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

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

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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	

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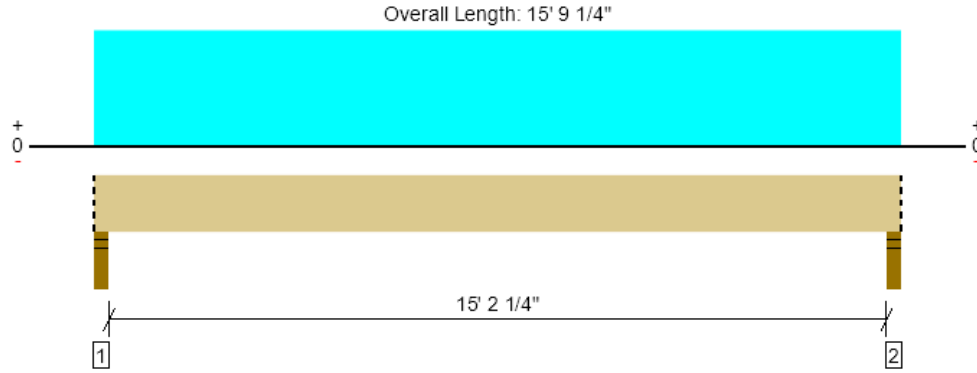


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File Name: East Town Crossing Building A

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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

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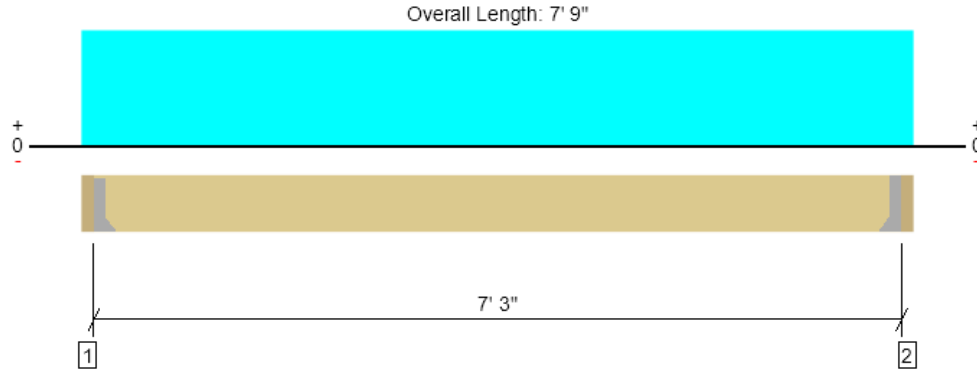
The product application, input design loads, dimensions and support information have been provided by ForteWEB Software Operator

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File Name: East Town Crossing Building A

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)

- 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.

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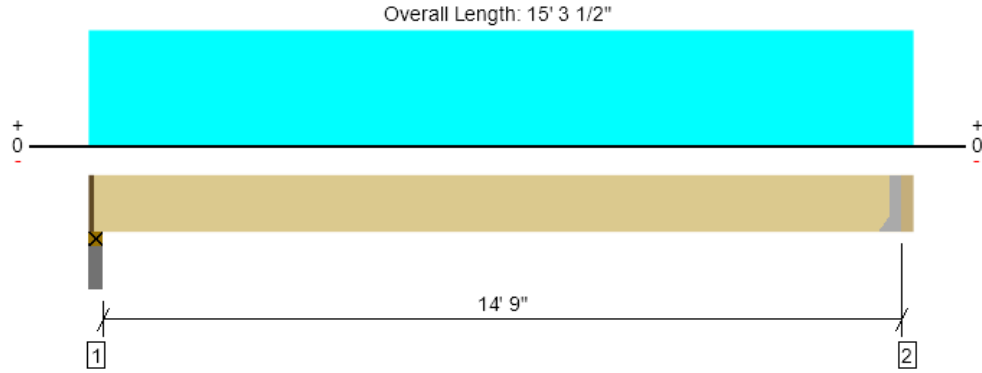
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File Name: East Town Crossing Building A

2nd Floor Framing, Long Short Stair Stringers
1 piece(s) 3 1/2" x 12" 24F-V4 DF Glulam



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)	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.

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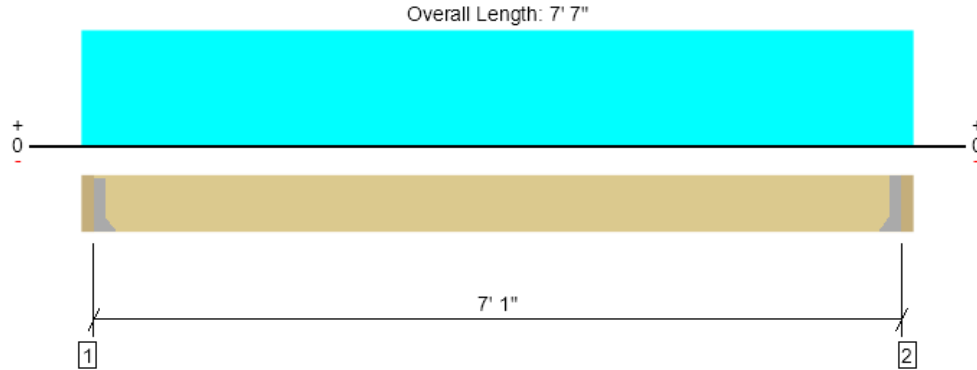
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2nd Floor Framing, 7'-1" Landing Joists
1 piece(s) 2 x 12 HF No.2 @ 16" OC



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"	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

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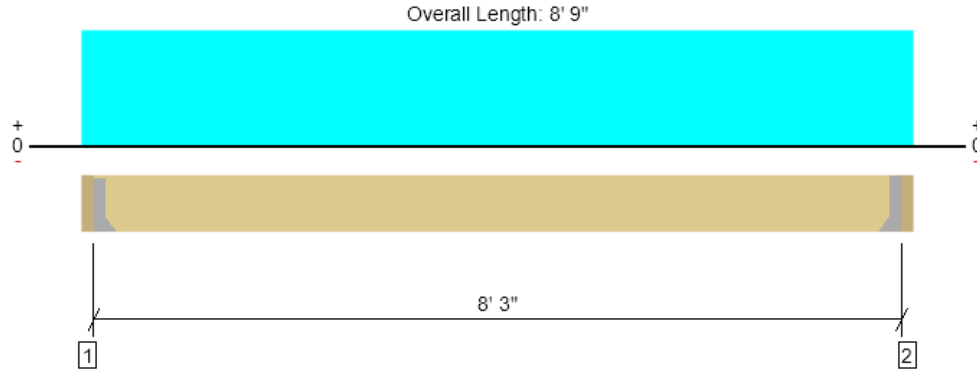
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2nd Floor Framing, 8'-3" Landing Joists
1 piece(s) 2 x 12 HF No.2 @ 12" OC



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

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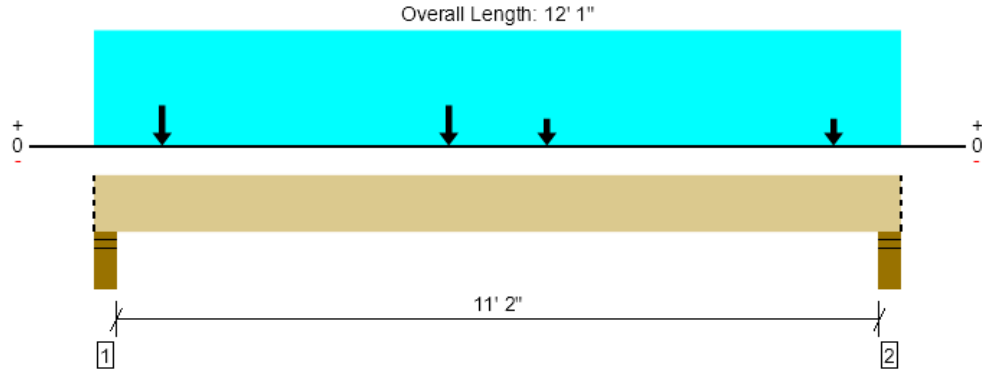
The product application, input design loads, dimensions and support information have been provided by ForteWEB Software Operator

ForteWEB Software Operator	Job Notes
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File Name: East Town Crossing Building A

2nd Floor Framing, Top Landing Beam
1 piece(s) 5 1/2" x 13 1/2" 24F-V4 DF Glulam



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

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	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)

- 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.

Member Length : 12' 1"
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	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.

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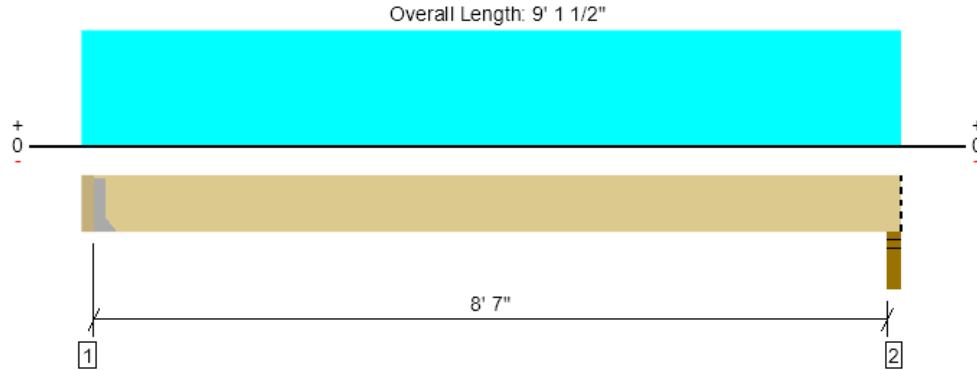
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2nd Floor Framing, Grid 1&8 Deck Beam
2 piece(s) 2 x 10 DF No.2



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)

- 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 : 8' 10 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 - 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.

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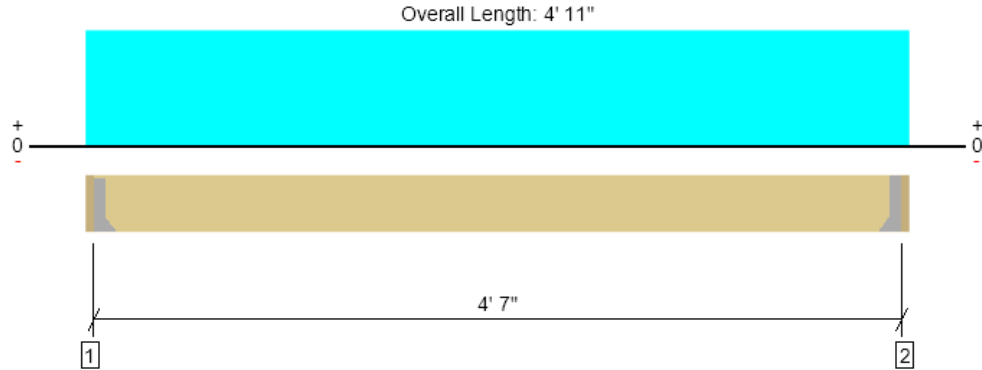
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File Name: East Town Crossing Building A

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

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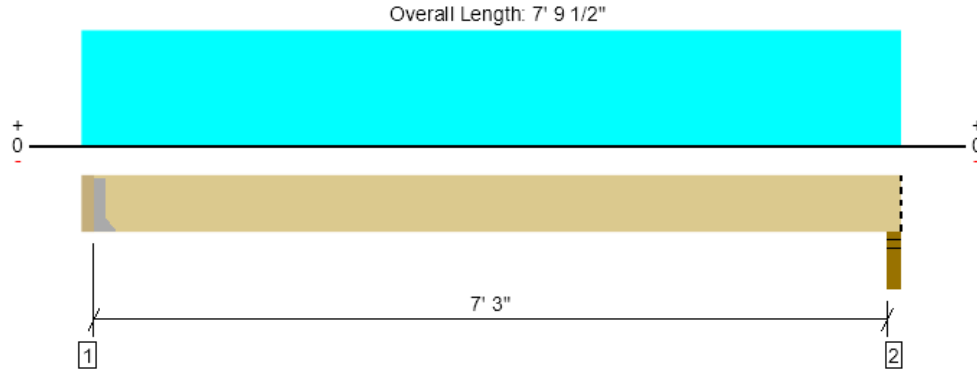
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File Name: East Town Crossing Building A

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)

- 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' 6 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 - 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.

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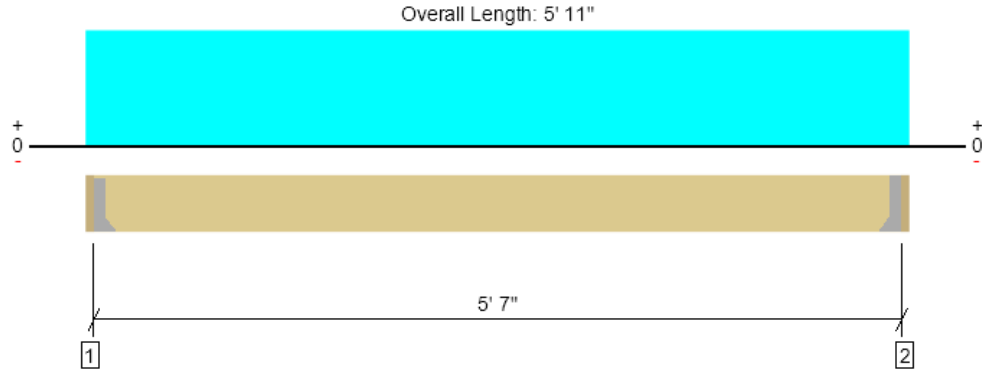
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File Name: East Town Crossing Building A

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

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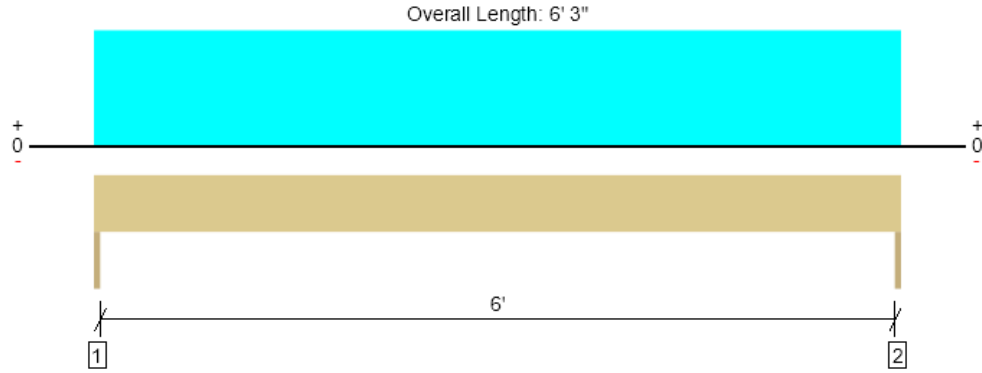
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File Name: East Town Crossing Building A

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)

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

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

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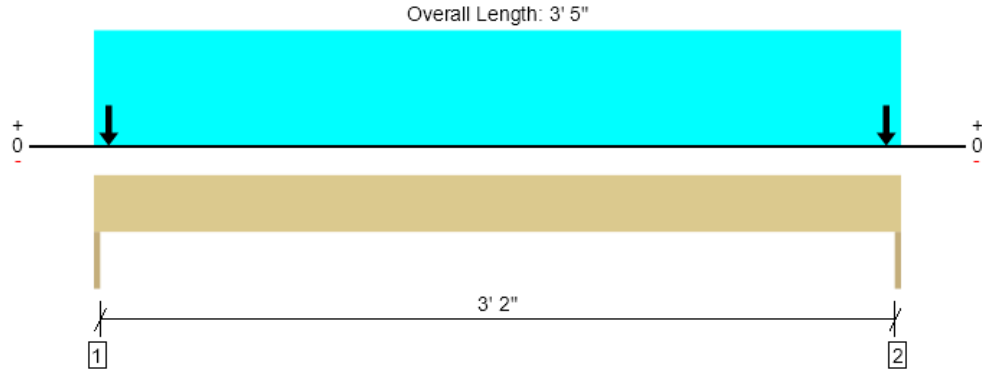
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2nd Floor Framing, Grid 3.1 (D-D.2) Door Header

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)

- 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"	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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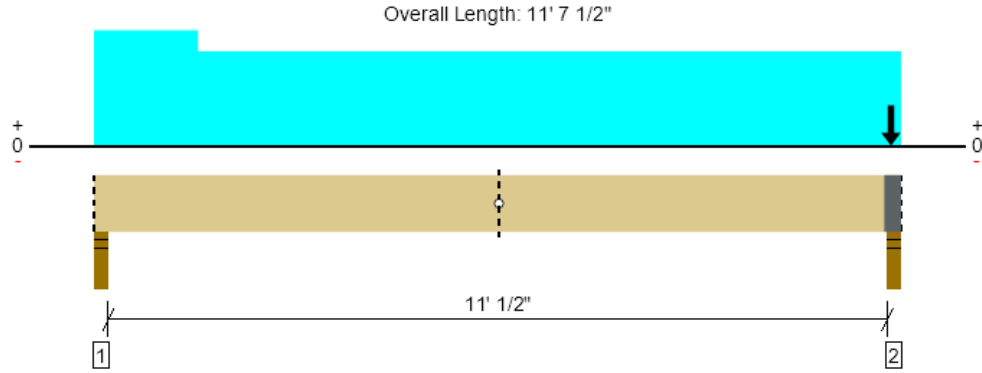
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2nd Floor Framing, Grid 3.1 (D.3-D.6) Flush Beam
2 piece(s) 1 3/4" x 11 1/4" 2.0E Microllam® LVL



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.

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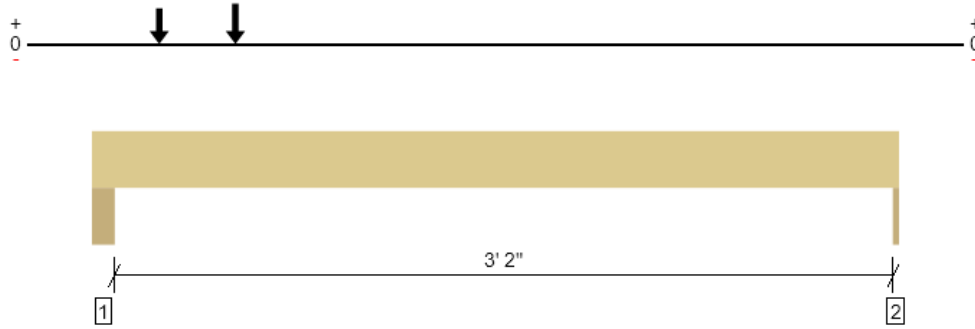
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2nd Floor Framing, Grid 3.6 (3-3.3) Door Header

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

Overall Length: 3' 9"



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)

- 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' 9"
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	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

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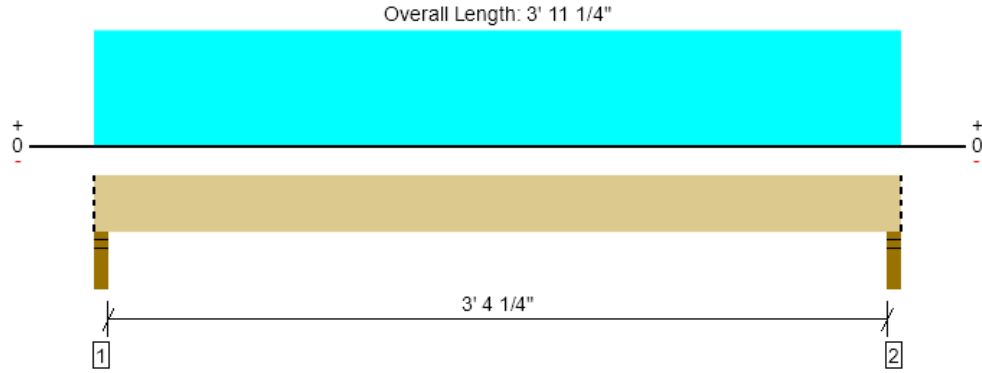
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2nd Floor Framing, Grid 3.3 (B.6-B-.8) Flush Beam

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)

- 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.

Member Length : 3' 11 1/4"
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"	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.

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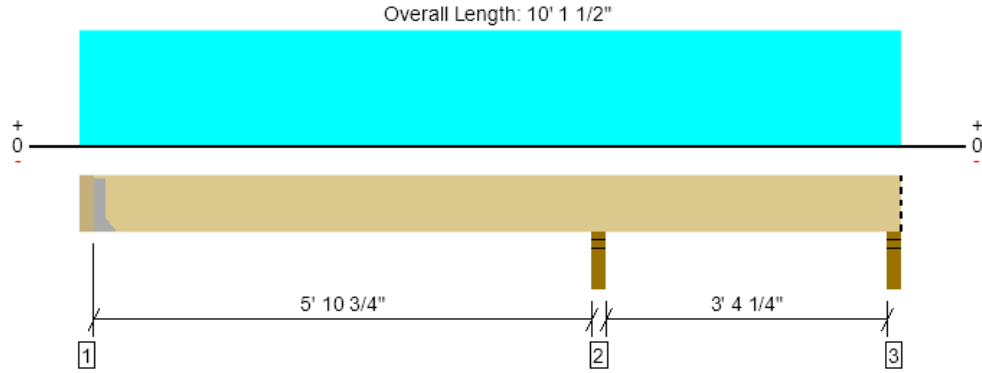
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2nd Floor Framing, Grid 4.1 (B.6-C) Flush Beam

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)

- 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.

Member Length : 9' 10"
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" 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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2nd Floor Framing, Grid 3.1D.3 Post

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

Post Height: 9'



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.

Drawing is Conceptual

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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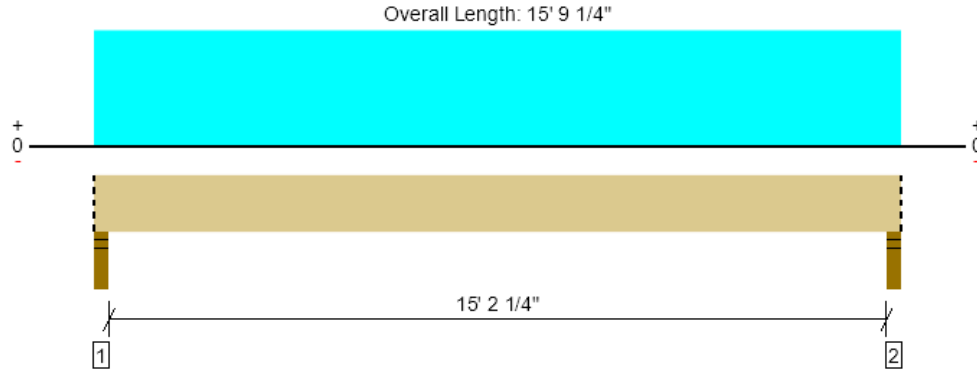
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Page 22 / 43

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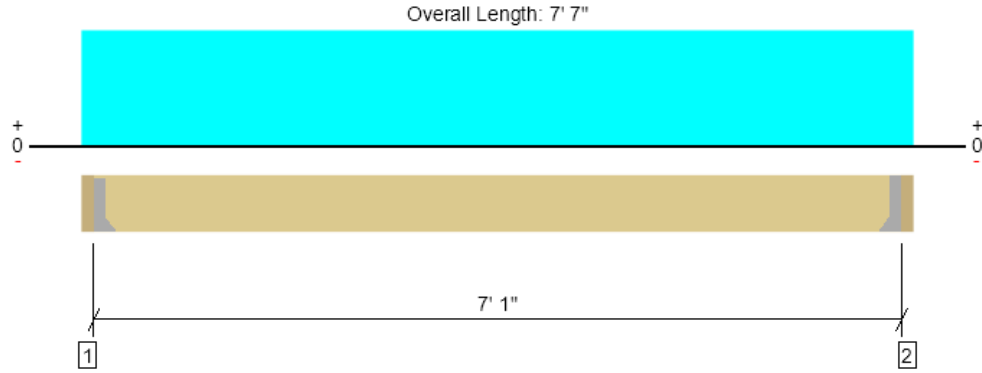
The product application, input design loads, dimensions and support information have been provided by ForteWEB Software Operator

ForteWEB Software Operator	Job Notes
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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, 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"	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	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

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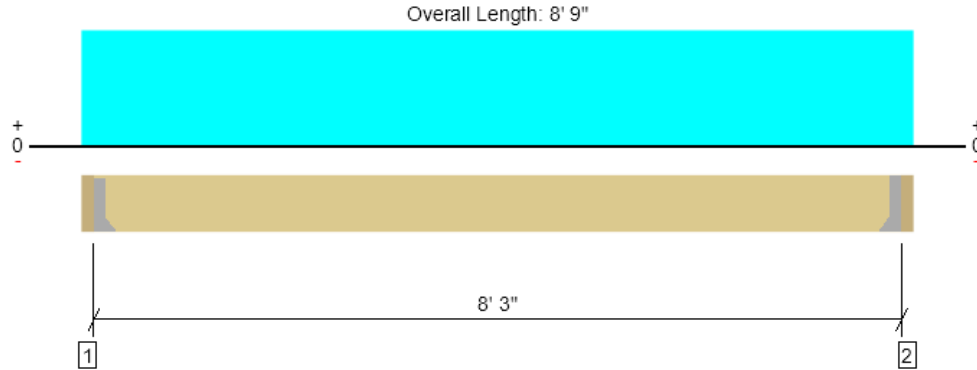
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ForteWEB v3.9, Engine: V8.4.3.94, Data: V8.1.7.3
File Name: East Town Crossing Building A

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

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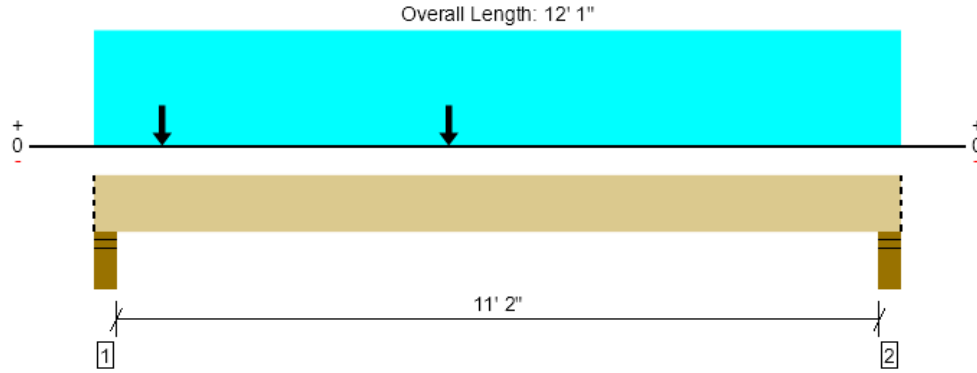
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File Name: East Town Crossing Building A

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)

- 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.

Member Length : 12' 1"
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	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.

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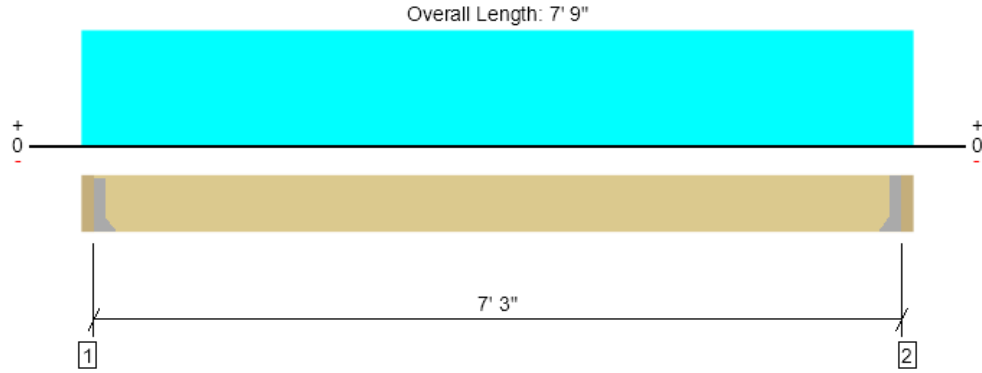
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File Name: East Town Crossing Building A

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.

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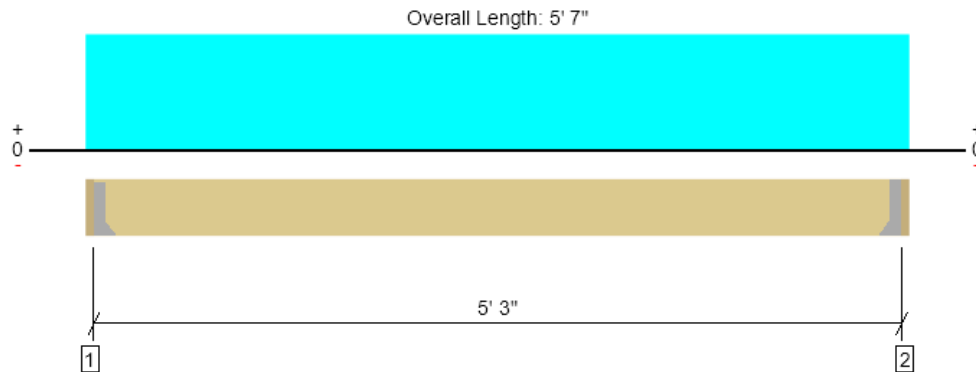
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File Name: East Town Crossing Building A

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

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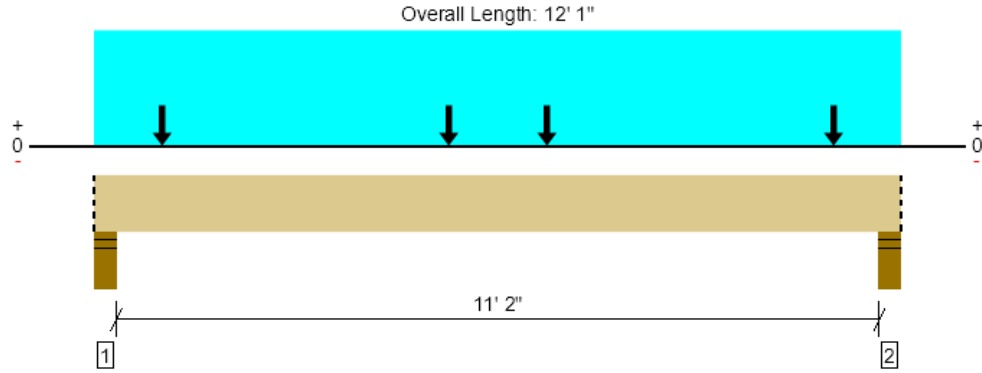
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File Name: East Town Crossing Building A

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)

- 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.

Member Length : 12' 1"
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	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.

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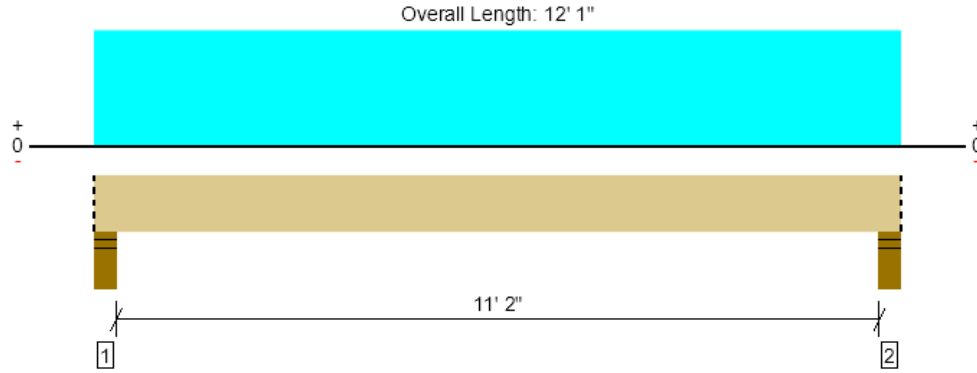
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3rd Floor Framing, Mid Landing Beam Outer
1 piece(s) 3 1/2" x 10 1/2" 24F-V4 DF Glulam



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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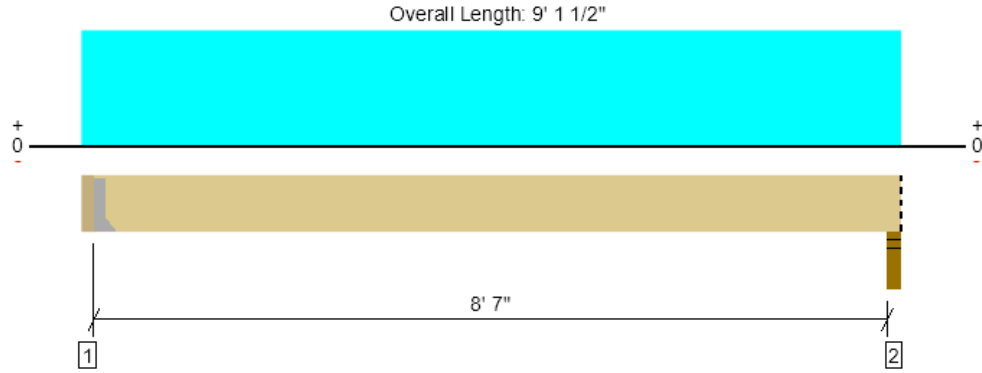
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3rd Floor Framing, Grid 1&8 Deck Beam
2 piece(s) 2 x 10 DF No.2



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)

- 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 : 8' 10 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 - 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.

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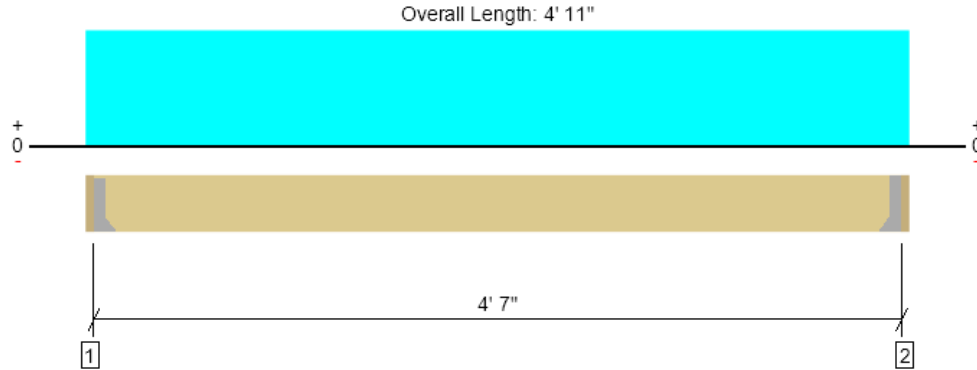
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File Name: East Town Crossing Building A

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

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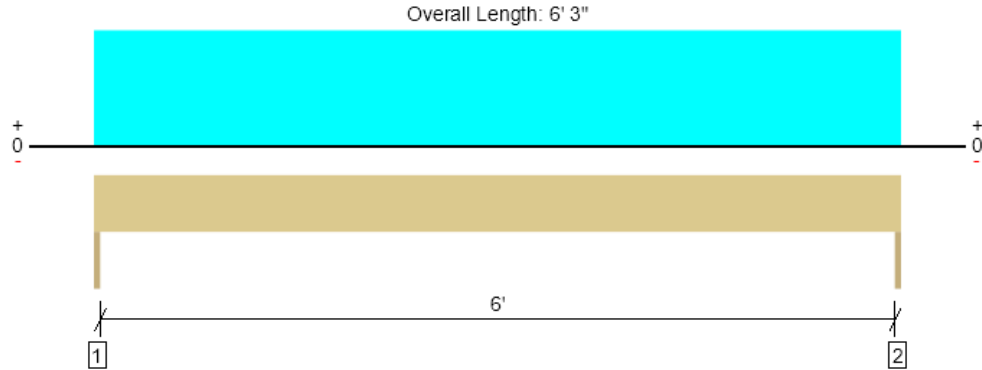
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File Name: East Town Crossing Building A

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)

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

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

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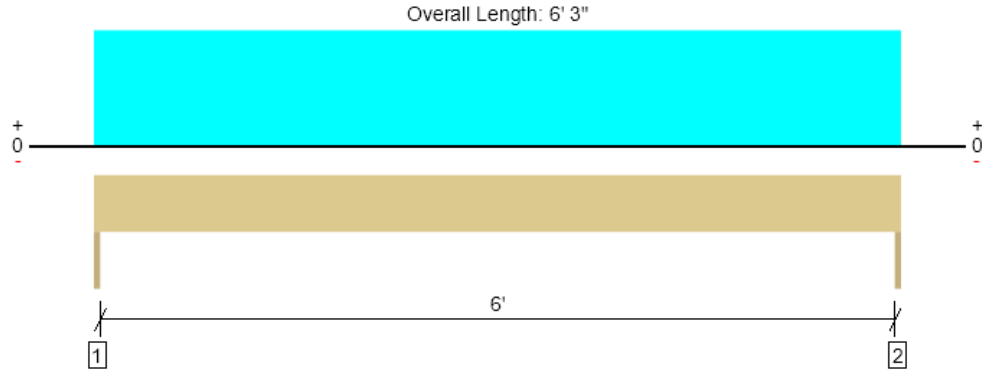
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3rd Floor Framing, 6' Window Header (Grid A)

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)

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

Member Length : 6' 3"
System : Wall
Member Type : Header
Building Use : Residential
Building Code : IBC 2018
Design Methodology : ASD

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Snow	Factored	
1 - Trimmer - HF	1.50"	1.50"	1.50"	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

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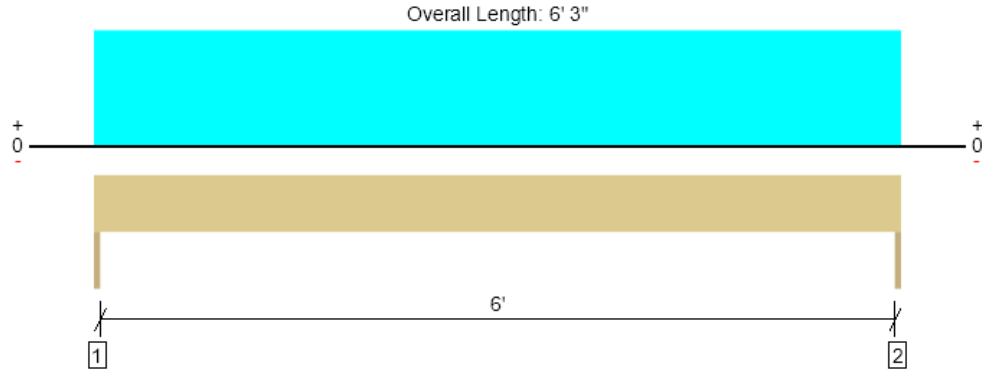
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3rd Floor Framing, 6' Window Header (Grid B)

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)

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

Member Length : 6' 3"
System : Wall
Member Type : Header
Building Use : Residential
Building Code : IBC 2018
Design Methodology : ASD

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Snow	Factored	
1 - Trimmer - HF	1.50"	1.50"	1.50"	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

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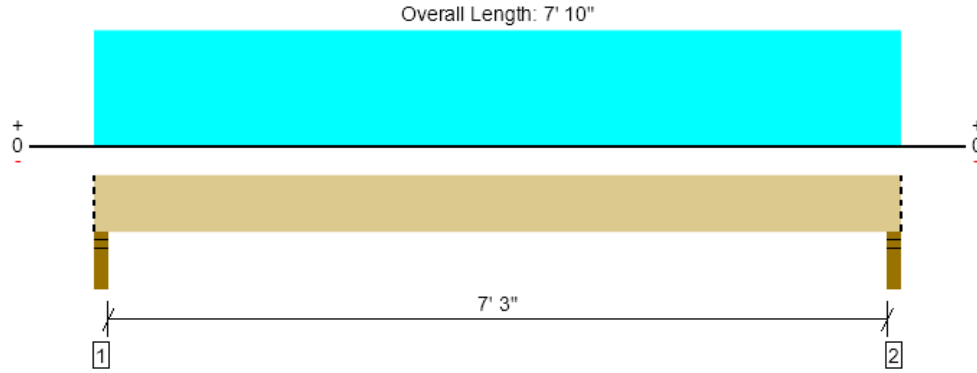
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3rd Floor Framing, Grid A - Deck Roof Beams
1 piece(s) 3 1/2" x 7 1/2" 24F-V4 DF Glulam



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.

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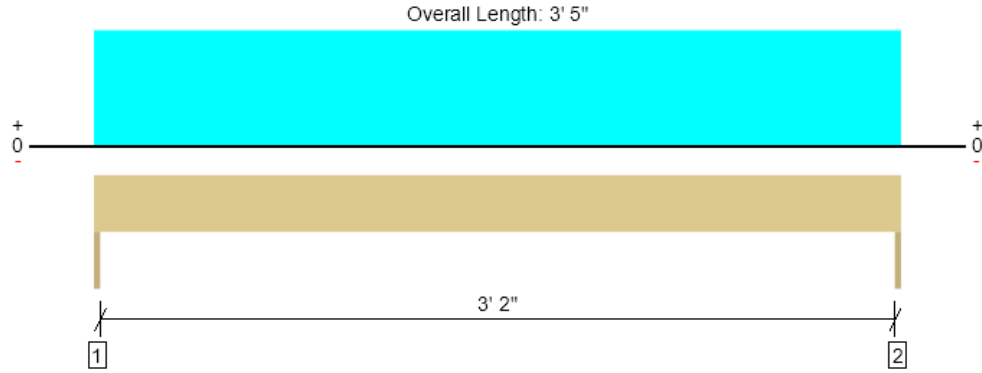
The product application, input design loads, dimensions and support information have been provided by ForteWEB Software Operator

ForteWEB Software Operator	Job Notes
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File Name: East Town Crossing Building A

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

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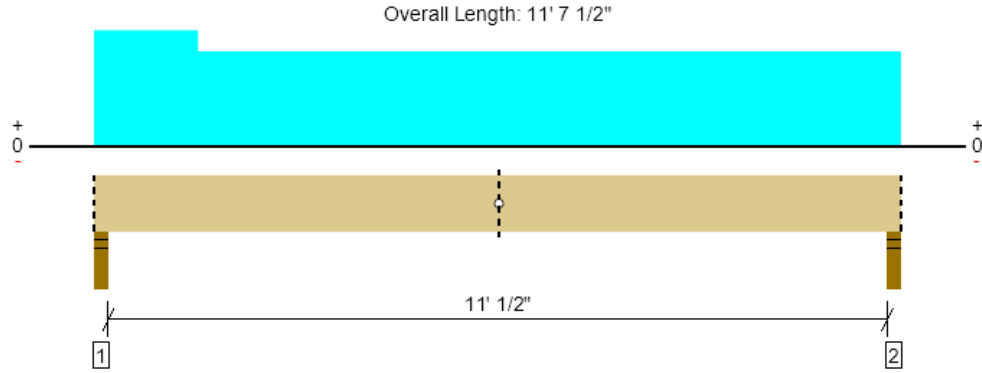
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File Name: East Town Crossing Building A

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.

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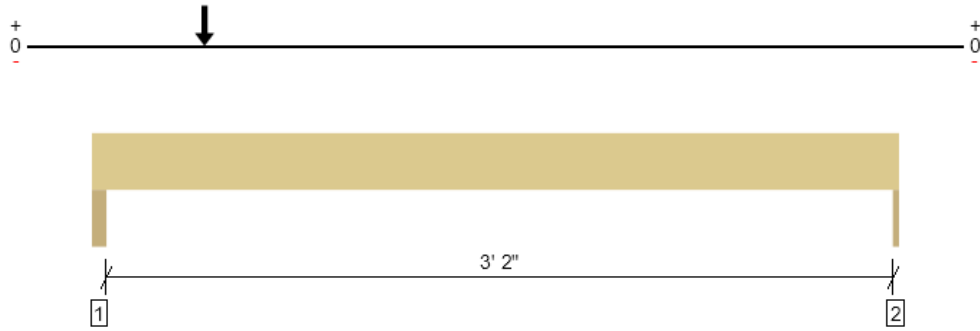


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3rd Floor Framing, Grid 3.6 (3-3.3) Door Header

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

Overall Length: 3' 7"



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)

- 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' 7"
 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	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

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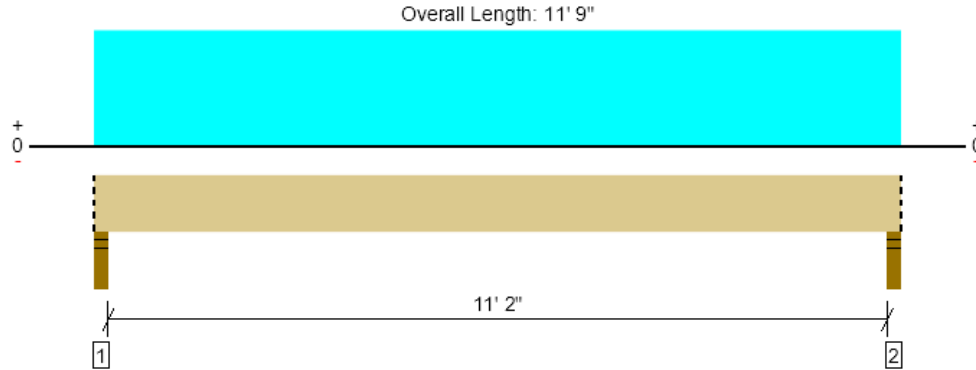
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 File Name: East Town Crossing Building A

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.

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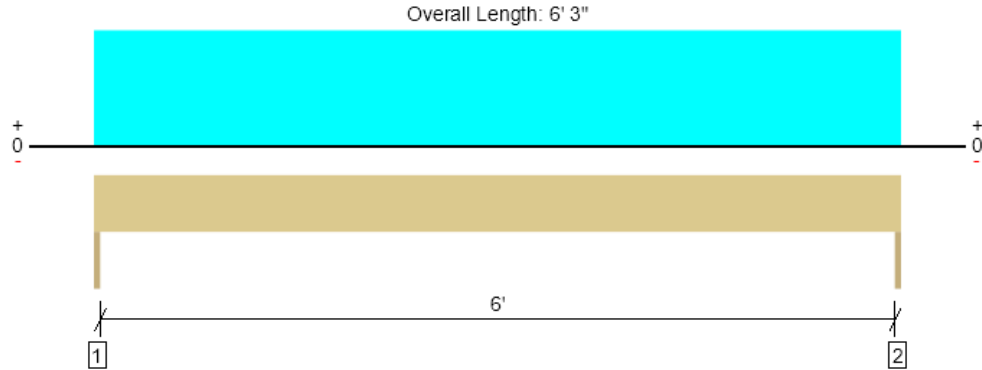
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File Name: East Town Crossing Building A

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)

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

Member Length : 6' 3"
System : Wall
Member Type : Header
Building Use : Residential
Building Code : IBC 2018
Design Methodology : ASD

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Snow	Factored	
1 - Trimmer - HF	1.50"	1.50"	1.50"	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

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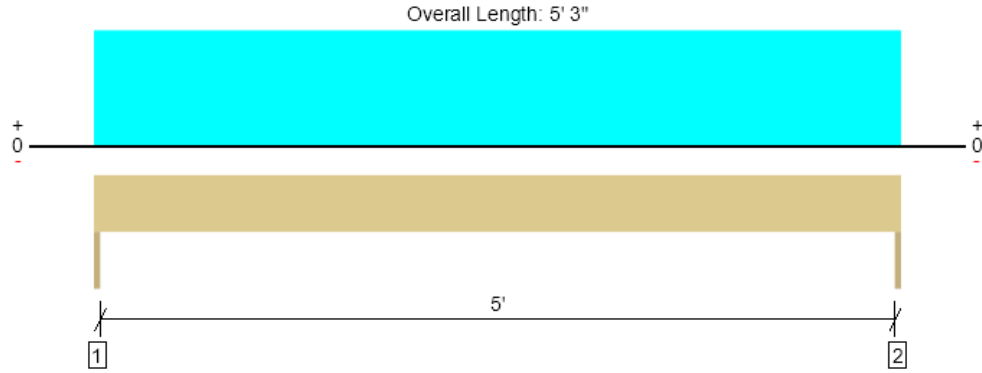
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Roof Framing, 5' Window Header (Grid H)
1 piece(s) 4 x 8 DF No.2



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)

- 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 : 5' 3"
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	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

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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 = $0.6 * W * L * \gamma * (SC + Thick - WT) = 0.6 * 2.50 * 2.50 * 62 * (0.67) = -0.2$ 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 = $\frac{\text{Resisting moment}}{\text{Overturning moment}} = \frac{3.3}{0.0} = 33.49 > 1.50$ OK

- Overturning about Z-Z

$$\text{- Moment } M_z = 0.6 * 0.0 + 0.6 * 0.0 = 0.0 \text{ k-ft}$$

$$\text{- Shear Force } V_x = 0.6 * 0.0 + 0.6 * 0.0 = 0.0 \text{ kip}$$

$$\text{Arm} = 0.00 + 8.0 / 12 = 0.67 \text{ ft}$$

$$\text{Moment} = 0.0 * 0.67 = 0.0 \text{ k-ft}$$

$$\text{- Passive Force} = 0.0 \text{ kip}$$

$$\text{Arm} = 0.27 \text{ ft}$$

$$\text{Moment} = 0.0 \text{ k-ft}$$

$$\text{- Overturning moment Z-Z} = 0.0 + 0.0 = 0.0 \text{ k-ft}$$

- Resisting about Z-Z

$$\text{- Footing weight} = 0.6 * W * L * Thick * Density = 0.6 * 2.50 * 2.50 * 8.0 / 12 * 0.15 = 0.4 \text{ kip}$$

$$\text{Arm} = L / 2 = 2.50 / 2 = 1.25 \text{ ft}$$

$$\text{Moment} = 0.4 * 1.25 = 0.5 \text{ k-ft}$$

$$\text{- Pedestal weight} = 0.6 * W * L * H * Density = 0.6 * 6.0 / 12 * 6.0 / 12 * 0.0 * 0.15 = 0.0 \text{ kip}$$

$$\text{Arm} = L / 2 - Offset = 2.50 / 2 - 0.0 / 12 = 1.25 \text{ ft}$$

$$\text{Moment} = 0.0 * 1.25 = 0.0 \text{ k-ft}$$

$$\text{- Soil cover} = 0.6 * W * L * SC * Density = 0.6 * (2.50 * 2.50 - 6.0 / 12 * 6.0 / 12) * 0.0 * 110 = 0.0 \text{ kip}$$

$$\text{Arm} = L / 2 = 2.50 / 2 = 1.25 \text{ ft}$$

$$\text{Moment} = 0.0 * 1.25 = 0.0 \text{ k-ft}$$

$$\text{- Buoyancy} = 0.6 * W * L * \gamma * (SC + Thick - WT) = 0.6 * 2.50 * 2.50 * 62 * (0.67) = -0.2 \text{ kip}$$

$$\text{Arm} = L / 2 = 2.50 / 2 = 1.25 \text{ ft}$$

$$\text{Moment} = 0.2 * 1.25 = -0.2 \text{ k-ft}$$

$$\text{- Axial force } P = 0.6 * 4.1 + 0.6 * 0.0 = 2.5 \text{ kip}$$

$$\text{Arm} = L / 2 - Offset = 2.50 / 2 - 0.0 / 12 = 1.25 \text{ ft}$$

$$\text{Moment} = 2.5 * 1.25 = 3.1 \text{ k-ft}$$

$$\text{- Resisting moment Z-Z} = 0.5 + 0.0 + 0.0 + 3.1 + -0.2 = 3.3 \text{ k-ft}$$

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

SOIL BEARING PRESSURES (Comb: D+L)

$$\text{Overturning moment X-X} = 0.0 + 0.0 = 0.0 \text{ k-ft}$$

$$\text{Resisting moment X-X} = 0.8 + 0.0 + 0.0 + -0.3 + 11.6 = 12.1 \text{ k-ft}$$

$$\text{Overturning moment Z-Z} = 0.0 + 0.0 = 0.0 \text{ k-ft}$$

$$\text{Resisting moment Z-Z} = 0.8 + 0.0 + 0.0 + -0.3 + 11.6 = 12.1 \text{ k-ft}$$

$$\text{Resisting force} = \text{Footing} + \text{Pedestal} + \text{Soil} - \text{Buoyancy} + P = 0.6 + 0.0 + 0.0 - 0.3 + 9.3 = 9.7 \text{ kip}$$

X-coordinate of resultant from maximum bearing corner:

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

Z-coordinate of resultant from maximum bearing corner:

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

$$X\text{-ecc} = \text{Length} / 2 - X_p = 2.50 / 2 - 1.25 = 0.00 \text{ ft}$$

$$Z\text{-ecc} = \text{Width} / 2 - Z_p = 2.50 / 2 - 1.25 = 0.00 \text{ ft}$$

$$\text{Area} = \text{Width} * \text{Length} = 2.50 * 2.50 = 6.3 \text{ ft}^2$$

$$S_x = \text{Length} * \text{Width}^2 / 6 = 2.50 * 2.50^2 / 6 = 2.6 \text{ ft}^3$$

$$S_z = \text{Width} * \text{Length}^2 / 6 = 2.50 * 2.50^2 / 6 = 2.6 \text{ ft}^3$$

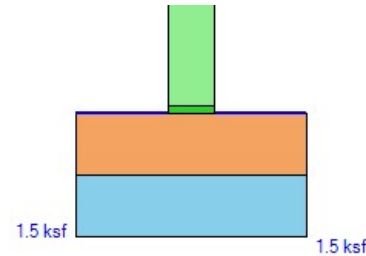
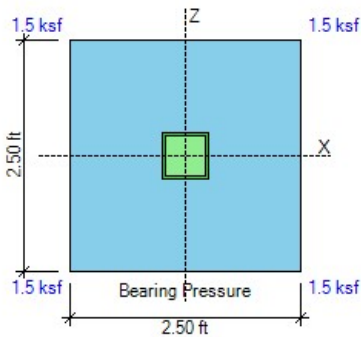
- Footing is in full bearing. Soil pressures are as follows:

$$P1 = P * (1/A + Z\text{-ecc} / S_x + X\text{-ecc} / S_z) = 9.7 * (1/6.3 + 0.00 / 2.6 + 0.00 / 2.6) = 1.55 \text{ ksf}$$

$$P2 = P * (1/A - Z\text{-ecc} / S_x + X\text{-ecc} / S_z) = 9.7 * (1/6.3 - 0.00 / 2.6 + 0.00 / 2.6) = 1.55 \text{ ksf}$$

$$P3 = P * (1/A - Z\text{-ecc} / S_x - X\text{-ecc} / S_z) = 9.7 * (1/6.3 - 0.00 / 2.6 - 0.00 / 2.6) = 1.55 \text{ ksf}$$

$$P4 = P * (1/A + Z\text{-ecc} / S_x - X\text{-ecc} / S_z) = 9.7 * (1/6.3 + 0.00 / 2.6 - 0.00 / 2.6) = 1.55 \text{ ksf}$$



SLIDING CALCULATIONS (Comb: 0.6D+0.6W)

Internal friction angle = 28.0 deg

Passive coefficient $k_p = 4.33$ (per Coulomb)Pressure at mid-depth = $k_p \cdot \text{Density} \cdot (\text{Cover} + \text{Thick} / 2) = 4.33 \cdot 110 \cdot (0.00 + 8.0 / 12 / 2) = 0.16$ ksfX-Passive force = $\text{Pressure} \cdot \text{Thick} \cdot \text{Width} = 0.16 \cdot 8.0 / 12 \cdot 2.50 = 0.3$ kipZ-Passive force = $\text{Pressure} \cdot \text{Thick} \cdot \text{Length} = 0.16 \cdot 8.0 / 12 \cdot 2.50 = 0.3$ kipFriction force = $\text{Resisting force} \cdot \text{Friction coeff.} = \text{Max}(0, 2.7 \cdot 0.35) = 0.9$ kip

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

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

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

UPLIFT CALCULATIONS (Comb: 0.6D+0.6W)

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

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

Concrete $f'_c = 2.5$ ksiSteel $f_y = 40.0$ ksi

Soil density = 110 pcf

Use Plain Concrete Shear Strength

$$\phi V_{cx} = \frac{4}{3} \cdot \phi \cdot \sqrt{f'_c} \cdot \text{Width} \cdot t / 1000 = \frac{4}{3} \cdot 0.60 \cdot \sqrt{2500} \cdot 2.5 \cdot 12 \cdot 8.0 / 1000 = 9.6 \text{ kip}$$

ACI 14.5.5.1

$$\phi V_{cz} = \frac{4}{3} \cdot \phi \cdot \sqrt{f'_c} \cdot \text{Length} \cdot t / 1000 = \frac{4}{3} \cdot 0.60 \cdot \sqrt{2500} \cdot 2.5 \cdot 12 \cdot 8.0 / 1000 = 9.6 \text{ kip}$$

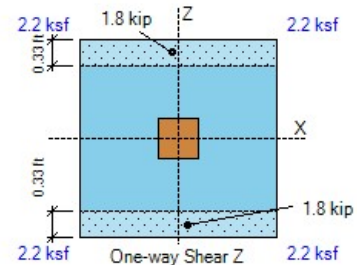
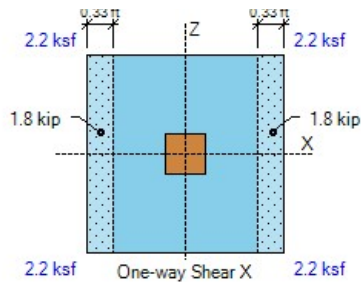
- Shear forces calculated as the volume of the bearing pressures under the effective areas:

$$\text{One-way shear } V_{ux} \text{ (- Side)} = 1.8 \text{ kip} < 9.6 \text{ kip} \quad \text{OK}$$

$$\text{One-way shear } V_{ux} \text{ (+ Side)} = 1.8 \text{ kip} < 9.6 \text{ kip} \quad \text{OK}$$

$$\text{One-way shear } V_{uz} \text{ (- Side)} = 1.8 \text{ kip} < 9.6 \text{ kip} \quad \text{OK}$$

$$\text{One-way shear } V_{uz} \text{ (+ Side)} = 1.8 \text{ kip} < 9.6 \text{ kip} \quad \text{OK}$$



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

$$\text{Plain } \phi M_{nx} = 5 * \phi * \sqrt{f_c} * L * \text{Thick}^2 / 6 = 5 * 0.60 * \sqrt{(2500)} * 2.50 * 8.0^2 / 6 / 1000 = 1.1 \text{ k-ft}$$

ACI Eq. (14.5.2.1a)

$$\text{Plain } \phi M_{nz} = 5 * \phi * \sqrt{f_c} * W * \text{Thick}^2 / 6 = 5 * 0.60 * \sqrt{(2500)} * 2.50 * 8.0^2 / 6 / 1000 = 1.1 \text{ k-ft}$$

- 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:

$$\text{Top moment -Mux (- Side)} = 0.0 \text{ k-ft} < 4.0 \text{ k-ft OK}$$

$$\text{Top moment -Mux (+ Side)} = 0.0 \text{ k-ft} < 4.0 \text{ k-ft OK}$$

$$\text{Top moment -Muz (- Side)} = 0.0 \text{ k-ft} < 4.0 \text{ k-ft OK}$$

$$\text{Top moment -Muz (+ Side)} = 0.0 \text{ k-ft} < 4.0 \text{ k-ft OK}$$

- Bottom Bars

No Bottom Reinforcement Provided at the Footing

Use Plain Concrete Flexural Strength at Bottom

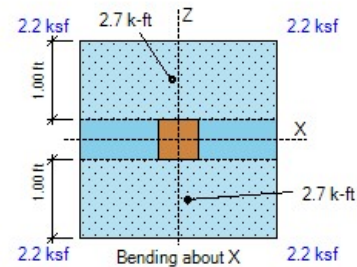
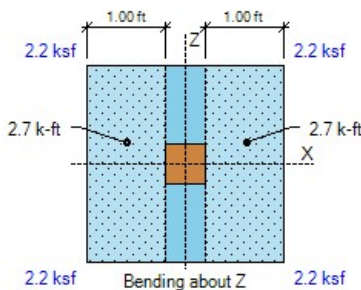
- Bottom moments calculated as the bearing minus the overburden pressures times the lever arm:

$$\text{Bottom moment Mux (- Side)} = 2.7 \text{ k-ft} < 4.0 \text{ k-ft OK} \quad \text{ratio} = 0.66$$

$$\text{Bottom moment Mux (+ Side)} = 2.7 \text{ k-ft} < 4.0 \text{ k-ft OK} \quad \text{ratio} = 0.66$$

$$\text{Bottom moment Muz (- Side)} = 2.7 \text{ k-ft} < 4.0 \text{ k-ft OK} \quad \text{ratio} = 0.66$$

$$\text{Bottom moment Muz (+ Side)} = 2.7 \text{ k-ft} < 4.0 \text{ k-ft OK} \quad \text{ratio} = 0.66$$



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

$$\text{Area } A1 = \text{col } L * \text{col } W = 6.0 * 6.0 = 36.0 \text{ in}^2$$

$$Sx = \text{col } W * \text{col } L^2 / 6 = 6.0 * 6.0^2 / 6 = 36.0 \text{ in}^3$$

$$Sz = \text{col } L * \text{col } W^2 / 6 = 6.0 * 6.0^2 / 6 = 36.0 \text{ in}^3$$

$$\text{Bearing } Pbu = P / A1 + Mz / Sx + Mx / Sz = 13.2 / 36.0 + 0.0 * 12 / 36.0 + 0.0 * 12 / 36.0 = 0.4 \text{ ksi}$$

$$\text{Min edge} = \text{Min} (L / 2 - X\text{-offset} - \text{col } L / 2, W / 2 - Z\text{-offset} - \text{col } W / 2)$$

$$\text{Min edge} = \text{Min} (2.50 * 12 / 2 - 0.0 - 6.0 / 2, 2.50 * 12 / 2 - 0.0 - 6.0 / 2) = 12.0 \text{ in}$$

$$\text{Area } A2 = \text{Min} [L * W, (\text{col } L + 2 * \text{Min edge}) * (\text{col } W + 2 * \text{Min edge})]$$

ACI R22.8.3.2

$$A2 = \text{Min} [2.50 * 12 * 2.5 * 12, (6.0 + 2 * 12.0) * (6.0 + 2 * 12.0)] = 900.0 \text{ in}^2$$

$$\text{Footing } \phi Pnc = \phi * 0.85 * f'c * \text{Min} [2, \sqrt{(A2 / A1)}] = 0.65 * 0.85 * 2.5 * \text{Min} [2, \sqrt{(900.0 / 36.0)}] = 2.8 \text{ ksi}$$

$$\text{Footing } \phi Pns = \phi * As * Fy / A1 = 0.0 \text{ ksi}$$

ACI 22.8.3.2

$$\text{Footing bearing } \phi Pn = \phi Pnc + \phi Pns = 2.8 + 0.0 = 2.8 \text{ ksi} > 0.4 \text{ psi OK}$$

Hooked $L_{dh} = \text{Max} (8 \text{ db}, 6, 0.02 * f_y / (f_c)^{1/2} * \text{Confining} * \text{Location} * \text{Concrete} * \text{db} * \text{ratio})$

ACI 25.4.3

$$L_{dh} = \text{Max} (8 \text{ db}, 6, 0.02 * 60.0 * 1000 / (2500)^{1/2} * 1.0 * 0.7 * 0.0 * 0.75 * 0.06) = 6.0 \text{ in}$$

Ld provided = Dowel length = $3.00 * 12 = 36.0 \text{ in} > 12.0 \text{ in OK}$

Ldh provided = Footing thickness - Cover = $8.00 - 3.0 = 5.0 \text{ in} < 6.0 \text{ in NG}$

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

$$\text{X-Edge} = \text{Length} / 2 - \text{Offset} - \text{Col} / 2 = 2.50 * 12 / 2 - 0.0 - 6.0 / 2 = 12.0 \text{ in} \quad \alpha_{sx} = 10$$

$$\text{Z-Edge} = \text{Width} / 2 - \text{Offset} - \text{Col} / 2 = 2.50 * 12 / 2 - 0.0 - 6.0 / 2 = 12.0 \text{ in} \quad \alpha_{sz} = 10$$

$$\alpha_s = \alpha_{sx} + \alpha_{sz} = 10 + 10 = 20 \quad \text{Col type} = \text{Corner} \quad \beta = L / W = 6.0 / 6.0 = 1.00$$

ACI 22.6.5.2

$$\text{Perimeter } b_o = \alpha_{sz} / 10 * (L + d / 2 + \text{X-Edge}) + \alpha_{sx} / 10 * (W + d / 2 + \text{Z-Edge})$$

ACI 22.6.4.2

$$b_o = 10 / 10 * (6.0 + 8.0 / 2 + 12.0) + 10 / 10 * (6.0 + 8.0 / 2 + 12.0) = 44.0 \text{ in}$$

$$\text{Area } A_{bo} = (L + d / 2 + \text{X-Edge}) * (W + d / 2 + \text{Z-Edge}) = (6.0 + 8.0 / 2 + 12.0) * (6.0 + 8.0 / 2 + 12.0) = 484.0 \text{ in}^2$$

Use Plain Concrete Shear Strength

$$\phi V_c = \phi * \text{Min} (1 + 2 / \beta, 2) * 4/3 * \sqrt{f_c}$$

ACI 14.5.5.1

$$\phi V_c = 0.60 * \text{Min} (1 + 2 / 1.00, 2) * 4/3 * \sqrt{2500} = 80.0 \text{ psi}$$

Punching force $F = P + \text{Overburden} * A_{bo} - \text{Bearing}$

$$F = 13.2 + 0.07 * 484.0 / 144 - 3.0 = 10.5 \text{ kip}$$

$$b_1 = L + d / 2 + \text{X-Edge} = 6.0 + 8.0 / 2 + 12.0 = 22.0 \text{ in} \quad b_2 = W + d / 2 + \text{Z-Edge} = 6.0 + 8.0 / 2 + 12.0 = 22.0 \text{ in}$$

$$\gamma_{vx} \text{ factor} = 1 - \frac{1}{1 + (2/3) \sqrt{(b_2 / b_1)}} = 1 - \frac{1}{1 + (2/3) \sqrt{(22.0 / 22.0)}} = 0.40$$

ACI Eq. (8.4.4.2.2)

$$\gamma_{vz} \text{ factor} = 1 - \frac{1}{1 + (2/3) \sqrt{(b_1 / b_2)}} = 1 - \frac{1}{1 + (2/3) \sqrt{(22.0 / 22.0)}} = 0.40$$

ACI Eq. (8.4.2.3.2)

$$X_{2z} = b_1^2 / 2 / (b_1 + b_2) = 22.0^2 / 2 / (22.0 + 22.0) = 5.5 \text{ in} \quad X_{2x} = b_2^2 / 2 / (b_2 + b_1) = 5.5 \text{ in}$$

$$J_{cz} = b_1 * d^3 / 12 + b_1^3 * d / 12 + b_1 * d * (b_1 / 2 - X_{2z})^2 + b_2 * d * X_{2z}^2$$

ACI R8.4.4.2.3

$$J_{cz} = 22.0 * 8.0^3 / 12 + 22.0^3 * 8.0 / 12 + 22.0 * 8.0 * (22.0 / 2 - 5.5)^2 + 22.0 * 8.0 * 5.5^2 = 18685 \text{ in}^4$$

$$J_{cx} = b_2 * d^3 / 12 + b_2^3 * d / 12 + b_2 * d * (b_2 / 2 - X_{2x})^2 + b_1 * d * X_{2x}^2$$

ACI R8.4.4.2.3

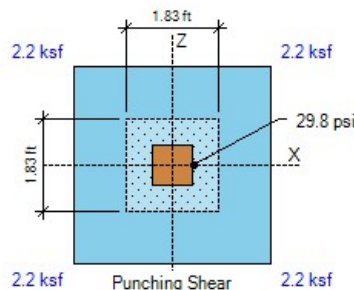
$$J_{cx} = 22.0 * 8.0^3 / 12 + 22.0^3 * 8.0 / 12 + 22.0 * 8.0 * (22.0 / 2 - 5.5)^2 + 22.0 * 8.0 * 5.5^2 = 18685 \text{ in}^4$$

$$\text{Stress due to } P = F / (b_o * d) * 1000 = 10.5 / (44.0 * 8.0) * 1000 = 29.8 \text{ psi}$$

$$\text{Stress due to } M_x = \gamma_{vx} * X\text{-OTM} * X_{2x} / J_{cx} = 0.40 * 0.0 * 12 * 5.5 / 18685 * 1000 = 0.0 \text{ psi}$$

$$\text{Stress due to } M_z = \gamma_{vz} * Z\text{-OTM} * X_{2z} / J_{cz} = 0.40 * 0.0 * 12 * 5.5 / 18685 * 1000 = 0.0 \text{ psi}$$

$$\text{Punching stress} = P\text{-stress} + M_x\text{-stress} + M_z\text{-stress} = 29.8 + 0.0 + 0.0 = 29.8 \text{ psi} < 80.0 \text{ psi OK}$$



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

$$\text{Area } A1 = \text{col } L * \text{col } W = 6.0 * 6.0 = 36.0 \text{ in}^2$$

$$Sx = \text{col } W * \text{col } L^2 / 6 = 6.0 * 6.0^2 / 6 = 36.0 \text{ in}^3$$

$$Sz = \text{col } L * \text{col } W^2 / 6 = 6.0 * 6.0^2 / 6 = 36.0 \text{ in}^3$$

$$\text{Bearing } Pbu = P / A1 + Mz / Sx + Mx / Sz = 13.2 / 36.0 + 0.0 * 12 / 36.0 + 0.0 * 12 / 36.0 = 0.4 \text{ ksi}$$

$$\text{Min edge} = \text{Min} (L / 2 - X\text{-offset} - \text{col } L / 2, W / 2 - Z\text{-offset} - \text{col } W / 2)$$

$$\text{Min edge} = \text{Min} (2.50 * 12 / 2 - 0.0 - 6.0 / 2, 2.50 * 12 / 2 - 0.0 - 6.0 / 2) = 12.0 \text{ in}$$

$$\text{Area } A2 = \text{Min} [L * W, (\text{col } L + 2 * \text{Min edge}) * (\text{col } W + 2 * \text{Min edge})]$$

ACI R22.8.3.2

$$A2 = \text{Min} [2.50 * 12 * 2.5 * 12, (6.0 + 2 * 12.0) * (6.0 + 2 * 12.0)] = 900.0 \text{ in}^2$$

$$\text{Footing } \phi Pnc = \phi * 0.85 * f'c * \text{Min} [2, \sqrt{(A2 / A1)}] = 0.65 * 0.85 * 2.5 * \text{Min} [2, \sqrt{(900.0 / 36.0)}] = 2.8 \text{ ksi}$$

$$\text{Footing } \phi Pns = \phi * As * Fy / A1 = 0.0 \text{ ksi}$$

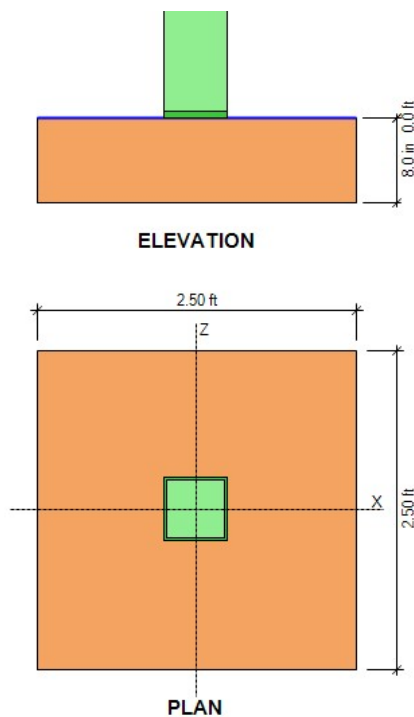
ACI 22.8.3.2

$$\text{Footing bearing } \phi Pn = \phi Pnc + \phi Pns = 2.8 + 0.0 = 2.8 \text{ ksi} > 0.4 \text{ psi OK}$$

DESIGN CODES

Concrete Design ACI 318-14

Load Combinations ASCE 7-10/16



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

$$\text{Arm} = 0.00 + 8.0 / 12 = 0.67 \text{ ft}$$

$$\text{Moment} = 0.0 * 0.67 = 0.0 \text{ k-ft}$$

- Passive Force = 0.0 kip

$$\text{Arm} = 0.27 \text{ ft}$$

$$\text{Moment} = 0.0 \text{ k-ft}$$

- 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

$$\text{Arm} = W / 2 = 2.50 / 2 = 1.25 \text{ ft}$$

$$\text{Moment} = 0.4 * 1.25 = 0.5 \text{ k-ft}$$

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

$$\text{Arm} = W / 2 - Offset = 2.50 / 2 - 0.0 / 12 = 1.25 \text{ ft}$$

$$\text{Moment} = 0.0 * 1.25 = 0.0 \text{ k-ft}$$

- 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

$$\text{Arm} = W / 2 = 2.50 / 2 = 1.25 \text{ ft}$$

$$\text{Moment} = 0.0 * 1.25 = 0.0 \text{ k-ft}$$

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

$$\text{Arm} = W / 2 = 2.50 / 2 = 1.25 \text{ ft}$$

$$\text{Moment} = 0.2 * 1.25 = -0.2 \text{ k-ft}$$

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

$$\text{Arm} = W / 2 - Offset = 2.50 / 2 - 0.0 / 12 = 1.25 \text{ ft}$$

$$\text{Moment} = 2.3 * 1.25 = 2.9 \text{ k-ft}$$

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

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

- Overturning about Z-Z

$$\text{- Moment } M_z = 0.6 * 0.0 + 0.6 * 0.0 = 0.0 \text{ k-ft}$$

$$\text{- Shear Force } V_x = 0.6 * 0.0 + 0.6 * 0.0 = 0.0 \text{ kip}$$

$$\text{Arm} = 0.00 + 8.0 / 12 = 0.67 \text{ ft}$$

$$\text{Moment} = 0.0 * 0.67 = 0.0 \text{ k-ft}$$

$$\text{- Passive Force} = 0.0 \text{ kip}$$

$$\text{Arm} = 0.27 \text{ ft}$$

$$\text{Moment} = 0.0 \text{ k-ft}$$

$$\text{- Overturning moment Z-Z} = 0.0 + 0.0 = 0.0 \text{ k-ft}$$

- Resisting about Z-Z

$$\text{- Footing weight} = 0.6 * W * L * Thick * Density = 0.6 * 2.50 * 2.50 * 8.0 / 12 * 0.15 = 0.4 \text{ kip}$$

$$\text{Arm} = L / 2 = 2.50 / 2 = 1.25 \text{ ft}$$

$$\text{Moment} = 0.4 * 1.25 = 0.5 \text{ k-ft}$$

$$\text{- Pedestal weight} = 0.6 * W * L * H * Density = 0.6 * 6.0 / 12 * 6.0 / 12 * 0.0 * 0.15 = 0.0 \text{ kip}$$

$$\text{Arm} = L / 2 - Offset = 2.50 / 2 - 0.0 / 12 = 1.25 \text{ ft}$$

$$\text{Moment} = 0.0 * 1.25 = 0.0 \text{ k-ft}$$

$$\text{- Soil cover} = 0.6 * W * L * SC * Density = 0.6 * (2.50 * 2.50 - 6.0 / 12 * 6.0 / 12) * 0.0 * 110 = 0.0 \text{ kip}$$

$$\text{Arm} = L / 2 = 2.50 / 2 = 1.25 \text{ ft}$$

$$\text{Moment} = 0.0 * 1.25 = 0.0 \text{ k-ft}$$

$$\text{- Buoyancy} = 0.6 * W * L * \gamma * (SC + Thick - WT) = 0.6 * 2.50 * 2.50 * 62 * (0.67) = -0.2 \text{ kip}$$

$$\text{Arm} = L / 2 = 2.50 / 2 = 1.25 \text{ ft}$$

$$\text{Moment} = 0.2 * 1.25 = -0.2 \text{ k-ft}$$

$$\text{- Axial force } P = 0.6 * 3.9 + 0.6 * 0.0 = 2.3 \text{ kip}$$

$$\text{Arm} = L / 2 - Offset = 2.50 / 2 - 0.0 / 12 = 1.25 \text{ ft}$$

$$\text{Moment} = 2.3 * 1.25 = 2.9 \text{ k-ft}$$

$$\text{- Resisting moment Z-Z} = 0.5 + 0.0 + 0.0 + 2.9 + -0.2 = 3.2 \text{ k-ft}$$

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

SOIL BEARING PRESSURES (Comb: D+L)

$$\text{Overturning moment X-X} = 0.0 + 0.0 = 0.0 \text{ k-ft}$$

$$\text{Resisting moment X-X} = 0.8 + 0.0 + 0.0 + -0.3 + 11.1 = 11.6 \text{ k-ft}$$

$$\text{Overturning moment Z-Z} = 0.0 + 0.0 = 0.0 \text{ k-ft}$$

$$\text{Resisting moment Z-Z} = 0.8 + 0.0 + 0.0 + -0.3 + 11.1 = 11.6 \text{ k-ft}$$

$$\text{Resisting force} = \text{Footing} + \text{Pedestal} + \text{Soil} - \text{Buoyancy} + P = 0.6 + 0.0 + 0.0 - 0.3 + 8.9 = 9.3 \text{ kip}$$

X-coordinate of resultant from maximum bearing corner:

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

Z-coordinate of resultant from maximum bearing corner:

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

$$X\text{-ecc} = \text{Length} / 2 - X_p = 2.50 / 2 - 1.25 = 0.00 \text{ ft}$$

$$Z\text{-ecc} = \text{Width} / 2 - Z_p = 2.50 / 2 - 1.25 = 0.00 \text{ ft}$$

$$\text{Area} = \text{Width} * \text{Length} = 2.50 * 2.50 = 6.3 \text{ ft}^2$$

$$S_x = \text{Length} * \text{Width}^2 / 6 = 2.50 * 2.50^2 / 6 = 2.6 \text{ ft}^3$$

$$S_z = \text{Width} * \text{Length}^2 / 6 = 2.50 * 2.50^2 / 6 = 2.6 \text{ ft}^3$$

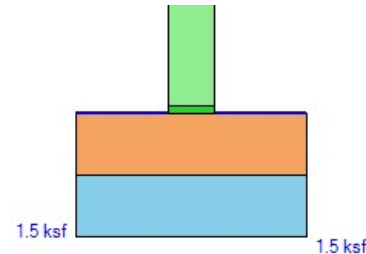
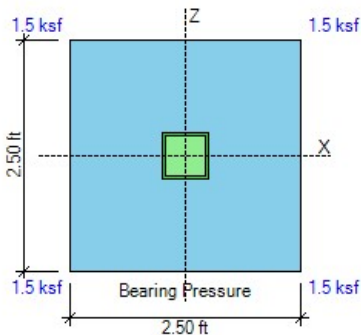
- Footing is in full bearing. Soil pressures are as follows:

$$P1 = P * (1/A + Z\text{-ecc} / S_x + X\text{-ecc} / S_z) = 9.3 * (1/6.3 + 0.00 / 2.6 + 0.00 / 2.6) = 1.48 \text{ ksf}$$

$$P2 = P * (1/A - Z\text{-ecc} / S_x + X\text{-ecc} / S_z) = 9.3 * (1/6.3 - 0.00 / 2.6 + 0.00 / 2.6) = 1.48 \text{ ksf}$$

$$P3 = P * (1/A - Z\text{-ecc} / S_x - X\text{-ecc} / S_z) = 9.3 * (1/6.3 - 0.00 / 2.6 - 0.00 / 2.6) = 1.48 \text{ ksf}$$

$$P4 = P * (1/A + Z\text{-ecc} / S_x - X\text{-ecc} / S_z) = 9.3 * (1/6.3 + 0.00 / 2.6 - 0.00 / 2.6) = 1.48 \text{ ksf}$$



SLIDING CALCULATIONS (Comb: 0.6D+0.6W)

Internal friction angle = 28.0 deg

Passive coefficient $k_p = 4.33$ (per Coulomb)Pressure at mid-depth = $k_p \cdot \text{Density} \cdot (\text{Cover} + \text{Thick} / 2) = 4.33 \cdot 110 \cdot (0.00 + 8.0 / 12 / 2) = 0.16$ ksfX-Passive force = $\text{Pressure} \cdot \text{Thick} \cdot \text{Width} = 0.16 \cdot 8.0 / 12 \cdot 2.50 = 0.3$ kipZ-Passive force = $\text{Pressure} \cdot \text{Thick} \cdot \text{Length} = 0.16 \cdot 8.0 / 12 \cdot 2.50 = 0.3$ kipFriction force = $\text{Resisting force} \cdot \text{Friction coeff.} = \text{Max}(0, 2.6 \cdot 0.35) = 0.9$ kip

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

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

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

UPLIFT CALCULATIONS (Comb: 0.6D+0.6W)

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

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

Concrete $f'_c = 2.5$ ksiSteel $f_y = 40.0$ ksi

Soil density = 110 pcf

Use Plain Concrete Shear Strength

$$\phi V_{cx} = \frac{4}{3} \cdot \phi \cdot \sqrt{f'_c} \cdot \text{Width} \cdot t / 1000 = \frac{4}{3} \cdot 0.60 \cdot \sqrt{(2500)} \cdot 2.5 \cdot 12 \cdot 8.0 / 1000 = 9.6 \text{ kip}$$

ACI 14.5.5.1

$$\phi V_{cz} = \frac{4}{3} \cdot \phi \cdot \sqrt{f'_c} \cdot \text{Length} \cdot t / 1000 = \frac{4}{3} \cdot 0.60 \cdot \sqrt{(2500)} \cdot 2.5 \cdot 12 \cdot 8.0 / 1000 = 9.6 \text{ kip}$$

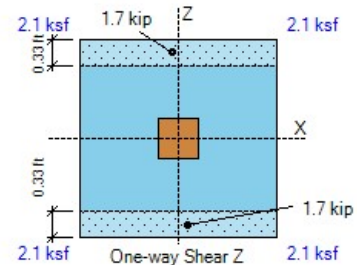
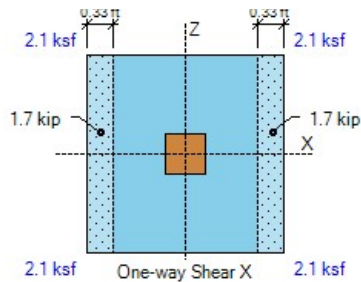
- Shear forces calculated as the volume of the bearing pressures under the effective areas:

$$\text{One-way shear } V_{ux} \text{ (- Side)} = 1.7 \text{ kip} < 9.6 \text{ kip OK}$$

$$\text{One-way shear } V_{ux} \text{ (+ Side)} = 1.7 \text{ kip} < 9.6 \text{ kip OK}$$

$$\text{One-way shear } V_{uz} \text{ (- Side)} = 1.7 \text{ kip} < 9.6 \text{ kip OK}$$

$$\text{One-way shear } V_{uz} \text{ (+ Side)} = 1.7 \text{ kip} < 9.6 \text{ kip OK}$$



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

$$\text{Plain } \phi M_{nx} = 5 * \phi * \sqrt{f_c} * L * \text{Thick}^2 / 6 = 5 * 0.60 * \sqrt{(2500)} * 2.50 * 8.0^2 / 6 / 1000 = 1.1 \text{ k-ft}$$

ACI Eq. (14.5.2.1a)

$$\text{Plain } \phi M_{nz} = 5 * \phi * \sqrt{f_c} * W * \text{Thick}^2 / 6 = 5 * 0.60 * \sqrt{(2500)} * 2.50 * 8.0^2 / 6 / 1000 = 1.1 \text{ k-ft}$$

- 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:

$$\text{Top moment -Mux (- Side)} = 0.0 \text{ k-ft} < 4.0 \text{ k-ft OK}$$

$$\text{Top moment -Mux (+ Side)} = 0.0 \text{ k-ft} < 4.0 \text{ k-ft OK}$$

$$\text{Top moment -Muz (- Side)} = 0.0 \text{ k-ft} < 4.0 \text{ k-ft OK}$$

$$\text{Top moment -Muz (+ Side)} = 0.0 \text{ k-ft} < 4.0 \text{ k-ft OK}$$

- Bottom Bars

No Bottom Reinforcement Provided at the Footing

Use Plain Concrete Flexural Strength at Bottom

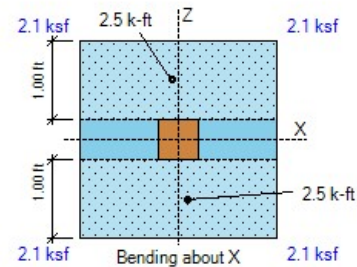
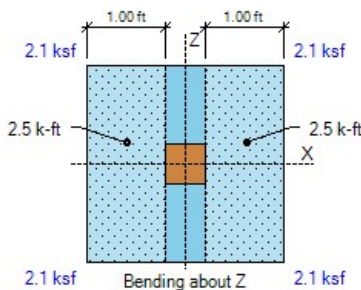
- Bottom moments calculated as the bearing minus the overburden pressures times the lever arm:

$$\text{Bottom moment Mux (- Side)} = 2.5 \text{ k-ft} < 4.0 \text{ k-ft OK} \quad \text{ratio} = 0.64$$

$$\text{Bottom moment Mux (+ Side)} = 2.5 \text{ k-ft} < 4.0 \text{ k-ft OK} \quad \text{ratio} = 0.64$$

$$\text{Bottom moment Muz (- Side)} = 2.5 \text{ k-ft} < 4.0 \text{ k-ft OK} \quad \text{ratio} = 0.64$$

$$\text{Bottom moment Muz (+ Side)} = 2.5 \text{ k-ft} < 4.0 \text{ k-ft OK} \quad \text{ratio} = 0.64$$



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

$$\text{Area } A1 = \text{col } L * \text{col } W = 6.0 * 6.0 = 36.0 \text{ in}^2$$

$$Sx = \text{col } W * \text{col } L^2 / 6 = 6.0 * 6.0^2 / 6 = 36.0 \text{ in}^3$$

$$Sz = \text{col } L * \text{col } W^2 / 6 = 6.0 * 6.0^2 / 6 = 36.0 \text{ in}^3$$

$$\text{Bearing } Pbu = P / A1 + Mz / Sx + Mx / Sz = 12.7 / 36.0 + 0.0 * 12 / 36.0 + 0.0 * 12 / 36.0 = 0.4 \text{ ksi}$$

$$\text{Min edge} = \text{Min} (L / 2 - X\text{-offset} - \text{col } L / 2, W / 2 - Z\text{-offset} - \text{col } W / 2)$$

$$\text{Min edge} = \text{Min} (2.50 * 12 / 2 - 0.0 - 6.0 / 2, 2.50 * 12 / 2 - 0.0 - 6.0 / 2) = 12.0 \text{ in}$$

$$\text{Area } A2 = \text{Min} [L * W, (\text{col } L + 2 * \text{Min edge}) * (\text{col } W + 2 * \text{Min edge})]$$

ACI R22.8.3.2

$$A2 = \text{Min} [2.50 * 12 * 2.5 * 12, (6.0 + 2 * 12.0) * (6.0 + 2 * 12.0)] = 900.0 \text{ in}^2$$

$$\text{Footing } \phi Pnc = \phi * 0.85 * f'c * \text{Min} [2, \sqrt{(A2 / A1)}] = 0.65 * 0.85 * 2.5 * \text{Min} [2, \sqrt{(900.0 / 36.0)}] = 2.8 \text{ ksi}$$

$$\text{Footing } \phi Pns = \phi * As * Fy / A1 = 0.0 \text{ ksi}$$

ACI 22.8.3.2

$$\text{Footing bearing } \phi Pn = \phi Pnc + \phi Pns = 2.8 + 0.0 = 2.8 \text{ ksi} > 0.4 \text{ psi OK}$$

Hooked $L_{dh} = \text{Max} (8 \text{ db}, 6, 0.02 * f_y / (f_c)^{1/2} * \text{Confining} * \text{Location} * \text{Concrete} * \text{db} * \text{ratio})$

ACI 25.4.3

$$L_{dh} = \text{Max} (8 \text{ db}, 6, 0.02 * 60.0 * 1000 / (2500)^{1/2} * 1.0 * 0.7 * 0.0 * 0.75 * 0.06) = 6.0 \text{ in}$$

Ld provided = Dowel length = $3.00 * 12 = 36.0 \text{ in} > 12.0 \text{ in OK}$

Ldh provided = Footing thickness - Cover = $8.00 - 3.0 = 5.0 \text{ in} < 6.0 \text{ in NG}$

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

$$\text{X-Edge} = \text{Length} / 2 - \text{Offset} - \text{Col} / 2 = 2.50 * 12 / 2 - 0.0 - 6.0 / 2 = 12.0 \text{ in} \quad \alpha_{sx} = 10$$

$$\text{Z-Edge} = \text{Width} / 2 - \text{Offset} - \text{Col} / 2 = 2.50 * 12 / 2 - 0.0 - 6.0 / 2 = 12.0 \text{ in} \quad \alpha_{sz} = 10$$

$$\alpha_s = \alpha_{sx} + \alpha_{sz} = 10 + 10 = 20 \quad \text{Col type} = \text{Corner} \quad \beta = L / W = 6.0 / 6.0 = 1.00$$

ACI 22.6.5.2

$$\text{Perimeter } b_o = \alpha_{sz} / 10 * (L + d / 2 + \text{X-Edge}) + \alpha_{sx} / 10 * (W + d / 2 + \text{Z-Edge})$$

ACI 22.6.4.2

$$b_o = 10 / 10 * (6.0 + 8.0 / 2 + 12.0) + 10 / 10 * (6.0 + 8.0 / 2 + 12.0) = 44.0 \text{ in}$$

$$\text{Area } A_{bo} = (L + d / 2 + \text{X-Edge}) * (W + d / 2 + \text{Z-Edge}) = (6.0 + 8.0 / 2 + 12.0) * (6.0 + 8.0 / 2 + 12.0) = 484.0 \text{ in}^2$$

Use Plain Concrete Shear Strength

$$\phi V_c = \phi * \text{Min} (1 + 2 / \beta, 2) * 4/3 * \sqrt{f_c}$$

ACI 14.5.5.1

$$\phi V_c = 0.60 * \text{Min} (1 + 2 / 1.00, 2) * 4/3 * \sqrt{2500} = 80.0 \text{ psi}$$

Punching force $F = P + \text{Overburden} * A_{bo} - \text{Bearing}$

$$F = 12.7 + 0.07 * 484.0 / 144 - 2.9 = 10.1 \text{ kip}$$

$$b_1 = L + d / 2 + \text{X-Edge} = 6.0 + 8.0 / 2 + 12.0 = 22.0 \text{ in} \quad b_2 = W + d / 2 + \text{Z-Edge} = 6.0 + 8.0 / 2 + 12.0 = 22.0 \text{ in}$$

$$\gamma_{vx} \text{ factor} = 1 - \frac{1}{1 + (2/3) \sqrt{(b_2 / b_1)}} = 1 - \frac{1}{1 + (2/3) \sqrt{(22.0 / 22.0)}} = 0.40$$

ACI Eq. (8.4.4.2.2)

$$\gamma_{vz} \text{ factor} = 1 - \frac{1}{1 + (2/3) \sqrt{(b_1 / b_2)}} = 1 - \frac{1}{1 + (2/3) \sqrt{(22.0 / 22.0)}} = 0.40$$

ACI Eq. (8.4.2.3.2)

$$X_{2z} = b_1^2 / 2 / (b_1 + b_2) = 22.0^2 / 2 / (22.0 + 22.0) = 5.5 \text{ in} \quad X_{2x} = b_2^2 / 2 / (b_2 + b_1) = 5.5 \text{ in}$$

$$J_{cz} = b_1 * d^3 / 12 + b_1^3 * d / 12 + b_1 * d * (b_1 / 2 - X_{2z})^2 + b_2 * d * X_{2z}^2$$

ACI R8.4.4.2.3

$$J_{cz} = 22.0 * 8.0^3 / 12 + 22.0^3 * 8.0 / 12 + 22.0 * 8.0 * (22.0 / 2 - 5.5)^2 + 22.0 * 8.0 * 5.5^2 = 18685 \text{ in}^4$$

$$J_{cx} = b_2 * d^3 / 12 + b_2^3 * d / 12 + b_2 * d * (b_2 / 2 - X_{2x})^2 + b_1 * d * X_{2x}^2$$

ACI R8.4.4.2.3

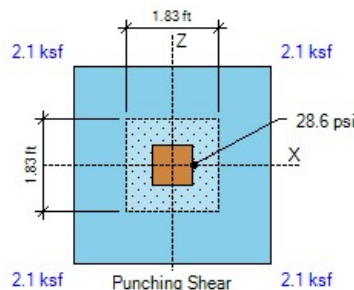
$$J_{cx} = 22.0 * 8.0^3 / 12 + 22.0^3 * 8.0 / 12 + 22.0 * 8.0 * (22.0 / 2 - 5.5)^2 + 22.0 * 8.0 * 5.5^2 = 18685 \text{ in}^4$$

$$\text{Stress due to } P = F / (b_o * d) * 1000 = 10.1 / (44.0 * 8.0) * 1000 = 28.6 \text{ psi}$$

$$\text{Stress due to } M_x = \gamma_{vx} * X\text{-OTM} * X_{2x} / J_{cx} = 0.40 * 0.0 * 12 * 5.5 / 18685 * 1000 = 0.0 \text{ psi}$$

$$\text{Stress due to } M_z = \gamma_{vz} * Z\text{-OTM} * X_{2z} / J_{cz} = 0.40 * 0.0 * 12 * 5.5 / 18685 * 1000 = 0.0 \text{ psi}$$

$$\text{Punching stress} = P\text{-stress} + M_x\text{-stress} + M_z\text{-stress} = 28.6 + 0.0 + 0.0 = 28.6 \text{ psi} < 80.0 \text{ psi OK}$$



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

$$\text{Area } A1 = \text{col } L * \text{col } W = 6.0 * 6.0 = 36.0 \text{ in}^2$$

$$Sx = \text{col } W * \text{col } L^2 / 6 = 6.0 * 6.0^2 / 6 = 36.0 \text{ in}^3$$

$$Sz = \text{col } L * \text{col } W^2 / 6 = 6.0 * 6.0^2 / 6 = 36.0 \text{ in}^3$$

$$\text{Bearing } Pbu = P / A1 + Mz / Sx + Mx / Sz = 12.7 / 36.0 + 0.0 * 12 / 36.0 + 0.0 * 12 / 36.0 = 0.4 \text{ ksi}$$

$$\text{Min edge} = \text{Min} (L / 2 - X\text{-offset} - \text{col } L / 2, W / 2 - Z\text{-offset} - \text{col } W / 2)$$

$$\text{Min edge} = \text{Min} (2.50 * 12 / 2 - 0.0 - 6.0 / 2, 2.50 * 12 / 2 - 0.0 - 6.0 / 2) = 12.0 \text{ in}$$

$$\text{Area } A2 = \text{Min} [L * W, (\text{col } L + 2 * \text{Min edge}) * (\text{col } W + 2 * \text{Min edge})]$$

ACI R22.8.3.2

$$A2 = \text{Min} [2.50 * 12 * 2.5 * 12, (6.0 + 2 * 12.0) * (6.0 + 2 * 12.0)] = 900.0 \text{ in}^2$$

$$\text{Footing } \phi Pnc = \phi * 0.85 * f'c * \text{Min} [2, \sqrt{(A2 / A1)}] = 0.65 * 0.85 * 2.5 * \text{Min} [2, \sqrt{(900.0 / 36.0)}] = 2.8 \text{ ksi}$$

$$\text{Footing } \phi Pns = \phi * As * Fy / A1 = 0.0 \text{ ksi}$$

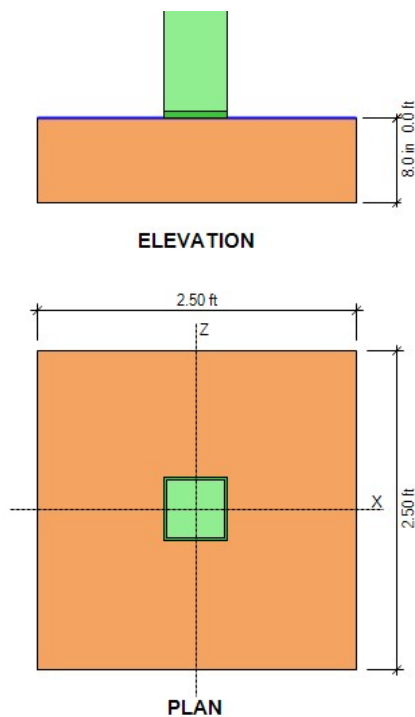
ACI 22.8.3.2

$$\text{Footing bearing } \phi Pn = \phi Pnc + \phi Pns = 2.8 + 0.0 = 2.8 \text{ ksi} > 0.4 \text{ psi OK}$$

DESIGN CODES

Concrete Design ACI 318-14

Load Combinations ASCE 7-10/16



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

$$\text{Arm} = 0.00 + 8.0 / 12 = 0.67 \text{ ft}$$

$$\text{Moment} = 0.0 * 0.67 = 0.0 \text{ k-ft}$$

- Passive Force = 0.0 kip

$$\text{Arm} = 0.27 \text{ ft}$$

$$\text{Moment} = 0.0 \text{ k-ft}$$

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

- Resisting about X-X

- Footing weight = $0.6 * W * L * \text{Thick} * \text{Density} = 0.6 * 3.50 * 3.50 * 8.0 / 12 * 0.15 = 0.7$ kip

$$\text{Arm} = W / 2 = 3.50 / 2 = 1.75 \text{ ft}$$

$$\text{Moment} = 0.7 * 1.75 = 1.3 \text{ k-ft}$$

- Pedestal weight = $0.6 * W * L * H * \text{Density} = 0.6 * 6.0 / 12 * 6.0 / 12 * 0.0 * 0.15 = 0.0$ kip

$$\text{Arm} = W / 2 - \text{Offset} = 3.50 / 2 - 0.0 / 12 = 1.75 \text{ ft}$$

$$\text{Moment} = 0.0 * 1.75 = 0.0 \text{ k-ft}$$

- Soil cover = $0.6 * W * L * SC * \text{Density} = 0.6 * (3.50 * 3.50 - 6.0 / 12 * 6.0 / 12) * 0.0 * 110 = 0.0$ kip

$$\text{Arm} = W / 2 = 3.50 / 2 = 1.75 \text{ ft}$$

$$\text{Moment} = 0.0 * 1.75 = 0.0 \text{ k-ft}$$

- Buoyancy = $0.6 * W * L * \gamma * (SC + \text{Thick} - WT) = 0.6 * 3.50 * 3.50 * 62 * (0.67) = -0.3$ kip

$$\text{Arm} = W / 2 = 3.50 / 2 = 1.75 \text{ ft}$$

$$\text{Moment} = 0.3 * 1.75 = -0.5 \text{ k-ft}$$

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

$$\text{Arm} = W / 2 - \text{Offset} = 3.50 / 2 - 0.0 / 12 = 1.75 \text{ ft}$$

$$\text{Moment} = 2.8 * 1.75 = 4.8 \text{ k-ft}$$

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

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

- Overturning about Z-Z

$$\text{- Moment } M_z = 0.6 * 0.0 + 0.6 * 0.0 = 0.0 \text{ k-ft}$$

$$\text{- Shear Force } V_x = 0.6 * 0.0 + 0.6 * 0.0 = 0.0 \text{ kip}$$

$$\text{Arm} = 0.00 + 8.0 / 12 = 0.67 \text{ ft}$$

$$\text{Moment} = 0.0 * 0.67 = 0.0 \text{ k-ft}$$

$$\text{- Passive Force} = 0.0 \text{ kip}$$

$$\text{Arm} = 0.27 \text{ ft}$$

$$\text{Moment} = 0.0 \text{ k-ft}$$

$$\text{- Overturning moment Z-Z} = 0.0 + 0.0 = 0.0 \text{ k-ft}$$

- Resisting about Z-Z

$$\text{- Footing weight} = 0.6 * W * L * Thick * Density = 0.6 * 3.50 * 3.50 * 8.0 / 12 * 0.15 = 0.7 \text{ kip}$$

$$\text{Arm} = L / 2 = 3.50 / 2 = 1.75 \text{ ft}$$

$$\text{Moment} = 0.7 * 1.75 = 1.3 \text{ k-ft}$$

$$\text{- Pedestal weight} = 0.6 * W * L * H * Density = 0.6 * 6.0 / 12 * 6.0 / 12 * 0.0 * 0.15 = 0.0 \text{ kip}$$

$$\text{Arm} = L / 2 - Offset = 3.50 / 2 - 0.0 / 12 = 1.75 \text{ ft}$$

$$\text{Moment} = 0.0 * 1.75 = 0.0 \text{ k-ft}$$

$$\text{- Soil cover} = 0.6 * W * L * SC * Density = 0.6 * (3.50 * 3.50 - 6.0 / 12 * 6.0 / 12) * 0.0 * 110 = 0.0 \text{ kip}$$

$$\text{Arm} = L / 2 = 3.50 / 2 = 1.75 \text{ ft}$$

$$\text{Moment} = 0.0 * 1.75 = 0.0 \text{ k-ft}$$

$$\text{- Buoyancy} = 0.6 * W * L * \gamma * (SC + Thick - WT) = 0.6 * 3.50 * 3.50 * 62 * (0.67) = -0.3 \text{ kip}$$

$$\text{Arm} = L / 2 = 3.50 / 2 = 1.75 \text{ ft}$$

$$\text{Moment} = 0.3 * 1.75 = -0.5 \text{ k-ft}$$

$$\text{- Axial force } P = 0.6 * 4.6 + 0.6 * 0.0 = 2.8 \text{ kip}$$

$$\text{Arm} = L / 2 - Offset = 3.50 / 2 - 0.0 / 12 = 1.75 \text{ ft}$$

$$\text{Moment} = 2.8 * 1.75 = 4.8 \text{ k-ft}$$

$$\text{- Resisting moment Z-Z} = 1.3 + 0.0 + 0.0 + 4.8 + -0.5 = 5.6 \text{ k-ft}$$

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

SOIL BEARING PRESSURES (Comb: D+L)

$$\text{Overturning moment X-X} = 0.0 + 0.0 = 0.0 \text{ k-ft}$$

$$\text{Resisting moment X-X} = 2.1 + 0.0 + 0.0 + -0.9 + 32.7 = 34.0 \text{ k-ft}$$

$$\text{Overturning moment Z-Z} = 0.0 + 0.0 = 0.0 \text{ k-ft}$$

$$\text{Resisting moment Z-Z} = 2.1 + 0.0 + 0.0 + -0.9 + 32.7 = 34.0 \text{ k-ft}$$

$$\text{Resisting force} = \text{Footing} + \text{Pedestal} + \text{Soil} - \text{Buoyancy} + P = 1.2 + 0.0 + 0.0 - 0.5 + 18.7 = 19.4 \text{ kip}$$

X-coordinate of resultant from maximum bearing corner:

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

Z-coordinate of resultant from maximum bearing corner:

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

$$X\text{-ecc} = \text{Length} / 2 - X_p = 3.50 / 2 - 1.75 = 0.00 \text{ ft}$$

$$Z\text{-ecc} = \text{Width} / 2 - Z_p = 3.50 / 2 - 1.75 = 0.00 \text{ ft}$$

$$\text{Area} = \text{Width} * \text{Length} = 3.50 * 3.50 = 12.3 \text{ ft}^2$$

$$S_x = \text{Length} * \text{Width}^2 / 6 = 3.50 * 3.50^2 / 6 = 7.1 \text{ ft}^3$$

$$S_z = \text{Width} * \text{Length}^2 / 6 = 3.50 * 3.50^2 / 6 = 7.1 \text{ ft}^3$$

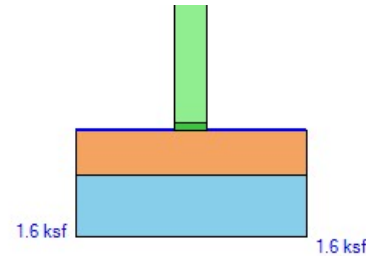
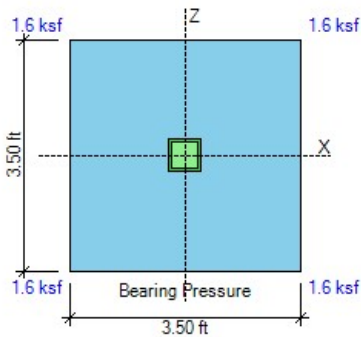
- Footing is in full bearing. Soil pressures are as follows:

$$P1 = P * (1/A + Z\text{-ecc} / S_x + X\text{-ecc} / S_z) = 19.4 * (1 / 12.3 + 0.00 / 7.1 + 0.00 / 7.1) = 1.58 \text{ ksf}$$

$$P2 = P * (1/A - Z\text{-ecc} / S_x + X\text{-ecc} / S_z) = 19.4 * (1 / 12.3 - 0.00 / 7.1 + 0.00 / 7.1) = 1.58 \text{ ksf}$$

$$P3 = P * (1/A - Z\text{-ecc} / S_x - X\text{-ecc} / S_z) = 19.4 * (1 / 12.3 - 0.00 / 7.1 - 0.00 / 7.1) = 1.58 \text{ ksf}$$

$$P4 = P * (1/A + Z\text{-ecc} / S_x - X\text{-ecc} / S_z) = 19.4 * (1 / 12.3 + 0.00 / 7.1 - 0.00 / 7.1) = 1.58 \text{ ksf}$$



SLIDING CALCULATIONS (Comb: 0.6D+0.6W)

Internal friction angle = 28.0 deg

Passive coefficient $k_p = 4.33$ (per Coulomb)Pressure at mid-depth = $k_p \cdot \text{Density} \cdot (\text{Cover} + \text{Thick} / 2) = 4.33 \cdot 110 \cdot (0.00 + 8.0 / 12 / 2) = 0.16 \text{ ksf}$ X-Passive force = $\text{Pressure} \cdot \text{Thick} \cdot \text{Width} = 0.16 \cdot 8.0 / 12 \cdot 3.50 = 0.4 \text{ kip}$ Z-Passive force = $\text{Pressure} \cdot \text{Thick} \cdot \text{Length} = 0.16 \cdot 8.0 / 12 \cdot 3.50 = 0.4 \text{ kip}$ Friction force = $\text{Resisting force} \cdot \text{Friction coeff.} = \text{Max}(0, 3.2 \cdot 0.35) = 1.1 \text{ kip}$

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

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

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

UPLIFT CALCULATIONS (Comb: 0.6D+0.6W)

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

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

Concrete $f'_c = 2.5 \text{ ksi}$ Steel $f_y = 40.0 \text{ ksi}$

Soil density = 110 pcf

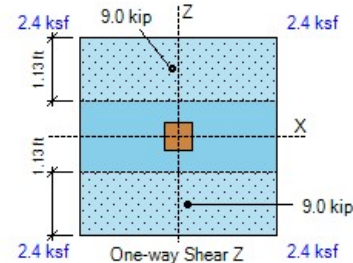
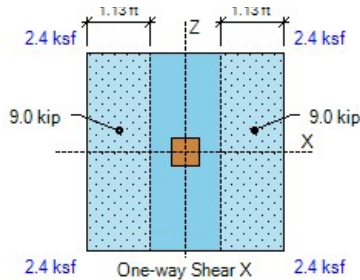
d Top X-dir = $\text{Thick} - \text{Cover} - \text{X-diameter} / 2 = 8.0 - 2.0 - 0.8 / 2 = 5.6 \text{ in}$ d Top Z-dir = $\text{Thick} - \text{Cover} - \text{X-diameter} - \text{Z-diameter} / 2 = 8.0 - 2.0 - 0.8 - 0.8 / 2 = 4.9 \text{ in}$ d Bot X-dir = $\text{Thick} - \text{Cover} - \text{X-diameter} / 2 = 8.0 - 3.0 - 0.5 / 2 = 4.8 \text{ in}$ d Bot Z-dir = $\text{Thick} - \text{Cover} - \text{X-diameter} - \text{Z-diameter} / 2 = 8.0 - 3.0 - 0.5 - 0.5 / 2 = 4.3 \text{ in}$ $\phi V_{cx} = 2 \cdot \phi \cdot \sqrt{f'_c} \cdot \text{Width} \cdot d / 1000 = 2 \cdot 0.75 \cdot \sqrt{(2500)} \cdot 3.5 \cdot 12 \cdot 4.8 / 1000 = 15.0 \text{ kip}$

ACI Eq. (22.5.5.1)

 $\phi V_{cz} = 2 \cdot \phi \cdot \sqrt{f'_c} \cdot \text{Length} \cdot d / 1000 = 2 \cdot 0.75 \cdot \sqrt{(2500)} \cdot 3.5 \cdot 12 \cdot 4.3 / 1000 = 13.4 \text{ kip}$

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

One-way shear V_{ux} (- Side) = 9.0 kip < 15.0 kip OKOne-way shear V_{ux} (+ Side) = 9.0 kip < 15.0 kip OKOne-way shear V_{uz} (- Side) = 9.0 kip < 13.4 kip OKOne-way shear V_{uz} (+ Side) = 9.0 kip < 13.4 kip OK



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

$$\text{Plain } \phi M_{nx} = 5 * \phi * \sqrt{f_c} * L * \text{Thick}^2 / 6 = 5 * 0.60 * \sqrt{(2500)} * 3.50 * 8.0^2 / 6 / 1000 = 1.5 \text{ k-ft}$$

ACI Eq. (14.5.2.1a)

$$\text{Plain } \phi M_{nz} = 5 * \phi * \sqrt{f_c} * W * \text{Thick}^2 / 6 = 5 * 0.60 * \sqrt{(2500)} * 3.50 * 8.0^2 / 6 / 1000 = 1.5 \text{ k-ft}$$

- 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:

$$\text{Top moment -Mux (- Side)} = 0.0 \text{ k-ft} < 5.6 \text{ k-ft OK}$$

$$\text{Top moment -Mux (+ Side)} = 0.0 \text{ k-ft} < 5.6 \text{ k-ft OK}$$

$$\text{Top moment -Muz (- Side)} = 0.0 \text{ k-ft} < 5.6 \text{ k-ft OK}$$

$$\text{Top moment -Muz (+ Side)} = 0.0 \text{ k-ft} < 5.6 \text{ k-ft OK}$$

- Bottom Bars

$$\text{Use 5 \#4 Z-Bars } \rho = A_s / b d = 1.0 / (3.50 * 12 * 4.3) = 0.0056$$

$$q = 0.0056 * 40 / 2.5 = 0.090$$

$$\text{Use 5 \#4 X-Bars } \rho = A_s / b d = 1.0 / (3.50 * 12 * 4.8) = 0.0050$$

$$q = 0.0050 * 40 / 2.5 = 0.080$$

$$\beta = L / W = 3.50 / 3.50 = 1.00 \quad \gamma_s = 2 * \beta / (\beta + 1) = 2 * 1.00 / (1.00 + 1) = 1.00$$

ACI 13.3.3.3

$$\text{Bending strength } \phi M_n = \phi * b * d^2 * f_c * q * (1 - 0.59 * q)$$

ACI 22.2.2

$$\phi M_{nx} = 0.90 * 3.50 * 12 * 4.3^2 * 2.5 * 0.090 * (1 - 0.59 * 0.090) = 12.1 \text{ k-ft}$$

$$\phi M_{nz} = 0.90 * 3.50 * 12 * 4.8^2 * 2.5 * 0.080 / 1.00 * (1 - 0.59 * 0.080 / 1.00) = 13.6 \text{ k-ft}$$

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

$$\text{Bottom moment Mux (- Side)} = 9.0 \text{ k-ft} < 12.1 \text{ k-ft OK} \quad \text{ratio} = 0.75$$

$$\text{Bottom moment Mux (+ Side)} = 9.1 \text{ k-ft} < 12.1 \text{ k-ft OK} \quad \text{ratio} = 0.75$$

$$\text{Bottom moment Muz (- Side)} = 9.0 \text{ k-ft} < 13.6 \text{ k-ft OK} \quad \text{ratio} = 0.67$$

$$\text{Bottom moment Muz (+ Side)} = 9.1 \text{ k-ft} < 13.6 \text{ k-ft OK} \quad \text{ratio} = 0.67$$

$$X\text{-As min} = 0.0018 * \text{Width} * \text{Thick} = 0.0018 * 3.50 * 12 * 8.0 = 0.6 \text{ in}^2 < 1.0 \text{ in}^2 \text{ OK}$$

ACI 8.6.1.1

$$Z\text{-As min} = 0.0018 * \text{Length} * \text{Thick} = 0.0018 * 3.50 * 12 * 8.0 = 0.6 \text{ in}^2 < 1.0 \text{ in}^2 \text{ OK}$$

ACI 8.6.1.1

$$X\text{-As max for 0.005 tension strain} = 3.20 \text{ in}^2 > 1.00 \text{ in}^2 \text{ OK}$$

ACI 21.2.2

$$Z\text{-As max for 0.005 tension strain} = 3.20 \text{ in}^2 > 1.00 \text{ in}^2 \text{ OK}$$

ACI 21.2.2

$$X\text{-Cover factor} = \text{Min} (2.5, (\text{Cover} + db / 2, \text{Spacing} / 2) / db) = \text{Min} (2.5, (3.0 + 0.50 / 2, 9.0 / 2) / 0.50) = 2.5$$

$$\text{Straight } X\text{-Ld} = \text{Max} (12.0, 3 / 40 * f_y / (f_c)^{1/2} * \text{Grade} * \text{Size} * \text{Casting} / \text{Cover} * db * \text{ratio})$$

ACI Eq. (25.4.2.3a)

$$X\text{-Ld} = \text{Max} (12.0, 3 / 40 * 40.0 * 1000 / (2500)^{1/2} * 1.0 * 0.8 * 1.0 / 2.5 * 0.50 * 0.67) = 12.0 \text{ in}$$

$$\text{Hooked } X\text{-Ldh} = \text{Max} (8 db, 6, 0.02 * f_y / (f_c)^{1/2} * \text{Confining} * \text{Location} * \text{Concrete} * db * \text{ratio}) =$$

ACI 25.4.3

$$X\text{-Ldh} = \text{Max} (8 db, 6, 0.02 * 40.0 * 1000 / (2500)^{1/2} * 1.0 * 0.7 * 0.0 * 0.50 * 0.67) = 6.0 \text{ in}$$

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

$$+X \text{ Ld provided} = (\text{Length} - \text{Col}) / 2 - \text{Offset} - \text{Cover} = 3.50 * 12 / 2 - 0.0 - 6.0 / 2 - 2.5 = 15.5 \text{ in} > 12.0 \text{ in OK} \quad 4 \text{ of } 7$$

$$Z\text{-Cover factor} = \text{Min} (2.5, (\text{Cover} + db / 2, \text{Spacing} / 2) / db) = \text{Min} (2.5, (3.0 + 0.50 / 2, 9.0 / 2) / 0.50) = 2.5$$

$$\text{Straight } Z\text{-Ld} = \text{Max} (12.0, 3 / 40 * f_y / (f_c)^{1/2} * \text{Grade} * \text{Size} * \text{Casting} / \text{Cover} * db * \text{ratio})$$

ACI Eq. (25.4.2.3a)

$$Z\text{-Ld} = \text{Max} (12.0, 3 / 40 * 40.0 * 1000 / (2500)^{1/2} * 1.0 * 0.8 * 1.0 / 2.5 * 0.50 * 0.67) = 12.0 \text{ in}$$

$$\text{Hooked } Z\text{-Ldh} = \text{Max} (8 \text{ db}, 6, 0.02 * f_y / (f_c)^{1/2} * \text{Confining} * \text{Location} * \text{Concrete} * db * \text{ratio}) =$$

ACI 25.4.3

$$Z\text{-Ldh} = \text{Max} (8 \text{ db}, 6, 0.02 * 40.0 * 1000 / (2500)^{1/2} * 1.0 * 0.7 * 0.0 * 0.50 * 0.75) = 6.0 \text{ in}$$

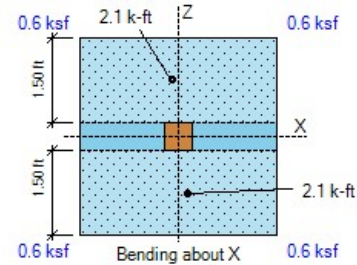
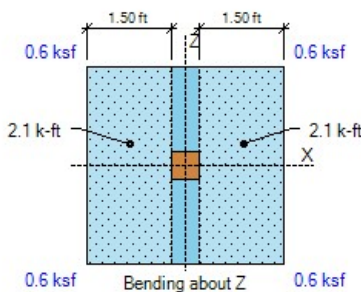
$$-Z \text{ Ld provided} = (\text{Width} - \text{Col}) / 2 + \text{Offset} - \text{Cover} = 3.50 * 12 / 2 + 0.0 - 6.0 / 2 - 2.5 = 15.5 \text{ in} > 12.0 \text{ in OK}$$

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

$$X\text{-bar spacing} = 9.0 \text{ in} < \text{Min} (3 * t, 18.0) = 18.0 \text{ in OK}$$

ACI 7.7.2.3

$$Z\text{-bar spacing} = 9.0 \text{ in} < \text{Min} (3 * t, 18.0) = 18.0 \text{ in OK}$$



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

$$\text{Area } A1 = \text{col } L * \text{col } W = 6.0 * 6.0 = 36.0 \text{ in}^2$$

$$Sx = \text{col } W * \text{col } L^2 / 6 = 6.0 * 6.0^2 / 6 = 36.0 \text{ in}^3$$

$$Sz = \text{col } L * \text{col } W^2 / 6 = 6.0 * 6.0^2 / 6 = 36.0 \text{ in}^3$$

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

$$\text{Min edge} = \text{Min} (L / 2 - X\text{-offset} - \text{col } L / 2, W / 2 - Z\text{-offset} - \text{col } W / 2)$$

$$\text{Min edge} = \text{Min} (3.50 * 12 / 2 - 0.0 - 6.0 / 2, 3.50 * 12 / 2 - 0.0 - 6.0 / 2) = 18.0 \text{ in}$$

$$\text{Area } A2 = \text{Min} [L * W, (\text{col } L + 2 * \text{Min edge}) * (\text{col } W + 2 * \text{Min edge})]$$

ACI R22.8.3.2

$$A2 = \text{Min} [3.50 * 12 * 3.5 * 12, (6.0 + 2 * 18.0) * (6.0 + 2 * 18.0)] = 1764.0 \text{ in}^2$$

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

$$\text{Footing } \phi Pns = \phi * As * Fy / A1 = 0.0 \text{ ksi}$$

ACI 22.8.3.2

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

Hooked $L_{dh} = \text{Max} (8 \text{ db}, 6, 0.02 * f_y / (f_c)^{1/2} * \text{Confining} * \text{Location} * \text{Concrete} * \text{db} * \text{ratio})$

ACI 25.4.3

$$L_{dh} = \text{Max} (8 \text{ db}, 6, 0.02 * 60.0 * 1000 / (2500)^{1/2} * 1.0 * 0.7 * 0.0 * 0.75 * 0.14) = 6.0 \text{ in}$$

Ld provided = Dowel length = $3.00 * 12 = 36.0 \text{ in} > 24.3 \text{ in OK}$

Ldh provided = Footing thickness - Cover = $8.00 - 3.0 = 5.0 \text{ in} < 6.0 \text{ in NG}$

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

$$\text{X-Edge} = d/2 = 4.5 / 2 = 2.3 \text{ in} \quad \text{asx} = 20$$

$$\text{Z-Edge} = d/2 = 4.5 / 2 = 2.3 \text{ in} \quad \text{asz} = 20$$

$$\text{as} = \text{asx} + \text{asz} = 20 + 20 = 40 \quad \text{Col type} = \text{Interior} \quad \beta = L / W = 6.0 / 6.0 = 1.00$$

ACI 22.6.5.2

$$\text{Perimeter } b_o = \text{asx} / 10 * (L + d/2 + \text{X-Edge}) + \text{asx} / 10 * (W + d/2 + \text{Z-Edge})$$

ACI 22.6.4.2

$$b_o = 20 / 10 * (6.0 + 4.5 / 2 + 2.3) + 20 / 10 * (6.0 + 4.5 / 2 + 2.3) = 42.0 \text{ in}$$

$$\text{Area } A_{bo} = (L + d/2 + \text{X-Edge}) * (W + d/2 + \text{Z-Edge}) = (6.0 + 4.5 / 2 + 2.3) * (6.0 + 4.5 / 2 + 2.3) = 110.3 \text{ in}^2$$

$$\phi V_c = \phi * \text{Min} (2 + 4 / \beta, \text{as} * d / b_o + 2, 4) * \sqrt{f_c}$$

ACI 22.6.5.2

$$\phi V_c = 0.75 * \text{Min} (2 + 4 / 1.00, 40 * 4.5 / 42.0 + 2, 4) * \sqrt{2500} = 150.0 \text{ psi}$$

Punching force $F = P + \text{Overburden} * A_{bo} - \text{Bearing}$

$$F = 28.1 + 0.07 * 110.3 / 144 - 1.8 = 26.3 \text{ kip}$$

$$b1 = L + d/2 + \text{X-Edge} = 6.0 + 4.5 / 2 + 2.3 = 10.5 \text{ in} \quad b2 = W + d/2 + \text{Z-Edge} = 6.0 + 4.5 / 2 + 2.3 = 10.5 \text{ in}$$

$$\gamma_{vx} \text{ factor} = 1 - \frac{1}{1 + (2/3) \sqrt{b2/b1}} = 1 - \frac{1}{1 + (2/3) \sqrt{10.5/10.5}} = 0.40$$

ACI Eq. (8.4.4.2.2)

$$\gamma_{vz} \text{ factor} = 1 - \frac{1}{1 + (2/3) \sqrt{b1/b2}} = 1 - \frac{1}{1 + (2/3) \sqrt{10.5/10.5}} = 0.40$$

ACI Eq. (8.4.2.3.2)

$$X2z = b1/2 = 10.5/2 = 5.3 \text{ in} \quad X2x = b2/2 = 10.5/2 = 5.3 \text{ in}$$

$$J_{cz} = b1 * d^3 / 6 + b1^3 * d / 6 + b1^2 * b2 * d / 2$$

ACI R8.4.4.2.3

$$J_{cz} = 10.5 * 4.5^3 / 6 + 10.5^3 * 4.5 / 6 + 10.5^2 * 10.5 * 4.5 / 2 = 3632 \text{ in}^4$$

$$J_{cx} = b2 * d^3 / 6 + b2^3 * d / 6 + b2^2 * b1 * d / 2$$

ACI R8.4.4.2.3

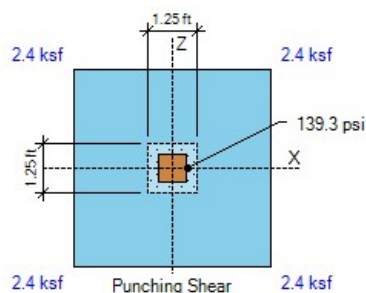
$$J_{cx} = 10.5 * 4.5^3 / 6 + 10.5^3 * 4.5 / 6 + 10.5^2 * 10.5 * 4.5 / 2 = 3632 \text{ in}^4$$

$$\text{Stress due to } P = F / (b_o * d) * 1000 = 26.3 / (42.0 * 4.5) * 1000 = 139.3 \text{ psi}$$

$$\text{Stress due to } M_x = \gamma_{vx} * X\text{-OTM} * X2x / J_{cx} = 0.40 * 0.0 * 12 * 5.3 / 3632 * 1000 = 0.0 \text{ psi}$$

$$\text{Stress due to } M_z = \gamma_{vz} * Z\text{-OTM} * X2z / J_{cz} = 0.40 * 0.0 * 12 * 5.3 / 3632 * 1000 = 0.0 \text{ psi}$$

$$\text{Punching stress} = P\text{-stress} + M_x\text{-stress} + M_z\text{-stress} = 139.3 + 0.0 + 0.0 = 139.3 \text{ psi} < 150.0 \text{ psi OK}$$



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

$$\text{Area } A1 = \text{col } L * \text{col } W = 6.0 * 6.0 = 36.0 \text{ in}^2$$

$$Sx = \text{col } W * \text{col } L^2 / 6 = 6.0 * 6.0^2 / 6 = 36.0 \text{ in}^3$$

$$Sz = \text{col } L * \text{col } W^2 / 6 = 6.0 * 6.0^2 / 6 = 36.0 \text{ in}^3$$

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

$$\text{Min edge} = \text{Min} (L / 2 - X\text{-offset} - \text{col } L / 2, W / 2 - Z\text{-offset} - \text{col } W / 2)$$

$$\text{Min edge} = \text{Min} (3.50 * 12 / 2 - 0.0 - 6.0 / 2, 3.50 * 12 / 2 - 0.0 - 6.0 / 2) = 18.0 \text{ in}$$

$$\text{Area } A2 = \text{Min} [L * W, (\text{col } L + 2 * \text{Min edge}) * (\text{col } W + 2 * \text{Min edge})]$$

ACI R22.8.3.2

$$A2 = \text{Min} [3.50 * 12 * 3.50 * 12, (6.0 + 2 * 18.0) * (6.0 + 2 * 18.0)] = 1764.0 \text{ in}^2$$

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

$$\text{Footing } \phi Pns = \phi * As * Fy / A1 = 0.0 \text{ ksi}$$

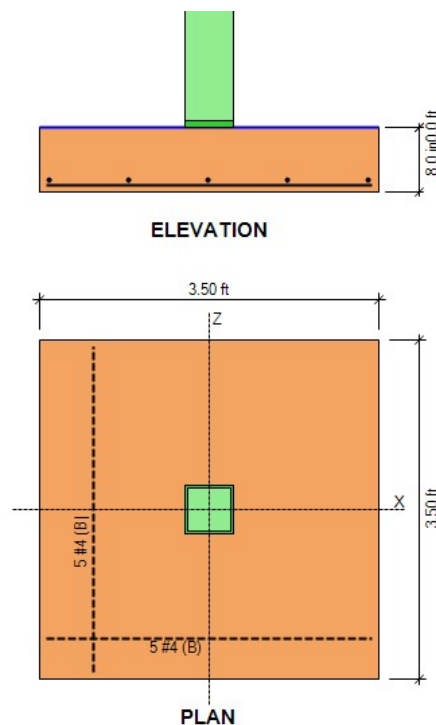
ACI 22.8.3.2

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

DESIGN CODES

Concrete Design ACI 318-14

Load Combinations ASCE 7-10/16



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

$$\text{Arm} = 0.00 + 8.0 / 12 = 0.67 \text{ ft}$$

$$\text{Moment} = 0.0 * 0.67 = 0.0 \text{ k-ft}$$

- Passive Force = 0.0 kip

$$\text{Arm} = 0.27 \text{ ft}$$

$$\text{Moment} = 0.0 \text{ k-ft}$$

- 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

$$\text{Arm} = W / 2 = 2.50 / 2 = 1.25 \text{ ft}$$

$$\text{Moment} = 0.4 * 1.25 = 0.5 \text{ k-ft}$$

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

$$\text{Arm} = W / 2 - Offset = 2.50 / 2 - 0.0 / 12 = 1.25 \text{ ft}$$

$$\text{Moment} = 0.0 * 1.25 = 0.0 \text{ k-ft}$$

- 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

$$\text{Arm} = W / 2 = 2.50 / 2 = 1.25 \text{ ft}$$

$$\text{Moment} = 0.0 * 1.25 = 0.0 \text{ k-ft}$$

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

$$\text{Arm} = W / 2 = 2.50 / 2 = 1.25 \text{ ft}$$

$$\text{Moment} = 0.2 * 1.25 = -0.2 \text{ k-ft}$$

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

$$\text{Arm} = W / 2 - Offset = 2.50 / 2 - 0.0 / 12 = 1.25 \text{ ft}$$

$$\text{Moment} = 2.5 * 1.25 = 3.1 \text{ k-ft}$$

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

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

- Overturning about Z-Z

$$\text{- Moment } M_z = 0.6 * 0.0 + 0.6 * 0.0 = 0.0 \text{ k-ft}$$

$$\text{- Shear Force } V_x = 0.6 * 0.0 + 0.6 * 0.0 = 0.0 \text{ kip}$$

$$\text{Arm} = 0.00 + 8.0 / 12 = 0.67 \text{ ft}$$

$$\text{Moment} = 0.0 * 0.67 = 0.0 \text{ k-ft}$$

$$\text{- Passive Force} = 0.0 \text{ kip}$$

$$\text{Arm} = 0.27 \text{ ft}$$

$$\text{Moment} = 0.0 \text{ k-ft}$$

$$\text{- Overturning moment Z-Z} = 0.0 + 0.0 = 0.0 \text{ k-ft}$$

- Resisting about Z-Z

$$\text{- Footing weight} = 0.6 * W * L * Thick * Density = 0.6 * 2.50 * 2.50 * 8.0 / 12 * 0.15 = 0.4 \text{ kip}$$

$$\text{Arm} = L / 2 = 2.50 / 2 = 1.25 \text{ ft}$$

$$\text{Moment} = 0.4 * 1.25 = 0.5 \text{ k-ft}$$

$$\text{- Pedestal weight} = 0.6 * W * L * H * Density = 0.6 * 6.0 / 12 * 6.0 / 12 * 0.0 * 0.15 = 0.0 \text{ kip}$$

$$\text{Arm} = L / 2 - Offset = 2.50 / 2 - 0.0 / 12 = 1.25 \text{ ft}$$

$$\text{Moment} = 0.0 * 1.25 = 0.0 \text{ k-ft}$$

$$\text{- Soil cover} = 0.6 * W * L * SC * Density = 0.6 * (2.50 * 2.50 - 6.0 / 12 * 6.0 / 12) * 0.0 * 110 = 0.0 \text{ kip}$$

$$\text{Arm} = L / 2 = 2.50 / 2 = 1.25 \text{ ft}$$

$$\text{Moment} = 0.0 * 1.25 = 0.0 \text{ k-ft}$$

$$\text{- Buoyancy} = 0.6 * W * L * \gamma * (SC + Thick - WT) = 0.6 * 2.50 * 2.50 * 62 * (0.67) = -0.2 \text{ kip}$$

$$\text{Arm} = L / 2 = 2.50 / 2 = 1.25 \text{ ft}$$

$$\text{Moment} = 0.2 * 1.25 = -0.2 \text{ k-ft}$$

$$\text{- Axial force } P = 0.6 * 4.1 + 0.6 * 0.0 = 2.5 \text{ kip}$$

$$\text{Arm} = L / 2 - Offset = 2.50 / 2 - 0.0 / 12 = 1.25 \text{ ft}$$

$$\text{Moment} = 2.5 * 1.25 = 3.1 \text{ k-ft}$$

$$\text{- Resisting moment Z-Z} = 0.5 + 0.0 + 0.0 + 3.1 + -0.2 = 3.3 \text{ k-ft}$$

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

SOIL BEARING PRESSURES (Comb: D+L)

$$\text{Overturning moment X-X} = 0.0 + 0.0 = 0.0 \text{ k-ft}$$

$$\text{Resisting moment X-X} = 0.8 + 0.0 + 0.0 + -0.3 + 11.6 = 12.1 \text{ k-ft}$$

$$\text{Overturning moment Z-Z} = 0.0 + 0.0 = 0.0 \text{ k-ft}$$

$$\text{Resisting moment Z-Z} = 0.8 + 0.0 + 0.0 + -0.3 + 11.6 = 12.1 \text{ k-ft}$$

$$\text{Resisting force} = \text{Footing} + \text{Pedestal} + \text{Soil} - \text{Buoyancy} + P = 0.6 + 0.0 + 0.0 - 0.3 + 9.3 = 9.7 \text{ kip}$$

X-coordinate of resultant from maximum bearing corner:

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

Z-coordinate of resultant from maximum bearing corner:

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

$$X\text{-ecc} = \text{Length} / 2 - X_p = 2.50 / 2 - 1.25 = 0.00 \text{ ft}$$

$$Z\text{-ecc} = \text{Width} / 2 - Z_p = 2.50 / 2 - 1.25 = 0.00 \text{ ft}$$

$$\text{Area} = \text{Width} * \text{Length} = 2.50 * 2.50 = 6.3 \text{ ft}^2$$

$$S_x = \text{Length} * \text{Width}^2 / 6 = 2.50 * 2.50^2 / 6 = 2.6 \text{ ft}^3$$

$$S_z = \text{Width} * \text{Length}^2 / 6 = 2.50 * 2.50^2 / 6 = 2.6 \text{ ft}^3$$

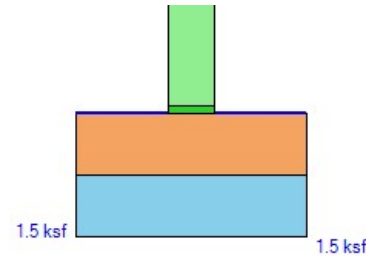
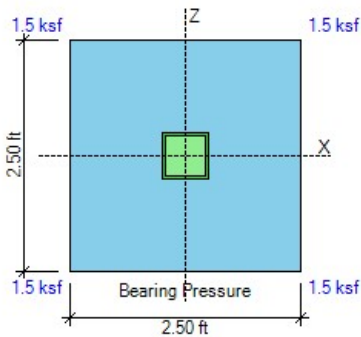
- Footing is in full bearing. Soil pressures are as follows:

$$P1 = P * (1/A + Z\text{-ecc} / S_x + X\text{-ecc} / S_z) = 9.7 * (1/6.3 + 0.00 / 2.6 + 0.00 / 2.6) = 1.55 \text{ ksf}$$

$$P2 = P * (1/A - Z\text{-ecc} / S_x + X\text{-ecc} / S_z) = 9.7 * (1/6.3 - 0.00 / 2.6 + 0.00 / 2.6) = 1.55 \text{ ksf}$$

$$P3 = P * (1/A - Z\text{-ecc} / S_x - X\text{-ecc} / S_z) = 9.7 * (1/6.3 - 0.00 / 2.6 - 0.00 / 2.6) = 1.55 \text{ ksf}$$

$$P4 = P * (1/A + Z\text{-ecc} / S_x - X\text{-ecc} / S_z) = 9.7 * (1/6.3 + 0.00 / 2.6 - 0.00 / 2.6) = 1.55 \text{ ksf}$$



SLIDING CALCULATIONS (Comb: 0.6D+0.6W)

Internal friction angle = 28.0 deg

Passive coefficient $k_p = 4.33$ (per Coulomb)Pressure at mid-depth = $k_p \cdot \text{Density} \cdot (\text{Cover} + \text{Thick} / 2) = 4.33 \cdot 110 \cdot (0.00 + 8.0 / 12 / 2) = 0.16$ ksfX-Passive force = $\text{Pressure} \cdot \text{Thick} \cdot \text{Width} = 0.16 \cdot 8.0 / 12 \cdot 2.50 = 0.3$ kipZ-Passive force = $\text{Pressure} \cdot \text{Thick} \cdot \text{Length} = 0.16 \cdot 8.0 / 12 \cdot 2.50 = 0.3$ kipFriction force = $\text{Resisting force} \cdot \text{Friction coeff.} = \text{Max}(0, 2.7 \cdot 0.35) = 0.9$ kip

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

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

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

UPLIFT CALCULATIONS (Comb: 0.6D+0.6W)

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

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

Concrete $f'_c = 2.5$ ksiSteel $f_y = 40.0$ ksi

Soil density = 110 pcf

Use Plain Concrete Shear Strength

$$\phi V_{cx} = \frac{4}{3} \cdot \phi \cdot \sqrt{f'_c} \cdot \text{Width} \cdot t / 1000 = \frac{4}{3} \cdot 0.60 \cdot \sqrt{(2500)} \cdot 2.5 \cdot 12 \cdot 8.0 / 1000 = 9.6 \text{ kip}$$

ACI 14.5.5.1

$$\phi V_{cz} = \frac{4}{3} \cdot \phi \cdot \sqrt{f'_c} \cdot \text{Length} \cdot t / 1000 = \frac{4}{3} \cdot 0.60 \cdot \sqrt{(2500)} \cdot 2.5 \cdot 12 \cdot 8.0 / 1000 = 9.6 \text{ kip}$$

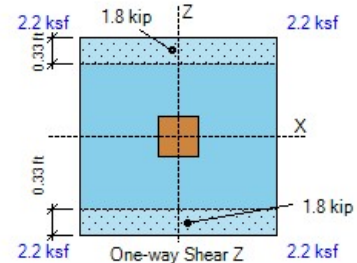
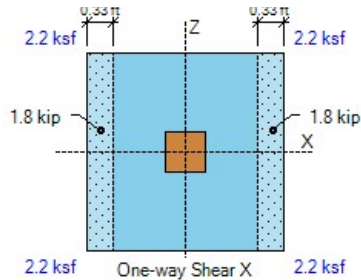
- Shear forces calculated as the volume of the bearing pressures under the effective areas:

$$\text{One-way shear } V_{ux} \text{ (- Side)} = 1.8 \text{ kip} < 9.6 \text{ kip} \quad \text{OK}$$

$$\text{One-way shear } V_{ux} \text{ (+ Side)} = 1.8 \text{ kip} < 9.6 \text{ kip} \quad \text{OK}$$

$$\text{One-way shear } V_{uz} \text{ (- Side)} = 1.8 \text{ kip} < 9.6 \text{ kip} \quad \text{OK}$$

$$\text{One-way shear } V_{uz} \text{ (+ Side)} = 1.8 \text{ kip} < 9.6 \text{ kip} \quad \text{OK}$$



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

$$\text{Plain } \phi M_{nx} = 5 * \phi * \sqrt{f_c} * L * \text{Thick}^2 / 6 = 5 * 0.60 * \sqrt{(2500)} * 2.50 * 8.0^2 / 6 / 1000 = 1.1 \text{ k-ft}$$

ACI Eq. (14.5.2.1a)

$$\text{Plain } \phi M_{nz} = 5 * \phi * \sqrt{f_c} * W * \text{Thick}^2 / 6 = 5 * 0.60 * \sqrt{(2500)} * 2.50 * 8.0^2 / 6 / 1000 = 1.1 \text{ k-ft}$$

- 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:

$$\text{Top moment -Mux (- Side)} = 0.0 \text{ k-ft} < 4.0 \text{ k-ft OK}$$

$$\text{Top moment -Mux (+ Side)} = 0.0 \text{ k-ft} < 4.0 \text{ k-ft OK}$$

$$\text{Top moment -Muz (- Side)} = 0.0 \text{ k-ft} < 4.0 \text{ k-ft OK}$$

$$\text{Top moment -Muz (+ Side)} = 0.0 \text{ k-ft} < 4.0 \text{ k-ft OK}$$

- Bottom Bars

No Bottom Reinforcement Provided at the Footing

Use Plain Concrete Flexural Strength at Bottom

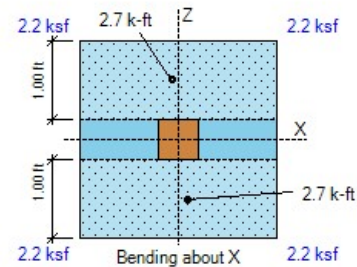
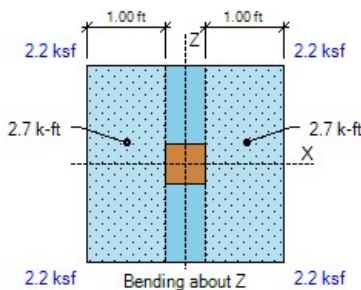
- Bottom moments calculated as the bearing minus the overburden pressures times the lever arm:

$$\text{Bottom moment Mux (- Side)} = 2.7 \text{ k-ft} < 4.0 \text{ k-ft OK} \quad \text{ratio} = 0.66$$

$$\text{Bottom moment Mux (+ Side)} = 2.7 \text{ k-ft} < 4.0 \text{ k-ft OK} \quad \text{ratio} = 0.66$$

$$\text{Bottom moment Muz (- Side)} = 2.7 \text{ k-ft} < 4.0 \text{ k-ft OK} \quad \text{ratio} = 0.66$$

$$\text{Bottom moment Muz (+ Side)} = 2.7 \text{ k-ft} < 4.0 \text{ k-ft OK} \quad \text{ratio} = 0.66$$



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

$$\text{Area } A1 = \text{col } L * \text{col } W = 6.0 * 6.0 = 36.0 \text{ in}^2$$

$$Sx = \text{col } W * \text{col } L^2 / 6 = 6.0 * 6.0^2 / 6 = 36.0 \text{ in}^3$$

$$Sz = \text{col } L * \text{col } W^2 / 6 = 6.0 * 6.0^2 / 6 = 36.0 \text{ in}^3$$

$$\text{Bearing } Pbu = P / A1 + Mz / Sx + Mx / Sz = 13.2 / 36.0 + 0.0 * 12 / 36.0 + 0.0 * 12 / 36.0 = 0.4 \text{ ksi}$$

$$\text{Min edge} = \text{Min} (L / 2 - X\text{-offset} - \text{col } L / 2, W / 2 - Z\text{-offset} - \text{col } W / 2)$$

$$\text{Min edge} = \text{Min} (2.50 * 12 / 2 - 0.0 - 6.0 / 2, 2.50 * 12 / 2 - 0.0 - 6.0 / 2) = 12.0 \text{ in}$$

$$\text{Area } A2 = \text{Min} [L * W, (\text{col } L + 2 * \text{Min edge}) * (\text{col } W + 2 * \text{Min edge})]$$

ACI R22.8.3.2

$$A2 = \text{Min} [2.50 * 12 * 2.5 * 12, (6.0 + 2 * 12.0) * (6.0 + 2 * 12.0)] = 900.0 \text{ in}^2$$

$$\text{Footing } \phi Pnc = \phi * 0.85 * f'c * \text{Min} [2, \sqrt{(A2 / A1)}] = 0.65 * 0.85 * 2.5 * \text{Min} [2, \sqrt{(900.0 / 36.0)}] = 2.8 \text{ ksi}$$

$$\text{Footing } \phi Pns = \phi * As * Fy / A1 = 0.0 \text{ ksi}$$

ACI 22.8.3.2

$$\text{Footing bearing } \phi Pn = \phi Pnc + \phi Pns = 2.8 + 0.0 = 2.8 \text{ ksi} > 0.4 \text{ psi OK}$$

Hooked $L_{dh} = \text{Max} (8 \text{ db}, 6, 0.02 * f_y / (f_c)^{1/2} * \text{Confining} * \text{Location} * \text{Concrete} * \text{db} * \text{ratio})$

ACI 25.4.3

$$L_{dh} = \text{Max} (8 \text{ db}, 6, 0.02 * 60.0 * 1000 / (2500)^{1/2} * 1.0 * 0.7 * 0.0 * 0.75 * 0.06) = 6.0 \text{ in}$$

Ld provided = Dowel length = $3.00 * 12 = 36.0 \text{ in} > 12.0 \text{ in OK}$

Ldh provided = Footing thickness - Cover = $8.00 - 3.0 = 5.0 \text{ in} < 6.0 \text{ in NG}$

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

$$X\text{-Edge} = \text{Length} / 2 - \text{Offset} - \text{Col} / 2 = 2.50 * 12 / 2 - 0.0 - 6.0 / 2 = 12.0 \text{ in} \quad \alpha_{sx} = 10$$

$$Z\text{-Edge} = \text{Width} / 2 - \text{Offset} - \text{Col} / 2 = 2.50 * 12 / 2 - 0.0 - 6.0 / 2 = 12.0 \text{ in} \quad \alpha_{sz} = 10$$

$$\alpha_s = \alpha_{sx} + \alpha_{sz} = 10 + 10 = 20 \quad \text{Col type} = \text{Corner} \quad \beta = L / W = 6.0 / 6.0 = 1.00$$

ACI 22.6.5.2

$$\text{Perimeter } b_o = \alpha_{sz} / 10 * (L + d / 2 + X\text{-Edge}) + \alpha_{sx} / 10 * (W + d / 2 + Z\text{-Edge})$$

ACI 22.6.4.2

$$b_o = 10 / 10 * (6.0 + 8.0 / 2 + 12.0) + 10 / 10 * (6.0 + 8.0 / 2 + 12.0) = 44.0 \text{ in}$$

$$\text{Area } A_{bo} = (L + d / 2 + X\text{-Edge}) * (W + d / 2 + Z\text{-Edge}) = (6.0 + 8.0 / 2 + 12.0) * (6.0 + 8.0 / 2 + 12.0) = 484.0 \text{ in}^2$$

Use Plain Concrete Shear Strength

$$\phi V_c = \phi * \text{Min} (1 + 2 / \beta, 2) * 4/3 * \sqrt{f_c}$$

ACI 14.5.5.1

$$\phi V_c = 0.60 * \text{Min} (1 + 2 / 1.00, 2) * 4/3 * \sqrt{2500} = 80.0 \text{ psi}$$

Punching force $F = P + \text{Overburden} * A_{bo} - \text{Bearing}$

$$F = 13.2 + 0.07 * 484.0 / 144 - 3.0 = 10.5 \text{ kip}$$

$$b_1 = L + d / 2 + X\text{-Edge} = 6.0 + 8.0 / 2 + 12.0 = 22.0 \text{ in} \quad b_2 = W + d / 2 + Z\text{-Edge} = 6.0 + 8.0 / 2 + 12.0 = 22.0 \text{ in}$$

$$\gamma_{vx} \text{ factor} = 1 - \frac{1}{1 + (2/3) \sqrt{(b_2 / b_1)}} = 1 - \frac{1}{1 + (2/3) \sqrt{(22.0 / 22.0)}} = 0.40$$

ACI Eq. (8.4.4.2.2)

$$\gamma_{vz} \text{ factor} = 1 - \frac{1}{1 + (2/3) \sqrt{(b_1 / b_2)}} = 1 - \frac{1}{1 + (2/3) \sqrt{(22.0 / 22.0)}} = 0.40$$

ACI Eq. (8.4.2.3.2)

$$X_{2z} = b_1^2 / 2 / (b_1 + b_2) = 22.0^2 / 2 / (22.0 + 22.0) = 5.5 \text{ in} \quad X_{2x} = b_2^2 / 2 / (b_2 + b_1) = 5.5 \text{ in}$$

$$J_{cz} = b_1 * d^3 / 12 + b_1^3 * d / 12 + b_1 * d * (b_1 / 2 - X_{2z})^2 + b_2 * d * X_{2z}^2$$

ACI R8.4.4.2.3

$$J_{cz} = 22.0 * 8.0^3 / 12 + 22.0^3 * 8.0 / 12 + 22.0 * 8.0 * (22.0 / 2 - 5.5)^2 + 22.0 * 8.0 * 5.5^2 = 18685 \text{ in}^4$$

$$J_{cx} = b_2 * d^3 / 12 + b_2^3 * d / 12 + b_2 * d * (b_2 / 2 - X_{2x})^2 + b_1 * d * X_{2x}^2$$

ACI R8.4.4.2.3

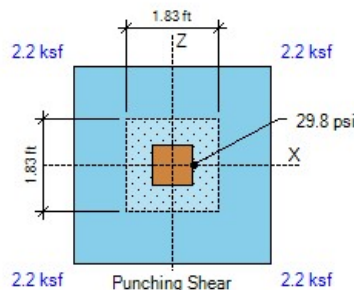
$$J_{cx} = 22.0 * 8.0^3 / 12 + 22.0^3 * 8.0 / 12 + 22.0 * 8.0 * (22.0 / 2 - 5.5)^2 + 22.0 * 8.0 * 5.5^2 = 18685 \text{ in}^4$$

$$\text{Stress due to } P = F / (b_o * d) * 1000 = 10.5 / (44.0 * 8.0) * 1000 = 29.8 \text{ psi}$$

$$\text{Stress due to } M_x = \gamma_{vx} * X\text{-OTM} * X_{2x} / J_{cx} = 0.40 * 0.0 * 12 * 5.5 / 18685 * 1000 = 0.0 \text{ psi}$$

$$\text{Stress due to } M_z = \gamma_{vz} * Z\text{-OTM} * X_{2z} / J_{cz} = 0.40 * 0.0 * 12 * 5.5 / 18685 * 1000 = 0.0 \text{ psi}$$

$$\text{Punching stress} = P\text{-stress} + M_x\text{-stress} + M_z\text{-stress} = 29.8 + 0.0 + 0.0 = 29.8 \text{ psi} < 80.0 \text{ psi OK}$$



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

$$\text{Area } A1 = \text{col } L * \text{col } W = 6.0 * 6.0 = 36.0 \text{ in}^2$$

$$Sx = \text{col } W * \text{col } L^2 / 6 = 6.0 * 6.0^2 / 6 = 36.0 \text{ in}^3$$

$$Sz = \text{col } L * \text{col } W^2 / 6 = 6.0 * 6.0^2 / 6 = 36.0 \text{ in}^3$$

$$\text{Bearing } Pbu = P / A1 + Mz / Sx + Mx / Sz = 13.2 / 36.0 + 0.0 * 12 / 36.0 + 0.0 * 12 / 36.0 = 0.4 \text{ ksi}$$

$$\text{Min edge} = \text{Min} (L / 2 - X\text{-offset} - \text{col } L / 2, W / 2 - Z\text{-offset} - \text{col } W / 2)$$

$$\text{Min edge} = \text{Min} (2.50 * 12 / 2 - 0.0 - 6.0 / 2, 2.50 * 12 / 2 - 0.0 - 6.0 / 2) = 12.0 \text{ in}$$

$$\text{Area } A2 = \text{Min} [L * W, (\text{col } L + 2 * \text{Min edge}) * (\text{col } W + 2 * \text{Min edge})]$$

ACI R22.8.3.2

$$A2 = \text{Min} [2.50 * 12 * 2.5 * 12, (6.0 + 2 * 12.0) * (6.0 + 2 * 12.0)] = 900.0 \text{ in}^2$$

$$\text{Footing } \phi Pnc = \phi * 0.85 * f'c * \text{Min} [2, \sqrt{(A2 / A1)}] = 0.65 * 0.85 * 2.5 * \text{Min} [2, \sqrt{(900.0 / 36.0)}] = 2.8 \text{ ksi}$$

$$\text{Footing } \phi Pns = \phi * As * Fy / A1 = 0.0 \text{ ksi}$$

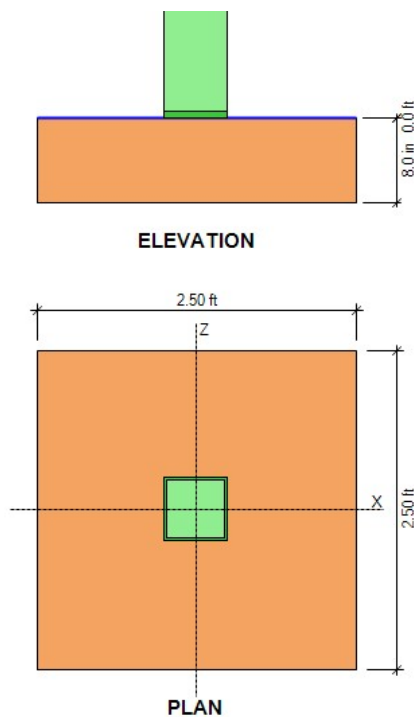
ACI 22.8.3.2

$$\text{Footing bearing } \phi Pn = \phi Pnc + \phi Pns = 2.8 + 0.0 = 2.8 \text{ ksi} > 0.4 \text{ psi OK}$$

DESIGN CODES

Concrete Design ACI 318-14

Load Combinations ASCE 7-10/16



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 = $0.6 * W * L * \gamma * (SC + Thick - WT) = 0.6 * 2.50 * 2.50 * 62 * (0.67) = -0.2$ 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 = $\frac{\text{Resisting moment}}{\text{Overturning moment}} = \frac{3.3}{0.0} = 33.49 > 1.50$ OK

- Overturning about Z-Z

$$\text{- Moment } M_z = 0.6 * 0.0 + 0.6 * 0.0 = 0.0 \text{ k-ft}$$

$$\text{- Shear Force } V_x = 0.6 * 0.0 + 0.6 * 0.0 = 0.0 \text{ kip}$$

$$\text{Arm} = 0.00 + 8.0 / 12 = 0.67 \text{ ft}$$

$$\text{Moment} = 0.0 * 0.67 = 0.0 \text{ k-ft}$$

$$\text{- Passive Force} = 0.0 \text{ kip}$$

$$\text{Arm} = 0.27 \text{ ft}$$

$$\text{Moment} = 0.0 \text{ k-ft}$$

$$\text{- Overturning moment Z-Z} = 0.0 + 0.0 = 0.0 \text{ k-ft}$$

- Resisting about Z-Z

$$\text{- Footing weight} = 0.6 * W * L * Thick * Density = 0.6 * 2.50 * 2.50 * 8.0 / 12 * 0.15 = 0.4 \text{ kip}$$

$$\text{Arm} = L / 2 = 2.50 / 2 = 1.25 \text{ ft}$$

$$\text{Moment} = 0.4 * 1.25 = 0.5 \text{ k-ft}$$

$$\text{- Pedestal weight} = 0.6 * W * L * H * Density = 0.6 * 6.0 / 12 * 6.0 / 12 * 0.0 * 0.15 = 0.0 \text{ kip}$$

$$\text{Arm} = L / 2 - Offset = 2.50 / 2 - 0.0 / 12 = 1.25 \text{ ft}$$

$$\text{Moment} = 0.0 * 1.25 = 0.0 \text{ k-ft}$$

$$\text{- Soil cover} = 0.6 * W * L * SC * Density = 0.6 * (2.50 * 2.50 - 6.0 / 12 * 6.0 / 12) * 0.0 * 110 = 0.0 \text{ kip}$$

$$\text{Arm} = L / 2 = 2.50 / 2 = 1.25 \text{ ft}$$

$$\text{Moment} = 0.0 * 1.25 = 0.0 \text{ k-ft}$$

$$\text{- Buoyancy} = 0.6 * W * L * \gamma * (SC + Thick - WT) = 0.6 * 2.50 * 2.50 * 62 * (0.67) = -0.2 \text{ kip}$$

$$\text{Arm} = L / 2 = 2.50 / 2 = 1.25 \text{ ft}$$

$$\text{Moment} = 0.2 * 1.25 = -0.2 \text{ k-ft}$$

$$\text{- Axial force } P = 0.6 * 4.1 + 0.6 * 0.0 = 2.5 \text{ kip}$$

$$\text{Arm} = L / 2 - Offset = 2.50 / 2 - 0.0 / 12 = 1.25 \text{ ft}$$

$$\text{Moment} = 2.5 * 1.25 = 3.1 \text{ k-ft}$$

$$\text{- Resisting moment Z-Z} = 0.5 + 0.0 + 0.0 + 3.1 + -0.2 = 3.3 \text{ k-ft}$$

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

SOIL BEARING PRESSURES (Comb: D+L)

$$\text{Overturning moment X-X} = 0.0 + 0.0 = 0.0 \text{ k-ft}$$

$$\text{Resisting moment X-X} = 0.8 + 0.0 + 0.0 + -0.3 + 11.6 = 12.1 \text{ k-ft}$$

$$\text{Overturning moment Z-Z} = 0.0 + 0.0 = 0.0 \text{ k-ft}$$

$$\text{Resisting moment Z-Z} = 0.8 + 0.0 + 0.0 + -0.3 + 11.6 = 12.1 \text{ k-ft}$$

$$\text{Resisting force} = \text{Footing} + \text{Pedestal} + \text{Soil} - \text{Buoyancy} + P = 0.6 + 0.0 + 0.0 - 0.3 + 9.3 = 9.7 \text{ kip}$$

X-coordinate of resultant from maximum bearing corner:

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

Z-coordinate of resultant from maximum bearing corner:

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

$$X\text{-ecc} = \text{Length} / 2 - X_p = 2.50 / 2 - 1.25 = 0.00 \text{ ft}$$

$$Z\text{-ecc} = \text{Width} / 2 - Z_p = 2.50 / 2 - 1.25 = 0.00 \text{ ft}$$

$$\text{Area} = \text{Width} * \text{Length} = 2.50 * 2.50 = 6.3 \text{ ft}^2$$

$$S_x = \text{Length} * \text{Width}^2 / 6 = 2.50 * 2.50^2 / 6 = 2.6 \text{ ft}^3$$

$$S_z = \text{Width} * \text{Length}^2 / 6 = 2.50 * 2.50^2 / 6 = 2.6 \text{ ft}^3$$

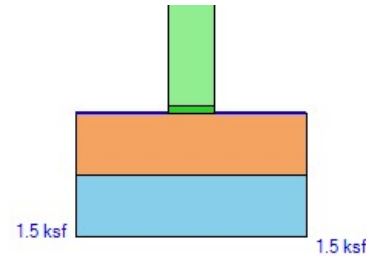
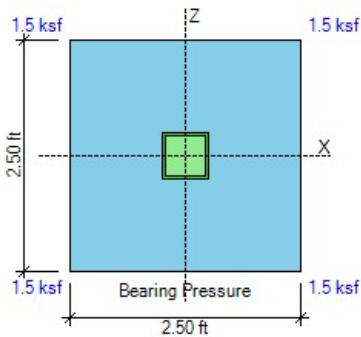
- Footing is in full bearing. Soil pressures are as follows:

$$P1 = P * (1/A + Z\text{-ecc} / S_x + X\text{-ecc} / S_z) = 9.7 * (1/6.3 + 0.00 / 2.6 + 0.00 / 2.6) = 1.55 \text{ ksf}$$

$$P2 = P * (1/A - Z\text{-ecc} / S_x + X\text{-ecc} / S_z) = 9.7 * (1/6.3 - 0.00 / 2.6 + 0.00 / 2.6) = 1.55 \text{ ksf}$$

$$P3 = P * (1/A - Z\text{-ecc} / S_x - X\text{-ecc} / S_z) = 9.7 * (1/6.3 - 0.00 / 2.6 - 0.00 / 2.6) = 1.55 \text{ ksf}$$

$$P4 = P * (1/A + Z\text{-ecc} / S_x - X\text{-ecc} / S_z) = 9.7 * (1/6.3 + 0.00 / 2.6 - 0.00 / 2.6) = 1.55 \text{ ksf}$$



SLIDING CALCULATIONS (Comb: 0.6D+0.6W)

Internal friction angle = 28.0 deg

Passive coefficient $k_p = 4.33$ (per Coulomb)Pressure at mid-depth = $k_p \cdot \text{Density} \cdot (\text{Cover} + \text{Thick} / 2) = 4.33 \cdot 110 \cdot (0.00 + 8.0 / 12 / 2) = 0.16$ ksfX-Passive force = $\text{Pressure} \cdot \text{Thick} \cdot \text{Width} = 0.16 \cdot 8.0 / 12 \cdot 2.50 = 0.3$ kipZ-Passive force = $\text{Pressure} \cdot \text{Thick} \cdot \text{Length} = 0.16 \cdot 8.0 / 12 \cdot 2.50 = 0.3$ kipFriction force = $\text{Resisting force} \cdot \text{Friction coeff.} = \text{Max}(0, 2.7 \cdot 0.35) = 0.9$ kip

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

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

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

UPLIFT CALCULATIONS (Comb: 0.6D+0.6W)

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

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

Concrete $f'_c = 2.5$ ksiSteel $f_y = 40.0$ ksi

Soil density = 110 pcf

Use Plain Concrete Shear Strength

$$\phi V_{cx} = \frac{4}{3} \cdot \phi \cdot \sqrt{f'_c} \cdot \text{Width} \cdot t / 1000 = \frac{4}{3} \cdot 0.60 \cdot \sqrt{2500} \cdot 2.5 \cdot 12 \cdot 8.0 / 1000 = 9.6 \text{ kip}$$

ACI 14.5.5.1

$$\phi V_{cz} = \frac{4}{3} \cdot \phi \cdot \sqrt{f'_c} \cdot \text{Length} \cdot t / 1000 = \frac{4}{3} \cdot 0.60 \cdot \sqrt{2500} \cdot 2.5 \cdot 12 \cdot 8.0 / 1000 = 9.6 \text{ kip}$$

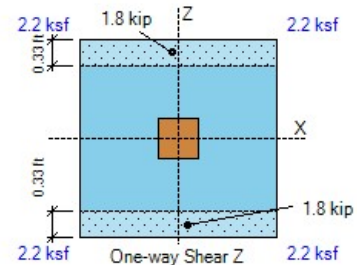
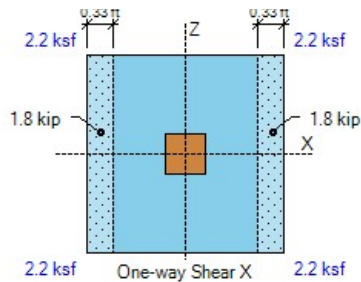
- Shear forces calculated as the volume of the bearing pressures under the effective areas:

$$\text{One-way shear } V_{ux} \text{ (- Side)} = 1.8 \text{ kip} < 9.6 \text{ kip} \quad \text{OK}$$

$$\text{One-way shear } V_{ux} \text{ (+ Side)} = 1.8 \text{ kip} < 9.6 \text{ kip} \quad \text{OK}$$

$$\text{One-way shear } V_{uz} \text{ (- Side)} = 1.8 \text{ kip} < 9.6 \text{ kip} \quad \text{OK}$$

$$\text{One-way shear } V_{uz} \text{ (+ Side)} = 1.8 \text{ kip} < 9.6 \text{ kip} \quad \text{OK}$$



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

$$\text{Plain } \phi M_{nx} = 5 * \phi * \sqrt{f_c} * L * \text{Thick}^2 / 6 = 5 * 0.60 * \sqrt{(2500)} * 2.50 * 8.0^2 / 6 / 1000 = 1.1 \text{ k-ft}$$

ACI Eq. (14.5.2.1a)

$$\text{Plain } \phi M_{nz} = 5 * \phi * \sqrt{f_c} * W * \text{Thick}^2 / 6 = 5 * 0.60 * \sqrt{(2500)} * 2.50 * 8.0^2 / 6 / 1000 = 1.1 \text{ k-ft}$$

- 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:

$$\text{Top moment -M}_{ux} \text{ (- Side)} = 0.0 \text{ k-ft} < 4.0 \text{ k-ft OK}$$

$$\text{Top moment -M}_{ux} \text{ (+ Side)} = 0.0 \text{ k-ft} < 4.0 \text{ k-ft OK}$$

$$\text{Top moment -M}_{uz} \text{ (- Side)} = 0.0 \text{ k-ft} < 4.0 \text{ k-ft OK}$$

$$\text{Top moment -M}_{uz} \text{ (+ Side)} = 0.0 \text{ k-ft} < 4.0 \text{ k-ft OK}$$

- Bottom Bars

No Bottom Reinforcement Provided at the Footing

Use Plain Concrete Flexural Strength at Bottom

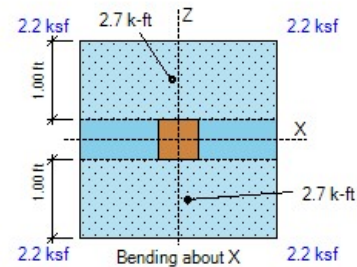
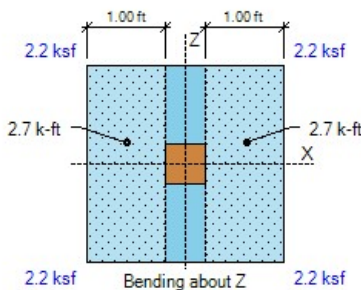
- Bottom moments calculated as the bearing minus the overburden pressures times the lever arm:

$$\text{Bottom moment M}_{ux} \text{ (- Side)} = 2.7 \text{ k-ft} < 4.0 \text{ k-ft OK} \quad \text{ratio} = 0.66$$

$$\text{Bottom moment M}_{ux} \text{ (+ Side)} = 2.7 \text{ k-ft} < 4.0 \text{ k-ft OK} \quad \text{ratio} = 0.66$$

$$\text{Bottom moment M}_{uz} \text{ (- Side)} = 2.7 \text{ k-ft} < 4.0 \text{ k-ft OK} \quad \text{ratio} = 0.66$$

$$\text{Bottom moment M}_{uz} \text{ (+ Side)} = 2.7 \text{ k-ft} < 4.0 \text{ k-ft OK} \quad \text{ratio} = 0.66$$



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

$$\text{Area } A1 = \text{col } L * \text{col } W = 6.0 * 6.0 = 36.0 \text{ in}^2$$

$$Sx = \text{col } W * \text{col } L^2 / 6 = 6.0 * 6.0^2 / 6 = 36.0 \text{ in}^3$$

$$Sz = \text{col } L * \text{col } W^2 / 6 = 6.0 * 6.0^2 / 6 = 36.0 \text{ in}^3$$

$$\text{Bearing } Pbu = P / A1 + Mz / Sx + Mx / Sz = 13.2 / 36.0 + 0.0 * 12 / 36.0 + 0.0 * 12 / 36.0 = 0.4 \text{ ksi}$$

$$\text{Min edge} = \text{Min} (L / 2 - X\text{-offset} - \text{col } L / 2, W / 2 - Z\text{-offset} - \text{col } W / 2)$$

$$\text{Min edge} = \text{Min} (2.50 * 12 / 2 - 0.0 - 6.0 / 2, 2.50 * 12 / 2 - 0.0 - 6.0 / 2) = 12.0 \text{ in}$$

$$\text{Area } A2 = \text{Min} [L * W, (\text{col } L + 2 * \text{Min edge}) * (\text{col } W + 2 * \text{Min edge})]$$

ACI R22.8.3.2

$$A2 = \text{Min} [2.50 * 12 * 2.5 * 12, (6.0 + 2 * 12.0) * (6.0 + 2 * 12.0)] = 900.0 \text{ in}^2$$

$$\text{Footing } \phi Pnc = \phi * 0.85 * f'c * \text{Min} [2, \sqrt{(A2 / A1)}] = 0.65 * 0.85 * 2.5 * \text{Min} [2, \sqrt{(900.0 / 36.0)}] = 2.8 \text{ ksi}$$

$$\text{Footing } \phi Pns = \phi * As * Fy / A1 = 0.0 \text{ ksi}$$

ACI 22.8.3.2

$$\text{Footing bearing } \phi Pn = \phi Pnc + \phi Pns = 2.8 + 0.0 = 2.8 \text{ ksi} > 0.4 \text{ psi OK}$$

Hooked $L_{dh} = \text{Max} (8 \text{ db}, 6, 0.02 * f_y / (f_c)^{1/2} * \text{Confining} * \text{Location} * \text{Concrete} * \text{db} * \text{ratio})$

ACI 25.4.3

$$L_{dh} = \text{Max} (8 \text{ db}, 6, 0.02 * 60.0 * 1000 / (2500)^{1/2} * 1.0 * 0.7 * 0.0 * 0.75 * 0.06) = 6.0 \text{ in}$$

Ld provided = Dowel length = $3.00 * 12 = 36.0 \text{ in} > 12.0 \text{ in OK}$

Ldh provided = Footing thickness - Cover = $8.00 - 3.0 = 5.0 \text{ in} < 6.0 \text{ in NG}$

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

$$X\text{-Edge} = \text{Length} / 2 - \text{Offset} - \text{Col} / 2 = 2.50 * 12 / 2 - 0.0 - 6.0 / 2 = 12.0 \text{ in} \quad \alpha_{sx} = 10$$

$$Z\text{-Edge} = \text{Width} / 2 - \text{Offset} - \text{Col} / 2 = 2.50 * 12 / 2 - 0.0 - 6.0 / 2 = 12.0 \text{ in} \quad \alpha_{sz} = 10$$

$$\alpha_s = \alpha_{sx} + \alpha_{sz} = 10 + 10 = 20 \quad \text{Col type} = \text{Corner} \quad \beta = L / W = 6.0 / 6.0 = 1.00$$

ACI 22.6.5.2

$$\text{Perimeter } b_o = \alpha_{sz} / 10 * (L + d / 2 + X\text{-Edge}) + \alpha_{sx} / 10 * (W + d / 2 + Z\text{-Edge})$$

ACI 22.6.4.2

$$b_o = 10 / 10 * (6.0 + 8.0 / 2 + 12.0) + 10 / 10 * (6.0 + 8.0 / 2 + 12.0) = 44.0 \text{ in}$$

$$\text{Area } A_{bo} = (L + d / 2 + X\text{-Edge}) * (W + d / 2 + Z\text{-Edge}) = (6.0 + 8.0 / 2 + 12.0) * (6.0 + 8.0 / 2 + 12.0) = 484.0 \text{ in}^2$$

Use Plain Concrete Shear Strength

$$\phi V_c = \phi * \text{Min} (1 + 2 / \beta, 2) * 4/3 * \sqrt{f_c}$$

ACI 14.5.5.1

$$\phi V_c = 0.60 * \text{Min} (1 + 2 / 1.00, 2) * 4/3 * \sqrt{2500} = 80.0 \text{ psi}$$

Punching force $F = P + \text{Overburden} * A_{bo} - \text{Bearing}$

$$F = 13.2 + 0.07 * 484.0 / 144 - 3.0 = 10.5 \text{ kip}$$

$$b_1 = L + d / 2 + X\text{-Edge} = 6.0 + 8.0 / 2 + 12.0 = 22.0 \text{ in} \quad b_2 = W + d / 2 + Z\text{-Edge} = 6.0 + 8.0 / 2 + 12.0 = 22.0 \text{ in}$$

$$\gamma_{vx} \text{ factor} = 1 - \frac{1}{1 + (2/3) \sqrt{(b_2 / b_1)}} = 1 - \frac{1}{1 + (2/3) \sqrt{(22.0 / 22.0)}} = 0.40$$

ACI Eq. (8.4.4.2.2)

$$\gamma_{vz} \text{ factor} = 1 - \frac{1}{1 + (2/3) \sqrt{(b_1 / b_2)}} = 1 - \frac{1}{1 + (2/3) \sqrt{(22.0 / 22.0)}} = 0.40$$

ACI Eq. (8.4.2.3.2)

$$X_{2z} = b_1^2 / 2 / (b_1 + b_2) = 22.0^2 / 2 / (22.0 + 22.0) = 5.5 \text{ in} \quad X_{2x} = b_2^2 / 2 / (b_2 + b_1) = 5.5 \text{ in}$$

$$J_{cz} = b_1 * d^3 / 12 + b_1^3 * d / 12 + b_1 * d * (b_1 / 2 - X_{2z})^2 + b_2 * d * X_{2z}^2$$

ACI R8.4.4.2.3

$$J_{cz} = 22.0 * 8.0^3 / 12 + 22.0^3 * 8.0 / 12 + 22.0 * 8.0 * (22.0 / 2 - 5.5)^2 + 22.0 * 8.0 * 5.5^2 = 18685 \text{ in}^4$$

$$J_{cx} = b_2 * d^3 / 12 + b_2^3 * d / 12 + b_2 * d * (b_2 / 2 - X_{2x})^2 + b_1 * d * X_{2x}^2$$

ACI R8.4.4.2.3

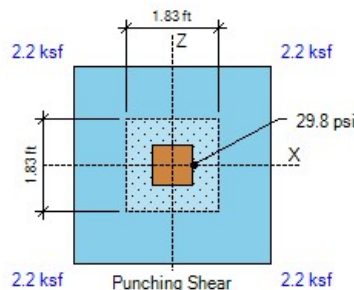
$$J_{cx} = 22.0 * 8.0^3 / 12 + 22.0^3 * 8.0 / 12 + 22.0 * 8.0 * (22.0 / 2 - 5.5)^2 + 22.0 * 8.0 * 5.5^2 = 18685 \text{ in}^4$$

$$\text{Stress due to } P = F / (b_o * d) * 1000 = 10.5 / (44.0 * 8.0) * 1000 = 29.8 \text{ psi}$$

$$\text{Stress due to } M_x = \gamma_{vx} * X\text{-OTM} * X_{2x} / J_{cx} = 0.40 * 0.0 * 12 * 5.5 / 18685 * 1000 = 0.0 \text{ psi}$$

$$\text{Stress due to } M_z = \gamma_{vz} * Z\text{-OTM} * X_{2z} / J_{cz} = 0.40 * 0.0 * 12 * 5.5 / 18685 * 1000 = 0.0 \text{ psi}$$

$$\text{Punching stress} = P\text{-stress} + M_x\text{-stress} + M_z\text{-stress} = 29.8 + 0.0 + 0.0 = 29.8 \text{ psi} < 80.0 \text{ psi OK}$$



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

$$\text{Area } A1 = \text{col } L * \text{col } W = 6.0 * 6.0 = 36.0 \text{ in}^2$$

$$Sx = \text{col } W * \text{col } L^2 / 6 = 6.0 * 6.0^2 / 6 = 36.0 \text{ in}^3$$

$$Sz = \text{col } L * \text{col } W^2 / 6 = 6.0 * 6.0^2 / 6 = 36.0 \text{ in}^3$$

$$\text{Bearing } Pbu = P / A1 + Mz / Sx + Mx / Sz = 13.2 / 36.0 + 0.0 * 12 / 36.0 + 0.0 * 12 / 36.0 = 0.4 \text{ ksi}$$

$$\text{Min edge} = \text{Min} (L / 2 - X\text{-offset} - \text{col } L / 2, W / 2 - Z\text{-offset} - \text{col } W / 2)$$

$$\text{Min edge} = \text{Min} (2.50 * 12 / 2 - 0.0 - 6.0 / 2, 2.50 * 12 / 2 - 0.0 - 6.0 / 2) = 12.0 \text{ in}$$

$$\text{Area } A2 = \text{Min} [L * W, (\text{col } L + 2 * \text{Min edge}) * (\text{col } W + 2 * \text{Min edge})]$$

ACI R22.8.3.2

$$A2 = \text{Min} [2.50 * 12 * 2.5 * 12, (6.0 + 2 * 12.0) * (6.0 + 2 * 12.0)] = 900.0 \text{ in}^2$$

$$\text{Footing } \phi Pnc = \phi * 0.85 * f'c * \text{Min} [2, \sqrt{(A2 / A1)}] = 0.65 * 0.85 * 2.5 * \text{Min} [2, \sqrt{(900.0 / 36.0)}] = 2.8 \text{ ksi}$$

$$\text{Footing } \phi Pns = \phi * As * Fy / A1 = 0.0 \text{ ksi}$$

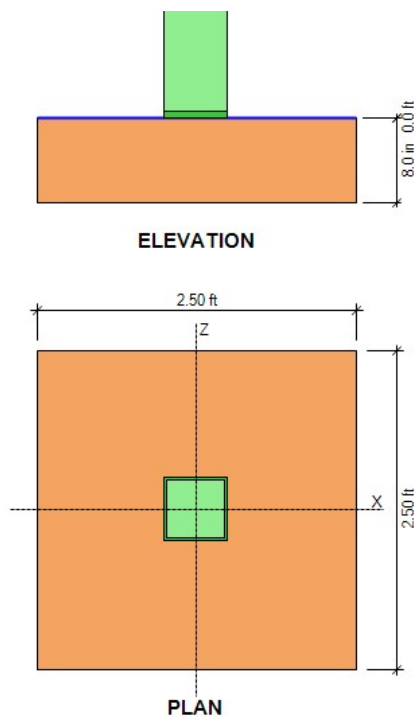
ACI 22.8.3.2

$$\text{Footing bearing } \phi Pn = \phi Pnc + \phi Pns = 2.8 + 0.0 = 2.8 \text{ ksi} > 0.4 \text{ psi OK}$$

DESIGN CODES

Concrete Design ACI 318-14

Load Combinations ASCE 7-10/16



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 = $0.6 * W * L * \gamma * (SC + Thick - WT) = 0.6 * 3.00 * 3.00 * 62 * (0.67) = -0.2$ 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 = $\frac{\text{Resisting moment}}{\text{Overturning moment}} = \frac{4.0}{0.0} = 39.83 > 1.50$ OK

- Overturning about Z-Z

$$\text{- Moment } M_z = 0.6 * 0.0 + 0.6 * 0.0 = 0.0 \text{ k-ft}$$

$$\text{- Shear Force } V_x = 0.6 * 0.0 + 0.6 * 0.0 = 0.0 \text{ kip}$$

$$\text{Arm} = 0.00 + 8.0 / 12 = 0.67 \text{ ft}$$

$$\text{Moment} = 0.0 * 0.67 = 0.0 \text{ k-ft}$$

$$\text{- Passive Force} = 0.0 \text{ kip}$$

$$\text{Arm} = 0.27 \text{ ft}$$

$$\text{Moment} = 0.0 \text{ k-ft}$$

$$\text{- Overturning moment Z-Z} = 0.0 + 0.0 = 0.0 \text{ k-ft}$$

- Resisting about Z-Z

$$\text{- Footing weight} = 0.6 * W * L * Thick * Density = 0.6 * 3.00 * 3.00 * 8.0 / 12 * 0.15 = 0.5 \text{ kip}$$

$$\text{Arm} = L / 2 = 3.00 / 2 = 1.50 \text{ ft}$$

$$\text{Moment} = 0.5 * 1.50 = 0.8 \text{ k-ft}$$

$$\text{- Pedestal weight} = 0.6 * W * L * H * Density = 0.6 * 6.0 / 12 * 6.0 / 12 * 0.0 * 0.15 = 0.0 \text{ kip}$$

$$\text{Arm} = L / 2 - Offset = 3.00 / 2 - 0.0 / 12 = 1.50 \text{ ft}$$

$$\text{Moment} = 0.0 * 1.50 = 0.0 \text{ k-ft}$$

$$\text{- Soil cover} = 0.6 * W * L * SC * Density = 0.6 * (3.00 * 3.00 - 6.0 / 12 * 6.0 / 12) * 0.0 * 110 = 0.0 \text{ kip}$$

$$\text{Arm} = L / 2 = 3.00 / 2 = 1.50 \text{ ft}$$

$$\text{Moment} = 0.0 * 1.50 = 0.0 \text{ k-ft}$$

$$\text{- Buoyancy} = 0.6 * W * L * \gamma * (SC + Thick - WT) = 0.6 * 3.00 * 3.00 * 62 * (0.67) = -0.2 \text{ kip}$$

$$\text{Arm} = L / 2 = 3.00 / 2 = 1.50 \text{ ft}$$

$$\text{Moment} = 0.2 * 1.50 = -0.3 \text{ k-ft}$$

$$\text{- Axial force } P = 0.6 * 3.9 + 0.6 * 0.0 = 2.3 \text{ kip}$$

$$\text{Arm} = L / 2 - Offset = 3.00 / 2 - 0.0 / 12 = 1.50 \text{ ft}$$

$$\text{Moment} = 2.3 * 1.50 = 3.5 \text{ k-ft}$$

$$\text{- Resisting moment Z-Z} = 0.8 + 0.0 + 0.0 + 3.5 - 0.3 = 4.0 \text{ k-ft}$$

$$\text{- Overturning safety factor Z-Z} = \frac{\text{Resisting moment}}{\text{Overturning moment}} = \frac{4.0}{0.0} = 39.83 > 1.50 \text{ OK}$$

SOIL BEARING PRESSURES (Comb: D+L)

$$\text{Overturning moment X-X} = 0.0 + 0.0 = 0.0 \text{ k-ft}$$

$$\text{Resisting moment X-X} = 1.4 + 0.0 + 0.0 + -0.6 + 23.6 = 24.3 \text{ k-ft}$$

$$\text{Overturning moment Z-Z} = 0.0 + 0.0 = 0.0 \text{ k-ft}$$

$$\text{Resisting moment Z-Z} = 1.4 + 0.0 + 0.0 + -0.6 + 23.6 = 24.3 \text{ k-ft}$$

$$\text{Resisting force} = \text{Footing} + \text{Pedestal} + \text{Soil} - \text{Buoyancy} + P = 0.9 + 0.0 + 0.0 - 0.4 + 15.7 = 16.2 \text{ kip}$$

X-coordinate of resultant from maximum bearing corner:

$$X_p = \frac{Z\text{-Resisting moment} - Z\text{-Overturning moment}}{\text{Resisting force}} = \frac{24.3 - 0.0}{16.2} = 1.50 \text{ ft}$$

Z-coordinate of resultant from maximum bearing corner:

$$Z_p = \frac{X\text{-Resisting moment} - X\text{-Overturning moment}}{\text{Resisting force}} = \frac{24.3 - 0.0}{16.2} = 1.50 \text{ ft}$$

$$X\text{-ecc} = \text{Length} / 2 - X_p = 3.00 / 2 - 1.50 = 0.00 \text{ ft}$$

$$Z\text{-ecc} = \text{Width} / 2 - Z_p = 3.00 / 2 - 1.50 = 0.00 \text{ ft}$$

$$\text{Area} = \text{Width} * \text{Length} = 3.00 * 3.00 = 9.0 \text{ ft}^2$$

$$S_x = \text{Length} * \text{Width}^2 / 6 = 3.00 * 3.00^2 / 6 = 4.5 \text{ ft}^3$$

$$S_z = \text{Width} * \text{Length}^2 / 6 = 3.00 * 3.00^2 / 6 = 4.5 \text{ ft}^3$$

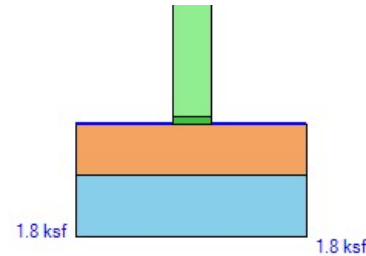
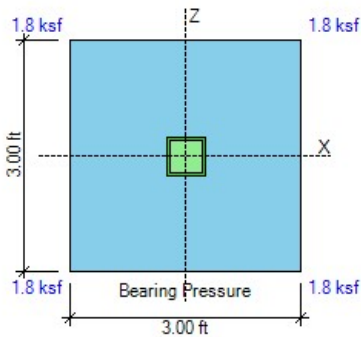
- Footing is in full bearing. Soil pressures are as follows:

$$P1 = P * (1/A + Z\text{-ecc} / S_x + X\text{-ecc} / S_z) = 16.2 * (1/9.0 + 0.00 / 4.5 + 0.00 / 4.5) = 1.80 \text{ ksf}$$

$$P2 = P * (1/A - Z\text{-ecc} / S_x + X\text{-ecc} / S_z) = 16.2 * (1/9.0 - 0.00 / 4.5 + 0.00 / 4.5) = 1.80 \text{ ksf}$$

$$P3 = P * (1/A - Z\text{-ecc} / S_x - X\text{-ecc} / S_z) = 16.2 * (1/9.0 - 0.00 / 4.5 - 0.00 / 4.5) = 1.80 \text{ ksf}$$

$$P4 = P * (1/A + Z\text{-ecc} / S_x - X\text{-ecc} / S_z) = 16.2 * (1/9.0 + 0.00 / 4.5 - 0.00 / 4.5) = 1.80 \text{ ksf}$$



SLIDING CALCULATIONS (Comb: 0.6D+0.6W)

Internal friction angle = 28.0 deg

Passive coefficient $k_p = 4.33$ (per Coulomb)Pressure at mid-depth = $k_p \cdot \text{Density} \cdot (\text{Cover} + \text{Thick} / 2) = 4.33 \cdot 110 \cdot (0.00 + 8.0 / 12 / 2) = 0.16$ ksfX-Passive force = $\text{Pressure} \cdot \text{Thick} \cdot \text{Width} = 0.16 \cdot 8.0 / 12 \cdot 3.00 = 0.3$ kipZ-Passive force = $\text{Pressure} \cdot \text{Thick} \cdot \text{Length} = 0.16 \cdot 8.0 / 12 \cdot 3.00 = 0.3$ kipFriction force = $\text{Resisting force} \cdot \text{Friction coeff.} = \text{Max}(0, 2.7 \cdot 0.35) = 0.9$ kip

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

$$\text{- Sliding safety factor X-X} = \frac{\text{X-Passive force} + \text{Friction}}{\text{X-Horizontal load}} = \frac{1.00 \cdot 0.3 + 1.00 \cdot 0.9}{0.0} = 12.47 > 1.50 \text{ OK}$$

$$\text{- Sliding safety factor Z-Z} = \frac{\text{Z-Passive force} + \text{Friction}}{\text{Z-Horizontal load}} = \frac{1.00 \cdot 0.3 + 1.00 \cdot 0.9}{0.0} = 12.47 > 1.50 \text{ OK}$$

UPLIFT CALCULATIONS (Comb: 0.6D+0.6W)

$$\text{- Uplift safety factor} = \frac{\text{Pedestal} + \text{Footing} + \text{Cover} - \text{Buoyancy}}{\text{Uplift load}} = \frac{0.0 + 0.5 + 0.0 - 0.2}{0.0} = 99.99 > 1.00 \text{ OK}$$

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

Concrete $f'_c = 2.5$ ksiSteel $f_y = 40.0$ ksi

Soil density = 110 pcf

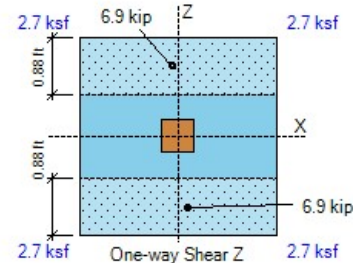
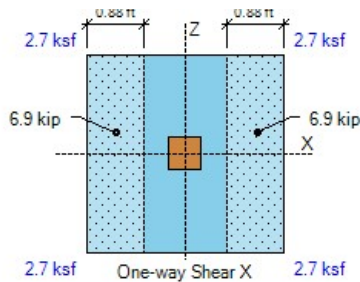
d Top X-dir = $\text{Thick} - \text{Cover} - \text{X-diameter} / 2 = 8.0 - 2.0 - 0.8 / 2 = 5.6$ ind Top Z-dir = $\text{Thick} - \text{Cover} - \text{X-diameter} - \text{Z-diameter} / 2 = 8.0 - 2.0 - 0.8 - 0.8 / 2 = 4.9$ ind Bot X-dir = $\text{Thick} - \text{Cover} - \text{X-diameter} / 2 = 8.0 - 3.0 - 0.5 / 2 = 4.8$ ind Bot Z-dir = $\text{Thick} - \text{Cover} - \text{X-diameter} - \text{Z-diameter} / 2 = 8.0 - 3.0 - 0.5 - 0.5 / 2 = 4.3$ in $\phi V_{cx} = 2 \cdot \phi \cdot \sqrt{f'_c} \cdot \text{Width} \cdot d / 1000 = 2 \cdot 0.75 \cdot \sqrt{(2500)} \cdot 3.0 \cdot 12 \cdot 4.8 / 1000 = 12.8$ kip

ACI Eq. (22.5.5.1)

 $\phi V_{cz} = 2 \cdot \phi \cdot \sqrt{f'_c} \cdot \text{Length} \cdot d / 1000 = 2 \cdot 0.75 \cdot \sqrt{(2500)} \cdot 3.0 \cdot 12 \cdot 4.3 / 1000 = 11.5$ kip

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

One-way shear V_{ux} (- Side) = 6.9 kip < 12.8 kip OKOne-way shear V_{ux} (+ Side) = 6.9 kip < 12.8 kip OKOne-way shear V_{uz} (- Side) = 6.9 kip < 11.5 kip OKOne-way shear V_{uz} (+ Side) = 6.9 kip < 11.5 kip OK



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

$$\text{Plain } \phi M_{nx} = 5 * \phi * \sqrt{f_c} * L * \text{Thick}^2 / 6 = 5 * 0.60 * \sqrt{(2500)} * 3.00 * 8.0^2 / 6 / 1000 = 1.3 \text{ k-ft}$$

ACI Eq. (14.5.2.1a)

$$\text{Plain } \phi M_{nz} = 5 * \phi * \sqrt{f_c} * W * \text{Thick}^2 / 6 = 5 * 0.60 * \sqrt{(2500)} * 3.00 * 8.0^2 / 6 / 1000 = 1.3 \text{ k-ft}$$

- 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:

$$\text{Top moment -Mux (- Side)} = 0.0 \text{ k-ft} < 4.8 \text{ k-ft OK}$$

$$\text{Top moment -Mux (+ Side)} = 0.0 \text{ k-ft} < 4.8 \text{ k-ft OK}$$

$$\text{Top moment -Muz (- Side)} = 0.0 \text{ k-ft} < 4.8 \text{ k-ft OK}$$

$$\text{Top moment -Muz (+ Side)} = 0.0 \text{ k-ft} < 4.8 \text{ k-ft OK}$$

- Bottom Bars

$$\text{Use 5 \#4 Z-Bars } \rho = A_s / b d = 1.0 / (3.00 * 12 * 4.3) = 0.0065$$

$$q = 0.0065 * 40 / 2.5 = 0.105$$

$$\text{Use 5 \#4 X-Bars } \rho = A_s / b d = 1.0 / (3.00 * 12 * 4.8) = 0.0058$$

$$q = 0.0058 * 40 / 2.5 = 0.094$$

$$\beta = L / W = 3.00 / 3.00 = 1.00 \quad \gamma_s = 2 * \beta / (\beta + 1) = 2 * 1.00 / (1.00 + 1) = 1.00$$

ACI 13.3.3.3

$$\text{Bending strength } \phi M_n = \phi * b * d^2 * f_c * q * (1 - 0.59 * q)$$

ACI 22.2.2

$$\phi M_{nx} = 0.90 * 3.00 * 12 * 4.3^2 * 2.5 * 0.105 * (1 - 0.59 * 0.105) = 12.0 \text{ k-ft}$$

$$\phi M_{nz} = 0.90 * 3.00 * 12 * 4.8^2 * 2.5 * 0.094 / 1.00 * (1 - 0.59 * 0.094 / 1.00) = 13.5 \text{ k-ft}$$

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

$$\text{Bottom moment Mux (- Side)} = 6.2 \text{ k-ft} < 12.0 \text{ k-ft OK} \quad \text{ratio} = 0.51$$

$$\text{Bottom moment Mux (+ Side)} = 6.2 \text{ k-ft} < 12.0 \text{ k-ft OK} \quad \text{ratio} = 0.51$$

$$\text{Bottom moment Muz (- Side)} = 6.2 \text{ k-ft} < 13.5 \text{ k-ft OK} \quad \text{ratio} = 0.46$$

$$\text{Bottom moment Muz (+ Side)} = 6.2 \text{ k-ft} < 13.5 \text{ k-ft OK} \quad \text{ratio} = 0.46$$

$$\text{X-As min} = 0.0018 * \text{Width} * \text{Thick} = 0.0018 * 3.00 * 12 * 8.0 = 0.5 \text{ in}^2 < 1.0 \text{ in}^2 \text{ OK}$$

ACI 8.6.1.1

$$\text{Z-As min} = 0.0018 * \text{Length} * \text{Thick} = 0.0018 * 3.00 * 12 * 8.0 = 0.5 \text{ in}^2 < 1.0 \text{ in}^2 \text{ OK}$$

ACI 8.6.1.1

$$\text{X-As max for 0.005 tension strain} = 2.74 \text{ in}^2 > 1.00 \text{ in}^2 \text{ OK}$$

ACI 21.2.2

$$\text{Z-As max for 0.005 tension strain} = 2.74 \text{ in}^2 > 1.00 \text{ in}^2 \text{ OK}$$

ACI 21.2.2

$$\text{X-Cover factor} = \text{Min} (2.5, (\text{Cover} + db / 2, \text{Spacing} / 2) / db) = \text{Min} (2.5, (3.0 + 0.50 / 2, 7.5 / 2) / 0.50) = 2.5$$

$$\text{Straight X-Ld} = \text{Max} (12.0, 3 / 40 * f_y / (f_c)^{1/2} * \text{Grade} * \text{Size} * \text{Casting} / \text{Cover} * db * \text{ratio})$$

ACI Eq. (25.4.2.3a)

$$\text{X-Ld} = \text{Max} (12.0, 3 / 40 * 40.0 * 1000 / (2500)^{1/2} * 1.0 * 0.8 * 1.0 / 2.5 * 0.50 * 0.46) = 12.0 \text{ in}$$

$$\text{Hooked X-Ldh} = \text{Max} (8 db, 6, 0.02 * f_y / (f_c)^{1/2} * \text{Confining} * \text{Location} * \text{Concrete} * db * \text{ratio}) =$$

ACI 25.4.3

$$\text{X-Ldh} = \text{Max} (8 db, 6, 0.02 * 40.0 * 1000 / (2500)^{1/2} * 1.0 * 0.7 * 0.0 * 0.50 * 0.46) = 6.0 \text{ in}$$

$$\text{-X Ld provided} = (\text{Length} - \text{Col}) / 2 + \text{Offset} - \text{Cover} = 3.00 * 12 / 2 + 0.0 - 6.0 / 2 - 2.5 = 12.5 \text{ in} > 12.0 \text{ in OK}$$

$$\text{+X Ld provided} = (\text{Length} - \text{Col}) / 2 - \text{Offset} - \text{Cover} = 3.00 * 12 / 2 - 0.0 - 6.0 / 2 - 2.5 = 12.5 \text{ in} > 12.0 \text{ in OK} \quad 4 \text{ of } 7$$

$$Z\text{-Cover factor} = \text{Min} (2.5, (\text{Cover} + db / 2, \text{Spacing} / 2) / db) = \text{Min} (2.5, (3.0 + 0.50 / 2, 7.5 / 2) / 0.50) = 2.5$$

$$\text{Straight } Z\text{-Ld} = \text{Max} (12.0, 3 / 40 * f_y / (f_c)^{1/2} * \text{Grade} * \text{Size} * \text{Casting} / \text{Cover} * db * \text{ratio})$$

ACI Eq. (25.4.2.3a)

$$Z\text{-Ld} = \text{Max} (12.0, 3 / 40 * 40.0 * 1000 / (2500)^{1/2} * 1.0 * 0.8 * 1.0 / 2.5 * 0.50 * 0.46) = 12.0 \text{ in}$$

$$\text{Hooked } Z\text{-Ldh} = \text{Max} (8 \text{ db}, 6, 0.02 * f_y / (f_c)^{1/2} * \text{Confining} * \text{Location} * \text{Concrete} * db * \text{ratio}) =$$

ACI 25.4.3

$$Z\text{-Ldh} = \text{Max} (8 \text{ db}, 6, 0.02 * 40.0 * 1000 / (2500)^{1/2} * 1.0 * 0.7 * 0.0 * 0.50 * 0.51) = 6.0 \text{ in}$$

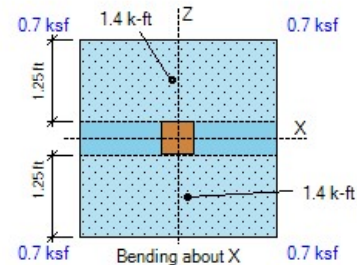
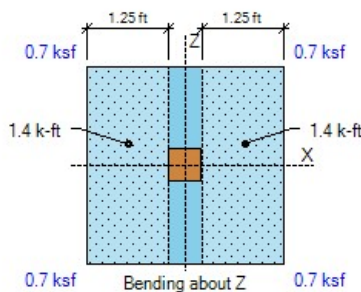
$$-Z \text{ Ld provided} = (\text{Width} - \text{Col}) / 2 + \text{Offset} - \text{Cover} = 3.00 * 12 / 2 + 0.0 - 6.0 / 2 - 2.5 = 12.5 \text{ in} > 12.0 \text{ in OK}$$

$$+Z \text{ Ld provided} = (\text{Width} - \text{Col}) / 2 - \text{Offset} - \text{Cover} = 3.00 * 12 / 2 - 0.0 - 6.0 / 2 - 2.5 = 12.5 \text{ in} > 12.0 \text{ in OK}$$

$$X\text{-bar spacing} = 7.5 \text{ in} < \text{Min} (3 * t, 18.0) = 18.0 \text{ in OK}$$

ACI 7.7.2.3

$$Z\text{-bar spacing} = 7.5 \text{ in} < \text{Min} (3 * t, 18.0) = 18.0 \text{ in OK}$$



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

$$\text{Area } A1 = \text{col } L * \text{col } W = 6.0 * 6.0 = 36.0 \text{ in}^2$$

$$Sx = \text{col } W * \text{col } L^2 / 6 = 6.0 * 6.0^2 / 6 = 36.0 \text{ in}^3$$

$$Sz = \text{col } L * \text{col } W^2 / 6 = 6.0 * 6.0^2 / 6 = 36.0 \text{ in}^3$$

$$\text{Bearing } Pbu = P / A1 + Mz / Sx + Mx / Sz = 23.6 / 36.0 + 0.0 * 12 / 36.0 + 0.0 * 12 / 36.0 = 0.7 \text{ ksi}$$

$$\text{Min edge} = \text{Min} (L / 2 - X\text{-offset} - \text{col } L / 2, W / 2 - Z\text{-offset} - \text{col } W / 2)$$

$$\text{Min edge} = \text{Min} (3.00 * 12 / 2 - 0.0 - 6.0 / 2, 3.00 * 12 / 2 - 0.0 - 6.0 / 2) = 15.0 \text{ in}$$

$$\text{Area } A2 = \text{Min} [L * W, (\text{col } L + 2 * \text{Min edge}) * (\text{col } W + 2 * \text{Min edge})]$$

ACI R22.8.3.2

$$A2 = \text{Min} [3.00 * 12 * 3.0 * 12, (6.0 + 2 * 15.0) * (6.0 + 2 * 15.0)] = 1296.0 \text{ in}^2$$

$$\text{Footing } \phi Pnc = \phi * 0.85 * f_c * \text{Min} [2, \sqrt{A2 / A1}] = 0.65 * 0.85 * 2.5 * \text{Min} [2, \sqrt{(1296.0 / 36.0)}] = 2.8 \text{ ksi}$$

$$\text{Footing } \phi Pns = \phi * As * Fy / A1 = 0.0 \text{ ksi}$$

ACI 22.8.3.2

$$\text{Footing bearing } \phi Pn = \phi Pnc + \phi Pns = 2.8 + 0.0 = 2.8 \text{ ksi} > 0.7 \text{ psi OK}$$

Hooked $L_{dh} = \text{Max} (8 \text{ db}, 6, 0.02 * f_y / (f_c)^{1/2} * \text{Confining} * \text{Location} * \text{Concrete} * \text{db} * \text{ratio})$

ACI 25.4.3

$$L_{dh} = \text{Max} (8 \text{ db}, 6, 0.02 * 60.0 * 1000 / (2500)^{1/2} * 1.0 * 0.7 * 0.0 * 0.75 * 0.11) = 6.0 \text{ in}$$

Ld provided = Dowel length = $3.00 * 12 = 36.0 \text{ in} > 20.4 \text{ in OK}$

Ldh provided = Footing thickness - Cover = $8.00 - 3.0 = 5.0 \text{ in} < 6.0 \text{ in NG}$

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

$$\text{X-Edge} = d/2 = 4.5 / 2 = 2.3 \text{ in} \quad \text{asx} = 20$$

$$\text{Z-Edge} = d/2 = 4.5 / 2 = 2.3 \text{ in} \quad \text{asz} = 20$$

$$\text{as} = \text{asx} + \text{asz} = 20 + 20 = 40 \quad \text{Col type} = \text{Interior} \quad \beta = L / W = 6.0 / 6.0 = 1.00$$

ACI 22.6.5.2

$$\text{Perimeter } b_o = \text{asx} / 10 * (L + d/2 + \text{X-Edge}) + \text{asx} / 10 * (W + d/2 + \text{Z-Edge})$$

ACI 22.6.4.2

$$b_o = 20 / 10 * (6.0 + 4.5 / 2 + 2.3) + 20 / 10 * (6.0 + 4.5 / 2 + 2.3) = 42.0 \text{ in}$$

$$\text{Area } A_{bo} = (L + d/2 + \text{X-Edge}) * (W + d/2 + \text{Z-Edge}) = (6.0 + 4.5 / 2 + 2.3) * (6.0 + 4.5 / 2 + 2.3) = 110.3 \text{ in}^2$$

$$\phi V_c = \phi * \text{Min} (2 + 4 / \beta, \text{as} * d / b_o + 2, 4) * \sqrt{f_c}$$

ACI 22.6.5.2

$$\phi V_c = 0.75 * \text{Min} (2 + 4 / 1.00, 40 * 4.5 / 42.0 + 2, 4) * \sqrt{2500} = 150.0 \text{ psi}$$

Punching force $F = P + \text{Overburden} * A_{bo} - \text{Bearing}$

$$F = 23.6 + 0.07 * 110.3 / 144 - 2.1 = 21.6 \text{ kip}$$

$$b1 = L + d/2 + \text{X-Edge} = 6.0 + 4.5 / 2 + 2.3 = 10.5 \text{ in} \quad b2 = W + d/2 + \text{Z-Edge} = 6.0 + 4.5 / 2 + 2.3 = 10.5 \text{ in}$$

$$\gamma_{vx} \text{ factor} = 1 - \frac{1}{1 + (2/3) \sqrt{b2/b1}} = 1 - \frac{1}{1 + (2/3) \sqrt{10.5/10.5}} = 0.40$$

ACI Eq. (8.4.4.2.2)

$$\gamma_{vz} \text{ factor} = 1 - \frac{1}{1 + (2/3) \sqrt{b1/b2}} = 1 - \frac{1}{1 + (2/3) \sqrt{10.5/10.5}} = 0.40$$

ACI Eq. (8.4.2.3.2)

$$X2z = b1/2 = 10.5/2 = 5.3 \text{ in} \quad X2x = b2/2 = 10.5/2 = 5.3 \text{ in}$$

$$J_{cz} = b1 * d^3 / 6 + b1^3 * d / 6 + b1^2 * b2 * d / 2$$

ACI R8.4.4.2.3

$$J_{cz} = 10.5 * 4.5^3 / 6 + 10.5^3 * 4.5 / 6 + 10.5^2 * 10.5 * 4.5 / 2 = 3632 \text{ in}^4$$

$$J_{cx} = b2 * d^3 / 6 + b2^3 * d / 6 + b2^2 * b1 * d / 2$$

ACI R8.4.4.2.3

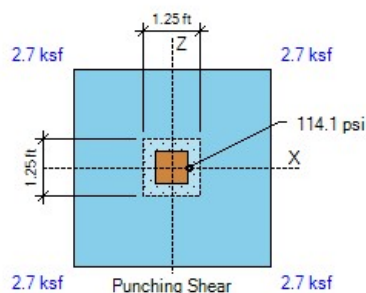
$$J_{cx} = 10.5 * 4.5^3 / 6 + 10.5^3 * 4.5 / 6 + 10.5^2 * 10.5 * 4.5 / 2 = 3632 \text{ in}^4$$

$$\text{Stress due to } P = F / (b_o * d) * 1000 = 21.6 / (42.0 * 4.5) * 1000 = 114.1 \text{ psi}$$

$$\text{Stress due to } M_x = \gamma_{vx} * X\text{-OTM} * X2x / J_{cx} = 0.40 * 0.0 * 12 * 5.3 / 3632 * 1000 = 0.0 \text{ psi}$$

$$\text{Stress due to } M_z = \gamma_{vz} * Z\text{-OTM} * X2z / J_{cz} = 0.40 * 0.0 * 12 * 5.3 / 3632 * 1000 = 0.0 \text{ psi}$$

$$\text{Punching stress} = P\text{-stress} + M_x\text{-stress} + M_z\text{-stress} = 114.1 + 0.0 + 0.0 = 114.1 \text{ psi} < 150.0 \text{ psi OK}$$



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

$$\text{Area } A1 = \text{col } L * \text{col } W = 6.0 * 6.0 = 36.0 \text{ in}^2$$

$$Sx = \text{col } W * \text{col } L^2 / 6 = 6.0 * 6.0^2 / 6 = 36.0 \text{ in}^3$$

$$Sz = \text{col } L * \text{col } W^2 / 6 = 6.0 * 6.0^2 / 6 = 36.0 \text{ in}^3$$

$$\text{Bearing } Pbu = P / A1 + Mz / Sx + Mx / Sz = 23.6 / 36.0 + 0.0 * 12 / 36.0 + 0.0 * 12 / 36.0 = 0.7 \text{ ksi}$$

$$\text{Min edge} = \text{Min} (L / 2 - X\text{-offset} - \text{col } L / 2, W / 2 - Z\text{-offset} - \text{col } W / 2)$$

$$\text{Min edge} = \text{Min} (3.00 * 12 / 2 - 0.0 - 6.0 / 2, 3.00 * 12 / 2 - 0.0 - 6.0 / 2) = 15.0 \text{ in}$$

$$\text{Area } A2 = \text{Min} [L * W, (\text{col } L + 2 * \text{Min edge}) * (\text{col } W + 2 * \text{Min edge})]$$

ACI R22.8.3.2

$$A2 = \text{Min} [3.00 * 12 * 3.00 * 12, (6.0 + 2 * 15.0) * (6.0 + 2 * 15.0)] = 1296.0 \text{ in}^2$$

$$\text{Footing } \phi Pnc = \phi * 0.85 * f'c * \text{Min} [2, \sqrt{A2 / A1}] = 0.65 * 0.85 * 2.5 * \text{Min} [2, \sqrt{(1296.0 / 36.0)}] = 2.8 \text{ ksi}$$

$$\text{Footing } \phi Pns = \phi * As * Fy / A1 = 0.0 \text{ ksi}$$

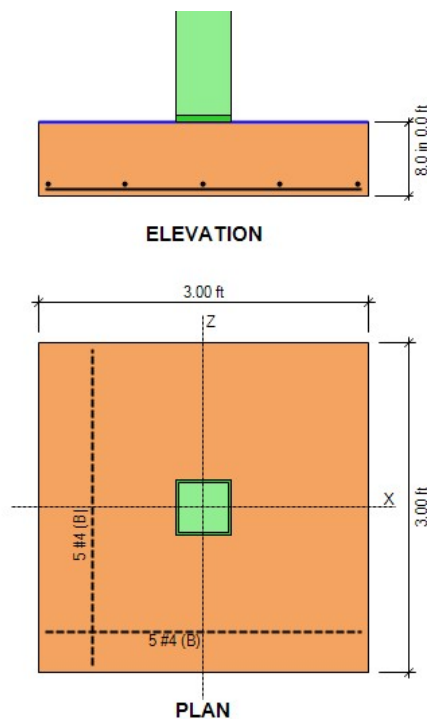
ACI 22.8.3.2

$$\text{Footing bearing } \phi Pn = \phi Pnc + \phi Pns = 2.8 + 0.0 = 2.8 \text{ ksi} > 0.7 \text{ psi OK}$$

DESIGN CODES

Concrete Design ACI 318-14

Load Combinations ASCE 7-10/16



GEOMETRY				SOIL PRESSURES (D+L)			
Footing Length (X-dir)	1.50	ft		Gross Allow. Soil Pressure	2.0	ksf	
Footing Width (Z-dir)	2.60	ft		Soil Pressure at Corner 1	2.0	ksf	
Footing Thickness	8.0	in	OK	Soil Pressure at Corner 2	2.0	ksf	
Soil Cover	0.00	ft		Soil Pressure at Corner 3	2.0	ksf	
Column Length (X-dir)	6.0	in		Soil Pressure at Corner 4	2.0	ksf	
Column Width (Z-dir)	6.0	in		Bearing Pressure Ratio	0.99	OK	
Offset (X-dir)	0.00	in	OK	Ftg. Area in Contact with Soil	100.0	%	
Offset (Z-dir)	0.00	in	OK	X-eccentricity / Ftg. Length	0.00	OK	
Base Plate (L x W)	6.0 x 6.0	in		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

$$\text{- Moment } M_x = 0.6 * 0.0 + 0.6 * 0.0 = 0.0 \text{ k-ft}$$

$$\text{- Shear Force } V_z = 0.6 * 0.0 + 0.6 * 0.0 = 0.0 \text{ kip}$$

$$\text{Arm} = 0.00 + 8.0 / 12 = 0.67 \text{ ft}$$

$$\text{Moment} = 0.0 * 0.67 = 0.0 \text{ k-ft}$$

$$\text{- Passive Force} = 0.0 \text{ kip}$$

$$\text{Arm} = 0.27 \text{ ft}$$

$$\text{Moment} = 0.0 \text{ k-ft}$$

$$\text{- Overturning moment } X-X = 0.0 + 0.0 = 0.0 \text{ k-ft}$$

- Resisting about X-X

$$\text{- Footing weight} = 0.6 * W * L * Thick * Density = 0.6 * 2.60 * 1.50 * 8.0 / 12 * 0.15 = 0.2 \text{ kip}$$

$$\text{Arm} = W / 2 = 2.60 / 2 = 1.30 \text{ ft}$$

$$\text{Moment} = 0.2 * 1.30 = 0.3 \text{ k-ft}$$

$$\text{- Pedestal weight} = 0.6 * W * L * H * Density = 0.6 * 6.0 / 12 * 6.0 / 12 * 0.0 * 0.15 = 0.0 \text{ kip}$$

$$\text{Arm} = W / 2 - Offset = 2.60 / 2 - 0.0 / 12 = 1.30 \text{ ft}$$

$$\text{Moment} = 0.0 * 1.30 = 0.0 \text{ k-ft}$$

$$\text{- Soil cover} = 0.6 * W * L * SC * Density = 0.6 * (2.60 * 1.50 - 6.0 / 12 * 6.0 / 12) * 0.0 * 110 = 0.0 \text{ kip}$$

$$\text{Arm} = W / 2 = 2.60 / 2 = 1.30 \text{ ft}$$

$$\text{Moment} = 0.0 * 1.30 = 0.0 \text{ k-ft}$$

$$\text{- Buoyancy} = 0.6 * W * L * \gamma * (SC + Thick - WT) = 0.6 * 2.60 * 1.50 * 62 * (0.67) = -0.1 \text{ kip}$$

$$\text{Arm} = W / 2 = 2.60 / 2 = 1.30 \text{ ft}$$

$$\text{Moment} = 0.1 * 1.30 = -0.1 \text{ k-ft}$$

$$\text{- Axial force } P = 0.6 * 3.0 + 0.6 * 0.0 = 1.8 \text{ kip}$$

$$\text{Arm} = W / 2 - Offset = 2.60 / 2 - 0.0 / 12 = 1.30 \text{ ft}$$

$$\text{Moment} = 1.8 * 1.30 = 2.3 \text{ k-ft}$$

$$\text{- Resisting moment } X-X = 0.3 + 0.0 + 0.0 + 2.3 + -0.1 = 2.5 \text{ k-ft}$$

$$\text{- Overturning safety factor } X-X = \frac{\text{Resisting moment}}{\text{Overturning moment}} = \frac{2.5}{0.0} = 25.18 > 1.50 \text{ OK}$$

- Overturning about Z-Z

$$\text{- Moment } M_z = 0.6 * 0.0 + 0.6 * 0.0 = 0.0 \text{ k-ft}$$

$$\text{- Shear Force } V_x = 0.6 * 0.0 + 0.6 * 0.0 = 0.0 \text{ kip}$$

$$\text{Arm} = 0.00 + 8.0 / 12 = 0.67 \text{ ft}$$

$$\text{Moment} = 0.0 * 0.67 = 0.0 \text{ k-ft}$$

$$\text{- Passive Force} = 0.0 \text{ kip}$$

$$\text{Arm} = 0.27 \text{ ft}$$

$$\text{Moment} = 0.0 \text{ k-ft}$$

$$\text{- Overturning moment Z-Z} = 0.0 + 0.0 = 0.0 \text{ k-ft}$$

- Resisting about Z-Z

$$\text{- Footing weight} = 0.6 * W * L * Thick * Density = 0.6 * 2.60 * 1.50 * 8.0 / 12 * 0.15 = 0.2 \text{ kip}$$

$$\text{Arm} = L / 2 = 1.50 / 2 = 0.75 \text{ ft}$$

$$\text{Moment} = 0.2 * 0.75 = 0.2 \text{ k-ft}$$

$$\text{- Pedestal weight} = 0.6 * W * L * H * Density = 0.6 * 6.0 / 12 * 6.0 / 12 * 0.0 * 0.15 = 0.0 \text{ kip}$$

$$\text{Arm} = L / 2 - Offset = 1.50 / 2 - 0.0 / 12 = 0.75 \text{ ft}$$

$$\text{Moment} = 0.0 * 0.75 = 0.0 \text{ k-ft}$$

$$\text{- Soil cover} = 0.6 * W * L * SC * Density = 0.6 * (2.60 * 1.50 - 6.0 / 12 * 6.0 / 12) * 0.0 * 110 = 0.0 \text{ kip}$$

$$\text{Arm} = L / 2 = 1.50 / 2 = 0.75 \text{ ft}$$

$$\text{Moment} = 0.0 * 0.75 = 0.0 \text{ k-ft}$$

$$\text{- Buoyancy} = 0.6 * W * L * \gamma * (SC + Thick - WT) = 0.6 * 2.60 * 1.50 * 62 * (0.67) = -0.1 \text{ kip}$$

$$\text{Arm} = L / 2 = 1.50 / 2 = 0.75 \text{ ft}$$

$$\text{Moment} = 0.1 * 0.75 = -0.1 \text{ k-ft}$$

$$\text{- Axial force } P = 0.6 * 3.0 + 0.6 * 0.0 = 1.8 \text{ kip}$$

$$\text{Arm} = L / 2 - Offset = 1.50 / 2 - 0.0 / 12 = 0.75 \text{ ft}$$

$$\text{Moment} = 1.8 * 0.75 = 1.4 \text{ k-ft}$$

$$\text{- Resisting moment Z-Z} = 0.2 + 0.0 + 0.0 + 1.4 + -0.1 = 1.5 \text{ k-ft}$$

$$\text{- Overturning safety factor Z-Z} = \frac{\text{Resisting moment}}{\text{Overturning moment}} = \frac{1.5}{0.0} = 14.52 > 1.50 \text{ OK}$$

SOIL BEARING PRESSURES (Comb: D+L)

$$\text{Overturning moment X-X} = 0.0 + 0.0 = 0.0 \text{ k-ft}$$

$$\text{Resisting moment X-X} = 0.5 + 0.0 + 0.0 + -0.2 + 9.8 = 10.0 \text{ k-ft}$$

$$\text{Overturning moment Z-Z} = 0.0 + 0.0 = 0.0 \text{ k-ft}$$

$$\text{Resisting moment Z-Z} = 0.3 + 0.0 + 0.0 + -0.1 + 5.6 = 5.8 \text{ k-ft}$$

$$\text{Resisting force} = \text{Footing} + \text{Pedestal} + \text{Soil} - \text{Buoyancy} + P = 0.4 + 0.0 + 0.0 - 0.2 + 7.5 = 7.7 \text{ kip}$$

X-coordinate of resultant from maximum bearing corner:

$$X_p = \frac{Z\text{-Resisting moment} - Z\text{-Overturning moment}}{\text{Resisting force}} = \frac{5.8 - 0.0}{7.7} = 0.75 \text{ ft}$$

Z-coordinate of resultant from maximum bearing corner:

$$Z_p = \frac{X\text{-Resisting moment} - X\text{-Overturning moment}}{\text{Resisting force}} = \frac{10.0 - 0.0}{7.7} = 1.30 \text{ ft}$$

$$X\text{-ecc} = \text{Length} / 2 - X_p = 1.50 / 2 - 0.75 = 0.00 \text{ ft}$$

$$Z\text{-ecc} = \text{Width} / 2 - Z_p = 2.60 / 2 - 1.30 = 0.00 \text{ ft}$$

$$\text{Area} = \text{Width} * \text{Length} = 2.60 * 1.50 = 3.9 \text{ ft}^2$$

$$S_x = \text{Length} * \text{Width}^2 / 6 = 1.50 * 2.60^2 / 6 = 1.7 \text{ ft}^3$$

$$S_z = \text{Width} * \text{Length}^2 / 6 = 2.60 * 1.50^2 / 6 = 1.0 \text{ ft}^3$$

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

$$P1 = P * (1/A + Z\text{-ecc} / S_x + X\text{-ecc} / S_z) = 7.7 * (1 / 3.9 + 0.00 / 1.7 + 0.00 / 1.0) = 1.98 \text{ ksf}$$

$$P2 = P * (1/A - Z\text{-ecc} / S_x + X\text{-ecc} / S_z) = 7.7 * (1 / 3.9 - 0.00 / 1.7 + 0.00 / 1.0) = 1.98 \text{ ksf}$$

$$P3 = P * (1/A - Z\text{-ecc} / S_x - X\text{-ecc} / S_z) = 7.7 * (1 / 3.9 - 0.00 / 1.7 - 0.00 / 1.0) = 1.98 \text{ ksf}$$

$$P4 = P * (1/A + Z\text{-ecc} / S_x - X\text{-ecc} / S_z) = 7.7 * (1 / 3.9 + 0.00 / 1.7 - 0.00 / 1.0) = 1.98 \text{ ksf}$$



SLIDING CALCULATIONS (Comb: 0.6D+0.6W)

Internal friction angle = 28.0 deg

Passive coefficient $k_p = 4.33$ (per Coulomb)Pressure at mid-depth = $k_p \cdot \text{Density} \cdot (\text{Cover} + \text{Thick} / 2) = 4.33 \cdot 110 \cdot (0.00 + 8.0 / 12 / 2) = 0.16$ ksfX-Passive force = $\text{Pressure} \cdot \text{Thick} \cdot \text{Width} = 0.16 \cdot 8.0 / 12 \cdot 2.60 = 0.3$ kipZ-Passive force = $\text{Pressure} \cdot \text{Thick} \cdot \text{Length} = 0.16 \cdot 8.0 / 12 \cdot 1.50 = 0.2$ kipFriction force = $\text{Resisting force} \cdot \text{Friction coeff.} = \text{Max}(0, 1.9 \cdot 0.35) = 0.7$ kip

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

$$\text{- Sliding safety factor X-X} = \frac{\text{X-Passive force} + \text{Friction}}{\text{X-Horizontal load}} = \frac{1.00 \cdot 0.3 + 1.00 \cdot 0.7}{0.0} = 9.53 > 1.50 \text{ OK}$$

$$\text{- Sliding safety factor Z-Z} = \frac{\text{Z-Passive force} + \text{Friction}}{\text{Z-Horizontal load}} = \frac{1.00 \cdot 0.2 + 1.00 \cdot 0.7}{0.0} = 8.36 > 1.50 \text{ OK}$$

UPLIFT CALCULATIONS (Comb: 0.6D+0.6W)

$$\text{- Uplift safety factor} = \frac{\text{Pedestal} + \text{Footing} + \text{Cover} - \text{Buoyancy}}{\text{Uplift load}} = \frac{0.0 + 0.2 + 0.0 - 0.1}{0.0} = 99.99 > 1.00 \text{ OK}$$

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

Concrete $f'_c = 2.5$ ksiSteel $f_y = 40.0$ ksi

Soil density = 110 pcf

Use Plain Concrete Shear Strength

$$\phi V_{cx} = \frac{4}{3} \cdot \phi \cdot \sqrt{f'_c} \cdot \text{Width} \cdot t / 1000 = \frac{4}{3} \cdot 0.60 \cdot \sqrt{(2500)} \cdot 2.6 \cdot 12 \cdot 8.0 / 1000 = 10.0 \text{ kip}$$

ACI 14.5.5.1

$$\phi V_{cz} = \frac{4}{3} \cdot \phi \cdot \sqrt{f'_c} \cdot \text{Length} \cdot t / 1000 = \frac{4}{3} \cdot 0.60 \cdot \sqrt{(2500)} \cdot 1.5 \cdot 12 \cdot 8.0 / 1000 = 5.8 \text{ kip}$$

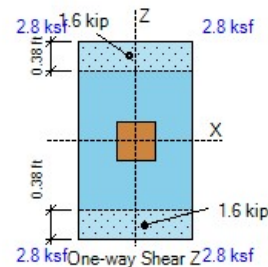
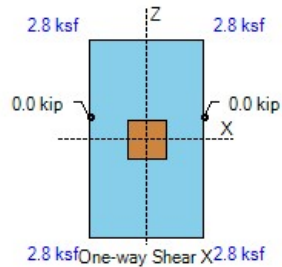
- Shear forces calculated as the volume of the bearing pressures under the effective areas:

$$\text{One-way shear } V_{ux} \text{ (- Side)} = 0.0 \text{ kip} < 10.0 \text{ kip OK}$$

$$\text{One-way shear } V_{ux} \text{ (+ Side)} = 0.0 \text{ kip} < 10.0 \text{ kip OK}$$

$$\text{One-way shear } V_{uz} \text{ (- Side)} = 1.6 \text{ kip} < 5.8 \text{ kip OK}$$

$$\text{One-way shear } V_{uz} \text{ (+ Side)} = 1.6 \text{ kip} < 5.8 \text{ kip OK}$$



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

$$\text{Plain } \phi M_{nx} = 5 * \phi * \sqrt{f_c} * L * \text{Thick}^2 / 6 = 5 * 0.60 * \sqrt{(2500)} * 1.50 * 8.0^2 / 6 / 1000 = 0.6 \text{ k-ft}$$

ACI Eq. (14.5.2.1a)

$$\text{Plain } \phi M_{nz} = 5 * \phi * \sqrt{f_c} * W * \text{Thick}^2 / 6 = 5 * 0.60 * \sqrt{(2500)} * 2.60 * 8.0^2 / 6 / 1000 = 1.1 \text{ k-ft}$$

- 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:

$$\text{Top moment -Mux (- Side)} = 0.0 \text{ k-ft} < 2.4 \text{ k-ft OK}$$

$$\text{Top moment -Mux (+ Side)} = 0.0 \text{ k-ft} < 2.4 \text{ k-ft OK}$$

$$\text{Top moment -Muz (- Side)} = 0.0 \text{ k-ft} < 4.2 \text{ k-ft OK}$$

$$\text{Top moment -Muz (+ Side)} = 0.0 \text{ k-ft} < 4.2 \text{ k-ft OK}$$

- Bottom Bars

No Bottom Reinforcement Provided at the Footing

Use Plain Concrete Flexural Strength at Bottom

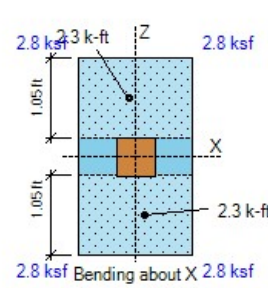
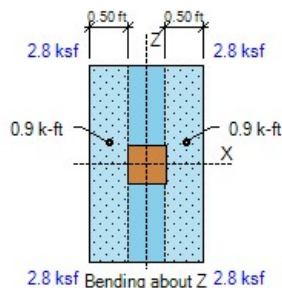
- Bottom moments calculated as the bearing minus the overburden pressures times the lever arm:

$$\text{Bottom moment Mux (- Side)} = 2.3 \text{ k-ft} < 2.4 \text{ k-ft OK} \quad \text{ratio} = 0.96$$

$$\text{Bottom moment Mux (+ Side)} = 2.3 \text{ k-ft} < 2.4 \text{ k-ft OK} \quad \text{ratio} = 0.96$$

$$\text{Bottom moment Muz (- Side)} = 0.9 \text{ k-ft} < 4.2 \text{ k-ft OK} \quad \text{ratio} = 0.22$$

$$\text{Bottom moment Muz (+ Side)} = 0.9 \text{ k-ft} < 4.2 \text{ k-ft OK} \quad \text{ratio} = 0.22$$



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

$$\text{Area } A1 = \text{col } L * \text{col } W = 6.0 * 6.0 = 36.0 \text{ in}^2$$

$$Sx = \text{col } W * \text{col } L^2 / 6 = 6.0 * 6.0^2 / 6 = 36.0 \text{ in}^3$$

$$Sz = \text{col } L * \text{col } W^2 / 6 = 6.0 * 6.0^2 / 6 = 36.0 \text{ in}^3$$

$$\text{Bearing } Pbu = P / A1 + Mz / Sx + Mx / Sz = 10.8 / 36.0 + 0.0 * 12 / 36.0 + 0.0 * 12 / 36.0 = 0.3 \text{ ksi}$$

$$\text{Min edge} = \text{Min} (L / 2 - X\text{-offset} - \text{col } L / 2, W / 2 - Z\text{-offset} - \text{col } W / 2)$$

$$\text{Min edge} = \text{Min} (1.50 * 12 / 2 - 0.0 - 6.0 / 2, 2.60 * 12 / 2 - 0.0 - 6.0 / 2) = 6.0 \text{ in}$$

$$\text{Area } A2 = \text{Min} [L * W, (\text{col } L + 2 * \text{Min edge}) * (\text{col } W + 2 * \text{Min edge})]$$

ACI R22.8.3.2

$$A2 = \text{Min} [1.50 * 12 * 2.6 * 12, (6.0 + 2 * 6.0) * (6.0 + 2 * 6.0)] = 324.0 \text{ in}^2$$

$$\text{Footing } \phi Pnc = \phi * 0.85 * f'c * \text{Min} [2, \sqrt{(A2 / A1)}] = 0.65 * 0.85 * 2.5 * \text{Min} [2, \sqrt{(324.0 / 36.0)}] = 2.8 \text{ ksi}$$

$$\text{Footing } \phi Pns = \phi * As * Fy / A1 = 0.0 \text{ ksi}$$

ACI 22.8.3.2

$$\text{Footing bearing } \phi Pn = \phi Pnc + \phi Pns = 2.8 + 0.0 = 2.8 \text{ ksi} > 0.3 \text{ psi OK}$$

Hooked $L_{dh} = \text{Max} (8 \text{ db}, 6, 0.02 * f_y / (f_c)^{1/2} * \text{Confining} * \text{Location} * \text{Concrete} * \text{db} * \text{ratio})$

ACI 25.4.3

$$L_{dh} = \text{Max} (8 \text{ db}, 6, 0.02 * 60.0 * 1000 / (2500)^{1/2} * 1.0 * 0.7 * 0.0 * 0.75 * 0.05) = 6.0 \text{ in}$$

Ld provided = Dowel length = $3.00 * 12 = 36.0 \text{ in} > 12.0 \text{ in OK}$

Ldh provided = Footing thickness - Cover = $8.00 - 3.0 = 5.0 \text{ in} < 6.0 \text{ in NG}$

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

$$\text{X-Edge} = \text{Length} / 2 - \text{Offset} - \text{Col} / 2 = 1.50 * 12 / 2 - 0.0 - 6.0 / 2 = 6.0 \text{ in} \quad \alpha_{sx} = 10$$

$$\text{Z-Edge} = \text{Width} / 2 - \text{Offset} - \text{Col} / 2 = 2.60 * 12 / 2 - 0.0 - 6.0 / 2 = 12.6 \text{ in} \quad \alpha_{sz} = 10$$

$$\alpha_s = \alpha_{sx} + \alpha_{sz} = 10 + 10 = 20 \quad \text{Col type} = \text{Corner} \quad \beta = L / W = 6.0 / 6.0 = 1.00$$

ACI 22.6.5.2

$$\text{Perimeter } b_o = \alpha_{sz} / 10 * (L + d / 2 + \text{X-Edge}) + \alpha_{sx} / 10 * (W + d / 2 + \text{Z-Edge})$$

ACI 22.6.4.2

$$b_o = 10 / 10 * (6.0 + 8.0 / 2 + 6.0) + 10 / 10 * (6.0 + 8.0 / 2 + 12.6) = 38.6 \text{ in}$$

$$\text{Area } A_{bo} = (L + d / 2 + \text{X-Edge}) * (W + d / 2 + \text{Z-Edge}) = (6.0 + 8.0 / 2 + 6.0) * (6.0 + 8.0 / 2 + 12.6) = 361.6 \text{ in}^2$$

Use Plain Concrete Shear Strength

$$\phi V_c = \phi * \text{Min} (1 + 2 / \beta, 2) * 4/3 * \sqrt{f_c}$$

ACI 14.5.5.1

$$\phi V_c = 0.60 * \text{Min} (1 + 2 / 1.00, 2) * 4/3 * \sqrt{2500} = 80.0 \text{ psi}$$

Punching force $F = P + \text{Overburden} * A_{bo} - \text{Bearing}$

$$F = 10.8 + 0.07 * 361.6 / 144 - 3.9 = 7.1 \text{ kip}$$

$$b_1 = L + d / 2 + \text{X-Edge} = 6.0 + 8.0 / 2 + 6.0 = 16.0 \text{ in} \quad b_2 = W + d / 2 + \text{Z-Edge} = 6.0 + 8.0 / 2 + 12.6 = 22.6 \text{ in}$$

$$\gamma_{vx} \text{ factor} = 1 - \frac{1}{1 + (2/3) \sqrt{b_2 / b_1}} = 1 - \frac{1}{1 + (2/3) \sqrt{22.6 / 16.0}} = 0.44$$

ACI Eq. (8.4.4.2.2)

$$\gamma_{vz} \text{ factor} = 1 - \frac{1}{1 + (2/3) \sqrt{b_1 / b_2}} = 1 - \frac{1}{1 + (2/3) \sqrt{16.0 / 22.6}} = 0.36$$

ACI Eq. (8.4.2.3.2)

$$X_{2z} = b_1^2 / 2 / (b_1 + b_2) = 16.0^2 / 2 / (16.0 + 22.6) = 3.3 \text{ in} \quad X_{2x} = b_2^2 / 2 / (b_2 + b_1) = 6.6 \text{ in}$$

$$J_{cz} = b_1 * d^3 / 12 + b_1^3 * d / 12 + b_1 * d * (b_1 / 2 - X_{2z})^2 + b_2 * d * X_{2z}^2$$

ACI R8.4.4.2.3

$$J_{cz} = 16.0 * 8.0^3 / 12 + 16.0^3 * 8.0 / 12 + 16.0 * 8.0 * (16.0 / 2 - 3.3)^2 + 22.6 * 8.0 * 3.3^2 = 8210 \text{ in}^4$$

$$J_{cx} = b_2 * d^3 / 12 + b_2^3 * d / 12 + b_2 * d * (b_2 / 2 - X_{2x})^2 + b_1 * d * X_{2x}^2$$

ACI R8.4.4.2.3

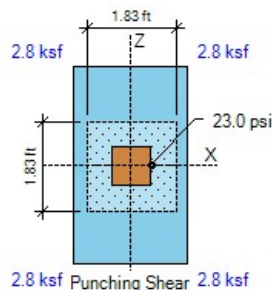
$$J_{cx} = 22.6 * 8.0^3 / 12 + 22.6^3 * 8.0 / 12 + 22.6 * 8.0 * (22.6 / 2 - 6.6)^2 + 16.0 * 8.0 * 6.6^2 = 18229 \text{ in}^4$$

$$\text{Stress due to } P = F / (b_o * d) * 1000 = 7.1 / (38.6 * 8.0) * 1000 = 23.0 \text{ psi}$$

$$\text{Stress due to } M_x = \gamma_{vx} * X\text{-OTM} * X_{2x} / J_{cx} = 0.44 * 0.0 * 12 * 6.6 / 18229 * 1000 = 0.0 \text{ psi}$$

$$\text{Stress due to } M_z = \gamma_{vz} * Z\text{-OTM} * X_{2z} / J_{cz} = 0.44 * 0.0 * 12 * 3.3 / 8210 * 1000 = 0.0 \text{ psi}$$

$$\text{Punching stress} = P\text{-stress} + M_x\text{-stress} + M_z\text{-stress} = 23.0 + 0.0 + 0.0 = 23.0 \text{ psi} < 80.0 \text{ psi OK}$$



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

$$\text{Area } A1 = \text{col } L * \text{col } W = 6.0 * 6.0 = 36.0 \text{ in}^2$$

$$Sx = \text{col } W * \text{col } L^2 / 6 = 6.0 * 6.0^2 / 6 = 36.0 \text{ in}^3$$

$$Sz = \text{col } L * \text{col } W^2 / 6 = 6.0 * 6.0^2 / 6 = 36.0 \text{ in}^3$$

$$\text{Bearing } Pbu = P / A1 + Mz / Sx + Mx / Sz = 10.8 / 36.0 + 0.0 * 12 / 36.0 + 0.0 * 12 / 36.0 = 0.3 \text{ ksi}$$

$$\text{Min edge} = \text{Min} (L / 2 - X\text{-offset} - \text{col } L / 2, W / 2 - Z\text{-offset} - \text{col } W / 2)$$

$$\text{Min edge} = \text{Min} (1.50 * 12 / 2 - 0.0 - 6.0 / 2, 2.60 * 12 / 2 - 0.0 - 6.0 / 2) = 6.0 \text{ in}$$

$$\text{Area } A2 = \text{Min} [L * W, (\text{col } L + 2 * \text{Min edge}) * (\text{col } W + 2 * \text{Min edge})]$$

ACI R22.8.3.2

$$A2 = \text{Min} [1.50 * 12 * 2.6 * 12, (6.0 + 2 * 6.0) * (6.0 + 2 * 6.0)] = 324.0 \text{ in}^2$$

$$\text{Footing } \phi Pnc = \phi * 0.85 * f'c * \text{Min} [2, \sqrt{A2 / A1}] = 0.65 * 0.85 * 2.5 * \text{Min} [2, \sqrt{(324.0 / 36.0)}] = 2.8 \text{ ksi}$$

$$\text{Footing } \phi Pns = \phi * As * Fy / A1 = 0.0 \text{ ksi}$$

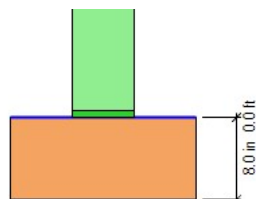
ACI 22.8.3.2

$$\text{Footing bearing } \phi Pn = \phi Pnc + \phi Pns = 2.8 + 0.0 = 2.8 \text{ ksi} > 0.3 \text{ psi OK}$$

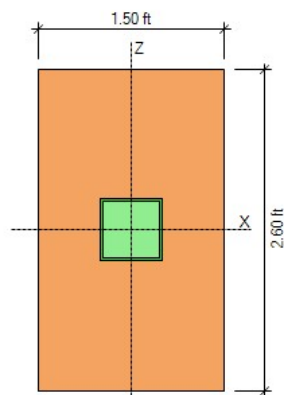
DESIGN CODES

Concrete Design ACI 318-14

Load Combinations ASCE 7-10/16



ELEVATION



PLAN

GEOMETRY				SOIL PRESSURES (D+L)			
Footing Length (X-dir)	2.00	ft		Gross Allow. Soil Pressure	2.0	ksf	
Footing Width (Z-dir)	2.60	ft		Soil Pressure at Corner 1	2.0	ksf	
Footing Thickness	8.0	in	OK	Soil Pressure at Corner 2	2.0	ksf	
Soil Cover	0.00	ft		Soil Pressure at Corner 3	2.0	ksf	
Column Length (X-dir)	6.0	in		Soil Pressure at Corner 4	2.0	ksf	
Column Width (Z-dir)	6.0	in		Bearing Pressure Ratio	0.99	OK	
Offset (X-dir)	0.00	in	OK	Ftg. Area in Contact with Soil	100.0	%	
Offset (Z-dir)	0.00	in	OK	X-eccentricity / Ftg. Length	0.00	OK	
Base Plate (L x W)	6.0 x 6.0	in		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

$$\text{- Moment } M_x = 0.6 * 0.0 + 0.6 * 0.0 = 0.0 \text{ k-ft}$$

$$\text{- Shear Force } V_z = 0.6 * 0.0 + 0.6 * 0.0 = 0.0 \text{ kip}$$

$$\text{Arm} = 0.00 + 8.0 / 12 = 0.67 \text{ ft}$$

$$\text{Moment} = 0.0 * 0.67 = 0.0 \text{ k-ft}$$

$$\text{- Passive Force} = 0.0 \text{ kip}$$

$$\text{Arm} = 0.27 \text{ ft}$$

$$\text{Moment} = 0.0 \text{ k-ft}$$

$$\text{- Overturning moment } X-X = 0.0 + 0.0 = 0.0 \text{ k-ft}$$

- Resisting about X-X

$$\text{- Footing weight} = 0.6 * W * L * Thick * Density = 0.6 * 2.60 * 2.00 * 8.0 / 12 * 0.15 = 0.3 \text{ kip}$$

$$\text{Arm} = W / 2 = 2.60 / 2 = 1.30 \text{ ft}$$

$$\text{Moment} = 0.3 * 1.30 = 0.4 \text{ k-ft}$$

$$\text{- Pedestal weight} = 0.6 * W * L * H * Density = 0.6 * 6.0 / 12 * 6.0 / 12 * 0.0 * 0.15 = 0.0 \text{ kip}$$

$$\text{Arm} = W / 2 - Offset = 2.60 / 2 - 0.0 / 12 = 1.30 \text{ ft}$$

$$\text{Moment} = 0.0 * 1.30 = 0.0 \text{ k-ft}$$

$$\text{- Soil cover} = 0.6 * W * L * SC * Density = 0.6 * (2.60 * 2.00 - 6.0 / 12 * 6.0 / 12) * 0.0 * 110 = 0.0 \text{ kip}$$

$$\text{Arm} = W / 2 = 2.60 / 2 = 1.30 \text{ ft}$$

$$\text{Moment} = 0.0 * 1.30 = 0.0 \text{ k-ft}$$

$$\text{- Buoyancy} = 0.6 * W * L * \gamma * (SC + Thick - WT) = 0.6 * 2.60 * 2.00 * 62 * (0.67) = -0.1 \text{ kip}$$

$$\text{Arm} = W / 2 = 2.60 / 2 = 1.30 \text{ ft}$$

$$\text{Moment} = 0.1 * 1.30 = -0.2 \text{ k-ft}$$

$$\text{- Axial force } P = 0.6 * 4.5 + 0.6 * 0.0 = 2.7 \text{ kip}$$

$$\text{Arm} = W / 2 - Offset = 2.60 / 2 - 0.0 / 12 = 1.30 \text{ ft}$$

$$\text{Moment} = 2.7 * 1.30 = 3.5 \text{ k-ft}$$

$$\text{- Resisting moment } X-X = 0.4 + 0.0 + 0.0 + 3.5 + -0.2 = 3.7 \text{ k-ft}$$

$$\text{- Overturning safety factor } X-X = \frac{\text{Resisting moment}}{\text{Overturning moment}} = \frac{3.7}{0.0} = 37.47 > 1.50 \text{ OK}$$

- Overturning about Z-Z

$$\text{- Moment } M_z = 0.6 * 0.0 + 0.6 * 0.0 = 0.0 \text{ k-ft}$$

$$\text{- Shear Force } V_x = 0.6 * 0.0 + 0.6 * 0.0 = 0.0 \text{ kip}$$

$$\text{Arm} = 0.00 + 8.0 / 12 = 0.67 \text{ ft}$$

$$\text{Moment} = 0.0 * 0.67 = 0.0 \text{ k-ft}$$

$$\text{- Passive Force} = 0.0 \text{ kip}$$

$$\text{Arm} = 0.27 \text{ ft}$$

$$\text{Moment} = 0.0 \text{ k-ft}$$

$$\text{- Overturning moment Z-Z} = 0.0 + 0.0 = 0.0 \text{ k-ft}$$

- Resisting about Z-Z

$$\text{- Footing weight} = 0.6 * W * L * Thick * Density = 0.6 * 2.60 * 2.00 * 8.0 / 12 * 0.15 = 0.3 \text{ kip}$$

$$\text{Arm} = L / 2 = 2.00 / 2 = 1.00 \text{ ft}$$

$$\text{Moment} = 0.3 * 1.00 = 0.3 \text{ k-ft}$$

$$\text{- Pedestal weight} = 0.6 * W * L * H * Density = 0.6 * 6.0 / 12 * 6.0 / 12 * 0.0 * 0.15 = 0.0 \text{ kip}$$

$$\text{Arm} = L / 2 - Offset = 2.00 / 2 - 0.0 / 12 = 1.00 \text{ ft}$$

$$\text{Moment} = 0.0 * 1.00 = 0.0 \text{ k-ft}$$

$$\text{- Soil cover} = 0.6 * W * L * SC * Density = 0.6 * (2.60 * 2.00 - 6.0 / 12 * 6.0 / 12) * 0.0 * 110 = 0.0 \text{ kip}$$

$$\text{Arm} = L / 2 = 2.00 / 2 = 1.00 \text{ ft}$$

$$\text{Moment} = 0.0 * 1.00 = 0.0 \text{ k-ft}$$

$$\text{- Buoyancy} = 0.6 * W * L * \gamma * (SC + Thick - WT) = 0.6 * 2.60 * 2.00 * 62 * (0.67) = -0.1 \text{ kip}$$

$$\text{Arm} = L / 2 = 2.00 / 2 = 1.00 \text{ ft}$$

$$\text{Moment} = 0.1 * 1.00 = -0.1 \text{ k-ft}$$

$$\text{- Axial force } P = 0.6 * 4.5 + 0.6 * 0.0 = 2.7 \text{ kip}$$

$$\text{Arm} = L / 2 - Offset = 2.00 / 2 - 0.0 / 12 = 1.00 \text{ ft}$$

$$\text{Moment} = 2.7 * 1.00 = 2.7 \text{ k-ft}$$

$$\text{- Resisting moment Z-Z} = 0.3 + 0.0 + 0.0 + 2.7 + -0.1 = 2.9 \text{ k-ft}$$

$$\text{- Overturning safety factor Z-Z} = \frac{\text{Resisting moment}}{\text{Overturning moment}} = \frac{2.9}{0.0} = 28.82 > 1.50 \text{ OK}$$

SOIL BEARING PRESSURES (Comb: D+L)

$$\text{Overturning moment X-X} = 0.0 + 0.0 = 0.0 \text{ k-ft}$$

$$\text{Resisting moment X-X} = 0.7 + 0.0 + 0.0 + -0.3 + 13.0 = 13.4 \text{ k-ft}$$

$$\text{Overturning moment Z-Z} = 0.0 + 0.0 = 0.0 \text{ k-ft}$$

$$\text{Resisting moment Z-Z} = 0.5 + 0.0 + 0.0 + -0.2 + 10.0 = 10.3 \text{ k-ft}$$

$$\text{Resisting force} = \text{Footing} + \text{Pedestal} + \text{Soil} - \text{Buoyancy} + P = 0.5 + 0.0 + 0.0 - 0.2 + 10.0 = 10.3 \text{ kip}$$

X-coordinate of resultant from maximum bearing corner:

$$X_p = \frac{Z\text{-Resisting moment} - Z\text{-Overturning moment}}{\text{Resisting force}} = \frac{10.3 - 0.0}{10.3} = 1.00 \text{ ft}$$

Z-coordinate of resultant from maximum bearing corner:

$$Z_p = \frac{X\text{-Resisting moment} - X\text{-Overturning moment}}{\text{Resisting force}} = \frac{13.4 - 0.0}{10.3} = 1.30 \text{ ft}$$

$$X\text{-ecc} = \text{Length} / 2 - X_p = 2.00 / 2 - 1.00 = 0.00 \text{ ft}$$

$$Z\text{-ecc} = \text{Width} / 2 - Z_p = 2.60 / 2 - 1.30 = 0.00 \text{ ft}$$

$$\text{Area} = \text{Width} * \text{Length} = 2.60 * 2.00 = 5.2 \text{ ft}^2$$

$$S_x = \text{Length} * \text{Width}^2 / 6 = 2.00 * 2.60^2 / 6 = 2.3 \text{ ft}^3$$

$$S_z = \text{Width} * \text{Length}^2 / 6 = 2.60 * 2.00^2 / 6 = 1.7 \text{ ft}^3$$

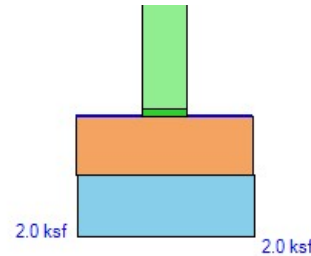
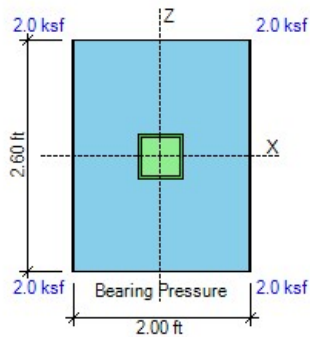
- Footing is in full bearing. Soil pressures are as follows:

$$P1 = P * (1/A + Z\text{-ecc} / S_x + X\text{-ecc} / S_z) = 10.3 * (1/5.2 + 0.00 / 2.3 + 0.00 / 1.7) = 1.98 \text{ ksf}$$

$$P2 = P * (1/A - Z\text{-ecc} / S_x + X\text{-ecc} / S_z) = 10.3 * (1/5.2 - 0.00 / 2.3 + 0.00 / 1.7) = 1.98 \text{ ksf}$$

$$P3 = P * (1/A - Z\text{-ecc} / S_x - X\text{-ecc} / S_z) = 10.3 * (1/5.2 - 0.00 / 2.3 - 0.00 / 1.7) = 1.98 \text{ ksf}$$

$$P4 = P * (1/A + Z\text{-ecc} / S_x - X\text{-ecc} / S_z) = 10.3 * (1/5.2 + 0.00 / 2.3 - 0.00 / 1.7) = 1.98 \text{ ksf}$$



SLIDING CALCULATIONS (Comb: 0.6D+0.6W)

Internal friction angle = 28.0 deg

Passive coefficient $k_p = 4.33$ (per Coulomb)Pressure at mid-depth = $k_p \cdot \text{Density} \cdot (\text{Cover} + \text{Thick} / 2) = 4.33 \cdot 110 \cdot (0.00 + 8.0 / 12 / 2) = 0.16$ ksfX-Passive force = $\text{Pressure} \cdot \text{Thick} \cdot \text{Width} = 0.16 \cdot 8.0 / 12 \cdot 2.60 = 0.3$ kipZ-Passive force = $\text{Pressure} \cdot \text{Thick} \cdot \text{Length} = 0.16 \cdot 8.0 / 12 \cdot 2.00 = 0.2$ kipFriction force = $\text{Resisting force} \cdot \text{Friction coeff.} = \text{Max}(0, 2.9 \cdot 0.35) = 1.0$ kip

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

$$\text{- Sliding safety factor X-X} = \frac{\text{X-Passive force} + \text{Friction}}{\text{X-Horizontal load}} = \frac{1.00 \cdot 0.3 + 1.00 \cdot 1.0}{0.0} = 12.84 > 1.50 \quad \text{OK}$$

$$\text{- Sliding safety factor Z-Z} = \frac{\text{Z-Passive force} + \text{Friction}}{\text{Z-Horizontal load}} = \frac{1.00 \cdot 0.2 + 1.00 \cdot 1.0}{0.0} = 12.20 > 1.50 \quad \text{OK}$$

UPLIFT CALCULATIONS (Comb: 0.6D+0.6W)

$$\text{- Uplift safety factor} = \frac{\text{Pedestal} + \text{Footing} + \text{Cover} - \text{Buoyancy}}{\text{Uplift load}} = \frac{0.0 + 0.3 + 0.0 - 0.1}{0.0} = 99.99 > 1.00 \quad \text{OK}$$

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

Concrete $f'_c = 2.5$ ksiSteel $f_y = 40.0$ ksi

Soil density = 110 pcf

Use Plain Concrete Shear Strength

$$\phi V_{cx} = \frac{4}{3} \cdot \phi \cdot \sqrt{f'_c} \cdot \text{Width} \cdot t / 1000 = \frac{4}{3} \cdot 0.60 \cdot \sqrt{2500} \cdot 2.6 \cdot 12 \cdot 8.0 / 1000 = 10.0 \text{ kip}$$

ACI 14.5.5.1

$$\phi V_{cz} = \frac{4}{3} \cdot \phi \cdot \sqrt{f'_c} \cdot \text{Length} \cdot t / 1000 = \frac{4}{3} \cdot 0.60 \cdot \sqrt{2500} \cdot 2.0 \cdot 12 \cdot 8.0 / 1000 = 7.7 \text{ kip}$$

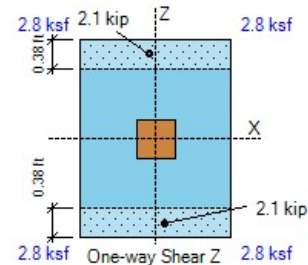
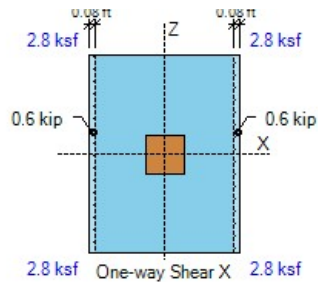
- Shear forces calculated as the volume of the bearing pressures under the effective areas:

$$\text{One-way shear } V_{ux} \text{ (- Side)} = 0.6 \text{ kip} < 10.0 \text{ kip} \quad \text{OK}$$

$$\text{One-way shear } V_{ux} \text{ (+ Side)} = 0.6 \text{ kip} < 10.0 \text{ kip} \quad \text{OK}$$

$$\text{One-way shear } V_{uz} \text{ (- Side)} = 2.1 \text{ kip} < 7.7 \text{ kip} \quad \text{OK}$$

$$\text{One-way shear } V_{uz} \text{ (+ Side)} = 2.1 \text{ kip} < 7.7 \text{ kip} \quad \text{OK}$$



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

$$\text{Plain } \phi M_{nx} = 5 * \phi * \sqrt{f_c} * L * \text{Thick}^2 / 6 = 5 * 0.60 * \sqrt{(2500)} * 2.00 * 8.0^2 / 6 / 1000 = 0.9 \text{ k-ft}$$

ACI Eq. (14.5.2.1a)

$$\text{Plain } \phi M_{nz} = 5 * \phi * \sqrt{f_c} * W * \text{Thick}^2 / 6 = 5 * 0.60 * \sqrt{(2500)} * 2.60 * 8.0^2 / 6 / 1000 = 1.1 \text{ k-ft}$$

- 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:

$$\text{Top moment -Mux (- Side)} = 0.0 \text{ k-ft} < 3.2 \text{ k-ft OK}$$

$$\text{Top moment -Mux (+ Side)} = 0.0 \text{ k-ft} < 3.2 \text{ k-ft OK}$$

$$\text{Top moment -Muz (- Side)} = 0.0 \text{ k-ft} < 4.2 \text{ k-ft OK}$$

$$\text{Top moment -Muz (+ Side)} = 0.0 \text{ k-ft} < 4.2 \text{ k-ft OK}$$

- Bottom Bars

No Bottom Reinforcement Provided at the Footing

Use Plain Concrete Flexural Strength at Bottom

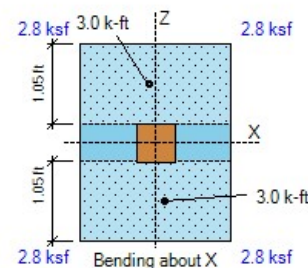
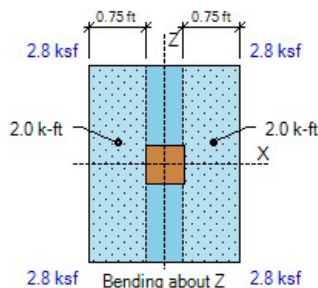
- Bottom moments calculated as the bearing minus the overburden pressures times the lever arm:

$$\text{Bottom moment Mux (- Side)} = 3.0 \text{ k-ft} < 3.2 \text{ k-ft OK} \quad \text{ratio} = 0.94$$

$$\text{Bottom moment Mux (+ Side)} = 3.0 \text{ k-ft} < 3.2 \text{ k-ft OK} \quad \text{ratio} = 0.94$$

$$\text{Bottom moment Muz (- Side)} = 2.0 \text{ k-ft} < 4.2 \text{ k-ft OK} \quad \text{ratio} = 0.48$$

$$\text{Bottom moment Muz (+ Side)} = 2.0 \text{ k-ft} < 4.2 \text{ k-ft OK} \quad \text{ratio} = 0.48$$



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

$$\text{Area } A1 = \text{col } L * \text{col } W = 6.0 * 6.0 = 36.0 \text{ in}^2$$

$$Sx = \text{col } W * \text{col } L^2 / 6 = 6.0 * 6.0^2 / 6 = 36.0 \text{ in}^3$$

$$Sz = \text{col } L * \text{col } W^2 / 6 = 6.0 * 6.0^2 / 6 = 36.0 \text{ in}^3$$

$$\text{Bearing } Pbu = P / A1 + Mz / Sx + Mx / Sz = 14.2 / 36.0 + 0.0 * 12 / 36.0 + 0.0 * 12 / 36.0 = 0.4 \text{ ksi}$$

$$\text{Min edge} = \text{Min} (L / 2 - X\text{-offset} - \text{col } L / 2, W / 2 - Z\text{-offset} - \text{col } W / 2)$$

$$\text{Min edge} = \text{Min} (2.00 * 12 / 2 - 0.0 - 6.0 / 2, 2.60 * 12 / 2 - 0.0 - 6.0 / 2) = 9.0 \text{ in}$$

$$\text{Area } A2 = \text{Min} [L * W, (\text{col } L + 2 * \text{Min edge}) * (\text{col } W + 2 * \text{Min edge})]$$

ACI R22.8.3.2

$$A2 = \text{Min} [2.00 * 12 * 2.6 * 12, (6.0 + 2 * 9.0) * (6.0 + 2 * 9.0)] = 576.0 \text{ in}^2$$

$$\text{Footing } \phi Pnc = \phi * 0.85 * f'c * \text{Min} [2, \sqrt{(A2 / A1)}] = 0.65 * 0.85 * 2.5 * \text{Min} [2, \sqrt{(576.0 / 36.0)}] = 2.8 \text{ ksi}$$

$$\text{Footing } \phi Pns = \phi * As * Fy / A1 = 0.0 \text{ ksi}$$

ACI 22.8.3.2

$$\text{Footing bearing } \phi Pn = \phi Pnc + \phi Pns = 2.8 + 0.0 = 2.8 \text{ ksi} > 0.4 \text{ psi OK}$$

Hooked $L_{dh} = \text{Max} (8 \text{ db}, 6, 0.02 * f_y / (f_c)^{1/2} * \text{Confining} * \text{Location} * \text{Concrete} * \text{db} * \text{ratio})$

ACI 25.4.3

$$L_{dh} = \text{Max} (8 \text{ db}, 6, 0.02 * 60.0 * 1000 / (2500)^{1/2} * 1.0 * 0.7 * 0.0 * 0.75 * 0.07) = 6.0 \text{ in}$$

Ld provided = Dowel length = $3.00 * 12 = 36.0 \text{ in} > 12.3 \text{ in OK}$

Ldh provided = Footing thickness - Cover = $8.00 - 3.0 = 5.0 \text{ in} < 6.0 \text{ in NG}$

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

$$X\text{-Edge} = \text{Length} / 2 - \text{Offset} - \text{Col} / 2 = 2.00 * 12 / 2 - 0.0 - 6.0 / 2 = 9.0 \text{ in} \quad \alpha_{sx} = 10$$

$$Z\text{-Edge} = \text{Width} / 2 - \text{Offset} - \text{Col} / 2 = 2.60 * 12 / 2 - 0.0 - 6.0 / 2 = 12.6 \text{ in} \quad \alpha_{sz} = 10$$

$$\alpha_s = \alpha_{sx} + \alpha_{sz} = 10 + 10 = 20 \quad \text{Col type} = \text{Corner} \quad \beta = L / W = 6.0 / 6.0 = 1.00$$

ACI 22.6.5.2

$$\text{Perimeter } b_o = \alpha_{sz} / 10 * (L + d / 2 + X\text{-Edge}) + \alpha_{sx} / 10 * (W + d / 2 + Z\text{-Edge})$$

ACI 22.6.4.2

$$b_o = 10 / 10 * (6.0 + 8.0 / 2 + 9.0) + 10 / 10 * (6.0 + 8.0 / 2 + 12.6) = 41.6 \text{ in}$$

$$\text{Area } A_{bo} = (L + d / 2 + X\text{-Edge}) * (W + d / 2 + Z\text{-Edge}) = (6.0 + 8.0 / 2 + 9.0) * (6.0 + 8.0 / 2 + 12.6) = 429.4 \text{ in}^2$$

Use Plain Concrete Shear Strength

$$\phi V_c = \phi * \text{Min} (1 + 2 / \beta, 2) * 4/3 * \sqrt{f_c}$$

ACI 14.5.5.1

$$\phi V_c = 0.60 * \text{Min} (1 + 2 / 1.00, 2) * 4/3 * \sqrt{2500} = 80.0 \text{ psi}$$

Punching force $F = P + \text{Overburden} * A_{bo} - \text{Bearing}$

$$F = 14.2 + 0.07 * 429.4 / 144 - 3.8 = 10.6 \text{ kip}$$

$$b_1 = L + d / 2 + X\text{-Edge} = 6.0 + 8.0 / 2 + 9.0 = 19.0 \text{ in} \quad b_2 = W + d / 2 + Z\text{-Edge} = 6.0 + 8.0 / 2 + 12.6 = 22.6 \text{ in}$$

$$\gamma_{vx} \text{ factor} = 1 - \frac{1}{1 + (2/3) \sqrt{(b_2 / b_1)}} = 1 - \frac{1}{1 + (2/3) \sqrt{(22.6 / 19.0)}} = 0.42$$

ACI Eq. (8.4.4.2.2)

$$\gamma_{vz} \text{ factor} = 1 - \frac{1}{1 + (2/3) \sqrt{(b_1 / b_2)}} = 1 - \frac{1}{1 + (2/3) \sqrt{(19.0 / 22.6)}} = 0.38$$

ACI Eq. (8.4.2.3.2)

$$X_{2z} = b_1^2 / 2 / (b_1 + b_2) = 19.0^2 / 2 / (19.0 + 22.6) = 4.3 \text{ in} \quad X_{2x} = b_2^2 / 2 / (b_2 + b_1) = 6.1 \text{ in}$$

$$J_{cz} = b_1 * d^3 / 12 + b_1^3 * d / 12 + b_1 * d * (b_1 / 2 - X_{2z})^2 + b_2 * d * X_{2z}^2$$

ACI R8.4.4.2.3

$$J_{cz} = 19.0 * 8.0^3 / 12 + 19.0^3 * 8.0 / 12 + 19.0 * 8.0 * (19.0 / 2 - 4.3)^2 + 22.6 * 8.0 * 4.3^2 = 12836 \text{ in}^4$$

$$J_{cx} = b_2 * d^3 / 12 + b_2^3 * d / 12 + b_2 * d * (b_2 / 2 - X_{2x})^2 + b_1 * d * X_{2x}^2$$

ACI R8.4.4.2.3

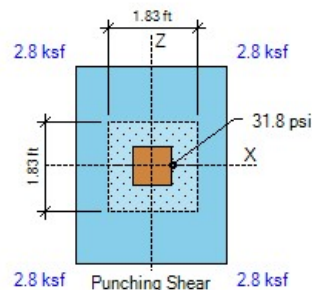
$$J_{cx} = 22.6 * 8.0^3 / 12 + 22.6^3 * 8.0 / 12 + 22.6 * 8.0 * (22.6 / 2 - 6.1)^2 + 19.0 * 8.0 * 6.1^2 = 19204 \text{ in}^4$$

$$\text{Stress due to } P = F / (b_o * d) * 1000 = 10.6 / (41.6 * 8.0) * 1000 = 31.8 \text{ psi}$$

$$\text{Stress due to } M_x = \gamma_{vx} * X\text{-OTM} * X_{2x} / J_{cx} = 0.42 * 0.0 * 12 * 6.1 / 19204 * 1000 = 0.0 \text{ psi}$$

$$\text{Stress due to } M_z = \gamma_{vz} * Z\text{-OTM} * X_{2z} / J_{cz} = 0.42 * 0.0 * 12 * 4.3 / 12836 * 1000 = 0.0 \text{ psi}$$

$$\text{Punching stress} = P\text{-stress} + M_x\text{-stress} + M_z\text{-stress} = 31.8 + 0.0 + 0.0 = 31.8 \text{ psi} < 80.0 \text{ psi OK}$$



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

$$\text{Area } A1 = \text{col } L * \text{col } W = 6.0 * 6.0 = 36.0 \text{ in}^2$$

$$Sx = \text{col } W * \text{col } L^2 / 6 = 6.0 * 6.0^2 / 6 = 36.0 \text{ in}^3$$

$$Sz = \text{col } L * \text{col } W^2 / 6 = 6.0 * 6.0^2 / 6 = 36.0 \text{ in}^3$$

$$\text{Bearing } Pbu = P / A1 + Mz / Sx + Mx / Sz = 14.2 / 36.0 + 0.0 * 12 / 36.0 + 0.0 * 12 / 36.0 = 0.4 \text{ ksi}$$

$$\text{Min edge} = \text{Min} (L / 2 - X\text{-offset} - \text{col } L / 2, W / 2 - Z\text{-offset} - \text{col } W / 2)$$

$$\text{Min edge} = \text{Min} (2.00 * 12 / 2 - 0.0 - 6.0 / 2, 2.60 * 12 / 2 - 0.0 - 6.0 / 2) = 9.0 \text{ in}$$

$$\text{Area } A2 = \text{Min} [L * W, (\text{col } L + 2 * \text{Min edge}) * (\text{col } W + 2 * \text{Min edge})]$$

ACI R22.8.3.2

$$A2 = \text{Min} [2.00 * 12 * 2.6 * 12, (6.0 + 2 * 9.0) * (6.0 + 2 * 9.0)] = 576.0 \text{ in}^2$$

$$\text{Footing } \phi Pnc = \phi * 0.85 * f'c * \text{Min} [2, \sqrt{(A2 / A1)}] = 0.65 * 0.85 * 2.5 * \text{Min} [2, \sqrt{(576.0 / 36.0)}] = 2.8 \text{ ksi}$$

$$\text{Footing } \phi Pns = \phi * As * Fy / A1 = 0.0 \text{ ksi}$$

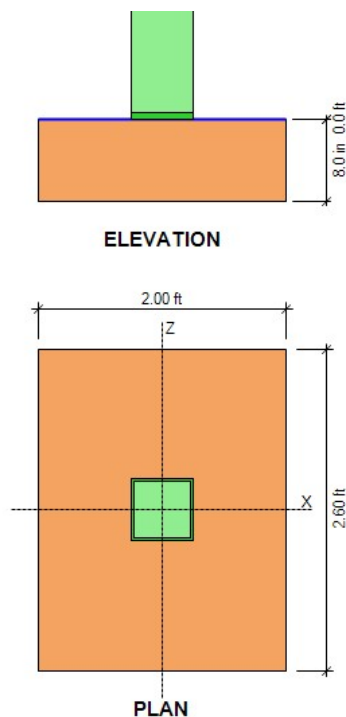
ACI 22.8.3.2

$$\text{Footing bearing } \phi Pn = \phi Pnc + \phi Pns = 2.8 + 0.0 = 2.8 \text{ ksi} > 0.4 \text{ psi OK}$$

DESIGN CODES

Concrete Design ACI 318-14

Load Combinations ASCE 7-10/16



6/2/2025

C. PIERUCCIONI, PE

ETC - BUILDING A

LATERAL ANALYSIS

1

WIND $V_{50} = 85 \text{ mph} + V_{ULT} 110 \text{ mph}$ Exp. B $K_{ZC} = 1.0$ $SLOPE = 0^\circ - 34^\circ$
 $h = 36'$ $\bar{A} = 1.06$

ZONE A $= 12.9 \text{ psf} \times 1.06 = 13.7 \text{ psf}$ 16.0 psf min

ZONE B $= 8.8 \text{ psf} \times 1.06 = 9.3 \text{ psf}$

ZONE C $= 10.2 \text{ psf} \times 1.06 = 10.8 \text{ psf}$ 16.0 psf min

ZONE D $= 7.0 \text{ psf} \times 1.06 = 7.4 \text{ psf}$ 8.0 psf min

SEISMIC $SOS = 0.831$ $R = 6.5$ $I_e = 1.0$

$C_s = (0.831 / (6.5 / 1.0)) / 1.4 = 0.091$

$W_{ROOF} = (35 \text{ psf} \times 2,632 \text{ sf}) = 92,120^\#$

$W_{LEVEL3} = (40 \text{ psf} \times 4,224 \text{ sf}) = 168,960^\#$

$W_{LEVEL2} = (40 \text{ psf} \times 3,894 \text{ sf}) = 155,760^\#$

$416,840^\#$

$h = 9'$

$h = 9'$

$h = 9'$

$h_R = 29'$

$h_3 = 20'$

$h_2 = 10'$

$V_s = 416,840^\# \times 0.091 = 37,932^\#$

$7,608,200$

$F_{ROOF} = \left[\frac{(92,120^\# \times 29')}{(92,120^\# \times 29') + (168,960^\# \times 20') + (155,760^\# \times 10')} \right] \times 37,932^\# = 13,319^\#$

$F_{LEVEL3} = \left[\frac{(168,960^\# \times 20')}{(92,120^\# \times 29') + (168,960^\# \times 20') + (155,760^\# \times 10')} \right] \times 37,932^\# = 16,847^\#$

$F_{LEVEL2} = \left[\frac{(155,760^\# \times 10')}{(92,120^\# \times 29') + (168,960^\# \times 20') + (155,760^\# \times 10')} \right] \times 37,932^\# = 7,766^\#$

GRID A-B

$$F_{2W} = (16.0 \text{ PSF} \times 133 \text{ SF}) + (9.3 \text{ PSF} \times 110 \text{ SF}) + (8.0 \text{ PSF} \times 30 \text{ SF}) = 3,391 \text{ \#}$$

$$F_{2E} = 16,347 \text{ \#} \times (1,092 \text{ SF} / 4,224 \text{ SF}) = 4,355 \text{ \#}$$

$$F_{1W} = 3,391 \text{ \#} + (16.0 \text{ PSF} \times 153 \text{ SF}) = 5,919 \text{ \#}$$

$$F_{1E} = 4,335 \text{ \#} + 7,766 \text{ \#} \times (931 \text{ SF} / 3,394 \text{ SF}) = 6,212 \text{ \#}$$

GRID C

$$F_{2W} = (16.0 \text{ PSF} \times 151 \text{ SF}) + (8.0 \text{ PSF} \times 36 \text{ SF}) = 2,704 \text{ \#}$$

$$F_{2E} = 13,319 \text{ \#} \times (1,323 \text{ SF} / 2,632 \text{ SF}) = 6,695 \text{ \#}$$

$$F_{1W} = 2,704 \text{ \#} + (16.0 \text{ PSF} \times 313 \text{ SF}) = 7,712 \text{ \#}$$

$$F_{1E} = 6,695 \text{ \#} + 16,347 \text{ \#} \times (2,098 \text{ SF} / 4,224 \text{ SF}) = 15,063 \text{ \#}$$

$$F_{1W} = 7,712 \text{ \#} + (16.0 \text{ PSF} \times 324 \text{ SF}) = 12,896 \text{ \#}$$

$$F_{1E} = 15,063 \text{ \#} + 7,766 \text{ \#} \times (2,078 \text{ SF} / 3,394 \text{ SF}) = 19,207 \text{ \#}$$

GRID G-H

$$F_{2W} = (16.0 \text{ PSF} \times 169 \text{ SF}) + (9.3 \text{ PSF} \times 55 \text{ SF}) + (8.0 \text{ PSF} \times 23 \text{ SF}) = 3,400 \text{ \#}$$

$$F_{2E} = 13,319 \text{ \#} \times (1,309 \text{ SF} / 2,632 \text{ SF}) = 6,624 \text{ \#}$$

$$F_{1W} = 3,400 \text{ \#} + (16.0 \text{ PSF} \times 194 \text{ SF}) = 6,504 \text{ \#}$$

$$F_{1E} = 6,624 \text{ \#} + 16,347 \text{ \#} \times (1,034 \text{ SF} / 4,224 \text{ SF}) = 10,748 \text{ \#}$$

$$F_{1W} = 6,504 \text{ \#} + (16.0 \text{ PSF} \times 194 \text{ SF}) = 9,608 \text{ \#}$$

$$F_{1E} = 10,748 \text{ \#} + 7,766 \text{ \#} \times (885 \text{ SF} / 3,394 \text{ SF}) = 12,513 \text{ \#}$$

GRID 1-2

$$F_{3W} = (16.0 \text{ PSF} \times 189 \text{ SF}) = 3,024 \text{ \#}$$

$$F_{3E} = 13,319 \text{ \#} \times (657 \text{ SF} / 2,632 \text{ SF}) = 3,705 \text{ \#}$$

$$F_{2W} = 3,024 \text{ \#} + (16.0 \text{ PSF} \times 176 \text{ SF}) = 5,840 \text{ \#}$$

$$F_{2E} = 3,705 \text{ \#} + 16,847 \text{ \#} \times (917 \text{ SF} / 4,224 \text{ SF}) = 7,362 \text{ \#}$$

$$F_{1W} = 5,840 \text{ \#} + (16.0 \text{ PSF} \times 176 \text{ SF}) = 8,656 \text{ \#}$$

$$F_{1E} = 7,362 \text{ \#} + 7,766 \text{ \#} \times (867 \text{ SF} / 3,894 \text{ SF}) = 9,091 \text{ \#}$$

GRID 4-5

$$F_{3W} = (16.0 \text{ PSF} \times 253 \text{ SF}) = 4,048 \text{ \#}$$

$$F_{3E} = 13,319 \text{ \#} \times (1,307 \text{ SF} / 2,632 \text{ SF}) = 6,614 \text{ \#}$$

$$F_{2W} = 4,048 \text{ \#} + (16.0 \text{ PSF} \times 355 \text{ SF}) = 9,728 \text{ \#}$$

$$F_{2E} = 6,614 \text{ \#} + 16,847 \text{ \#} \times (2,087 \text{ SF} / 4,224 \text{ SF}) = 14,938 \text{ \#}$$

$$F_{1W} = 9,728 \text{ \#} + (16.0 \text{ PSF} \times 353 \text{ SF}) = 15,376 \text{ \#}$$

$$F_{1E} = 14,938 \text{ \#} + 7,766 \text{ \#} \times (1,872 \text{ SF} / 3,894 \text{ SF}) = 18,671 \text{ \#}$$

GRID 3-8

$$F_{3W} = (16.0 \text{ PSF} \times 181 \text{ SF}) = 2,896 \text{ \#}$$

$$F_{3E} = 13,319 \text{ \#} \times (668 \text{ SF} / 2,632 \text{ SF}) = 3,380 \text{ \#}$$

$$F_{2W} = 2,896 \text{ \#} + (16.0 \text{ PSF} \times 203 \text{ SF}) = 6,144 \text{ \#}$$

$$F_{2E} = 3,380 \text{ \#} + 16,847 \text{ \#} \times (1,122 \text{ SF} / 4,224 \text{ SF}) = 8,246 \text{ \#}$$

$$F_{1W} = 6,144 \text{ \#} + (16.0 \text{ PSF} \times 201 \text{ SF}) = 9,360 \text{ \#}$$

$$F_{1E} = 8,246 \text{ \#} + 7,766 \text{ \#} \times (1,154 \text{ SF} / 3,894 \text{ SF}) = 10,548 \text{ \#}$$

11'-9"
FTAD

GRID A (LEVEL 2) $FE = 4,355^{\#} \times 11.75' / 44.25' = 1,156^{\#}$ FTAD

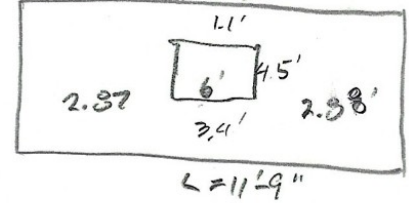
$$VE = 201 PLF$$

USE W1 $VE_{ALLOW} = 242 PLF$

HOLD DOWNS

$$TE = 385^{\#} \times 1.25 - 1/2(2015^{\#} \times 9' \times 5.99') - 1/2(1215^{\#} \times 4.5' \times 5.99') = 418^{\#}$$

USE MS T37 W/ 2 STOPS $TE_{ALLOW} = 2,140^{\#} \times 1.4 / 1.6 = 1,873^{\#}$



h=9'

20'-9"
FTAD

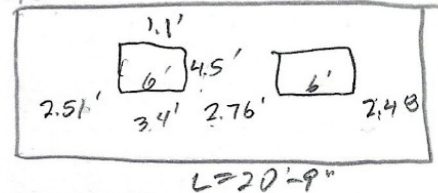
GRID B (LEVEL 2) $FE = 4,355^{\#} \times 20.75' / 44.25' = 2,019^{\#}$ FTAD

$$VE = 336 PLF$$

USE W2 $VE_{ALLOW} = 353 PLF$

HOLD DOWNS

$$TE = 920^{\#} \times 1.25 - 1/2(2015^{\#} \times 15' \times 10.39') - 1/2(1215^{\#} \times 4.5' \times 10.39') = -687^{\#}$$

So NO HDS
REQ'D

h=9'

L=11'-9"

GRID A (LEVEL 1) $FE = 6,212^{\#} \times 11.75' / 44.25' = 1,650^{\#}$ FTAD

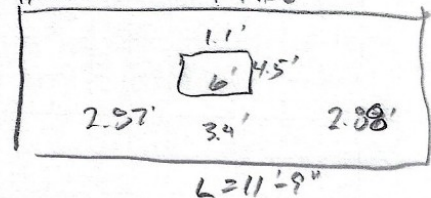
$$VE = 287 PLF$$

USE W2 $VE_{ALLOW} = 353 PLF$

HOLD DOWNS

$$TE = 1,264^{\#} \times 1.25 + 418^{\#} - 1/2(3015^{\#} \times 0.67' \times 5.99') - 1/2(1215^{\#} \times 9' \times 5.99') = 1,622^{\#}$$

USE HDOZ-SDS 2.5 W/ 2 STOPS $TE_{ALLOW} = 2,215^{\#} \times 1.4 / 1.6 = 1,938^{\#}$



h=9'

20'-9" FTAD GRID B (LEVEL 1) $FE = 6,212^{\#} \times 20.75' / 44.25' = 2,913^{\#}$ FTAD

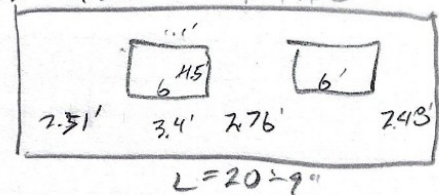
$$VE = 484 PLF$$

USE W4 $VE_{ALLOW} = 595 PLF$

HOLD DOWNS

$$TE = 1,327^{\#} \times 1.25 - 687^{\#} - 1/2(3015^{\#} \times 0.67' \times 10.39') - 1/2(1215^{\#} \times 9' \times 10.39') = 307^{\#}$$

USE HDOZ-SDS 2.5 W/ 2 STOPS $TE_{ALLOW} = 2,215^{\#} \times 1.4 / 1.6 = 1,938^{\#}$



h=9'

GRID C (LEVEL 3) FE = 6,695#

$$V_E = 6,695\# / 61.5' = 109\text{PIF}$$

USE 

$$V_{E\text{ALLOW}} = 242\text{PIF} \times 0.9 = 218\text{PIF}$$

HOLD DOWNS

$$T_E = \left(\frac{6,695\# \times 9'}{0.9 \times 61.5'} \right) \times 1.25 - \frac{1}{2} (20\text{PIF} \times 17.6' \times 32.33') - \frac{1}{2} (12\text{PIF} \times 4.5' \times 32.33') = -5,474\#$$

SO NO HD - REQUIRED

PERF. WALL

$A_o/A_w = 4\%$	$L = 6'-6"$	$h = 9'$
$A_{ch}/A_w = 95\%$		
$C_o = 0.9$		

$$L_T = 64'-8"$$

GRID C (LEVEL 2) FE = 15,063#

3 SEGMENTS

$$L = 6'-6"$$

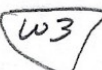
$$h = 9'$$

$$V_E = 15,063\# / 38.42' = 392\text{PIF}$$

$$L = 13'-4"$$

$$L = 18'-7"$$

$$L_T = 38'-5"$$

USE 

$$V_{E\text{ALLOW}} = 456\text{PIF}$$

HOLD DOWNS

$$L = 6'-6" \quad T_E = 392\text{PIF} \times 9' \times 1.25 - \frac{1}{2} (20\text{PIF} \times 17.6' \times 32.33') - \frac{1}{2} (12\text{PIF} \times 13.5' \times 32.33') = 3,572\#$$

USE MST60 w/ 2 STUDS

$$T_{E\text{ALLOW}} = 5,405\# \times 1.4 / 1.6 = 4,729\#$$

$$L = 13'-4" \quad T_E = 392\text{PIF} \times 9' \times 1.25 - \frac{1}{2} (20\text{PIF} \times 29' \times 6.67') - \frac{1}{2} (40\text{PIF} \times 3.67' \times 6.67') - \frac{1}{2} (24\text{PIF} \times 13.5' \times 6.67') = 1,513\#$$

USE MST37 w/ 2 STUDS

$$T_{E\text{ALLOW}} = 2,140\# \times 1.4 / 1.6 = 1,873\#$$

$$L = 18'-7" \quad T_E = 392\text{PIF} \times 9' \times 1.25 - \frac{1}{2} (20\text{PIF} \times 29' \times 9.29') - \frac{1}{2} (8\text{PIF} \times 13.5' \times 9.29') = 1,307\#$$

USE (2) HDU2-SDS2.5 w/ 2 STUDS

$$T_{E\text{ALLOW}} = 2,215\# \times 1.4 / 1.6 = 1,938\#$$

GRID C (LEVEL 1) FE = 19,207#

3 SEGMENTS

$$L = 6'-6"$$

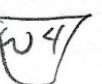
$$h = 9'$$

$$V_E = 19,207\# / 38.42' = 500\text{PIF}$$

$$L = 13'-4"$$

$$L = 18'-7"$$

$$L_T = 38'-5"$$

USE 

$$V_{E\text{ALLOW}} = 595\text{PIF}$$

HOLD DOWNS

$$L = 6'-6" \quad T_E = 500\text{PIF} \times 9' \times 1.25 + 3,572\# - \frac{1}{2} (12\text{PIF} \times 9' \times 32.33') = 9,021\#$$

USE HDU14-SDS2.5 w/ 4 STUDS

$$T_{E\text{ALLOW}} = 12,475\# \times 1.4 / 1.6 = 10,871\#$$

$$L = 13'-4" \quad T_E = 500\text{PIF} \times 9' \times 1.25 + 1,513\# - \frac{1}{2} (40\text{PIF} \times 3.67' \times 6.67') - \frac{1}{2} (12\text{PIF} \times 9' \times 6.67') = 6,293\#$$

USE HDU11-SDS2.5 w/ 3 STUDS

$$T_{E\text{ALLOW}} = 8,030\# \times 1.4 / 1.6 = 7,026\#$$

$$L = 18'-7" \quad T_E = 500\text{PIF} \times 9' \times 1.25 + 1,307\# - \frac{1}{2} (8\text{PIF} \times 9' \times 9.29') = 6,598\#$$

USE HDU11-SDS2.5 w/ 4 STUDS

$$T_{E\text{ALLOW}} = 8,030\# \times 1.4 / 1.6 = 7,026\#$$

6/2/2025

C. PIEROCCIONI, PE

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SHEAR

6

GRID 6 (LEVEL 3) $FE = 6,624^{\#} \times 13.33' / 56.33' = 1,567^{\#}$ $FTAO$

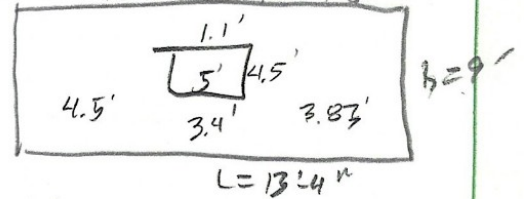
$$VE = 235PIF$$

$$USE \text{ (W1) } VEA_{allow} = 242PIF$$

HOLD DOWNS

$$TE = 1,058^{\#} \times 1.25 - (20PSF \times 17.5' \times 6.67') - \frac{1}{2}(12PSF \times 4.5' \times 6.67') = -25^{\#}$$

So NO HOLD DOWNS



GRID 6 (LEVEL 2) $FE = 10,748^{\#} \times 13.33' / 56.33' = 2,545^{\#}$ $FTAO$

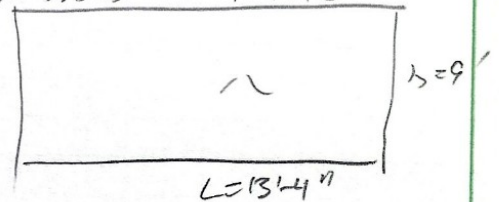
$$VE = 382PIF$$

$$USE \text{ (W3) } VEA_{allow} = 456PIF$$

HOLD DOWNS

$$TE = 1,718^{\#} \times 1.25 - 25^{\#} - \frac{1}{2}(30PSF \times 0.67' \times 6.67') - \frac{1}{2}(12PSF \times 9' \times 6.67') = 1,695^{\#}$$

$$USE \text{ MST37 W/ 2 STUDS } TFA_{allow} = 2,140^{\#} \times 1.41 / 1.6 = 1,873^{\#}$$



GRID 6 (LEVEL 1) $FE = 12,513^{\#} \times 13.33' / 56.33' = 2,961^{\#}$ $FTAO$

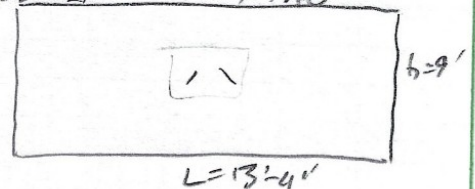
$$VE = 444PIF$$

$$USE \text{ (W3) } VEA_{allow} = 456PIF$$

HOLD DOWNS

$$TE = 1,999^{\#} \times 1.25 + 1,695^{\#} - \frac{1}{2}(30PSF \times 0.67' \times 6.67') - \frac{1}{2}(12PSF \times 9' \times 6.67') = 3,767^{\#}$$

$$USE \text{ HDU5-SDS2.5 W/ 2 STUDS } TE = 4,340^{\#} \times 1.41 / 1.6 = 3,798^{\#}$$



6/12/2025

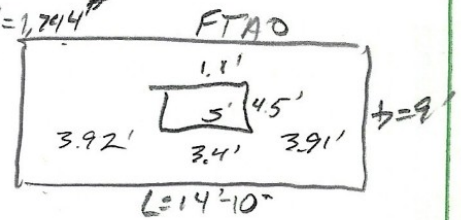
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SHEAR

7

14'-10"
FTADGRID H (LEVEL 3) $FE = 6,624^{\#} \times 14.83' / 56.33' = 1,744^{\#}$

VEE 235PIF

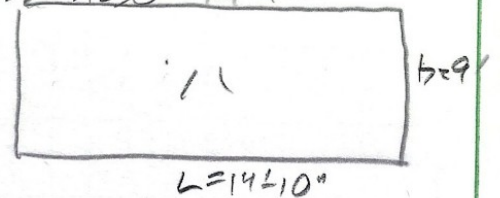
USE  VEA110WHOLD DOWNS

$$TE = 1,058^{\#} \times 1.25 - 1/2(200^{\#} \times 18.67' \times 7.42') - 1/2(1215^{\#} \times 4.5' \times 2.42') = -263^{\#}$$

SO NO HDS
REQ'D

GRID H (LEVEL 2) $FE = 10,748^{\#} \times 14.83' / 56.33' = 2,830^{\#}$ FTAD

VEE 332PIF

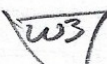
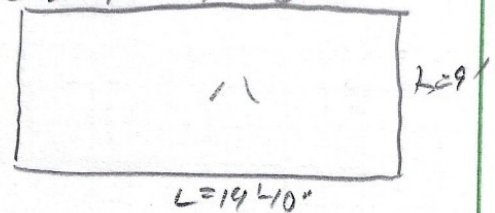
USE  VEA110W = 456PIFHOLD DOWNS

$$TE = 1,717^{\#} \times 1.25 - 263^{\#} - 1/2(300^{\#} \times 0.67' \times 7.42') - 1/2(1215^{\#} \times 9' \times 7.42') = 1,408^{\#}$$

USE AST37 W/2 STUOS $TE_{110W} = 2,140^{\#} \times 1.4 / 1.6 = 1,973^{\#}$

GRID H (LEVEL 1) $FE = 12,513^{\#} \times 14.83' / 56.33' = 3,844^{\#}$ FTAD

VEE 444PIF

USE  VEA110W = 456PIFHOLD DOWNS

$$TE = 1,499^{\#} \times 1.25 + 1,408^{\#} - 1/2(300^{\#} \times 0.67' \times 7.42') - 1/2(1215^{\#} \times 9' \times 7.42') = 3,931^{\#}$$

USE HDS-SDS2.5 W/2 STUOS $TE_{110W} = 4,340^{\#} \times 1.4 / 1.6 = 3,795^{\#}$

6/2/2025

C. PIERUCCIONI, PE

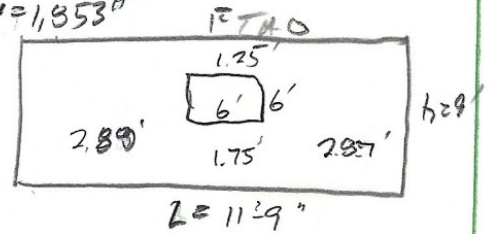
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8

11'4" FTAD
GLOBGRID 1 (LEVEL 3) $FE = 3,705^{\#} \times 11.75' / 23.5' = 1,853^{\#}$

VE = 473 PLF

USE W4 $VE_{ALLOW} = 595 \text{ PLF}$ HOLD DOWNS

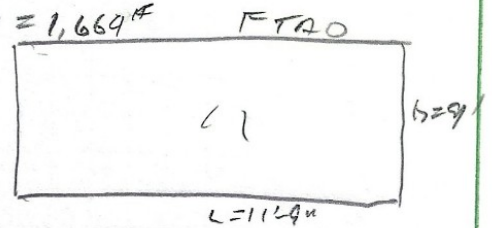
$$TE = 1,419^{\#} \times 1.25' - \frac{1}{2} (20 \text{ PSF} \times 1' \times 5.88') - \frac{1}{2} (12 \text{ PSF} \times 4.5' \times 5.88') = 1,556^{\#}$$

USE M532 W/2 STOPS

$$TE_{ALLOW} = 2,140^{\#} \times 1.4' / 1.6 = 1,823^{\#}$$

GRID 1 (LEVEL 2) $FE = 7,362^{\#} \times 11.75' / 52' = 1,669^{\#}$

VE = 425 PLF

USE W4 $VE_{ALLOW} = 595 \text{ PLF}$ HOLD DOWNS

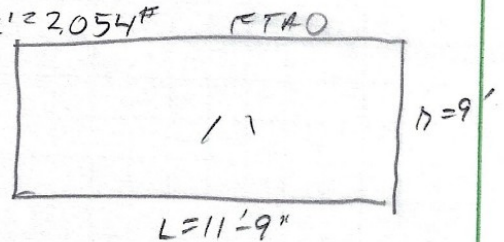
$$TE = 1,275^{\#} \times 1.25' + 1,556^{\#} - \frac{1}{2} (30 \text{ PSF} \times 7.5' \times 5.88') - \frac{1}{2} (12 \text{ PSF} \times 9' \times 5.88') = 1,953^{\#}$$

USE M548 W/2 STOPS

$$TE_{ALLOW} = 3,425^{\#} \times 1.4' / 1.6 = 2,997^{\#}$$

GRID 1 (LEVEL 1) $FE = 9,091^{\#} \times 11.75' / 52' = 2,054^{\#}$

VE = 524 PLF

USE W4 $VE_{ALLOW} = 595 \text{ PLF}$ HOLD DOWNS

$$TE = 1,573^{\#} \times 1.25' + 1,953^{\#} - \frac{1}{2} (30 \text{ PSF} \times 7.5' \times 5.88') - \frac{1}{2} (12 \text{ PSF} \times 9' \times 5.88') = 2,940^{\#}$$

USE ITDUS-SDS25 W/2 STOPS

$$TE_{ALLOW} = 4,340^{\#} \times 1.4' / 1.6 = 3,798^{\#}$$

FAD
1129
GRD 1.2

GRD 1.2 (LEVEL 3) $FE = 3,705^{\#} \times 11.75' / 23.5' = 1,853^{\#}$

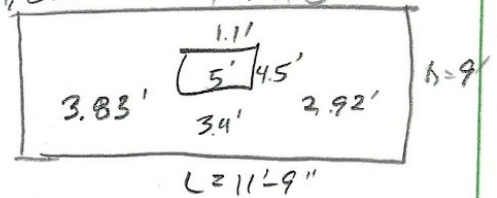
$VE = 315 \text{ PLF}$

USE  $VE_{ALLOW} = 353 \text{ PLF}$

HOLD DOWNS

$TE = 1,419^{\#} \times 1.25 - 1/2(20 \text{ PLF} \times 1' \times 5.88') - 1/2(12 \text{ PLF} \times 4.5' \times 5.88') = 1,556^{\#}$

USE 15T37W/2STUDS $TE_{ALLOW} = 2,140^{\#} \times 1.4 / 1.6 = 1,978^{\#}$



GRD 1.2 (LEVEL 2) $FE = 11.75' / 52' = 1.664^{\#}$

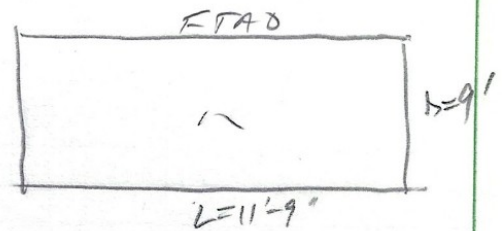
$VE = 283 \text{ PLF}$

USE  $VE_{ALLOW} = 353 \text{ PLF}$

HOLD DOWNS

$TE = 1,275^{\#} \times 1.25 + 1,556^{\#} - 1/2(30 \text{ PLF} \times 6.5' \times 5.88') - 1/2(12 \text{ PLF} \times 9' \times 5.88') = 2,259^{\#}$

USE 15T48W/2STUDS $TE_{ALLOW} = 3,425^{\#} \times 1.4 / 1.6 = 2,997^{\#}$



GRD 1.2 (LEVEL 1) $FE = 11.75' / 52' = 2.054^{\#}$

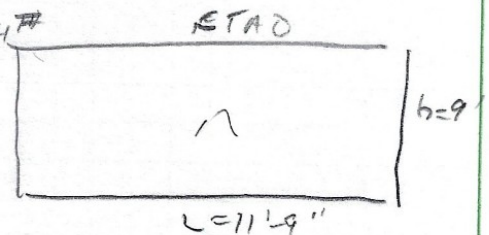
$VE = 350 \text{ PLF}$

USE  $VE_{ALLOW} = 353 \text{ PLF}$

HOLD DOWNS

$TE = 1,573^{\#} \times 1.25 + 2,259^{\#} - 1/2(30 \text{ PLF} \times 6.5' \times 5.88') - 1/2(12 \text{ PLF} \times 9' \times 5.88') = 3,339^{\#}$

USE 14D05SDS2.5W/2STUDS $TE_{ALLOW} = 4,340^{\#} \times 1.4 / 1.6 = 3,798^{\#}$



6/2/2025

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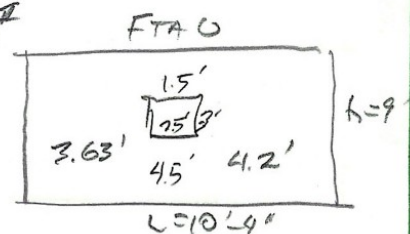
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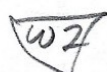
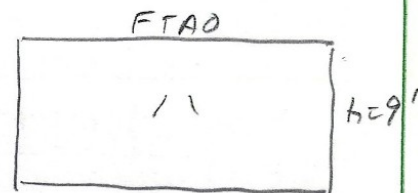
10

10'-4" FTAO GRID 22 (LEVEL 2) $FE = 7,362^{\#} \times 10.33' / 52' = 1,462^{\#}$

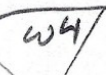
VE = 2120 PLF

USE  VE ALLOW = 2420 PLFHOLD DOWNS $TE = 1,274^{\#} \times 1.25 - 1/2(20PSF \times 14.5 \times 16') - 1/2(12PSF \times 4.5 \times 5.16') = 1,402^{\#}$ USE MST37 W/ 2 STOPS $TE ALLOW = 2,140^{\#} \times 1.44 / 1.6 = 1,873^{\#}$ GRID 22 (LEVEL 1) $FE = 9,091^{\#} \times 10.33' / 52' = 1,806^{\#}$

VE = 762 PLF

USE  VE ALLOW = 553 PLFHOLD DOWNS $TE = 1,573^{\#} \times 1.25 + 1,402^{\#} - 1/2(30PSF \times 5.1 \times 5.16') - 1/2(12PSF \times 9' \times 5.16') = 2,694^{\#}$ USE HD U4-S057.5 W/ 2 STOPS $TE ALLOW = 3,285^{\#} \times 1.44 / 1.6 = 2,874^{\#}$ 

3'-2" FTAO

GRID 2 (LEVEL 2) - $FE = 7,362^{\#} \times 13.16' / 52' = 2,571^{\#}$ 2 SEGMENTS $L = 3'-7"$ $h = 9'$
 $L = 3'-7"$
 $LT = 7'-2"$ $VE = 2,571^{\#} / 7.16' = 359 \text{ PLF}$ USE  $VE ALLOW = 456 \text{ PLF} \times (1.25 - 0.125 \times 9' / 3.58') = 427 \text{ PLF}$ HOLD DOWNS $TE = 359 \text{ PLF} \times 9' \times 1.25 - 1/2(20PSF \times 1' \times 1.79') - 1/2(12PSF \times 4.5' \times 1.79') = 3,973^{\#}$ USE MST60 W/ 2 STOPS $TE ALLOW = 5,405^{\#} \times 1.44 / 1.6 = 4,729^{\#}$ GRID 2 (LEVEL 1) $FE = 9,091^{\#} \times 13.16' / 52' = 3,175^{\#}$ 2 SEGMENTS $L = 8'-7"$ $h = 9'$
 $L = 3'-7"$
 $LT = 7'-2"$ $VE = 3,175^{\#} / 7.16' = 443 \text{ PLF}$ USE  $VE ALLOW = 595 \text{ PLF} \times (1.25 - 0.125 \times 9' / 3.58') = 557 \text{ PLF}$ HOLD DOWNS $TE = 443 \text{ PLF} \times 9' \times 1.25 + 3,973^{\#} - 1/2(30PSF \times 5.9' \times 1.79') - 1/2(12PSF \times 9' \times 1.79') = 8,702^{\#}$ USE HD U14-S057.5 W/ 4 STOPS $TE ALLOW = 12,425^{\#} \times 1.44 / 1.6 = 10,372^{\#}$

6/2/2025

C. P. BROCCIONI, PE

ETC - BUILDING A

SHEAR

11

GRID 485 (LEVEL 3) $FE = 6,614 \# \times 29'4" / 59'8" = 3,307 \#$ $1 \text{ SEC. } L = 29'4"$
 $h = 9'$

$VE = 3,307 \# / 29.33' = 113 \text{ p.f.}$

USE  $VE_{ALLOW} = 242 \text{ p.f.}$

HOLD DOWNS

$TE = 113 \text{ p.f.} \times 9' \times 1.25 - 1/2 (30 \text{ p.s.f.} \times 14.67') - 1/2 (12 \text{ p.s.f.} \times 14.67') = 726 \#$

USE MST 37 W/ 2 STOPS $TE_{ALLOW} = 2,140 \# \times 1.4 / 1.6 = 1,873 \#$

GRID 485 (LEVEL 2) $FE = 14,938 \# \times 29'4" / 84'8" = 5,169 \#$ $1 \text{ SEC. } L = 29'4"$
 $h = 9'$

$VE = 5,169 \# / 29.33' = 176 \text{ p.f.}$

USE  $VE_{ALLOW} = 242 \text{ p.f.}$

HOLD DOWNS

$TE = 176 \text{ p.f.} \times 9' \times 1.25 + 726 \# - 1/2 (30 \text{ p.s.f.} \times 14.67') - 1/2 (12 \text{ p.s.f.} \times 14.67') = 354 \#$

USE MST 37 W/ 2 STOPS $TE_{ALLOW} = 2,140 \# \times 1.4 / 1.6 = 1,873 \#$

GRID 485 (LEVEL 1) $FE = 18,671 \# \times 29'4" / 84'8" = 6,493 \#$ $1 \text{ SEC. } L = 29'4"$
 $h = 9'$

$VE = 6,493 \# / 29.33' = 221 \text{ p.f.}$

USE  $VE_{ALLOW} = 242 \text{ p.f.}$

HOLD DOWNS

$TE = 221 \text{ p.f.} \times 9' \times 1.25 + 354 \# - 1/2 (30 \text{ p.s.f.} \times 14.67') - 1/2 (12 \text{ p.s.f.} \times 14.67') = 490 \#$

USE HDU 2-S002.5 W/ 2 STOPS $TE_{ALLOW} = 2,215 \# \times 1.4 / 1.6 = 1,939 \#$

6/2/2025

C. PIEROCCIONI, PE

ETC-BUILDING A

SHEAR

12

$L=26'-1"$ GRID 5 (LEVEL 2) $FE = 14,938 \text{ lb} \times 26'-1" / 84'-9" = 4,597 \text{ lb}$ 1 SEC. $L=26'-1"$
 $VE = 4,597 \text{ lb} / 26.08' = 176 \text{ plf}$ $h=9'$

USE  $VE_{allow} = 242 \text{ plf}$

HOLD DOWNS

$TE = 176 \text{ plf} \times 9' \times 1.25 - \frac{1}{2}(20 \text{ psf} \times 1' \times 13.04') - \frac{1}{2}(815 \text{ psf} \times 4.5' \times 13.04') = 1,618 \text{ lb}$

USE (2) HDU2-SDS2.5 w/2 STOPS $TE_{allow} = 2,215 \text{ lb} \times 1.4 / 1.6 = 1,938 \text{ lb}$

GRID 5 (LEVEL 1) $FE = 18,671 \text{ lb} \times 26'-1" / 84'-9" = 5,773 \text{ lb}$ 1 SEC. $L=26'-1"$
 $VE = 5,773 \text{ lb} / 26.08' = 221 \text{ plf}$ $h=9'$

USE  $VE_{allow} = 242 \text{ plf}$

HOLD DOWNS

$TE = 221 \text{ plf} \times 9' \times 1.25 + 1,618 \text{ lb} - \frac{1}{2}(30 \text{ psf} \times 6.1' \times 13.04') - \frac{1}{2}(815 \text{ psf} \times 9' \times 13.04') = 2,691 \text{ lb}$

USE HDU4-SDS2.5 w/2 STOPS $TE_{allow} = 3,285 \text{ lb} \times 1.4 / 1.6 = 2,874 \text{ lb}$

6/2/2025

C. PIERUCCIONI, A.E.

ETC-BUILDING A

SHEAR

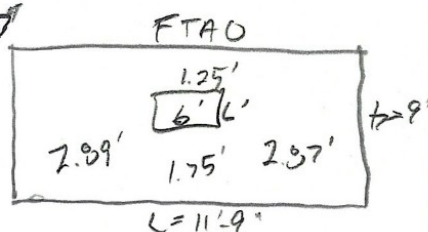
13

11'-9" FTAD
GRID 6GRID 7 (LEVEL 2) $FE = 3,380^{\#} \times 11.75' / 23' = 1,727^{\#}$

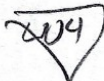
VEE 441 PLF

USE  VEA_{allow} = 450 PLFHOLD DOWNS

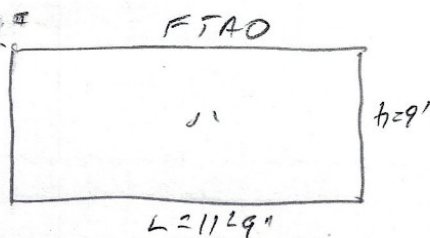
$$TE = 1,323^{\#} \times 1.25 - \frac{1}{2}(20 \text{ PSF} \times 1' \times 5.99') - \frac{1}{2}(120 \text{ PSF} \times 4.5' \times 5.99') = 1,436^{\#}$$

USE AST 37 W / Z STUDS $TE_{allow} = 2,140^{\#} \times 1.4 / 1.6 = 1,973^{\#}$ GRID 7 (LEVEL 2) $FE = 8,426^{\#} \times 11.75' / 52' = 1,903^{\#}$

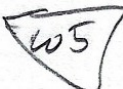
VEE 486 PLF

USE  VEA_{allow} = 595 PLFHOLD DOWNS

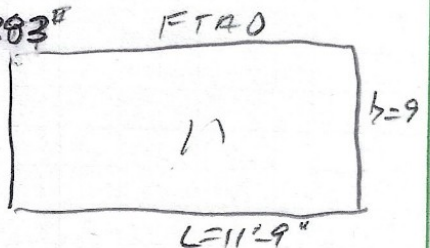
$$TE = 1,458^{\#} \times 1.25 + 1,436^{\#} - \frac{1}{2}(30 \text{ PSF} \times 7' \times 5.99') - \frac{1}{2}(120 \text{ PSF} \times 9' \times 5.99') = 2,324^{\#}$$

USE AST 48 W / Z STUDS $TE_{allow} = 3,425^{\#} \times 1.4 / 1.6 = 2,997^{\#}$ GRID 7 (LEVEL 1) $FE = 10,578^{\#} \times 11.75' / 52' = 2,383^{\#}$

VEE 608 PLF

USE  VEA_{allow} = 716 PLFHOLD DOWNS

$$TE = 1,925^{\#} \times 1.25 + 2,324^{\#} - \frac{1}{2}(30 \text{ PSF} \times 7' \times 5.99') - \frac{1}{2}(120 \text{ PSF} \times 9' \times 5.99') = 3,690^{\#}$$

USE HDO 5-SOS 2.5 W / Z STUDS $TE_{allow} = 4,340^{\#} \times 1.4 / 1.6 = 3,798^{\#}$ 

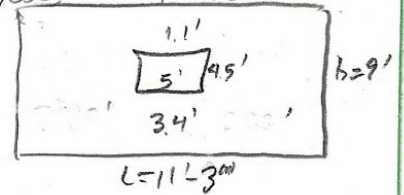
6/2/2025

C. PIERUCCIONI, PE

ETC - BUILDING A

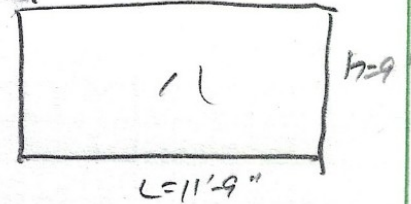
SHEAR

14

11'-3" FTOD
GRID C-DGRID 6.8 (LEVEL 3) $FE = 3,380^{\#} \times 11.25' / 22' = 1,653^{\#}$ $VE = 294 \text{ PIF}$ USE $\nabla W2$ $VE_{ALLOW} = 353 \text{ PIF}$ HOLD DOWNS

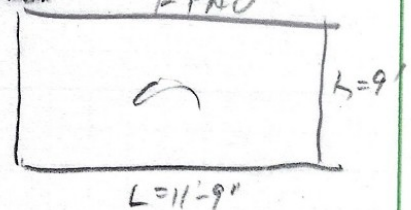
$$TE = 1,322^{\#} \times 1.25 - 1/2 (20 \text{ PSF} \times 1 \times 5.12') - 1/2 (12 \text{ PSF} \times 4.5 \times 5.12') = 1,463^{\#}$$

$$\text{USE MST3T w/ 2 STUOS} \quad TE_{ALLOW} = 2,140^{\#} \times 1.4 / 1.6 = 1,973^{\#}$$

GRID 6.8 (LEVEL 2) $FE = 8,246^{\#} \times 11.25' / 52' = 1,794^{\#}$ $VE = 317 \text{ PIF}$ USE $\nabla W2$ $VE_{ALLOW} = 353 \text{ PIF}$ HOLD DOWNS

$$TE = 1,477^{\#} \times 1.25 + 1,463^{\#} - 1/2 (30 \text{ PSF} \times 6.5 \times 5.12') - 1/2 (12 \text{ PSF} \times 9 \times 5.12') = 2,471^{\#}$$

$$\text{USE MST4B w/ 2 STUOS} \quad TE_{ALLOW} = 3,475^{\#} \times 1.4 / 1.6 = 2,997^{\#}$$

GRID 6.8 (LEVEL 1) $FE = 10,548^{\#} \times 11.25' / 52' = 2,282^{\#}$ $VE = 406 \text{ PIF}$ USE $\nabla W3$ $VE_{ALLOW} = 450 \text{ PIF}$ HOLD DOWNS

$$TE = 1,826^{\#} \times 1.25 + 2,471^{\#} - 1/2 (30 \text{ PSF} \times 6.5 \times 5.12') - 1/2 (12 \text{ PSF} \times 9 \times 5.12') = 3,978^{\#}$$

$$\text{USE H1208-5057.5 w/ 2 STUOS} \quad TE_{ALLOW} = 5,820^{\#} \times 1.4 / 1.6 = 5,093^{\#}$$

6/2/2025

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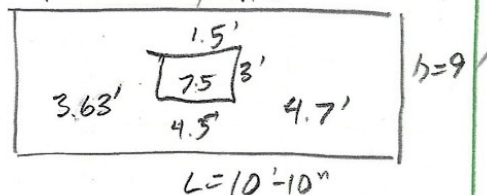
ETC-BUILDING A

SHEAR

15

GR 10 7.5 (LEVEL 2) $FE = 8,246 \# \times 10.83' / 52' = 1,717 \#$ F_{TAO}

VE = 238 PLF

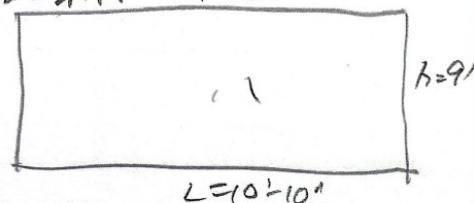
USE  $VE_{ALLOW} = 242 \text{ PLF}$ HOLD DOWNS

$$TE = 1,427 \# \times 1.25 - 1/2 (20 \text{ PSF} \times 1' \times 5.42') - 1/2 (12 \text{ PSF} \times 4.5' \times 5.42') = 1,437 \#$$

USE MST37 W/ 2 STUDS $TE_{ALLOW} = 2,140 \# \times 1.4' / 1.6' = 1,878 \#$

GR 10 7.5 (LEVEL 1) $FE = 10,549 \# \times 10.83' / 52' = 2,197 \#$ F_{TAO}

VE = 304 PLF

USE  $VE_{ALLOW} = 353 \text{ PLF}$ HOLD DOWNS

$$TE = 1,826 \# \times 1.25 + 1,437 \# - 1/2 (30 \text{ PSF} \times 5.1' \times 5.42') - 1/2 (12 \text{ PSF} \times 9' \times 5.42') = 3,012 \#$$

USE HDOUS-SDS7.5 W/ 2 STUDS $TE_{ALLOW} = 4,340 \# \times 1.4' / 1.6' = 3,798 \#$

GR 10 8 (LEVEL 2) $FE = 8,246 \# \times 18.16' / 52' = 2,830 \#$ 2 SEGS $L = 3'-7"$ $h = 9'$
 $L = 3'-7"$
 $L_T = 7'-2"$

$$VE = 2,830 \# / 7.16' = 402 \text{ PLF}$$

USE  $VE_{ALLOW} = 456 \text{ PLF} \times (1.25 - 0.125 \times 9' / 3.58') = 427 \text{ PLF}$ HOLD DOWNS

$$TE = 402 \text{ PLF} \times 9' \times 1.25 - 1/2 (20 \text{ PSF} \times 1' \times 1.79') - 1/2 (12 \text{ PSF} \times 4.5' \times 1.79') = 4,459 \#$$

USE MST60 W/ 2 STUDS $TE_{ALLOW} = 5,405 \# \times 1.4' / 1.6' = 4,729 \#$

GR 10 8 (LEVEL 1) $FE = 10,549 \# \times 18.16' / 52' = 3,694 \#$ 2 SEGS $L = 3'-7"$ $h = 9'$
 $L = 3'-7"$
 $L_T = 7'-2"$

$$VE = 3,694 \# / 7.16' = 514 \text{ PLF}$$

USE  $VE_{ALLOW} = 595 \text{ PLF} \times (1.25 - 0.125 \times 9' / 3.58') = 557 \text{ PLF}$ HOLD DOWNS

$$TE = 514 \text{ PLF} \times 9' \times 1.25 + 4,459 \# - 1/2 (20 \text{ PSF} \times 5.8' \times 1.79') - 1/2 (12 \text{ PSF} \times 9' \times 1.79') = 7,995 \#$$

USE HDOU14-SDS2.5 W/ 4 STUDS $TE_{ALLOW} = 2,425 \# \times 1.4' / 1.6' = 2,092 \#$



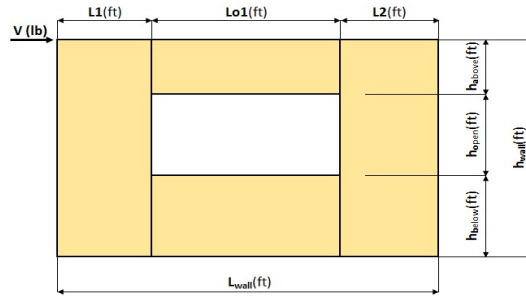
Force Transfer Around Openings Calculator

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	ha	1.10 ft
L2	2.88 ft	ho	4.50 ft
hwall	9.00 ft	hb	3.40 ft
Lwall	11.75 ft	Lo1	6.00 ft
		Wall Pier Aspect Ratio	Adj. Factor
		P1=ho/L1=	1.57 N/A
		P2=ho/L2=	1.56 N/A

1. Hold-down forces: $H = Vh_{wall}/L_{wall}$ 885 lbf

2. Unit shear above + below opening
First opening: $va1 = vb1 = H/(h_a + h_b) = 197 \text{ plf}$

3. Total boundary force above + below openings
First opening: $O1 = va1 \times (Lo1) = 1181 \text{ lbf}$

4. Corner forces
 $F1 = O1(L1)/(L1+L2) = 589 \text{ lbf}$
 $F2 = O1(L2)/(L1+L2) = 591 \text{ lbf}$

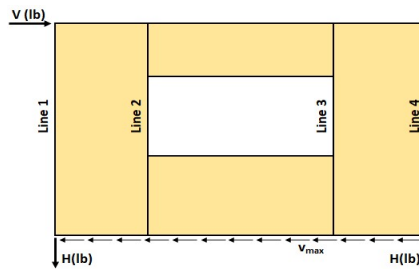
5. Tributary length of openings
 $T1 = (L1 \times Lo1)/(L1+L2) = 2.99 \text{ ft}$
 $T2 = (L2 \times Lo1)/(L1+L2) = 3.01 \text{ ft}$

6. Unit shear beside opening
 $v1 = (V/L)(L1+T1)/L1 = 201 \text{ plf}$
 $v2 = (V/L)(T2+L2)/L2 = 201 \text{ plf}$
Check $v1 \times L1 + v2 \times L2 = V?$ 1156 lbf OK

7. Resistance to corner forces
 $R1 = v1 \times L1 = 577 \text{ lbf}$
 $R2 = v2 \times L2 = 579 \text{ lbf}$

8. Difference corner force + resistance
 $R1 - F1 = -12 \text{ lbf}$
 $R2 - F2 = -12 \text{ lbf}$

9. Unit shear in corner zones
 $vc1 = (R1 - F1)/L1 = -4 \text{ plf}$
 $vc2 = (R2 - F2)/L2 = -4 \text{ plf}$



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:

Designer: Chon Pieruccioni, PE

Date: 6/2/2025

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_i 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

$$\Delta = \frac{8vh^3}{EAb} + \frac{vh}{Gt} + 0.75he_n + d_a \frac{h}{b}$$
 (Equation 23-2)

	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.)

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	Term 2	Term 3	Term 4	Term 1	Term 2	Term 3	Term 4
Bending	Shear	Fastener	HD-1	Bending	Shear	Fastener	HD-2
	0.022	0.034	0.233		0.013	0.021	0.090
Sum				Sum			
Pier 2 (left)				Pier 2 (right)			
Term 1	Term 2	Term 3	Term 4	Term 1	Term 2	Term 3	Term 4
Bending	Shear	Fastener	HD-1	Bending	Shear	Fastener	HD-2
	0.013	0.021	0.090		0.022	0.034	0.233
Sum				Sum			

Total Defl.

(in.) %drift

Project Information

Code:

Designer: Chon Pieruccioni, PE

Date: 6/2/2025

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

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 (in.)

HD Capacity: 2140 (lbf)

HD Deflection: 0.088 (in.)

Three-Term Equation Deflection Check

$$\delta_{sw} = \frac{8vh^3}{EAb} + \frac{vh}{1000G_a} + \frac{h\Delta_a}{b} \quad (4.3-1)$$

	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	Term 2	Term 3	Term 1	Term 2	Term 3
Bending	Shear	Fastener	Bending	Shear	Fastener
	0.121	0.233		0.075	0.090
Sum			Sum		
Pier 2 (left)			Pier 2 (right)		
Term 1	Term 2	Term 3	Term 1	Term 2	Term 3
Bending	Shear	Fastener	Bending	Shear	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.



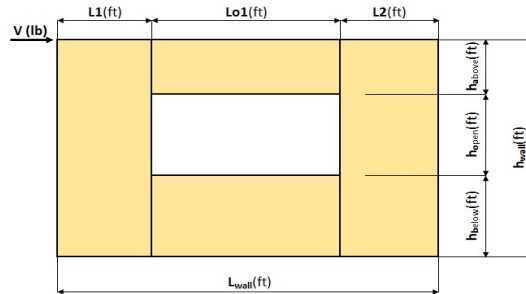
Force Transfer Around Openings Calculator

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		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 _a /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: $H = Vh_{wall}/L_{wall}$ 1264 lbf

2. Unit shear above + below opening
First opening: $va1 = vb1 = H/(h_a + h_b) = 281 \text{ plf}$

3. Total boundary force above + below openings
First opening: $O1 = va1 \times (Lo1) = 1685 \text{ lbf}$

4. Corner forces
 $F1 = O1(L1)/(L1+L2) = 841 \text{ lbf}$
 $F2 = O1(L2)/(L1+L2) = 844 \text{ lbf}$

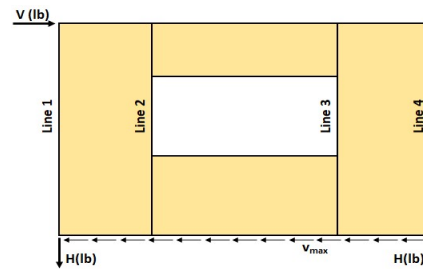
5. Tributary length of openings
 $T1 = (L1 \times Lo1)/(L1+L2) = 2.99 \text{ ft}$
 $T2 = (L2 \times Lo1)/(L1+L2) = 3.01 \text{ ft}$

6. Unit shear beside opening
 $v1 = (V/L)(L1+T1)/L1 = 287 \text{ plf}$
 $v2 = (V/L)(T2+L2)/L2 = 287 \text{ plf}$
Check $v1 \times L1 + v2 \times L2 = V?$ 1650 lbf OK

7. Resistance to corner forces
 $R1 = v1 \times L1 = 824 \text{ lbf}$
 $R2 = v2 \times L2 = 826 \text{ lbf}$

8. Difference corner force + resistance
 $R1 - F1 = -18 \text{ lbf}$
 $R2 - F2 = -18 \text{ lbf}$

9. Unit shear in corner zones
 $vc1 = (R1 - F1)/L1 = -6 \text{ plf}$
 $vc2 = (R2 - F2)/L2 = -6 \text{ plf}$



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:

Designer: Chon Pieruccioni, PE

Date: 6/2/2025

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_i 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

$$\Delta = \frac{8vh^3}{EAb} + \frac{vh}{Gt} + 0.75he_n + d_a \frac{h}{b}$$

(Equation 23-2)

	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.)

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	Term 2	Term 3	Term 4	Term 1	Term 2	Term 3	Term 4
Bending	Shear	Fastener	HD-1	Bending	Shear	Fastener	HD-2
	0.031	0.029	0.333		0.019	0.018	0.129
Sum				Sum			
Pier 2 (left)				Pier 2 (right)			
Term 1	Term 2	Term 3	Term 4	Term 1	Term 2	Term 3	Term 4
Bending	Shear	Fastener	HD-1	Bending	Shear	Fastener	HD-2
	0.019	0.018	0.128		0.031	0.029	0.332
Sum				Sum			

Total Defl.

(in.) %drift

Project Information

Code:

Designer: Chon Pieruccioni, PE

Date: 6/2/2025

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

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

HD Capacity: 2140

HD Deflection: 0.088

(in.)

(lbf)

(in.)

Three-Term Equation Deflection Check

$$\delta_{sw} = \frac{8vh^3}{EAb} + \frac{vh}{1000G_a} + \frac{h\Delta_a}{b} \quad (4.3-1)$$

	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	Term 2	Term 3	Term 1	Term 2	Term 3
Bending	Shear	Fastener	Bending	Shear	Fastener
	0.117	0.333		0.073	0.129
Sum			Sum		
Pier 2 (left)			Pier 2 (right)		
Term 1	Term 2	Term 3	Term 1	Term 2	Term 3
Bending	Shear	Fastener	Bending	Shear	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.



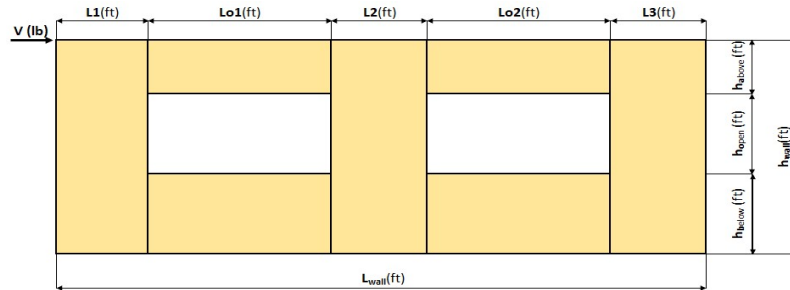
Force Transfer Around Openings Calculator

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	ha1	ha2	Wall Pier Aspect Ratio	Adj. Factor
L2	2.76 ft	ho1	ho2	P1=ho/L1=	N/A
L3	2.48 ft	hb1	hb2	P2=ho/L2=	N/A
hwall	9.00 ft	Lo1	Lo2	P3=ho/L3=	N/A
Lwall	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: $O1 = va1 \times (Lo1) = 1227$ lbf
Second opening: $O2 = va2 \times (Lo2) = 1227$ 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

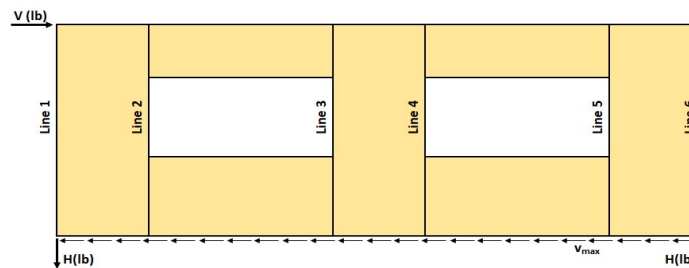
$R1 = v1*L1 = 549$ lbf
 $R2 = v2*L2 = 926$ lbf
 $R3 = v3*L3 = 544$ lbf

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	0
Line 3: $vc2(h_a1+h_b1)+v2(h_o1)-va1(h_a1+h_b1)=0?$	-590	1511	0
Line 4: $va2(h_a2+h_b2)-v2(h_o2)-vc2(h_a2+h_b2)=0?$	920	1511	0
Line 5: $va2(h_a2+h_b2)-vc3(h_a2+h_b2)-v3(h_o2)=0?$	920	-67	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.

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 $V_{\text{unfactored}}$

2019

(lbf)

Wood End Post Values:

Species:

HF#2

E:

1.30E+06

(psi)

Nail Type:

8d common

(penny weight)

Sheathing Type:

7/16 OSB

Grade:

APA Rated Sheathing

Enter individual post sizes below.

G_1 Override:

G_2 Override:

C_{ej}

4.00

Nail Spacing:

4

Pier 1

4

Pier 3

(in.)

HD Capacity:

2140

2140

(lbf)

HD Deflection:

0.088

0.088

(in.)

Four-Term Equation Deflection Check

$$\Delta = \frac{8vh^3}{FAb} + \frac{vh}{Gt} + 0.75he_n + d_a \frac{h}{b} \quad (\text{Equation 23-2})$$

[illegible]

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	(in.) %drift
Defl.	

Project Information

Code:

Designer: Chon Pieruccioni, PE

Date: 6/2/2025

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

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 3

Nail Spacing: 4

4

(in.)

HD Capacity: 2140

2140

(lbf)

HD Deflection: 0.088

0.088

(in.)

Three-Term Equation Deflection Check

$$\delta_{sw} = \frac{8vh^3}{EAb} + \frac{vh}{1000G_a} + \frac{h\Delta_a}{b}$$

(4.3-1)

	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	Term 2	Term 3	Term 1	Term 2	Term 3
Bending	Shear	Fastener	Bending	Shear	Fastener
	0.089	0.290		0.056	0.112
Sum			Sum		
Pier 2 (left)			Pier 2 (right)		
Term 1	Term 2	Term 3	Term 1	Term 2	Term 3
Bending	Shear	Fastener	Bending	Shear	Fastener
	0.085	0.157		0.085	0.157
Sum			Sum		
Pier 3 (left)			Pier 3 (right)		
Term 1	Term 2	Term 3	Term 1	Term 2	Term 3
Bending	Shear	Fastener	Bending	Shear	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.



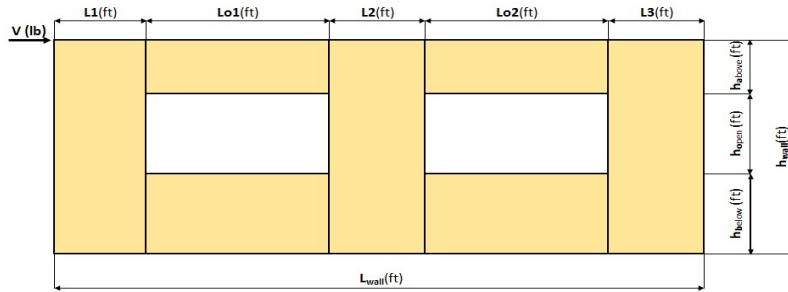
Force Transfer Around Openings Calculator

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

V	2913 lbf	Opening 1	Opening 2	Adj. Factor Method =	1.25-0.125h/bs
L1	2.51 ft	ha1	ha2	Wall Pier Aspect Ratio	Adj. Factor
L2	2.76 ft	ho1	ho2	P1=ho/L1=	1.79
L3	2.48 ft	hb1	hb2	P2=ho/L2=	1.63
hwall	9.00 ft	Lo1	Lo2	P3=ho/L3=	1.81
Lwall	19.75 ft				

1. Hold-down forces: $H = Vh_{wall}/L_{wall}$ 1327 lbf

2. Unit shear above + below opening
First opening: $va1 = vb1 = H/(h_a1+h_b1) = 295 \text{ plf}$
Second opening: $va2 = vb2 = H/(h_a2+h_b2) = 295 \text{ plf}$

3. Total boundary force above + below openings
First opening: $O1 = va1 \times (Lo1) = 1770 \text{ lbf}$
Second opening: $O2 = va2 \times (Lo2) = 1770 \text{ lbf}$

4. Corner forces
 $F1 = O1(L1)/(L1+L2) = 843 \text{ lbf}$
 $F2 = O1(L2)/(L1+L2) = 927 \text{ lbf}$
 $F3 = O2(L2)/(L2+L3) = 932 \text{ lbf}$
 $F4 = O2(L3)/(L2+L3) = 838 \text{ lbf}$

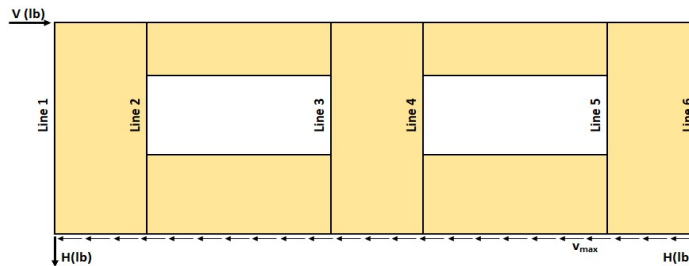
5. Tributary length of openings
 $T1 = (L1*Lo1)/(L1+L2) = 2.86 \text{ ft}$
 $T2 = (L2*Lo1)/(L1+L2) = 3.14 \text{ ft}$
 $T3 = (L2*Lo2)/(L2+L3) = 3.16 \text{ ft}$
 $T4 = (L3*Lo2)/(L2+L3) = 2.84 \text{ ft}$

6. Unit shear beside opening
 $v1 = (V/L)(L1+T1)/L1 = 315 \text{ plf}$
 $v2 = (V/L)(T2+L2+T3)/L2 = 484 \text{ plf}$
 $v3 = (V/L)(T4+L3)/L3 = 316 \text{ plf}$
Check $v1*L1+v2*L2+v3*L3=V?$ 2913 lbf OK

7. Resistance to corner forces
 $R1 = v1*L1 = 792 \text{ lbf}$
 $R2 = v2*L2 = 1337 \text{ lbf}$
 $R3 = v3*L3 = 785 \text{ lbf}$

8. Difference corner force + resistance
 $R1-F1 = -51 \text{ lbf}$
 $R2-F2-F3 = -523 \text{ lbf}$
 $R3-F4 = -53 \text{ lbf}$

9. Unit shear in corner zones
 $vc1 = (R1-F1)/L1 = -20 \text{ plf}$
 $vc2 = (R2-F2-F3)/L2 = -189 \text{ plf}$
 $vc3 = (R3-F4)/L3 = -21 \text{ plf}$



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	0
Line 3: $vc2(h_a1+h_b1)+v2(h_o2)-va1(h_a1+h_b1)=0?$	-852	2179	1327
Line 4: $va2(h_a2+h_b2)-v2(h_o2)-vc2(h_a2+h_b2)=0?$	1327	2179	-852
Line 5: $va2(h_a2+h_b2)-vc3(h_a2+h_b2)-v3(h_o2)=0?$	1327	-96	1424
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:	
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		Wood End Post Values:		Nail Type:		8d common (penny weight)	
Grade:		APA Rated Sheathing		Species:		HF#2			
				E:		1.30E+06 (psi)			
G _l Override:				Enter individual post sizes below.					
G _a Override:				C _d :		4.00			

Four-Term Equation Deflection Check

$$\Delta = \frac{8vh^3}{EAb} + \frac{vh}{Gt} + 0.75he_n + d_a \frac{h}{b}$$
(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	Term 2	Term 3	Term 4	Term 1	Term 2	Term 3	Term 4
Bending	Shear	Fastener	HD-1	Bending	Shear	Fastener	HD-2
	0.034	0.005	0.419		0.021	0.003	0.162
Sum				Sum			
Pier 2 (left)				Pier 2 (right)			
Term 1	Term 2	Term 3	Term 4	Term 1	Term 2	Term 3	Term 4
Bending	Shear	Fastener	HD-1	Bending	Shear	Fastener	HD-2
	0.032	0.011	0.226		0.032	0.011	0.226
Sum				Sum			
Pier 3 (left)				Pier 3 (right)			
Term 1	Term 2	Term 3	Term 4	Term 1	Term 2	Term 3	Term 4
Bending	Shear	Fastener	HD-1	Bending	Shear	Fastener	HD-2
	0.021	0.003	0.165		0.034	0.005	0.425
Sum				Sum			

Total
Defl.
(in.)
%drift

Project Information

Code:

Designer: Chon Pieruccioni, PE

Date: 6/2/2025

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

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 3

Nail Spacing: 2 (in.)

HD Capacity: 2140 (lbf)

HD Deflection: 0.088 (in.)

Three-Term Equation Deflection Check

$$\delta_{sw} = \frac{8vh^3}{EAb} + \frac{vh}{1000G_a} + \frac{h\Delta_a}{b}$$

(4.3-1)

	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	Term 2	Term 3	Term 1	Term 2	Term 3
Bending	Shear	Fastener	Bending	Shear	Fastener
	0.068	0.419		0.042	0.162
Sum			Sum		
Pier 2 (left)			Pier 2 (right)		
Term 1	Term 2	Term 3	Term 1	Term 2	Term 3
Bending	Shear	Fastener	Bending	Shear	Fastener
	0.065	0.226		0.065	0.226
Sum			Sum		
Pier 3 (left)			Pier 3 (right)		
Term 1	Term 2	Term 3	Term 1	Term 2	Term 3
Bending	Shear	Fastener	Bending	Shear	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.



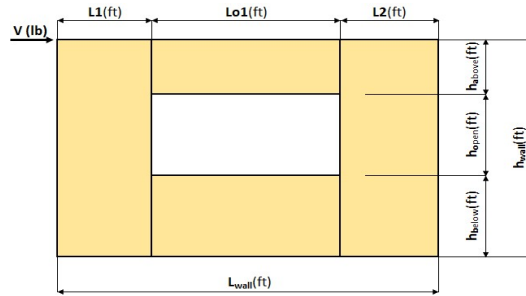
Force Transfer Around Openings Calculator

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	ha	1.10 ft
L2	3.83 ft	ho	4.50 ft
hwall	9.00 ft	hb	3.40 ft
Lwall	13.33 ft	Lo1	5.00 ft
		Wall Pier Aspect Ratio	Adj. Factor
		P1=h _o /L1=	1.00 N/A
		P2=h _o /L2=	1.17 N/A

1. Hold-down forces: $H = Vh_{wall}/L_{wall}$ 1058 lbf

2. Unit shear above + below opening
First opening: $va1 = vb1 = H/(h_a + h_b) = 235 \text{ plf}$

3. Total boundary force above + below openings
First opening: $O1 = va1 \times (Lo1) = 1176 \text{ lbf}$

4. Corner forces
 $F1 = O1(L1)/(L1+L2) = 635 \text{ lbf}$
 $F2 = O1(L2)/(L1+L2) = 540 \text{ lbf}$

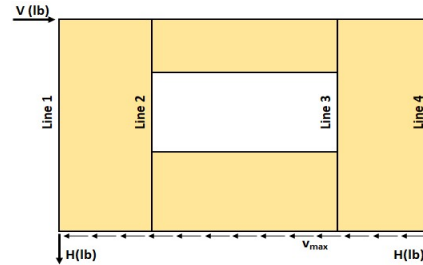
5. Tributary length of openings
 $T1 = (L1 \times Lo1)/(L1+L2) = 2.70 \text{ ft}$
 $T2 = (L2 \times Lo1)/(L1+L2) = 2.30 \text{ ft}$

6. Unit shear beside opening
 $v1 = (V/L)(L1+T1)/L1 = 188 \text{ plf}$
 $v2 = (V/L)(T2+L2)/L2 = 188 \text{ plf}$
Check $v1 \times L1 + v2 \times L2 = V?$ 1567 lbf OK

7. Resistance to corner forces
 $R1 = v1 \times L1 = 847 \text{ lbf}$
 $R2 = v2 \times L2 = 720 \text{ lbf}$

8. Difference corner force + resistance
 $R1 - F1 = 211 \text{ lbf}$
 $R2 - F2 = 180 \text{ lbf}$

9. Unit shear in corner zones
 $vc1 = (R1 - F1)/L1 = 47 \text{ plf}$
 $vc2 = (R2 - F2)/L2 = 47 \text{ plf}$



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
Line 3: $va1(h_a + h_b) - vc2(h_a + h_b) - v1(h_o) = 0?$	1058	211	847
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:

Designer: Chon Pieruccioni, PE

Date: 6/2/2025

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_i 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

$$\Delta = \frac{8vh^3}{EAb} + \frac{vh}{Gt} + 0.75he_n + d_a \frac{h}{b}$$

(Equation 23-2)

	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.)

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	Term 2	Term 3	Term 4	Term 1	Term 2	Term 3	Term 4
Bending	Shear	Fastener	HD-1	Bending	Shear	Fastener	HD-2
	0.020	0.028	0.139		0.013	0.017	0.054
Sum				Sum			
Pier 2 (left)				Pier 2 (right)			
Term 1	Term 2	Term 3	Term 4	Term 1	Term 2	Term 3	Term 4
Bending	Shear	Fastener	HD-1	Bending	Shear	Fastener	HD-2
	0.013	0.017	0.063		0.020	0.028	0.164
Sum				Sum			

Total

Defl.

(in.)
%drift

Project Information

Code:

Designer: Chon Pieruccioni, PE

Date: 6/2/2025

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

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 (in.)

HD Capacity: 2140 (lbf)

HD Deflection: 0.088 (in.)

Three-Term Equation Deflection Check

$$\delta_{sw} = \frac{8vh^3}{EAb} + \frac{vh}{1000G_a} + \frac{h\Delta_a}{b} \quad (4.3-1)$$

	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	Term 2	Term 3	Term 1	Term 2	Term 3
Bending	Shear	Fastener	Bending	Shear	Fastener
	0.113	0.139		0.070	0.054
Sum			Sum		
Pier 2 (left)			Pier 2 (right)		
Term 1	Term 2	Term 3	Term 1	Term 2	Term 3
Bending	Shear	Fastener	Bending	Shear	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.



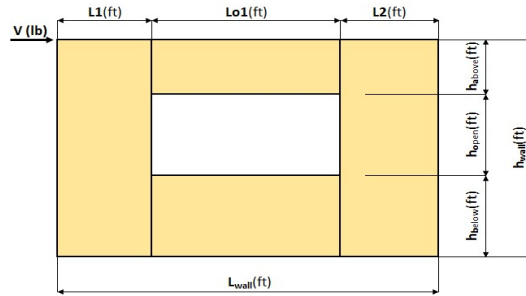
Force Transfer Around Openings Calculator

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
h _{wall}	9.00 ft	h _b	3.40 ft	P2=h _o /L2=	1.17
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: $va1 = vb1 = H/(h_a + h_b) = 382 \text{ plf}$

3. Total boundary force above + below openings
First opening: $O1 = va1 \times (Lo1) = 1909 \text{ lbf}$

4. Corner forces
 $F1 = O1(L1)/(L1+L2) = 1031 \text{ lbf}$
 $F2 = O1(L2)/(L1+L2) = 878 \text{ lbf}$

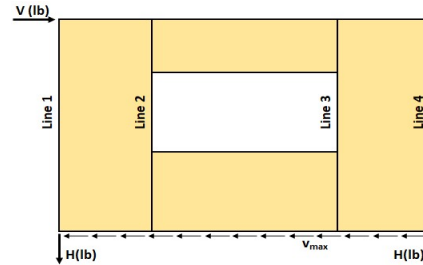
5. Tributary length of openings
 $T1 = (L1 \times Lo1)/(L1+L2) = 2.70 \text{ ft}$
 $T2 = (L2 \times Lo1)/(L1+L2) = 2.30 \text{ ft}$

6. Unit shear beside opening
 $v1 = (V/L)(L1+T1)/L1 = 306 \text{ plf}$
 $v2 = (V/L)(T2+L2)/L2 = 306 \text{ plf}$
Check $v1 \times L1 + v2 \times L2 = V?$ 2545 lbf OK

7. Resistance to corner forces
 $R1 = v1 \times L1 = 1375 \text{ lbf}$
 $R2 = v2 \times L2 = 1170 \text{ lbf}$

8. Difference corner force + resistance
 $R1 - F1 = 343 \text{ lbf}$
 $R2 - F2 = 292 \text{ lbf}$

9. Unit shear in corner zones
 $vc1 = (R1 - F1)/L1 = 76 \text{ plf}$
 $vc2 = (R2 - F2)/L2 = 76 \text{ plf}$



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:

Designer: Chon Pieruccioni, PE

Date: 6/2/2025

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_i 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)

Nail Spacing: 3

HD Capacity: 2140

HD Deflection: 0.088

Pier 1

Pier 2

(in.)

(lbf)

(in.)

Four-Term Equation Deflection Check

$$\Delta = \frac{8vh^3}{EAb} + \frac{vh}{Gt} + 0.75he_n + d_a \frac{h}{b}$$

(Equation 23-2)

	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.)

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	Term 2	Term 3	Term 4	Term 1	Term 2	Term 3	Term 4
Bending	Shear	Fastener	HD-1	Bending	Shear	Fastener	HD-2
	0.033	0.015	0.226		0.020	0.009	0.088
Sum				Sum			
Pier 2 (left)				Pier 2 (right)			
Term 1	Term 2	Term 3	Term 4	Term 1	Term 2	Term 3	Term 4
Bending	Shear	Fastener	HD-1	Bending	Shear	Fastener	HD-2
	0.020	0.009	0.103		0.033	0.015	0.266
Sum				Sum			

Total Defl.

(in.) %drift

Project Information

Code:

Designer: Chon Pieruccioni, PE

Date: 6/2/2025

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

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

HD Capacity: 2140

HD Deflection: 0.088

(in.)

(lbf)

(in.)

Three-Term Equation Deflection Check

$$\delta_{sw} = \frac{8vh^3}{Eab} + \frac{vh}{1000G_a} + \frac{h\Delta_a}{b}$$

(4.3-1)

	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.)

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	Term 2	Term 3	Term 1	Term 2	Term 3
Bending	Shear	Fastener	Bending	Shear	Fastener
	0.098	0.226		0.061	0.088
Sum			Sum		
Pier 2 (left)			Pier 2 (right)		
Term 1	Term 2	Term 3	Term 1	Term 2	Term 3
Bending	Shear	Fastener	Bending	Shear	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.



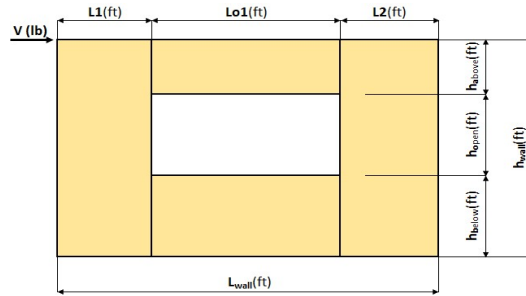
Force Transfer Around Openings Calculator

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	ha	Wall Pier Aspect Ratio	Adj. Factor
L2	3.83 ft	ho	P1=h _o /L1=	1.00
h _{wall}	9.00 ft	hb	P2=h _o /L2=	1.17
L _{wall}	13.33 ft	Lo1		N/A

1. Hold-down forces: $H = Vh_{\text{wall}}/L_{\text{wall}}$ 1999 lbf

2. Unit shear above + below opening
First opening: $va1 = vb1 = H/(h_a + h_b) = 444 \text{ plf}$

3. Total boundary force above + below openings
First opening: $O1 = va1 \times (Lo1) = 2221 \text{ lbf}$

4. Corner forces
 $F1 = O1(L1)/(L1+L2) = 1200 \text{ lbf}$
 $F2 = O1(L2)/(L1+L2) = 1021 \text{ lbf}$

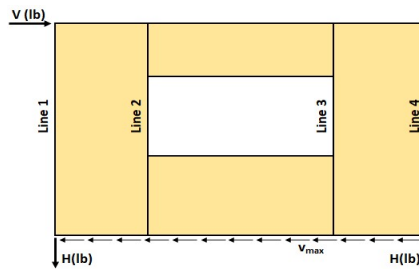
5. Tributary length of openings
 $T1 = (L1 \times Lo1)/(L1+L2) = 2.70 \text{ ft}$
 $T2 = (L2 \times Lo1)/(L1+L2) = 2.30 \text{ ft}$

6. Unit shear beside opening
 $v1 = (V/L)(L1+T1)/L1 = 355 \text{ plf}$
 $v2 = (V/L)(T2+L2)/L2 = 355 \text{ plf}$
Check $v1 \times L1 + v2 \times L2 = V?$ 2961 lbf OK

7. Resistance to corner forces
 $R1 = v1 \times L1 = 1600 \text{ lbf}$
 $R2 = v2 \times L2 = 1361 \text{ lbf}$

8. Difference corner force + resistance
 $R1 - F1 = 400 \text{ lbf}$
 $R2 - F2 = 340 \text{ lbf}$

9. Unit shear in corner zones
 $vc1 = (R1 - F1)/L1 = 89 \text{ plf}$
 $vc2 = (R2 - F2)/L2 = 89 \text{ plf}$



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:

Designer: Chon Pieruccioni, PE

Date: 6/2/2025

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_i 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)

Nail Spacing: 3

HD Capacity: 2140

HD Deflection: 0.088

Pier 1

Pier 2

(in.)

(lbf)

(in.)

Four-Term Equation Deflection Check

$$\Delta = \frac{8vh^3}{EAb} + \frac{vh}{Gt} + 0.75he_n + d_a \frac{h}{b}$$

(Equation 23-2)

	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.)

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	Term 2	Term 3	Term 4	Term 1	Term 2	Term 3	Term 4
Bending	Shear	Fastener	HD-1	Bending	Shear	Fastener	HD-2
	0.038	0.023	0.263		0.024	0.015	0.102
Sum				Sum			
Pier 2 (left)				Pier 2 (right)			
Term 1	Term 2	Term 3	Term 4	Term 1	Term 2	Term 3	Term 4
Bending	Shear	Fastener	HD-1	Bending	Shear	Fastener	HD-2
	0.024	0.015	0.120		0.038	0.023	0.309
Sum				Sum			

Total

Defl.

(in.)

%drift

Project Information

Code:

Designer: Chon Pieruccioni, PE

Date: 6/2/2025

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

HD Capacity: 2140

HD Deflection: 0.088

(in.)

(lbf)

(in.)

Three-Term Equation Deflection Check

$$\delta_{sw} = \frac{8vh^3}{EAb} + \frac{vh}{1000G_a} + \frac{h\Delta_a}{b} \quad (4.3-1)$$

	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.)

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	Term 2	Term 3	Term 1	Term 2	Term 3
Bending	Shear	Fastener	Bending	Shear	Fastener
	0.114	0.263		0.071	0.102
Sum			Sum		
Pier 2 (left)			Pier 2 (right)		
Term 1	Term 2	Term 3	Term 1	Term 2	Term 3
Bending	Shear	Fastener	Bending	Shear	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.



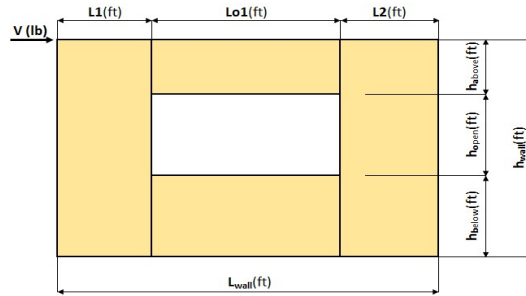
Force Transfer Around Openings Calculator

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		Adj. Factor Method = 1.25-0.125h/bs	
L1	4.92 ft	h _a	1.10 ft	Wall Pier Aspect Ratio	Adj. Factor
L2	4.91 ft	h _o	4.50 ft	P1=h _o /L1=	0.91
h _{wall}	9.00 ft	h _b	3.40 ft	P2=h _o /L2=	0.92
L _{wall}	14.83 ft	Lo1	5.00 ft		

1. Hold-down forces: $H = Vh_{wall}/L_{wall}$ 1058 lbf

2. Unit shear above + below opening
First opening: $va1 = vb1 = H/(h_a + h_b) = 235 \text{ plf}$

3. Total boundary force above + below openings
First opening: $O1 = va1 \times (Lo1) = 1176 \text{ lbf}$

4. Corner forces
 $F1 = O1(L1)/(L1+L2) = 589 \text{ lbf}$
 $F2 = O1(L2)/(L1+L2) = 587 \text{ lbf}$

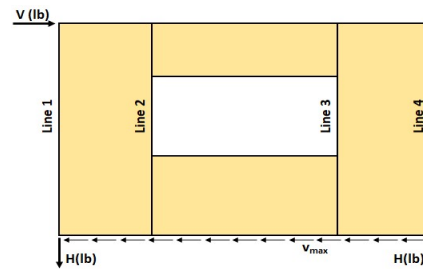
5. Tributary length of openings
 $T1 = (L1 \times Lo1)/(L1+L2) = 2.50 \text{ ft}$
 $T2 = (L2 \times Lo1)/(L1+L2) = 2.50 \text{ ft}$

6. Unit shear beside opening
 $v1 = (V/L)(L1+T1)/L1 = 177 \text{ plf}$
 $v2 = (V/L)(T2+L2)/L2 = 177 \text{ plf}$
Check $v1 \times L1 + v2 \times L2 = V?$ 1744 lbf OK

7. Resistance to corner forces
 $R1 = v1 \times L1 = 873 \text{ lbf}$
 $R2 = v2 \times L2 = 871 \text{ lbf}$

8. Difference corner force + resistance
 $R1 - F1 = 284 \text{ lbf}$
 $R2 - F2 = 284 \text{ lbf}$

9. Unit shear in corner zones
 $vc1 = (R1 - F1)/L1 = 58 \text{ plf}$
 $vc2 = (R2 - F2)/L2 = 58 \text{ plf}$



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:

Designer: Chon Pieruccioni, PE

Date: 6/2/2025

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_i 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)

Nail Spacing: 6

HD Capacity: 2140

HD Deflection: 0.088

Pier 1

Pier 2

(in.)

(lbf)

(in.)

Four-Term Equation Deflection Check

$$\Delta = \frac{8vh^3}{EAb} + \frac{vh}{Gt} + 0.75he_n + d_a \frac{h}{b}$$

(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	Term 2	Term 3	Term 4	Term 1	Term 2	Term 3	Term 4
Bending	Shear	Fastener	HD-1	Bending	Shear	Fastener	HD-2
	0.019	0.023	0.120		0.012	0.015	0.047
Sum				Sum			
Pier 2 (left)				Pier 2 (right)			
Term 1	Term 2	Term 3	Term 4	Term 1	Term 2	Term 3	Term 4
Bending	Shear	Fastener	HD-1	Bending	Shear	Fastener	HD-2
	0.012	0.015	0.047		0.019	0.023	0.120
Sum				Sum			

Total Defl.

(in.) %drift

Project Information

Code:

Designer: Chon Pieruccioni, PE

Date: 6/2/2025

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

HD Capacity: 2140

HD Deflection: 0.088

(in.)

(lbf)

(in.)

Three-Term Equation Deflection Check

$$\delta_{sw} = \frac{8vh^3}{Eab} + \frac{vh}{1000G_a} + \frac{h\Delta_a}{b} \quad (4.3-1)$$

	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	Term 2	Term 3	Term 1	Term 2	Term 3
Bending	Shear	Fastener	Bending	Shear	Fastener
	0.106	0.120		0.066	0.047
Sum			Sum		
Pier 2 (left)			Pier 2 (right)		
Term 1	Term 2	Term 3	Term 1	Term 2	Term 3
Bending	Shear	Fastener	Bending	Shear	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.



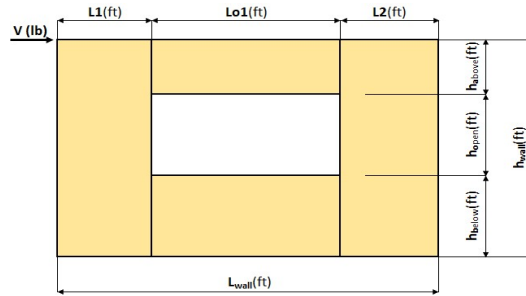
Force Transfer Around Openings Calculator

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		Adj. Factor Method = 1.25-0.125h/bs	
L1	4.92 ft	h _a	1.10 ft	Wall Pier Aspect Ratio	Adj. Factor
L2	4.91 ft	h _o	4.50 ft	P1=h _o /L1=	0.91
h _{wall}	9.00 ft	h _b	3.40 ft	P2=h _o /L2=	0.92
L _{wall}	14.83 ft	Lo1	5.00 ft		

1. Hold-down forces: $H = Vh_{wall}/L_{wall}$ 1717 lbf

2. Unit shear above + below opening
First opening: $va1 = vb1 = H/(h_a + h_b) = 382 \text{ plf}$

3. Total boundary force above + below openings
First opening: $O1 = va1 \times (Lo1) = 1908 \text{ lbf}$

4. Corner forces
 $F1 = O1(L1)/(L1+L2) = 955 \text{ lbf}$
 $F2 = O1(L2)/(L1+L2) = 953 \text{ lbf}$

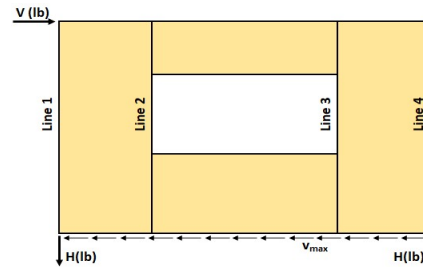
5. Tributary length of openings
 $T1 = (L1 \times Lo1)/(L1+L2) = 2.50 \text{ ft}$
 $T2 = (L2 \times Lo1)/(L1+L2) = 2.50 \text{ ft}$

6. Unit shear beside opening
 $v1 = (V/L)(L1+T1)/L1 = 288 \text{ plf}$
 $v2 = (V/L)(T2+L2)/L2 = 288 \text{ plf}$
Check $v1 \times L1 + v2 \times L2 = V?$ 2830 lbf OK

7. Resistance to corner forces
 $R1 = v1 \times L1 = 1416 \text{ lbf}$
 $R2 = v2 \times L2 = 1414 \text{ lbf}$

8. Difference corner force + resistance
 $R1 - F1 = 461 \text{ lbf}$
 $R2 - F2 = 460 \text{ lbf}$

9. Unit shear in corner zones
 $vc1 = (R1 - F1)/L1 = 94 \text{ plf}$
 $vc2 = (R2 - F2)/L2 = 94 \text{ plf}$



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:

Designer: Chon Pieruccioni, PE

Date: 6/2/2025

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_i 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

$$\Delta = \frac{8vh^3}{EAb} + \frac{vh}{Gt} + 0.75he_n + d_a \frac{h}{b}$$
 (Equation 23-2)

Pier 1-L

Pier 1-R

Pier 2-L

Pier 2-R

$V_{unfactored}$: 288288288 (plf)

E : 1.30E+061.30E+061.30E+061.30E+06 (psi)

h : 9.005.605.609.00 (ft)

Qty:

Stud Size:

A Override:

A :

G_i : 83,50083,50083,50083,500 (lbf/in.)

Nail Spacing: 3333 (in.)

V_n : 72727272 (plf)

e_n : 0.00180.00180.00180.0018 (in.)

b : 4.924.924.914.91 (ft)

HD Capacity: 2140214021402140 (lbf)

HD Defl: 0.0880.0880.0880.088 (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	Term 2	Term 3	Term 4	Term 1	Term 2	Term 3	Term 4
Bending	Shear	Fastener	HD-1	Bending	Shear	Fastener	HD-2
	0.031	0.012	0.195		0.019	0.008	0.075
Sum				Sum			
Pier 2 (left)				Pier 2 (right)			
Term 1	Term 2	Term 3	Term 4	Term 1	Term 2	Term 3	Term 4
Bending	Shear	Fastener	HD-1	Bending	Shear	Fastener	HD-2
	0.019	0.008	0.076		0.031	0.012	0.195
Sum				Sum			

Total Defl.

(in.) %drift

Project Information

Code:

Designer: Chon Pieruccioni, PE

Date: 6/2/2025

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

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

HD Capacity: 2140

HD Deflection: 0.088

(in.)

(lbf)

(in.)

Three-Term Equation Deflection Check

$$\delta_{sw} = \frac{8vh^3}{EAb} + \frac{vh}{1000G_a} + \frac{h\Delta_a}{b}$$

(4.3-1)

	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.)

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	Term 2	Term 3	Term 1	Term 2	Term 3
Bending	Shear	Fastener	Bending	Shear	Fastener
	0.093	0.195		0.058	0.075
Sum			Sum		
Pier 2 (left)			Pier 2 (right)		
Term 1	Term 2	Term 3	Term 1	Term 2	Term 3
Bending	Shear	Fastener	Bending	Shear	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.



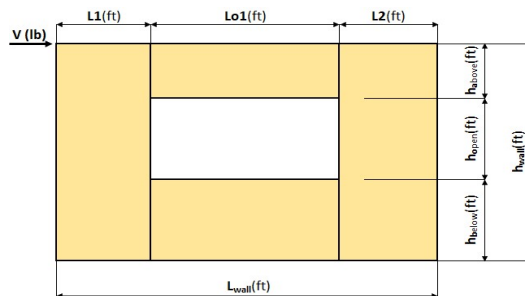
Force Transfer Around Openings Calculator

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		Adj. Factor Method = 1.25-0.125h/bs	
L1	4.92 ft	h _a	1.10 ft	Wall Pier Aspect Ratio	
L2	4.91 ft	h _o	4.50 ft	P1=h _o /L1=	0.91
h _{wall}	9.00 ft	h _b	3.40 ft	P2=h _o /L2=	0.92
L _{wall}	14.83 ft	Lo1	5.00 ft		

1. Hold-down forces: $H = Vh_{wall}/L_{wall}$ 1999 lbf

2. Unit shear above + below opening
First opening: $va1 = vb1 = H/(h_a + h_b) = 444 \text{ plf}$

3. Total boundary force above + below openings
First opening: $O1 = va1 \times (Lo1) = 2221 \text{ lbf}$

4. Corner forces
 $F1 = O1(L1)/(L1+L2) = 1112 \text{ lbf}$
 $F2 = O1(L2)/(L1+L2) = 1109 \text{ lbf}$

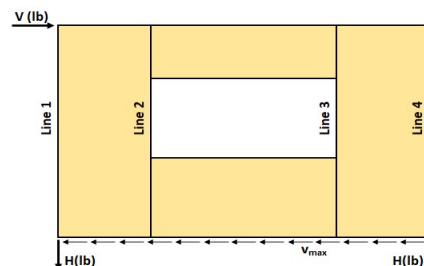
5. Tributary length of openings
 $T1 = (L1 \times Lo1)/(L1+L2) = 2.50 \text{ ft}$
 $T2 = (L2 \times Lo1)/(L1+L2) = 2.50 \text{ ft}$

6. Unit shear beside opening
 $v1 = (V/L)(L1+T1)/L1 = 335 \text{ plf}$
 $v2 = (V/L)(T2+L2)/L2 = 335 \text{ plf}$
Check $v1 \times L1 + v2 \times L2 = V?$ 3294 lbf OK

7. Resistance to corner forces
 $R1 = v1 \times L1 = 1649 \text{ lbf}$
 $R2 = v2 \times L2 = 1645 \text{ lbf}$

8. Difference corner force + resistance
 $R1 - F1 = 537 \text{ lbf}$
 $R2 - F2 = 536 \text{ lbf}$

9. Unit shear in corner zones
 $vc1 = (R1 - F1)/L1 = 109 \text{ plf}$
 $vc2 = (R2 - F2)/L2 = 109 \text{ plf}$



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:

Designer: Chon Pieruccioni, PE

Date: 6/2/2025

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)

Sheathing Type: 7/16 OSB

Grade: APA Rated Sheathing

G_i 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

$$\Delta = \frac{8vh^3}{EAb} + \frac{vh}{Gt} + 0.75he_n + d_a \frac{h}{b}$$
 (Equation 23-2)

	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.)

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	Term 2	Term 3	Term 4	Term 1	Term 2	Term 3	Term 4
Bending	Shear	Fastener	HD-1	Bending	Shear	Fastener	HD-2
	0.036	0.020	0.227		0.022	0.012	0.088
Sum				Sum			
Pier 2 (left)				Pier 2 (right)			
Term 1	Term 2	Term 3	Term 4	Term 1	Term 2	Term 3	Term 4
Bending	Shear	Fastener	HD-1	Bending	Shear	Fastener	HD-2
	0.022	0.012	0.088		0.036	0.020	0.227
Sum				Sum			

Total Defl.

(in.) %drift

Project Information

Code:

Designer: Chon Pieruccioni, PE

Date: 6/2/2025

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)

Sheathing Type: 7/16 OSB

Grade: APA Rated Sheathing

G_i Override:

G_a Override:

Wood End Post Values:

Species: HF#2

E: 1.30E+06 (psi)

C_d : 4.00

Nail Type: 8d common (penny weight)

	Pier 1	Pier 2
Nail Spacing:	3	3
HD Capacity:	2140	2140
HD Deflection:	0.088	0.088

Three-Term Equation Deflection Check

$$\delta_{sw} = \frac{8vh^3}{Eab} + \frac{vh}{1000G_a} + \frac{h\Delta_a}{b}$$

(4.3-1)

	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	Term 2	Term 3	Term 1	Term 2	Term 3
Bending	Shear	Fastener	Bending	Shear	Fastener
	0.108	0.227		0.067	0.088
Sum			Sum		
Pier 2 (left)			Pier 2 (right)		
Term 1	Term 2	Term 3	Term 1	Term 2	Term 3
Bending	Shear	Fastener	Bending	Shear	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.



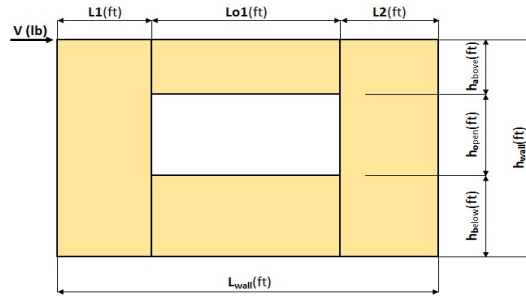
Force Transfer Around Openings Calculator

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	Adj. Factor Method = 1.25-0.125h/bs
L1	2.88 ft	ha	1.25 ft
L2	2.87 ft	ho	6.00 ft
hwall	9.00 ft	hb	1.75 ft
Lwall	11.75 ft	Lo1	6.00 ft
		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: $H = Vh_{\text{wall}}/L_{\text{wall}}$ 1419 lbf

2. Unit shear above + below opening
First opening: $va1 = vb1 = H/(h_a + h_b) = 473 \text{ plf}$

3. Total boundary force above + below openings
First opening: $O1 = va1 \times (Lo1) = 2839 \text{ lbf}$

4. Corner forces
 $F1 = O1(L1)/(L1+L2) = 1422 \text{ lbf}$
 $F2 = O1(L2)/(L1+L2) = 1417 \text{ lbf}$

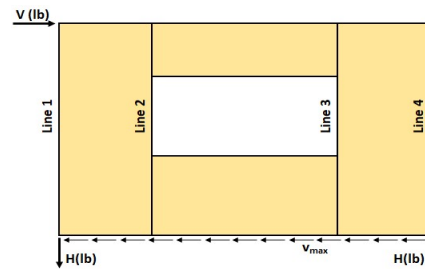
5. Tributary length of openings
 $T1 = (L1 \times Lo1)/(L1+L2) = 3.01 \text{ ft}$
 $T2 = (L2 \times Lo1)/(L1+L2) = 2.99 \text{ ft}$

6. Unit shear beside opening
 $v1 = (V/L)(L1+T1)/L1 = 322 \text{ plf}$
 $v2 = (V/L)(T2+L2)/L2 = 322 \text{ plf}$
Check $v1 \times L1 + v2 \times L2 = V?$ 1853 lbf OK

7. Resistance to corner forces
 $R1 = v1 \times L1 = 928 \text{ lbf}$
 $R2 = v2 \times L2 = 925 \text{ lbf}$

8. Difference corner force + resistance
 $R1 - F1 = -494 \text{ lbf}$
 $R2 - F2 = -492 \text{ lbf}$

9. Unit shear in corner zones
 $vc1 = (R1 - F1)/L1 = -171 \text{ plf}$
 $vc2 = (R2 - F2)/L2 = -171 \text{ plf}$



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:

Designer: Chon Pieruccioni, PE

Date: 6/2/2025

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_i 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)

Nail Spacing: 2

HD Capacity: 2140

HD Deflection: 0.088

Pier 1

Pier 2

(in.)

(lbf)

(in.)

Four-Term Equation Deflection Check

$$\Delta = \frac{8vh^3}{EAb} + \frac{vh}{Gt} + 0.75he_n + d_a \frac{h}{b}$$
 (Equation 23-2)

Pier 1-L

Pier 1-R

Pier 2-L

Pier 2-R

$V_{unfactored}$: 322

E: 1.30E+06

h: 9.00

Qty:

Stud Size:

A Override:

A:

G_i : 83,500

Nail Spacing: 2

V_n : 54

e_n : 0.0008

b: 2.88

HD Capacity: 2140

HD Defl: 0.088

322

1.30E+06

7.25

83,500

2

54

0.0008

2.88

2140

0.088

322

1.30E+06

9.00

83,500

2

54

0.0008

2.87

2140

0.088

322

1.30E+06

7.25

83,500

2

54

0.0008

2.87

2140

0.088

322

1.30E+06

9.00

83,500

2

54

0.0008

2.87

2140

0.088

(plf)

(psi)

(ft)

(in.²)

(in.²)

(lbf/in.)

(in.)

(plf)

(in.)

(ft)

(lbf)

(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	Term 2	Term 3	Term 4	Term 1	Term 2	Term 3	Term 4
Bending	Shear	Fastener	HD-1	Bending	Shear	Fastener	HD-2
	0.035	0.005	0.373		0.028	0.004	0.242
Sum				Sum			
Pier 2 (left)				Pier 2 (right)			
Term 1	Term 2	Term 3	Term 4	Term 1	Term 2	Term 3	Term 4
Bending	Shear	Fastener	HD-1	Bending	Shear	Fastener	HD-2
	0.028	0.004	0.243		0.035	0.005	0.374
Sum				Sum			

Total Defl.

(in.) %drift

Project Information

Code:

Designer: Chon Pieruccioni, PE

Date: 6/2/2025

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

HD Capacity: 2140

2140

HD Deflection: 0.088

0.088

Three-Term Equation Deflection Check

$$\delta_{sw} = \frac{8vh^3}{EAb} + \frac{vh}{1000G_a} + \frac{h\Delta_a}{b} \quad (4.3-1)$$

	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	Term 2	Term 3	Term 1	Term 2	Term 3
Bending	Shear	Fastener	Bending	Shear	Fastener
	0.069	0.373		0.056	0.242
Sum			Sum		
Pier 2 (left)			Pier 2 (right)		
Term 1	Term 2	Term 3	Term 1	Term 2	Term 3
Bending	Shear	Fastener	Bending	Shear	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.



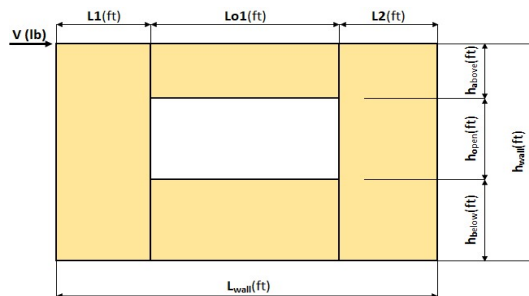
Force Transfer Around Openings Calculator

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		Adj. Factor Method = 1.25-0.125h/bs	
L1	2.88 ft	h _a	1.25 ft	Wall Pier Aspect Ratio	
L2	2.87 ft	h _o	6.00 ft	Adj. Factor	
h _{wall}	9.00 ft	h _b	1.75 ft	P1=h _o /L1=	2.08
L _{wall}	11.75 ft	Lo1	6.00 ft	P2=h _o /L2=	2.09
					0.990
					0.989

1. Hold-down forces: $H = Vh_{\text{wall}}/L_{\text{wall}}$ 1275 lbf

2. Unit shear above + below opening
First opening: $va1 = vb1 = H/(h_a + h_b) = 425 \text{ plf}$

3. Total boundary force above + below openings
First opening: $O1 = va1 \times (Lo1) = 2549 \text{ lbf}$

4. Corner forces
 $F1 = O1(L1)/(L1+L2) = 1277 \text{ lbf}$
 $F2 = O1(L2)/(L1+L2) = 1272 \text{ lbf}$

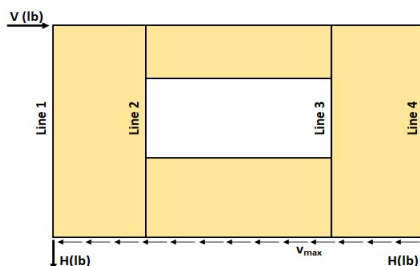
5. Tributary length of openings
 $T1 = (L1 \times Lo1)/(L1+L2) = 3.01 \text{ ft}$
 $T2 = (L2 \times Lo1)/(L1+L2) = 2.99 \text{ ft}$

6. Unit shear beside opening
 $v1 = (V/L)(L1+T1)/L1 = 289 \text{ plf}$
 $v2 = (V/L)(T2+L2)/L2 = 289 \text{ plf}$
Check $v1 \times L1 + v2 \times L2 = V?$ 1664 lbf OK

7. Resistance to corner forces
 $R1 = v1 \times L1 = 833 \text{ lbf}$
 $R2 = v2 \times L2 = 831 \text{ lbf}$

8. Difference corner force + resistance
 $R1 - F1 = -443 \text{ lbf}$
 $R2 - F2 = -442 \text{ lbf}$

9. Unit shear in corner zones
 $vc1 = (R1 - F1)/L1 = -154 \text{ plf}$
 $vc2 = (R2 - F2)/L2 = -154 \text{ plf}$



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:

Designer: Chon Pieruccioni, PE

Date: 6/2/2025

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_i 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)

Nail Spacing: 2

HD Capacity: 2140

HD Deflection: 0.088

Pier 1

Pier 2

2

2140

0.088

(in.)

(lbf)

(in.)

Four-Term Equation Deflection Check

$$\Delta = \frac{8vh^3}{EAb} + \frac{vh}{Gt} + 0.75he_n + d_a \frac{h}{b}$$

(Equation 23-2)

	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.)

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	Term 2	Term 3	Term 4	Term 1	Term 2	Term 3	Term 4
Bending	Shear	Fastener	HD-1	Bending	Shear	Fastener	HD-2
	0.031	0.004	0.335		0.025	0.003	0.217
Sum				Sum			
Pier 2 (left)				Pier 2 (right)			
Term 1	Term 2	Term 3	Term 4	Term 1	Term 2	Term 3	Term 4
Bending	Shear	Fastener	HD-1	Bending	Shear	Fastener	HD-2
	0.025	0.003	0.218		0.031	0.004	0.336
Sum				Sum			

Total

Defl.

(in.)

%drift

Project Information

Code:

Designer: Chon Pieruccioni, PE

Date: 6/2/2025

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_i Override:

G_a Override:

Wood End Post Values:

Species: HF#2

E: 1.30E+06 (psi)

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.)

Three-Term Equation Deflection Check

$$\delta_{sw} = \frac{8vh^3}{EAb} + \frac{vh}{1000G_a} + \frac{h\Delta_a}{b}$$

(4.3-1)

	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	Term 2	Term 3	Term 1	Term 2	Term 3
Bending	Shear	Fastener	Bending	Shear	Fastener
	0.062	0.335		0.050	0.217
Sum			Sum		
Pier 2 (left)			Pier 2 (right)		
Term 1	Term 2	Term 3	Term 1	Term 2	Term 3
Bending	Shear	Fastener	Bending	Shear	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.



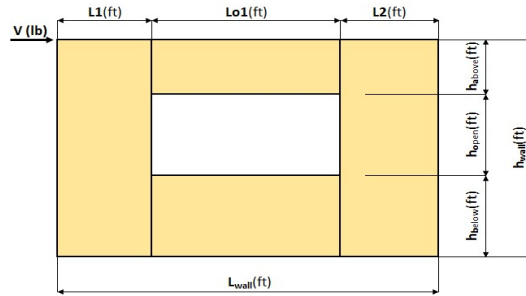
Force Transfer Around Openings Calculator

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 _a /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: $va1 = vb1 = H/(h_a + h_b) = 524 \text{ plf}$

3. Total boundary force above + below openings
First opening: $O1 = va1 \times (Lo1) = 3147 \text{ lbf}$

4. Corner forces
 $F1 = O1(L1)/(L1+L2) = 1576 \text{ lbf}$
 $F2 = O1(L2)/(L1+L2) = 1571 \text{ lbf}$

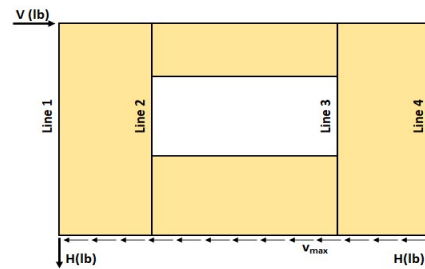
5. Tributary length of openings
 $T1 = (L1 \times Lo1)/(L1+L2) = 3.01 \text{ ft}$
 $T2 = (L2 \times Lo1)/(L1+L2) = 2.99 \text{ ft}$

6. Unit shear beside opening
 $v1 = (V/L)(L1+T1)/L1 = 357 \text{ plf}$
 $v2 = (V/L)(T2+L2)/L2 = 357 \text{ plf}$
Check $v1 \times L1 + v2 \times L2 = V?$ 2054 lbf OK

7. Resistance to corner forces
 $R1 = v1 \times L1 = 1029 \text{ lbf}$
 $R2 = v2 \times L2 = 1025 \text{ lbf}$

8. Difference corner force + resistance
 $R1 - F1 = -547 \text{ lbf}$
 $R2 - F2 = -545 \text{ lbf}$

9. Unit shear in corner zones
 $vc1 = (R1 - F1)/L1 = -190 \text{ plf}$
 $vc2 = (R2 - F2)/L2 = -190 \text{ plf}$



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:

Designer: Chon Pieruccioni, PE

Date: 6/2/2025

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_i 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)

Nail Spacing: 2

HD Capacity: 2140

HD Deflection: 0.088

Pier 1

Pier 2

(in.)

(lbf)

(in.)

Four-Term Equation Deflection Check

$$\Delta = \frac{8vh^3}{EAb} + \frac{vh}{Gt} + 0.75he_n + d_a \frac{h}{b}$$

(Equation 23-2)

	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.)

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	Term 2	Term 3	Term 4	Term 1	Term 2	Term 3	Term 4
Bending	Shear	Fastener	HD-1	Bending	Shear	Fastener	HD-2
	0.039	0.007	0.413		0.031	0.006	0.268
Sum				Sum			
Pier 2 (left)				Pier 2 (right)			
Term 1	Term 2	Term 3	Term 4	Term 1	Term 2	Term 3	Term 4
Bending	Shear	Fastener	HD-1	Bending	Shear	Fastener	HD-2
	0.031	0.006	0.269		0.039	0.007	0.415
Sum				Sum			

Total

Defl.

(in.)

%drift

Project Information

Code:

Designer: Chon Pieruccioni, PE

Date: 6/2/2025

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

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

HD Capacity: 2140

2140

HD Deflection: 0.088

0.088

Three-Term Equation Deflection Check

$$\delta_{sw} = \frac{8vh^3}{EAb} + \frac{vh}{1000G_a} + \frac{h\Delta_a}{b} \quad (4.3-1)$$

	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.)

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	Term 2	Term 3	Term 1	Term 2	Term 3
Bending	Shear	Fastener	Bending	Shear	Fastener
	0.077	0.413		0.062	0.268
Sum			Sum		
Pier 2 (left)			Pier 2 (right)		
Term 1	Term 2	Term 3	Term 1	Term 2	Term 3
Bending	Shear	Fastener	Bending	Shear	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.



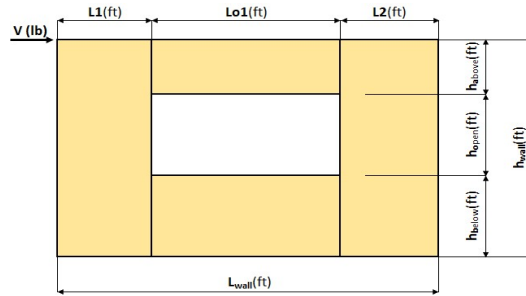
Force Transfer Around Openings Calculator

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
h _{wall}	9.00 ft	h _b	3.40 ft	P2=h _o /L2=	1.54
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: $va1 = vb1 = H/(h_a + h_b) = 315 \text{ plf}$

3. Total boundary force above + below openings
First opening: $O1 = va1 \times (Lo1) = 1577 \text{ lbf}$

4. Corner forces
 $F1 = O1(L1)/(L1+L2) = 895 \text{ lbf}$
 $F2 = O1(L2)/(L1+L2) = 682 \text{ lbf}$

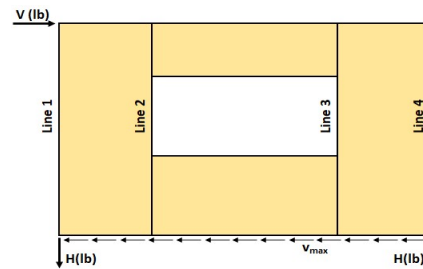
5. Tributary length of openings
 $T1 = (L1 \times Lo1)/(L1+L2) = 2.84 \text{ ft}$
 $T2 = (L2 \times Lo1)/(L1+L2) = 2.16 \text{ ft}$

6. Unit shear beside opening
 $v1 = (V/L)(L1+T1)/L1 = 275 \text{ plf}$
 $v2 = (V/L)(T2+L2)/L2 = 275 \text{ plf}$
Check $v1 \times L1 + v2 \times L2 = V?$ 1853 lbf OK

7. Resistance to corner forces
 $R1 = v1 \times L1 = 1051 \text{ lbf}$
 $R2 = v2 \times L2 = 802 \text{ lbf}$

8. Difference corner force + resistance
 $R1 - F1 = 157 \text{ lbf}$
 $R2 - F2 = 119 \text{ lbf}$

9. Unit shear in corner zones
 $vc1 = (R1 - F1)/L1 = 41 \text{ plf}$
 $vc2 = (R2 - F2)/L2 = 41 \text{ plf}$



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:

Designer: Chon Pieruccioni, PE

Date: 6/2/2025

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_i 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)

Nail Spacing: 4

HD Capacity: 2140

HD Deflection: 0.088

Pier 1

Pier 2

(in.)

(lbf)

(in.)

Four-Term Equation Deflection Check

$$\Delta = \frac{8vh^3}{EAb} + \frac{vh}{Gt} + 0.75he_n + d_a \frac{h}{b}$$
 (Equation 23-2)

Pier 1-L

Pier 1-R

Pier 2-L

Pier 2-R

$V_{unfactored}$: 275

275

275

275

E: 1.30E+06

1.30E+06

1.30E+06

1.30E+06

h: 9.00

5.60

5.60

9.00

Qty:

Stud Size:

A Override:

A:

G_i : 83,500

83,500

83,500

83,500

Nail Spacing: 4

4

4

4

V_n : 92

92

92

92

e_n : 0.0038

0.0038

0.0038

0.0038

b: 3.83

3.83

2.92

2.92

HD Capacity: 2140

2140

2140

2140

HD Defl: 0.088

0.088

0.088

0.088

(plf)

(psi)

(ft)

(in.²)

(in.²)

(lbf/in.)

(in.)

(plf)

(in.)

(ft)

(lbf)

(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	Term 2	Term 3	Term 4	Term 1	Term 2	Term 3	Term 4
Bending	Shear	Fastener	HD-1	Bending	Shear	Fastener	HD-2
	0.030	0.026	0.239		0.018	0.016	0.092
Sum				Sum			
Pier 2 (left)				Pier 2 (right)			
Term 1	Term 2	Term 3	Term 4	Term 1	Term 2	Term 3	Term 4
Bending	Shear	Fastener	HD-1	Bending	Shear	Fastener	HD-2
	0.018	0.016	0.121		0.030	0.026	0.313
Sum				Sum			

Total Defl.

(in.) %drift

Project Information

Code:

Designer: Chon Pieruccioni, PE

Date: 6/2/2025

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

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

HD Capacity: 2140

HD Deflection: 0.088

(in.)

(lbf)

(in.)

Three-Term Equation Deflection Check

$$\delta_{sw} = \frac{8vh^3}{EAb} + \frac{vh}{1000G_a} + \frac{h\Delta_a}{b} \quad (4.3-1)$$

	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	Term 2	Term 3	Term 1	Term 2	Term 3
Bending	Shear	Fastener	Bending	Shear	Fastener
	0.112	0.239		0.070	0.092
Sum			Sum		
Pier 2 (left)			Pier 2 (right)		
Term 1	Term 2	Term 3	Term 1	Term 2	Term 3
Bending	Shear	Fastener	Bending	Shear	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.



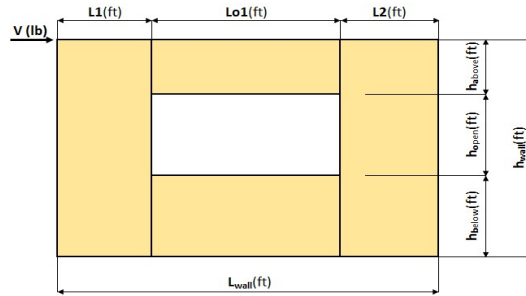
Force Transfer Around Openings Calculator

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
h _{wall}	9.00 ft	h _b	3.40 ft	P2=h _o /L2=	1.54
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: $O1 = va1 \times (Lo1) =$ 1416 lbf

4. Corner forces
 $F1 = O1(L1)/(L1+L2) =$ 804 lbf
 $F2 = O1(L2)/(L1+L2) =$ 613 lbf

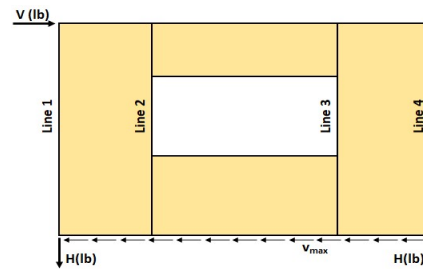
5. Tributary length of openings
 $T1 = (L1 \times Lo1)/(L1+L2) =$ 2.84 ft
 $T2 = (L2 \times Lo1)/(L1+L2) =$ 2.16 ft

6. Unit shear beside opening
 $v1 = (V/L)(L1+T1)/L1 =$ 247 plf
 $v2 = (V/L)(T2+L2)/L2 =$ 247 plf
Check $v1 \times L1 + v2 \times L2 = V?$ 1664 lbf OK

7. Resistance to corner forces
 $R1 = v1 \times L1 =$ 944 lbf
 $R2 = v2 \times L2 =$ 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:

Designer: Chon Pieruccioni, PE

Date: 6/2/2025

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_i 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)

Nail Spacing: 4

HD Capacity: 2140

HD Deflection: 0.088

Pier 1

Pier 2

(in.)

(lbf)

(in.)

Four-Term Equation Deflection Check

$$\Delta = \frac{8vh^3}{EAb} + \frac{vh}{Gt} + 0.75he_n + d_a \frac{h}{b}$$
 (Equation 23-2)

Pier 1-L

Pier 1-R

Pier 2-L

Pier 2-R

$V_{unfactored}$: 247247247247 (plf)

E : 1.30E+061.30E+061.30E+061.30E+06 (psi)

h : 9.005.605.609.00 (ft)

Qty :

$Stud Size$:

A Override:

A :

G_i : 83,50083,50083,50083,500 (lbf/in.)

$Nail Spacing$: 4444 (in.)

V_n : 82828282 (plf)

e_n : 0.00270.00270.00270.0027 (in.)

b : 3.833.832.922.92 (ft)

$HD Capacity$: 2140214021402140 (lbf)

$HD Defl$: 0.0880.0880.0880.088 (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	Term 2	Term 3	Term 4	Term 1	Term 2	Term 3	Term 4
Bending	Shear	Fastener	HD-1	Bending	Shear	Fastener	HD-2
	0.027	0.019	0.214		0.017	0.012	0.083
Sum				Sum			
Pier 2 (left)				Pier 2 (right)			
Term 1	Term 2	Term 3	Term 4	Term 1	Term 2	Term 3	Term 4
Bending	Shear	Fastener	HD-1	Bending	Shear	Fastener	HD-2
	0.017	0.012	0.109		0.027	0.019	0.281
Sum				Sum			

Total Defl.

(in.) %drift

Project Information

Code:

Designer: Chon Pieruccioni, PE

Date: 6/2/2025

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

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 (in.)

HD Capacity: 2140 (lbf)

HD Deflection: 0.088 (in.)

Three-Term Equation Deflection Check

$$\delta_{sw} = \frac{8vh^3}{EAb} + \frac{vh}{1000G_a} + \frac{h\Delta_a}{b} \quad (4.3-1)$$

	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.)

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	Term 2	Term 3	Term 1	Term 2	Term 3
Bending	Shear	Fastener	Bending	Shear	Fastener
	0.101	0.214		0.063	0.083
Sum			Sum		
Pier 2 (left)			Pier 2 (right)		
Term 1	Term 2	Term 3	Term 1	Term 2	Term 3
Bending	Shear	Fastener	Bending	Shear	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.



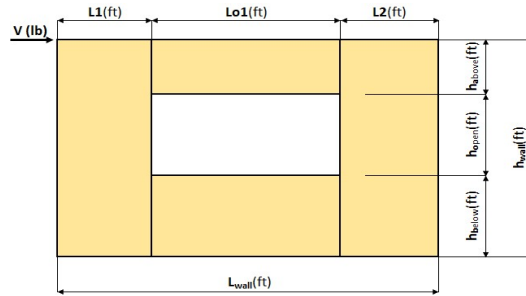
Force Transfer Around Openings Calculator

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
h _{wall}	9.00 ft	h _b	3.40 ft	P2=h _o /L2=	1.54
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: $va1 = vb1 = H/(h_a + h_b) = 350 \text{ plf}$

3. Total boundary force above + below openings
First opening: $O1 = va1 \times (Lo1) = 1748 \text{ lbf}$

4. Corner forces
 $F1 = O1(L1)/(L1+L2) = 992 \text{ lbf}$
 $F2 = O1(L2)/(L1+L2) = 756 \text{ lbf}$

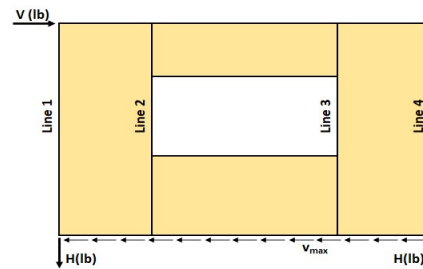
5. Tributary length of openings
 $T1 = (L1 \times Lo1)/(L1+L2) = 2.84 \text{ ft}$
 $T2 = (L2 \times Lo1)/(L1+L2) = 2.16 \text{ ft}$

6. Unit shear beside opening
 $v1 = (V/L)(L1+T1)/L1 = 304 \text{ plf}$
 $v2 = (V/L)(T2+L2)/L2 = 304 \text{ plf}$
Check $v1 \times L1 + v2 \times L2 = V?$ 2054 lbf OK

7. Resistance to corner forces
 $R1 = v1 \times L1 = 1165 \text{ lbf}$
 $R2 = v2 \times L2 = 889 \text{ lbf}$

8. Difference corner force + resistance
 $R1 - F1 = 174 \text{ lbf}$
 $R2 - F2 = 132 \text{ lbf}$

9. Unit shear in corner zones
 $vc1 = (R1 - F1)/L1 = 45 \text{ plf}$
 $vc2 = (R2 - F2)/L2 = 45 \text{ plf}$



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	0
Line 3: $va1(h_a + h_b) - vc2(h_a + h_b) - v1(h_o) = 0?$	1573	204	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:

Designer: Chon Pieruccioni, PE

Date: 6/2/2025

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_i 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)

Nail Spacing: 4

HD Capacity: 2140

HD Deflection: 0.088

Pier 1

Pier 2

(in.)

(lbf)

(in.)

Four-Term Equation Deflection Check

$$\Delta = \frac{8vh^3}{EAb} + \frac{vh}{Gt} + 0.75he_n + d_a \frac{h}{b}$$
 (Equation 23-2)

Pier 1-L

Pier 1-R

Pier 2-L

Pier 2-R

$V_{unfactored}$: 304304304304 (plf)

E : 1.30E+061.30E+061.30E+061.30E+06 (psi)

h : 9.005.605.609.00 (ft)

Qty:

Stud Size:

A Override:

A :

G_i : 83,50083,50083,50083,500 (lbf/in.)

Nail Spacing: 4444 (in.)

V_n : 101101101101 (plf)

e_n : 0.00520.00520.00520.0052 (in.)

b : 3.833.832.922.92 (ft)

HD Capacity: 2140214021402140 (lbf)

HD Defl: 0.0880.0880.0880.088 (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	Term 2	Term 3	Term 4	Term 1	Term 2	Term 3	Term 4
Bending	Shear	Fastener	HD-1	Bending	Shear	Fastener	HD-2
	0.033	0.035	0.265		0.020	0.022	0.102
Sum				Sum			
Pier 2 (left)				Pier 2 (right)			
Term 1	Term 2	Term 3	Term 4	Term 1	Term 2	Term 3	Term 4
Bending	Shear	Fastener	HD-1	Bending	Shear	Fastener	HD-2
	0.020	0.022	0.134		0.033	0.035	0.347
Sum				Sum			

Total Defl.

(in.) %drift

Project Information

Code:

Designer: Chon Pieruccioni, PE

Date: 6/2/2025

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

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

HD Capacity: 2140

HD Deflection: 0.088

(in.)

(lbf)

(in.)

Three-Term Equation Deflection Check

$$\delta_{sw} = \frac{8vh^3}{EAb} + \frac{vh}{1000G_a} + \frac{h\Delta_a}{b} \quad (4.3-1)$$

	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.)

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	Term 2	Term 3	Term 1	Term 2	Term 3
Bending	Shear	Fastener	Bending	Shear	Fastener
	0.124	0.265		0.077	0.102
Sum			Sum		
Pier 2 (left)			Pier 2 (right)		
Term 1	Term 2	Term 3	Term 1	Term 2	Term 3
Bending	Shear	Fastener	Bending	Shear	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.



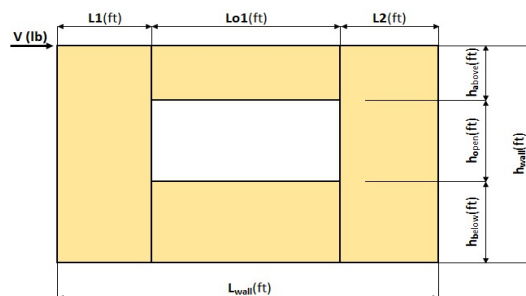
Force Transfer Around Openings Calculator

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		Adj. Factor Method = 1.25-0.125h/bs	
L1	3.63 ft	h _a	1.50 ft	Wall Pier Aspect Ratio	
L2	4.20 ft	h _o	3.00 ft	P1=h _o /L1=	0.83
h _{wall}	9.00 ft	h _b	4.50 ft	P2=h _o /L2=	0.71
L _{wall}	10.33 ft	Lo1	2.50 ft		

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

2. Unit shear above + below opening
First opening: $va1 = vb1 = H/(h_a + h_b) = 212 \text{ plf}$

3. Total boundary force above + below openings
First opening: $O1 = va1 \times (Lo1) = 531 \text{ lbf}$

4. Corner forces
 $F1 = O1(L1)/(L1+L2) = 246 \text{ lbf}$
 $F2 = O1(L2)/(L1+L2) = 285 \text{ lbf}$

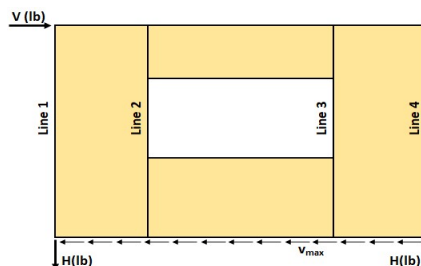
5. Tributary length of openings
 $T1 = (L1 \times Lo1)/(L1+L2) = 1.16 \text{ ft}$
 $T2 = (L2 \times Lo1)/(L1+L2) = 1.34 \text{ ft}$

6. Unit shear beside opening
 $v1 = (V/L)(L1+T1)/L1 = 187 \text{ plf}$
 $v2 = (V/L)(T2+L2)/L2 = 187 \text{ plf}$
Check $v1 \times L1 + v2 \times L2 = V?$ 1462 lbf OK

7. Resistance to corner forces
 $R1 = v1 \times L1 = 678 \text{ lbf}$
 $R2 = v2 \times L2 = 784 \text{ lbf}$

8. Difference corner force + resistance
 $R1 - F1 = 432 \text{ lbf}$
 $R2 - F2 = 500 \text{ lbf}$

9. Unit shear in corner zones
 $vc1 = (R1 - F1)/L1 = 119 \text{ plf}$
 $vc2 = (R2 - F2)/L2 = 119 \text{ plf}$



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:

Designer: Chon Pieruccioni, PE

Date: 6/2/2025

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_i 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

$$\Delta = \frac{8vh^3}{EAb} + \frac{vh}{Gt} + 0.75he_n + d_a \frac{h}{b}$$
 (Equation 23-2)

	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.)

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	Term 2	Term 3	Term 4	Term 1	Term 2	Term 3	Term 4
Bending	Shear	Fastener	HD-1	Bending	Shear	Fastener	HD-2
	0.020	0.027	0.171		0.010	0.014	0.043
Sum				Sum			
Pier 2 (left)				Pier 2 (right)			
Term 1	Term 2	Term 3	Term 4	Term 1	Term 2	Term 3	Term 4
Bending	Shear	Fastener	HD-1	Bending	Shear	Fastener	HD-2
	0.010	0.014	0.037		0.020	0.027	0.148
Sum				Sum			

Total Defl.

(in.) %drift

Project Information

Code:

Designer: Chon Pieruccioni, PE

Date: 6/2/2025

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

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

HD Capacity: 2140

HD Deflection: 0.088

(in.)

(lbf)

(in.)

Three-Term Equation Deflection Check

$$\delta_{sw} = \frac{8vh^3}{Eab} + \frac{vh}{1000G_a} + \frac{h\Delta_a}{b} \quad (4.3-1)$$

	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	Term 2	Term 3	Term 1	Term 2	Term 3
Bending	Shear	Fastener	Bending	Shear	Fastener
	0.112	0.171		0.056	0.043
Sum			Sum		
Pier 2 (left)			Pier 2 (right)		
Term 1	Term 2	Term 3	Term 1	Term 2	Term 3
Bending	Shear	Fastener	Bending	Shear	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.



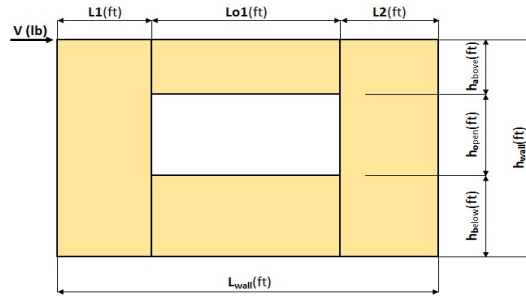
Force Transfer Around Openings Calculator

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		Adj. Factor Method = 1.25-0.125h/bs	
L1	3.63 ft	h _a	1.50 ft	Wall Pier Aspect Ratio	Adj. Factor
L2	4.20 ft	h _o	3.00 ft	P1=h _o /L1=	0.83
h _{wall}	9.00 ft	h _b	4.50 ft	P2=h _o /L2=	0.71
L _{wall}	10.33 ft	Lo1	2.50 ft		

1. Hold-down forces: $H = Vh_{wall}/L_{wall}$ 1573 lbf

2. Unit shear above + below opening
First opening: $va1 = vb1 = H/(h_a + h_b) = 262 \text{ plf}$

3. Total boundary force above + below openings
First opening: $O1 = va1 \times (Lo1) = 656 \text{ lbf}$

4. Corner forces
 $F1 = O1(L1)/(L1+L2) = 304 \text{ lbf}$
 $F2 = O1(L2)/(L1+L2) = 352 \text{ lbf}$

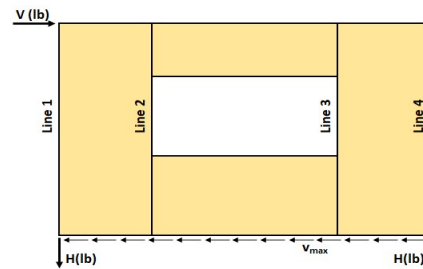
5. Tributary length of openings
 $T1 = (L1 \times Lo1)/(L1+L2) = 1.16 \text{ ft}$
 $T2 = (L2 \times Lo1)/(L1+L2) = 1.34 \text{ ft}$

6. Unit shear beside opening
 $v1 = (V/L)(L1+T1)/L1 = 231 \text{ plf}$
 $v2 = (V/L)(T2+L2)/L2 = 231 \text{ plf}$
Check $v1 \times L1 + v2 \times L2 = V?$ 1806 lbf OK

7. Resistance to corner forces
 $R1 = v1 \times L1 = 837 \text{ lbf}$
 $R2 = v2 \times L2 = 969 \text{ lbf}$

8. Difference corner force + resistance
 $R1 - F1 = 533 \text{ lbf}$
 $R2 - F2 = 617 \text{ lbf}$

9. Unit shear in corner zones
 $vc1 = (R1 - F1)/L1 = 147 \text{ plf}$
 $vc2 = (R2 - F2)/L2 = 147 \text{ plf}$



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:

Designer: Chon Pieruccioni, PE

Date: 6/2/2025

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_i 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)

Nail Spacing: 4

HD Capacity: 2140

HD Deflection: 0.088

Pier 1

Pier 2

(in.)

(lbf)

(in.)

Four-Term Equation Deflection Check

$$\Delta = \frac{8vh^3}{EAb} + \frac{vh}{Gt} + 0.75he_n + d_a \frac{h}{b}$$

(Equation 23-2)

	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.)

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	Term 2	Term 3	Term 4	Term 1	Term 2	Term 3	Term 4
Bending	Shear	Fastener	HD-1	Bending	Shear	Fastener	HD-2
	0.025	0.015	0.212		0.012	0.008	0.053
Sum				Sum			
Pier 2 (left)				Pier 2 (right)			
Term 1	Term 2	Term 3	Term 4	Term 1	Term 2	Term 3	Term 4
Bending	Shear	Fastener	HD-1	Bending	Shear	Fastener	HD-2
	0.012	0.008	0.046		0.025	0.015	0.183
Sum				Sum			

Total Defl.

(in.) %drift

Project Information

Code:

Designer: Chon Pieruccioni, PE

Date: 6/2/2025

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

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

HD Capacity: 2140

HD Deflection: 0.088

(in.)

(lbf)

(in.)

Three-Term Equation Deflection Check

$$\delta_{sw} = \frac{8vh^3}{EAb} + \frac{vh}{1000G_a} + \frac{h\Delta_a}{b}$$

(4.3-1)

	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.)

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	Term 2	Term 3	Term 1	Term 2	Term 3
Bending	Shear	Fastener	Bending	Shear	Fastener
	0.094	0.212		0.047	0.053
Sum			Sum		
Pier 2 (left)			Pier 2 (right)		
Term 1	Term 2	Term 3	Term 1	Term 2	Term 3
Bending	Shear	Fastener	Bending	Shear	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.



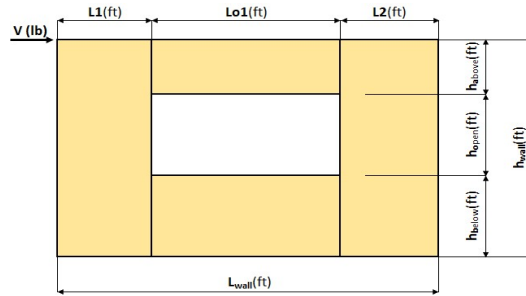
Force Transfer Around Openings Calculator

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: $va1 = vb1 = H/(h_a + h_b) = 441 \text{ plf}$

3. Total boundary force above + below openings
First opening: $O1 = va1 \times (Lo1) = 2646 \text{ lbf}$

4. Corner forces
 $F1 = O1(L1)/(L1+L2) = 1325 \text{ lbf}$
 $F2 = O1(L2)/(L1+L2) = 1321 \text{ lbf}$

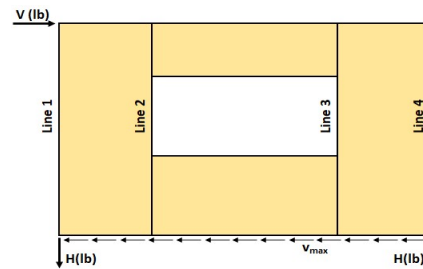
5. Tributary length of openings
 $T1 = (L1 \times Lo1)/(L1+L2) = 3.01 \text{ ft}$
 $T2 = (L2 \times Lo1)/(L1+L2) = 2.99 \text{ ft}$

6. Unit shear beside opening
 $v1 = (V/L)(L1+T1)/L1 = 300 \text{ plf}$
 $v2 = (V/L)(T2+L2)/L2 = 300 \text{ plf}$
Check $v1 \times L1 + v2 \times L2 = V?$ 1727 lbf OK

7. Resistance to corner forces
 $R1 = v1 \times L1 = 865 \text{ lbf}$
 $R2 = v2 \times L2 = 862 \text{ lbf}$

8. Difference corner force + resistance
 $R1 - F1 = -460 \text{ lbf}$
 $R2 - F2 = -459 \text{ lbf}$

9. Unit shear in corner zones
 $vc1 = (R1 - F1)/L1 = -160 \text{ plf}$
 $vc2 = (R2 - F2)/L2 = -160 \text{ plf}$



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:

Designer: Chon Pieruccioni, PE

Date: 6/2/2025

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_i 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)

Nail Spacing: 2

HD Capacity: 2140

HD Deflection: 0.088

Pier 1

Pier 2

(in.)

(lbf)

(in.)

Four-Term Equation Deflection Check

$$\Delta = \frac{8vh^3}{EAb} + \frac{vh}{Gt} + 0.75he_n + d_a \frac{h}{b}$$
 (Equation 23-2)

Pier 1-L

Pier 1-R

Pier 2-L

Pier 2-R

$V_{unfactored}$: 300

E: 1.30E+06

h: 9.00

Qty:

Stud Size:

A Override:

A:

G_i : 83,500

Nail Spacing: 2

V_n : 50

e_n : 0.0006

b: 2.88

HD Capacity: 2140

HD Defl: 0.088

300

1.30E+06

7.25

83,500

2

50

0.0006

2.88

2140

0.088

300

1.30E+06

7.25

83,500

2

50

0.0006

2.88

2140

0.088

300

1.30E+06

7.25

83,500

2

50

0.0006

2.87

2140

0.088

300

1.30E+06

9.00

83,500

2

50

0.0006

2.87

2140

0.088

(plf)

(psi)

(ft)

(in.²)

(in.²)

(lbf/in.)

(in.)

(plf)

(in.)

(ft)

(lbf)

(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	Term 2	Term 3	Term 4	Term 1	Term 2	Term 3	Term 4
Bending	Shear	Fastener	HD-1	Bending	Shear	Fastener	HD-2
	0.032	0.004	0.347		0.026	0.003	0.225
Sum				Sum			
Pier 2 (left)				Pier 2 (right)			
Term 1	Term 2	Term 3	Term 4	Term 1	Term 2	Term 3	Term 4
Bending	Shear	Fastener	HD-1	Bending	Shear	Fastener	HD-2
	0.026	0.003	0.226		0.032	0.004	0.349
Sum				Sum			

Total Defl.

(in.) %drift

Project Information

Code:

Designer: Chon Pieruccioni, PE

Date: 6/2/2025

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

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

HD Capacity: 2140

HD Deflection: 0.088

(in.)

(lbf)

(in.)

Three-Term Equation Deflection Check

$$\delta_{sw} = \frac{8vh^3}{EAb} + \frac{vh}{1000G_a} + \frac{h\Delta_a}{b} \quad (4.3-1)$$

	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	Term 2	Term 3	Term 1	Term 2	Term 3
Bending	Shear	Fastener	Bending	Shear	Fastener
	0.064	0.347		0.052	0.225
Sum			Sum		
Pier 2 (left)			Pier 2 (right)		
Term 1	Term 2	Term 3	Term 1	Term 2	Term 3
Bending	Shear	Fastener	Bending	Shear	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.



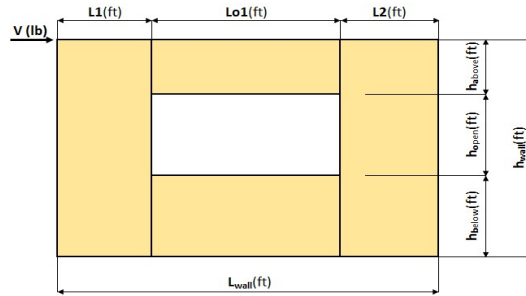
Force Transfer Around Openings Calculator

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	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 _a /L1=	2.08	0.990
h _{wall}	9.00 ft	h _b	1.75 ft	P2=h _b /L2=	2.09	0.989
L _{wall}	11.75 ft	Lo1	6.00 ft			

1. Hold-down forces: $H = Vh_{\text{wall}}/L_{\text{wall}}$ 1458 lbf

2. Unit shear above + below opening
First opening: $va1 = vb1 = H/(h_a + h_b) = 486 \text{ plf}$

3. Total boundary force above + below openings
First opening: $O1 = va1 \times (Lo1) = 2915 \text{ lbf}$

4. Corner forces
 $F1 = O1(L1)/(L1+L2) = 1460 \text{ lbf}$
 $F2 = O1(L2)/(L1+L2) = 1455 \text{ lbf}$

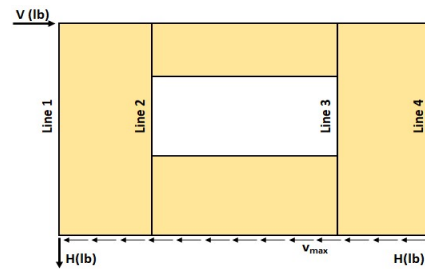
5. Tributary length of openings
 $T1 = (L1 \times Lo1)/(L1+L2) = 3.01 \text{ ft}$
 $T2 = (L2 \times Lo1)/(L1+L2) = 2.99 \text{ ft}$

6. Unit shear beside opening
 $v1 = (V/L)(L1+T1)/L1 = 331 \text{ plf}$
 $v2 = (V/L)(T2+L2)/L2 = 331 \text{ plf}$
Check $v1 \times L1 + v2 \times L2 = V?$ 1903 lbf OK

7. Resistance to corner forces
 $R1 = v1 \times L1 = 953 \text{ lbf}$
 $R2 = v2 \times L2 = 950 \text{ lbf}$

8. Difference corner force + resistance
 $R1 - F1 = -507 \text{ lbf}$
 $R2 - F2 = -505 \text{ lbf}$

9. Unit shear in corner zones
 $vc1 = (R1 - F1)/L1 = -176 \text{ plf}$
 $vc2 = (R2 - F2)/L2 = -176 \text{ plf}$



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	0
Line 3: $va1(h_a + h_b) - vc2(h_a + h_b) - v1(h_o) = 0?$	1458	-528	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:

Designer: Chon Pieruccioni, PE

Date: 6/2/2025

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_i 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)

Nail Spacing: 2

HD Capacity: 2140

HD Deflection: 0.088

Pier 1

Pier 2

2

2140

0.088

(in.)

(lbf)

(in.)

Four-Term Equation Deflection Check

$$\Delta = \frac{8vh^3}{EAb} + \frac{vh}{Gt} + 0.75he_n + d_a \frac{h}{b}$$

(Equation 23-2)

	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.)

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	Term 2	Term 3	Term 4	Term 1	Term 2	Term 3	Term 4
Bending	Shear	Fastener	HD-1	Bending	Shear	Fastener	HD-2
	0.036	0.006	0.383		0.029	0.004	0.248
Sum				Sum			
Pier 2 (left)				Pier 2 (right)			
Term 1	Term 2	Term 3	Term 4	Term 1	Term 2	Term 3	Term 4
Bending	Shear	Fastener	HD-1	Bending	Shear	Fastener	HD-2
	0.029	0.004	0.249		0.036	0.006	0.384
Sum				Sum			

Total

Defl.

(in.)

%drift

Project Information

Code:

Designer: Chon Pieruccioni, PE

Date: 6/2/2025

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

HD Capacity: 2140

HD Deflection: 0.088

(in.)

(lbf)

(in.)

Three-Term Equation Deflection Check

$$\delta_{sw} = \frac{8vh^3}{EAb} + \frac{vh}{1000G_a} + \frac{h\Delta_a}{b} \quad (4.3-1)$$

	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.)

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	Term 2	Term 3	Term 1	Term 2	Term 3
Bending	Shear	Fastener	Bending	Shear	Fastener
	0.071	0.383		0.057	0.248
Sum			Sum		
Pier 2 (left)			Pier 2 (right)		
Term 1	Term 2	Term 3	Term 1	Term 2	Term 3
Bending	Shear	Fastener	Bending	Shear	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.



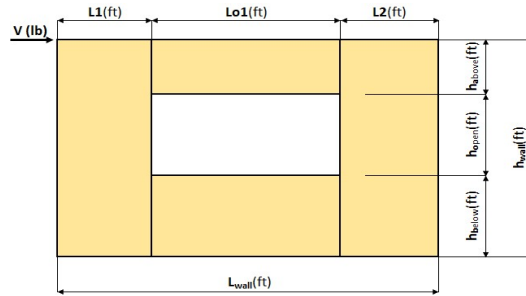
Force Transfer Around Openings Calculator

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 _a /L1=	2.08
h _{wall}	9.00 ft	h _b	1.75 ft	P2=h _o /L2=	2.09
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: $va1 = vb1 = H/(h_a + h_b) = 608 \text{ plf}$

3. Total boundary force above + below openings
First opening: $O1 = va1 \times (Lo1) = 3651 \text{ lbf}$

4. Corner forces
 $F1 = O1(L1)/(L1+L2) = 1828 \text{ lbf}$
 $F2 = O1(L2)/(L1+L2) = 1822 \text{ lbf}$

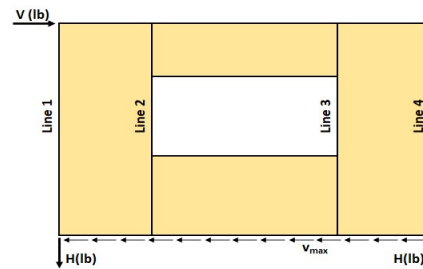
5. Tributary length of openings
 $T1 = (L1 \times Lo1)/(L1+L2) = 3.01 \text{ ft}$
 $T2 = (L2 \times Lo1)/(L1+L2) = 2.99 \text{ ft}$

6. Unit shear beside opening
 $v1 = (V/L)(L1+T1)/L1 = 414 \text{ plf}$
 $v2 = (V/L)(T2+L2)/L2 = 414 \text{ plf}$
Check $v1 \times L1 + v2 \times L2 = V?$ 2383 lbf OK

7. Resistance to corner forces
 $R1 = v1 \times L1 = 1194 \text{ lbf}$
 $R2 = v2 \times L2 = 1189 \text{ lbf}$

8. Difference corner force + resistance
 $R1 - F1 = -635 \text{ lbf}$
 $R2 - F2 = -633 \text{ lbf}$

9. Unit shear in corner zones
 $vc1 = (R1 - F1)/L1 = -220 \text{ plf}$
 $vc2 = (R2 - F2)/L2 = -220 \text{ plf}$



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:

Designer: Chon Pieruccioni, PE

Date: 6/2/2025

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_i 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)

Nail Spacing: 2

HD Capacity: 2140

HD Deflection: 0.088

Pier 1

Pier 2

2

2140

0.088

(in.)

(lbf)

(in.)

Four-Term Equation Deflection Check

$$\Delta = \frac{8vh^3}{EAb} + \frac{vh}{Gt} + 0.75he_n + d_a \frac{h}{b}$$

(Equation 23-2)

	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.)

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	Term 2	Term 3	Term 4	Term 1	Term 2	Term 3	Term 4
Bending	Shear	Fastener	HD-1	Bending	Shear	Fastener	HD-2
	0.045	0.011	0.479		0.036	0.009	0.311
Sum				Sum			
Pier 2 (left)				Pier 2 (right)			
Term 1	Term 2	Term 3	Term 4	Term 1	Term 2	Term 3	Term 4
Bending	Shear	Fastener	HD-1	Bending	Shear	Fastener	HD-2
	0.036	0.009	0.312		0.045	0.011	0.481
Sum				Sum			

Total Defl.

(in.) %drift

Project Information

Code:

Designer: Chon Pieruccioni, PE

Date: 6/2/2025

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)

Pier 1

Pier 2

Nail Spacing: 2 (in.)

HD Capacity: 2140 (lbf)

HD Deflection: 0.088 (in.)

G_t Override:

G_a Override:

C_d: 4.00

Three-Term Equation Deflection Check

$$\delta_{sw} = \frac{8vh^3}{Eab} + \frac{vh}{1000G_a} + \frac{h\Delta_a}{b} \quad (4.3-1)$$

	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	Term 2	Term 3	Term 1	Term 2	Term 3
Bending	Shear	Fastener	Bending	Shear	Fastener
	0.096	0.479		0.077	0.311
Sum			Sum		
Pier 2 (left)			Pier 2 (right)		
Term 1	Term 2	Term 3	Term 1	Term 2	Term 3
Bending	Shear	Fastener	Bending	Shear	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.



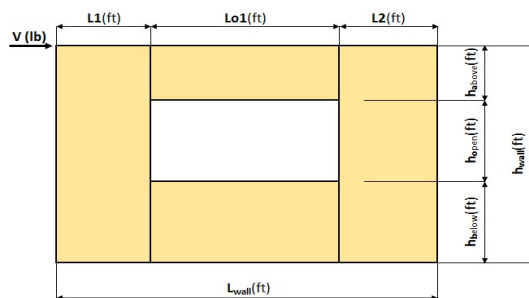
Force Transfer Around Openings Calculator

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		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
h _{wall}	9.00 ft	h _b	3.40 ft	P2=h _o /L2=	1.54
L _{wall}	11.25 ft	Lo1	5.00 ft		

1. Hold-down forces: $H = Vh_{\text{wall}}/L_{\text{wall}}$ 1322 lbf

2. Unit shear above + below opening
First opening: $va1 = vb1 = H/(h_a + h_b) = 294 \text{ plf}$

3. Total boundary force above + below openings
First opening: $O1 = va1 \times (Lo1) = 1469 \text{ lbf}$

4. Corner forces
 $F1 = O1(L1)/(L1+L2) = 783 \text{ lbf}$
 $F2 = O1(L2)/(L1+L2) = 686 \text{ lbf}$

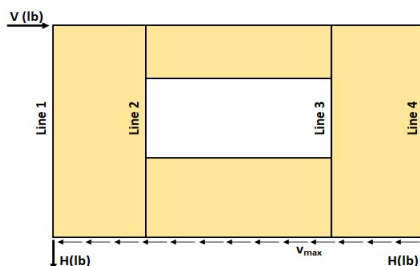
5. Tributary length of openings
 $T1 = (L1 \times Lo1)/(L1+L2) = 2.66 \text{ ft}$
 $T2 = (L2 \times Lo1)/(L1+L2) = 2.34 \text{ ft}$

6. Unit shear beside opening
 $v1 = (V/L)(L1+T1)/L1 = 264 \text{ plf}$
 $v2 = (V/L)(T2+L2)/L2 = 264 \text{ plf}$
Check $v1 \times L1 + v2 \times L2 = V?$ 1653 lbf OK

7. Resistance to corner forces
 $R1 = v1 \times L1 = 881 \text{ lbf}$
 $R2 = v2 \times L2 = 772 \text{ lbf}$

8. Difference corner force + resistance
 $R1 - F1 = 98 \text{ lbf}$
 $R2 - F2 = 86 \text{ lbf}$

9. Unit shear in corner zones
 $vc1 = (R1 - F1)/L1 = 29 \text{ plf}$
 $vc2 = (R2 - F2)/L2 = 29 \text{ plf}$



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:

Designer: Chon Pieruccioni, PE

Date: 6/2/2025

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_i 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)

Nail Spacing: 4

HD Capacity: 2140

HD Deflection: 0.088

Pier 1

Pier 2

(in.)

(lbf)

(in.)

Four-Term Equation Deflection Check

$$\Delta = \frac{8vh^3}{EAb} + \frac{vh}{Gt} + 0.75he_n + d_a \frac{h}{b}$$
 (Equation 23-2)

Pier 1-L

Pier 1-R

Pier 2-L

Pier 2-R

$V_{unfactored}$: 264

E: 1.30E+06

h: 9.00

Qty:

Stud Size:

A Override:

A:

G_i : 83,500

Nail Spacing: 4

V_n : 88

e_n : 0.0034

b: 3.33

HD Capacity: 2140

HD Defl: 0.088

264

1.30E+06

5.60

83,500

4

88

0.0034

3.33

2140

0.088

264

1.30E+06

5.60

83,500

4

88

0.0034

2.92

2140

0.088

264

1.30E+06

9.00

83,500

4

88

0.0034

2.92

2140

0.088

264

1.30E+06

9.00

83,500

4

88

0.0034

2.92

2140

0.088

(plf)

(psi)

(ft)

(in.²)

(in.²)

(lbf/in.)

(in.)

(plf)

(in.)

(ft)

(lbf)

(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	Term 2	Term 3	Term 4	Term 1	Term 2	Term 3	Term 4
Bending	Shear	Fastener	HD-1	Bending	Shear	Fastener	HD-2
	0.029	0.023	0.265		0.018	0.014	0.102
Sum				Sum			
Pier 2 (left)				Pier 2 (right)			
Term 1	Term 2	Term 3	Term 4	Term 1	Term 2	Term 3	Term 4
Bending	Shear	Fastener	HD-1	Bending	Shear	Fastener	HD-2
	0.018	0.014	0.117		0.029	0.023	0.302
Sum				Sum			

Total Defl.

(in.) %drift

Project Information

Code:

Designer: Chon Pieruccioni, PE

Date: 6/2/2025

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

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 (in.)

HD Capacity: 2140 (lbf)

HD Deflection: 0.088 (in.)

Three-Term Equation Deflection Check

$$\delta_{sw} = \frac{8vh^3}{Eab} + \frac{vh}{1000G_a} + \frac{h\Delta_a}{b} \quad (4.3-1)$$

	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	Term 2	Term 3	Term 1	Term 2	Term 3
Bending	Shear	Fastener	Bending	Shear	Fastener
	0.108	0.265		0.067	0.102
Sum			Sum		
Pier 2 (left)			Pier 2 (right)		
Term 1	Term 2	Term 3	Term 1	Term 2	Term 3
Bending	Shear	Fastener	Bending	Shear	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.



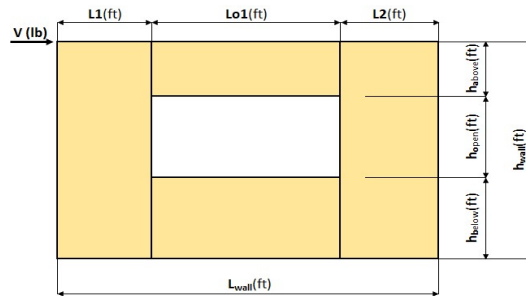
Force Transfer Around Openings Calculator

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 _a /L1=	1.35
h _{wall}	9.00 ft	h _b	3.40 ft	P2=h _o /L2=	1.54
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: $va1 = vb1 = H/(h_a + h_b) = 317 \text{ plf}$

3. Total boundary force above + below openings
First opening: $O1 = va1 \times (Lo1) = 1586 \text{ lbf}$

4. Corner forces
 $F1 = O1(L1)/(L1+L2) = 845 \text{ lbf}$
 $F2 = O1(L2)/(L1+L2) = 741 \text{ lbf}$

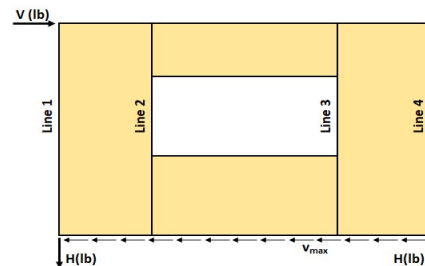
5. Tributary length of openings
 $T1 = (L1 \times Lo1)/(L1+L2) = 2.66 \text{ ft}$
 $T2 = (L2 \times Lo1)/(L1+L2) = 2.34 \text{ ft}$

6. Unit shear beside opening
 $v1 = (V/L)(L1+T1)/L1 = 285 \text{ plf}$
 $v2 = (V/L)(T2+L2)/L2 = 285 \text{ plf}$
Check $v1 \times L1 + v2 \times L2 = V?$ 1784 lbf OK

7. Resistance to corner forces
 $R1 = v1 \times L1 = 951 \text{ lbf}$
 $R2 = v2 \times L2 = 833 \text{ lbf}$

8. Difference corner force + resistance
 $R1 - F1 = 106 \text{ lbf}$
 $R2 - F2 = 93 \text{ lbf}$

9. Unit shear in corner zones
 $vc1 = (R1 - F1)/L1 = 32 \text{ plf}$
 $vc2 = (R2 - F2)/L2 = 32 \text{ plf}$



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	0
Line 3: $va1(h_a + h_b) - vc2(h_a + h_b) - v1(h_o) = 0?$	1427	143	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:

Designer: Chon Pieruccioni, PE

Date: 6/2/2025

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_i 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)

Nail Spacing: 4

HD Capacity: 2140

HD Deflection: 0.088

Pier 1

Pier 2

(in.)

(lbf)

(in.)

Four-Term Equation Deflection Check

$$\Delta = \frac{8vh^3}{EAb} + \frac{vh}{Gt} + 0.75he_n + d_a \frac{h}{b}$$
 (Equation 23-2)

Pier 1-L

Pier 1-R

Pier 2-L

Pier 2-R

$V_{unfactored}$: 285

E: 1.30E+06

h: 9.00

Qty:

Stud Size:

A Override:

A:

G_i : 83,500

Nail Spacing: 4

V_n : 95

e_n : 0.0043

b: 3.33

HD Capacity: 2140

HD Defl: 0.088

285

1.30E+06

5.60

83,500

4

95

0.0043

3.33

2140

0.088

285

1.30E+06

5.60

83,500

4

95

0.0043

2.92

2140

0.088

285

1.30E+06

5.60

83,500

4

95

0.0043

2.92

2140

0.088

285

1.30E+06

9.00

83,500

4

95

0.0043

2.92

2140

0.088

(plf)

(psi)

(ft)

(in.²)

(in.²)

(lbf/in.)

(in.)

(plf)

(in.)

(ft)

(lbf)

(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	Term 2	Term 3	Term 4	Term 1	Term 2	Term 3	Term 4
Bending	Shear	Fastener	HD-1	Bending	Shear	Fastener	HD-2
	0.031	0.029	0.286		0.019	0.018	0.111
Sum				Sum			
Pier 2 (left)				Pier 2 (right)			
Term 1	Term 2	Term 3	Term 4	Term 1	Term 2	Term 3	Term 4
Bending	Shear	Fastener	HD-1	Bending	Shear	Fastener	HD-2
	0.019	0.018	0.126		0.031	0.029	0.326
Sum				Sum			

Total Defl.

(in.) %drift

Project Information

Code:

Designer: Chon Pieruccioni, PE

Date: 6/2/2025

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

HD Capacity: 2140

HD Deflection: 0.088

(in.)

(lbf)

(in.)

Three-Term Equation Deflection Check

$$\delta_{sw} = \frac{8vh^3}{Eab} + \frac{vh}{1000G_a} + \frac{h\Delta_a}{b} \quad (4.3-1)$$

	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.)

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	Term 2	Term 3	Term 1	Term 2	Term 3
Bending	Shear	Fastener	Bending	Shear	Fastener
	0.117	0.286		0.073	0.111
Sum			Sum		
Pier 2 (left)			Pier 2 (right)		
Term 1	Term 2	Term 3	Term 1	Term 2	Term 3
Bending	Shear	Fastener	Bending	Shear	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.



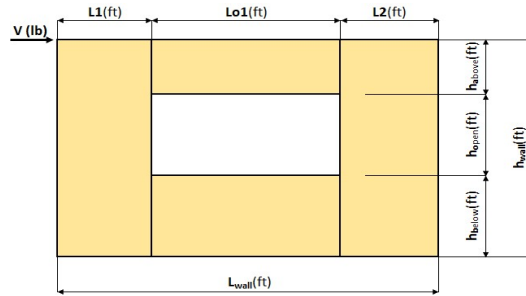
Force Transfer Around Openings Calculator

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 _a /L1=	1.35
h _{wall}	9.00 ft	h _b	3.40 ft	P2=h _o /L2=	1.54
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: $O1 = va1 \times (Lo1) =$ 2028 lbf

4. Corner forces
 $F1 = O1(L1)/(L1+L2) =$ 1081 lbf
 $F2 = O1(L2)/(L1+L2) =$ 948 lbf

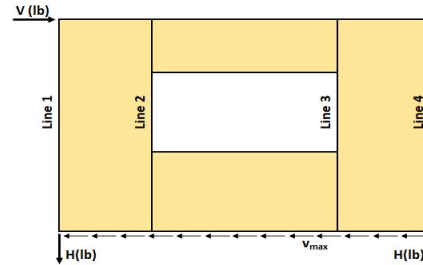
5. Tributary length of openings
 $T1 = (L1 \times Lo1)/(L1+L2) =$ 2.66 ft
 $T2 = (L2 \times Lo1)/(L1+L2) =$ 2.34 ft

6. Unit shear beside opening
 $v1 = (V/L)(L1+T1)/L1 =$ 365 plf
 $v2 = (V/L)(T2+L2)/L2 =$ 365 plf
Check $v1 \times L1 + v2 \times L2 = V?$ 2282 lbf OK

7. Resistance to corner forces
 $R1 = v1 \times L1 =$ 1216 lbf
 $R2 = v2 \times L2 =$ 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:

Designer: Chon Pieruccioni, PE

Date: 6/2/2025

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_i 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

$$\Delta = \frac{8vh^3}{EAb} + \frac{vh}{Gt} + 0.75he_n + d_a \frac{h}{b}$$
 (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_i :

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	Term 2	Term 3	Term 4	Term 1	Term 2	Term 3	Term 4
Bending	Shear	Fastener	HD-1	Bending	Shear	Fastener	HD-2
	0.039	0.061	0.365		0.024	0.038	0.141
Sum				Sum			
Pier 2 (left)				Pier 2 (right)			
Term 1	Term 2	Term 3	Term 4	Term 1	Term 2	Term 3	Term 4
Bending	Shear	Fastener	HD-1	Bending	Shear	Fastener	HD-2
	0.024	0.038	0.161		0.039	0.061	0.416
Sum				Sum			

Total

Defl.

(in.)

%drift

Project Information

Code:

Designer: Chon Pieruccioni, PE

Date: 6/2/2025

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

HD Capacity: 2140

HD Deflection: 0.088

(in.)

(lbf)

(in.)

Three-Term Equation Deflection Check

$$\delta_{sw} = \frac{8vh^3}{Eab} + \frac{vh}{1000G_a} + \frac{h\Delta_a}{b} \quad (4.3-1)$$

	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	Term 2	Term 3	Term 1	Term 2	Term 3
Bending	Shear	Fastener	Bending	Shear	Fastener
	0.149	0.365		0.093	0.141
Sum			Sum		
Pier 2 (left)			Pier 2 (right)		
Term 1	Term 2	Term 3	Term 1	Term 2	Term 3
Bending	Shear	Fastener	Bending	Shear	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.



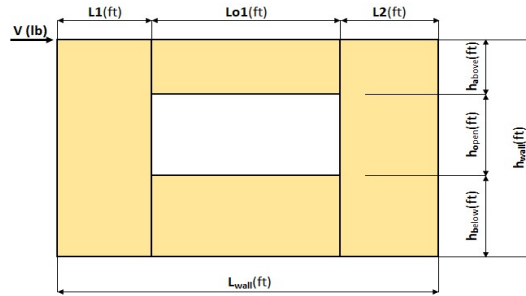
Force Transfer Around Openings Calculator

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
h _{wall}	9.00 ft	h _b	4.50 ft	P2=h _o /L2=	0.64
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: $O1 = va1 \times (Lo1) =$ 595 lbf

4. Corner forces
 $F1 = O1(L1)/(L1+L2) =$ 259 lbf
 $F2 = O1(L2)/(L1+L2) =$ 335 lbf

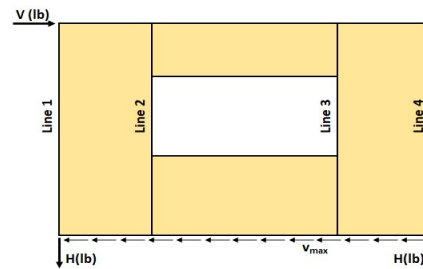
5. Tributary length of openings
 $T1 = (L1 \times Lo1)/(L1+L2) =$ 1.09 ft
 $T2 = (L2 \times Lo1)/(L1+L2) =$ 1.41 ft

6. Unit shear beside opening
 $v1 = (V/L)(L1+T1)/L1 =$ 206 plf
 $v2 = (V/L)(T2+L2)/L2 =$ 206 plf
Check $v1 \times L1 + v2 \times L2 = V?$ 1717 lbf OK

7. Resistance to corner forces
 $R1 = v1 \times L1 =$ 748 lbf
 $R2 = v2 \times L2 =$ 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:

Designer: Chon Pieruccioni, PE

Date: 6/2/2025

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_i 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

$$\Delta = \frac{8vh^3}{EAb} + \frac{vh}{Gt} + 0.75he_n + d_a \frac{h}{b}$$
 (Equation 23-2)

	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.)

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	Term 2	Term 3	Term 4	Term 1	Term 2	Term 3	Term 4
Bending	Shear	Fastener	HD-1	Bending	Shear	Fastener	HD-2
	0.022	0.037	0.189		0.011	0.018	0.047
Sum				Sum			
Pier 2 (left)				Pier 2 (right)			
Term 1	Term 2	Term 3	Term 4	Term 1	Term 2	Term 3	Term 4
Bending	Shear	Fastener	HD-1	Bending	Shear	Fastener	HD-2
	0.011	0.018	0.037		0.022	0.037	0.146
Sum				Sum			

Total Defl.

(in.) %drift

Project Information

Code:

Designer: Chon Pieruccioni, PE

Date: 6/2/2025

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)

Pier 1	Pier 2
Nail Spacing: 6	Nail Spacing: 6
HD Capacity: 2140	HD Capacity: 2140
HD Deflection: 0.088	HD Deflection: 0.088

G_i Override:

G_a Override:

C_d : 4.00

Three-Term Equation Deflection Check

$$\delta_{sw} = \frac{8vh^3}{EAb} + \frac{vh}{1000G_a} + \frac{h\Delta_a}{b} \quad (4.3-1)$$

	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	Term 2	Term 3	Term 1	Term 2	Term 3
Bending	Shear	Fastener	Bending	Shear	Fastener
	0.124	0.189		0.062	0.047
Sum			Sum		
Pier 2 (left)			Pier 2 (right)		
Term 1	Term 2	Term 3	Term 1	Term 2	Term 3
Bending	Shear	Fastener	Bending	Shear	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.



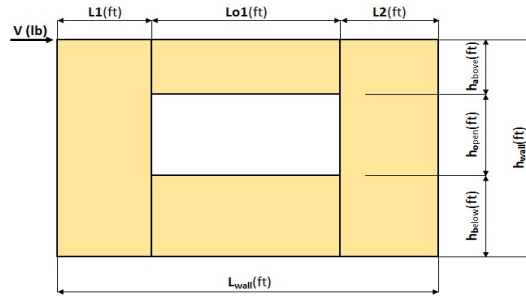
Force Transfer Around Openings Calculator

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
h _{wall}	9.00 ft	h _b	4.50 ft	P2=h _o /L2=	0.64
L _{wall}	10.83 ft	Lo1	2.50 ft		

1. Hold-down forces: $H = Vh_{\text{wall}}/L_{\text{wall}}$ 1826 lbf

2. Unit shear above + below opening
First opening: $va1 = vb1 = H/(h_a + h_b) = 304 \text{ plf}$

3. Total boundary force above + below openings
First opening: $O1 = va1 \times (Lo1) = 761 \text{ lbf}$

4. Corner forces
 $F1 = O1(L1)/(L1+L2) = 332 \text{ lbf}$
 $F2 = O1(L2)/(L1+L2) = 429 \text{ lbf}$

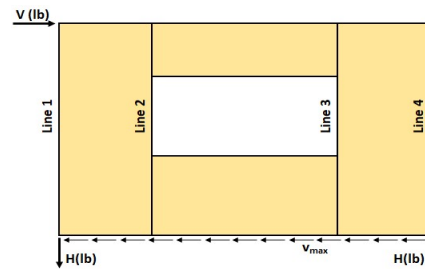
5. Tributary length of openings
 $T1 = (L1 \times Lo1)/(L1+L2) = 1.09 \text{ ft}$
 $T2 = (L2 \times Lo1)/(L1+L2) = 1.41 \text{ ft}$

6. Unit shear beside opening
 $v1 = (V/L)(L1+T1)/L1 = 264 \text{ plf}$
 $v2 = (V/L)(T2+L2)/L2 = 264 \text{ plf}$
Check $v1 \times L1 + v2 \times L2 = V?$ 2197 lbf OK

7. Resistance to corner forces
 $R1 = v1 \times L1 = 957 \text{ lbf}$
 $R2 = v2 \times L2 = 1240 \text{ lbf}$

8. Difference corner force + resistance
 $R1 - F1 = 626 \text{ lbf}$
 $R2 - F2 = 810 \text{ lbf}$

9. Unit shear in corner zones
 $vc1 = (R1 - F1)/L1 = 172 \text{ plf}$
 $vc2 = (R2 - F2)/L2 = 172 \text{ plf}$



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:

Designer: Chon Pieruccioni, PE

Date: 6/2/2025

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_i 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)

Nail Spacing: 6

HD Capacity: 2140

HD Deflection: 0.088

Pier 1

Pier 2

(in.)

(lbf)

(in.)

Four-Term Equation Deflection Check

$$\Delta = \frac{8vh^3}{EAb} + \frac{vh}{Gt} + 0.75he_n + d_a \frac{h}{b}$$
 (Equation 23-2)

Pier 1-L

Pier 1-R

Pier 2-L

Pier 2-R

$V_{unfactored}$: 264264264264 (plf)

E : 1.30E+061.30E+061.30E+061.30E+06 (psi)

h : 9.004.504.509.00 (ft)

Qty :

$Stud Size$:

A Override:

A :

G_i : 83,50083,50083,50083,500 (lbf/in.)

$Nail Spacing$: 6666 (in.)

V_n : 132132132132 (plf)

e_n : 0.01150.01150.01150.0115 (in.)

b : 3.633.634.704.70 (ft)

$HD Capacity$: 2140214021402140 (lbf)

$HD Defl$: 0.0880.0880.0880.088 (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	Term 2	Term 3	Term 4	Term 1	Term 2	Term 3	Term 4
Bending	Shear	Fastener	HD-1	Bending	Shear	Fastener	HD-2
	0.028	0.077	0.242		0.014	0.039	0.061
Sum				Sum			
Pier 2 (left)				Pier 2 (right)			
Term 1	Term 2	Term 3	Term 4	Term 1	Term 2	Term 3	Term 4
Bending	Shear	Fastener	HD-1	Bending	Shear	Fastener	HD-2
	0.014	0.039	0.047		0.028	0.077	0.187
Sum				Sum			

Total Defl.

(in.) %drift

Project Information

Code:

Designer: Chon Pieruccioni, PE

Date: 6/2/2025

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

$$\delta_{sw} = \frac{8vh^3}{Eab} + \frac{vh}{1000G_a} + \frac{h\Delta_a}{b} \quad (4.3-1)$$

	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	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	Term 2	Term 3	Term 1	Term 2	Term 3
Bending	Shear	Fastener	Bending	Shear	Fastener
	0.158	0.242		0.079	0.061
Sum			Sum		
Pier 2 (left)			Pier 2 (right)		
Term 1	Term 2	Term 3	Term 1	Term 2	Term 3
Bending	Shear	Fastener	Bending	Shear	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.