny weight)

	Pier 1	Pier 2	
Nail Spacing:	2	2	(in.)
HD Capacity:	2140	2140	(lbf)
HD Deflection:	0.088	0.088	(in.)

Four-Term Equation Deflection Check

$$\Delta = \frac{8vh^3}{EAb} + \frac{vh}{Gt} + 0.75he_n + d_a\frac{h}{b} \qquad \text{(Equation 23-2)}$$

Sheathing Type: 7/16 OSB APA Rated Sheathing
Nail Type: 8d common

	Pier 1-L	Pier 1-R	Pier 2-L	Pier 2-R	
$v_{unfactored}$:	357	357	357	357	(plf)
E:	1.30E+06	1.30E+06	1.30E+06	1.30E+06	(psi)
h:	9.00	7.25	7.25	9.00	(ft)
Qty:					
Stud Size:					
A Override:					(in.^{2})
A:					(in.^{2})
G_t:	83,500	83,500	83,500	83,500	(lbf/in.)
Nail Spacing:	2	2	2	2	(in.)
V_n:	60	60	60	60	(plf)
e_n:	0.0010	0.0010	0.0010	0.0010	(in.)
b:	2.88	2.88	2.87	2.87	(ft)
HD Capacity:	2140	2140	2140	2140	(lbf)
HD Defl:	0.088	0.088	0.088	0.088	(in.)

Check Total Deflection of Wall System

Pier 1 (left)				Pier 1 (right)			
Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 4 HD-1	Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 4 HD-2
	0.039	0.007	0.413		0.031	0.006	0.268
		Sum				Sum	
Pier 2 (left)				**Pier 2 (right)**			
Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 4 HD-1	Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 4 HD-2
	0.031	0.006	0.269		0.039	0.007	0.415
		Sum				Sum	

Total Defl.	
	(in.) %drift

Project Information

Code:		**Date:**	6/2/2025
Designer:	Chon Pieruccioni, PE		
Client:			
Project:	East Town Crossing - Building A		
Wall Line:	Grid 1 - 11'-9" Segment - L1E		

Shear Wall Deflection Calculation Variables

Unfactored Shear Load $V_{unfactored}$: 2054 (lbf)

Wood End Post Values:

Sheathing Type:	7/16 OSB	Species:	HF#2	Nail Type:	8d common (penny weight)
Grade:	APA Rated Sheathing	E:	1.30E+06 (psi)		

				Pier 1	Pier 2	
G_t Override:			Nail Spacing:	2	2	(in.)
G_a Override:		C_d: 4.00	HD Capacity:	2140	2140	(lbf)
			HD Deflection:	0.088	0.088	(in.)

Three-Term Equation Deflection Check

$$\delta_{sw} = \frac{8vh^3}{EAb} + \frac{vh}{1000G_a} + \frac{h\Delta_a}{b} \qquad (4.3\text{-}1)$$

Sheathing Type: 7/16 OSB APA Rated Sheathing

Nail Type: 8d common

	Pier 1-L	Pier 1-R	Pier 2-L	Pier 2-R	
$v_{unfactored}$:	357	357	357	357	(plf)
E:	1.30E+06	1.30E+06	1.30E+06	1.30E+06	(psi)
h:	9.00	7.25	7.25	9.00	(ft)
Qty:					
Stud Size:					
A Override:					(in.^{2})
A:					(in.^{2})
G_a:	42.0	42.0	42.0	42.0	(kips/in.)
b:	2.88	2.88	2.87	2.87	(ft)
HD Capacity:	2140	2140	2140	2140	(lbf)
HD Defl:	0.088	0.088	0.088	0.088	(in.)

Check Total Deflection of Wall System

Pier 1 (left)			Pier 1 (right)		
Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 1 Bending	Term 2 Shear	Term 3 Fastener
	0.077	0.413		0.062	0.268
	Sum			Sum	
Pier 2 (left)			**Pier 2 (right)**		
Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 1 Bending	Term 2 Shear	Term 3 Fastener
	0.062	0.269		0.077	0.415
	Sum			Sum	

Total Defl.	
	(in.)
	%drift

Comment: The 3-term equation is calibrated to be approximately equal to 4-term equation at 1.4*ASD capacity.

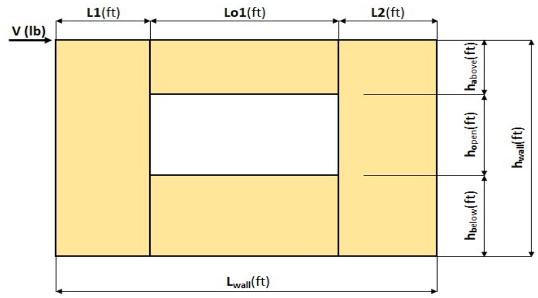
Force Transfer Around Openings Calculator

ONE OPENING

The force transfer around openings (FTAO) method of shear wall analysis is an approach that aims to reinforce the wall such that it performs as if there was no opening. This approach lends certain advantages over segmented shear walls: more versatility, because it allows for narrower wall segments while still meeting the height-to-width ratios and, often, fewer required hold-downs.

Project Information

Code:	**Date:** 6/2/2025
Designer:	Chon Pieruccioni, PE
Client:	
Project:	East Town Crossing - Building A
Wall Line:	Grid 1.2 - 11'-9" Segment - L3E



Shear Wall Calculation Variables

V	1853 lbf		Opening 1	Adj. Factor Method =	1.25-0.125h/bs
L1	3.83 ft	h_a	1.10 ft	Wall Pier Aspect Ratio	Adj. Factor
L2	2.92 ft	h_o	4.50 ft	$P1=h_o/L1=$ 1.17	N/A
h_{wall}	9.00 ft	h_b	3.40 ft	$P2=h_o/L2=$ 1.54	N/A
L_{wall}	11.75 ft	Lo1	5.00 ft		

1. Hold-down forces: $H = Vh_{wall}/L_{wall}$ 1419 lbf

2. Unit shear above + below opening

First opening: $va1 = vb1 = H/(h_a+h_b) =$ 315 plf

3. Total boundary force above + below openings

First opening: O1 = va1 x (Lo1) = 1577 lbf

4. Corner forces

F1 = O1(L1)/(L1+L2) = 895 lbf

F2 = O1(L2)/(L1+L2) = 682 lbf

5. Tributary length of openings

T1 = (L1*Lo1)/(L1+L2) = 2.84 ft

T2 = (L2*Lo1)/(L1+L2) = 2.16 ft

6. Unit shear beside opening

v1 = (V/L)(L1+T1)/L1 = 275 plf

v2 = (V/L)(T2+L2)/L2 = 275 plf

Check v1*L1+v2*L2=V? 1853 lbf **OK**

7. Resistance to corner forces

R1 = v1*L1 = 1051 lbf

R2 = v2*L2 = 802 lbf

8. Difference corner force + resistance

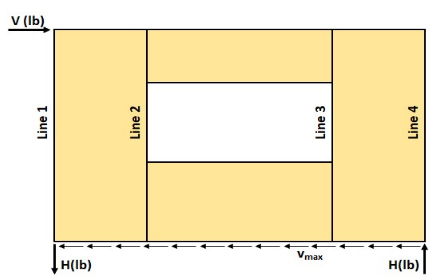
R1-F1 = 157 lbf

R2-F2 = 119 lbf

9. Unit shear in corner zones

vc1 = (R1-F1)/L1 = 41 plf

vc2 = (R2-F2)/L2 = 41 plf



Check Summary of Shear Values for One Opening

Line 1: $vc1(h_a+h_b)+v1(h_o)=H$?		184	1235	1419 lbf
Line 2: $va1(h_a+h_b)-vc1(h_a+h_b)-v1(h_o)=0$?	1419	184	1235	0
Line 3: $va1(h_a+h_b)-vc2(h_a+h_b)-v1(h_o)=0$?	1419	184	1235	0
Line 4: $vc2(h_a+h_b)+v2(h_o)=H$?		184	1235	1419 lbf

Design Summary*

Req. Sheathing Capacity	315 plf	**4-Term Deflection**		**3-Term Deflection**	
Req. Strap Force	895 lbf	**4-Term Story Drift %**		**3-Term Story Drift %**	
Req. HD Force (H)	1419 lbf				
Req. Shear Wall Anchorage Force (v_{max})	158 plf				

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

Project Information

Code:		**Date:**	6/2/2025
Designer:	Chon Pieruccioni, PE		
Client:			
Project:	East Town Crossing - Building A		
Wall Line:	Grid 1.2 - 11'-9" Segment - L3E		

Shear Wall Deflection Calculation Variables

Unfactored Shear Load $V_{unfactored}$: 1853 (lbf)

Sheathing Type: 7/16 OSB
Grade: APA Rated Sheathing

G_t Override:
G_a Override:

Wood End Post Values:
Species: HF#2
E: 1.30E+06 (psi)

Enter individual post sizes below.

C_d: 4.00

Nail Type: 8d common (penny weight)

	Pier 1	Pier 2	
Nail Spacing:	4	4	(in.)
HD Capacity:	2140	2140	(lbf)
HD Deflection:	0.088	0.088	(in.)

Four-Term Equation Deflection Check

$$\Delta = \frac{8vh^3}{EAb} + \frac{vh}{Gt} + 0.75he_n + d_a\frac{h}{b} \quad \text{(Equation 23-2)}$$

Sheathing Type: 7/16 OSB APA Rated Sheathing
Nail Type: 8d common

	Pier 1-L	Pier 1-R	Pier 2-L	Pier 2-R	
$v_{unfactored}$:	275	275	275	275	(plf)
E:	1.30E+06	1.30E+06	1.30E+06	1.30E+06	(psi)
h:	9.00	5.60	5.60	9.00	(ft)
Qty:					
Stud Size:					
A Override:					(in.^{2})
A:					(in.^{2})
G_t:	83,500	83,500	83,500	83,500	(lbf/in.)
Nail Spacing:	4	4	4	4	(in.)
V_n:	92	92	92	92	(plf)
e_n:	0.0038	0.0038	0.0038	0.0038	(in.)
b:	3.83	3.83	2.92	2.92	(ft)
HD Capacity:	2140	2140	2140	2140	(lbf)
HD Defl:	0.088	0.088	0.088	0.088	(in.)

Check Total Deflection of Wall System

Pier 1 (left)				Pier 1 (right)			
Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 4 HD-1	Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 4 HD-2
	0.030	0.026	0.239		0.018	0.016	0.092
		Sum				Sum	
Pier 2 (left)				**Pier 2 (right)**			
Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 4 HD-1	Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 4 HD-2
	0.018	0.016	0.121		0.030	0.026	0.313
		Sum				Sum	

Total Defl.	
	(in.)
	%drift

Project Information

Code:		**Date:**	6/2/2025
Designer:	Chon Pieruccioni, PE		
Client:			
Project:	East Town Crossing - Building A		
Wall Line:	Grid 1.2 - 11'-9" Segment - L3E		

Shear Wall Deflection Calculation Variables

Unfactored Shear Load $V_{unfactored}$: 1853 (lbf)

Wood End Post Values:

Sheathing Type: 7/16 OSB

Grade: APA Rated Sheathing

Species: HF#2

E: 1.30E+06 (psi)

Nail Type: 8d common (penny weight)

G_t Override:

G_a Override:

C_d: 4.00

	Pier 1	Pier 2	
Nail Spacing:	4	4	(in.)
HD Capacity:	2140	2140	(lbf)
HD Deflection:	0.088	0.088	(in.)

Three-Term Equation Deflection Check

$$\delta_{sw} = \frac{8vh^3}{EAb} + \frac{vh}{1000G_a} + \frac{h\Delta_a}{b} \qquad (4.3\text{-}1)$$

	Pier 1-L	Pier 1-R	Pier 2-L	Pier 2-R	
$v_{unfactored}$:	275	275	275	275	(plf)
E:	1.30E+06	1.30E+06	1.30E+06	1.30E+06	(psi)
h:	9.00	5.60	5.60	9.00	(ft)
Qty:					
Stud Size:					
A Override:					(in.^{2})
A:					(in.^{2})
G_a:	22.0	22.0	22.0	22.0	(kips/in.)
b:	3.83	3.83	2.92	2.92	(ft)
HD Capacity:	2140	2140	2140	2140	(lbf)
HD Defl:	0.088	0.088	0.088	0.088	(in.)

Sheathing Type: 7/16 OSB APA Rated Sheathing

Nail Type: 8d common

Check Total Deflection of Wall System

Pier 1 (left)			Pier 1 (right)		
Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 1 Bending	Term 2 Shear	Term 3 Fastener
	0.112	0.239		0.070	0.092
	Sum			Sum	
Pier 2 (left)			**Pier 2 (right)**		
Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 1 Bending	Term 2 Shear	Term 3 Fastener
	0.070	0.121		0.112	0.313
	Sum			Sum	

Total Defl.	
	(in.)
	%drift

Comment: The 3-term equation is calibrated to be approximately equal to 4-term equation at 1.4*ASD capacity.

APA

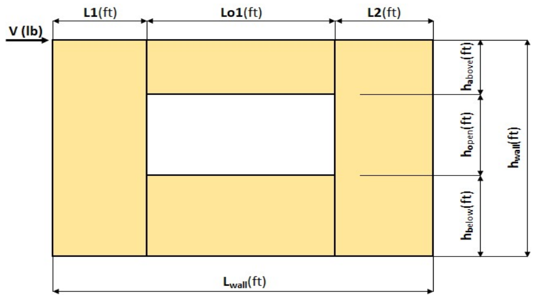
Force Transfer Around Openings Calculator

ONE OPENING

The force transfer around openings (FTAO) method of shear wall analysis is an approach that aims to reinforce the wall such that it performs as if there was no opening. This approach lends certain advantages over segmented shear walls: more versatility, because it allows for narrower wall segments while still meeting the height-to-width ratios and, often, fewer required hold-downs.

Project Information

Code:	**Date:** 6/2/2025
Designer:	Chon Pieruccioni, PE
Client:	
Project:	East Town Crossing - Building A
Wall Line:	Grid 1.2 - 11'-9" Segment - L2E



Shear Wall Calculation Variables

V	1664 lbf		Opening 1	Adj. Factor Method =	1.25-0.125h/bs
L1	3.83 ft	h_a	1.10 ft	Wall Pier Aspect Ratio	Adj. Factor
L2	2.92 ft	h_o	4.50 ft	$P1=h_o/L1=$ 1.17	N/A
h_{wall}	9.00 ft	h_b	3.40 ft	$P2=h_o/L2=$ 1.54	N/A
L_{wall}	11.75 ft	Lo1	5.00 ft		

1. Hold-down forces: $H = Vh_{wall}/L_{wall}$ — 1275 lbf

2. Unit shear above + below opening

First opening: $va1 = vb1 = H/(h_a+h_b) =$ 283 plf

3. Total boundary force above + below openings

First opening: O1 = va1 x (Lo1) = 1416 lbf

4. Corner forces

F1 = O1(L1)/(L1+L2) = 804 lbf

F2 = O1(L2)/(L1+L2) = 613 lbf

5. Tributary length of openings

T1 = (L1*Lo1)/(L1+L2) = 2.84 ft

T2 = (L2*Lo1)/(L1+L2) = 2.16 ft

6. Unit shear beside opening

v1 = (V/L)(L1+T1)/L1 = 247 plf

v2 = (V/L)(T2+L2)/L2 = 247 plf

Check v1*L1+v2*L2=V? 1664 lbf **OK**

7. Resistance to corner forces

R1 = v1*L1 = 944 lbf

R2 = v2*L2 = 720 lbf

8. Difference corner force + resistance

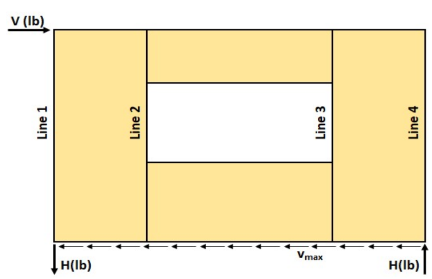
R1-F1 = 141 lbf

R2-F2 = 107 lbf

9. Unit shear in corner zones

vc1 = (R1-F1)/L1 = 37 plf

vc2 = (R2-F2)/L2 = 37 plf



Check Summary of Shear Values for One Opening

Line 1: $vc1(h_a+h_b)+v1(h_o)=H$?		165	1109	1275 lbf
Line 2: $va1(h_a+h_b)-vc1(h_a+h_b)-v1(h_o)=0$?	1275	165	1109	0
Line 3: $va1(h_a+h_b)-vc2(h_a+h_b)-v1(h_o)=0$?	1275	165	1109	0
Line 4: $vc2(h_a+h_b)+v2(h_o)=H$?		165	1109	1275 lbf

Design Summary*

Req. Sheathing Capacity	283 plf	**4-Term Deflection**		**3-Term Deflection**	
Req. Strap Force	804 lbf	**4-Term Story Drift %**		**3-Term Story Drift %**	
Req. HD Force (H)	1275 lbf				
Req. Shear Wall Anchorage Force (v_{max})	142 plf				

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

Project Information

Code:		**Date:**	6/2/2025
Designer:	Chon Pieruccioni, PE		
Client:			
Project:	East Town Crossing - Building A		
Wall Line:	Grid 1.2 - 11'-9" Segment - L2E		

Shear Wall Deflection Calculation Variables

Unfactored Shear Load $V_{unfactored}$: 1664 (lbf)

Sheathing Type: 7/16 OSB
Grade: APA Rated Sheathing

G_t Override:
G_a Override:

Wood End Post Values:
Species: HF#2
E: 1.30E+06 (psi)

Enter individual post sizes below.

C_d: 4.00

Nail Type: 8d common (penny weight)

	Pier 1	Pier 2	
Nail Spacing:	4	4	(in.)
HD Capacity:	2140	2140	(lbf)
HD Deflection:	0.088	0.088	(in.)

Four-Term Equation Deflection Check

$$\Delta = \frac{8vh^3}{EAb} + \frac{vh}{Gt} + 0.75he_n + d_a\frac{h}{b} \qquad \text{(Equation 23-2)}$$

Sheathing Type: 7/16 OSB APA Rated Sheathing
Nail Type: 8d common

	Pier 1-L	Pier 1-R	Pier 2-L	Pier 2-R	
$v_{unfactored}$:	247	247	247	247	(plf)
E:	1.30E+06	1.30E+06	1.30E+06	1.30E+06	(psi)
h:	9.00	5.60	5.60	9.00	(ft)
Qty:					
Stud Size:					
A Override:					(in.^{2})
A:					(in.^{2})
G_t:	83,500	83,500	83,500	83,500	(lbf/in.)
Nail Spacing:	4	4	4	4	(in.)
V_n:	82	82	82	82	(plf)
e_n:	0.0027	0.0027	0.0027	0.0027	(in.)
b:	3.83	3.83	2.92	2.92	(ft)
HD Capacity:	2140	2140	2140	2140	(lbf)
HD Defl:	0.088	0.088	0.088	0.088	(in.)

Check Total Deflection of Wall System

Pier 1 (left)				Pier 1 (right)			
Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 4 HD-1	Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 4 HD-2
	0.027	0.019	0.214		0.017	0.012	0.083
		Sum				Sum	
Pier 2 (left)				**Pier 2 (right)**			
Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 4 HD-1	Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 4 HD-2
	0.017	0.012	0.109		0.027	0.019	0.281
		Sum				Sum	

Total Defl.	
	(in.) %drift

Project Information

Code:		**Date:**	6/2/2025
Designer:	Chon Pieruccioni, PE		
Client:			
Project:	East Town Crossing - Building A		
Wall Line:	Grid 1.2 - 11'-9" Segment - L2E		

Shear Wall Deflection Calculation Variables

Unfactored Shear Load $V_{unfactored}$: 1664 (lbf)

Wood End Post Values:

Sheathing Type:	7/16 OSB	Species:	HF#2	Nail Type:	8d common (penny weight)
Grade:	APA Rated Sheathing	E:	1.30E+06 (psi)		

				Pier 1	Pier 2	
G_t Override:			Nail Spacing:	4	4	(in.)
G_a Override:		C_d: 4.00	HD Capacity:	2140	2140	(lbf)
			HD Deflection:	0.088	0.088	(in.)

Three-Term Equation Deflection Check

$$\delta_{sw} = \frac{8vh^3}{EAb} + \frac{vh}{1000G_a} + \frac{h\Delta_a}{b} \quad (4.3\text{-}1)$$

Sheathing Type: 7/16 OSB APA Rated Sheathing

Nail Type: 8d common

	Pier 1-L	Pier 1-R	Pier 2-L	Pier 2-R	
$v_{unfactored}$:	247	247	247	247	(plf)
E:	1.30E+06	1.30E+06	1.30E+06	1.30E+06	(psi)
h:	9.00	5.60	5.60	9.00	(ft)
Qty:					
Stud Size:					
A Override:					(in.^{2})
A:					(in.^{2})
G_a:	22.0	22.0	22.0	22.0	(kips/in.)
b:	3.83	3.83	2.92	2.92	(ft)
HD Capacity:	2140	2140	2140	2140	(lbf)
HD Defl:	0.088	0.088	0.088	0.088	(in.)

Check Total Deflection of Wall System

Pier 1 (left)			Pier 1 (right)		
Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 1 Bending	Term 2 Shear	Term 3 Fastener
	0.101	0.214		0.063	0.083
	Sum			Sum	
Pier 2 (left)			**Pier 2 (right)**		
Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 1 Bending	Term 2 Shear	Term 3 Fastener
	0.063	0.109		0.101	0.281
	Sum			Sum	

Total Defl.	
	(in.)
	%drift

Comment: The 3-term equation is calibrated to be approximately equal to 4-term equation at 1.4*ASD capacity.

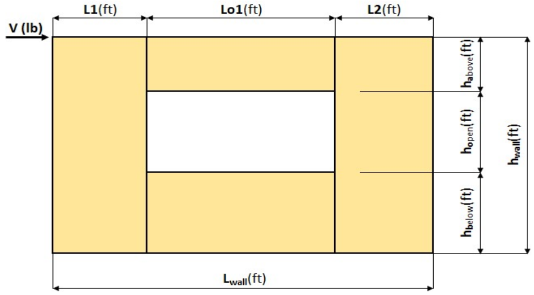
Force Transfer Around Openings Calculator

ONE OPENING

The force transfer around openings (FTAO) method of shear wall analysis is an approach that aims to reinforce the wall such that it performs as if there was no opening. This approach lends certain advantages over segmented shear walls: more versatility, because it allows for narrower wall segments while still meeting the height-to-width ratios and, often, fewer required hold-downs.

Project Information

Code:		**Date:**	6/2/2025
Designer:	Chon Pieruccioni, PE		
Client:			
Project:	East Town Crossing - Building A		
Wall Line:	Grid 1.2 - 11'-9" Segment - L1E		



Shear Wall Calculation Variables

V	2054 lbf	Opening 1		Adj. Factor Method =	1.25-0.125h/bs	
L1	3.83 ft	h_a	1.10 ft	Wall Pier Aspect Ratio		Adj. Factor
L2	2.92 ft	h_o	4.50 ft	$P1=h_o/L1=$	1.17	N/A
h_{wall}	9.00 ft	h_b	3.40 ft	$P2=h_o/L2=$	1.54	N/A
L_{wall}	11.75 ft	Lo1	5.00 ft			

1. Hold-down forces: $H = Vh_{wall}/L_{wall}$ — 1573 lbf

2. Unit shear above + below opening

First opening: $va1 = vb1 = H/(h_a+h_b) =$ 350 plf

3. Total boundary force above + below openings

First opening: O1 = va1 x (Lo1) = 1748 lbf

4. Corner forces

F1 = O1(L1)/(L1+L2) = 992 lbf

F2 = O1(L2)/(L1+L2) = 756 lbf

5. Tributary length of openings

T1 = (L1*Lo1)/(L1+L2) = 2.84 ft

T2 = (L2*Lo1)/(L1+L2) = 2.16 ft

6. Unit shear beside opening

v1 = (V/L)(L1+T1)/L1 = 304 plf

v2 = (V/L)(T2+L2)/L2 = 304 plf

Check v1*L1+v2*L2=V? 2054 lbf **OK**

7. Resistance to corner forces

R1 = v1*L1 = 1165 lbf

R2 = v2*L2 = 889 lbf

8. Difference corner force + resistance

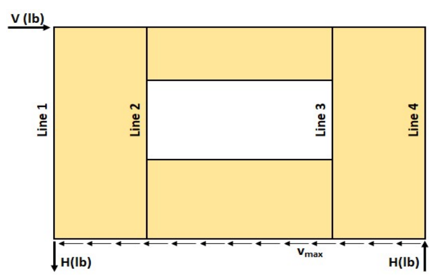
R1-F1 = 174 lbf

R2-F2 = 132 lbf

9. Unit shear in corner zones

vc1 = (R1-F1)/L1 = 45 plf

vc2 = (R2-F2)/L2 = 45 plf



Check Summary of Shear Values for One Opening

Line 1: $vc1(h_a+h_b)+v1(h_o)=H?$		204	1369	1573 lbf
Line 2: $va1(h_a+h_b)-vc1(h_a+h_b)-v1(h_o)=0?$	1573	204	1369	0
Line 3: $va1(h_a+h_b)-vc2(h_a+h_b)-v1(h_o)=0?$	1573	204	1369	0
Line 4: $vc2(h_a+h_b)+v2(h_o)=H?$		204	1369	1573 lbf

Design Summary*

Req. Sheathing Capacity	350 plf	**4-Term Deflection**		**3-Term Deflection**	
Req. Strap Force	992 lbf	**4-Term Story Drift %**		**3-Term Story Drift %**	
Req. HD Force (H)	1573 lbf				
Req. Shear Wall Anchorage Force (v_{max})	175 plf				

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

Project Information

Code:		**Date:**	6/2/2025
Designer:	Chon Pieruccioni, PE		
Client:			
Project:	East Town Crossing - Building A		
Wall Line:	Grid 1.2 - 11'-9" Segment - L1E		

Shear Wall Deflection Calculation Variables

Unfactored Shear Load $V_{unfactored}$: 2054 (lbf)

Sheathing Type: 7/16 OSB
Grade: APA Rated Sheathing

G_t Override:
G_a Override:

Wood End Post Values:
Species: HF#2
E: 1.30E+06 (psi)

Enter individual post sizes below.

C_d: 4.00

Nail Type: 8d common (penny weight)

	Pier 1	Pier 2	
Nail Spacing:	4	4	(in.)
HD Capacity:	2140	2140	(lbf)
HD Deflection:	0.088	0.088	(in.)

Four-Term Equation Deflection Check

$$\Delta = \frac{8vh^3}{EAb} + \frac{vh}{Gt} + 0.75he_n + d_a\frac{h}{b} \qquad \text{(Equation 23-2)}$$

Sheathing Type: 7/16 OSB APA Rated Sheathing
Nail Type: 8d common

	Pier 1-L	Pier 1-R	Pier 2-L	Pier 2-R	
$v_{unfactored}$:	304	304	304	304	(plf)
E:	1.30E+06	1.30E+06	1.30E+06	1.30E+06	(psi)
h:	9.00	5.60	5.60	9.00	(ft)
Qty:					
Stud Size:					
A Override:					(in.^{2})
A:					(in.^{2})
G_t:	83,500	83,500	83,500	83,500	(lbf/in.)
Nail Spacing:	4	4	4	4	(in.)
V_n:	101	101	101	101	(plf)
e_n:	0.0052	0.0052	0.0052	0.0052	(in.)
b:	3.83	3.83	2.92	2.92	(ft)
HD Capacity:	2140	2140	2140	2140	(lbf)
HD Defl:	0.088	0.088	0.088	0.088	(in.)

Check Total Deflection of Wall System

Pier 1 (left)				Pier 1 (right)			
Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 4 HD-1	Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 4 HD-2
	0.033	0.035	0.265		0.020	0.022	0.102
		Sum				Sum	
Pier 2 (left)				**Pier 2 (right)**			
Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 4 HD-1	Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 4 HD-2
	0.020	0.022	0.134		0.033	0.035	0.347
		Sum				Sum	

Total Defl.	
	(in.) %drift

Project Information

Code:		**Date:**	6/2/2025
Designer:	Chon Pieruccioni, PE		
Client:			
Project:	East Town Crossing - Building A		
Wall Line:	Grid 1.2 - 11'-9" Segment - L1E		

Shear Wall Deflection Calculation Variables

Unfactored Shear Load $V_{unfactored}$: 2054 (lbf)

Wood End Post Values:

Sheathing Type:	7/16 OSB	Species:	HF#2	Nail Type:	8d common (penny weight)
Grade:	APA Rated Sheathing	E:	1.30E+06 (psi)		

				Pier 1	Pier 2	
G_t Override:			Nail Spacing:	4	4	(in.)
G_a Override:		C_d: 4.00	HD Capacity:	2140	2140	(lbf)
			HD Deflection:	0.088	0.088	(in.)

Three-Term Equation Deflection Check

$$\delta_{sw} = \frac{8vh^3}{EAb} + \frac{vh}{1000G_a} + \frac{h\Delta_a}{b} \qquad (4.3\text{-}1)$$

Sheathing Type: 7/16 OSB APA Rated Sheathing

Nail Type: 8d common

	Pier 1-L	Pier 1-R	Pier 2-L	Pier 2-R	
$v_{unfactored}$:	304	304	304	304	(plf)
E:	1.30E+06	1.30E+06	1.30E+06	1.30E+06	(psi)
h:	9.00	5.60	5.60	9.00	(ft)
Qty:					
Stud Size:					
A Override:					(in.^{2})
A:					(in.^{2})
G_a:	22.0	22.0	22.0	22.0	(kips/in.)
b:	3.83	3.83	2.92	2.92	(ft)
HD Capacity:	2140	2140	2140	2140	(lbf)
HD Defl:	0.088	0.088	0.088	0.088	(in.)

Check Total Deflection of Wall System

Pier 1 (left)			Pier 1 (right)		
Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 1 Bending	Term 2 Shear	Term 3 Fastener
	0.124	0.265		0.077	0.102
	Sum			Sum	
Pier 2 (left)			**Pier 2 (right)**		
Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 1 Bending	Term 2 Shear	Term 3 Fastener
	0.077	0.134		0.124	0.347
	Sum			Sum	

Total Defl.	
	(in.)
	%drift

Comment: The 3-term equation is calibrated to be approximately equal to 4-term equation at 1.4*ASD capacity.

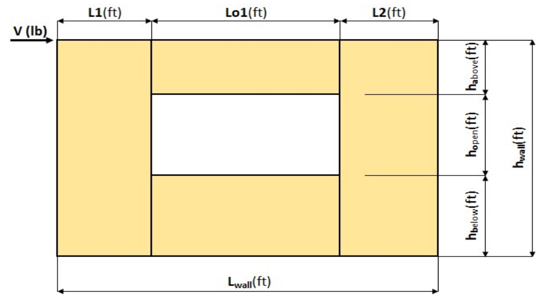
Force Transfer Around Openings Calculator

ONE OPENING

The force transfer around openings (FTAO) method of shear wall analysis is an approach that aims to reinforce the wall such that it performs as if there was no opening. This approach lends certain advantages over segmented shear walls: more versatility, because it allows for narrower wall segments while still meeting the height-to-width ratios and, often, fewer required hold-downs.

Project Information

Code:		**Date:**	6/2/2025
Designer:	Chon Pieruccioni, PE		
Client:			
Project:	East Town Crossing - Building A		
Wall Line:	Grid 2.2 - 10'-4" Segment - L2E		



Shear Wall Calculation Variables

V	1462 lbf	Opening 1		
L1	3.63 ft	h_a	1.50 ft	
L2	4.20 ft	h_o	3.00 ft	
h_{wall}	9.00 ft	h_b	4.50 ft	
L_{wall}	10.33 ft	Lo1	2.50 ft	

Adj. Factor Method = 1.25-0.125h/bs

Wall Pier Aspect Ratio		Adj. Factor
$P1=h_o/L1=$	0.83	N/A
$P2=h_o/L2=$	0.71	N/A

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

2. Unit shear above + below opening

First opening: $va1 = vb1 = H/(h_a+h_b) =$ 212 plf

3. Total boundary force above + below openings

First opening: O1 = va1 x (Lo1) = 531 lbf

4. Corner forces

F1 = O1(L1)/(L1+L2) = 246 lbf

F2 = O1(L2)/(L1+L2) = 285 lbf

5. Tributary length of openings

T1 = (L1*Lo1)/(L1+L2) = 1.16 ft

T2 = (L2*Lo1)/(L1+L2) = 1.34 ft

6. Unit shear beside opening

v1 = (V/L)(L1+T1)/L1 = 187 plf

v2 = (V/L)(T2+L2)/L2 = 187 plf

Check v1*L1+v2*L2=V? 1462 lbf **OK**

7. Resistance to corner forces

R1 = v1*L1 = 678 lbf

R2 = v2*L2 = 784 lbf

8. Difference corner force + resistance

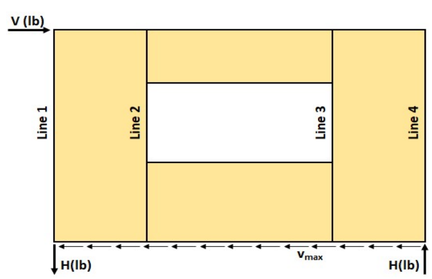
R1-F1 = 432 lbf

R2-F2 = 500 lbf

9. Unit shear in corner zones

vc1 = (R1-F1)/L1 = 119 plf

vc2 = (R2-F2)/L2 = 119 plf



Check Summary of Shear Values for One Opening

Line 1: $vc1(h_a+h_b)+v1(h_o)=H$?		714	560	1274 lbf
Line 2: $va1(h_a+h_b)-vc1(h_a+h_b)-v1(h_o)=0$?	1274	714	560	0
Line 3: $va1(h_a+h_b)-vc2(h_a+h_b)-v1(h_o)=0$?	1274	714	560	0
Line 4: $vc2(h_a+h_b)+v2(h_o)=H$?		714	560	1274 lbf

Design Summary*

Req. Sheathing Capacity	212 plf	**4-Term Deflection**		**3-Term Deflection**	
Req. Strap Force	285 lbf	**4-Term Story Drift %**		**3-Term Story Drift %**	
Req. HD Force (H)	1274 lbf				
Req. Shear Wall Anchorage Force (v_{max})	142 plf				

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

Project Information

Code:		**Date:**	6/2/2025
Designer:	Chon Pieruccioni, PE		
Client:			
Project:	East Town Crossing - Building A		
Wall Line:	Grid 2.2 - 10'-4" Segment - L2E		

Shear Wall Deflection Calculation Variables

Unfactored Shear Load $V_{unfactored}$: 1462 (lbf)

Sheathing Type: 7/16 OSB
Grade: APA Rated Sheathing

G_t Override:
G_a Override:

Wood End Post Values:
Species: HF#2
E: 1.30E+06 (psi)

Enter individual post sizes below.

C_d: 4.00

Nail Type: 8d common (penny weight)

	Pier 1	Pier 2	
Nail Spacing:	6	6	(in.)
HD Capacity:	2140	2140	(lbf)
HD Deflection:	0.088	0.088	(in.)

Four-Term Equation Deflection Check

$$\Delta = \frac{8vh^3}{EAb} + \frac{vh}{Gt} + 0.75he_n + d_a\frac{h}{b} \qquad \text{(Equation 23-2)}$$

Sheathing Type: 7/16 OSB APA Rated Sheathing
Nail Type: 8d common

	Pier 1-L	Pier 1-R	Pier 2-L	Pier 2-R	
$v_{unfactored}$:	187	187	187	187	(plf)
E:	1.30E+06	1.30E+06	1.30E+06	1.30E+06	(psi)
h:	9.00	4.50	4.50	9.00	(ft)
Qty:					
Stud Size:					
A Override:					(in.^{2})
A:					(in.^{2})
G_t:	83,500	83,500	83,500	83,500	(lbf/in.)
Nail Spacing:	6	6	6	6	(in.)
V_n:	93	93	93	93	(plf)
e_n:	0.0040	0.0040	0.0040	0.0040	(in.)
b:	3.63	3.63	4.20	4.20	(ft)
HD Capacity:	2140	2140	2140	2140	(lbf)
HD Defl:	0.088	0.088	0.088	0.088	(in.)

Check Total Deflection of Wall System

Pier 1 (left)				Pier 1 (right)			
Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 4 HD-1	Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 4 HD-2
	0.020	0.027	0.171		0.010	0.014	0.043
		Sum				Sum	
Pier 2 (left)				**Pier 2 (right)**			
Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 4 HD-1	Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 4 HD-2
	0.010	0.014	0.037		0.020	0.027	0.148
		Sum				Sum	

Total Defl.	
	(in.) %drift

Project Information

Code:		**Date:**	6/2/2025
Designer:	Chon Pieruccioni, PE		
Client:			
Project:	East Town Crossing - Building A		
Wall Line:	Grid 2.2 - 10'-4" Segment - L2E		

Shear Wall Deflection Calculation Variables

Unfactored Shear Load $V_{unfactored}$: 1462 (lbf)

Wood End Post Values:

Sheathing Type: 7/16 OSB
Grade: APA Rated Sheathing

Species: HF#2
E: 1.30E+06 (psi)

Nail Type: 8d common (penny weight)

G_t Override:
G_a Override:

C_d: 4.00

	Pier 1	Pier 2	
Nail Spacing:	6	6	(in.)
HD Capacity:	2140	2140	(lbf)
HD Deflection:	0.088	0.088	(in.)

Three-Term Equation Deflection Check

$$\delta_{sw} = \frac{8vh^3}{EAb} + \frac{vh}{1000G_a} + \frac{h\Delta_a}{b} \qquad (4.3\text{-}1)$$

	Pier 1-L	Pier 1-R	Pier 2-L	Pier 2-R	
$v_{unfactored}$:	187	187	187	187	(plf)
E:	1.30E+06	1.30E+06	1.30E+06	1.30E+06	(psi)
h:	9.00	4.50	4.50	9.00	(ft)
Qty:					
Stud Size:					
A Override:					(in.^{2})
A:					(in.^{2})
G_a:	15.0	15.0	15.0	15.0	(kips/in.)
b:	3.63	3.63	4.20	4.20	(ft)
HD Capacity:	2140	2140	2140	2140	(lbf)
HD Defl:	0.088	0.088	0.088	0.088	(in.)

Sheathing Type: 7/16 OSB APA Rated Sheathing
Nail Type: 8d common

Check Total Deflection of Wall System

Pier 1 (left)			Pier 1 (right)		
Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 1 Bending	Term 2 Shear	Term 3 Fastener
	0.112	0.171		0.056	0.043
	Sum			Sum	
Pier 2 (left)			**Pier 2 (right)**		
Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 1 Bending	Term 2 Shear	Term 3 Fastener
	0.056	0.037		0.112	0.148
	Sum			Sum	

Total Defl.	
	(in.)
	%drift

Comment: The 3-term equation is calibrated to be approximately equal to 4-term equation at 1.4*ASD capacity.

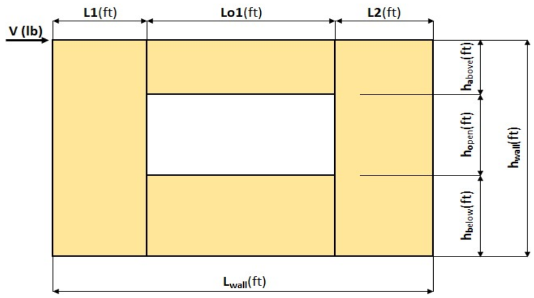
Force Transfer Around Openings Calculator

ONE OPENING

The force transfer around openings (FTAO) method of shear wall analysis is an approach that aims to reinforce the wall such that it performs as if there was no opening. This approach lends certain advantages over segmented shear walls: more versatility, because it allows for narrower wall segments while still meeting the height-to-width ratios and, often, fewer required hold-downs.

Project Information

Code:		**Date:**	6/2/2025
Designer:	Chon Pieruccioni, PE		
Client:			
Project:	East Town Crossing - Building A		
Wall Line:	Grid 2.2 - 10'-4" Segment - L1E		



Shear Wall Calculation Variables

V	1806 lbf		Opening 1		
L1	3.63 ft	h_a	1.50 ft		
L2	4.20 ft	h_o	3.00 ft		
h_{wall}	9.00 ft	h_b	4.50 ft		
L_{wall}	10.33 ft	Lo1	2.50 ft		

Adj. Factor Method =		1.25-0.125h/bs
Wall Pier Aspect Ratio		Adj. Factor
$P1=h_o/L1=$	0.83	N/A
$P2=h_o/L2=$	0.71	N/A

1. Hold-down forces: $H = Vh_{wall}/L_{wall}$ 1573 lbf

2. Unit shear above + below opening

First opening: $va1 = vb1 = H/(h_a+h_b) =$ 262 plf

3. Total boundary force above + below openings

First opening: O1 = va1 x (Lo1) = 656 lbf

4. Corner forces

F1 = O1(L1)/(L1+L2) = 304 lbf

F2 = O1(L2)/(L1+L2) = 352 lbf

5. Tributary length of openings

T1 = (L1*Lo1)/(L1+L2) = 1.16 ft

T2 = (L2*Lo1)/(L1+L2) = 1.34 ft

6. Unit shear beside opening

v1 = (V/L)(L1+T1)/L1 = 231 plf

v2 = (V/L)(T2+L2)/L2 = 231 plf

Check v1*L1+v2*L2=V? 1806 lbf **OK**

7. Resistance to corner forces

R1 = v1*L1 = 837 lbf

R2 = v2*L2 = 969 lbf

8. Difference corner force + resistance

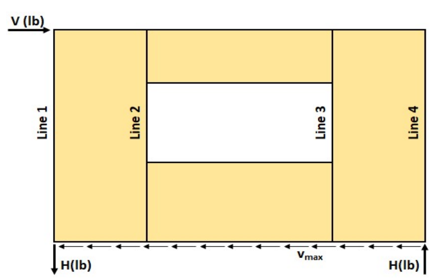
R1-F1 = 533 lbf

R2-F2 = 617 lbf

9. Unit shear in corner zones

vc1 = (R1-F1)/L1 = 147 plf

vc2 = (R2-F2)/L2 = 147 plf



Check Summary of Shear Values for One Opening

Line 1: $vc1(h_a+h_b)+v1(h_o)=H$?		882	692	1573 lbf
Line 2: $va1(h_a+h_b)-vc1(h_a+h_b)-v1(h_o)=0$?	1573	882	692	0
Line 3: $va1(h_a+h_b)-vc2(h_a+h_b)-v1(h_o)=0$?	1573	882	692	0
Line 4: $vc2(h_a+h_b)+v2(h_o)=H$?		882	692	1573 lbf

Design Summary*

Req. Sheathing Capacity	262 plf	**4-Term Deflection**		**3-Term Deflection**	
Req. Strap Force	352 lbf	**4-Term Story Drift %**		**3-Term Story Drift %**	
Req. HD Force (H)	1573 lbf				
Req. Shear Wall Anchorage Force (v_{max})	175 plf				

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

Project Information

Code:		**Date:**	6/2/2025
Designer:	Chon Pieruccioni, PE		
Client:			
Project:	East Town Crossing - Building A		
Wall Line:	Grid 2.2 - 10'-4" Segment - L1E		

Shear Wall Deflection Calculation Variables

Unfactored Shear Load $V_{unfactored}$: 1806 (lbf)

Sheathing Type: 7/16 OSB
Grade: APA Rated Sheathing

G_t Override:
G_a Override:

Wood End Post Values:
Species: HF#2
E: 1.30E+06 (psi)

Enter individual post sizes below.

C_d: 4.00

Nail Type: 8d common (penny weight)

	Pier 1	Pier 2	
Nail Spacing:	4	4	(in.)
HD Capacity:	2140	2140	(lbf)
HD Deflection:	0.088	0.088	(in.)

Four-Term Equation Deflection Check

$$\Delta = \frac{8vh^3}{EAb} + \frac{vh}{Gt} + 0.75he_n + d_a\frac{h}{b} \qquad \text{(Equation 23-2)}$$

Sheathing Type: 7/16 OSB APA Rated Sheathing
Nail Type: 8d common

	Pier 1-L	Pier 1-R	Pier 2-L	Pier 2-R	
$v_{unfactored}$:	231	231	231	231	(plf)
E:	1.30E+06	1.30E+06	1.30E+06	1.30E+06	(psi)
h:	9.00	4.50	4.50	9.00	(ft)
Qty:					
Stud Size:					
A Override:					(in.^{2})
A:					(in.^{2})
G_t:	83,500	83,500	83,500	83,500	(lbf/in.)
Nail Spacing:	4	4	4	4	(in.)
V_n:	77	77	77	77	(plf)
e_n:	0.0022	0.0022	0.0022	0.0022	(in.)
b:	3.63	3.63	4.20	4.20	(ft)
HD Capacity:	2140	2140	2140	2140	(lbf)
HD Defl:	0.088	0.088	0.088	0.088	(in.)

Check Total Deflection of Wall System

Pier 1 (left)				Pier 1 (right)			
Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 4 HD-1	Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 4 HD-2
	0.025	0.015	0.212		0.012	0.008	0.053
		Sum				Sum	
Pier 2 (left)				**Pier 2 (right)**			
Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 4 HD-1	Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 4 HD-2
	0.012	0.008	0.046		0.025	0.015	0.183
		Sum				Sum	

Total Defl.	
	(in.) %drift

Project Information

Code:		**Date:**	6/2/2025
Designer:	Chon Pieruccioni, PE		
Client:			
Project:	East Town Crossing - Building A		
Wall Line:	Grid 2.2 - 10'-4" Segment - L1E		

Shear Wall Deflection Calculation Variables

Unfactored Shear Load $V_{unfactored}$: 1806 (lbf)

Wood End Post Values:

Sheathing Type:	7/16 OSB	Species:	HF#2		Nail Type:	8d common	(penny weight)
Grade:	APA Rated Sheathing	E:	1.30E+06	(psi)			

					Pier 1	Pier 2	
G_t Override:			Nail Spacing:		4	4	(in.)
G_a Override:		C_d: 4.00	HD Capacity:		2140	2140	(lbf)
			HD Deflection:		0.088	0.088	(in.)

Three-Term Equation Deflection Check

$$\delta_{sw} = \frac{8vh^3}{EAb} + \frac{vh}{1000G_a} + \frac{h\Delta_a}{b} \qquad (4.3\text{-}1)$$

Sheathing Type: 7/16 OSB APA Rated Sheathing

Nail Type: 8d common

	Pier 1-L	Pier 1-R	Pier 2-L	Pier 2-R	
$v_{unfactored}$:	231	231	231	231	(plf)
E:	1.30E+06	1.30E+06	1.30E+06	1.30E+06	(psi)
h:	9.00	4.50	4.50	9.00	(ft)
Qty:					
Stud Size:					
A Override:					(in.^{2})
A:					(in.^{2})
G_a:	22.0	22.0	22.0	22.0	(kips/in.)
b:	3.63	3.63	4.20	4.20	(ft)
HD Capacity:	2140	2140	2140	2140	(lbf)
HD Defl:	0.088	0.088	0.088	0.088	(in.)

Check Total Deflection of Wall System

Pier 1 (left)			Pier 1 (right)		
Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 1 Bending	Term 2 Shear	Term 3 Fastener
	0.094	0.212		0.047	0.053
	Sum			Sum	
Pier 2 (left)			**Pier 2 (right)**		
Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 1 Bending	Term 2 Shear	Term 3 Fastener
	0.047	0.046		0.094	0.183
	Sum			Sum	

Total Defl.	
	(in.)
	%drift

Comment: The 3-term equation is calibrated to be approximately equal to 4-term equation at 1.4*ASD capacity.

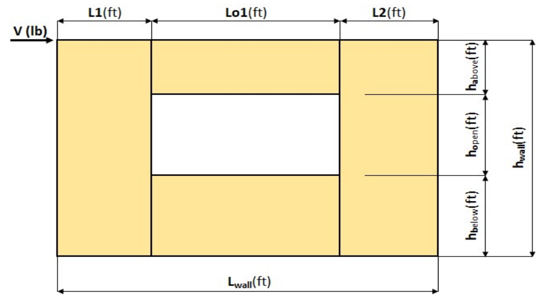
Force Transfer Around Openings Calculator

ONE OPENING

The force transfer around openings (FTAO) method of shear wall analysis is an approach that aims to reinforce the wall such that it performs as if there was no opening. This approach lends certain advantages over segmented shear walls: more versatility, because it allows for narrower wall segments while still meeting the height-to-width ratios and, often, fewer required hold-downs.

Project Information

Code:		**Date:**	6/2/2025
Designer:	Chon Pieruccioni, PE		
Client:			
Project:	East Town Crossing - Building A		
Wall Line:	Grid 7 - 11'-9" Segment - L3E		



Shear Wall Calculation Variables

V	1727 lbf		Opening 1	Adj. Factor Method =	1.25-0.125h/bs
L1	2.88 ft	h_a	1.25 ft	Wall Pier Aspect Ratio	Adj. Factor
L2	2.87 ft	h_o	6.00 ft	$P1=h_o/L1=$ 2.08	0.990
h_{wall}	9.00 ft	h_b	1.75 ft	$P2=h_o/L2=$ 2.09	0.989
L_{wall}	11.75 ft	Lo1	6.00 ft		

1. Hold-down forces: $H = Vh_{wall}/L_{wall}$ 1323 lbf

2. Unit shear above + below opening

First opening: $va1 = vb1 = H/(h_a+h_b)$ =	441 plf

3. Total boundary force above + below openings

First opening: O1 = va1 x (Lo1) =	2646 lbf

4. Corner forces

F1 = O1(L1)/(L1+L2) =	1325 lbf
F2 = O1(L2)/(L1+L2) =	1321 lbf

5. Tributary length of openings

T1 = (L1*Lo1)/(L1+L2) =	3.01 ft
T2 = (L2*Lo1)/(L1+L2) =	2.99 ft

6. Unit shear beside opening

v1 = (V/L)(L1+T1)/L1 =	300 plf	
v2 = (V/L)(T2+L2)/L2 =	300 plf	
Check v1*L1+v2*L2=V?	1727 lbf	**OK**

7. Resistance to corner forces

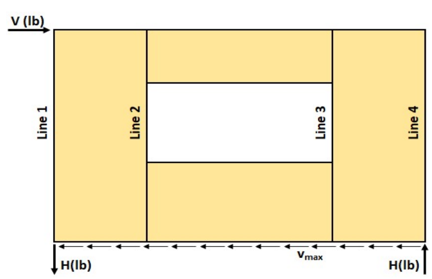
R1 = v1*L1 =	865 lbf
R2 = v2*L2 =	862 lbf

8. Difference corner force + resistance

R1-F1 =	-460 lbf
R2-F2 =	-459 lbf

9. Unit shear in corner zones

vc1 = (R1-F1)/L1 =	-160 plf
vc2 = (R2-F2)/L2 =	-160 plf



Check Summary of Shear Values for One Opening

Line 1: $vc1(h_a+h_b)+v1(h_o)=H$?		-479	1802	1323 lbf
Line 2: $va1(h_a+h_b)-vc1(h_a+h_b)-v1(h_o)=0$?	1323	-479	1802	0
Line 3: $va1(h_a+h_b)-vc2(h_a+h_b)-v1(h_o)=0$?	1323	-479	1802	0
Line 4: $vc2(h_a+h_b)+v2(h_o)=H$?		-479	1802	1323 lbf

Design Summary*

Req. Sheathing Capacity	441 plf	**4-Term Deflection**		**3-Term Deflection**	
Req. Strap Force	1325 lbf	**4-Term Story Drift %**		**3-Term Story Drift %**	
Req. HD Force (H)	1323 lbf				
Req. Shear Wall Anchorage Force (v_{max})	147 plf				

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

Project Information

Code:		**Date:**	6/2/2025
Designer:	Chon Pieruccioni, PE		
Client:			
Project:	East Town Crossing - Building A		
Wall Line:	Grid 7 - 11'-9" Segment - L3E		

Shear Wall Deflection Calculation Variables

Unfactored Shear Load $V_{unfactored}$: 1727 (lbf)

Sheathing Type: 7/16 OSB
Grade: APA Rated Sheathing

G_t Override:
G_a Override:

Wood End Post Values:
Species: HF#2
E: 1.30E+06 (psi)

Enter individual post sizes below.

C_d: 4.00

Nail Type: 8d common (penny weight)

	Pier 1	Pier 2	
Nail Spacing:	2	2	(in.)
HD Capacity:	2140	2140	(lbf)
HD Deflection:	0.088	0.088	(in.)

Four-Term Equation Deflection Check

$$\Delta = \frac{8vh^3}{EAb} + \frac{vh}{Gt} + 0.75he_n + d_a\frac{h}{b} \qquad \text{(Equation 23-2)}$$

Sheathing Type: 7/16 OSB APA Rated Sheathing
Nail Type: 8d common

	Pier 1-L	Pier 1-R	Pier 2-L	Pier 2-R	
$v_{unfactored}$:	300	300	300	300	(plf)
E:	1.30E+06	1.30E+06	1.30E+06	1.30E+06	(psi)
h:	9.00	7.25	7.25	9.00	(ft)
Qty:					
Stud Size:					
A Override:					(in.^{2})
A:					(in.^{2})
G_t:	83,500	83,500	83,500	83,500	(lbf/in.)
Nail Spacing:	2	2	2	2	(in.)
V_n:	50	50	50	50	(plf)
e_n:	0.0006	0.0006	0.0006	0.0006	(in.)
b:	2.88	2.88	2.87	2.87	(ft)
HD Capacity:	2140	2140	2140	2140	(lbf)
HD Defl:	0.088	0.088	0.088	0.088	(in.)

Check Total Deflection of Wall System

Pier 1 (left)				Pier 1 (right)			
Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 4 HD-1	Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 4 HD-2
	0.032	0.004	0.347		0.026	0.003	0.225
		Sum				Sum	
Pier 2 (left)				**Pier 2 (right)**			
Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 4 HD-1	Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 4 HD-2
	0.026	0.003	0.226		0.032	0.004	0.349
		Sum				Sum	

Total Defl.	
	(in.)
	%drift

Project Information

Code:		**Date:**	6/2/2025
Designer:	Chon Pieruccioni, PE		
Client:			
Project:	East Town Crossing - Building A		
Wall Line:	Grid 7 - 11'-9" Segment - L3E		

Shear Wall Deflection Calculation Variables

Unfactored Shear Load $V_{unfactored}$: 1727 (lbf)

Wood End Post Values:

Sheathing Type:	7/16 OSB	Species:	HF#2	Nail Type:	8d common (penny weight)
Grade:	APA Rated Sheathing	E:	1.30E+06 (psi)		

			Pier 1	Pier 2	
G_t Override:		Nail Spacing:	2	2	(in.)
G_a Override:	C_d: 4.00	HD Capacity:	2140	2140	(lbf)
		HD Deflection:	0.088	0.088	(in.)

Three-Term Equation Deflection Check

$$\delta_{sw} = \frac{8vh^3}{EAb} + \frac{vh}{1000G_a} + \frac{h\Delta_a}{b} \qquad (4.3\text{-}1)$$

	Pier 1-L	Pier 1-R	Pier 2-L	Pier 2-R	
$v_{unfactored}$:	300	300	300	300	(plf)
E:	1.30E+06	1.30E+06	1.30E+06	1.30E+06	(psi)
h:	9.00	7.25	7.25	9.00	(ft)
Qty:					
Stud Size:					
A Override:					(in.^{2})
A:					(in.^{2})
G_a:	42.0	42.0	42.0	42.0	(kips/in.)
b:	2.88	2.88	2.87	2.87	(ft)
HD Capacity:	2140	2140	2140	2140	(lbf)
HD Defl:	0.088	0.088	0.088	0.088	(in.)

Sheathing Type: 7/16 OSB APA Rated Sheathing

Nail Type: 8d common

Check Total Deflection of Wall System

Pier 1 (left)			Pier 1 (right)		
Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 1 Bending	Term 2 Shear	Term 3 Fastener
	0.064	0.347		0.052	0.225
	Sum			Sum	
Pier 2 (left)			**Pier 2 (right)**		
Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 1 Bending	Term 2 Shear	Term 3 Fastener
	0.052	0.226		0.064	0.349
	Sum			Sum	

Total Defl.	
	(in.)
	%drift

Comment: The 3-term equation is calibrated to be approximately equal to 4-term equation at 1.4*ASD capacity.

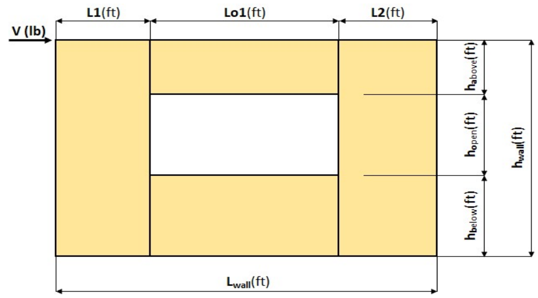
Force Transfer Around Openings Calculator

ONE OPENING

The force transfer around openings (FTAO) method of shear wall analysis is an approach that aims to reinforce the wall such that it performs as if there was no opening. This approach lends certain advantages over segmented shear walls: more versatility, because it allows for narrower wall segments while still meeting the height-to-width ratios and, often, fewer required hold-downs.

Project Information

Code:		**Date:**	6/2/2025
Designer:	Chon Pieruccioni, PE		
Client:			
Project:	East Town Crossing - Building A		
Wall Line:	Grid 7 - 11'-9" Segment - L2E		



Shear Wall Calculation Variables

V	1903 lbf		
L1	2.88 ft	h_a	1.25 ft
L2	2.87 ft	h_o	6.00 ft
h_{wall}	9.00 ft	h_b	1.75 ft
L_{wall}	11.75 ft	Lo1	6.00 ft

Opening 1

Adj. Factor Method = 1.25-0.125h/bs

Wall Pier Aspect Ratio		Adj. Factor
P1=h_o/L1=	2.08	0.990
P2=h_o/L2=	2.09	0.989

1. Hold-down forces: H = Vh_{wall}/L_{wall} 1458 lbf

2. Unit shear above + below opening

First opening: va1 = vb1 = H/(h_a+h_b) = 486 plf

3. Total boundary force above + below openings

First opening: O1 = va1 x (Lo1) = 2915 lbf

4. Corner forces

F1 = O1(L1)/(L1+L2) = 1460 lbf

F2 = O1(L2)/(L1+L2) = 1455 lbf

5. Tributary length of openings

T1 = (L1*Lo1)/(L1+L2) = 3.01 ft

T2 = (L2*Lo1)/(L1+L2) = 2.99 ft

6. Unit shear beside opening

v1 = (V/L)(L1+T1)/L1 = 331 plf

v2 = (V/L)(T2+L2)/L2 = 331 plf

Check v1*L1+v2*L2=V? 1903 lbf **OK**

7. Resistance to corner forces

R1 = v1*L1 = 953 lbf

R2 = v2*L2 = 950 lbf

8. Difference corner force + resistance

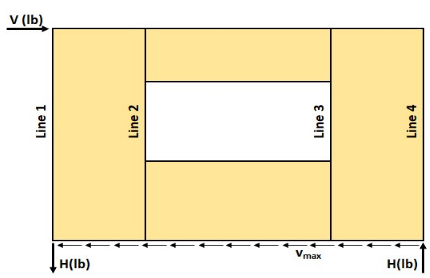
R1-F1 = -507 lbf

R2-F2 = -505 lbf

9. Unit shear in corner zones

vc1 = (R1-F1)/L1 = -176 plf

vc2 = (R2-F2)/L2 = -176 plf



Check Summary of Shear Values for One Opening

Line 1: vc1(h_a+h_b)+v1(h_o)=H?		-528	1986	1458 lbf
Line 2: va1(h_a+h_b)-vc1(h_a+h_b)-v1(h_o)=0?	1458	-528	1986	0
Line 3: va1(h_a+h_b)-vc2(h_a+h_b)-v1(h_o)=0?	1458	-528	1986	0
Line 4: vc2(h_a+h_b)+v2(h_o)=H?		-528	1986	1458 lbf

Design Summary*

Req. Sheathing Capacity	486 plf	**4-Term Deflection**		**3-Term Deflection**	
Req. Strap Force	1460 lbf	**4-Term Story Drift %**		**3-Term Story Drift %**	
Req. HD Force (H)	1458 lbf				
Req. Shear Wall Anchorage Force (v_{max})	162 plf				

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

Project Information

Code:
Date: 6/2/2025
Designer: Chon Pieruccioni, PE
Client:
Project: East Town Crossing - Building A
Wall Line: Grid 7 - 11'-9" Segment - L2E

Shear Wall Deflection Calculation Variables

Unfactored Shear Load $V_{unfactored}$: 1903 (lbf)

Sheathing Type: 7/16 OSB
Grade: APA Rated Sheathing

G_t Override:
G_a Override:

Wood End Post Values:
Species: HF#2
E: 1.30E+06 (psi)

Enter individual post sizes below.

C_d: 4.00

Nail Type: 8d common (penny weight)

	Pier 1	Pier 2	
Nail Spacing:	2	2	(in.)
HD Capacity:	2140	2140	(lbf)
HD Deflection:	0.088	0.088	(in.)

Four-Term Equation Deflection Check

$$\Delta = \frac{8vh^3}{EAb} + \frac{vh}{Gt} + 0.75he_n + d_a\frac{h}{b} \qquad \text{(Equation 23-2)}$$

Sheathing Type: 7/16 OSB APA Rated Sheathing
Nail Type: 8d common

	Pier 1-L	Pier 1-R	Pier 2-L	Pier 2-R	
$v_{unfactored}$:	331	331	331	331	(plf)
E:	1.30E+06	1.30E+06	1.30E+06	1.30E+06	(psi)
h:	9.00	7.25	7.25	9.00	(ft)
Qty:					
Stud Size:					
A Override:					(in.^{2})
A:					(in.^{2})
G_t:	83,500	83,500	83,500	83,500	(lbf/in.)
Nail Spacing:	2	2	2	2	(in.)
V_n:	55	55	55	55	(plf)
e_n:	0.0008	0.0008	0.0008	0.0008	(in.)
b:	2.88	2.88	2.87	2.87	(ft)
HD Capacity:	2140	2140	2140	2140	(lbf)
HD Defl:	0.088	0.088	0.088	0.088	(in.)

Check Total Deflection of Wall System

Pier 1 (left)				Pier 1 (right)			
Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 4 HD-1	Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 4 HD-2
	0.036	0.006	0.383		0.029	0.004	0.248
		Sum				Sum	
Pier 2 (left)				**Pier 2 (right)**			
Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 4 HD-1	Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 4 HD-2
	0.029	0.004	0.249		0.036	0.006	0.384
		Sum				Sum	

Total Defl.	
	(in.) %drift

Project Information

Code:		**Date:**	6/2/2025
Designer:	Chon Pieruccioni, PE		
Client:			
Project:	East Town Crossing - Building A		
Wall Line:	Grid 7 - 11'-9" Segment - L2E		

Shear Wall Deflection Calculation Variables

Unfactored Shear Load $V_{unfactored}$: 1903 (lbf)

Sheathing Type: 7/16 OSB
Grade: APA Rated Sheathing

Wood End Post Values:
Species: HF#2
E: 1.30E+06 (psi)

Nail Type: 8d common (penny weight)

G_t Override:
G_a Override:

C_d: 4.00

	Pier 1	Pier 2	
Nail Spacing:	2	2	(in.)
HD Capacity:	2140	2140	(lbf)
HD Deflection:	0.088	0.088	(in.)

Three-Term Equation Deflection Check

$$\delta_{sw} = \frac{8vh^3}{EAb} + \frac{vh}{1000G_a} + \frac{h\Delta_a}{b} \qquad (4.3\text{-}1)$$

Sheathing Type: 7/16 OSB APA Rated Sheathing
Nail Type: 8d common

	Pier 1-L	Pier 1-R	Pier 2-L	Pier 2-R	
$v_{unfactored}$:	331	331	331	331	(plf)
E:	1.30E+06	1.30E+06	1.30E+06	1.30E+06	(psi)
h:	9.00	7.25	7.25	9.00	(ft)
Qty:					
Stud Size:					
A Override:					(in.^{2})
A:					(in.^{2})
G_a:	42.0	42.0	42.0	42.0	(kips/in.)
b:	2.88	2.88	2.87	2.87	(ft)
HD Capacity:	2140	2140	2140	2140	(lbf)
HD Defl:	0.088	0.088	0.088	0.088	(in.)

Check Total Deflection of Wall System

Pier 1 (left)			Pier 1 (right)		
Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 1 Bending	Term 2 Shear	Term 3 Fastener
	0.071	0.383		0.057	0.248
	Sum			Sum	
Pier 2 (left)			**Pier 2 (right)**		
Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 1 Bending	Term 2 Shear	Term 3 Fastener
	0.057	0.249		0.071	0.384
	Sum			Sum	

Total Defl.	
	(in.)
	%drift

Comment: The 3-term equation is calibrated to be approximately equal to 4-term equation at 1.4*ASD capacity.

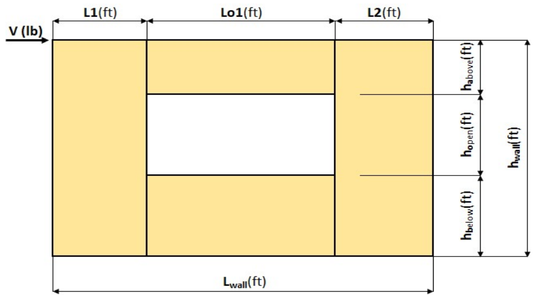
Force Transfer Around Openings Calculator

ONE OPENING

The force transfer around openings (FTAO) method of shear wall analysis is an approach that aims to reinforce the wall such that it performs as if there was no opening. This approach lends certain advantages over segmented shear walls: more versatility, because it allows for narrower wall segments while still meeting the height-to-width ratios and, often, fewer required hold-downs.

Project Information

Code:		**Date:**	6/2/2025
Designer:	Chon Pieruccioni, PE		
Client:			
Project:	East Town Crossing - Building A		
Wall Line:	Grid 7 - 11'-9" Segment - L1E		



Shear Wall Calculation Variables

V	2383 lbf		Opening 1		Adj. Factor Method =	1.25-0.125h/bs	
L1	2.88 ft	h_a	1.25 ft		Wall Pier Aspect Ratio		Adj. Factor
L2	2.87 ft	h_o	6.00 ft		$P1=h_o/L1=$	2.08	0.990
h_{wall}	9.00 ft	h_b	1.75 ft		$P2=h_o/L2=$	2.09	0.989
L_{wall}	11.75 ft	Lo1	6.00 ft				

1. Hold-down forces: $H = Vh_{wall}/L_{wall}$ 1825 lbf

2. Unit shear above + below opening

First opening: $va1 = vb1 = H/(h_a+h_b) =$ 608 plf

3. Total boundary force above + below openings

First opening: O1 = va1 x (Lo1) = 3651 lbf

4. Corner forces

F1 = O1(L1)/(L1+L2) = 1828 lbf

F2 = O1(L2)/(L1+L2) = 1822 lbf

5. Tributary length of openings

T1 = (L1*Lo1)/(L1+L2) = 3.01 ft

T2 = (L2*Lo1)/(L1+L2) = 2.99 ft

6. Unit shear beside opening

v1 = (V/L)(L1+T1)/L1 = 414 plf

v2 = (V/L)(T2+L2)/L2 = 414 plf

Check v1*L1+v2*L2=V? 2383 lbf **OK**

7. Resistance to corner forces

R1 = v1*L1 = 1194 lbf

R2 = v2*L2 = 1189 lbf

8. Difference corner force + resistance

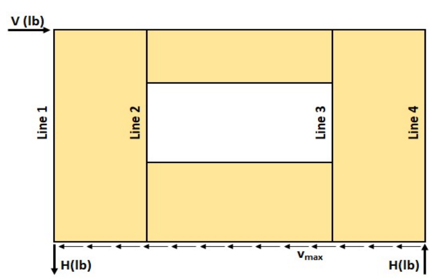
R1-F1 = -635 lbf

R2-F2 = -633 lbf

9. Unit shear in corner zones

vc1 = (R1-F1)/L1 = -220 plf

vc2 = (R2-F2)/L2 = -220 plf



Check Summary of Shear Values for One Opening

Line 1: $vc1(h_a+h_b)+v1(h_o)=H$?		-661	2487	1825 lbf
Line 2: $va1(h_a+h_b)-vc1(h_a+h_b)-v1(h_o)=0$?	1825	-661	2487	0
Line 3: $va1(h_a+h_b)-vc2(h_a+h_b)-v1(h_o)=0$?	1825	-661	2487	0
Line 4: $vc2(h_a+h_b)+v2(h_o)=H$?		-661	2487	1825 lbf

Design Summary*

Req. Sheathing Capacity	608 plf	**4-Term Deflection**		**3-Term Deflection**	
Req. Strap Force	1828 lbf	**4-Term Story Drift %**		**3-Term Story Drift %**	
Req. HD Force (H)	1825 lbf				
Req. Shear Wall Anchorage Force (v_{max})	203 plf				

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

Project Information

Code:		**Date:**	6/2/2025
Designer:	Chon Pieruccioni, PE		
Client:			
Project:	East Town Crossing - Building A		
Wall Line:	Grid 7 - 11'-9" Segment - L1E		

Shear Wall Deflection Calculation Variables

Unfactored Shear Load $V_{unfactored}$: 2383 (lbf)

Sheathing Type: 15/32 OSB
Grade: APA Rated Sheathing

G_t Override:
G_a Override:

Wood End Post Values:
Species: HF#2
E: 1.30E+06 (psi)

Enter individual post sizes below.

C_d: 4.00

Nail Type: 8d common (penny weight)

	Pier 1	Pier 2	
Nail Spacing:	2	2	(in.)
HD Capacity:	2140	2140	(lbf)
HD Deflection:	0.088	0.088	(in.)

Four-Term Equation Deflection Check

$$\Delta = \frac{8vh^3}{EAb} + \frac{vh}{Gt} + 0.75he_n + d_a\frac{h}{b} \qquad \text{(Equation 23-2)}$$

Sheathing Type: 15/32 OSB APA Rated Sheathing
Nail Type: 8d common

	Pier 1-L	Pier 1-R	Pier 2-L	Pier 2-R	
$v_{unfactored}$:	414	414	414	414	(plf)
E:	1.30E+06	1.30E+06	1.30E+06	1.30E+06	(psi)
h:	9.00	7.25	7.25	9.00	(ft)
Qty:					
Stud Size:					
A Override:					(in.^{2})
A:					(in.^{2})
G_t:	83,500	83,500	83,500	83,500	(lbf/in.)
Nail Spacing:	2	2	2	2	(in.)
V_n:	69	69	69	69	(plf)
e_n:	0.0016	0.0016	0.0016	0.0016	(in.)
b:	2.88	2.88	2.87	2.87	(ft)
HD Capacity:	2140	2140	2140	2140	(lbf)
HD Defl:	0.088	0.088	0.088	0.088	(in.)

Check Total Deflection of Wall System

Pier 1 (left)				Pier 1 (right)			
Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 4 HD-1	Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 4 HD-2
	0.045	0.011	0.479		0.036	0.009	0.311
		Sum				Sum	
Pier 2 (left)				**Pier 2 (right)**			
Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 4 HD-1	Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 4 HD-2
	0.036	0.009	0.312		0.045	0.011	0.481
		Sum				Sum	

Total Defl.	
	(in.)
	%drift

Project Information

Code:		**Date:**	6/2/2025
Designer:	Chon Pieruccioni, PE		
Client:			
Project:	East Town Crossing - Building A		
Wall Line:	Grid 7 - 11'-9" Segment - L1E		

Shear Wall Deflection Calculation Variables

Unfactored Shear Load $V_{unfactored}$: 2383 (lbf)

Sheathing Type: 15/32 OSB
Grade: APA Rated Sheathing

Wood End Post Values:
Species: HF#2
E: 1.30E+06 (psi)

Nail Type: 8d common (penny weight)

G_t Override:
G_a Override:

C_d: 4.00

	Pier 1	Pier 2	
Nail Spacing:	2	2	(in.)
HD Capacity:	2140	2140	(lbf)
HD Deflection:	0.088	0.088	(in.)

Three-Term Equation Deflection Check

$$\delta_{sw} = \frac{8vh^3}{EAb} + \frac{vh}{1000G_a} + \frac{h\Delta_a}{b} \qquad (4.3\text{-}1)$$

	Pier 1-L	Pier 1-R	Pier 2-L	Pier 2-R	
$v_{unfactored}$:	414	414	414	414	(plf)
E:	1.30E+06	1.30E+06	1.30E+06	1.30E+06	(psi)
h:	9.00	7.25	7.25	9.00	(ft)
Qty:					
Stud Size:					
A Override:					(in.^{2})
A:					(in.^{2})
G_a:	39.0	39.0	39.0	39.0	(kips/in.)
b:	2.88	2.88	2.87	2.87	(ft)
HD Capacity:	2140	2140	2140	2140	(lbf)
HD Defl:	0.088	0.088	0.088	0.088	(in.)

Sheathing Type: 15/32 OSB APA Rated Sheathing
Nail Type: 8d common

Check Total Deflection of Wall System

Pier 1 (left)			Pier 1 (right)		
Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 1 Bending	Term 2 Shear	Term 3 Fastener
	0.096	0.479		0.077	0.311
	Sum			Sum	
Pier 2 (left)			**Pier 2 (right)**		
Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 1 Bending	Term 2 Shear	Term 3 Fastener
	0.077	0.312		0.096	0.481
	Sum			Sum	

Total Defl.	
	(in.)
	%drift

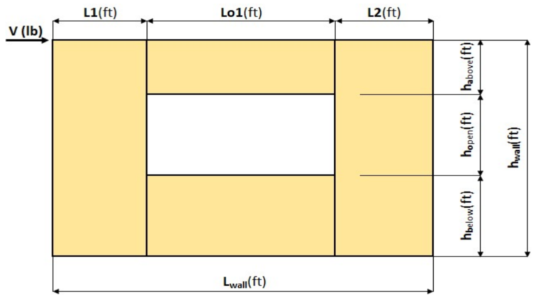
Comment: The 3-term equation is calibrated to be approximately equal to 4-term equation at 1.4*ASD capacity.

Force Transfer Around Openings Calculator
ONE OPENING

The force transfer around openings (FTAO) method of shear wall analysis is an approach that aims to reinforce the wall such that it performs as if there was no opening. This approach lends certain advantages over segmented shear walls: more versatility, because it allows for narrower wall segments while still meeting the height-to-width ratios and, often, fewer required hold-downs.

Project Information

Code:		**Date:** 6/2/2025
Designer:	Chon Pieruccioni, PE	
Client:		
Project:	East Town Crossing - Building A	
Wall Line:	Grid 6.8 - 11'-3" Segment - L3E	



Shear Wall Calculation Variables

V	1653 lbf		Opening 1		
L1	3.33 ft	h_a	1.10 ft		
L2	2.92 ft	h_o	4.50 ft		
h_{wall}	9.00 ft	h_b	3.40 ft		
L_{wall}	11.25 ft	Lo1	5.00 ft		

Adj. Factor Method =		1.25-0.125h/bs
Wall Pier Aspect Ratio		Adj. Factor
$P1=h_o/L1=$	1.35	N/A
$P2=h_o/L2=$	1.54	N/A

1. Hold-down forces: $H = Vh_{wall}/L_{wall}$ 1322 lbf

2. Unit shear above + below opening

First opening: $va1 = vb1 = H/(h_a+h_b) =$ 294 plf

3. Total boundary force above + below openings

First opening: O1 = va1 x (Lo1) = 1469 lbf

4. Corner forces

F1 = O1(L1)/(L1+L2) = 783 lbf
F2 = O1(L2)/(L1+L2) = 686 lbf

5. Tributary length of openings

T1 = (L1*Lo1)/(L1+L2) = 2.66 ft
T2 = (L2*Lo1)/(L1+L2) = 2.34 ft

6. Unit shear beside opening

v1 = (V/L)(L1+T1)/L1 = 264 plf
v2 = (V/L)(T2+L2)/L2 = 264 plf
Check v1*L1+v2*L2=V? 1653 lbf **OK**

7. Resistance to corner forces

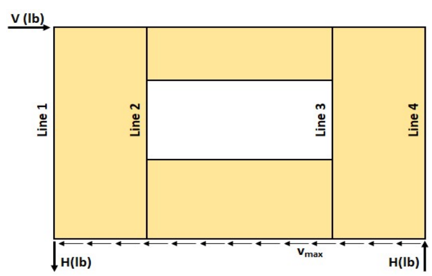
R1 = v1*L1 = 881 lbf
R2 = v2*L2 = 772 lbf

8. Difference corner force + resistance

R1-F1 = 98 lbf
R2-F2 = 86 lbf

9. Unit shear in corner zones

vc1 = (R1-F1)/L1 = 29 plf
vc2 = (R2-F2)/L2 = 29 plf



Check Summary of Shear Values for One Opening

Line 1: $vc1(h_a+h_b)+v1(h_o)=H$?		132	1190	1322 lbf
Line 2: $va1(h_a+h_b)-vc1(h_a+h_b)-v1(h_o)=0$?	1322	132	1190	0
Line 3: $va1(h_a+h_b)-vc2(h_a+h_b)-v1(h_o)=0$?	1322	132	1190	0
Line 4: $vc2(h_a+h_b)+v2(h_o)=H$?		132	1190	1322 lbf

Design Summary*

Req. Sheathing Capacity	294 plf	**4-Term Deflection**		**3-Term Deflection**	
Req. Strap Force	783 lbf	**4-Term Story Drift %**		**3-Term Story Drift %**	
Req. HD Force (H)	1322 lbf				
Req. Shear Wall Anchorage Force (v_{max})	147 plf				

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

Project Information

Code:		**Date:**	6/2/2025
Designer:	Chon Pieruccioni, PE		
Client:			
Project:	East Town Crossing - Building A		
Wall Line:	Grid 6.8 - 11'-3" Segment - L3E		

Shear Wall Deflection Calculation Variables

Unfactored Shear Load $V_{unfactored}$: 1653 (lbf)

Sheathing Type: 7/16 OSB
Grade: APA Rated Sheathing

G_t Override:
G_a Override:

Wood End Post Values:
Species: HF#2
E: 1.30E+06 (psi)

Enter individual post sizes below.

C_d: 4.00

Nail Type: 8d common (penny weight)

	Pier 1	Pier 2	
Nail Spacing:	4	4	(in.)
HD Capacity:	2140	2140	(lbf)
HD Deflection:	0.088	0.088	(in.)

Four-Term Equation Deflection Check

$$\Delta = \frac{8vh^3}{EAb} + \frac{vh}{Gt} + 0.75he_n + d_a\frac{h}{b} \qquad \text{(Equation 23-2)}$$

	Pier 1-L	Pier 1-R	Pier 2-L	Pier 2-R	
$v_{unfactored}$:	264	264	264	264	(plf)
E:	1.30E+06	1.30E+06	1.30E+06	1.30E+06	(psi)
h:	9.00	5.60	5.60	9.00	(ft)
Qty:					
Stud Size:					
A Override:					($in.^2$)
A:					($in.^2$)
G_t:	83,500	83,500	83,500	83,500	(lbf/in.)
Nail Spacing:	4	4	4	4	(in.)
V_n:	88	88	88	88	(plf)
e_n:	0.0034	0.0034	0.0034	0.0034	(in.)
b:	3.33	3.33	2.92	2.92	(ft)
HD Capacity:	2140	2140	2140	2140	(lbf)
HD Defl:	0.088	0.088	0.088	0.088	(in.)

Sheathing Type: 7/16 OSB APA Rated Sheathing
Nail Type: 8d common

Check Total Deflection of Wall System

Pier 1 (left)				Pier 1 (right)			
Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 4 HD-1	Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 4 HD-2
	0.029	0.023	0.265		0.018	0.014	0.102
		Sum				Sum	
Pier 2 (left)				**Pier 2 (right)**			
Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 4 HD-1	Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 4 HD-2
	0.018	0.014	0.117		0.029	0.023	0.302
		Sum				Sum	

Total Defl.	
	(in.)
	%drift

Project Information

Code:		**Date:**	6/2/2025
Designer:	Chon Pieruccioni, PE		
Client:			
Project:	East Town Crossing - Building A		
Wall Line:	Grid 6.8 - 11'-3" Segment - L3E		

Shear Wall Deflection Calculation Variables

Unfactored Shear Load $V_{unfactored}$: 1653 (lbf)

Wood End Post Values:

Sheathing Type: 7/16 OSB
Grade: APA Rated Sheathing

Species: HF#2
E: 1.30E+06 (psi)

Nail Type: 8d common (penny weight)

G_t Override:
G_a Override:

C_d: 4.00

	Pier 1	Pier 2	
Nail Spacing:	4	4	(in.)
HD Capacity:	2140	2140	(lbf)
HD Deflection:	0.088	0.088	(in.)

Three-Term Equation Deflection Check

$$\delta_{sw} = \frac{8vh^3}{EAb} + \frac{vh}{1000G_a} + \frac{h\Delta_a}{b} \quad (4.3\text{-}1)$$

	Pier 1-L	Pier 1-R	Pier 2-L	Pier 2-R	
$v_{unfactored}$:	264	264	264	264	(plf)
E:	1.30E+06	1.30E+06	1.30E+06	1.30E+06	(psi)
h:	9.00	5.60	5.60	9.00	(ft)
Qty:					
Stud Size:					
A Override:					(in.^{2})
A:					(in.^{2})
G_a:	22.0	22.0	22.0	22.0	(kips/in.)
b:	3.33	3.33	2.92	2.92	(ft)
HD Capacity:	2140	2140	2140	2140	(lbf)
HD Defl:	0.088	0.088	0.088	0.088	(in.)

Sheathing Type: 7/16 OSB APA Rated Sheathing
Nail Type: 8d common

Check Total Deflection of Wall System

Pier 1 (left)			Pier 1 (right)		
Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 1 Bending	Term 2 Shear	Term 3 Fastener
	0.108	0.265		0.067	0.102
	Sum			Sum	
Pier 2 (left)			**Pier 2 (right)**		
Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 1 Bending	Term 2 Shear	Term 3 Fastener
	0.067	0.117		0.108	0.302
	Sum			Sum	

Total Defl.	
	(in.)
	%drift

Comment: The 3-term equation is calibrated to be approximately equal to 4-term equation at 1.4*ASD capacity.

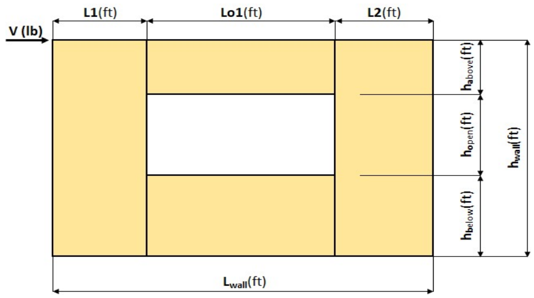
Force Transfer Around Openings Calculator

ONE OPENING

The force transfer around openings (FTAO) method of shear wall analysis is an approach that aims to reinforce the wall such that it performs as if there was no opening. This approach lends certain advantages over segmented shear walls: more versatility, because it allows for narrower wall segments while still meeting the height-to-width ratios and, often, fewer required hold-downs.

Project Information

Code:		**Date:** 6/2/2025
Designer:	Chon Pieruccioni, PE	
Client:		
Project:	East Town Crossing - Building A	
Wall Line:	Grid 6.8 - 11'-3" Segment - L2E	



Shear Wall Calculation Variables

V	1784 lbf		Opening 1	Adj. Factor Method =	1.25-0.125h/bs	
L1	3.33 ft	h_a	1.10 ft	Wall Pier Aspect Ratio		Adj. Factor
L2	2.92 ft	h_o	4.50 ft	$P1=h_o/L1=$	1.35	N/A
h_{wall}	9.00 ft	h_b	3.40 ft	$P2=h_o/L2=$	1.54	N/A
L_{wall}	11.25 ft	Lo1	5.00 ft			

1. Hold-down forces: $H = Vh_{wall}/L_{wall}$ 1427 lbf

2. Unit shear above + below opening

First opening: $va1 = vb1 = H/(h_a+h_b) =$ 317 plf

3. Total boundary force above + below openings

First opening: O1 = va1 x (Lo1) = 1586 lbf

4. Corner forces

F1 = O1(L1)/(L1+L2) = 845 lbf

F2 = O1(L2)/(L1+L2) = 741 lbf

5. Tributary length of openings

T1 = (L1*Lo1)/(L1+L2) = 2.66 ft

T2 = (L2*Lo1)/(L1+L2) = 2.34 ft

6. Unit shear beside opening

v1 = (V/L)(L1+T1)/L1 = 285 plf

v2 = (V/L)(T2+L2)/L2 = 285 plf

Check v1*L1+v2*L2=V? 1784 lbf **OK**

7. Resistance to corner forces

R1 = v1*L1 = 951 lbf

R2 = v2*L2 = 833 lbf

8. Difference corner force + resistance

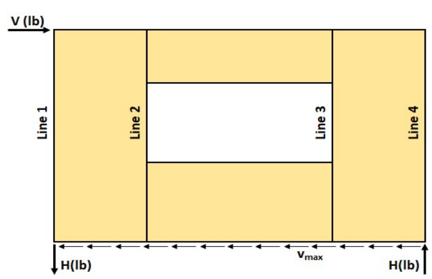
R1-F1 = 106 lbf

R2-F2 = 93 lbf

9. Unit shear in corner zones

vc1 = (R1-F1)/L1 = 32 plf

vc2 = (R2-F2)/L2 = 32 plf



Check Summary of Shear Values for One Opening

Line 1: $vc1(h_a+h_b)+v1(h_o)=H?$		143	1284	1427 lbf
Line 2: $va1(h_a+h_b)-vc1(h_a+h_b)-v1(h_o)=0?$	1427	143	1284	0
Line 3: $va1(h_a+h_b)-vc2(h_a+h_b)-v1(h_o)=0?$	1427	143	1284	0
Line 4: $vc2(h_a+h_b)+v2(h_o)=H?$		143	1284	1427 lbf

Design Summary*

Req. Sheathing Capacity	317 plf	**4-Term Deflection**		**3-Term Deflection**	
Req. Strap Force	845 lbf	**4-Term Story Drift %**		**3-Term Story Drift %**	
Req. HD Force (H)	1427 lbf				
Req. Shear Wall Anchorage Force (v_{max})	159 plf				

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

Project Information

Code:		**Date:**	6/2/2025
Designer:	Chon Pieruccioni, PE		
Client:			
Project:	East Town Crossing - Building A		
Wall Line:	Grid 6.8 - 11'-3" Segment - L2E		

Shear Wall Deflection Calculation Variables

Unfactored Shear Load $V_{unfactored}$: 1784 (lbf)

Sheathing Type: 7/16 OSB
Grade: APA Rated Sheathing

G_t Override:
G_a Override:

Wood End Post Values:
Species: HF#2
E: 1.30E+06 (psi)

Enter individual post sizes below.

C_d: 4.00

Nail Type: 8d common (penny weight)

	Pier 1	Pier 2	
Nail Spacing:	4	4	(in.)
HD Capacity:	2140	2140	(lbf)
HD Deflection:	0.088	0.088	(in.)

Four-Term Equation Deflection Check

$$\Delta = \frac{8vh^3}{EAb} + \frac{vh}{Gt} + 0.75he_n + d_a\frac{h}{b} \qquad \text{(Equation 23-2)}$$

Sheathing Type: 7/16 OSB APA Rated Sheathing
Nail Type: 8d common

	Pier 1-L	Pier 1-R	Pier 2-L	Pier 2-R	
$v_{unfactored}$:	285	285	285	285	(plf)
E:	1.30E+06	1.30E+06	1.30E+06	1.30E+06	(psi)
h:	9.00	5.60	5.60	9.00	(ft)
Qty:					
Stud Size:					
A Override:					(in.^{2})
A:					(in.^{2})
G_t:	83,500	83,500	83,500	83,500	(lbf/in.)
Nail Spacing:	4	4	4	4	(in.)
V_n:	95	95	95	95	(plf)
e_n:	0.0043	0.0043	0.0043	0.0043	(in.)
b:	3.33	3.33	2.92	2.92	(ft)
HD Capacity:	2140	2140	2140	2140	(lbf)
HD Defl:	0.088	0.088	0.088	0.088	(in.)

Check Total Deflection of Wall System

Pier 1 (left)				Pier 1 (right)			
Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 4 HD-1	Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 4 HD-2
	0.031	0.029	0.286		0.019	0.018	0.111
		Sum				Sum	
Pier 2 (left)				**Pier 2 (right)**			
Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 4 HD-1	Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 4 HD-2
	0.019	0.018	0.126		0.031	0.029	0.326
		Sum				Sum	

Total Defl.	
	(in.)
	%drift

Project Information

Code:		**Date:**	6/2/2025
Designer:	Chon Pieruccioni, PE		
Client:			
Project:	East Town Crossing - Building A		
Wall Line:	Grid 6.8 - 11'-3" Segment - L2E		

Shear Wall Deflection Calculation Variables

Unfactored Shear Load $V_{unfactored}$: 1784 (lbf)

Sheathing Type: 7/16 OSB
Grade: APA Rated Sheathing

Wood End Post Values:
Species: HF#2
E: 1.30E+06 (psi)

Nail Type: 8d common (penny weight)

G_t Override:
G_a Override:

C_d: 4.00

	Pier 1	Pier 2	
Nail Spacing:	4	4	(in.)
HD Capacity:	2140	2140	(lbf)
HD Deflection:	0.088	0.088	(in.)

Three-Term Equation Deflection Check

$$\delta_{sw} = \frac{8vh^3}{EAb} + \frac{vh}{1000G_a} + \frac{h\Delta_a}{b} \qquad (4.3\text{-}1)$$

Sheathing Type: 7/16 OSB APA Rated Sheathing
Nail Type: 8d common

	Pier 1-L	Pier 1-R	Pier 2-L	Pier 2-R	
$v_{unfactored}$:	285	285	285	285	(plf)
E:	1.30E+06	1.30E+06	1.30E+06	1.30E+06	(psi)
h:	9.00	5.60	5.60	9.00	(ft)
Qty:					
Stud Size:					
A Override:					($in.^2$)
A:					($in.^2$)
G_a:	22.0	22.0	22.0	22.0	(kips/in.)
b:	3.33	3.33	2.92	2.92	(ft)
HD Capacity:	2140	2140	2140	2140	(lbf)
HD Defl:	0.088	0.088	0.088	0.088	(in.)

Check Total Deflection of Wall System

Pier 1 (left)			Pier 1 (right)		
Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 1 Bending	Term 2 Shear	Term 3 Fastener
	0.117	0.286		0.073	0.111
	Sum			Sum	
Pier 2 (left)			**Pier 2 (right)**		
Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 1 Bending	Term 2 Shear	Term 3 Fastener
	0.073	0.126		0.117	0.326
	Sum			Sum	

Total Defl.	
	(in.)
	%drift

Comment: The 3-term equation is calibrated to be approximately equal to 4-term equation at 1.4*ASD capacity.

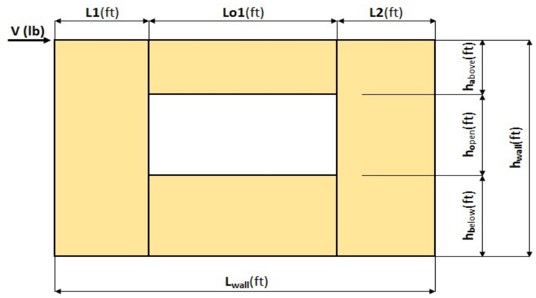
Force Transfer Around Openings Calculator

ONE OPENING

The force transfer around openings (FTAO) method of shear wall analysis is an approach that aims to reinforce the wall such that it performs as if there was no opening. This approach lends certain advantages over segmented shear walls: more versatility, because it allows for narrower wall segments while still meeting the height-to-width ratios and, often, fewer required hold-downs.

Project Information

Code:		**Date:** 6/2/2025
Designer:	Chon Pieruccioni, PE	
Client:		
Project:	East Town Crossing - Building A	
Wall Line:	Grid 6.8 - 11'-3" Segment - L1E	



Shear Wall Calculation Variables

V	2282 lbf		Opening 1	Adj. Factor Method =	1.25-0.125h/bs		
L1	3.33 ft	h_a	1.10 ft	Wall Pier Aspect Ratio		Adj. Factor	
L2	2.92 ft	h_o	4.50 ft	$P1=h_o/L1=$	1.35	N/A	
h_{wall}	9.00 ft	h_b	3.40 ft	$P2=h_o/L2=$	1.54	N/A	
L_{wall}	11.25 ft	Lo1	5.00 ft				

1. Hold-down forces: $H = Vh_{wall}/L_{wall}$ 1826 lbf

2. Unit shear above + below opening

First opening: $va1 = vb1 = H/(h_a+h_b) =$ 406 plf

3. Total boundary force above + below openings

First opening: O1 = va1 x (Lo1) = 2028 lbf

4. Corner forces

F1 = O1(L1)/(L1+L2) = 1081 lbf

F2 = O1(L2)/(L1+L2) = 948 lbf

5. Tributary length of openings

T1 = (L1*Lo1)/(L1+L2) = 2.66 ft

T2 = (L2*Lo1)/(L1+L2) = 2.34 ft

6. Unit shear beside opening

v1 = (V/L)(L1+T1)/L1 = 365 plf

v2 = (V/L)(T2+L2)/L2 = 365 plf

Check v1*L1+v2*L2=V? 2282 lbf **OK**

7. Resistance to corner forces

R1 = v1*L1 = 1216 lbf

R2 = v2*L2 = 1066 lbf

8. Difference corner force + resistance

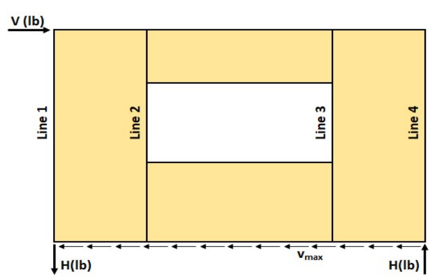
R1-F1 = 135 lbf

R2-F2 = 118 lbf

9. Unit shear in corner zones

vc1 = (R1-F1)/L1 = 41 plf

vc2 = (R2-F2)/L2 = 41 plf



Check Summary of Shear Values for One Opening

Line 1: $vc1(h_a+h_b)+v1(h_o)=H$?		183	1643	1826 lbf
Line 2: $va1(h_a+h_b)-vc1(h_a+h_b)-v1(h_o)=0$?	1826	183	1643	0
Line 3: $va1(h_a+h_b)-vc2(h_a+h_b)-v1(h_o)=0$?	1826	183	1643	0
Line 4: $vc2(h_a+h_b)+v2(h_o)=H$?		183	1643	1826 lbf

Design Summary*

Req. Sheathing Capacity	406 plf	**4-Term Deflection**		**3-Term Deflection**	
Req. Strap Force	1081 lbf	**4-Term Story Drift %**		**3-Term Story Drift %**	
Req. HD Force (H)	1826 lbf				
Req. Shear Wall Anchorage Force (v_{max})	203 plf				

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

Project Information

Code:		**Date:**	6/2/2025
Designer:	Chon Pieruccioni, PE		
Client:			
Project:	East Town Crossing - Building A		
Wall Line:	Grid 6.8 - 11'-3" Segment - L1E		

Shear Wall Deflection Calculation Variables

Unfactored Shear Load $V_{unfactored}$: 2282 (lbf)

Sheathing Type: 7/16 OSB
Grade: APA Rated Sheathing

G_t Override:
G_a Override:

Wood End Post Values:
Species: HF#2
E: 1.30E+06 (psi)

Enter individual post sizes below.

C_d: 4.00

Nail Type: 8d common (penny weight)

	Pier 1	Pier 2	
Nail Spacing:	4	4	(in.)
HD Capacity:	2140	2140	(lbf)
HD Deflection:	0.088	0.088	(in.)

Four-Term Equation Deflection Check

$$\Delta = \frac{8vh^3}{EAb} + \frac{vh}{Gt} + 0.75he_n + d_a\frac{h}{b} \qquad \text{(Equation 23-2)}$$

	Pier 1-L	Pier 1-R	Pier 2-L	Pier 2-R	
$v_{unfactored}$:	365	365	365	365	(plf)
E:	1.30E+06	1.30E+06	1.30E+06	1.30E+06	(psi)
h:	9.00	5.60	5.60	9.00	(ft)
Qty:					
Stud Size:					
A Override:					(in.^{2})
A:					(in.^{2})
G_t:	83,500	83,500	83,500	83,500	(lbf/in.)
Nail Spacing:	4	4	4	4	(in.)
V_n:	122	122	122	122	(plf)
e_n:	0.0090	0.0090	0.0090	0.0090	(in.)
b:	3.33	3.33	2.92	2.92	(ft)
HD Capacity:	2140	2140	2140	2140	(lbf)
HD Defl:	0.088	0.088	0.088	0.088	(in.)

Sheathing Type: 7/16 OSB APA Rated Sheathing
Nail Type: 8d common

Check Total Deflection of Wall System

Pier 1 (left)				Pier 1 (right)			
Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 4 HD-1	Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 4 HD-2
	0.039	0.061	0.365		0.024	0.038	0.141
		Sum				Sum	
Pier 2 (left)				**Pier 2 (right)**			
Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 4 HD-1	Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 4 HD-2
	0.024	0.038	0.161		0.039	0.061	0.416
		Sum				Sum	

Total Defl.	
	(in.)
	%drift

Project Information

Code:		**Date:**	6/2/2025
Designer:	Chon Pieruccioni, PE		
Client:			
Project:	East Town Crossing - Building A		
Wall Line:	Grid 6.8 - 11'-3" Segment - L1E		

Shear Wall Deflection Calculation Variables

Unfactored Shear Load $V_{unfactored}$: 2282 (lbf)

Sheathing Type: 7/16 OSB
Grade: APA Rated Sheathing

Wood End Post Values:
Species: HF#2
E: 1.30E+06 (psi)

Nail Type: 8d common (penny weight)

G_t Override:
G_a Override:

C_d: 4.00

	Pier 1	Pier 2	
Nail Spacing:	4	4	(in.)
HD Capacity:	2140	2140	(lbf)
HD Deflection:	0.088	0.088	(in.)

Three-Term Equation Deflection Check

$$\delta_{sw} = \frac{8vh^3}{EAb} + \frac{vh}{1000G_a} + \frac{h\Delta_a}{b} \quad (4.3\text{-}1)$$

	Pier 1-L	Pier 1-R	Pier 2-L	Pier 2-R	
$v_{unfactored}$:	365	365	365	365	(plf)
E:	1.30E+06	1.30E+06	1.30E+06	1.30E+06	(psi)
h:	9.00	5.60	5.60	9.00	(ft)
Qty:					
Stud Size:					
A Override:					$(in.^2)$
A:					$(in.^2)$
G_a:	22.0	22.0	22.0	22.0	(kips/in.)
b:	3.33	3.33	2.92	2.92	(ft)
HD Capacity:	2140	2140	2140	2140	(lbf)
HD Defl:	0.088	0.088	0.088	0.088	(in.)

Sheathing Type: 7/16 OSB APA Rated Sheathing

Nail Type: 8d common

Check Total Deflection of Wall System

Pier 1 (left)			Pier 1 (right)		
Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 1 Bending	Term 2 Shear	Term 3 Fastener
	0.149	0.365		0.093	0.141
	Sum			Sum	
Pier 2 (left)			**Pier 2 (right)**		
Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 1 Bending	Term 2 Shear	Term 3 Fastener
	0.093	0.161		0.149	0.416
	Sum			Sum	

Total Defl.	
	(in.)
	%drift

Comment: The 3-term equation is calibrated to be approximately equal to 4-term equation at 1.4*ASD capacity.

APA

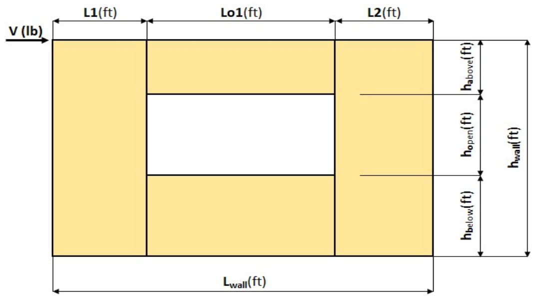
Force Transfer Around Openings Calculator

ONE OPENING

The force transfer around openings (FTAO) method of shear wall analysis is an approach that aims to reinforce the wall such that it performs as if there was no opening. This approach lends certain advantages over segmented shear walls: more versatility, because it allows for narrower wall segments while still meeting the height-to-width ratios and, often, fewer required hold-downs.

Project Information

Code:		**Date:**	6/2/2025
Designer:	Chon Pieruccioni, PE		
Client:			
Project:	East Town Crossing - Building A		
Wall Line:	Grid 7.5 - 10'-10" Segment - L2E		



Shear Wall Calculation Variables

V	1717 lbf		Opening 1		Adj. Factor Method =		1.25-0.125h/bs
L1	3.63 ft	h_a	1.50 ft		Wall Pier Aspect Ratio		Adj. Factor
L2	4.70 ft	h_o	3.00 ft		$P1=h_o/L1=$	0.83	N/A
h_{wall}	9.00 ft	h_b	4.50 ft		$P2=h_o/L2=$	0.64	N/A
L_{wall}	10.83 ft	Lo1	2.50 ft				

1. Hold-down forces: $H = Vh_{wall}/L_{wall}$ 1427 lbf

2. Unit shear above + below opening

First opening: $va1 = vb1 = H/(h_a+h_b) =$	238 plf

3. Total boundary force above + below openings

First opening: O1 = va1 x (Lo1) =	595 lbf

4. Corner forces

F1 = O1(L1)/(L1+L2) =	259 lbf
F2 = O1(L2)/(L1+L2) =	335 lbf

5. Tributary length of openings

T1 = (L1*Lo1)/(L1+L2) =	1.09 ft
T2 = (L2*Lo1)/(L1+L2) =	1.41 ft

6. Unit shear beside opening

v1 = (V/L)(L1+T1)/L1 =	206 plf	
v2 = (V/L)(T2+L2)/L2 =	206 plf	
Check v1*L1+v2*L2=V?	1717 lbf	**OK**

7. Resistance to corner forces

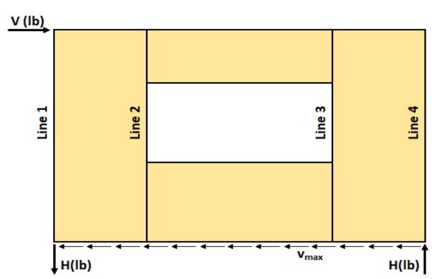
R1 = v1*L1 =	748 lbf
R2 = v2*L2 =	969 lbf

8. Difference corner force + resistance

R1-F1 =	489 lbf
R2-F2 =	633 lbf

9. Unit shear in corner zones

vc1 = (R1-F1)/L1 =	135 plf
vc2 = (R2-F2)/L2 =	135 plf



Check Summary of Shear Values for One Opening

Line 1: $vc1(h_a+h_b)+v1(h_o)=H?$		809	618	1427 lbf
Line 2: $va1(h_a+h_b)-vc1(h_a+h_b)-v1(h_o)=0?$	1427	809	618	0
Line 3: $va1(h_a+h_b)-vc2(h_a+h_b)-v1(h_o)=0?$	1427	809	618	0
Line 4: $vc2(h_a+h_b)+v2(h_o)=H?$		809	618	1427 lbf

Design Summary*

Req. Sheathing Capacity	238 plf	**4-Term Deflection**		**3-Term Deflection**	
Req. Strap Force	335 lbf	**4-Term Story Drift %**		**3-Term Story Drift %**	
Req. HD Force (H)	1427 lbf				
Req. Shear Wall Anchorage Force (v_{max})	159 plf				

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

Project Information

Code:		**Date:**	6/2/2025
Designer:	Chon Pieruccioni, PE		
Client:			
Project:	East Town Crossing - Building A		
Wall Line:	Grid 7.5 - 10'-10" Segment - L2E		

Shear Wall Deflection Calculation Variables

Unfactored Shear Load $V_{unfactored}$: 1717 (lbf)

Sheathing Type: 7/16 OSB
Grade: APA Rated Sheathing

G_t Override:
G_a Override:

Wood End Post Values:
Species: HF#2
E: 1.30E+06 (psi)

Enter individual post sizes below.

C_d: 4.00

Nail Type: 8d common (penny weight)

	Pier 1	Pier 2	
Nail Spacing:	6	6	(in.)
HD Capacity:	2140	2140	(lbf)
HD Deflection:	0.088	0.088	(in.)

Four-Term Equation Deflection Check

$$\Delta = \frac{8vh^3}{EAb} + \frac{vh}{Gt} + 0.75he_n + d_a\frac{h}{b} \qquad \text{(Equation 23-2)}$$

Sheathing Type: 7/16 OSB APA Rated Sheathing
Nail Type: 8d common

	Pier 1-L	Pier 1-R	Pier 2-L	Pier 2-R	
$v_{unfactored}$:	206	206	206	206	(plf)
E:	1.30E+06	1.30E+06	1.30E+06	1.30E+06	(psi)
h:	9.00	4.50	4.50	9.00	(ft)
Qty:					
Stud Size:					
A Override:					(in.^{2})
A:					(in.^{2})
G_t:	83,500	83,500	83,500	83,500	(lbf/in.)
Nail Spacing:	6	6	6	6	(in.)
V_n:	103	103	103	103	(plf)
e_n:	0.0054	0.0054	0.0054	0.0054	(in.)
b:	3.63	3.63	4.70	4.70	(ft)
HD Capacity:	2140	2140	2140	2140	(lbf)
HD Defl:	0.088	0.088	0.088	0.088	(in.)

Check Total Deflection of Wall System

Pier 1 (left)				Pier 1 (right)			
Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 4 HD-1	Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 4 HD-2
	0.022	0.037	0.189		0.011	0.018	0.047
		Sum				Sum	
Pier 2 (left)				**Pier 2 (right)**			
Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 4 HD-1	Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 4 HD-2
	0.011	0.018	0.037		0.022	0.037	0.146
		Sum				Sum	

Total Defl.	
	(in.) %drift

Project Information

Code:		**Date:**	6/2/2025
Designer:	Chon Pieruccioni, PE		
Client:			
Project:	East Town Crossing - Building A		
Wall Line:	Grid 7.5 - 10'-10" Segment - L2E		

Shear Wall Deflection Calculation Variables

Unfactored Shear Load $V_{unfactored}$: 1717 (lbf)

Sheathing Type: 7/16 OSB
Grade: APA Rated Sheathing

Wood End Post Values:
Species: HF#2
E: 1.30E+06 (psi)

Nail Type: 8d common (penny weight)

G_t Override:
G_a Override:

C_d: 4.00

	Pier 1	Pier 2	
Nail Spacing:	6	6	(in.)
HD Capacity:	2140	2140	(lbf)
HD Deflection:	0.088	0.088	(in.)

Three-Term Equation Deflection Check

$$\delta_{sw} = \frac{8vh^3}{EAb} + \frac{vh}{1000G_a} + \frac{h\Delta_a}{b} \qquad (4.3\text{-}1)$$

	Pier 1-L	Pier 1-R	Pier 2-L	Pier 2-R	
$v_{unfactored}$:	206	206	206	206	(plf)
E:	1.30E+06	1.30E+06	1.30E+06	1.30E+06	(psi)
h:	9.00	4.50	4.50	9.00	(ft)
Qty:					
Stud Size:					
A Override:					(in.^{2})
A:					(in.^{2})
G_a:	15.0	15.0	15.0	15.0	(kips/in.)
b:	3.63	3.63	4.70	4.70	(ft)
HD Capacity:	2140	2140	2140	2140	(lbf)
HD Defl:	0.088	0.088	0.088	0.088	(in.)

Sheathing Type: 7/16 OSB APA Rated Sheathing

Nail Type: 8d common

Check Total Deflection of Wall System

Pier 1 (left)			Pier 1 (right)		
Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 1 Bending	Term 2 Shear	Term 3 Fastener
	0.124	0.189		0.062	0.047
	Sum			Sum	
Pier 2 (left)			**Pier 2 (right)**		
Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 1 Bending	Term 2 Shear	Term 3 Fastener
	0.062	0.037		0.124	0.146
	Sum			Sum	

Total Defl.	
	(in.)
	%drift

Comment: The 3-term equation is calibrated to be approximately equal to 4-term equation at 1.4*ASD capacity.

APA

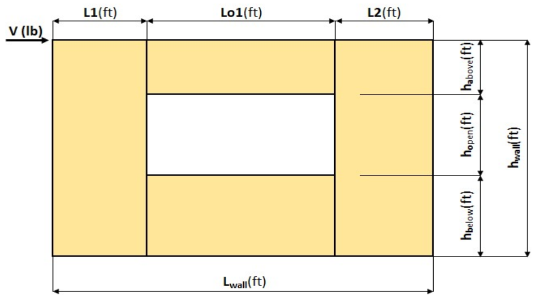
Force Transfer Around Openings Calculator

ONE OPENING

The force transfer around openings (FTAO) method of shear wall analysis is an approach that aims to reinforce the wall such that it performs as if there was no opening. This approach lends certain advantages over segmented shear walls: more versatility, because it allows for narrower wall segments while still meeting the height-to-width ratios and, often, fewer required hold-downs.

Project Information

Code:		**Date:**	6/2/2025
Designer:	Chon Pieruccioni, PE		
Client:			
Project:	East Town Crossing - Building A		
Wall Line:	Grid 7.5 - 10'-10" Segment - L1E		



Shear Wall Calculation Variables

V	2197 lbf		Opening 1		Adj. Factor Method =	1.25-0.125h/bs	
L1	3.63 ft	h_a	1.50 ft		Wall Pier Aspect Ratio		Adj. Factor
L2	4.70 ft	h_o	3.00 ft		$P1=h_o/L1=$	0.83	N/A
h_{wall}	9.00 ft	h_b	4.50 ft		$P2=h_o/L2=$	0.64	N/A
L_{wall}	10.83 ft	Lo1	2.50 ft				

1. Hold-down forces: $H = Vh_{wall}/L_{wall}$ 1826 lbf

2. Unit shear above + below opening

First opening: $va1 = vb1 = H/(h_a+h_b) =$	304 plf

3. Total boundary force above + below openings

First opening: O1 = va1 x (Lo1) =	761 lbf

4. Corner forces

F1 = O1(L1)/(L1+L2) =	332 lbf
F2 = O1(L2)/(L1+L2) =	429 lbf

5. Tributary length of openings

T1 = (L1*Lo1)/(L1+L2) =	1.09 ft
T2 = (L2*Lo1)/(L1+L2) =	1.41 ft

6. Unit shear beside opening

v1 = (V/L)(L1+T1)/L1 =	264 plf	
v2 = (V/L)(T2+L2)/L2 =	264 plf	
Check v1*L1+v2*L2=V?	2197 lbf	**OK**

7. Resistance to corner forces

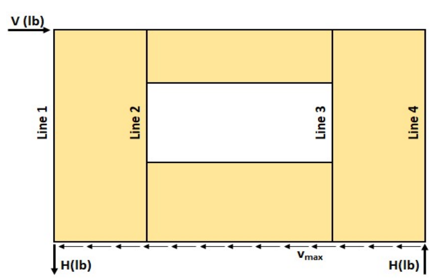
R1 = v1*L1 =	957 lbf
R2 = v2*L2 =	1240 lbf

8. Difference corner force + resistance

R1-F1 =	626 lbf
R2-F2 =	810 lbf

9. Unit shear in corner zones

vc1 = (R1-F1)/L1 =	172 plf
vc2 = (R2-F2)/L2 =	172 plf



Check Summary of Shear Values for One Opening

Line 1: $vc1(h_a+h_b)+v1(h_o)=H?$		1035	791	1826 lbf
Line 2: $va1(h_a+h_b)-vc1(h_a+h_b)-v1(h_o)=0?$	1826	1035	791	0
Line 3: $va1(h_a+h_b)-vc2(h_a+h_b)-v1(h_o)=0?$	1826	1035	791	0
Line 4: $vc2(h_a+h_b)+v2(h_o)=H?$		1035	791	1826 lbf

Design Summary*

Req. Sheathing Capacity	304 plf	**4-Term Deflection**		**3-Term Deflection**	
Req. Strap Force	429 lbf	**4-Term Story Drift %**		**3-Term Story Drift %**	
Req. HD Force (H)	1826 lbf				
Req. Shear Wall Anchorage Force (v_{max})	203 plf				

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

Project Information

Code:		**Date:**	6/2/2025
Designer:	Chon Pieruccioni, PE		
Client:			
Project:	East Town Crossing - Building A		
Wall Line:	Grid 7.5 - 10'-10" Segment - L1E		

Shear Wall Deflection Calculation Variables

Unfactored Shear Load $V_{unfactored}$: 2197 (lbf)

Sheathing Type: 7/16 OSB
Grade: APA Rated Sheathing

G_t Override:
G_a Override:

Wood End Post Values:
Species: HF#2
E: 1.30E+06 (psi)

Enter individual post sizes below.

C_d: 4.00

Nail Type: 8d common (penny weight)

	Pier 1	Pier 2	
Nail Spacing:	6	6	(in.)
HD Capacity:	2140	2140	(lbf)
HD Deflection:	0.088	0.088	(in.)

Four-Term Equation Deflection Check

$$\Delta = \frac{8vh^3}{EAb} + \frac{vh}{Gt} + 0.75he_n + d_a\frac{h}{b} \qquad \text{(Equation 23-2)}$$

Sheathing Type: 7/16 OSB APA Rated Sheathing
Nail Type: 8d common

	Pier 1-L	Pier 1-R	Pier 2-L	Pier 2-R	
$v_{unfactored}$:	264	264	264	264	(plf)
E:	1.30E+06	1.30E+06	1.30E+06	1.30E+06	(psi)
h:	9.00	4.50	4.50	9.00	(ft)
Qty:					
Stud Size:					
A Override:					(in.^{2})
A:					(in.^{2})
G_t:	83,500	83,500	83,500	83,500	(lbf/in.)
Nail Spacing:	6	6	6	6	(in.)
V_n:	132	132	132	132	(plf)
e_n:	0.0115	0.0115	0.0115	0.0115	(in.)
b:	3.63	3.63	4.70	4.70	(ft)
HD Capacity:	2140	2140	2140	2140	(lbf)
HD Defl:	0.088	0.088	0.088	0.088	(in.)

Check Total Deflection of Wall System

Pier 1 (left)				Pier 1 (right)			
Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 4 HD-1	Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 4 HD-2
	0.028	0.077	0.242		0.014	0.039	0.061
		Sum				Sum	
Pier 2 (left)				**Pier 2 (right)**			
Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 4 HD-1	Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 4 HD-2
	0.014	0.039	0.047		0.028	0.077	0.187
		Sum				Sum	

Total Defl.	
	(in.)
	%drift

Project Information

Code:		**Date:**	6/2/2025
Designer:	Chon Pieruccioni, PE		
Client:			
Project:	East Town Crossing - Building A		
Wall Line:	Grid 7.5 - 10'-10" Segment - L1E		

Shear Wall Deflection Calculation Variables

Unfactored Shear Load $V_{unfactored}$: 2197 (lbf)

Sheathing Type: 7/16 OSB
Grade: APA Rated Sheathing

Wood End Post Values:
Species: HF#2
E: 1.30E+06 (psi)

Nail Type: 8d common (penny weight)

G_t Override:
G_a Override:

C_d: 4.00

	Pier 1	Pier 2	
Nail Spacing:	6	6	(in.)
HD Capacity:	2140	2140	(lbf)
HD Deflection:	0.088	0.088	(in.)

Three-Term Equation Deflection Check

$$\delta_{sw} = \frac{8vh^3}{EAb} + \frac{vh}{1000G_a} + \frac{h\Delta_a}{b} \qquad (4.3\text{-}1)$$

Sheathing Type: 7/16 OSB APA Rated Sheathing
Nail Type: 8d common

	Pier 1-L	Pier 1-R	Pier 2-L	Pier 2-R	
$v_{unfactored}$:	264	264	264	264	(plf)
E:	1.30E+06	1.30E+06	1.30E+06	1.30E+06	(psi)
h:	9.00	4.50	4.50	9.00	(ft)
Qty:					
Stud Size:					
A Override:					(in.^{2})
A:					(in.^{2})
G_a:	15.0	15.0	15.0	15.0	(kips/in.)
b:	3.63	3.63	4.70	4.70	(ft)
HD Capacity:	2140	2140	2140	2140	(lbf)
HD Defl:	0.088	0.088	0.088	0.088	(in.)

Check Total Deflection of Wall System

Pier 1 (left)			Pier 1 (right)		
Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 1 Bending	Term 2 Shear	Term 3 Fastener
	0.158	0.242		0.079	0.061
	Sum			Sum	
Pier 2 (left)			**Pier 2 (right)**		
Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 1 Bending	Term 2 Shear	Term 3 Fastener
	0.079	0.047		0.158	0.187
	Sum			Sum	

Total Defl.	
	(in.)
	%drift

Comment: The 3-term equation is calibrated to be approximately equal to 4-term equation at 1.4*ASD capacity.