City of Puyallup
Development & Permitting Services
ISSUED PERMIT

Building	Planning
Engineering	Public Works
Fire	Traffic

These calculations must be on site and made available by the Permittee for all inspections.

PRCNC20250692

ENGINEERING ANALYSIS FOR: EAST TOWN CROSSING APARTMENTS 3002 E PIONEER WAY PUYALLUP, WA 98372 PARCEL NO: 0420264053 CLUBHOUSE | MANAGER'S APARTMENT

DESIGN CRITERIA

BUILDING CODE: 2021 INTERNATIONAL BUILDING CODE (IBC) AS AMENDED BY THE LOCAL JURISDICTION.

VERTICAL LOADS

ROOF LIVE LOAD:	*25 PSF (SNOW)*
ROOF DEAD LOAD:	*20 PSF*
RESIDENTIAL FLOOR LIVE LOAD:	*40 PSF (REDUCIBLE) : 60 PSF (FOR DECKS)*
FLOOR DEAD LOAD:	*30 PSF (INCLUDES 1½" GYP TOPPING)*

SNOW DESIGN DATA (ASCE 7-16)
FLAT SNOW LOAD: N/A
SNOW EXPOSURE FACTOR, Ce=1.0,
SNOW IMPORTANCE FACTOR, Is=1.0,
THERMAL FACTOR, Ct=1.1

WIND DESIGN DATA (ASCE 7-16)
BASIC WIND SPEED (ASD) V= 85MPH
ULTIMATE WIND SPEED V= 110MPH
RISK CATEGORY: II EXPOSURE: B
IMPORTANCE FACTOR, Iw= 1.0
TOPOGRAPHIC FACTOR, Kzt= 1.0

SEISMIC DESIGN DATA (ASCE7-16)
SEISMIC RESPONSE SYSTEM: WOOD SHEARWALLS
EQUIVALENT LATERAL FORCE PROCEDURE (ASCE 7-16)
RISK CATEGORY: II *SEISMIC IMPORTANCE FACTOR, Ie= 1.0*
MAPPED SPECTRAL RESPONSE ACCELERATION: Ss=1.24, S1=0.476
DESIGN SPECTRAL RESPONSE ACCELERATION: Sds=0.831, Sd1=0.476
SITE CLASS: D *SEISMIC DESIGN CATEGORY: D*
SEISMIC RESPONSE COEFFICIENT: Cs= 0.091
DESIGN BASE SHEAR: 21,805#
SOIL PROPERTIES:
BEARING CAPACITY: 2,000 PSF
LATERAL CAPACITY: 250 PSF/FT

01

REVISION 1

ADDED MISSING HEADER CALCULATIONS FOR 12' DECK WALL OPENING ON GRID E

City of Puyallup
Building
REVIEWED FOR COMPLIANCE
BSnowden
07/16/2025
2:47:04 PM

42816 REGISTERED PROFESSIONAL ENGINEER

PIERUCCIONI E&C, LLC
CHON PIERUCCIONI, PE
3128 N. BENNETT ST. TACOMA, WA 98407
PIERUCCIONIENGINEERING@GMAIL.COM
206.949.7866

EAST TOWN CROSSING
CLUBHOUSE WITH MANAGER'S APARTMENT
PIONEER & SHAW PUYALLUP WA

REVISIONS

01	CITY REVIEW COMMENTS

REVISIONS

ENGINEER:	CP
CHECKED BY:	CP
DATE:	2025.02.28
TITLE:	STRUCTURAL ANALYSIS
PROJECT # :	----

Level 2 and Lower Roof			
Member Name	**Results (Max UTIL %)**	**Current Solution**	**Comments**
13'-4" Floor Joist	Passed (78% M)	1 piece(s) 2 x 12 DF No.2 @ 16" OC	
16'-0" Floor Joist	Passed (40% ΔT)	1 piece(s) 1 3/4" x 11 1/4" 2.0E Microllam® LVL @ 16" OC	
17'-4" Floor Joist	Passed (51% ΔT)	1 piece(s) 1 3/4" x 11 1/4" 2.0E Microllam® LVL @ 16" OC	
10' Deck Joist	Passed (59% M)	2 piece(s) 2 x 8 HF No.2 @ 16" OC	
Grid H (3-5.5) Floor Beam	Passed (87% ΔT)	2 piece(s) 1 3/4" x 11 1/4" 2.0E Microllam® LVL	
Grid H (5.5-7) Floor Beam	Passed (73% ΔT)	3 piece(s) 1 3/4" x 11 1/4" 2.0E Microllam® LVL	
Grid 5.5 (F-I) Floor Beam	Passed (93% M+)	1 piece(s) 5 1/2" x 19 1/2" 24F-V4 DF Glulam	
Grid 5.5I Post	Passed (79% f_c)	1 piece(s) 6 x 8 DF No.2	
Grid 5.5F Post	Passed (86% f_c)	1 piece(s) 6 x 6 DF No.2	
Grid 7 (F-I) Floor Beam	Passed (92% V)	1 piece(s) 5 1/2" x 22 1/2" 24F-V4 DF Glulam	
Grid 7I Post	Passed (97% f_c)	1 piece(s) 6 x 10 DF No.2	
Grid 7F Post	Passed (86% f_c)	1 piece(s) 6 x 8 DF No.2	
Grid F (3-5.5) Floor Beam	Passed (81% ΔL)	3 piece(s) 1 3/4" x 11 1/4" 2.0E Microllam® LVL	
Grid E.5 (5-5.5) Floor Beam	Passed (56% M)	1 piece(s) 4 x 12 DF No.2	
Grid E (5-5.5) Floor Beam	Passed (99% M)	1 piece(s) 4 x 12 DF No.2	
Grid 8 (E-E.4) Floor Beam	Passed (79% M)	1 piece(s) 2 x 12 DF No.2	
Grid 8 (E.4-F) Floor Beam	Passed (87% M)	1 piece(s) 4 x 12 DF No.2	
Grid E (5.5-9) Floor Beam	Passed (89% M+)	1 piece(s) 5 1/2" x 19 1/2" 24F-V4 DF Glulam	
Grid 5.5E Post	Passed (80% f_c)	1 piece(s) 6 x 8 DF No.2	Could not find information based on inputs for ~1.
Grid 9 (D.2-E.2) 6' Window Header	Passed (99% V)	1 piece(s) 5 1/2" x 10 1/2" 24F-V4 DF Glulam	
Grid 9E Post	Passed (82% f_c)	1 piece(s) 6 x 6 DF No.2	
Grid I - Corbal Roof Beam	Passed (100% ΔT)	1 piece(s) 5 1/2" x 7 1/2" 24F-V4 DF Glulam	
Grid E - Corbal Roof Beam	Passed (43% ΔT)	1 piece(s) 5 1/2" x 7 1/2" 24F-V4 DF Glulam	
Grid 5 - Corbal Roof Beam	Passed (36% M)	1 piece(s) 6 x 8 DF No.2	
Grid 1 - Corbal Roof Beam	Passed (11% M)	1 piece(s) 6 x 8 DF No.2	
Upper Roof			
Member Name	**Results (Max UTIL %)**	**Current Solution**	**Comments**
Grid 7 (F.5-G.6) 9' Window Header	Passed (88% M+)	1 piece(s) 5 1/2" x 13 1/2" 24F-V4 DF Glulam	
Grid H - 9' Window Header	Passed (59% M)	1 piece(s) 4 x 8 DF No.2	
Grid 2 - 9' Window Header	Passed (59% M)	1 piece(s) 4 x 8 DF No.2	
Grid 2 - 7' Window Header	Passed (36% M)	1 piece(s) 4 x 8 DF No.2	
Grid E - 12' Header	Passed (73% M+)	1 piece(s) 3 1/2" x 12" 24F-V4 DF Glulam	

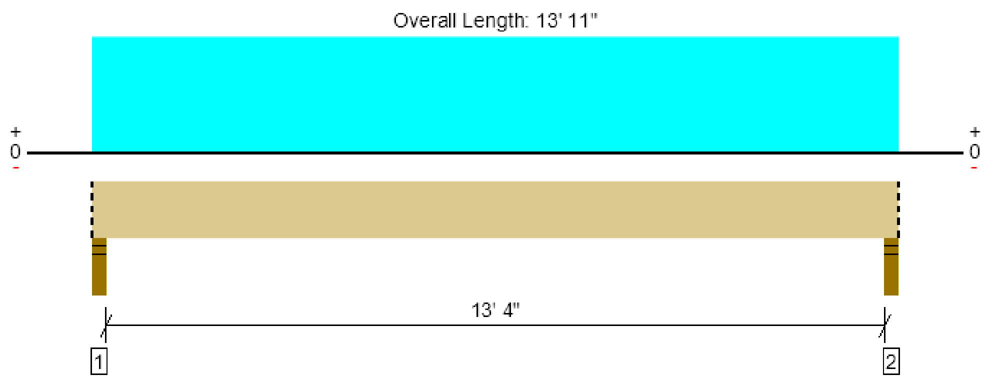
ForteWEB Software Operator	Job Notes
Chon Pieruccioni Pieruccioni Engineering (206) 949-7866 cpieru@hotmail.com	

MEMBER REPORT

PASSED

Level 2 and Lower Roof, 13'-4" Floor Joist

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)	649 @ 2 1/2"	2126 (3.50")	Passed (31%)	--	1.0 D + 1.0 L (All Spans)
Shear (lbs)	535 @ 1' 2 3/4"	2025	Passed (26%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	2126 @ 6' 11 1/2"	2729	Passed (78%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.140 @ 6' 11 1/2"	0.450	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.245 @ 6' 11 1/2"	0.675	Passed (L/661)	--	1.0 D + 1.0 L (All Spans)
TJ-Pro™ Rating	N/A	N/A	N/A	--	N/A

Member Length : 13' 11"
System : Floor
Member Type : Joist
Building Use : Residential
Building Code : IBC 2021
Design Methodology : ASD

- Deflection criteria: LL (L/360) and TL (L/240).
- Allowed moment does not reflect the adjustment for the beam stability factor.
- 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"	278	371	649	Blocking
2 - Stud wall - HF	3.50"	3.50"	1.50"	278	371	649	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)	5' 11" o/c	
Bottom Edge (Lu)	13' 11" 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 13' 11"	16"	30.0	40.0	Default Load

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

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

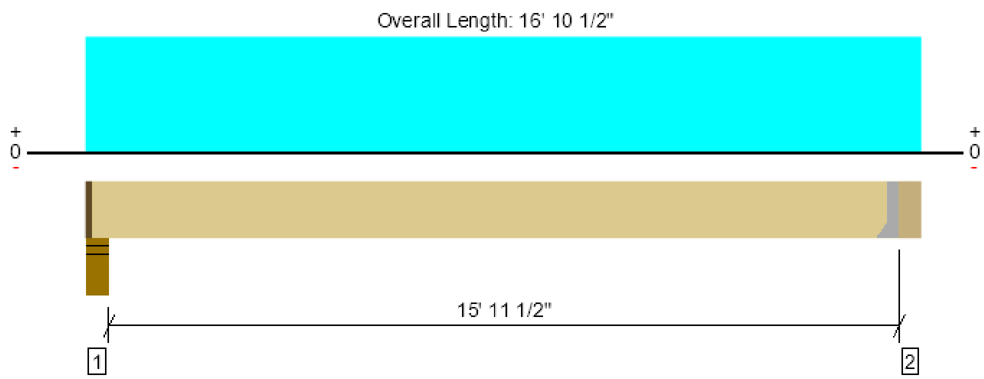
FORTEWEB

MEMBER REPORT

PASSED

Level 2 and Lower Roof, 16'-0" Floor Joist

1 piece(s) 1 3/4" x 11 1/4" 2.0E Microllam® LVL @ 16" OC



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Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	749 @ 16' 5"	1969 (1.50")	Passed (38%)	--	1.0 D + 1.0 L (All Spans)
Shear (lbs)	661 @ 15' 5 3/4"	3741	Passed (18%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	3002 @ 8' 4 3/4"	8391	Passed (36%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.182 @ 8' 4 3/4"	0.535	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.319 @ 8' 4 3/4"	0.802	Passed (L/604)	--	1.0 D + 1.0 L (All Spans)
TJ-Pro™ Rating	53	40	Passed	--	--

Member Length : 16' 3 1/2"
System : Floor
Member Type : Joist
Building Use : Residential
Building Code : IBC 2021
Design Methodology : ASD

- Deflection criteria: LL (L/360) and TL (L/240).
- Allowed moment does not reflect the adjustment for the beam stability factor.
- A 4% increase in the moment capacity has been added to account for repetitive member usage.
- A structural analysis of the deck has not been performed.
- Deflection analysis is based on composite action with a single layer of 23/32" Weyerhaeuser Edge™ Panel (24" Span Rating) that is glued and nailed down.
- Additional considerations for the TJ-Pro™ Rating include: 1/2" Gypsum ceiling.

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Floor Live	Factored	
1 - Stud wall - HF	5.50"	4.00"	1.50"	336	448	784	1 1/2" Rim Board
2 - Hanger on 11 1/4" GLB beam	5.50"	Hanger[1]	1.50"	339	452	791	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)	15' o/c	
Bottom Edge (Lu)	16' 4" 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	IUS1.81/9.5	2.00"	N/A	8-10dx1.5	2-10dx1.5	

- 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 16' 10 1/2"	16"	30.0	40.0	Default Load

Weyerhaeuser Notes

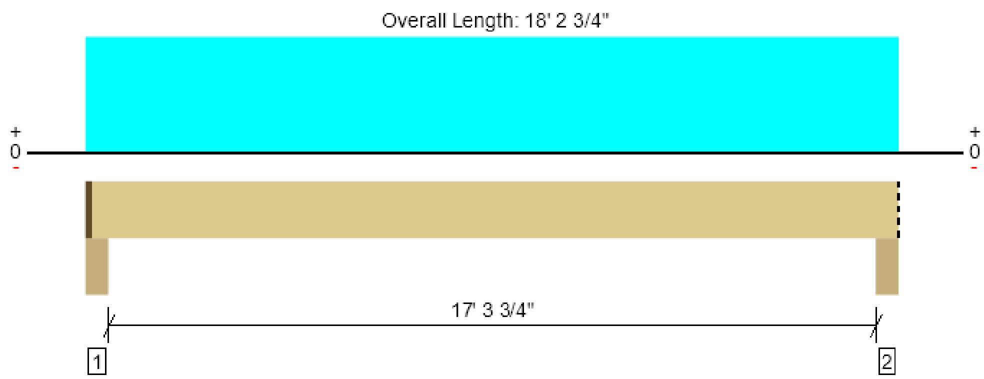
ForteWEB Software Operator	Job Notes
Chon Pieruccioni Pieruccioni Engineering (206) 949-7866 cpieru@hotmail.com	

Weyerhaeuser

7/7/2025 3:30:00 PM UTC
ForteWEB v3.9, Engine: V8.4.3.94, Data: V8.1.7.3
File Name: East Town Crossing - Clubhouse

Level 2 and Lower Roof, 17'-4" Floor Joist

1 piece(s) 1 3/4" x 11 1/4" 2.0E Microllam® LVL @ 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)	839 @ 4 1/2"	4550 (4.00")	Passed (18%)	--	1.0 D + 1.0 L (All Spans)
Shear (lbs)	720 @ 1' 4 3/4"	3741	Passed (19%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	3564 @ 9' 1 3/8"	8391	Passed (42%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.254 @ 9' 1 3/8"	0.583	Passed (L/824)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.445 @ 9' 1 3/8"	0.874	Passed (L/471)	--	1.0 D + 1.0 L (All Spans)
TJ-Pro™ Rating	50	40	Passed	--	--

Member Length : 18' 1 1/4"
System : Floor
Member Type : Joist
Building Use : Residential
Building Code : IBC 2021
Design Methodology : ASD

- Deflection criteria: LL (L/360) and TL (L/240).
- Allowed moment does not reflect the adjustment for the beam stability factor.
- A 4% increase in the moment capacity has been added to account for repetitive member usage.
- A structural analysis of the deck has not been performed.
- Deflection analysis is based on composite action with a single layer of 23/32" Weyerhaeuser Edge™ Panel (24" Span Rating) that is glued and nailed down.
- Additional considerations for the TJ-Pro™ Rating include: 1/2" Gypsum ceiling.

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Floor Live	Factored	
1 - Beam - GLB	5.50"	4.00"	1.50"	365	486	851	1 1/2" Rim Board
2 - Beam - GLB	5.50"	5.50"	1.50"	365	486	851	Blocking

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

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	12' 5" o/c	
Bottom Edge (Lu)	18' 1" 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 18' 2 3/4"	16"	30.0	40.0	Default Load

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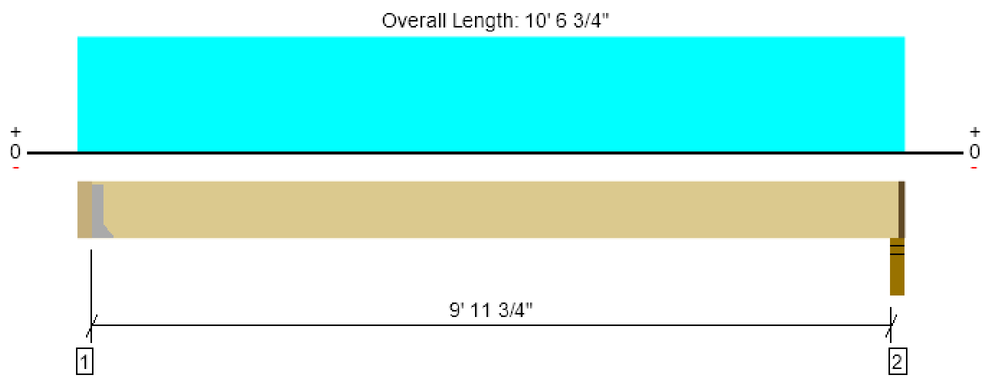
The product application, input design loads, dimensions and support information have been provided by ForteWEB Software Operator

MEMBER REPORT

PASSED

Level 2 and Lower Roof, 10' Deck Joist

2 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)	604 @ 3 1/2"	1823 (1.50")	Passed (33%)	--	1.0 D + 1.0 L (All Spans)
Shear (lbs)	531 @ 10 3/4"	2175	Passed (24%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	1519 @ 5' 3 7/8"	2569	Passed (59%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.149 @ 5' 3 7/8"	0.335	Passed (L/810)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.224 @ 5' 3 7/8"	0.503	Passed (L/540)	--	1.0 D + 1.0 L (All Spans)
TJ-Pro™ Rating	N/A	N/A	N/A	--	N/A

Member Length : 10' 1 3/4"
System : Floor
Member Type : Joist
Building Use : Residential
Building Code : IBC 2021
Design Methodology : ASD

- Deflection criteria: LL (L/360) and TL (L/240).
- Allowed moment does not reflect the adjustment for the beam stability factor.
- 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" HF beam	3.50"	Hanger[1]	1.50"	213	426	639	See note [1]
2 - Stud wall - HF	3.50"	2.00"	1.50"	210	419	629	1 1/2" Rim Board

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

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	10' 2" o/c	
Bottom Edge (Lu)	10' 2" o/c	

•Maximum allowable bracing intervals based on applied load.

Connector: Simpson Strong-Tie

Support	Model	Seat Length	Top Fasteners	Face Fasteners	Member Fasteners	Accessories
1 - Face Mount Hanger	LUS26-2	2.00"	N/A	4-10dx1.5	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 10' 6 3/4"	16"	30.0	60.0	Default Load

Weyerhaeuser Notes

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

Weyerhaeuser

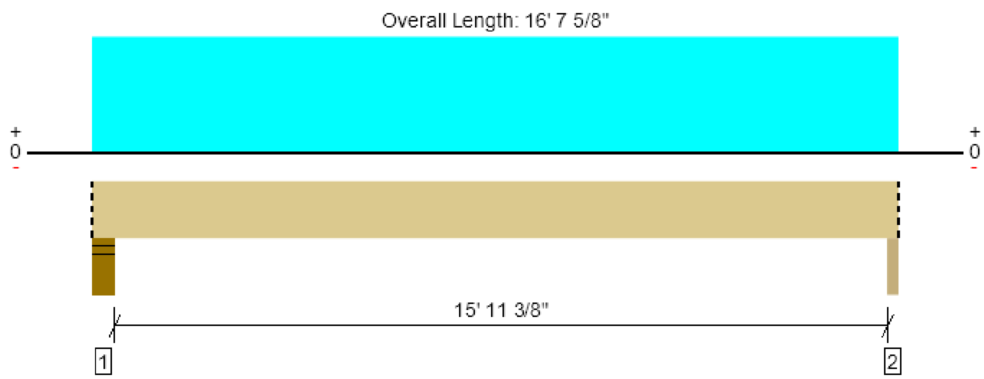
7/7/2025 3:30:00 PM UTC

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

File Name: East Town Crossing - Clubhouse

Level 2 and Lower Roof, Grid H (3-5.5) Floor Beam

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



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Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	2963 @ 16' 6 3/8"	6256 (2.75")	Passed (47%)	--	1.0 D + 1.0 S (All Spans)
Shear (lbs)	2541 @ 1' 4 3/4"	8603	Passed (30%)	1.15	1.0 D + 1.0 S (All Spans)
Moment (Ft-lbs)	11845 @ 8' 5 3/16"	18558	Passed (64%)	1.15	1.0 D + 1.0 S (All Spans)
Live Load Defl. (in)	0.222 @ 8' 5 3/16"	0.540	Passed (L/877)	--	1.0 D + 1.0 S (All Spans)
Total Load Defl. (in)	0.708 @ 8' 5 3/16"	0.810	Passed (L/274)	--	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.

Member Length : 16' 7 5/8"
System : Floor
Member Type : Drop Beam
Building Use : Residential
Building Code : IBC 2021
Design Methodology : ASD

Supports	Bearing Length			Loads to Supports (lbs)				Accessories
	Total	Available	Required	Dead	Floor Live	Snow	Factored	
1 - Stud wall - HF	5.50"	5.50"	2.15"	2092	225	953	3046	Blocking
2 - Beam - GLB	2.75"	2.75"	1.50"	2036	219	927	2963	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' 2" o/c	
Bottom Edge (Lu)	16' 8" o/c	

•Maximum allowable bracing intervals based on applied load.

Vertical Loads	Location (Side)	Tributary Width	Dead (0.90)	Floor Live (1.00)	Snow (1.15)	Comments
0 - Self Weight (PLF)	0 to 16' 7 5/8"	N/A	11.5	--	--	
1 - Uniform (PSF)	0 to 16' 7 5/8" (Front)	8"	30.0	40.0	--	Default Load
2 - Uniform (PSF)	0 to 16' 7 5/8" (Front)	9 1/4"	24.0	--	25.0	Lower Roof
3 - Uniform (PSF)	0 to 16' 7 5/8" (Front)	3' 9"	24.0	--	25.0	Upper Roof
4 - Uniform (PLF)	0 to 16' 7 5/8" (Top)	N/A	108.0	--	--	Wall

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

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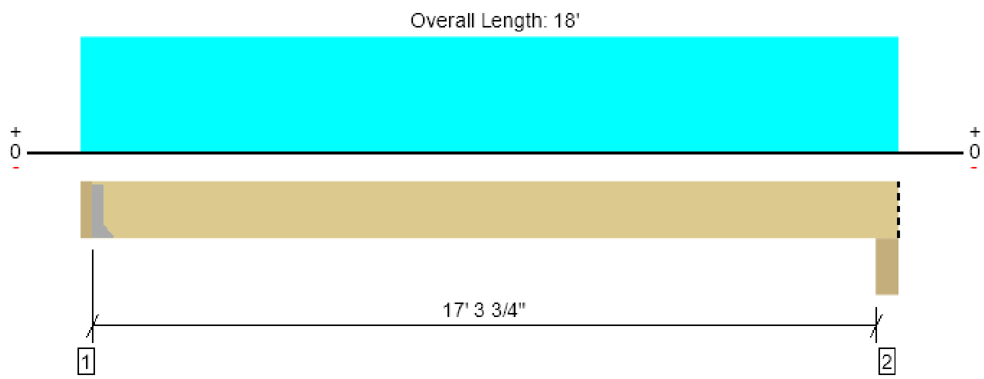
The product application, input design loads, dimensions and support information have been provided by ForteWEB Software Operator

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

Level 2 and Lower Roof, Grid H (5.5-7) Floor Beam

3 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)	3199 @ 2 3/4"	5906 (1.50")	Passed (54%)	--	1.0 D + 1.0 S (All Spans)
Shear (lbs)	2855 @ 1' 2"	12905	Passed (22%)	1.15	1.0 D + 1.0 S (All Spans)
Moment (Ft-lbs)	13946 @ 8' 11 3/8"	27837	Passed (50%)	1.15	1.0 D + 1.0 S (All Spans)
Live Load Defl. (in)	0.197 @ 8' 11 3/8"	0.581	Passed (L/999+)	--	1.0 D + 1.0 S (All Spans)
Total Load Defl. (in)	0.640 @ 8' 11 3/8"	0.872	Passed (L/327)	--	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.

Member Length : 17' 9 1/4"
System : Floor
Member Type : Drop Beam
Building Use : Residential
Building Code : IBC 2021
Design Methodology : ASD

Supports	Bearing Length			Loads to Supports (lbs)				Accessories
	Total	Available	Required	Dead	Floor Live	Snow	Factored	
1 - Hanger on 11 1/4" GLB beam	2.75"	Hanger[1]	1.50"	2268	239	1011	3279	See note [1]
2 - Beam - GLB	5.50"	5.50"	1.50"	2298	241	1023	3321	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)	17' 9" o/c	
Bottom Edge (Lu)	17' 9" o/c	

•Maximum allowable bracing intervals based on applied load.

Connector: Simpson Strong-Tie						
Support	Model	Seat Length	Top Fasteners	Face Fasteners	Member Fasteners	Accessories
1 - Face Mount Hanger	HU612	2.50"	N/A	22-16d	8-16d	

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

Vertical Loads	Location (Side)	Tributary Width	Dead (0.90)	Floor Live (1.00)	Snow (1.15)	Comments
0 - Self Weight (PLF)	2 3/4" to 18'	N/A	17.2	--	--	
1 - Uniform (PSF)	0 to 18' (Front)	8"	30.0	40.0	--	Default Load
2 - Uniform (PSF)	0 to 18' (Front)	9 1/4"	24.0	--	25.0	Lower Roof
3 - Uniform (PSF)	0 to 18' (Front)	3' 9"	24.0	--	25.0	Upper Roof
4 - Uniform (PLF)	0 to 18' (Top)	N/A	108.0	--	--	Wall

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

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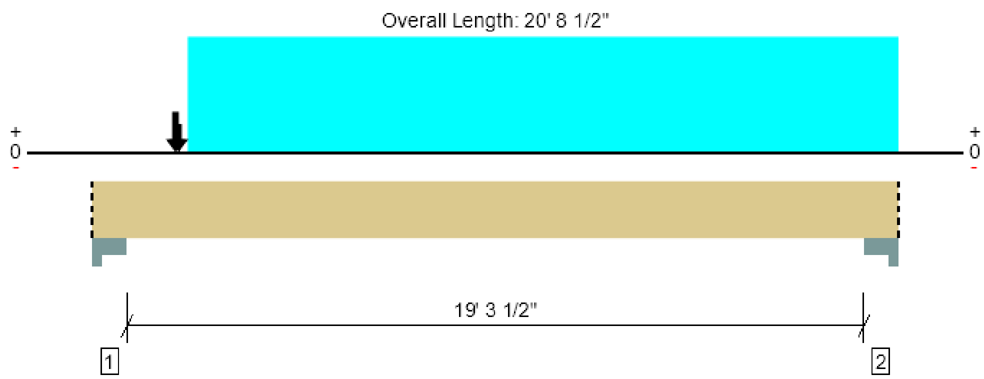
The product application, input design loads, dimensions and support information have been provided by ForteWEB Software Operator

Level 2 and Lower Roof, Grid 5.5 (F-I) Floor Beam

1 piece(s) 5 1/2" x 19 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)	16569 @ 7"	30388 (8.50")	Passed (55%)	--	1.0 D + 0.75 L + 0.75 S (All Spans)
Shear (lbs)	15403 @ 2' 4"	18948	Passed (81%)	1.00	1.0 D + 1.0 L (All Spans)
Pos Moment (Ft-lbs)	61928 @ 9' 11 9/16"	66418	Passed (93%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.359 @ 10' 4 5/16"	0.489	Passed (L/653)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.705 @ 10' 2 7/8"	0.977	Passed (L/333)	--	1.0 D + 1.0 L (All Spans)

Member Length : 20' 8 1/2"
System : Floor
Member Type : Flush Beam
Building Use : Residential
Building Code : IBC 2021
Design Methodology : ASD

- Deflection criteria: LL (L/480) and TL (L/240).
- Allowed moment does not reflect the adjustment for the beam stability factor.
- Volume factor of 0.95 was calculated for positive bending using length L = 19' 6 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	Snow	Factored	
1 - Column Cap - steel	8.50"	8.50"	4.63"	10111	5979	2632	16569	Blocking
2 - Column Cap - steel	8.50"	8.50"	3.61"	5951	6941	233	12891	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)	18' 5" o/c	
Bottom Edge (Lu)	20' 9" o/c	

•Maximum allowable bracing intervals based on applied load.

Vertical Loads	Location (Side)	Tributary Width	Dead (0.90)	Floor Live (1.00)	Snow (1.15)	Comments
0 - Self Weight (PLF)	0 to 20' 8 1/2"	N/A	26.1	--	--	
1 - Uniform (PSF)	2' 5 1/2" to 20' 8 1/2" (Front)	16' 9 1/4"	30.0	40.0	--	Floor
2 - Point (lb)	2' 1 3/4" (Back)	N/A	2036	219	927	Linked from: Grid H (3-5.5) Floor Beam, Support 2
3 - Point (lb)	2' 1 3/4" (Back)	N/A	2036	219	927	Linked from: Grid H (3-5.5) Floor Beam, Support 2
4 - Point (lb)	2' 2 3/4" (Front)	N/A	2268	239	1011	Linked from: Grid H (5.5-7) Floor Beam, Support 1

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

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Level 2 and Lower Roof, Grid 5.5I Post

1 piece(s) 6 x 8 DF No.2

Post Height: 10' 6"

Drawing is Conceptual

Design Results	Actual	Allowed	Result	LDF	Load: Combination
Slenderness	23	50	Passed (46%)	--	--
Compression (lbs)	16090	20446	Passed (79%)	1.00	1.0 D + 1.0 L
Base Bearing (lbs)	16569	1225125	Passed (1%)	--	1.0 D + 0.75 L + 0.75 S
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.

Supports	Type	Material
Base	Plate	Steel

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

Max Unbraced Length	Comments
Full Member Length	No bracing assumed.

Vertical Load	Dead (0.90)	Floor Live (1.00)	Snow (1.15)	Comments
1 - Point (lb)	10111	5979	2632	Linked from: Grid 5.5 (F-I) Floor Beam, Support 1

Weyerhaeuser Notes

Level 2 and Lower Roof, Grid 5.5F Post

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

Post Height: 10' 6"

Drawing is Conceptual

Design Results	Actual	Allowed	Result	LDF	Load: Combination
Slenderness	23	50	Passed (46%)	--	--
Compression (lbs)	12892	14994	Passed (86%)	1.00	1.0 D + 1.0 L
Base Bearing (lbs)	12892	898425	Passed (1%)	--	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.

Supports	Type	Material
Base	Plate	Steel

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

Max Unbraced Length	Comments
Full Member Length	No bracing assumed.

Vertical Load	Dead (0.90)	Floor Live (1.00)	Snow (1.15)	Comments
1 - Point (lb)	5951	6941	233	Linked from: Grid 5.5 (F-I) Floor Beam, Support 2

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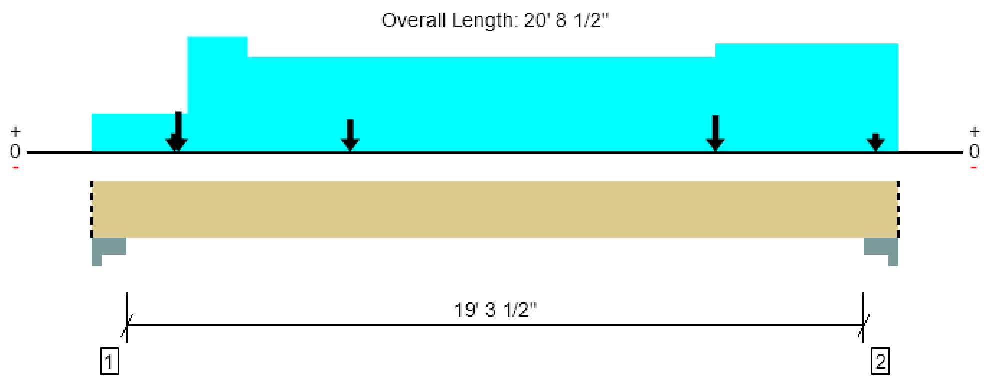
The product application, input design loads, dimensions and support information have been provided by ForteWEB Software Operator

Level 2 and Lower Roof, Grid 7 (F-I) Floor Beam

1 piece(s) 5 1/2" x 22 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)	24680 @ 7"	30388 (8.50")	Passed (81%)	--	1.0 D + 0.525 E + 0.75 L + 0.75 S (All Spans)
Shear (lbs)	20050 @ 2' 7"	21863	Passed (92%)	1.00	1.0 D + 1.0 L (All Spans)
Pos Moment (Ft-lbs)	87741 @ 8' 8"	100245	Passed (88%)	1.15	1.0 D + 0.75 L + 0.75 S (All Spans)
Live Load Defl. (in)	0.286 @ 10' 2 5/8"	0.489	Passed (L/820)	--	1.0 D + 0.75 L + 0.75 S (All Spans)
Total Load Defl. (in)	0.674 @ 10' 2 7/8"	0.977	Passed (L/348)	--	1.0 D + 0.75 L + 0.75 S (All Spans)

Member Length : 20' 8 1/2"
System : Floor
Member Type : Flush Beam
Building Use : Residential
Building Code : IBC 2021
Design Methodology : ASD

- Deflection criteria: LL (L/480) and TL (L/240).
- Allowed moment does not reflect the adjustment for the beam stability factor.
- Volume factor of 0.94 was calculated for positive bending using length L = 19' 6 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	Snow	Seismic	Factored	
1 - Column Cap - steel	8.50"	8.50"	6.90"	13747	9251	4946	543/-543	24680	Blocking
2 - Column Cap - steel	8.50"	8.50"	5.35"	10903	4305	6232	637/-637	19140	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)	18' o/c	
Bottom Edge (Lu)	20' 9" o/c	

•Maximum allowable bracing intervals based on applied load.

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Vertical Loads	Location (Side)	Tributary Width	Dead (0.90)	Floor Live (1.00)	Snow (1.15)	Seismic (1.60)	Comments
0 - Self Weight (PLF)	0 to 20' 8 1/2"	N/A	30.1	--	--	--	
1 - Uniform (PSF)	2' 5 1/2" to 20' 8 1/2" (Front)	9' 3/4"	30.0	40.0	--	--	Floor
2 - Uniform (PSF)	0 to 20' 8 1/2" (Front)	9 3/4"	24.0	--	25.0	--	Lower Roof
3 - Uniform (PLF)	0 to 20' 8 1/2" (Top)	N/A	108.0	--	--	--	Wall
4 - Uniform (PSF)	0 to 4' (Front)	3' 8 3/4"	24.0	--	25.0	--	Upper Roof
5 - Uniform (PSF)	16' 1/8" to 20' 8 1/2" (Front)	2' 6"	24.0	--	25.0	--	Upper Roof
6 - Point (lb)	2' 1 1/2" (Front)	N/A	--	--	--	590	Seismic Strap
7 - Point (lb)	20' 1 1/2" (Front)	N/A	--	--	--	590	Seismic Strap
8 - Point (lb)	2' 2 3/4" (Back)	N/A	5951	6941	233	--	Linked from: Grid 5.5 (F-I) Floor Beam, Support 2
9 - Point (lb)	6' 7 5/8" (Top)	N/A	4306	--	4325	--	Linked from: Grid 7 (F.5-G.6) 9' Window Header, Support 1
10 - Point (lb)	16' 1/8" (Top)	N/A	5526	--	5533	--	Linked from: Grid 7 (F.5-G.6) 9' Window Header, Support 2

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

Weyerhaeuser Notes

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

Level 2 and Lower Roof, Grid 7I Post

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

Post Height: 10' 6"

Drawing is Conceptual

Design Results	Actual	Allowed	Result	LDF	Load: Combination [Load Group]
Slenderness	23	50	Passed (46%)	--	--
Compression (lbs)	22998	23719	Passed (97%)	1.00	1.0 D + 1.0 L [1]
Base Bearing (lbs)	24680	1551825	Passed (2%)	--	1.0 D + 0.525 E + 0.75 L + 0.75 S [1]
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.
- Lumber grading provisions must be extended over the length of the member per NDS 4.2.5.5.

Supports	Type	Material
Base	Plate	Steel

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

Max Unbraced Length	Comments
Full Member Length	No bracing assumed.

Vertical Load	Dead (0.90)	Floor Live (1.00)	Snow (1.15)	Seismic (1.60)	Comments
1 - Point (lb)	13747	9251	4946	543/-543	Linked from: Grid 7 (F-I) Floor Beam, Support 1

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Level 2 and Lower Roof, Grid 7F Post

1 piece(s) 6 x 8 DF No.2

Post Height: 10' 6"

Drawing is Conceptual

Design Results	Actual	Allowed	Result	LDF	Load: Combination [Load Group]
Slenderness	23	50	Passed (46%)	--	--
Compression (lbs)	18806	21893	Passed (86%)	1.15	1.0 D + 0.75 L + 0.75 S [1]
Base Bearing (lbs)	19140	1225125	Passed (2%)	--	1.0 D + 0.525 E + 0.75 L + 0.75 S [1]
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.

Supports	Type	Material
Base	Plate	Steel

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

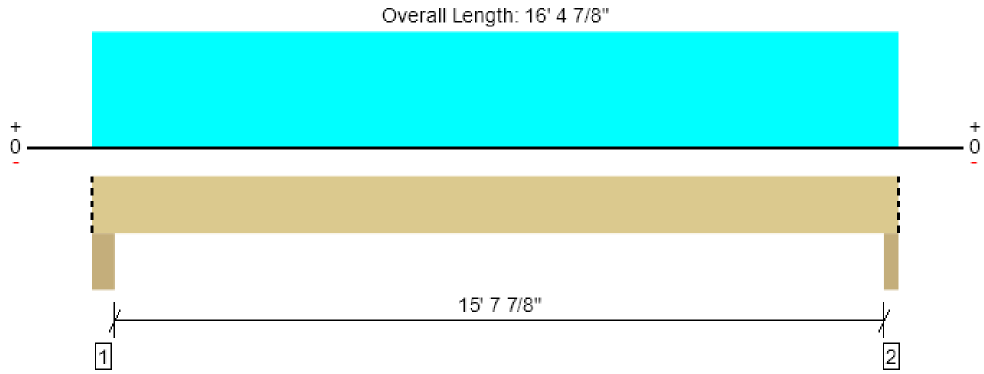
Max Unbraced Length	Comments
Full Member Length	No bracing assumed.

Vertical Load	Dead (0.90)	Floor Live (1.00)	Snow (1.15)	Seismic (1.60)	Comments
1 - Point (lb)	10903	4305	6232	637/-637	Linked from: Grid 7 (F-I) Floor Beam, Support 2

Weyerhaeuser Notes

Level 2 and Lower Roof, Grid F (3-5.5) Floor Beam

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



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Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	3882 @ 16' 2 7/8"	13322 (3.50")	Passed (29%)	--	1.0 D + 1.0 L (All Spans)
Shear (lbs)	3294 @ 1' 4 3/4"	11222	Passed (29%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	15119 @ 8' 3 7/16"	24206	Passed (62%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.321 @ 8' 3 7/16"	0.398	Passed (L/595)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.582 @ 8' 3 7/16"	0.795	Passed (L/328)	--	1.0 D + 1.0 L (All Spans)

Member Length : 16' 4 7/8"
System : Floor
Member Type : Flush Beam
Building Use : Residential
Building Code : IBC 2021
Design Methodology : ASD

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

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Floor Live	Factored	
1 - Column - HF	5.50"	5.50"	1.50"	1779	2182	3961	Blocking
2 - Column - HF	3.50"	3.50"	1.50"	1744	2138	3882	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)	16' 5" o/c	
Bottom Edge (Lu)	16' 5" 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 16' 4 7/8"	N/A	17.2	--	
1 - Uniform (PSF)	0 to 16' 4 7/8" (Front)	6' 7"	30.0	40.0	Floor

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

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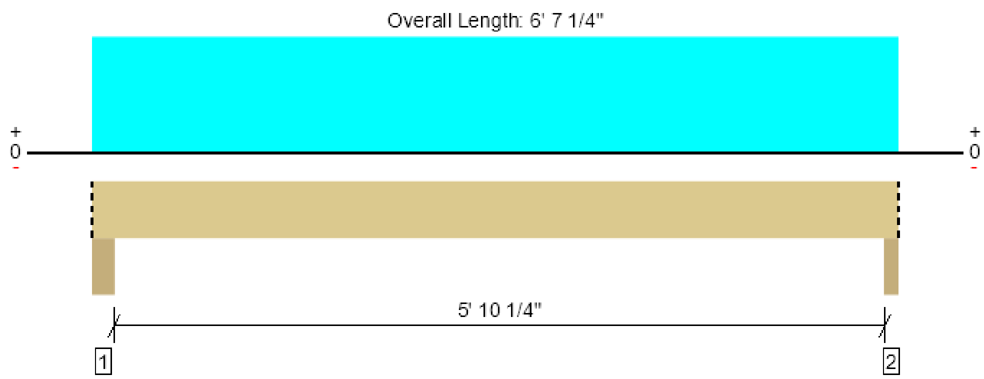
Level 2 and Lower Roof, Grid E.5 (5-5.5) Floor Beam

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)	2360 @ 6' 5 1/4"	7656 (3.50")	Passed (31%)	--	1.0 D + 1.0 L (All Spans)
Shear (lbs)	1459 @ 1' 4 3/4"	4725	Passed (31%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	3415 @ 3' 4 5/8"	6091	Passed (56%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.019 @ 3' 4 5/8"	0.153	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.034 @ 3' 4 5/8"	0.305	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)

Member Length : 6' 7 1/4"
System : Floor
Member Type : Flush Beam
Building Use : Residential
Building Code : IBC 2021
Design Methodology : ASD

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

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Floor Live	Factored	
1 - Column - HF	5.50"	5.50"	1.50"	1083	1399	2483	Blocking
2 - Column - HF	3.50"	3.50"	1.50"	1030	1330	2360	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' 7" o/c	
Bottom Edge (Lu)	6' 7" 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 6' 7 1/4"	N/A	10.0	--	
1 - Uniform (PSF)	0 to 6' 7 1/4" (Front)	10' 4"	30.0	40.0	Floor

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

Weyerhaeuser Notes

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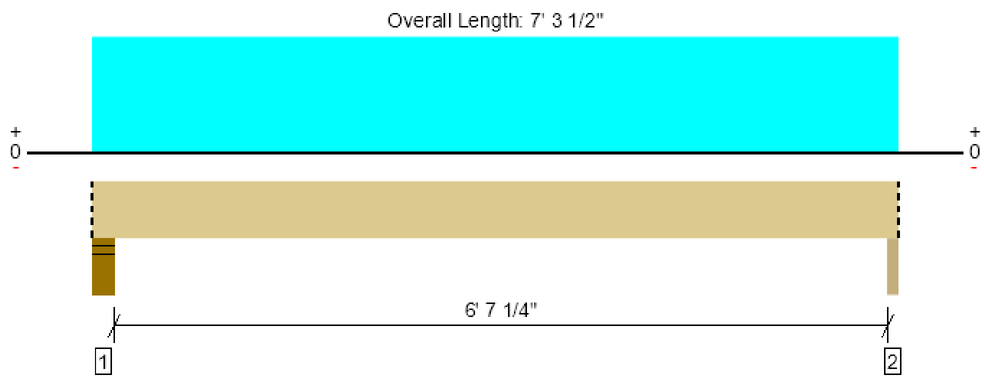
The product application, input design loads, dimensions and support information have been provided by ForteWEB Software Operator

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

PASSED

Level 2 and Lower Roof, Grid E (5-5.5) Floor 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)	4186 @ 7' 2 1/4"	6016 (2.75")	Passed (70%)	--	1.0 D + 0.75 L + 0.75 S (All Spans)
Shear (lbs)	2803 @ 1' 4 3/4"	5434	Passed (52%)	1.15	1.0 D + 0.75 L + 0.75 S (All Spans)
Moment (Ft-lbs)	6961 @ 3' 9 1/8"	7004	Passed (99%)	1.15	1.0 D + 0.75 L + 0.75 S (All Spans)
Live Load Defl. (in)	0.036 @ 3' 9 1/8"	0.228	Passed (L/999+)	--	1.0 D + 0.75 L + 0.75 S (All Spans)
Total Load Defl. (in)	0.089 @ 3' 9 1/8"	0.343	Passed (L/928)	--	1.0 D + 0.75 L + 0.75 S (All Spans)

Member Length : 7' 3 1/2"
System : Floor
Member Type : Drop Beam
Building Use : Residential
Building Code : IBC 2021
Design Methodology : ASD

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

Supports	Bearing Length			Loads to Supports (lbs)				Accessories
	Total	Available	Required	Dead	Floor Live	Snow	Factored	
1 - Stud wall - HF	5.50"	5.50"	3.14"	2622	821	1626	4457	Blocking
2 - Beam - GLB	2.75"	2.75"	1.91"	2463	771	1527	4186	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' o/c	
Bottom Edge (Lu)	7' 4" o/c	

•Maximum allowable bracing intervals based on applied load.

Vertical Loads	Location (Side)	Tributary Width	Dead (0.90)	Floor Live (1.00)	Snow (1.15)	Comments
0 - Self Weight (PLF)	0 to 7' 3 1/2"	N/A	10.0	--	--	
1 - Uniform (PSF)	0 to 7' 3 1/2" (Front)	5' 5 1/2"	30.0	40.0	--	Floor
2 - Uniform (PSF)	0 to 7' 3 1/2" (Front)	1'	24.0	--	25.0	Lower Roof
3 - Uniform (PSF)	0 to 7' 3 1/2" (Front)	16' 3 1/2"	24.0	--	25.0	Upper Roof
4 - Uniform (PLF)	0 to 7' 3 1/2" (Top)	N/A	108.0	--	--	Wall

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

Weyerhaeuser Notes

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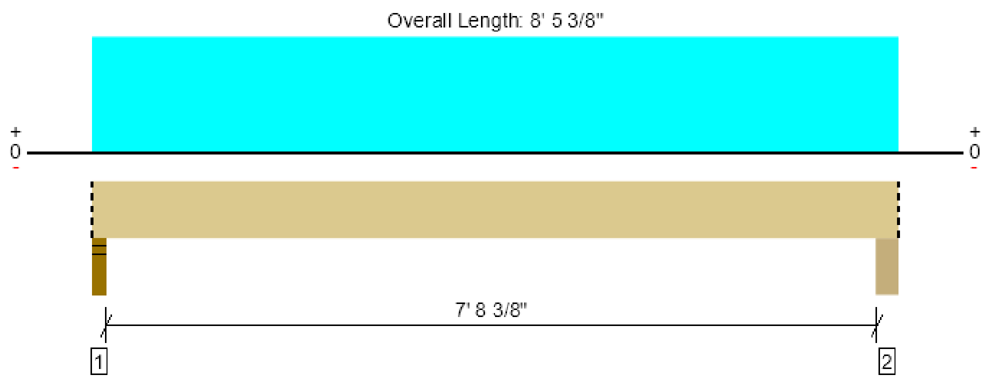
FORTEWEB

MEMBER REPORT

PASSED

Level 2 and Lower Roof, Grid 8 (E-E.4) Floor Beam

1 piece(s) 2 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)	1130 @ 2"	2126 (3.50")	Passed (53%)	--	1.0 D + 1.0 S (All Spans)
Shear (lbs)	795 @ 1' 2 3/4"	2329	Passed (34%)	1.15	1.0 D + 1.0 S (All Spans)
Moment (Ft-lbs)	2156 @ 4' 1 11/16"	2729	Passed (79%)	1.15	1.0 D + 1.0 S (All Spans)
Live Load Defl. (in)	0.026 @ 4' 1 11/16"	0.397	Passed (L/999+)	--	1.0 D + 1.0 S (All Spans)
Total Load Defl. (in)	0.086 @ 4' 1 11/16"	0.530	Passed (L/999+)	--	1.0 D + 1.0 S (All Spans)

- Deflection criteria: LL (L/240) and TL (L/180).
- Allowed moment does not reflect the adjustment for the beam stability factor.
- Applicable calculations are based on NDS.

Member Length : 8' 5 3/8"
System : Roof
Member Type : Flush Beam
Building Use : Residential
Building Code : IBC 2021
Design Methodology : ASD
Member Pitch : 0/12

Supports	Bearing Length			Loads to Supports (lbs)				Accessories
	Total	Available	Required	Dead	Floor Live	Snow	Factored	
1 - Stud wall - HF	3.50"	3.50"	1.86"	792	110	339	1130	Blocking
2 - Beam - GLB	5.50"	5.50"	1.50"	824	115	352	1176	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)	5' 10" o/c	
Bottom Edge (Lu)	8' 5" o/c	

•Maximum allowable bracing intervals based on applied load.

Vertical Loads	Location (Side)	Tributary Width	Dead (0.90)	Floor Live (1.00)	Snow (1.15)	Comments
0 - Self Weight (PLF)	0 to 8' 5 3/8"	N/A	4.3	--	--	
1 - Uniform (PSF)	0 to 8' 5 3/8" (Front)	8"	30.0	40.0	--	Floor
2 - Uniform (PSF)	0 to 8' 5 3/8" (Front)	9 1/4"	18.0	--	25.0	Lower Roof
3 - Uniform (PLF)	0 to 8' 5 3/8" (Top)	N/A	108.0	--	--	Wall
4 - Uniform (PSF)	0 to 8' 5 3/8" (Front)	2' 6"	18.0	--	25.0	Upper Roof

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

Weyerhaeuser Notes

ForteWEB Software Operator	Job Notes
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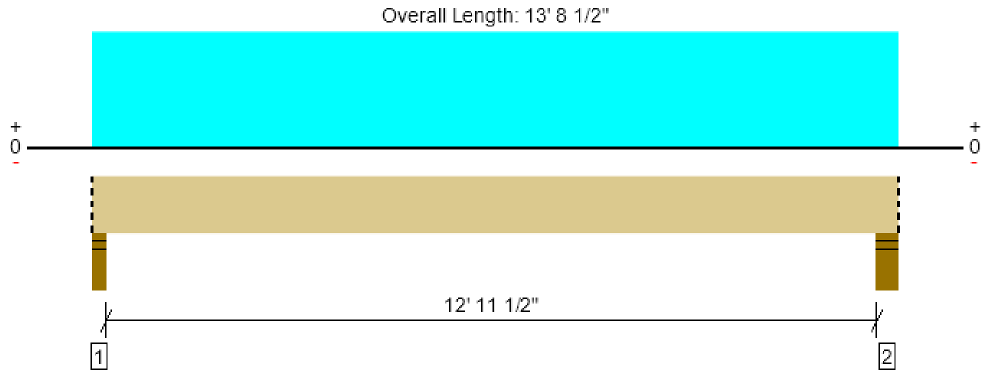
Level 2 and Lower Roof, Grid 8 (E.4-F) Floor Beam

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)	1887 @ 2"	4961 (3.50")	Passed (38%)	--	1.0 D + 1.0 S (All Spans)
Shear (lbs)	1545 @ 1' 2 3/4"	5434	Passed (28%)	1.15	1.0 D + 1.0 S (All Spans)
Moment (Ft-lbs)	6078 @ 6' 9 1/4"	7004	Passed (87%)	1.15	1.0 D + 1.0 S (All Spans)
Live Load Defl. (in)	0.084 @ 6' 9 1/4"	0.660	Passed (L/999+)	--	1.0 D + 1.0 S (All Spans)
Total Load Defl. (in)	0.287 @ 6' 9 1/4"	0.881	Passed (L/552)	--	1.0 D + 1.0 S (All Spans)

Member Length : 13' 8 1/2"
System : Roof
Member Type : Flush Beam
Building Use : Residential
Building Code : IBC 2021
Design Methodology : ASD
Member Pitch : 0/12

- Deflection criteria: LL (L/240) and TL (L/180).
- Allowed moment does not reflect the adjustment for the beam stability factor.
- Applicable calculations are based on NDS.

Supports	Bearing Length			Loads to Supports (lbs)				Accessories
	Total	Available	Required	Dead	Floor Live	Snow	Factored	
1 - Stud wall - HF	3.50"	3.50"	1.50"	1333	181	554	1887	Blocking
2 - Stud wall - HF	5.50"	5.50"	1.50"	1366	185	567	1934	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)	13' 9" o/c	
Bottom Edge (Lu)	13' 9" o/c	

•Maximum allowable bracing intervals based on applied load.

Vertical Loads	Location (Side)	Tributary Width	Dead (0.90)	Floor Live (1.00)	Snow (1.15)	Comments
0 - Self Weight (PLF)	0 to 13' 8 1/2"	N/A	10.0	--	--	
1 - Uniform (PSF)	0 to 13' 8 1/2" (Front)	8"	30.0	40.0	--	Floor
2 - Uniform (PSF)	0 to 13' 8 1/2" (Front)	9 1/4"	18.0	--	25.0	Lower Roof
3 - Uniform (PLF)	0 to 13' 8 1/2" (Top)	N/A	108.0	--	--	Wall
4 - Uniform (PSF)	0 to 13' 8 1/2" (Front)	2' 6"	18.0	--	25.0	Upper Roof

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

Weyerhaeuser Notes

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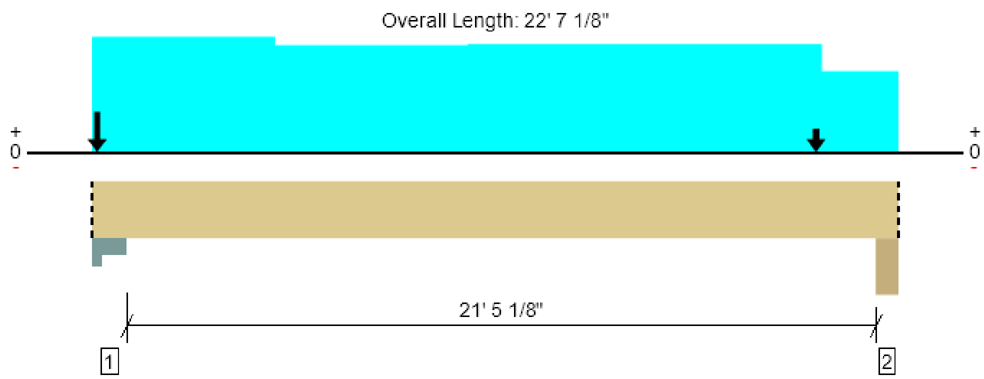
Level 2 and Lower Roof, Grid E (5.5-9) Floor Beam

1 piece(s) 5 1/2" x 19 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)	13224 @ 22' 3 1/8"	19663 (5.50")	Passed (67%)	--	1.0 D + 1.0 S (All Spans)
Shear (lbs)	11379 @ 20' 6 1/8"	21790	Passed (52%)	1.15	1.0 D + 0.75 L + 0.75 S (All Spans)
Pos Moment (Ft-lbs)	67608 @ 11' 5 15/16"	75592	Passed (89%)	1.15	1.0 D + 0.75 L + 0.75 S (All Spans)
Live Load Defl. (in)	0.374 @ 11' 5 3/16"	0.723	Passed (L/696)	--	1.0 D + 0.75 L + 0.75 S (All Spans)
Total Load Defl. (in)	0.938 @ 11' 5 5/16"	1.084	Passed (L/277)	--	1.0 D + 0.75 L + 0.75 S (All Spans)

Member Length : 22' 7 1/8"
System : Floor
Member Type : Drop Beam
Building Use : Residential
Building Code : IBC 2021
Design Methodology : ASD

- Deflection criteria: LL (L/360) and TL (L/240).
- Allowed moment does not reflect the adjustment for the beam stability factor.
- Volume factor of 0.94 was calculated for positive bending using length L = 21' 8 1/8".
- The effects of positive or negative camber have not been accounted for when calculating deflection.
- The specified glulam is assumed to have its strong laminations at the bottom of the beam. Install with proper side up as indicated by the manufacturer.
- Applicable calculations are based on NDS.

Supports	Bearing Length			Loads to Supports (lbs)				Accessories
	Total	Available	Required	Dead	Floor Live	Snow	Factored	
1 - Column Cap - steel	8.50"	8.50"	4.89"	10422	2921	6497	17485	Blocking
2 - Column - DF	5.50"	5.50"	3.70"	8075	1684	5149	13224	Blocking

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

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	20' o/c	
Bottom Edge (Lu)	22' 7" o/c	

•Maximum allowable bracing intervals based on applied load.

Vertical Loads	Location (Side)	Tributary Width	Dead (0.90)	Floor Live (1.00)	Snow (1.15)	Comments
0 - Self Weight (PLF)	0 to 22' 7 1/8"	N/A	26.1	--	--	
1 - Uniform (PSF)	0 to 5' 1 1/2" (Front)	5' 5 1/2"	30.0	40.0	--	Floor
2 - Uniform (PSF)	5' 1 1/2" to 10' 6 1/4" (Front)	4' 1"	30.0	40.0	--	Floor
3 - Uniform (PSF)	10' 6 1/4" to 20' 5 1/4" (Front)	4' 4"	30.0	40.0	--	Floor
4 - Uniform (PSF)	0 to 22' 7 1/8" (Front)	1'	24.0	--	25.0	Lower Roof
5 - Uniform (PSF)	0 to 22' 7 1/8" (Top)	16' 3 1/2"	24.0	--	25.0	Upper Roof
6 - Uniform (PLF)	0 to 22' 7 1/8" (Top)	N/A	108.0	--	--	Wall
7 - Point (lb)	1 3/4" (Top)	N/A	2463	771	1527	Linked from: Grid E (5-5.5) Floor Beam, Support 2
8 - Point (lb)	20' 3 3/8" (Top)	N/A	824	115	352	Linked from: Grid 8 (E-E.4) Floor Beam, Support 2

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

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

Level 2 and Lower Roof, Grid 5.5E Post

1 piece(s) 6 x 8 DF No.2

Post Height: 10' 6"

Drawing is Conceptual

Design Results	Actual	Allowed	Result	LDF	Load: Combination
Slenderness	23	50	Passed (46%)	--	--
Compression (lbs)	17486	21893	Passed (80%)	1.15	1.0 D + 0.75 L + 0.75 S
Base Bearing (lbs)	17486	1225125	Passed (1%)	--	1.0 D + 0.75 L + 0.75 S
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.

Supports	Type	Material
Base	Plate	Steel

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

Max Unbraced Length	Comments
Full Member Length	No bracing assumed.

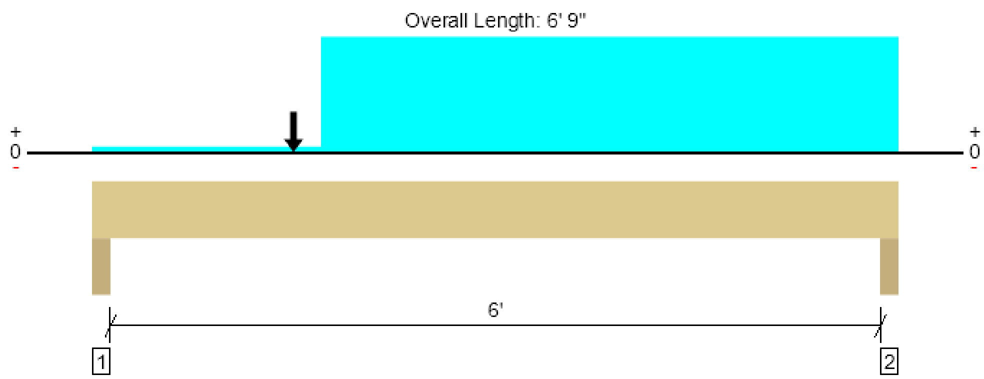
Vertical Load	Dead (0.90)	Floor Live (1.00)	Snow (1.15)	Comments
1 - Point (lb)	10422	2921	6497	Linked from: Grid E (5.5-9) Floor Beam, Support 1

Weyerhaeuser Notes

PASSED

Level 2 and Lower Roof, Grid 9 (D.2-E.2) 6' Window Header

1 piece(s) 5 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)	11719 @ 3"	16088 (4.50")	Passed (73%)	--	1.0 D + 1.0 S (All Spans)
Shear (lbs)	11652 @ 1' 3"	11733	Passed (99%)	1.15	1.0 D + 1.0 S (All Spans)
Pos Moment (Ft-lbs)	16771 @ 1' 8 1/4"	23244	Passed (72%)	1.15	1.0 D + 1.0 S (All Spans)
Live Load Defl. (in)	0.044 @ 3' 1 7/8"	0.208	Passed (L/999+)	--	1.0 D + 1.0 S (All Spans)
Total Load Defl. (in)	0.105 @ 3' 1 5/8"	0.313	Passed (L/713)	--	1.0 D + 1.0 S (All Spans)

Member Length : 6' 9"
System : Wall
Member Type : Header
Building Use : Residential
Building Code : IBC 2021
Design Methodology : ASD

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

Supports	Bearing Length			Loads to Supports (lbs)				Accessories
	Total	Available	Required	Dead	Floor Live	Snow	Factored	
1 - Trimmer - HF	4.50"	4.50"	3.28"	6995	1297	4724	11719	None
2 - Trimmer - HF	4.50"	4.50"	1.61"	3216	387	2548	5765	None

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

•Maximum allowable bracing intervals based on applied load.

Vertical Loads	Location	Tributary Width	Dead (0.90)	Floor Live (1.00)	Snow (1.15)	Comments
0 - Self Weight (PLF)	0 to 6' 9"	N/A	14.0	--	--	
1 - Uniform (PSF)	0 to 1' 11"	9 3/4"	24.0	--	25.0	Lower Roof
2 - Uniform (PSF)	1' 11" to 6' 9"	17' 3"	24.0	--	25.0	Lower Roof
3 - Point (lb)	1' 8 1/4"	N/A	8075	1684	5149	Linked from: Grid E (5.5-9) Floor Beam, Support 2

Weyerhaeuser Notes

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Weyerhaeuser

Level 2 and Lower Roof, Grid 9E Post

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

Post Height: 10' 6"

Design Results	Actual	Allowed	Result	LDF	Load: Combination
Slenderness	23	50	Passed (46%)	--	--
Compression (lbs)	13224	16055	Passed (82%)	1.15	1.0 D + 1.0 S
Base Bearing (lbs)	13224	898425	Passed (1%)	--	1.0 D + 1.0 S
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.

Drawing is Conceptual

Supports	Type	Material
Base	Plate	Steel

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

Max Unbraced Length	Comments
Full Member Length	No bracing assumed.

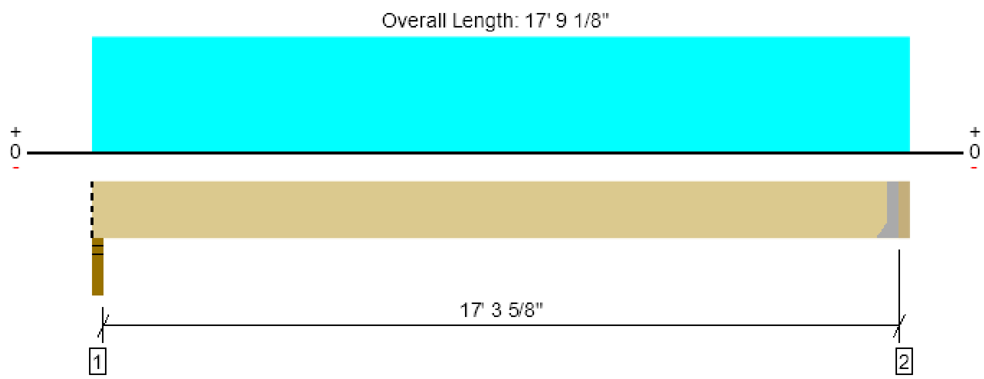
Vertical Load	Dead (0.90)	Floor Live (1.00)	Snow (1.15)	Comments
1 - Point (lb)	8075	1684	5149	Linked from: Grid E (5.5-9) Floor Beam, Support 2

Weyerhaeuser Notes

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

Level 2 and Lower Roof, Grid I - Corbal Roof Beam

1 piece(s) 5 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)	1690 @ 17' 6 3/8"	5363 (1.50")	Passed (32%)	--	1.0 D + 1.0 S (All Spans)
Shear (lbs)	1568 @ 16' 10 7/8"	8381	Passed (19%)	1.15	1.0 D + 1.0 S (All Spans)
Pos Moment (Ft-lbs)	7362 @ 8' 9 13/16"	11859	Passed (62%)	1.15	1.0 D + 1.0 S (All Spans)
Live Load Defl. (in)	0.559 @ 8' 9 13/16"	0.871	Passed (L/374)	--	1.0 D + 1.0 S (All Spans)
Total Load Defl. (in)	1.156 @ 8' 9 13/16"	1.162	Passed (L/181)	--	1.0 D + 1.0 S (All Spans)

Member Length : 17' 6 3/8"
System : Roof
Member Type : Drop Beam
Building Use : Residential
Building Code : IBC 2021
Design Methodology : ASD
Member Pitch : 0/12

- 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 = 17' 5 1/8".
- The effects of positive or negative camber have not been accounted for when calculating deflection.
- The specified glulam is assumed to have its strong laminations at the bottom of the beam. Install with proper side up as indicated by the manufacturer.
- Applicable calculations are based on NDS.

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Snow	Factored	
1 - Stud wall - HF	2.75"	2.75"	1.50"	883	827	1710	Blocking
2 - Hanger on 7 1/2" HF beam	2.75"	Hanger[1]	1.50"	893	838	1732	See note [1]

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

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	17' 6" o/c	
Bottom Edge (Lu)	17' 6" o/c	

- Maximum allowable bracing intervals based on applied load.

Connector: Simpson Strong-Tie

Support	Model	Seat Length	Top Fasteners	Face Fasteners	Member Fasteners	Accessories
2 - Face Mount Hanger	HU68	2.50"	N/A	14-10d	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)	Snow (1.15)	Comments
0 - Self Weight (PLF)	0 to 17' 6 3/8"	N/A	10.0	--	
1 - Uniform (PSF)	0 to 17' 9 1/8" (Front)	3' 9"	24.0	25.0	Default Load

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

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Weyerhaeuser

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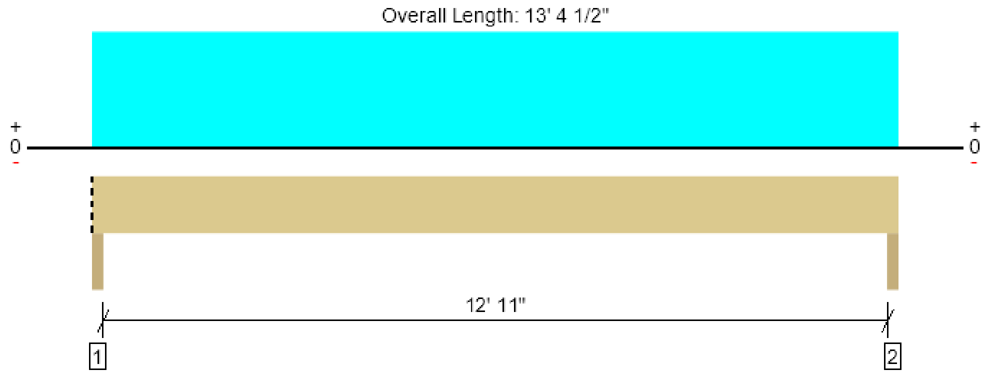
The product application, input design loads, dimensions and support information have been provided by ForteWEB Software Operator

MEMBER REPORT

PASSED

Level 2 and Lower Roof, Grid E - Corbal Roof Beam

1 piece(s) 5 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)	1297 @ 1 1/4"	6126 (2.75")	Passed (21%)	--	1.0 D + 1.0 S (All Spans)
Shear (lbs)	1131 @ 10 1/4"	8381	Passed (13%)	1.15	1.0 D + 1.0 S (All Spans)
Pos Moment (Ft-lbs)	4202 @ 6' 8 1/4"	11859	Passed (35%)	1.15	1.0 D + 1.0 S (All Spans)
Live Load Defl. (in)	0.182 @ 6' 8 1/4"	0.658	Passed (L/867)	--	1.0 D + 1.0 S (All Spans)
Total Load Defl. (in)	0.377 @ 6' 8 1/4"	0.878	Passed (L/419)	--	1.0 D + 1.0 S (All Spans)

Member Length : 13' 4 1/2"
System : Roof
Member Type : Drop Beam
Building Use : Residential
Building Code : IBC 2021
Design Methodology : ASD
Member Pitch : 0/12

- 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 = 13' 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	Snow	Factored	
1 - Beam - HF	2.75"	2.75"	1.50"	670	627	1297	Blocking
2 - Beam - HF	2.75"	2.75"	1.50"	670	627	1297	None

• 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)	13' 5" o/c	
Bottom Edge (Lu)	13' 5" 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 13' 4 1/2"	N/A	10.0	--	
1 - Uniform (PSF)	0 to 13' 4 1/2" (Front)	3' 9"	24.0	25.0	Default Load

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

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ForteWEB Software Operator	Job Notes
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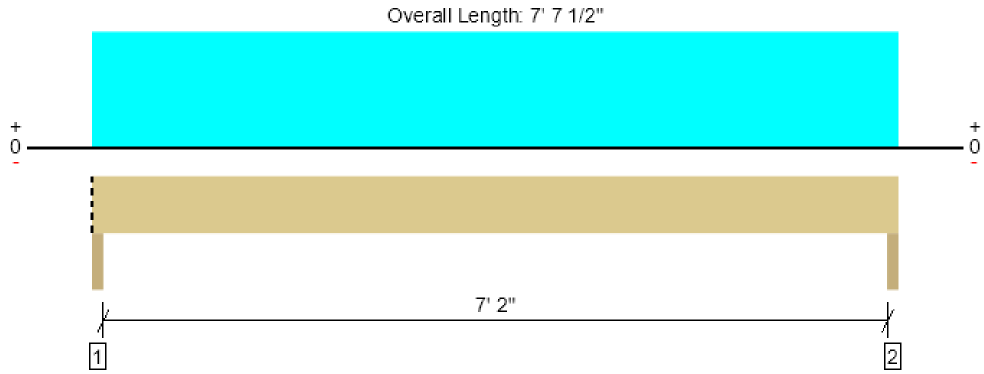
Level 2 and Lower Roof, Grid 5 - Corbal Roof Beam

1 piece(s) 6 x 8 DF No.2



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Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	741 @ 1 1/4"	6126 (2.75")	Passed (12%)	--	1.0 D + 1.0 S (All Spans)
Shear (lbs)	575 @ 10 1/4"	5376	Passed (11%)	1.15	1.0 D + 1.0 S (All Spans)
Moment (Ft-lbs)	1336 @ 3' 9 3/4"	3706	Passed (36%)	1.15	1.0 D + 1.0 S (All Spans)
Live Load Defl. (in)	0.025 @ 3' 9 3/4"	0.371	Passed (L/999+)	--	1.0 D + 1.0 S (All Spans)
Total Load Defl. (in)	0.053 @ 3' 9 3/4"	0.494	Passed (L/999+)	--	1.0 D + 1.0 S (All Spans)

Member Length : 7' 7 1/2"
System : Roof
Member Type : Drop Beam
Building Use : Residential
Building Code : IBC 2021
Design Methodology : ASD
Member Pitch : 0/12

- Deflection criteria: LL (L/240) and TL (L/180).
- Allowed moment does not reflect the adjustment for the beam stability factor.
- Applicable calculations are based on NDS.

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Snow	Factored	
1 - Beam - HF	2.75"	2.75"	1.50"	383	357	741	Blocking
2 - Beam - HF	2.75"	2.75"	1.50"	383	357	741	None

• 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' 8" o/c	
Bottom Edge (Lu)	7' 8" 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' 7 1/2"	N/A	10.4	--	
1 - Uniform (PSF)	0 to 7' 7 1/2" (Front)	3' 9"	24.0	25.0	Default Load

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

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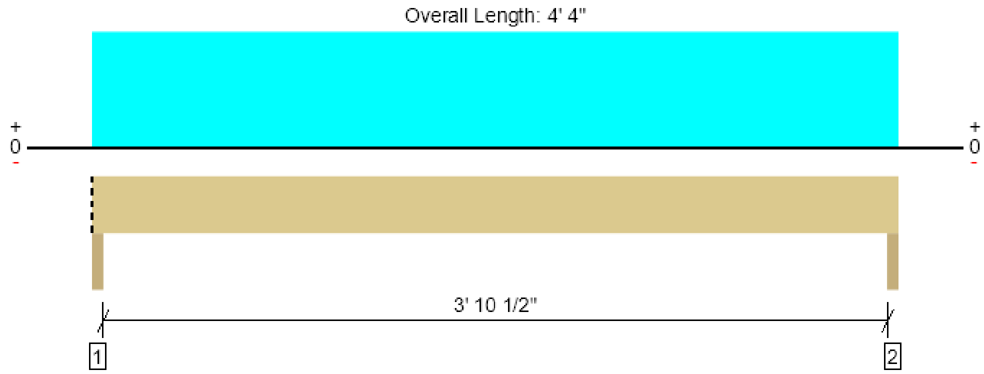
MEMBER REPORT

PASSED

Level 2 and Lower Roof, Grid 1 - Corbal Roof Beam

1 piece(s) 6 x 8 DF No.2



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Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	421 @ 1 1/4"	6126 (2.75")	Passed (7%)	--	1.0 D + 1.0 S (All Spans)
Shear (lbs)	255 @ 10 1/4"	5376	Passed (5%)	1.15	1.0 D + 1.0 S (All Spans)
Moment (Ft-lbs)	413 @ 2' 2"	3706	Passed (11%)	1.15	1.0 D + 1.0 S (All Spans)
Live Load Defl. (in)	0.002 @ 2' 2"	0.206	Passed (L/999+)	--	1.0 D + 1.0 S (All Spans)
Total Load Defl. (in)	0.005 @ 2' 2"	0.275	Passed (L/999+)	--	1.0 D + 1.0 S (All Spans)

Member Length : 4' 4"
System : Roof
Member Type : Drop Beam
Building Use : Residential
Building Code : IBC 2021
Design Methodology : ASD
Member Pitch : 0/12

- Deflection criteria: LL (L/240) and TL (L/180).
- Allowed moment does not reflect the adjustment for the beam stability factor.
- Applicable calculations are based on NDS.

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Snow	Factored	
1 - Beam - HF	2.75"	2.75"	1.50"	218	203	421	Blocking
2 - Beam - HF	2.75"	2.75"	1.50"	218	203	421	None

• 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)	4' 4" o/c	
Bottom Edge (Lu)	4' 4" 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 4' 4"	N/A	10.4	--	
1 - Uniform (PSF)	0 to 4' 4" (Front)	3' 9"	24.0	25.0	Default Load

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

Weyerhaeuser Notes

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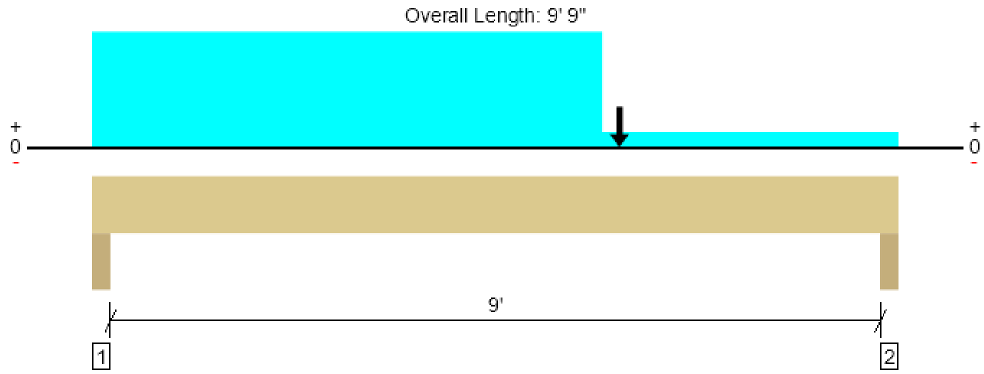
7/7/2025 3:30:00 PM UTC
ForteWEB v3.9, Engine: V8.4.3.94, Data: V8.1.7.3
File Name: East Town Crossing - Clubhouse

Upper Roof, Grid 7 (F.5-G.6) 9' Window Header

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)	11059 @ 9' 6"	16088 (4.50")	Passed (69%)	--	1.0 D + 1.0 S (All Spans)
Shear (lbs)	10848 @ 8' 3"	15085	Passed (72%)	1.15	1.0 D + 1.0 S (All Spans)
Pos Moment (Ft-lbs)	33762 @ 6' 4 1/2"	38424	Passed (88%)	1.15	1.0 D + 1.0 S (All Spans)
Live Load Defl. (in)	0.110 @ 5' 1 1/4"	0.308	Passed (L/999+)	--	1.0 D + 1.0 S (All Spans)
Total Load Defl. (in)	0.220 @ 5' 1 1/4"	0.463	Passed (L/504)	--	1.0 D + 1.0 S (All Spans)

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

- Deflection criteria: LL (L/360) and TL (L/240).
- Allowed moment does not reflect the adjustment for the beam stability factor.
- Volume factor of 1.00 was calculated for positive bending using length L = 9' 3".
- The effects of positive or negative camber have not been accounted for when calculating deflection.
- The specified glulam is assumed to have its strong laminations at the bottom of the beam. Install with proper side up as indicated by the manufacturer.
- Applicable calculations are based on NDS.

	Bearing Length			Loads to Supports (lbs)			
Supports	Total	Available	Required	Dead	Snow	Factored	Accessories
1 - Trimmer - HF	4.50"	4.50"	2.41"	4306	4325	8632	None
2 - Trimmer - HF	4.50"	4.50"	3.09"	5526	5533	11059	None

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	9' 9" o/c	
Bottom Edge (Lu)	9' 9" 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 9' 9"	N/A	18.0	--	
1 - Uniform (PSF)	0 to 6' 2"	18' 10 1/4"	24.0	25.0	Default Load
2 - Uniform (PSF)	6' 2" to 9' 9"	2' 6"	24.0	25.0	Default Load
3 - Point (lb)	6' 4 1/2"	N/A	6646	6728	Linked from: Grid G (3-7) GT (for loads only), Support 2

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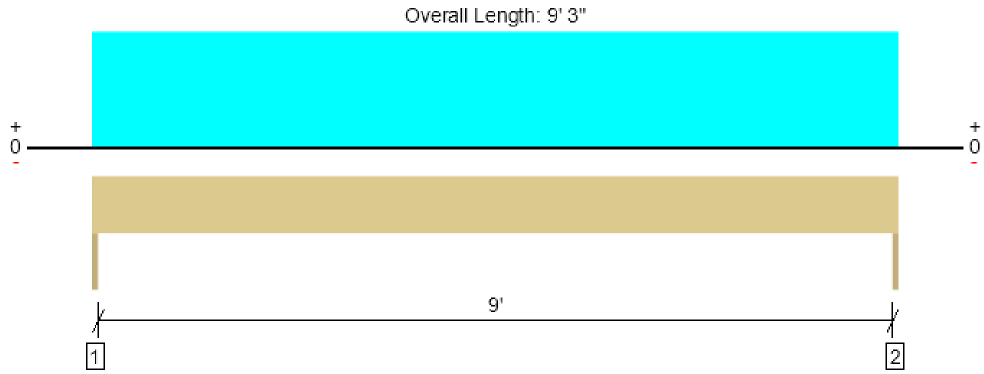
Upper Roof, Grid H - 9' Window Header

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)	880 @ 0	3281 (1.50")	Passed (27%)	--	1.0 D + 1.0 S (All Spans)
Shear (lbs)	741 @ 8 3/4"	3502	Passed (21%)	1.15	1.0 D + 1.0 S (All Spans)
Moment (Ft-lbs)	2036 @ 4' 7 1/2"	3438	Passed (59%)	1.15	1.0 D + 1.0 S (All Spans)
Live Load Defl. (in)	0.087 @ 4' 7 1/2"	0.308	Passed (L/999+)	--	1.0 D + 1.0 S (All Spans)
Total Load Defl. (in)	0.176 @ 4' 7 1/2"	0.313	Passed (L/630)	--	1.0 D + 1.0 S (All Spans)

Member Length : 9' 3"
System : Wall
Member Type : Header
Building Use : Residential
Building Code : IBC 2021
Design Methodology : ASD

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

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Snow	Factored	
1 - Trimmer - HF	1.50"	1.50"	1.50"	447	434	880	None
2 - Trimmer - HF	1.50"	1.50"	1.50"	447	434	880	None

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	9' 3" o/c	
Bottom Edge (Lu)	9' 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 9' 3"	N/A	6.4	--	
1 - Uniform (PSF)	0 to 9' 3"	3' 9"	24.0	25.0	Default Load

Weyerhaeuser Notes

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

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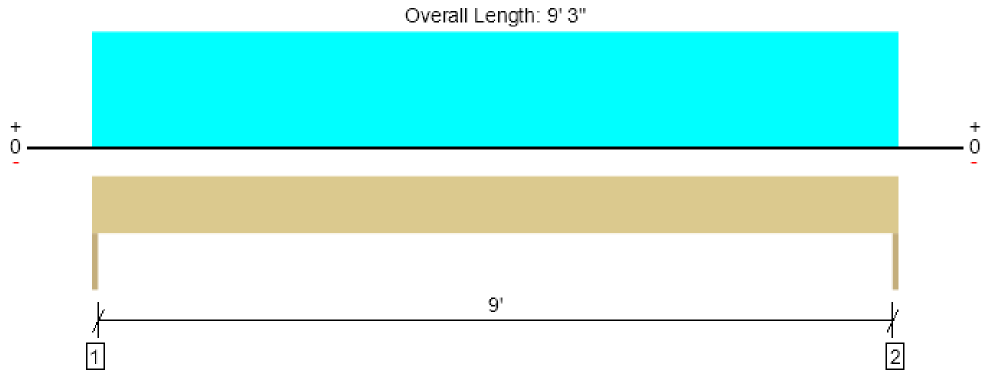
7/7/2025 3:30:00 PM UTC
ForteWEB v3.9, Engine: V8.4.3.94, Data: V8.1.7.3
File Name: East Town Crossing - Clubhouse

Upper Roof, Grid 2 - 9' Window Header

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



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Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	880 @ 0	3281 (1.50")	Passed (27%)	--	1.0 D + 1.0 S (All Spans)
Shear (lbs)	741 @ 8 3/4"	3502	Passed (21%)	1.15	1.0 D + 1.0 S (All Spans)
Moment (Ft-lbs)	2036 @ 4' 7 1/2"	3438	Passed (59%)	1.15	1.0 D + 1.0 S (All Spans)
Live Load Defl. (in)	0.087 @ 4' 7 1/2"	0.308	Passed (L/999+)	--	1.0 D + 1.0 S (All Spans)
Total Load Defl. (in)	0.176 @ 4' 7 1/2"	0.313	Passed (L/630)	--	1.0 D + 1.0 S (All Spans)

Member Length : 9' 3"
System : Wall
Member Type : Header
Building Use : Residential
Building Code : IBC 2021
Design Methodology : ASD

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

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Snow	Factored	
1 - Trimmer - HF	1.50"	1.50"	1.50"	447	434	880	None
2 - Trimmer - HF	1.50"	1.50"	1.50"	447	434	880	None

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	9' 3" o/c	
Bottom Edge (Lu)	9' 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 9' 3"	N/A	6.4	--	
1 - Uniform (PSF)	0 to 9' 3"	3' 9"	24.0	25.0	Default Load

Weyerhaeuser Notes

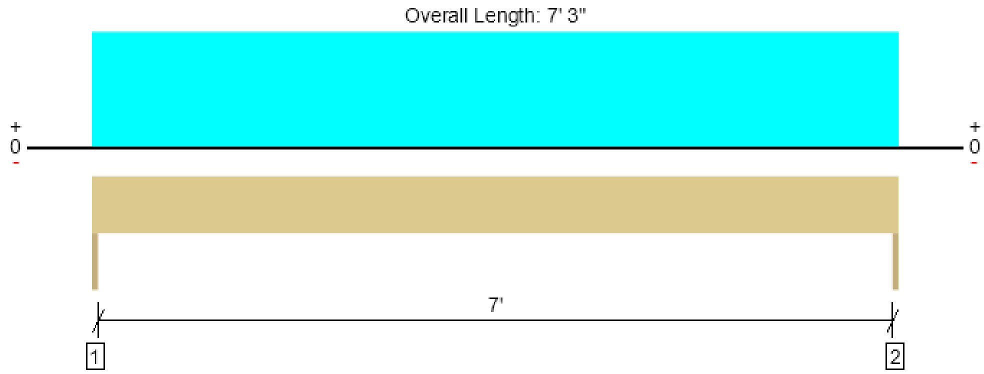
ForteWEB Software Operator	Job Notes
Chon Pieruccioni Pieruccioni Engineering (206) 949-7866 cpieru@hotmail.com	

MEMBER REPORT

PASSED

Upper Roof, Grid 2 - 7' Window Header

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



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Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	690 @ 0	3281 (1.50")	Passed (21%)	--	1.0 D + 1.0 S (All Spans)
Shear (lbs)	551 @ 8 3/4"	3502	Passed (16%)	1.15	1.0 D + 1.0 S (All Spans)
Moment (Ft-lbs)	1250 @ 3' 7 1/2"	3438	Passed (36%)	1.15	1.0 D + 1.0 S (All Spans)
Live Load Defl. (in)	0.033 @ 3' 7 1/2"	0.242	Passed (L/999+)	--	1.0 D + 1.0 S (All Spans)
Total Load Defl. (in)	0.067 @ 3' 7 1/2"	0.313	Passed (L/999+)	--	1.0 D + 1.0 S (All Spans)

Member Length : 7' 3"
System : Wall
Member Type : Header
Building Use : Residential
Building Code : IBC 2021
Design Methodology : ASD

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

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Snow	Factored	
1 - Trimmer - HF	1.50"	1.50"	1.50"	350	340	690	None
2 - Trimmer - HF	1.50"	1.50"	1.50"	350	340	690	None

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.

Vertical Loads	Location	Tributary Width	Dead (0.90)	Snow (1.15)	Comments
0 - Self Weight (PLF)	0 to 7' 3"	N/A	6.4	--	
1 - Uniform (PSF)	0 to 7' 3"	3' 9"	24.0	25.0	Default Load

Weyerhaeuser Notes

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

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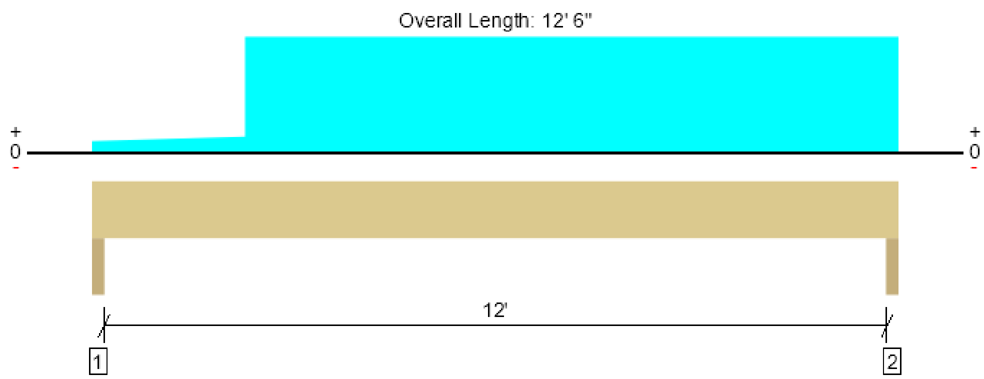
ForteWEB v3.9, Engine: V8.4.3.94, Data: V8.1.7.3
File Name: East Town Crossing - Clubhouse

MEMBER REPORT

PASSED

Upper Roof, Grid E - 12' Header

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



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Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	4873 @ 12' 4 1/2"	6825 (3.00")	Passed (71%)	--	1.0 D + 1.0 S (All Spans)
Shear (lbs)	3870 @ 11' 3"	8533	Passed (45%)	1.15	1.0 D + 1.0 S (All Spans)
Pos Moment (Ft-lbs)	14186 @ 6' 5 3/16"	19320	Passed (73%)	1.15	1.0 D + 1.0 S (All Spans)
Live Load Defl. (in)	0.212 @ 6' 3 5/8"	0.408	Passed (L/695)	--	1.0 D + 1.0 S (All Spans)
Total Load Defl. (in)	0.417 @ 6' 3 3/4"	0.613	Passed (L/352)	--	1.0 D + 1.0 S (All Spans)

Member Length : 12' 6"
System : Wall
Member Type : Header
Building Use : Residential
Building Code : IBC 2021
Design Methodology : ASD

- Deflection criteria: LL (L/360) and TL (L/240).
- Allowed moment does not reflect the adjustment for the beam stability factor.
- Volume factor of 1.00 was calculated for positive bending using length L = 12' 3".
- The effects of positive or negative camber have not been accounted for when calculating deflection.
- The specified glulam is assumed to have its strong laminations at the bottom of the beam. Install with proper side up as indicated by the manufacturer.
- Applicable calculations are based on NDS.

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Snow	Factored	
1 - Trimmer - HF	3.00"	3.00"	1.53"	1660	1823	3482	None
2 - Trimmer - HF	3.00"	3.00"	2.14"	2413	2460	4873	None

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	12' 6" o/c	
Bottom Edge (Lu)	12' 6" o/c	

•Maximum allowable bracing intervals based on applied load.

Vertical Loads	Location	Tributary Width	Dead (0.90)	Snow (1.15)	Comments
0 - Self Weight (PLF)	0 to 12' 6"	N/A	10.2	--	
1 - Tapered (PSF)	0 to 2' 4 1/2"	2' 8" to 3' 9"	1.4	25.0	Default Load
2 - Uniform (PSF)	2' 4 1/2" to 12' 6"	16' 2"	24.0	25.0	Default Load

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ASDIP Foundation 5.6.0.0 **SPREAD FOOTING DESIGN** www.asdipsoft.com

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

SOIL PRESSURES (D+0.7S)			
Gross Allow. Soil Pressure	2.0	ksf	
Soil Pressure at Corner 1	1.9	ksf	
Soil Pressure at Corner 2	1.9	ksf	
Soil Pressure at Corner 3	1.9	ksf	
Soil Pressure at Corner 4	1.9	ksf	
Bearing Pressure Ratio	0.94		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	6.7	0.0	0.0	6.7	0.0	0.0	kip
Moment about X Mx ..	0.0	0.0	0.0	0.0	0.0	0.0	k-ft
Moment about Z Mz ..	0.0	0.0	0.0	0.0	0.0	0.0	k-ft
Shear Force Vx	0.0	0.0	0.0	0.0	0.0	0.0	kip
Shear Force Vz	0.0	0.0	0.0	0.0	0.0	0.0	kip

OVERTURNING CALCULATIONS (Comb: 0.6D+0.6W)

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

- Resisting about X-X
- Footing weight = *0.6 * W * L * Thick * Density =* 0.6 * 2.50 * 2.50 * 8.0 / 12 * 0.15 = 0.4 kip
  Arm = *W / 2 =* 2.50 / 2 = 1.25 ft   Moment = 0.4 * 1.25 = 0.5 k-ft
- Pedestal weight = *0.6 * W * L * H * Density =* 0.6 * 6.0 / 12 * 6.0 / 12 * 0.0 * 0.15 = 0.0 kip
  Arm = *W / 2 - Offset =* 2.50 / 2 - 0.0 / 12 = 1.25 ft   Moment = 0.0 * 1.25 = 0.0 k-ft
- Soil cover = *0.6 * W * L * SC * Density =* 0.6 * (2.50 * 2.50 - 6.0 / 12 * 6.0 / 12) * 0.0 * 110 = 0.0 kip
  Arm = *W / 2 =* 2.50 / 2 = 1.25 ft   Moment = 0.0 * 1.25 = 0.0 k-ft
- Buoyancy = *0.6 * W * L * γ * (SC + Thick - WT) =* 0.6 * 2.50 * 2.50 * 62 * (0.67) = -0.2 kip
  Arm = *W / 2 =* 2.50 / 2 = 1.25 ft   Moment = 0.2 * 1.25 = -0.2 k-ft
- Axial force P = 0.6 * 6.7 + 0.6 * 0.0 = 4.0 kip
  Arm = *W / 2 - Offset =* 2.50 / 2 - 0.0 / 12 = 1.25 ft   Moment = 4.0 * 1.25 = 5.0 k-ft
- Resisting moment X-X = 0.5 + 0.0 + 0.0 + 5.0 + -0.2 = 5.3 k-ft

- Overturning safety factor X-X = $\frac{\textit{Resisting moment}}{\textit{Overturning moment}} = \frac{5.3}{0.0}$ = 52.99 > 1.50 OK

- Overturning about Z-Z

- Moment Mz = 0.6 * 0.0 + 0.6 * 0.0 = 0.0 k-ft
- Shear Force Vx = 0.6 * 0.0 + 0.6 * 0.0 = 0.0 kip
  Arm = 0.00 + 8.0 / 12 = 0.67 ft Moment = 0.0 * 0.67 = 0.0 k-ft
- Passive Force = 0.0 kip Arm = 0.27 ft Moment = 0.0 k-ft
- Overturning moment Z-Z = 0.0 + 0.0 = 0.0 k-ft

- Resisting about Z-Z

- Footing weight = *0.6 * W * L * Thick * Density =* 0.6 * 2.50 * 2.50 * 8.0 / 12 * 0.15 = 0.4 kip
  Arm = *L / 2 =* 2.50 / 2 = 1.25 ft Moment = 0.4 * 1.25 = 0.5 k-ft
- Pedestal weight = *0.6 * W * L * H * Density =* 0.6 * 6.0 / 12 * 6.0 / 12 * 0.0 * 0.15 = 0.0 kip
  Arm = *L / 2 - Offset =* 2.50 / 2 - 0.0 / 12 = 1.25 ft Moment = 0.0 * 1.25 = 0.0 k-ft
- Soil cover = *0.6 * W * L * SC * Density =* 0.6 * (2.50 * 2.50 - 6.0 / 12 * 6.0 / 12) * 0.0 * 110 = 0.0 kip
  Arm = *L / 2 =* 2.50 / 2 = 1.25 ft Moment = 0.0 * 1.25 = 0.0 k-ft
- Buoyancy = *0.6 * W * L * γ * (SC + Thick - WT) =* 0.6 * 2.50 * 2.50 * 62 * (0.67) = -0.2 kip
  Arm = *L / 2 =* 2.50 / 2 = 1.25 ft Moment = 0.2 * 1.25 = -0.2 k-ft
- Axial force P = 0.6 * 6.7 + 0.6 * 0.0 = 4.0 kip
  Arm = *L / 2 - Offset =* 2.50 / 2 - 0.0 / 12 = 1.25 ft Moment = 4.0 * 1.25 = 5.0 k-ft
- Resisting moment Z-Z = 0.5 + 0.0 + 0.0 + 5.0 + -0.2 = 5.3 k-ft
- Overturning safety factor Z-Z = $\dfrac{\text{Resisting moment}}{\text{Overturning moment}} = \dfrac{5.3}{0.0}$ = 52.99 > 1.50 OK

SOIL BEARING PRESSURES (Comb: D+0.7S)

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

Resisting moment X-X = 0.8 + 0.0 + 0.0 + -0.3 + 14.2 = 14.7 k-ft

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

Resisting moment Z-Z = 0.8 + 0.0 + 0.0 + -0.3 + 14.2 = 14.7 k-ft

Resisting force = *Footing + Pedestal + Soil - Buoyancy + P =* 0.6 + 0.0 + 0.0 - 0.3 + 11.4 = 11.8 kip

X-coordinate of resultant from maximum bearing corner:

$$Xp = \frac{\text{Z-Resisting moment - Z-Overturning moment}}{\text{Resisting force}} = \frac{14.7 - 0.0}{11.8} = 1.25 \text{ ft}$$

Z-coordinate of resultant from maximum bearing corner:

$$Zp = \frac{\text{X-Resisting moment - X-Overturning moment}}{\text{Resisting force}} = \frac{14.7 - 0.0}{11.8} = 1.25 \text{ ft}$$

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

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

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

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

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

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

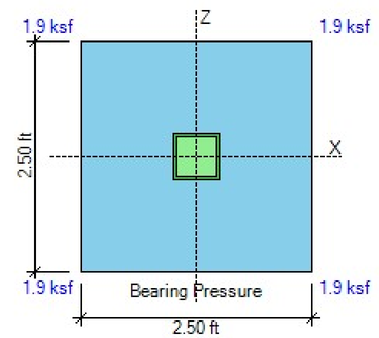
P1 = *P * (1/ A + Z-ecc / Sx + X-ecc / Sz) =* 11.8 * (1 / 6.3 + 0.00 / 2.6 + 0.00 / 2.6) = 1.88 ksf

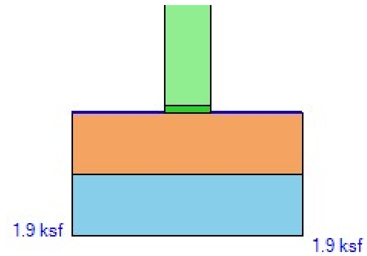
P2 = *P * (1/ A - Z-ecc / Sx + X-ecc / Sz) =* 11.8 * (1 / 6.3 - 0.00 / 2.6 + 0.00 / 2.6) = 1.88 ksf

P3 = *P * (1/ A - Z-ecc / Sx - X-ecc / Sz) =* 11.8 * (1 / 6.3 - 0.00 / 2.6 - 0.00 / 2.6) = 1.88 ksf

P4 = *P * (1/ A + Z-ecc / Sx - X-ecc / Sz) =* 11.8 * (1 / 6.3 + 0.00 / 2.6 - 0.00 / 2.6) = 1.88 ksf

ASDIP Foundation 5.6.0.0 **SPREAD FOOTING DESIGN** *www.asdipsoft.com*





SLIDING CALCULATIONS (Comb: 0.6D+0.6W)

Internal friction angle = 28.0 deg

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

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

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

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

Friction force = *Resisting force * Friction coeff. =* Max (0, 4.2 * 0.35) = 1.5 kip

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

- Sliding safety factor X-X = $\dfrac{\text{X-Passive force + Friction}}{\text{X-Horizontal load}} = \dfrac{1.00 * 0.3 + 1.00 * 1.5}{0.0}$ = 17.48 > 1.50 OK

- Sliding safety factor Z-Z = $\dfrac{\text{Z-Passive force + Friction}}{\text{Z-Horizontal load}} = \dfrac{1.00 * 0.3 + 1.00 * 1.5}{0.0}$ = 17.48 > 1.50 OK

UPLIFT CALCULATIONS (Comb: 0.6D+0.6W)

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

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

Concrete f'c = 2.5 ksi Steel fy = 60.0 ksi Soil density = 110 pcf

Use Plain Concrete Shear Strength

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

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

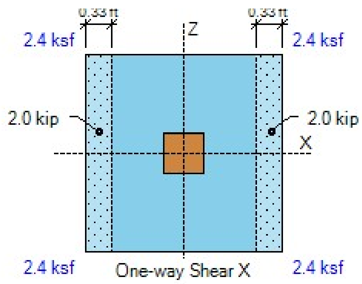
- Shear forces calculated as the volume of the bearing pressures under the effective areas:

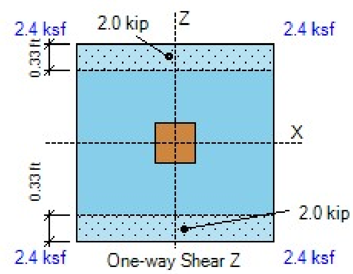
One-way shear Vux (- Side) = 2.0 kip < 9.6 kip OK

One-way shear Vux (+ Side) = 2.0 kip < 9.6 kip OK

One-way shear Vuz (- Side) = 2.0 kip < 9.6 kip OK

One-way shear Vuz (+ Side) = 2.0 kip < 9.6 kip OK





FLEXURE CALCULATIONS (Comb: 1.2D+S+0.5W)

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

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

- Top Bars

No Top Reinforcement Provided at the Footing

Use Plain Concrete Flexural Strength at Top

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

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

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

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

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

- Bottom Bars

No Bottom Reinforcement Provided at the Footing

Use Plain Concrete Flexural Strength at Bottom

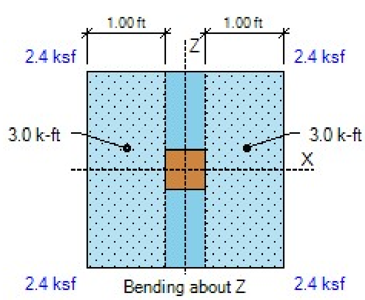
- Bottom moments calculated as the bearing minus the overburden pressures times the lever arm:

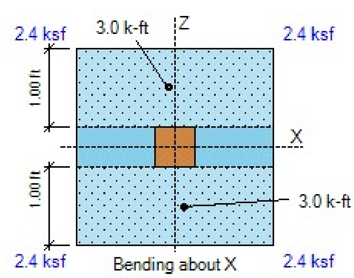
Bottom moment Mux (- Side) = 3.0 k-ft < 4.0 k-ft OK ratio = 0.74

Bottom moment Mux (+ Side) = 3.0 k-ft < 4.0 k-ft OK ratio = 0.74

Bottom moment Muz (- Side) = 3.0 k-ft < 4.0 k-ft OK ratio = 0.74

Bottom moment Muz (+ Side) = 3.0 k-ft < 4.0 k-ft OK ratio = 0.74





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

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

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

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

Bearing *Pbu = P / A1 + Mz / Sx + Mx / Sz =* 14.7 / 36.0 + 0.0 * 12 / 36.0 + 0.0 * 12 / 36.0 = 0.4 ksi

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

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

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

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

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

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

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

Hooked *Ldh = Max (8 db, 6, 1 / 55 * fy / (f'c)½ * Confining * Location * Concrete * db^1.5)* ACI 25.4.3

Ldh = Max (8 db, 6, 1 / 55 * 60.0 * 1000 / (2500)½ * 1.6 * 1.0 * 0.8 * 0.75^1.5) = 17.4 in

Ld provided = *Dowel length =* 3.00 * 12 = 36.0 in > 12.7 in OK

Ldh provided = *Footing thickness - Cover =* 8.00 - 3.0 = 5.0 in < 17.4 in NG

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

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

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

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

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

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

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

Use Plain Concrete Shear Strength

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

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

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

F = 14.7 + 0.07 * 484.0 / 144 - 3.3 = 11.7 kip

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

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

ACI Eq. (8.4.4.2.2)

ACI Eq. (8.4.2.3.2)

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

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

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

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

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

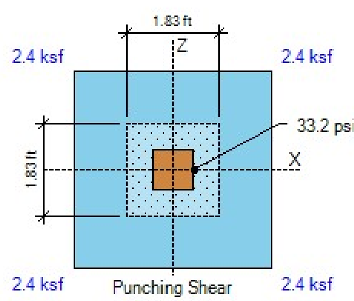
Jcz = 22.0 * 8.0³ / 12 + 22.0³ * 8.0 / 12 + 22.0 * 8.0 * (22.0 / 2 * 5.5)² + 22.0 * 8.0 * 5.5² = 18685 in⁴

Stress due to P = *F / (bo * d) * 1000 =* 11.7 / (44.0 * 8.0) * 1000 = 33.2 psi

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

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

Punching stress = *P-stress + Mx-stress + Mz-stress =* 33.2 + 0.0 + 0.0 = 33.2 psi < 80.0 psi OK



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

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

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

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

Bearing *Pbu = P / A1 + Mz / Sx + Mx / Sz* = 14.7 / 36.0 + 0.0 * 12 / 36.0 + 0.0 * 12 / 36.0 = 0.4 ksi

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

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

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

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

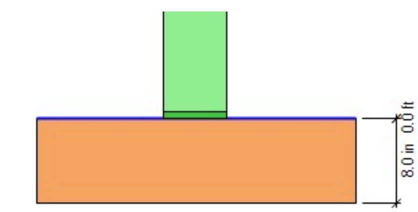
Footing *ϕPnc = ϕ * 0.85 * f'c * Min [2, √(A2 / A1)]* = 0.65 * 0.85 * 2.5 * Min [2, √(900.0 / 36.0)] = 2.8 ksi

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

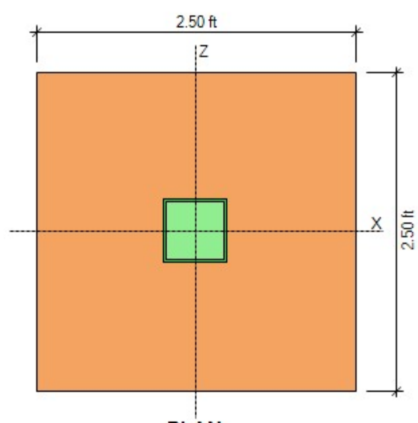
Footing bearing *ϕPn = ϕPnc + ϕPns* = 2.8 + 0.0 = 2.8 ksi > 0.4 psi OK

DESIGN CODES

Concrete Design	ACI 318-19
Load Combinations	ASCE 7-22



ELEVATION



PLAN

ASDIP Foundation 5.6.0.0 **SPREAD FOOTING DESIGN** www.asdipsoft.com

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+0.75L+0.525S)

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	0.92		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	10.4	2.9	0.0	6.5	0.0	0.0	kip
Moment about X Mx ..	0.0	0.0	0.0	0.0	0.0	0.0	k-ft
Moment about Z Mz ..	0.0	0.0	0.0	0.0	0.0	0.0	k-ft
Shear Force Vx	0.0	0.0	0.0	0.0	0.0	0.0	kip
Shear Force Vz	0.0	0.0	0.0	0.0	0.0	0.0	kip

OVERTURNING CALCULATIONS (Comb: 0.6D+0.6W)

- Overturning about X-X

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

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

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

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

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

- Resisting about X-X

- Footing weight = *0.6 * W * L * Thick * Density =* 0.6 * 3.00 * 3.00 * 8.0 / 12 * 0.15 = 0.5 kip

  Arm = *W / 2 =* 3.00 / 2 = 1.50 ft Moment = 0.5 * 1.50 = 0.8 k-ft

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

  Arm = *W / 2 - Offset =* 3.00 / 2 - 0.0 / 12 = 1.50 ft Moment = 0.0 * 1.50 = 0.0 k-ft

- Soil cover = *0.6 * W * L * SC * Density =* 0.6 * (3.00 * 3.00 - 6.0 / 12 * 6.0 / 12) * 0.0 * 110 = 0.0 kip

  Arm = *W / 2 =* 3.00 / 2 = 1.50 ft Moment = 0.0 * 1.50 = 0.0 k-ft

- Buoyancy = *0.6 * W * L * γ * (SC + Thick - WT) =* 0.6 * 3.00 * 3.00 * 62 * (0.67) = -0.2 kip

  Arm = *W / 2 =* 3.00 / 2 = 1.50 ft Moment = 0.2 * 1.50 = -0.3 k-ft

- Axial force P = 0.6 * 10.4 + 0.6 * 0.0 = 6.2 kip

  Arm = *W / 2 - Offset =* 3.00 / 2 - 0.0 / 12 = 1.50 ft Moment = 6.2 * 1.50 = 9.4 k-ft

- Resisting moment X-X = 0.8 + 0.0 + 0.0 + 9.4 + -0.3 = 9.8 k-ft

- Overturning safety factor X-X = $\frac{\textit{Resisting moment}}{\textit{Overturning moment}} = \frac{9.8}{0.0}$ = 98.33 > 1.50 OK

- Overturning about Z-Z

- Moment Mz = 0.6 * 0.0 + 0.6 * 0.0 = 0.0 k-ft
- Shear Force Vx = 0.6 * 0.0 + 0.6 * 0.0 = 0.0 kip
  Arm = 0.00 + 8.0 / 12 = 0.67 ft Moment = 0.0 * 0.67 = 0.0 k-ft
- Passive Force = 0.0 kip Arm = 0.27 ft Moment = 0.0 k-ft
- Overturning moment Z-Z = 0.0 + 0.0 = 0.0 k-ft

- Resisting about Z-Z

- Footing weight = *0.6 * W * L * Thick * Density =* 0.6 * 3.00 * 3.00 * 8.0 / 12 * 0.15 = 0.5 kip
  Arm = *L / 2 =* 3.00 / 2 = 1.50 ft Moment = 0.5 * 1.50 = 0.8 k-ft
- Pedestal weight = *0.6 * W * L * H * Density =* 0.6 * 6.0 / 12 * 6.0 / 12 * 0.0 * 0.15 = 0.0 kip
  Arm = *L / 2 - Offset =* 3.00 / 2 - 0.0 / 12 = 1.50 ft Moment = 0.0 * 1.50 = 0.0 k-ft
- Soil cover = *0.6 * W * L * SC * Density =* 0.6 * (3.00 * 3.00 - 6.0 / 12 * 6.0 / 12) * 0.0 * 110 = 0.0 kip
  Arm = *L / 2 =* 3.00 / 2 = 1.50 ft Moment = 0.0 * 1.50 = 0.0 k-ft
- Buoyancy = *0.6 * W * L * γ * (SC + Thick - WT) =* 0.6 * 3.00 * 3.00 * 62 * (0.67) = -0.2 kip
  Arm = *L / 2 =* 3.00 / 2 = 1.50 ft Moment = 0.2 * 1.50 = -0.3 k-ft
- Axial force P = 0.6 * 10.4 + 0.6 * 0.0 = 6.2 kip
  Arm = *L / 2 - Offset =* 3.00 / 2 - 0.0 / 12 = 1.50 ft Moment = 6.2 * 1.50 = 9.4 k-ft
- Resisting moment Z-Z = 0.8 + 0.0 + 0.0 + 9.4 + -0.3 = 9.8 k-ft
- Overturning safety factor Z-Z = $\frac{\textit{Resisting moment}}{\textit{Overturning moment}} = \frac{9.8}{0.0}$ = 98.33 > 1.50 OK

SOIL BEARING PRESSURES (Comb: D+0.75L+0.525S)

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

Resisting moment X-X = 1.4 + 0.0 + 0.0 + -0.6 + 24.0 = 24.8 k-ft

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

Resisting moment Z-Z = 1.4 + 0.0 + 0.0 + -0.6 + 24.0 = 24.8 k-ft

Resisting force = *Footing + Pedestal + Soil - Buoyancy + P =* 0.9 + 0.0 + 0.0 - 0.4 + 16.0 = 16.5 kip

X-coordinate of resultant from maximum bearing corner:

$$Xp = \frac{\textit{Z-Resisting moment - Z-Overturning moment}}{\textit{Resisting force}} = \frac{24.8 - 0.0}{16.5} = 1.50 \text{ ft}$$

Z-coordinate of resultant from maximum bearing corner:

$$Zp = \frac{\textit{X-Resisting moment - X-Overturning moment}}{\textit{Resisting force}} = \frac{24.8 - 0.0}{16.5} = 1.50 \text{ ft}$$

X-ecc = *Length / 2 - Xp =* 3.00 / 2 - 1.50 = 0.00 ft

Z-ecc = *Width / 2 - Zp =* 3.00 / 2 - 1.50 = 0.00 ft

Area = *Width * Length =* 3.00 * 3.00 = 9.0 ft²

Sx = *Length * Width² / 6 =* 3.00 * 3.00² / 6 = 4.5 ft³

Sz = *Width * Length² / 6 =* 3.00 * 3.00² / 6 = 4.5 ft³

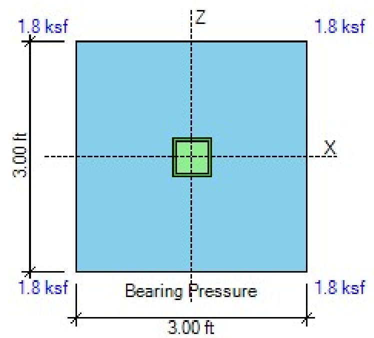
- Footing is in full bearing. Soil pressures are as follows:

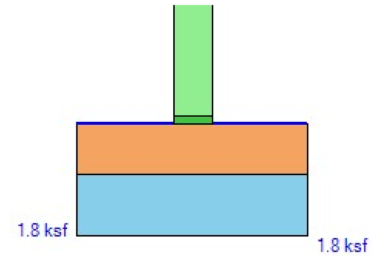
P1 = *P * (1/ A + Z-ecc / Sx + X-ecc / Sz) =* 16.5 * (1 / 9.0 + 0.00 / 4.5 + 0.00 / 4.5) = 1.83 ksf

P2 = *P * (1/ A - Z-ecc / Sx + X-ecc / Sz) =* 16.5 * (1 / 9.0 - 0.00 / 4.5 + 0.00 / 4.5) = 1.83 ksf

P3 = *P * (1/ A - Z-ecc / Sx - X-ecc / Sz) =* 16.5 * (1 / 9.0 - 0.00 / 4.5 - 0.00 / 4.5) = 1.83 ksf

P4 = *P * (1/ A + Z-ecc / Sx - X-ecc / Sz) =* 16.5 * (1 / 9.0 + 0.00 / 4.5 - 0.00 / 4.5) = 1.83 ksf





SLIDING CALCULATIONS (Comb: 0.6D+0.6W)

Internal friction angle = 28.0 deg

Passive coefficient *kp = 4.33 (per Coulomb)*

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

X-Passive force = *Pressure * Thick * Width =* 0.16 * 8.0 / 12 * 3.00 = 0.3 kip

Z-Passive force = *Pressure * Thick * Length =* 0.16 * 8.0 / 12 * 3.00 = 0.3 kip

Friction force = *Resisting force * Friction coeff. =* Max (0, 6.6 * 0.35) = 2.3 kip

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

- Sliding safety factor X-X = $\frac{\text{X-Passive force + Friction}}{\text{X-Horizontal load}} = \frac{1.00 * 0.3 + 1.00 * 2.3}{0.0}$ = 26.12 > 1.50 OK

- Sliding safety factor Z-Z = $\frac{\text{Z-Passive force + Friction}}{\text{Z-Horizontal load}} = \frac{1.00 * 0.3 + 1.00 * 2.3}{0.0}$ = 26.12 > 1.50 OK

UPLIFT CALCULATIONS (Comb: 0.6D+0.6W)

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

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

Concrete f'c = 2.5 ksi Steel fy = 60.0 ksi Soil density = 110 pcf

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

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

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

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

$\phi Vcx = 8 * \rho^{1/3} * \sqrt{f'c} * Width * d$ = 8 * (0.0047)⅓ * √(2500) * 3.0 * 12 * 4.8 / 1000 = 8.7 kip ACI 22.5.5.1

$\phi Vcz = 8 * \rho^{1/3} * \sqrt{f'c} * Length * d$ = 8 * (0.0052)⅓ * √(2500) * 3.0 * 12 * 4.3 / 1000 = 8.1 kip

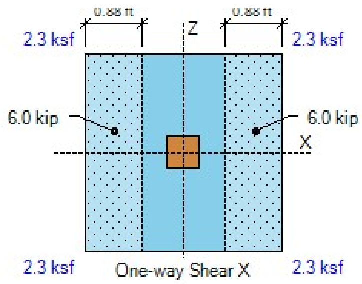
- Shear forces calculated as the volume of the bearing pressures under the effective areas:

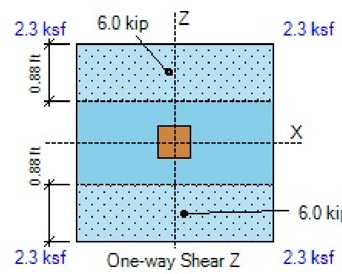
One-way shear Vux (- Side) = 6.0 kip < 8.7 kip OK

One-way shear Vux (+ Side) = 6.0 kip < 8.7 kip OK

One-way shear Vuz (- Side) = 6.0 kip < 8.1 kip OK

One-way shear Vuz (+ Side) = 6.0 kip < 8.1 kip OK





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

Plain ϕMnx = *5 * ϕ * √(f'c) * L * Thick^2 / 6* = 5 * 0.60 * √(2500) * 3.00 * 8.0^2 / 6 / 1000 = 1.3 k-ft — ACI Eq. (14.5.2.1a)

Plain ϕMnz = *5 * ϕ * √(f'c) * W * Thick^2 / 6* = 5 * 0.60 * √(2500) * 3.00 * 8.0^2 / 6 / 1000 = 1.3 k-ft

- Top Bars

No Top Reinforcement Provided at the Footing

Use Plain Concrete Flexural Strength at Top

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

Top moment -Mux (- Side) = 0.0 k-ft < 4.8 k-ft OK

Top moment -Mux (+ Side) = 0.0 k-ft < 4.8 k-ft OK

Top moment -Muz (- Side) = 0.0 k-ft < 4.8 k-ft OK

Top moment -Muz (+ Side) = 0.0 k-ft < 4.8 k-ft OK

- Bottom Bars

Use 4 #4 Z-Bars — *ρ = As / b d* = 0.8 / (3.00 * 12 * 4.3) = 0.0052 — q = 0.0052 * 60 / 2.5 = 0.125

Use 4 #4 X-Bars — *ρ = As / b d* = 0.8 / (3.00 * 12 * 4.8) = 0.0047 — q = 0.0047 * 60 / 2.5 = 0.112

β = L / W = 3.00 / 3.00 = 1.00 — *γs = 2 * β / (β + 1)* = 2 * 1.00 / (1.00 + 1) = 1.00 — ACI 13.3.3.3

Bending strength *ϕMn = ϕ * b * d^2 * f'c * q * (1 - 0.59 * q)* — ACI 22.2.2

ϕMnx = 0.90 * 3.00 * 12 * 4.3^2 * 2.5 * 0.125 * (1 - 0.59 * 0.125) = 14.2 k-ft

ϕMnz = 0.90 * 3.00 * 12 * 4.8^2 * 2.5 * 0.112 / 1.00 * (1 - 0.59 * 0.112 / 1.00) = 16.0 k-ft

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

Bottom moment Mux (- Side) = 5.3 k-ft < 14.2 k-ft OK ratio = 0.38

Bottom moment Mux (+ Side) = 5.3 k-ft < 14.2 k-ft OK ratio = 0.38

Bottom moment Muz (- Side) = 5.3 k-ft < 16.0 k-ft OK ratio = 0.33

Bottom moment Muz (+ Side) = 5.3 k-ft < 16.0 k-ft OK ratio = 0.33

X-As min = *0.0018 * Width * Thick* = 0.0018 * 3.00 * 12 * 8.0 = 0.5 in^2 < 0.8 in^2 OK — ACI 8.6.1.1

Z-As min = *0.0018 * Length * Thick* = 0.0018 * 3.00 * 12 * 8.0 = 0.5 in^2 < 0.8 in^2 OK — ACI 8.6.1.1

X-As max for fy/Es + 0.003 tension strain = 1.81 in^2 > 0.80 in^2 OK — ACI 21.2.2

Z-As max for fy/Es + 0.003 tension strain = 1.81 in^2 > 0.80 in^2 OK — ACI 21.2.2

X-Cover factor = *Min (2.5, (Cover + db /2, Spacing / 2) / db)* = Min (2.5, (3.0 + 0.50 / 2, 10.0 / 2) / 0.50) = 2.5

Straight *X-Ld = Max (12.0, 3 / 40 * fy / (f'c)½ * Grade * Size * Casting / Cover * db * ratio)* — ACI Eq. (25.4.2.4a)

X-Ld = Max (12.0, 3 / 40 * 60.0 * 1000 / (2500)½ * 1.0 * 0.8 * 1.0 / 2.5 * 0.50 * 0.33) = 12.0 in

Hooked *X-Ldh = Max (8 db, 6, 1 / 55 * fy / (f'c)½ * Confining * Location * Concrete * db^1.5) =* — ACI 25.4.3

X-Ldh = Max (8 db, 6, 1 / 55 * 60.0 * 1000 / (2500)½ * 1.0 * 1.0 * 0.8 * 0.50^1.5) = 6.0 in

-X Ld provided = *(Length - Col) / 2 + Offset - Cover =* 3.00 * 12 / 2 + 0.0 - 6.0 / 2 - 2.5 = 12.5 in > 12.0 in OK

+X Ld provided = *(Length - Col) / 2 - Offset - Cover =* 3.00 * 12 / 2 - 0.0 - 6.0 / 2 - 2.5 = 12.5 in > 12.0 in OK

Z-Cover factor = *Min (2.5, (Cover + db /2, Spacing / 2) / db)* = Min (2.5, (3.0 + 0.50 / 2, 10.0 / 2) / 0.50) = 2.5

Straight *Z-Ld = Max (12.0, 3 / 40 * fy / (f'c)½ * Grade * Size * Casting / Cover * db * ratio)* ACI Eq. (25.4.2.4a)

Z-Ld = Max (12.0, 3 / 40 * 60.0 * 1000 / (2500)½ * 1.0 * 0.8 * 1.0 / 2.5 * 0.50 * 0.33) = 12.0 in

Hooked *Z-Ldh = Max (8 db, 6, 1 / 55 * fy / (f'c)½ * Confining * Location * Concrete * db^1.5) =* ACI 25.4.3

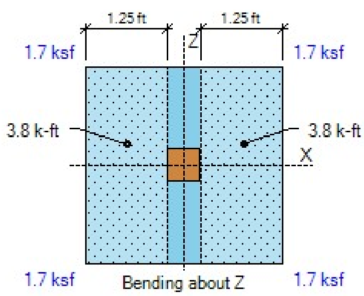
Z-Ldh = Max (8 db, 6, 1 / 55 * 60.0 * 1000 / (2500)½ * 1.0 * 1.0 * 0.8 * 0.50^1.5) = 6.0 in

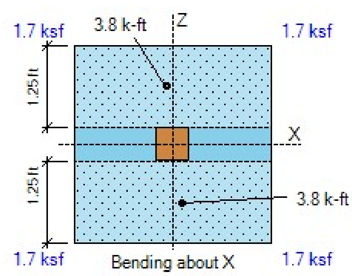
-Z Ld provided = *(Width - Col) / 2 + Offset - Cover =* 3.00 * 12 / 2 + 0.0 - 6.0 / 2 - 2.5 = 12.5 in > 12.0 in OK

+Z Ld provided = *(Width - Col) / 2 - Offset - Cover =* 3.00 * 12 / 2 - 0.0 - 6.0 / 2 - 2.5 = 12.5 in > 12.0 in OK

X-bar spacing = 10.0 in < Min (3 * t, 18.0) = 18.0 in OK ACI 7.7.2.3

Z-bar spacing = 10.0 in < Min (3 * t, 18.0) = 18.0 in OK





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

Area *A1 = col L * col W =* 6.0 * 6.0 = 36.0 in^2

Sx = *col W * col L² / 6 =* 6.0 * 6.0^2 / 6 = 36.0 in^3

Sz = *col L * col W² / 6 =* 6.0 * 6.0^2 / 6 = 36.0 in^3

Bearing *Pbu = P / A1 + Mz / Sx + Mx / Sz =* 20.4 / 36.0 + 0.0 * 12 / 36.0 + 0.0 * 12 / 36.0 = 0.6 ksi

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

Min edge = Min (3.00 * 12 / 2 - 0.0 - 6.0 / 2, 3.00 * 12 / 2 - 0.0 - 6.0 / 2 = 15.0 in

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

A2 = Min [3.00 * 12 * 3.0 * 12, (6.0 + 2 * 15.0) * (6.0 + 2 * 15.0)] = 1296.0 in^2

Footing *ϕPnc = ϕ * 0.85 * f'c * Min [2, √(A2 / A1)] =* 0.65 * 0.85 * 2.5 * Min [2, √(1296.0 / 36.0)] = 2.8 ksi

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

Footing bearing *ϕPn = ϕPnc + ϕPns =* 2.8 + 0.0 = 2.8 ksi > 0.6 psi OK

Hooked *Ldh = Max (8 db, 6, 1 / 55 * fy / (f'c)½ * Confining * Location * Concrete * db^1.5)* ACI 25.4.3

Ldh = Max (8 db, 6, 1 / 55 * 60.0 * 1000 / (2500)½ * 1.6 * 1.0 * 0.8 * 0.75^1.5) = 17.4 in

Ld provided = *Dowel length =* 3.00 * 12 = 36.0 in > 17.6 in OK

Ldh provided = *Footing thickness - Cover =* 8.00 - 3.0 = 5.0 in < 17.4 in NG

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

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

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

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

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

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

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

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

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

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

F = 20.4 + 0.07 * 110.3 / 144 - 1.8 = 18.7 kip

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

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

ACI Eq. (8.4.4.2.2)

ACI Eq. (8.4.2.3.2)

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

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

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

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

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

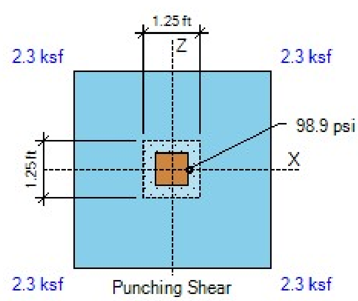
Jcx = 10.5 * 4.5³ / 6 + 10.5³ * 4.5 / 6 + 10.5² * 10.5 * 4.5 / 2 = 3632 in⁴

Stress due to P = *F / (bo * d) * 1000 =* 18.7 / (42.0 * 4.5) * 1000 = 98.9 psi

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

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

Punching stress = *P-stress + Mx-stress + Mz-stress =* 98.9 + 0.0 + 0.0 = 98.9 psi < 150.0 psi OK



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

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

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

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

Bearing *Pbu = P / A1 + Mz / Sx + Mx / Sz =* 20.4 / 36.0 + 0.0 * 12 / 36.0 + 0.0 * 12 / 36.0 = 0.6 ksi

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

Min edge = Min (3.00 * 12 / 2 - 0.0 - 6.0 / 2, 3.00 * 12 / 2 - 0.0 - 6.0 / 2 = 15.0 in

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

A2 = Min [3.00 * 12 * 3.0 * 12, (6.0 + 2 * 15.0) * (6.0 + 2 * 15.0)] = 1296.0 in²

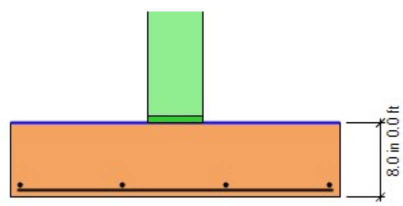
Footing *ϕPnc = ϕ * 0.85 * f'c * Min [2, √(A2 / A1)] =* 0.65 * 0.85 * 2.5 * Min [2, √(1296.0 / 36.0)] = 2.8 ksi

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

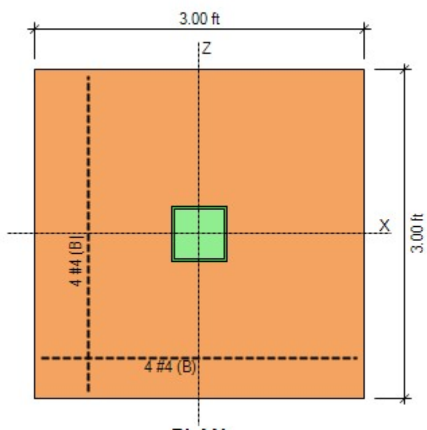
Footing bearing *ϕPn = ϕPnc + ϕPns =* 2.8 + 0.0 = 2.8 ksi > 0.6 psi OK

DESIGN CODES

Concrete Design	ACI 318-19
Load Combinations	ASCE 7-22



ELEVATION



PLAN

ASDIP Foundation 5.6.0.0 **SPREAD FOOTING DESIGN** www.asdipsoft.com

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	1.9	ksf	
Soil Pressure at Corner 2	1.9	ksf	
Soil Pressure at Corner 3	1.9	ksf	
Soil Pressure at Corner 4	1.9	ksf	
Bearing Pressure Ratio	0.96		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	7.7	9.1	0.0	0.3	0.0	0.0	kip
Moment about X Mx ..	0.0	0.0	0.0	0.0	0.0	0.0	k-ft
Moment about Z Mz ..	0.0	0.0	0.0	0.0	0.0	0.0	k-ft
Shear Force Vx	0.0	0.0	0.0	0.0	0.0	0.0	kip
Shear Force Vz	0.0	0.0	0.0	0.0	0.0	0.0	kip

OVERTURNING CALCULATIONS (Comb: 0.6D+0.6W)

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

- Resisting about X-X
- Footing weight = *0.6 * W * L * Thick * Density =* 0.6 * 3.00 * 3.00 * 8.0 / 12 * 0.15 = 0.5 kip
  Arm = *W / 2 =* 3.00 / 2 = 1.50 ft   Moment = 0.5 * 1.50 = 0.8 k-ft
- Pedestal weight = *0.6 * W * L * H * Density =* 0.6 * 6.0 / 12 * 6.0 / 12 * 0.0 * 0.15 = 0.0 kip
  Arm = *W / 2 - Offset =* 3.00 / 2 - 0.0 / 12 = 1.50 ft   Moment = 0.0 * 1.50 = 0.0 k-ft
- Soil cover = *0.6 * W * L * SC * Density =* 0.6 * (3.00 * 3.00 - 6.0 / 12 * 6.0 / 12) * 0.0 * 110 = 0.0 kip
  Arm = *W / 2 =* 3.00 / 2 = 1.50 ft   Moment = 0.0 * 1.50 = 0.0 k-ft
- Buoyancy = *0.6 * W * L * γ * (SC + Thick - WT) =* 0.6 * 3.00 * 3.00 * 62 * (0.67) = -0.2 kip
  Arm = *W / 2 =* 3.00 / 2 = 1.50 ft   Moment = 0.2 * 1.50 = -0.3 k-ft
- Axial force P = 0.6 * 7.7 + 0.6 * 0.0 = 4.6 kip
  Arm = *W / 2 - Offset =* 3.00 / 2 - 0.0 / 12 = 1.50 ft   Moment = 4.6 * 1.50 = 6.9 k-ft
- Resisting moment X-X = 0.8 + 0.0 + 0.0 + 6.9 + -0.3 = 7.4 k-ft

- Overturning safety factor X-X = $\dfrac{\textit{Resisting moment}}{\textit{Overturning moment}} = \dfrac{7.4}{0.0}$ = 74.03 > 1.50 OK

- Overturning about Z-Z

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

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

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

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

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

- Resisting about Z-Z

- Footing weight = *0.6 * W * L * Thick * Density =* 0.6 * 3.00 * 3.00 * 8.0 / 12 * 0.15 = 0.5 kip

Arm = *L / 2 =* 3.00 / 2 = 1.50 ft Moment = 0.5 * 1.50 = 0.8 k-ft

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

Arm = *L / 2 - Offset =* 3.00 / 2 - 0.0 / 12 = 1.50 ft Moment = 0.0 * 1.50 = 0.0 k-ft

- Soil cover = *0.6 * W * L * SC * Density =* 0.6 * (3.00 * 3.00 - 6.0 / 12 * 6.0 / 12) * 0.0 * 110 = 0.0 kip

Arm = *L / 2 =* 3.00 / 2 = 1.50 ft Moment = 0.0 * 1.50 = 0.0 k-ft

- Buoyancy = *0.6 * W * L * γ * (SC + Thick - WT) =* 0.6 * 3.00 * 3.00 * 62 * (0.67) = -0.2 kip

Arm = *L / 2 =* 3.00 / 2 = 1.50 ft Moment = 0.2 * 1.50 = -0.3 k-ft

- Axial force P = 0.6 * 7.7 + 0.6 * 0.0 = 4.6 kip

Arm = *L / 2 - Offset =* 3.00 / 2 - 0.0 / 12 = 1.50 ft Moment = 4.6 * 1.50 = 6.9 k-ft

- Resisting moment Z-Z = 0.8 + 0.0 + 0.0 + 6.9 + -0.3 = 7.4 k-ft

- Overturning safety factor Z-Z = $\dfrac{\textit{Resisting moment}}{\textit{Overturning moment}} = \dfrac{7.4}{0.0}$ = 74.03 > 1.50 OK

SOIL BEARING PRESSURES (Comb: D+L)

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

Resisting moment X-X = 1.4 + 0.0 + 0.0 + -0.6 + 25.2 = 26.0 k-ft

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

Resisting moment Z-Z = 1.4 + 0.0 + 0.0 + -0.6 + 25.2 = 26.0 k-ft

Resisting force = *Footing + Pedestal + Soil - Buoyancy + P =* 0.9 + 0.0 + 0.0 - 0.4 + 16.8 = 17.3 kip

X-coordinate of resultant from maximum bearing corner:

Xp = $\dfrac{\textit{Z-Resisting moment - Z-Overturning moment}}{\textit{Resisting force}} = \dfrac{26.0 - 0.0}{17.3}$ = 1.50 ft

Z-coordinate of resultant from maximum bearing corner:

Zp = $\dfrac{\textit{X-Resisting moment - X-Overturning moment}}{\textit{Resisting force}} = \dfrac{26.0 - 0.0}{17.3}$ = 1.50 ft

X-ecc = *Length / 2 - Xp =*3.00 / 2 - 1.50 = 0.00 ft

Z-ecc = *Width / 2 - Zp =* 3.00 / 2 - 1.50 = 0.00 ft

Area = *Width * Length =* 3.00 * 3.00 = 9.0 ft²

Sx = *Length * Width² / 6 =* 3.00 * 3.00² / 6 = 4.5 ft³

Sz = *Width * Length² / 6 =* 3.00 * 3.00² / 6 = 4.5 ft³

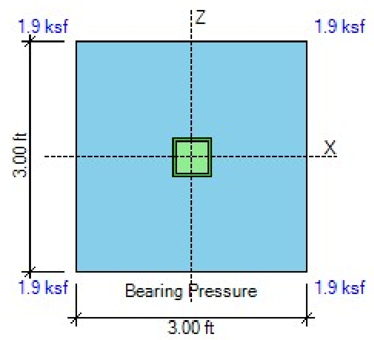
- Footing is in full bearing. Soil pressures are as follows:

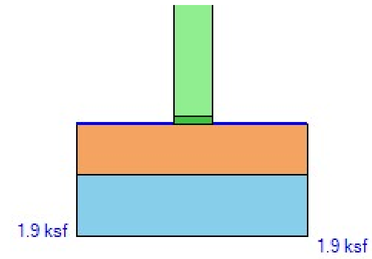
P1 = *P * (1/ A + Z-ecc / Sx + X-ecc / Sz) =* 17.3 * (1 / 9.0 + 0.00 / 4.5 + 0.00 / 4.5) = 1.93 ksf

P2 = *P * (1/ A - Z-ecc / Sx + X-ecc / Sz) =* 17.3 * (1 / 9.0 - 0.00 / 4.5 + 0.00 / 4.5) = 1.93 ksf

P3 = *P * (1/ A - Z-ecc / Sx - X-ecc / Sz) =* 17.3 * (1 / 9.0 - 0.00 / 4.5 - 0.00 / 4.5) = 1.93 ksf

P4 = *P * (1/ A + Z-ecc / Sx - X-ecc / Sz) =* 17.3 * (1 / 9.0 + 0.00 / 4.5 - 0.00 / 4.5) = 1.93 ksf





SLIDING CALCULATIONS (Comb: 0.6D+0.6W)

Internal friction angle = 28.0 deg

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

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

X-Passive force = *Pressure * Thick * Width =* 0.16 * 8.0 / 12 * 3.00 = 0.3 kip

Z-Passive force = *Pressure * Thick * Length =* 0.16 * 8.0 / 12 * 3.00 = 0.3 kip

Friction force = *Resisting force * Friction coeff. =* Max (0, 4.9 * 0.35) = 1.7 kip

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

- Sliding safety factor X-X = $\dfrac{\textit{X-Passive force + Friction}}{\textit{X-Horizontal load}} = \dfrac{1.00 * 0.3 + 1.00 * 1.7}{0.0}$ = 20.45 > 1.50 OK

- Sliding safety factor Z-Z = $\dfrac{\textit{Z-Passive force + Friction}}{\textit{Z-Horizontal load}} = \dfrac{1.00 * 0.3 + 1.00 * 1.7}{0.0}$ = 20.45 > 1.50 OK

UPLIFT CALCULATIONS (Comb: 0.6D+0.6W)

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

ONE-WAY SHEAR CALCULATIONS (Comb: 1.2D+1.6L+0.3S)

Concrete f'c = 2.5 ksi | Steel fy = 60.0 ksi | Soil density = 110 pcf

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

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

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

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

$\phi Vcx = 8 * \rho^{1/3} * \sqrt{f'c} * Width * d =$ 8 * (0.0047)⅓ * √(2500) * 3.0 * 12 * 4.8 / 1000 = 8.7 kip ACI 22.5.5.1

$\phi Vcz = 8 * \rho^{1/3} * \sqrt{f'c} * Length * d =$ 8 * (0.0052)⅓ * √(2500) * 3.0 * 12 * 4.3 / 1000 = 8.1 kip

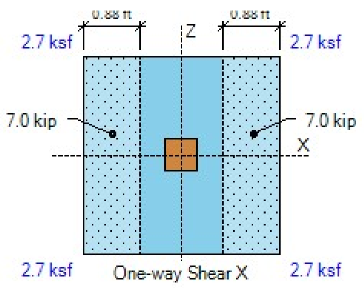
- Shear forces calculated as the volume of the bearing pressures under the effective areas:

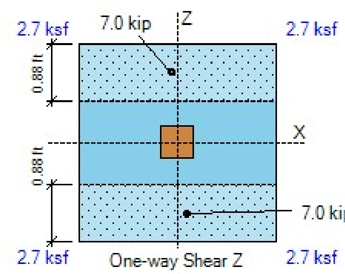
One-way shear Vux (- Side) = 7.0 kip < 8.7 kip OK

One-way shear Vux (+ Side) = 7.0 kip < 8.7 kip OK

One-way shear Vuz (- Side) = 7.0 kip < 8.1 kip OK

One-way shear Vuz (+ Side) = 7.0 kip < 8.1 kip OK





FLEXURE CALCULATIONS (Comb: 1.2D+1.6L+0.3S)

Plain ϕMnx = *5 * ϕ * √(f'c) * L * Thick² / 6* = 5 * 0.60 * √(2500) * 3.00 * 8.0² / 6 / 1000 = 1.3 k-ft — ACI Eq. (14.5.2.1a)

Plain ϕMnz = *5 * ϕ * √(f'c) * W * Thick² / 6* = 5 * 0.60 * √(2500) * 3.00 * 8.0² / 6 / 1000 = 1.3 k-ft

- Top Bars

No Top Reinforcement Provided at the Footing

Use Plain Concrete Flexural Strength at Top

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

Top moment -Mux (- Side) = 0.0 k-ft < 4.8 k-ft OK

Top moment -Mux (+ Side) = 0.0 k-ft < 4.8 k-ft OK

Top moment -Muz (- Side) = 0.0 k-ft < 4.8 k-ft OK

Top moment -Muz (+ Side) = 0.0 k-ft < 4.8 k-ft OK

- Bottom Bars

Use 4 #4 Z-Bars *ρ = As / b d* = 0.8 / (3.00 * 12 * 4.3) = 0.0052 — q = 0.0052 * 60 / 2.5 = 0.125

Use 4 #4 X-Bars *ρ = As / b d* = 0.8 / (3.00 * 12 * 4.8) = 0.0047 — q = 0.0047 * 60 / 2.5 = 0.112

β = L / W = 3.00 / 3.00 = 1.00 *γs = 2 * β / (β + 1)* = 2 * 1.00 / (1.00 + 1) = 1.00 — ACI 13.3.3.3

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

ϕMnx = 0.90 * 3.00 * 12 * 4.3² * 2.5 * 0.125 * (1 - 0.59 * 0.125) = 14.2 k-ft

ϕMnz = 0.90 * 3.00 * 12 * 4.8² * 2.5 * 0.112 / 1.00 * (1 - 0.59 * 0.112 / 1.00) = 16.0 k-ft

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

Bottom moment Mux (- Side) = 6.2 k-ft < 14.2 k-ft OK ratio = 0.44

Bottom moment Mux (+ Side) = 6.2 k-ft < 14.2 k-ft OK ratio = 0.44

Bottom moment Muz (- Side) = 6.2 k-ft < 16.0 k-ft OK ratio = 0.39

Bottom moment Muz (+ Side) = 6.2 k-ft < 16.0 k-ft OK ratio = 0.39

X-As min = *0.0018 * Width * Thick* = 0.0018 * 3.00 * 12 * 8.0 = 0.5 in² < 0.8 in² OK — ACI 8.6.1.1

Z-As min = *0.0018 * Length * Thick* = 0.0018 * 3.00 * 12 * 8.0 = 0.5 in² < 0.8 in² OK — ACI 8.6.1.1

X-As max for fy/Es + 0.003 tension strain = 1.81 in² > 0.80 in² OK — ACI 21.2.2

Z-As max for fy/Es + 0.003 tension strain = 1.81 in² > 0.80 in² OK — ACI 21.2.2

X-Cover factor = *Min (2.5, (Cover + db /2, Spacing / 2) / db)* = Min (2.5, (3.0 + 0.50 / 2, 10.0 / 2) / 0.50) = 2.5

Straight *X-Ld = Max (12.0, 3 / 40 * fy / (f'c)½ * Grade * Size * Casting / Cover * db * ratio)* — ACI Eq. (25.4.2.4a)

X-Ld = Max (12.0, 3 / 40 * 60.0 * 1000 / (2500)½ * 1.0 * 0.8 * 1.0 / 2.5 * 0.50 * 0.39) = 12.0 in

Hooked *X-Ldh = Max (8 db, 6, 1 / 55 * fy / (f'c)½ * Confining * Location * Concrete * db^1.5) =* — ACI 25.4.3

X-Ldh = Max (8 db, 6, 1 / 55 * 60.0 * 1000 / (2500)½ * 1.0 * 1.0 * 0.8 * 0.50^1.5) = 6.0 in

-X Ld provided = *(Length - Col) / 2 + Offset - Cover =* 3.00 * 12 / 2 + 0.0 - 6.0 / 2 - 2.5 = 12.5 in > 12.0 in OK

+X Ld provided = *(Length - Col) / 2 - Offset - Cover =* 3.00 * 12 / 2 - 0.0 - 6.0 / 2 - 2.5 = 12.5 in > 12.0 in OK

Z-Cover factor = *Min (2.5, (Cover + db /2, Spacing / 2) / db) =* Min (2.5, (3.0 + 0.50 / 2, 10.0 / 2) / 0.50) = 2.5

Straight *Z-Ld = Max (12.0, 3 / 40 * fy / (f'c)$^{1/2}$ * Grade * Size * Casting / Cover * db * ratio)* ACI Eq. (25.4.2.4a)

Z-Ld = Max (12.0, 3 / 40 * 60.0 * 1000 / (2500)$^{1/2}$ * 1.0 * 0.8 * 1.0 / 2.5 * 0.50 * 0.39) = 12.0 in

Hooked *Z-Ldh = Max (8 db, 6, 1 / 55 * fy / (f'c)$^{1/2}$ * Confining * Location * Concrete * db^1.5) =* ACI 25.4.3

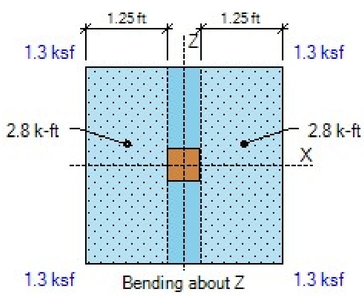
Z-Ldh = Max (8 db, 6, 1 / 55 * 60.0 * 1000 / (2500)$^{1/2}$ * 1.0 * 1.0 * 0.8 * 0.50^1.5) = 6.0 in

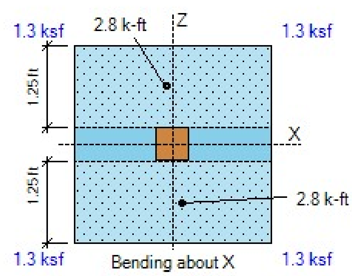
-Z Ld provided = *(Width - Col) / 2 + Offset - Cover =* 3.00 * 12 / 2 + 0.0 - 6.0 / 2 - 2.5 = 12.5 in > 12.0 in OK

+Z Ld provided =*(Width - Col) / 2 - Offset - Cover =* 3.00 * 12 / 2 - 0.0 - 6.0 / 2 - 2.5 = 12.5 in > 12.0 in OK

X-bar spacing = 10.0 in < Min (3 * t, 18.0) = 18.0 in OK ACI 7.7.2.3

Z-bar spacing = 10.0 in < Min (3 * t, 18.0) = 18.0 in OK





LOAD TRANSFER CALCULATIONS (Comb: 1.2D+1.6L+0.3S)

Area *A1 = col L * col W =* 6.0 * 6.0 = 36.0 in^2

Sx = *col W * col L^2 / 6 =* 6.0 * 6.0^2 / 6 = 36.0 in^3

Sz = *col L * col W^2 / 6 =* 6.0 * 6.0^2 / 6 = 36.0 in^3

Bearing *Pbu = P / A1 + Mz / Sx + Mx / Sz =* 23.9 / 36.0 + 0.0 * 12 / 36.0 + 0.0 * 12 / 36.0 = 0.7 ksi

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

Min edge = Min (3.00 * 12 / 2 - 0.0 - 6.0 / 2, 3.00 * 12 / 2 - 0.0 - 6.0 / 2 = 15.0 in

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

A2 = Min [3.00 * 12 * 3.0 * 12, (6.0 + 2 * 15.0) * (6.0 + 2 * 15.0)] = 1296.0 in^2

Footing *ϕPnc = ϕ * 0.85 * f'c * Min [2, √(A2 / A1)] =* 0.65 * 0.85 * 2.5 * Min [2, √(1296.0 / 36.0)] = 2.8 ksi

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

Footing bearing *ϕPn = ϕPnc + ϕPns =* 2.8 + 0.0 = 2.8 ksi > 0.7 psi OK

Hooked *Ldh = Max (8 db, 6, 1 / 55 * fy / (f'c)½ * Confining * Location * Concrete * db^1.5)* ACI 25.4.3

Ldh = Max (8 db, 6, 1 / 55 * 60.0 * 1000 / (2500)½ * 1.6 * 1.0 * 0.8 * 0.75^1.5) = 17.4 in

Ld provided = *Dowel length =* 3.00 * 12 = 36.0 in > 20.6 in OK

Ldh provided = *Footing thickness - Cover =* 8.00 - 3.0 = 5.0 in < 17.4 in NG

PUNCHING SHEAR CALCULATIONS (Comb: 1.2D+1.6L+0.3S)

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

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

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

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

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

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

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

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

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

F = 23.9 + 0.07 * 110.3 / 144 - 2.1 = 21.8 kip

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

$$\gamma vx \text{ factor} = 1 - \frac{1}{1 + (2/3)\sqrt{b2 / b1}} = 1 - \frac{1}{1 + (2/3)\sqrt{10.5 / 10.5}} = 0.40$$ 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 in X2x = *b2 / 2 =* 10.5 / 2 = 5.3 in

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

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

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

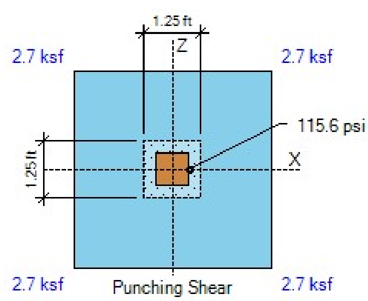
Jcx = 10.5 * 4.5³ / 6 + 10.5³ * 4.5 / 6 + 10.5² * 10.5 * 4.5 / 2 = 3632 in⁴

Stress due to P = *F / (bo * d) * 1000 =* 21.8 / (42.0 * 4.5) * 1000 = 115.6 psi

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

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

Punching stress = *P-stress + Mx-stress + Mz-stress =* 115.6 + 0.0 + 0.0 = 115.6 psi < 150.0 psi OK



SPREAD FOOTING DESIGN

LOAD TRANSFER CALCULATIONS (Comb: 1.2D+1.6L+0.3S)

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

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

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

Bearing *Pbu = P / A1 + Mz / Sx + Mx / Sz* = 23.9 / 36.0 + 0.0 * 12 / 36.0 + 0.0 * 12 / 36.0 = 0.7 ksi

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

Min edge = Min (3.00 * 12 / 2 - 0.0 - 6.0 / 2, 3.00 * 12 / 2 - 0.0 - 6.0 / 2 = 15.0 in

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

A2 = Min [3.00 * 12 * 3.0 * 12, (6.0 + 2 * 15.0) * (6.0 + 2 * 15.0)] = 1296.0 in²

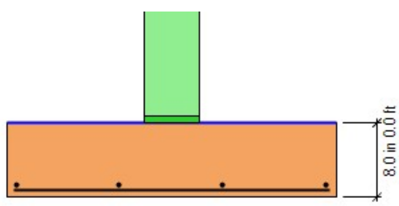
Footing *ϕPnc = ϕ * 0.85 * f'c * Min [2, √(A2 / A1)]* = 0.65 * 0.85 * 2.5 * Min [2, √(1296.0 / 36.0)] = 2.8 ksi

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

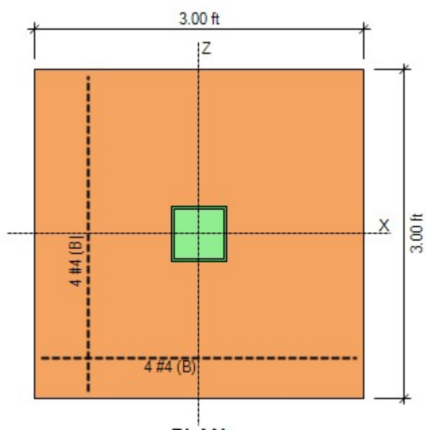
Footing bearing *ϕPn = ϕPnc + ϕPns* = 2.8 + 0.0 = 2.8 ksi > 0.7 psi OK

DESIGN CODES

Concrete Design	ACI 318-19
Load Combinations	ASCE 7-22



ELEVATION



PLAN

SPREAD FOOTING DESIGN

GEOMETRY

Footing Length (X-dir)	3.00	ft	
Footing Width (Z-dir)	3.00	ft	
Footing Thickness	8.0	in	OK
Soil Cover	0.00	ft	
Column Length (X-dir)	6.0	in	
Column Width (Z-dir)	6.0	in	
Offset (X-dir)	0.00	in	OK
Offset (Z-dir)	0.00	in	OK
Base Plate (L x W)	6.0 x 6.0	in	

SOIL PRESSURES (D+L)

Gross Allow. Soil Pressure	2.0	ksf	
Soil Pressure at Corner 1	1.8	ksf	
Soil Pressure at Corner 2	1.8	ksf	
Soil Pressure at Corner 3	1.8	ksf	
Soil Pressure at Corner 4	1.8	ksf	
Bearing Pressure Ratio	0.92		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	10.1	6.0	0.0	2.6	0.0	0.0	kip
Moment about X Mx ..	0.0	0.0	0.0	0.0	0.0	0.0	k-ft
Moment about Z Mz ..	0.0	0.0	0.0	0.0	0.0	0.0	k-ft
Shear Force Vx	0.0	0.0	0.0	0.0	0.0	0.0	kip
Shear Force Vz	0.0	0.0	0.0	0.0	0.0	0.0	kip

OVERTURNING CALCULATIONS (Comb: 0.6D+0.6W)

- Overturning about X-X

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

- Resisting about X-X

- Footing weight = *0.6 * W * L * Thick * Density =* 0.6 * 3.00 * 3.00 * 8.0 / 12 * 0.15 = 0.5 kip
  Arm = *W / 2 =* 3.00 / 2 = 1.50 ft   Moment = 0.5 * 1.50 = 0.8 k-ft
- Pedestal weight = *0.6 * W * L * H * Density =* 0.6 * 6.0 / 12 * 6.0 / 12 * 0.0 * 0.15 = 0.0 kip
  Arm = *W / 2 - Offset =* 3.00 / 2 - 0.0 / 12 = 1.50 ft   Moment = 0.0 * 1.50 = 0.0 k-ft
- Soil cover = *0.6 * W * L * SC * Density =* 0.6 * (3.00 * 3.00 - 6.0 / 12 * 6.0 / 12) * 0.0 * 110 = 0.0 kip
  Arm = *W / 2 =* 3.00 / 2 = 1.50 ft   Moment = 0.0 * 1.50 = 0.0 k-ft
- Buoyancy = *0.6 * W * L * γ * (SC + Thick - WT) =* 0.6 * 3.00 * 3.00 * 62 * (0.67) = -0.2 kip
  Arm = *W / 2 =* 3.00 / 2 = 1.50 ft   Moment = 0.2 * 1.50 = -0.3 k-ft
- Axial force P = 0.6 * 10.1 + 0.6 * 0.0 = 6.1 kip
  Arm = *W / 2 - Offset =* 3.00 / 2 - 0.0 / 12 = 1.50 ft   Moment = 6.1 * 1.50 = 9.1 k-ft
- Resisting moment X-X = 0.8 + 0.0 + 0.0 + 9.1 + -0.3 = 9.6 k-ft

- Overturning safety factor X-X = $\frac{\textit{Resisting moment}}{\textit{Overturning moment}} = \frac{9.6}{0.0}$ = 95.63 > 1.50 OK

- Overturning about Z-Z

- Moment Mz = 0.6 * 0.0 + 0.6 * 0.0 = 0.0 k-ft
- Shear Force Vx = 0.6 * 0.0 + 0.6 * 0.0 = 0.0 kip
  Arm = 0.00 + 8.0 / 12 = 0.67 ft   Moment = 0.0 * 0.67 = 0.0 k-ft
- Passive Force = 0.0 kip   Arm = 0.27 ft   Moment = 0.0 k-ft
- Overturning moment Z-Z = 0.0 + 0.0 = 0.0 k-ft

- Resisting about Z-Z

- Footing weight = *0.6 * W * L * Thick * Density =* 0.6 * 3.00 * 3.00 * 8.0 / 12 * 0.15 = 0.5 kip
  Arm = *L / 2 =* 3.00 / 2 = 1.50 ft   Moment = 0.5 * 1.50 = 0.8 k-ft
- Pedestal weight = *0.6 * W * L * H * Density =* 0.6 * 6.0 / 12 * 6.0 / 12 * 0.0 * 0.15 = 0.0 kip
  Arm = *L / 2 - Offset =* 3.00 / 2 - 0.0 / 12 = 1.50 ft   Moment = 0.0 * 1.50 = 0.0 k-ft
- Soil cover = *0.6 * W * L * SC * Density =* 0.6 * (3.00 * 3.00 - 6.0 / 12 * 6.0 / 12) * 0.0 * 110 = 0.0 kip
  Arm = *L / 2 =* 3.00 / 2 = 1.50 ft   Moment = 0.0 * 1.50 = 0.0 k-ft
- Buoyancy = *0.6 * W * L * γ * (SC + Thick - WT) =* 0.6 * 3.00 * 3.00 * 62 * (0.67) = -0.2 kip
  Arm = *L / 2 =* 3.00 / 2 = 1.50 ft   Moment = 0.2 * 1.50 = -0.3 k-ft
- Axial force P = 0.6 * 10.1 + 0.6 * 0.0 = 6.1 kip
  Arm = *L / 2 - Offset =* 3.00 / 2 - 0.0 / 12 = 1.50 ft   Moment = 6.1 * 1.50 = 9.1 k-ft
- Resisting moment Z-Z = 0.8 + 0.0 + 0.0 + 9.1 + -0.3 = 9.6 k-ft

- Overturning safety factor Z-Z = $\dfrac{\textit{Resisting moment}}{\textit{Overturning moment}} = \dfrac{9.6}{0.0}$ = 95.63 > 1.50 OK

SOIL BEARING PRESSURES (Comb: D+L)

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

Resisting moment X-X = 1.4 + 0.0 + 0.0 + -0.6 + 24.2 = 24.9 k-ft

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

Resisting moment Z-Z = 1.4 + 0.0 + 0.0 + -0.6 + 24.2 = 24.9 k-ft

Resisting force = *Footing + Pedestal + Soil - Buoyancy + P =* 0.9 + 0.0 + 0.0 - 0.4 + 16.1 = 16.6 kip

X-coordinate of resultant from maximum bearing corner:

$$Xp = \frac{\text{Z-Resisting moment - Z-Overturning moment}}{\text{Resisting force}} = \frac{24.9 - 0.0}{16.6} = 1.50 \text{ ft}$$

Z-coordinate of resultant from maximum bearing corner:

$$Zp = \frac{\text{X-Resisting moment - X-Overturning moment}}{\text{Resisting force}} = \frac{24.9 - 0.0}{16.6} = 1.50 \text{ ft}$$

X-ecc = *Length / 2 - Xp =* 3.00 / 2 - 1.50 = 0.00 ft

Z-ecc = *Width / 2 - Zp =* 3.00 / 2 - 1.50 = 0.00 ft

Area = *Width * Length =* 3.00 * 3.00 = 9.0 ft²

Sx = *Length * Width² / 6 =* 3.00 * 3.00² / 6 = 4.5 ft³

Sz = *Width * Length² / 6 =* 3.00 * 3.00² / 6 = 4.5 ft³

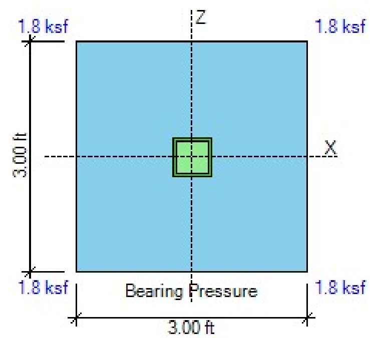
- Footing is in full bearing. Soil pressures are as follows:

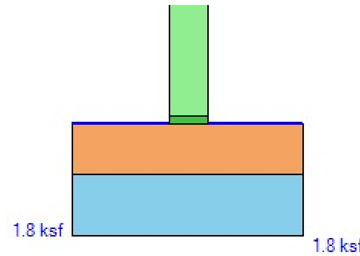
P1 = *P * (1/ A + Z-ecc / Sx + X-ecc / Sz) =* 16.6 * (1 / 9.0 + 0.00 / 4.5 + 0.00 / 4.5) = 1.85 ksf

P2 = *P * (1/ A - Z-ecc / Sx + X-ecc / Sz) =* 16.6 * (1 / 9.0 - 0.00 / 4.5 + 0.00 / 4.5) = 1.85 ksf

P3 = *P * (1/ A - Z-ecc / Sx - X-ecc / Sz) =* 16.6 * (1 / 9.0 - 0.00 / 4.5 - 0.00 / 4.5) = 1.85 ksf

P4 = *P * (1/ A + Z-ecc / Sx - X-ecc / Sz) =* 16.6 * (1 / 9.0 + 0.00 / 4.5 - 0.00 / 4.5) = 1.85 ksf





SLIDING CALCULATIONS (Comb: 0.6D+0.6W)

Internal friction angle = 28.0 deg

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

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

X-Passive force = *Pressure * Thick * Width =* 0.16 * 8.0 / 12 * 3.00 = 0.3 kip

Z-Passive force = *Pressure * Thick * Length =* 0.16 * 8.0 / 12 * 3.00 = 0.3 kip

Friction force = *Resisting force * Friction coeff. =* Max (0, 6.4 * 0.35) = 2.2 kip

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

- Sliding safety factor X-X = $\dfrac{\text{X-Passive force + Friction}}{\text{X-Horizontal load}} = \dfrac{1.00 * 0.3 + 1.00 * 2.2}{0.0}$ = 25.49 > 1.50 OK

- Sliding safety factor Z-Z = $\dfrac{\text{Z-Passive force + Friction}}{\text{Z-Horizontal load}} = \dfrac{1.00 * 0.3 + 1.00 * 2.2}{0.0}$ = 25.49 > 1.50 OK

UPLIFT CALCULATIONS (Comb: 0.6D+0.6W)

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

ONE-WAY SHEAR CALCULATIONS (Comb: 1.2D+1.6L+0.3S)

Concrete f'c = 2.5 ksi Steel fy = 60.0 ksi Soil density = 110 pcf

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

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

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

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

$\phi V_{cx} = 8 * \rho^{1/3} * \sqrt{f'c} * Width * d$ = 8 * $(0.0047)^{1/3}$ * $\sqrt{2500}$ * 3.0 * 12 * 4.8 / 1000 = 8.7 kip ACI 22.5.5.1

$\phi V_{cz} = 8 * \rho^{1/3} * \sqrt{f'c} * Length * d$ = 8 * $(0.0052)^{1/3}$ * $\sqrt{2500}$ * 3.0 * 12 * 4.3 / 1000 = 8.1 kip

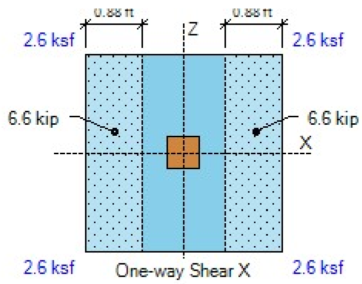
- Shear forces calculated as the volume of the bearing pressures under the effective areas:

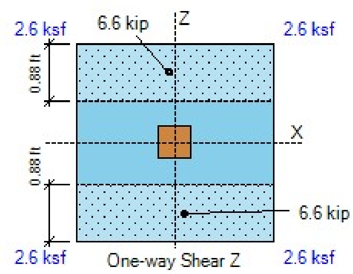
One-way shear Vux (- Side) = 6.6 kip < 8.7 kip OK

One-way shear Vux (+ Side) = 6.6 kip < 8.7 kip OK

One-way shear Vuz (- Side) = 6.6 kip < 8.1 kip OK

One-way shear Vuz (+ Side) = 6.6 kip < 8.1 kip OK





FLEXURE CALCULATIONS (Comb: 1.2D+1.6L+0.3S)

Plain ϕMnx = *5 * ϕ * √(f'c) * L * Thick² / 6* = 5 * 0.60 * √(2500) * 3.00 * 8.0² / 6 / 1000 = 1.3 k-ft — ACI Eq. (14.5.2.1a)

Plain ϕMnz = *5 * ϕ * √(f'c) * W * Thick² / 6* = 5 * 0.60 * √(2500) * 3.00 * 8.0² / 6 / 1000 = 1.3 k-ft

- Top Bars

No Top Reinforcement Provided at the Footing

Use Plain Concrete Flexural Strength at Top

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

Top moment -Mux (- Side) = 0.0 k-ft < 4.8 k-ft OK

Top moment -Mux (+ Side) = 0.0 k-ft < 4.8 k-ft OK

Top moment -Muz (- Side) = 0.0 k-ft < 4.8 k-ft OK

Top moment -Muz (+ Side) = 0.0 k-ft < 4.8 k-ft OK

- Bottom Bars

Use 4 #4 Z-Bars — *ρ = As / b d* = 0.8 / (3.00 * 12 * 4.3) = 0.0052 — q = 0.0052 * 60 / 2.5 = 0.125

Use 4 #4 X-Bars — *ρ = As / b d* = 0.8 / (3.00 * 12 * 4.8) = 0.0047 — q = 0.0047 * 60 / 2.5 = 0.112

β = L / W = 3.00 / 3.00 = 1.00 — *γs = 2 * β / (β + 1)* = 2 * 1.00 / (1.00 + 1) = 1.00 — ACI 13.3.3.3

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

ϕMnx = 0.90 * 3.00 * 12 * 4.3² * 2.5 * 0.125 * (1 - 0.59 * 0.125) = 14.2 k-ft

ϕMnz = 0.90 * 3.00 * 12 * 4.8² * 2.5 * 0.112 / 1.00 * (1 - 0.59 * 0.112 / 1.00) = 16.0 k-ft

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

Bottom moment Mux (- Side) = 5.9 k-ft < 14.2 k-ft OK — ratio = 0.41

Bottom moment Mux (+ Side) = 5.9 k-ft < 14.2 k-ft OK — ratio = 0.41

Bottom moment Muz (- Side) = 5.9 k-ft < 16.0 k-ft OK — ratio = 0.37

Bottom moment Muz (+ Side) = 5.9 k-ft < 16.0 k-ft OK — ratio = 0.37

X-As min = *0.0018 * Width * Thick* = 0.0018 * 3.00 * 12 * 8.0 = 0.5 in² < 0.8 in² OK — ACI 8.6.1.1

Z-As min = *0.0018 * Length * Thick* = 0.0018 * 3.00 * 12 * 8.0 = 0.5 in² < 0.8 in² OK — ACI 8.6.1.1

X-As max for fy/Es + 0.003 tension strain = 1.81 in² > 0.80 in² OK — ACI 21.2.2

Z-As max for fy/Es + 0.003 tension strain = 1.81 in² > 0.80 in² OK — ACI 21.2.2

X-Cover factor = *Min (2.5, (Cover + db /2, Spacing / 2) / db)* = Min (2.5, (3.0 + 0.50 / 2, 10.0 / 2) / 0.50) = 2.5

Straight *X-Ld = Max (12.0, 3 / 40 * fy / (f'c)½ * Grade * Size * Casting / Cover * db * ratio)* — ACI Eq. (25.4.2.4a)

X-Ld = Max (12.0, 3 / 40 * 60.0 * 1000 / (2500)½ * 1.0 * 0.8 * 1.0 / 2.5 * 0.50 * 0.37) = 12.0 in

Hooked *X-Ldh = Max (8 db, 6, 1 / 55 * fy / (f'c)½ * Confining * Location * Concrete * db^1.5) =* — ACI 25.4.3

X-Ldh = Max (8 db, 6, 1 / 55 * 60.0 * 1000 / (2500)½ * 1.0 * 1.0 * 0.8 * 0.50^1.5) = 6.0 in

-X Ld provided = *(Length - Col) / 2 + Offset - Cover* = 3.00 * 12 / 2 + 0.0 - 6.0 / 2 - 2.5 = 12.5 in > 12.0 in OK

+X Ld provided = *(Length - Col) / 2 - Offset - Cover* = 3.00 * 12 / 2 - 0.0 - 6.0 / 2 - 2.5 = 12.5 in > 12.0 in OK

Z-Cover factor = *Min (2.5, (Cover + db /2, Spacing / 2) / db)* = Min (2.5, (3.0 + 0.50 / 2, 10.0 / 2) / 0.50) = 2.5

Straight *Z-Ld = Max (12.0, 3 / 40 * fy / (f'c)½ * Grade * Size * Casting / Cover * db * ratio)* ACI Eq. (25.4.2.4a)

Z-Ld = Max (12.0, 3 / 40 * 60.0 * 1000 / (2500)½ * 1.0 * 0.8 * 1.0 / 2.5 * 0.50 * 0.37) = 12.0 in

Hooked *Z-Ldh = Max (8 db, 6, 1 / 55 * fy / (f'c)½ * Confining * Location * Concrete * db^1.5) =* ACI 25.4.3

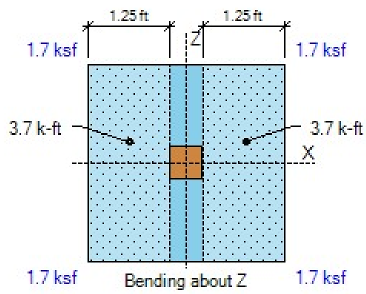
Z-Ldh = Max (8 db, 6, 1 / 55 * 60.0 * 1000 / (2500)½ * 1.0 * 1.0 * 0.8 * 0.50^1.5) = 6.0 in

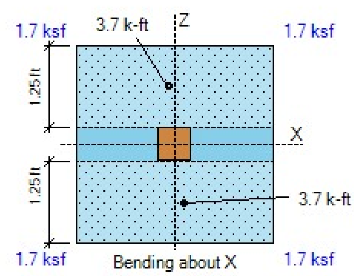
-Z Ld provided = *(Width - Col) / 2 + Offset - Cover =* 3.00 * 12 / 2 + 0.0 - 6.0 / 2 - 2.5 = 12.5 in > 12.0 in OK

+Z Ld provided = *(Width - Col) / 2 - Offset - Cover =* 3.00 * 12 / 2 - 0.0 - 6.0 / 2 - 2.5 = 12.5 in > 12.0 in OK

X-bar spacing = 10.0 in < Min (3 * t, 18.0) = 18.0 in OK ACI 7.7.2.3

Z-bar spacing = 10.0 in < Min (3 * t, 18.0) = 18.0 in OK





LOAD TRANSFER CALCULATIONS (Comb: 1.2D+1.6L+0.3S)

Area *A1 = col L * col W =* 6.0 * 6.0 = 36.0 in^2

Sx = *col W * col L² / 6 =* 6.0 * 6.0² / 6 = 36.0 in^3

Sz = *col L * col W² / 6 =* 6.0 * 6.0² / 6 = 36.0 in^3

Bearing *Pbu = P / A1 + Mz / Sx + Mx / Sz =* 22.5 / 36.0 + 0.0 * 12 / 36.0 + 0.0 * 12 / 36.0 = 0.6 ksi

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

Min edge = Min (3.00 * 12 / 2 - 0.0 - 6.0 / 2, 3.00 * 12 / 2 - 0.0 - 6.0 / 2 = 15.0 in

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

A2 = Min [3.00 * 12 * 3.0 * 12, (6.0 + 2 * 15.0) * (6.0 + 2 * 15.0)] = 1296.0 in^2

Footing *ϕPnc = ϕ * 0.85 * f'c * Min [2, √(A2 / A1)] =* 0.65 * 0.85 * 2.5 * Min [2, √(1296.0 / 36.0)] = 2.8 ksi

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

Footing bearing *ϕPn = ϕPnc + ϕPns* = 2.8 + 0.0 = 2.8 ksi > 0.6 psi OK

Hooked *Ldh = Max (8 db, 6, 1 / 55 * fy / (f'c)½ * Confining * Location * Concrete * db^1.5)* ACI 25.4.3

Ldh = Max (8 db, 6, 1 / 55 * 60.0 * 1000 / (2500)½ * 1.6 * 1.0 * 0.8 * 0.75^1.5) = 17.4 in

Ld provided = *Dowel length =* 3.00 * 12 = 36.0 in > 19.4 in OK

Ldh provided = *Footing thickness - Cover =* 8.00 - 3.0 = 5.0 in < 17.4 in NG

PUNCHING SHEAR CALCULATIONS (Comb: 1.2D+1.6L+0.3S)

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

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

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

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

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

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

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

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

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

F = 22.5 + 0.07 * 110.3 / 144 - 2.0 = 20.6 kip

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

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

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

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

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

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

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

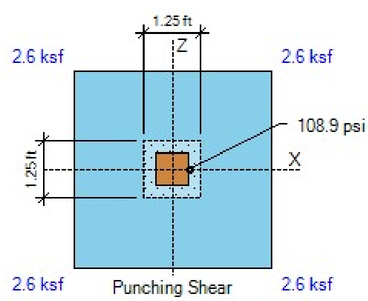
Jcx = 10.5 * 4.5³ / 6 + 10.5³ * 4.5 / 6 + 10.5² * 10.5 * 4.5 / 2 = 3632 in⁴

Stress due to P = *F / (bo * d) * 1000 =* 20.6 / (42.0 * 4.5) * 1000 = 108.9 psi

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

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

Punching stress = *P-stress + Mx-stress + Mz-stress =* 108.9 + 0.0 + 0.0 = 108.9 psi < 150.0 psi OK



LOAD TRANSFER CALCULATIONS (Comb: 1.2D+1.6L+0.3S)

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

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

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

Bearing *Pbu = P / A1 + Mz / Sx + Mx / Sz* = 22.5 / 36.0 + 0.0 * 12 / 36.0 + 0.0 * 12 / 36.0 = 0.6 ksi

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

Min edge = Min (3.00 * 12 / 2 - 0.0 - 6.0 / 2, 3.00 * 12 / 2 - 0.0 - 6.0 / 2 = 15.0 in

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

A2 = Min [3.00 * 12 * 3.0 * 12, (6.0 + 2 * 15.0) * (6.0 + 2 * 15.0)] = 1296.0 in²

Footing *ϕPnc = ϕ * 0.85 * f'c * Min [2, √(A2 / A1)]* = 0.65 * 0.85 * 2.5 * Min [2, √(1296.0 / 36.0)] = 2.8 ksi

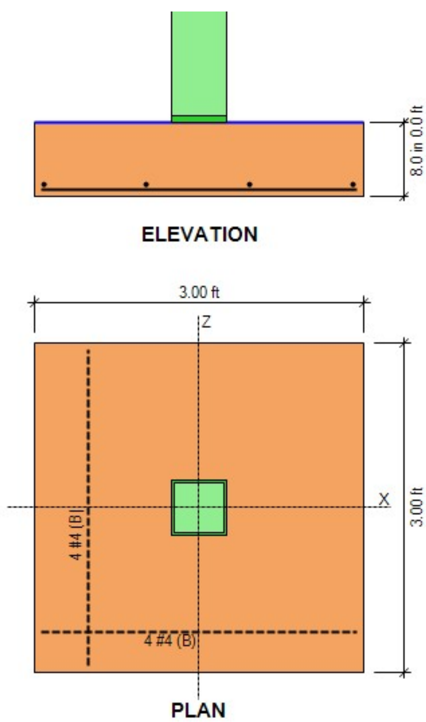
Footing *ϕPns = ϕ * As * Fy / A1* = 0.0 ksi ACI 22.8.3.2

Footing bearing *ϕPn = ϕPnc + ϕPns* = 2.8 + 0.0 = 2.8 ksi > 0.6 psi OK

DESIGN CODES

Concrete Design .... ACI 318-19

Load Combinations .... ASCE 7-22



SPREAD FOOTING DESIGN

GEOMETRY			
Footing Length (X-dir)	3.00	ft	
Footing Width (Z-dir)	3.00	ft	
Footing Thickness	8.0	in	OK
Soil Cover	0.00	ft	
Column Length (X-dir)	6.0	in	
Column Width (Z-dir)	6.0	in	
Offset (X-dir)	0.00	in	OK
Offset (Z-dir)	0.00	in	OK
Base Plate (L x W)	6.0 x 6.0	in	

SOIL PRESSURES (D+0.75L+0.525S)			
Gross Allow. Soil Pressure	2.0	ksf	
Soil Pressure at Corner 1	2.0	ksf	
Soil Pressure at Corner 2	2.0	ksf	
Soil Pressure at Corner 3	2.0	ksf	
Soil Pressure at Corner 4	2.0	ksf	
Bearing Pressure Ratio	0.99		OK
Ftg. Area in Contact with Soil	100.0	%	
X-eccentricity / Ftg. Length	0.00		OK
Z-eccentricity / Ftg. Width	0.00		OK

APPLIED LOADS

	Dead	Live	RLive	Snow	Wind	Seismic	
Axial Force P	10.9	4.3	0.0	6.2	0.0	0.0	kip
Moment about X Mx ..	0.0	0.0	0.0	0.0	0.0	0.0	k-ft
Moment about Z Mz ..	0.0	0.0	0.0	0.0	0.0	0.0	k-ft
Shear Force Vx	0.0	0.0	0.0	0.0	0.0	0.0	kip
Shear Force Vz	0.0	0.0	0.0	0.0	0.0	0.0	kip

OVERTURNING CALCULATIONS (Comb: 0.6D+0.6W)

- Overturning about X-X

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

- Resisting about X-X

- Footing weight = *0.6 * W * L * Thick * Density =* 0.6 * 3.00 * 3.00 * 8.0 / 12 * 0.15 = 0.5 kip
  Arm = *W / 2 =* 3.00 / 2 = 1.50 ft   Moment = 0.5 * 1.50 = 0.8 k-ft
- Pedestal weight = *0.6 * W * L * H * Density =* 0.6 * 6.0 / 12 * 6.0 / 12 * 0.0 * 0.15 = 0.0 kip
  Arm = *W / 2 - Offset =* 3.00 / 2 - 0.0 / 12 = 1.50 ft   Moment = 0.0 * 1.50 = 0.0 k-ft
- Soil cover = *0.6 * W * L * SC * Density =* 0.6 * (3.00 * 3.00 - 6.0 / 12 * 6.0 / 12) * 0.0 * 110 = 0.0 kip
  Arm = *W / 2 =* 3.00 / 2 = 1.50 ft   Moment = 0.0 * 1.50 = 0.0 k-ft
- Buoyancy = *0.6 * W * L * γ * (SC + Thick - WT) =* 0.6 * 3.00 * 3.00 * 62 * (0.67) = -0.2 kip
  Arm = *W / 2 =* 3.00 / 2 = 1.50 ft   Moment = 0.2 * 1.50 = -0.3 k-ft
- Axial force P = 0.6 * 10.9 + 0.6 * 0.0 = 6.5 kip
  Arm = *W / 2 - Offset =* 3.00 / 2 - 0.0 / 12 = 1.50 ft   Moment = 6.5 * 1.50 = 9.8 k-ft
- Resisting moment X-X = 0.8 + 0.0 + 0.0 + 9.8 + -0.3 = 10.3 k-ft

- Overturning safety factor X-X = $\frac{\text{Resisting moment}}{\text{Overturning moment}} = \frac{10.3}{0.0}$ = 99.99 > 1.50 OK

- Overturning about Z-Z
- Moment Mz = 0.6 * 0.0 + 0.6 * 0.0 = 0.0 k-ft
- Shear Force Vx = 0.6 * 0.0 + 0.6 * 0.0 = 0.0 kip
  Arm = 0.00 + 8.0 / 12 = 0.67 ft Moment = 0.0 * 0.67 = 0.0 k-ft
- Passive Force = 0.0 kip Arm = 0.27 ft Moment = 0.0 k-ft
- Overturning moment Z-Z = 0.0 + 0.0 = 0.0 k-ft

- Resisting about Z-Z
- Footing weight = *0.6 * W * L * Thick * Density =* 0.6 * 3.00 * 3.00 * 8.0 / 12 * 0.15 = 0.5 kip
  Arm = *L / 2 =* 3.00 / 2 = 1.50 ft Moment = 0.5 * 1.50 = 0.8 k-ft
- Pedestal weight = *0.6 * W * L * H * Density =* 0.6 * 6.0 / 12 * 6.0 / 12 * 0.0 * 0.15 = 0.0 kip
  Arm = *L / 2 - Offset =* 3.00 / 2 - 0.0 / 12 = 1.50 ft Moment = 0.0 * 1.50 = 0.0 k-ft
- Soil cover = *0.6 * W * L * SC * Density =* 0.6 * (3.00 * 3.00 - 6.0 / 12 * 6.0 / 12) * 0.0 * 110 = 0.0 kip
  Arm = *L / 2 =* 3.00 / 2 = 1.50 ft Moment = 0.0 * 1.50 = 0.0 k-ft
- Buoyancy = *0.6 * W * L * γ * (SC + Thick - WT) =* 0.6 * 3.00 * 3.00 * 62 * (0.67) = -0.2 kip
  Arm = *L / 2 =* 3.00 / 2 = 1.50 ft Moment = 0.2 * 1.50 = -0.3 k-ft
- Axial force P = 0.6 * 10.9 + 0.6 * 0.0 = 6.5 kip
  Arm = *L / 2 - Offset =* 3.00 / 2 - 0.0 / 12 = 1.50 ft Moment = 6.5 * 1.50 = 9.8 k-ft
- Resisting moment Z-Z = 0.8 + 0.0 + 0.0 + 9.8 + -0.3 = 10.3 k-ft
- Overturning safety factor Z-Z = $\frac{\textit{Resisting moment}}{\textit{Overturning moment}} = \frac{10.3}{0.0}$ = 99.99 > 1.50 OK

SOIL BEARING PRESSURES (Comb: D+0.75L+0.525S)

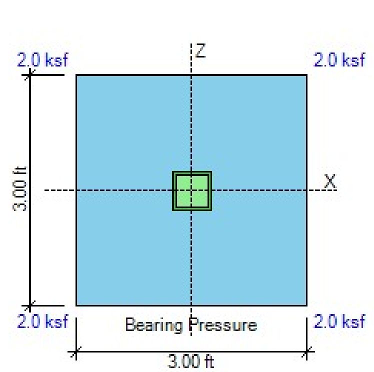
Overturning moment X-X = 0.0 + 0.0 = 0.0 k-ft
Resisting moment X-X = 1.4 + 0.0 + 0.0 + -0.6 + 26.1 = 26.9 k-ft
Overturning moment Z-Z = 0.0 + 0.0 = 0.0 k-ft
Resisting moment Z-Z = 1.4 + 0.0 + 0.0 + -0.6 + 26.1 = 26.9 k-ft
Resisting force = *Footing + Pedestal + Soil - Buoyancy + P =* 0.9 + 0.0 + 0.0 - 0.4 + 17.4 = 17.9 kip
X-coordinate of resultant from maximum bearing corner:

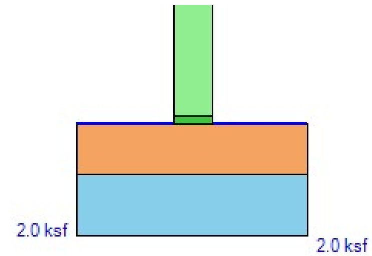
$$Xp = \frac{\textit{Z-Resisting moment - Z-Overturning moment}}{\textit{Resisting force}} = \frac{26.9 - 0.0}{17.9} = 1.50 \text{ ft}$$

Z-coordinate of resultant from maximum bearing corner:

$$Zp = \frac{\textit{X-Resisting moment - X-Overturning moment}}{\textit{Resisting force}} = \frac{26.9 - 0.0}{17.9} = 1.50 \text{ ft}$$

X-ecc = *Length / 2 - Xp =*3.00 / 2 - 1.50 = 0.00 ft
Z-ecc = *Width / 2 - Zp =* 3.00 / 2 - 1.50 = 0.00 ft
Area = *Width * Length =* 3.00 * 3.00 = 9.0 ft²
Sx = *Length * Width² / 6 =* 3.00 * 3.00² / 6 = 4.5 ft³
Sz = *Width * Length² / 6 =* 3.00 * 3.00² / 6 = 4.5 ft³
- Footing is in full bearing. Soil pressures are as follows:
P1 = *P * (1/ A + Z-ecc / Sx + X-ecc / Sz) =* 17.9 * (1 / 9.0 + 0.00 / 4.5 + 0.00 / 4.5) = 1.99 ksf
P2 = *P * (1/ A - Z-ecc / Sx + X-ecc / Sz) =* 17.9 * (1 / 9.0 - 0.00 / 4.5 + 0.00 / 4.5) = 1.99 ksf
P3 = *P * (1/ A - Z-ecc / Sx - X-ecc / Sz) =* 17.9 * (1 / 9.0 - 0.00 / 4.5 - 0.00 / 4.5) = 1.99 ksf
P4 = *P * (1/ A + Z-ecc / Sx - X-ecc / Sz) =* 17.9 * (1 / 9.0 + 0.00 / 4.5 - 0.00 / 4.5) = 1.99 ksf





SLIDING CALCULATIONS (Comb: 0.6D+0.6W)

Internal friction angle = 28.0 deg

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

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

X-Passive force = *Pressure * Thick * Width =* 0.16 * 8.0 / 12 * 3.00 = 0.3 kip

Z-Passive force = *Pressure * Thick * Length =* 0.16 * 8.0 / 12 * 3.00 = 0.3 kip

Friction force = *Resisting force * Friction coeff. =* Max (0, 6.9 * 0.35) = 2.4 kip

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

- Sliding safety factor X-X = $\dfrac{\textit{X-Passive force + Friction}}{\textit{X-Horizontal load}} = \dfrac{1.00 * 0.3 + 1.00 * 2.4}{0.0}$ = 27.17 > 1.50 OK

- Sliding safety factor Z-Z = $\dfrac{\textit{Z-Passive force + Friction}}{\textit{Z-Horizontal load}} = \dfrac{1.00 * 0.3 + 1.00 * 2.4}{0.0}$ = 27.17 > 1.50 OK

UPLIFT CALCULATIONS (Comb: 0.6D+0.6W)

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

ONE-WAY SHEAR CALCULATIONS (Comb: 1.2D+1.6L+0.3S)

Concrete f'c = 2.5 ksi Steel fy = 60.0 ksi Soil density = 110 pcf

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

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

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

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

$\phi Vcx = 8 * \rho^{1/3} * \sqrt{f'c} * Width * d$ = 8 * (0.0047)⅓ * √(2500) * 3.0 * 12 * 4.8 / 1000 = 8.7 kip ACI 22.5.5.1

$\phi Vcz = 8 * \rho^{1/3} * \sqrt{f'c} * Length * d$ = 8 * (0.0052)⅓ * √(2500) * 3.0 * 12 * 4.3 / 1000 = 8.1 kip

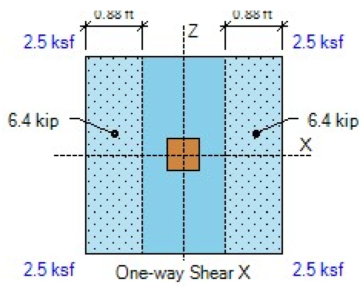
- Shear forces calculated as the volume of the bearing pressures under the effective areas:

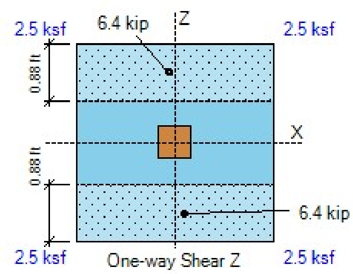
One-way shear Vux (- Side) = 6.4 kip < 8.7 kip OK

One-way shear Vux (+ Side) = 6.4 kip < 8.7 kip OK

One-way shear Vuz (- Side) = 6.4 kip < 8.1 kip OK

One-way shear Vuz (+ Side) = 6.4 kip < 8.1 kip OK





FLEXURE CALCULATIONS (Comb: 1.2D+1.6L+0.3S)

Plain ϕMnx = *5 * ϕ * √(f'c) * L * Thick² / 6* = 5 * 0.60 * √(2500) * 3.00 * 8.0² / 6 / 1000 = 1.3 k-ft — ACI Eq. (14.5.2.1a)

Plain ϕMnz = *5 * ϕ * √(f'c) * W * Thick² / 6* = 5 * 0.60 * √(2500) * 3.00 * 8.0² / 6 / 1000 = 1.3 k-ft

- Top Bars

No Top Reinforcement Provided at the Footing

Use Plain Concrete Flexural Strength at Top

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

Top moment -Mux (- Side) = 0.0 k-ft < 4.8 k-ft OK

Top moment -Mux (+ Side) = 0.0 k-ft < 4.8 k-ft OK

Top moment -Muz (- Side) = 0.0 k-ft < 4.8 k-ft OK

Top moment -Muz (+ Side) = 0.0 k-ft < 4.8 k-ft OK

- Bottom Bars

Use 4 #4 Z-Bars — $\rho = As / b\, d$ = 0.8 / (3.00 * 12 * 4.3) = 0.0052 — q = 0.0052 * 60 / 2.5 = 0.125

Use 4 #4 X-Bars — $\rho = As / b\, d$ = 0.8 / (3.00 * 12 * 4.8) = 0.0047 — q = 0.0047 * 60 / 2.5 = 0.112

$\beta = L / W$ = 3.00 / 3.00 = 1.00 — $\gamma s = 2 * \beta / (\beta + 1)$ = 2 * 1.00 / (1.00 + 1) = 1.00 — ACI 13.3.3.3

Bending strength $\phi Mn = \phi * b * d^2 * f'c * q * (1 - 0.59 * q)$ — ACI 22.2.2

ϕMnx = 0.90 * 3.00 * 12 * 4.3² * 2.5 * 0.125 * (1 - 0.59 * 0.125) = 14.2 k-ft

ϕMnz = 0.90 * 3.00 * 12 * 4.8² * 2.5 * 0.112 / 1.00 * (1 - 0.59 * 0.112 / 1.00) = 16.0 k-ft

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

Bottom moment Mux (- Side) = 5.7 k-ft < 14.2 k-ft OK ratio = 0.40

Bottom moment Mux (+ Side) = 5.7 k-ft < 14.2 k-ft OK ratio = 0.40

Bottom moment Muz (- Side) = 5.7 k-ft < 16.0 k-ft OK ratio = 0.36

Bottom moment Muz (+ Side) = 5.7 k-ft < 16.0 k-ft OK ratio = 0.36

X-As min = *0.0018 * Width * Thick* = 0.0018 * 3.00 * 12 * 8.0 = 0.5 in² < 0.8 in² OK — ACI 8.6.1.1

Z-As min = *0.0018 * Length * Thick* = 0.0018 * 3.00 * 12 * 8.0 = 0.5 in² < 0.8 in² OK — ACI 8.6.1.1

X-As max for fy/Es + 0.003 tension strain = 1.81 in² > 0.80 in² OK — ACI 21.2.2

Z-As max for fy/Es + 0.003 tension strain = 1.81 in² > 0.80 in² OK — ACI 21.2.2

X-Cover factor = *Min (2.5, (Cover + db /2, Spacing / 2) / db)* = Min (2.5, (3.0 + 0.50 / 2, 10.0 / 2) / 0.50) = 2.5

Straight *X-Ld = Max (12.0, 3 / 40 * fy / (f'c)½ * Grade * Size * Casting / Cover * db * ratio)* — ACI Eq. (25.4.2.4a)

X-Ld = Max (12.0, 3 / 40 * 60.0 * 1000 / (2500)½ * 1.0 * 0.8 * 1.0 / 2.5 * 0.50 * 0.36) = 12.0 in

Hooked *X-Ldh = Max (8 db, 6, 1 / 55 * fy / (f'c)½ * Confining * Location * Concrete * db^1.5) =* — ACI 25.4.3

X-Ldh = Max (8 db, 6, 1 / 55 * 60.0 * 1000 / (2500)½ * 1.0 * 1.0 * 0.8 * 0.50^1.5) = 6.0 in

-X Ld provided = *(Length - Col) / 2 + Offset - Cover* = 3.00 * 12 / 2 + 0.0 - 6.0 / 2 - 2.5 = 12.5 in > 12.0 in OK

+X Ld provided = *(Length - Col) / 2 - Offset - Cover* = 3.00 * 12 / 2 - 0.0 - 6.0 / 2 - 2.5 = 12.5 in > 12.0 in OK

Z-Cover factor = *Min (2.5, (Cover + db /2, Spacing / 2) / db) =* Min (2.5, (3.0 + 0.50 / 2, 10.0 / 2) / 0.50) = 2.5

Straight *Z-Ld = Max (12.0, 3 / 40 * fy / (f'c)½ * Grade * Size * Casting / Cover * db * ratio)* ACI Eq. (25.4.2.4a)

Z-Ld = Max (12.0, 3 / 40 * 60.0 * 1000 / (2500)½ * 1.0 * 0.8 * 1.0 / 2.5 * 0.50 * 0.36) = 12.0 in

Hooked *Z-Ldh = Max (8 db, 6, 1 / 55 * fy / (f'c)½ * Confining * Location * Concrete * db^1.5) =* ACI 25.4.3

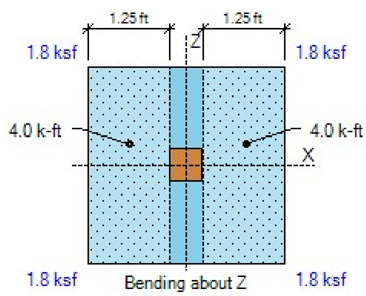
Z-Ldh = Max (8 db, 6, 1 / 55 * 60.0 * 1000 / (2500)½ * 1.0 * 1.0 * 0.8 * 0.50^1.5) = 6.0 in

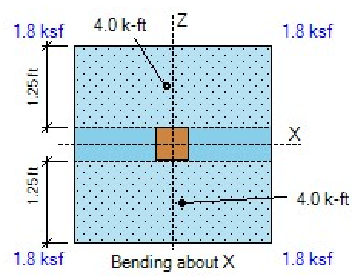
-Z Ld provided = *(Width - Col) / 2 + Offset - Cover =* 3.00 * 12 / 2 + 0.0 - 6.0 / 2 - 2.5 = 12.5 in > 12.0 in OK

+Z Ld provided =*(Width - Col) / 2 - Offset - Cover =* 3.00 * 12 / 2 - 0.0 - 6.0 / 2 - 2.5 = 12.5 in > 12.0 in OK

X-bar spacing = 10.0 in < Min (3 * t, 18.0) = 18.0 in OK ACI 7.7.2.3

Z-bar spacing = 10.0 in < Min (3 * t, 18.0) = 18.0 in OK





LOAD TRANSFER CALCULATIONS (Comb: 1.2D+1.6L+0.3S)

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

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

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

Bearing *Pbu = P / A1 + Mz / Sx + Mx / Sz =* 21.8 / 36.0 + 0.0 * 12 / 36.0 + 0.0 * 12 / 36.0 = 0.6 ksi

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

Min edge = Min (3.00 * 12 / 2 - 0.0 - 6.0 / 2, 3.00 * 12 / 2 - 0.0 - 6.0 / 2 = 15.0 in

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

A2 = Min [3.00 * 12 * 3.0 * 12, (6.0 + 2 * 15.0) * (6.0 + 2 * 15.0)] = 1296.0 in²

Footing *ϕPnc = ϕ * 0.85 * f'c * Min [2, √(A2 / A1)] =* 0.65 * 0.85 * 2.5 * Min [2, √(1296.0 / 36.0)] = 2.8 ksi

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

Footing bearing *ϕPn = ϕPnc + ϕPns =* 2.8 + 0.0 = 2.8 ksi > 0.6 psi OK

Hooked *Ldh = Max (8 db, 6, 1 / 55 * fy / (f'c)½ * Confining * Location * Concrete * db^1.5)* ACI 25.4.3

Ldh = Max (8 db, 6, 1 / 55 * 60.0 * 1000 / (2500)½ * 1.6 * 1.0 * 0.8 * 0.75^1.5) = 17.4 in

Ld provided = *Dowel length =* 3.00 * 12 = 36.0 in > 18.8 in OK

Ldh provided = *Footing thickness - Cover =* 8.00 - 3.0 = 5.0 in < 17.4 in NG

PUNCHING SHEAR CALCULATIONS (Comb: 1.2D+1.6L+0.3S)

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

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

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

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

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

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

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

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

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

F = 21.8 + 0.07 * 110.3 / 144 - 1.9 = 20.0 kip

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

$$\gamma vx \text{ factor} = 1 - \frac{1}{1 + (2/3)\sqrt{(b2 / b1)}} = 1 - \frac{1}{1 + (2/3)\sqrt{(10.5 / 10.5)}} = 0.40$$ 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 in X2x = *b2 / 2 =* 10.5 / 2 = 5.3 in

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

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

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

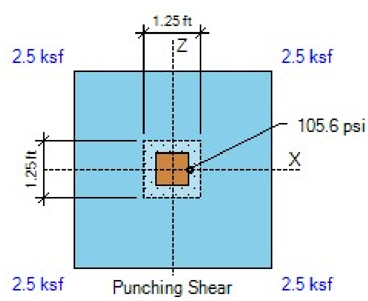
Jcx = 10.5 * 4.5³ / 6 + 10.5³ * 4.5 / 6 + 10.5² * 10.5 * 4.5 / 2 = 3632 in⁴

Stress due to P = *F / (bo * d) * 1000 =* 20.0 / (42.0 * 4.5) * 1000 = 105.6 psi

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

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

Punching stress = *P-stress + Mx-stress + Mz-stress =* 105.6 + 0.0 + 0.0 = 105.6 psi < 150.0 psi OK



LOAD TRANSFER CALCULATIONS (Comb: 1.2D+1.6L+0.3S)

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

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

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

Bearing *Pbu = P / A1 + Mz / Sx + Mx / Sz* = 21.8 / 36.0 + 0.0 * 12 / 36.0 + 0.0 * 12 / 36.0 = 0.6 ksi

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

Min edge = Min (3.00 * 12 / 2 - 0.0 - 6.0 / 2, 3.00 * 12 / 2 - 0.0 - 6.0 / 2 = 15.0 in

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

A2 = Min [3.00 * 12 * 3.0 * 12, (6.0 + 2 * 15.0) * (6.0 + 2 * 15.0)] = 1296.0 in²

Footing *ϕPnc = ϕ * 0.85 * f'c * Min [2, √(A2 / A1)]* = 0.65 * 0.85 * 2.5 * Min [2, √(1296.0 / 36.0)] = 2.8 ksi

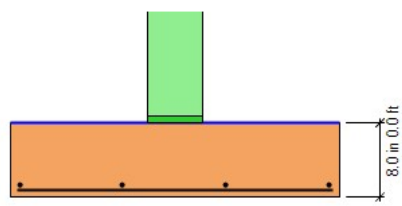
Footing *ϕPns = ϕ * As * Fy / A1* = 0.0 ksi ACI 22.8.3.2

Footing bearing *ϕPn = ϕPnc + ϕPns* = 2.8 + 0.0 = 2.8 ksi > 0.6 psi OK

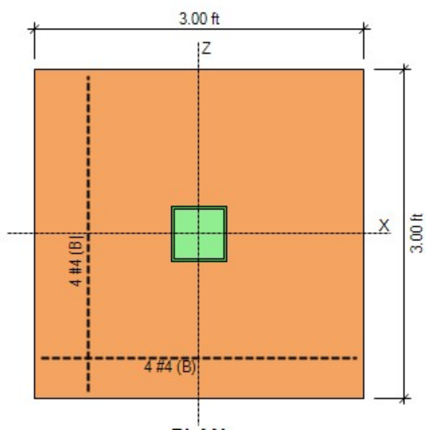
DESIGN CODES

Concrete Design .... ACI 318-19

Load Combinations .... ASCE 7-22



ELEVATION



PLAN

ASDIP Foundation 5.6.0.0 **SPREAD FOOTING DESIGN** www.asdipsoft.com

GEOMETRY

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

SOIL PRESSURES (D+0.75L+0.525S)

Gross Allow. Soil Pressure	2.0	ksf	
Soil Pressure at Corner 1	2.0	ksf	
Soil Pressure at Corner 2	2.0	ksf	
Soil Pressure at Corner 3	2.0	ksf	
Soil Pressure at Corner 4	2.0	ksf	
Bearing Pressure Ratio	0.98		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	13.8	9.3	0.0	5.0	0.0	0.0	kip
Moment about X Mx ..	0.0	0.0	0.0	0.0	0.0	0.0	k-ft
Moment about Z Mz ..	0.0	0.0	0.0	0.0	0.0	0.0	k-ft
Shear Force Vx	0.0	0.0	0.0	0.0	0.0	0.0	kip
Shear Force Vz	0.0	0.0	0.0	0.0	0.0	0.0	kip

OVERTURNING CALCULATIONS (Comb: 0.6D+0.6W)

- Overturning about X-X

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

- Resisting about X-X

- Footing weight = *0.6 * W * L * Thick * Density =* 0.6 * 3.50 * 3.50 * 8.0 / 12 * 0.15 = 0.7 kip
  Arm = *W / 2 =* 3.50 / 2 = 1.75 ft   Moment = 0.7 * 1.75 = 1.3 k-ft
- Pedestal weight = *0.6 * W * L * H * Density =* 0.6 * 6.0 / 12 * 6.0 / 12 * 0.0 * 0.15 = 0.0 kip
  Arm = *W / 2 - Offset =* 3.50 / 2 - 0.0 / 12 = 1.75 ft   Moment = 0.0 * 1.75 = 0.0 k-ft
- Soil cover = *0.6 * W * L * SC * Density =* 0.6 * (3.50 * 3.50 - 6.0 / 12 * 6.0 / 12) * 0.0 * 110 = 0.0 kip
  Arm = *W / 2 =* 3.50 / 2 = 1.75 ft   Moment = 0.0 * 1.75 = 0.0 k-ft
- Buoyancy = *0.6 * W * L * γ * (SC + Thick - WT) =* 0.6 * 3.50 * 3.50 * 62 * (0.67) = -0.3 kip
  Arm = *W / 2 =* 3.50 / 2 = 1.75 ft   Moment = 0.3 * 1.75 = -0.5 k-ft
- Axial force P = 0.6 * 13.8 + 0.6 * 0.0 = 8.3 kip
  Arm = *W / 2 - Offset =* 3.50 / 2 - 0.0 / 12 = 1.75 ft   Moment = 8.3 * 1.75 = 14.5 k-ft
- Resisting moment X-X = 1.3 + 0.0 + 0.0 + 14.5 + -0.5 = 15.2 k-ft

$$\text{- Overturning safety factor X-X} = \frac{\textit{Resisting moment}}{\textit{Overturning moment}} = \frac{15.2}{0.0} = 99.99 > 1.50 \quad \text{OK}$$

- Overturning about Z-Z

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

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

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

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

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

- Resisting about Z-Z

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

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

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

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

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

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

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

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

- Axial force P = 0.6 * 13.8 + 0.6 * 0.0 = 8.3 kip

Arm = *L / 2 - Offset =* 3.50 / 2 - 0.0 / 12 = 1.75 ft   Moment = 8.3 * 1.75 = 14.5 k-ft

- Resisting moment Z-Z = 1.3 + 0.0 + 0.0 + 14.5 + -0.5 = 15.2 k-ft

- Overturning safety factor Z-Z = $\dfrac{\text{Resisting moment}}{\text{Overturning moment}} = \dfrac{15.2}{0.0}$ = 99.99 > 1.50 OK

SOIL BEARING PRESSURES (Comb: D+0.75L+0.525S)

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

Resisting moment X-X = 2.1 + 0.0 + 0.0 + -0.9 + 41.0 = 42.2 k-ft

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

Resisting moment Z-Z = 2.1 + 0.0 + 0.0 + -0.9 + 41.0 = 42.2 k-ft

Resisting force = *Footing + Pedestal + Soil - Buoyancy + P =* 1.2 + 0.0 + 0.0 - 0.5 + 23.4 = 24.1 kip

X-coordinate of resultant from maximum bearing corner:

Xp = $\dfrac{\text{Z-Resisting moment - Z-Overturning moment}}{\text{Resisting force}} = \dfrac{42.2 - 0.0}{24.1}$ = 1.75 ft

Z-coordinate of resultant from maximum bearing corner:

Zp = $\dfrac{\text{X-Resisting moment - X-Overturning moment}}{\text{Resisting force}} = \dfrac{42.2 - 0.0}{24.1}$ = 1.75 ft

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

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

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

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

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

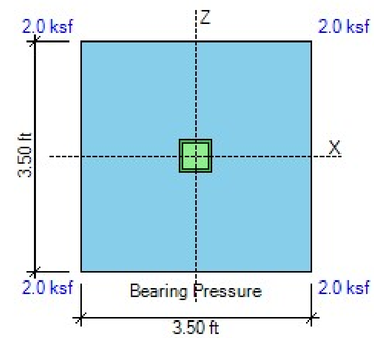
- Footing is in full bearing. Soil pressures are as follows:

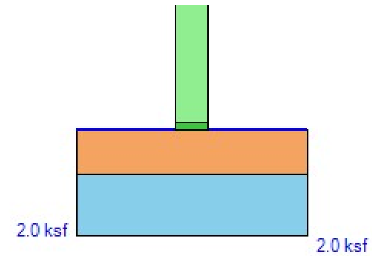
P1 = *P * (1/ A + Z-ecc / Sx + X-ecc / Sz) =* 24.1 * (1 / 12.3 + 0.00 / 7.1 + 0.00 / 7.1) = 1.97 ksf

P2 = *P * (1/ A - Z-ecc / Sx + X-ecc / Sz) =* 24.1 * (1 / 12.3 - 0.00 / 7.1 + 0.00 / 7.1) = 1.97 ksf

P3 = *P * (1/ A - Z-ecc / Sx - X-ecc / Sz) =* 24.1 * (1 / 12.3 - 0.00 / 7.1 - 0.00 / 7.1) = 1.97 ksf

P4 = *P * (1/ A + Z-ecc / Sx - X-ecc / Sz) =* 24.1 * (1 / 12.3 + 0.00 / 7.1 - 0.00 / 7.1) = 1.97 ksf





SLIDING CALCULATIONS (Comb: 0.6D+0.6W)

Internal friction angle = 28.0 deg

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

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

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

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

Friction force = *Resisting force * Friction coeff. =* Max (0, 8.7 * 0.35) = 3.0 kip

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

- Sliding safety factor X-X = $\dfrac{\text{X-Passive force + Friction}}{\text{X-Horizontal load}} = \dfrac{1.00 * 0.4 + 1.00 * 3.0}{0.0}$ = 34.18 > 1.50 OK

- Sliding safety factor Z-Z = $\dfrac{\text{Z-Passive force + Friction}}{\text{Z-Horizontal load}} = \dfrac{1.00 * 0.4 + 1.00 * 3.0}{0.0}$ = 34.18 > 1.50 OK

UPLIFT CALCULATIONS (Comb: 0.6D+0.6W)

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

ONE-WAY SHEAR CALCULATIONS (Comb: 1.2D+1.6L+0.3S)

Concrete f'c = 2.5 ksi   Steel fy = 60.0 ksi   Soil density = 110 pcf

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

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

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

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

$\phi Vcx = 8 * \rho^{1/3} * \sqrt{f'c} * Width * d$ = 8 * (0.0040)⅓ * √(2500) * 3.5 * 12 * 4.8 / 1000 = 9.7 kip   ACI 22.5.5.1

$\phi Vcz = 8 * \rho^{1/3} * \sqrt{f'c} * Length * d$ = 8 * (0.0045)⅓ * √(2500) * 3.5 * 12 * 4.3 / 1000 = 9.0 kip

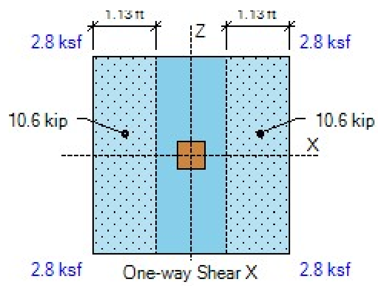
- Shear forces calculated as the volume of the bearing pressures under the effective areas:

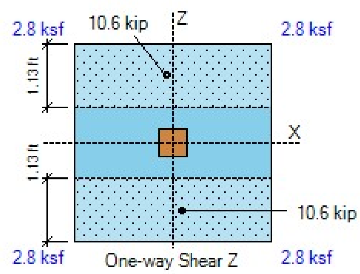
One-way shear Vux (- Side) = 10.6 kip > 9.7 kip NG

One-way shear Vux (+ Side) = 10.6 kip > 9.7 kip NG

One-way shear Vuz (- Side) = 10.6 kip > 9.0 kip NG

One-way shear Vuz (+ Side) = 10.6 kip > 9.0 kip NG





FLEXURE CALCULATIONS (Comb: 1.2D+1.6L+0.3S)

Plain ϕMnx = *5 * ϕ * √(f'c) * L * Thick² / 6* = 5 * 0.60 * √(2500) * 3.50 * 8.0² / 6 / 1000 = 1.5 k-ft ACI Eq. (14.5.2.1a)

Plain ϕMnz = *5 * ϕ * √(f'c) * W * Thick² / 6* = 5 * 0.60 * √(2500) * 3.50 * 8.0² / 6 / 1000 = 1.5 k-ft

- Top Bars

No Top Reinforcement Provided at the Footing

Use Plain Concrete Flexural Strength at Top

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

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

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

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

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

- Bottom Bars

Use 4 #4 Z-Bars *ρ = As / b d* = 0.8 / (3.50 * 12 * 4.3) = 0.0045 q = 0.0045 * 60 / 2.5 = 0.108

Use 4 #4 X-Bars *ρ = As / b d* = 0.8 / (3.50 * 12 * 4.8) = 0.0040 q = 0.0040 * 60 / 2.5 = 0.096

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

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

ϕMnx = 0.90 * 3.50 * 12 * 4.3² * 2.5 * 0.108 * (1 - 0.59 * 0.108) = 14.3 k-ft

ϕMnz = 0.90 * 3.50 * 12 * 4.8² * 2.5 * 0.096 / 1.00 * (1 - 0.59 * 0.096 / 1.00) = 16.1 k-ft

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

Bottom moment Mux (- Side) = 10.6 k-ft < 14.3 k-ft OK ratio = 0.74

Bottom moment Mux (+ Side) = 10.6 k-ft < 14.3 k-ft OK ratio = 0.74

Bottom moment Muz (- Side) = 10.6 k-ft < 16.1 k-ft OK ratio = 0.66

Bottom moment Muz (+ Side) = 10.6 k-ft < 16.1 k-ft OK ratio = 0.66

X-As min = *0.0018 * Width * Thick* = 0.0018 * 3.50 * 12 * 8.0 = 0.6 in² < 0.8 in² OK ACI 8.6.1.1

Z-As min = *0.0018 * Length * Thick* = 0.0018 * 3.50 * 12 * 8.0 = 0.6 in² < 0.8 in² OK ACI 8.6.1.1

X-As max for fy/Es + 0.003 tension strain = 2.12 in² > 0.80 in² OK ACI 21.2.2

Z-As max for fy/Es + 0.003 tension strain = 2.12 in² > 0.80 in² OK ACI 21.2.2

X-Cover factor = *Min (2.5, (Cover + db /2, Spacing / 2) / db)* = Min (2.5, (3.0 + 0.50 / 2, 12.0 / 2) / 0.50) = 2.5

Straight *X-Ld = Max (12.0, 3 / 40 * fy / (f'c)½ * Grade * Size * Casting / Cover * db * ratio)* ACI Eq. (25.4.2.4a)

X-Ld = Max (12.0, 3 / 40 * 60.0 * 1000 / (2500)½ * 1.0 * 0.8 * 1.0 / 2.5 * 0.50 * 0.66) = 12.0 in

Hooked *X-Ldh = Max (8 db, 6, 1 / 55 * fy / (f'c)½ * Confining * Location * Concrete * db^1.5) =* ACI 25.4.3

X-Ldh = Max (8 db, 6, 1 / 55 * 60.0 * 1000 / (2500)½ * 1.0 * 1.0 * 0.8 * 0.50^1.5) = 6.0 in

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

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

Z-Cover factor = *Min (2.5, (Cover + db /2, Spacing / 2) / db) =* Min (2.5, (3.0 + 0.50 / 2, 12.0 / 2) / 0.50) = 2.5

Straight *Z-Ld = Max (12.0, 3 / 40 * fy / (f'c)½ * Grade * Size * Casting / Cover * db * ratio)* ACI Eq. (25.4.2.4a)

Z-Ld = Max (12.0, 3 / 40 * 60.0 * 1000 / (2500)½ * 1.0 * 0.8 * 1.0 / 2.5 * 0.50 * 0.66) = 12.0 in

Hooked *Z-Ldh = Max (8 db, 6, 1 / 55 * fy / (f'c)½ * Confining * Location * Concrete * db^1.5) =* ACI 25.4.3

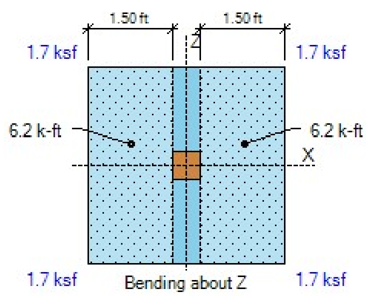
Z-Ldh = Max (8 db, 6, 1 / 55 * 60.0 * 1000 / (2500)½ * 1.0 * 1.0 * 0.8 * 0.50^1.5) = 6.0 in

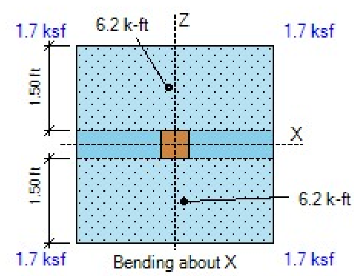
-Z Ld provided = *(Width - Col) / 2 + Offset - Cover =* 3.50 * 12 / 2 + 0.0 - 6.0 / 2 - 2.5 = 15.5 in > 12.0 in OK

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

X-bar spacing = 12.0 in < Min (3 * t, 18.0) = 18.0 in OK ACI 7.7.2.3

Z-bar spacing = 12.0 in < Min (3 * t, 18.0) = 18.0 in OK





LOAD TRANSFER CALCULATIONS (Comb: 1.2D+1.6L+0.3S)

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

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

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

Bearing *Pbu = P / A1 + Mz / Sx + Mx / Sz =* 32.9 / 36.0 + 0.0 * 12 / 36.0 + 0.0 * 12 / 36.0 = 0.9 ksi

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

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

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

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

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

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

Footing bearing *ϕPn = ϕPnc + ϕPns =* 2.8 + 0.0 = 2.8 ksi > 0.9 psi OK

Hooked *Ldh = Max (8 db, 6, 1 / 55 * fy / (f'c)½ * Confining * Location * Concrete * db^1.5)* ACI 25.4.3

Ldh = Max (8 db, 6, 1 / 55 * 60.0 * 1000 / (2500)½ * 1.6 * 1.0 * 0.8 * 0.75^1.5) = 17.4 in

Ld provided = *Dowel length =* 3.00 * 12 = 36.0 in > 28.5 in OK

Ldh provided = *Footing thickness - Cover =* 8.00 - 3.0 = 5.0 in < 17.4 in NG

PUNCHING SHEAR CALCULATIONS (Comb: 1.2D+1.6L+0.3S)

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

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

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

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

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

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

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

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

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

F = 32.9 + 0.07 * 110.3 / 144 - 2.1 = 30.9 kip

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

$$\gamma vx \text{ factor} = 1 - \frac{1}{1 + (2/3)\sqrt{(b2 / b1)}} = 1 - \frac{1}{1 + (2/3)\sqrt{(10.5 / 10.5)}} = 0.40$$ ACI Eq. (8.4.4.2.2)

ACI Eq. (8.4.2.3.2)
$$\gamma vz \text{ factor} = 1 - \frac{1}{1 + (2/3)\sqrt{(b1 / b2)}} = 1 - \frac{1}{1 + (2/3)\sqrt{(10.5 / 10.5)}} = 0.40$$

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

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

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

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

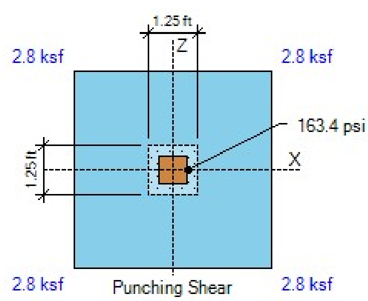
Jcx = 10.5 * 4.5³ / 6 + 10.5³ * 4.5 / 6 + 10.5² * 10.5 * 4.5 / 2 = 3632 in⁴

Stress due to P = *F / (bo * d) * 1000 =* 30.9 / (42.0 * 4.5) * 1000 = 163.4 psi

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

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

Punching stress = *P-stress + Mx-stress + Mz-stress =* 163.4 + 0.0 + 0.0 = 163.4 psi > 150.0 psi NG



LOAD TRANSFER CALCULATIONS (Comb: 1.2D+1.6L+0.3S)

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

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

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

Bearing *Pbu = P / A1 + Mz / Sx + Mx / Sz* = 32.9 / 36.0 + 0.0 * 12 / 36.0 + 0.0 * 12 / 36.0 = 0.9 ksi

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

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

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

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

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

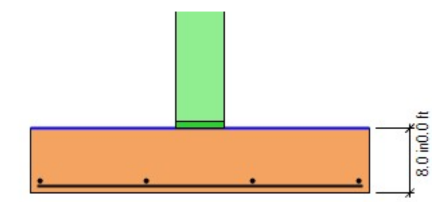
Footing *ϕPns = ϕ * As * Fy / A1* = 0.0 ksi ACI 22.8.3.2

Footing bearing *ϕPn = ϕPnc + ϕPns* = 2.8 + 0.0 = 2.8 ksi > 0.9 psi OK

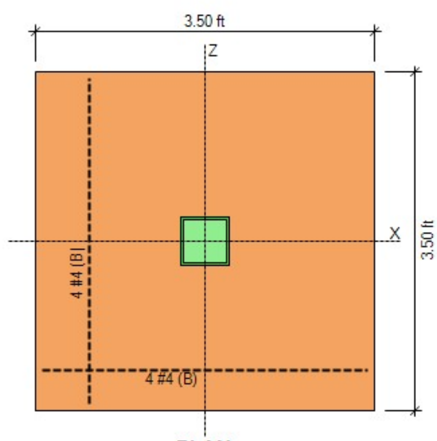
DESIGN CODES

Concrete Design .... ACI 318-19

Load Combinations .... ASCE 7-22



ELEVATION



PLAN

SPREAD FOOTING DESIGN

GEOMETRY

Footing Length (X-dir)	3.00	ft	
Footing Width (Z-dir)	3.00	ft	
Footing Thickness	8.0	in	OK
Soil Cover	0.00	ft	
Column Length (X-dir)	6.0	in	
Column Width (Z-dir)	6.0	in	
Offset (X-dir)	0.00	in	OK
Offset (Z-dir)	0.00	in	OK
Base Plate (L x W)	6.0 x 6.0	in	

SOIL PRESSURES (D+0.75L+0.525S)

Gross Allow. Soil Pressure	2.0	ksf
Soil Pressure at Corner 1	1.4	ksf
Soil Pressure at Corner 2	1.4	ksf
Soil Pressure at Corner 3	1.4	ksf
Soil Pressure at Corner 4	1.4	ksf
Bearing Pressure Ratio	0.70	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	8.1	1.7	0.0	5.2	0.0	0.0	kip
Moment about X Mx ..	0.0	0.0	0.0	0.0	0.0	0.0	k-ft
Moment about Z Mz ..	0.0	0.0	0.0	0.0	0.0	0.0	k-ft
Shear Force Vx	0.0	0.0	0.0	0.0	0.0	0.0	kip
Shear Force Vz	0.0	0.0	0.0	0.0	0.0	0.0	kip

OVERTURNING CALCULATIONS (Comb: 0.6D+0.6W)

- Overturning about X-X

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

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

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

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

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

- Resisting about X-X

- Footing weight = *0.6 * W * L * Thick * Density =* 0.6 * 3.00 * 3.00 * 8.0 / 12 * 0.15 = 0.5 kip

  Arm = *W / 2 =* 3.00 / 2 = 1.50 ft Moment = 0.5 * 1.50 = 0.8 k-ft

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

  Arm = *W / 2 - Offset =* 3.00 / 2 - 0.0 / 12 = 1.50 ft Moment = 0.0 * 1.50 = 0.0 k-ft

- Soil cover = *0.6 * W * L * SC * Density =* 0.6 * (3.00 * 3.00 - 6.0 / 12 * 6.0 / 12) * 0.0 * 110 = 0.0 kip

  Arm = *W / 2 =* 3.00 / 2 = 1.50 ft Moment = 0.0 * 1.50 = 0.0 k-ft

- Buoyancy = *0.6 * W * L * γ * (SC + Thick - WT) =* 0.6 * 3.00 * 3.00 * 62 * (0.67) = -0.2 kip

  Arm = *W / 2 =* 3.00 / 2 = 1.50 ft Moment = 0.2 * 1.50 = -0.3 k-ft

- Axial force P = 0.6 * 8.1 + 0.6 * 0.0 = 4.9 kip

  Arm = *W / 2 - Offset =* 3.00 / 2 - 0.0 / 12 = 1.50 ft Moment = 4.9 * 1.50 = 7.3 k-ft

- Resisting moment X-X = 0.8 + 0.0 + 0.0 + 7.3 + -0.3 = 7.8 k-ft

- Overturning safety factor X-X = $\dfrac{\textit{Resisting moment}}{\textit{Overturning moment}} = \dfrac{7.8}{0.0}$ = 77.63 > 1.50 OK

- Overturning about Z-Z

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

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

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

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

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

- Resisting about Z-Z

- Footing weight = *0.6 * W * L * Thick * Density =* 0.6 * 3.00 * 3.00 * 8.0 / 12 * 0.15 = 0.5 kip

  Arm = *L / 2 =* 3.00 / 2 = 1.50 ft   Moment = 0.5 * 1.50 = 0.8 k-ft

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

  Arm = *L / 2 - Offset =* 3.00 / 2 - 0.0 / 12 = 1.50 ft   Moment = 0.0 * 1.50 = 0.0 k-ft

- Soil cover = *0.6 * W * L * SC * Density =* 0.6 * (3.00 * 3.00 - 6.0 / 12 * 6.0 / 12) * 0.0 * 110 = 0.0 kip

  Arm = *L / 2 =* 3.00 / 2 = 1.50 ft   Moment = 0.0 * 1.50 = 0.0 k-ft

- Buoyancy = *0.6 * W * L * γ * (SC + Thick - WT) =* 0.6 * 3.00 * 3.00 * 62 * (0.67) = -0.2 kip

  Arm = *L / 2 =* 3.00 / 2 = 1.50 ft   Moment = 0.2 * 1.50 = -0.3 k-ft

- Axial force P = 0.6 * 8.1 + 0.6 * 0.0 = 4.9 kip

  Arm = *L / 2 - Offset =* 3.00 / 2 - 0.0 / 12 = 1.50 ft   Moment = 4.9 * 1.50 = 7.3 k-ft

- Resisting moment Z-Z = 0.8 + 0.0 + 0.0 + 7.3 + -0.3 = 7.8 k-ft

- Overturning safety factor Z-Z = $\frac{\textit{Resisting moment}}{\textit{Overturning moment}} = \frac{7.8}{0.0}$ = 77.63 > 1.50 OK

SOIL BEARING PRESSURES (Comb: D+0.75L+0.525S)

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

Resisting moment X-X = 1.4 + 0.0 + 0.0 + -0.6 + 18.2 = 18.9 k-ft

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

Resisting moment Z-Z = 1.4 + 0.0 + 0.0 + -0.6 + 18.2 = 18.9 k-ft

Resisting force = *Footing + Pedestal + Soil - Buoyancy + P =* 0.9 + 0.0 + 0.0 - 0.4 + 12.1 = 12.6 kip

X-coordinate of resultant from maximum bearing corner:

Xp = $\frac{\textit{Z-Resisting moment - Z-Overturning moment}}{\textit{Resisting force}} = \frac{18.9 - 0.0}{12.6}$ = 1.50 ft

Z-coordinate of resultant from maximum bearing corner:

Zp = $\frac{\textit{X-Resisting moment - X-Overturning moment}}{\textit{Resisting force}} = \frac{18.9 - 0.0}{12.6}$ = 1.50 ft

X-ecc = *Length / 2 - Xp =* 3.00 / 2 - 1.50 = 0.00 ft

Z-ecc = *Width / 2 - Zp =* 3.00 / 2 - 1.50 = 0.00 ft

Area = *Width * Length =* 3.00 * 3.00 = 9.0 ft²

Sx = *Length * Width² / 6 =* 3.00 * 3.00² / 6 = 4.5 ft³

Sz = *Width * Length² / 6 =* 3.00 * 3.00² / 6 = 4.5 ft³

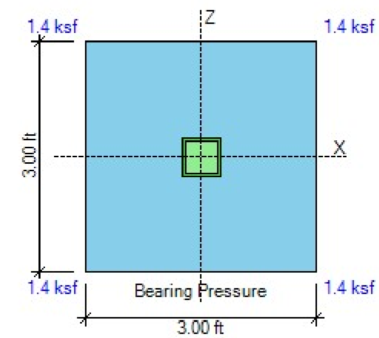
- Footing is in full bearing. Soil pressures are as follows:

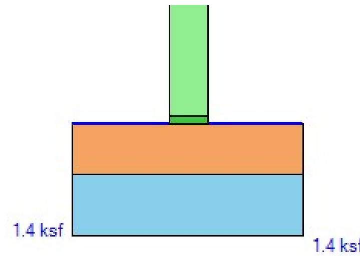
P1 = *P * (1/ A + Z-ecc / Sx + X-ecc / Sz) =* 12.6 * (1 / 9.0 + 0.00 / 4.5 + 0.00 / 4.5) = 1.40 ksf

P2 = *P * (1/ A - Z-ecc / Sx + X-ecc / Sz) =* 12.6 * (1 / 9.0 - 0.00 / 4.5 + 0.00 / 4.5) = 1.40 ksf

P3 = *P * (1/ A - Z-ecc / Sx - X-ecc / Sz) =* 12.6 * (1 / 9.0 - 0.00 / 4.5 - 0.00 / 4.5) = 1.40 ksf

P4 = *P * (1/ A + Z-ecc / Sx - X-ecc / Sz) =* 12.6 * (1 / 9.0 + 0.00 / 4.5 - 0.00 / 4.5) = 1.40 ksf





SLIDING CALCULATIONS (Comb: 0.6D+0.6W)

Internal friction angle = 28.0 deg

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

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

X-Passive force = *Pressure * Thick * Width =* 0.16 * 8.0 / 12 * 3.00 = 0.3 kip

Z-Passive force = *Pressure * Thick * Length =* 0.16 * 8.0 / 12 * 3.00 = 0.3 kip

Friction force = *Resisting force * Friction coeff. =* Max (0, 5.2 * 0.35) = 1.8 kip

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

- Sliding safety factor X-X = $\dfrac{\text{X-Passive force + Friction}}{\text{X-Horizontal load}} = \dfrac{1.00 * 0.3 + 1.00 * 1.8}{0.0}$ = 21.29 > 1.50 OK

- Sliding safety factor Z-Z = $\dfrac{\text{Z-Passive force + Friction}}{\text{Z-Horizontal load}} = \dfrac{1.00 * 0.3 + 1.00 * 1.8}{0.0}$ = 21.29 > 1.50 OK

UPLIFT CALCULATIONS (Comb: 0.6D+0.6W)

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

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

Concrete f'c = 2.5 ksi | Steel fy = 60.0 ksi | Soil density = 110 pcf

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

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

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

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

$\phi Vcx = 8 * \rho^{1/3} * \sqrt{f'c} * Width * d$ = 8 * (0.0047)⅓ * √(2500) * 3.0 * 12 * 4.8 / 1000 = 8.7 kip — ACI 22.5.5.1

$\phi Vcz = 8 * \rho^{1/3} * \sqrt{f'c} * Length * d$ = 8 * (0.0052)⅓ * √(2500) * 3.0 * 12 * 4.3 / 1000 = 8.1 kip

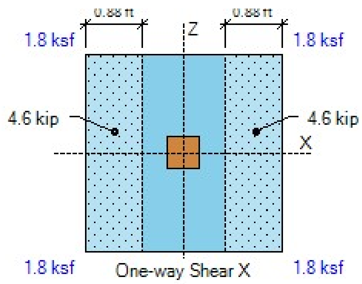
- Shear forces calculated as the volume of the bearing pressures under the effective areas:

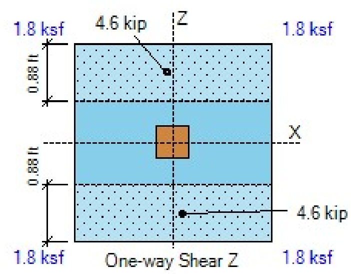
One-way shear Vux (- Side) = 4.6 kip < 8.7 kip OK

One-way shear Vux (+ Side) = 4.6 kip < 8.7 kip OK

One-way shear Vuz (- Side) = 4.6 kip < 8.1 kip OK

One-way shear Vuz (+ Side) = 4.6 kip < 8.1 kip OK





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

Plain ϕMnx = *5 * ϕ * √(f'c) * L * Thick² / 6* = 5 * 0.60 * √(2500) * 3.00 * 8.0² / 6 / 1000 = 1.3 k-ft — ACI Eq. (14.5.2.1a)

Plain ϕMnz = *5 * ϕ * √(f'c) * W * Thick² / 6* = 5 * 0.60 * √(2500) * 3.00 * 8.0² / 6 / 1000 = 1.3 k-ft

- Top Bars

No Top Reinforcement Provided at the Footing

Use Plain Concrete Flexural Strength at Top

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

Top moment -Mux (- Side) = 0.0 k-ft < 4.8 k-ft OK

Top moment -Mux (+ Side) = 0.0 k-ft < 4.8 k-ft OK

Top moment -Muz (- Side) = 0.0 k-ft < 4.8 k-ft OK

Top moment -Muz (+ Side) = 0.0 k-ft < 4.8 k-ft OK

- Bottom Bars

Use 4 #4 Z-Bars — ρ = *As / b d* = 0.8 / (3.00 * 12 * 4.3) = 0.0052 — q = 0.0052 * 60 / 2.5 = 0.125

Use 4 #4 X-Bars — ρ = *As / b d* = 0.8 / (3.00 * 12 * 4.8) = 0.0047 — q = 0.0047 * 60 / 2.5 = 0.112

β = *L / W* = 3.00 / 3.00 = 1.00 — γs = *2 * β / (β + 1)* = 2 * 1.00 / (1.00 + 1) = 1.00 — ACI 13.3.3.3

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

ϕMnx = 0.90 * 3.00 * 12 * 4.3² * 2.5 * 0.125 * (1 - 0.59 * 0.125) = 14.2 k-ft

ϕMnz = 0.90 * 3.00 * 12 * 4.8² * 2.5 * 0.112 / 1.00 * (1 - 0.59 * 0.112 / 1.00) = 16.0 k-ft

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

Bottom moment Mux (- Side) = 4.1 k-ft < 14.2 k-ft OK ratio = 0.29

Bottom moment Mux (+ Side) = 4.1 k-ft < 14.2 k-ft OK ratio = 0.29

Bottom moment Muz (- Side) = 4.1 k-ft < 16.0 k-ft OK ratio = 0.26

Bottom moment Muz (+ Side) = 4.1 k-ft < 16.0 k-ft OK ratio = 0.26

X-As min = *0.0018 * Width * Thick* = 0.0018 * 3.00 * 12 * 8.0 = 0.5 in² < 0.8 in² OK — ACI 8.6.1.1

Z-As min = *0.0018 * Length * Thick* = 0.0018 * 3.00 * 12 * 8.0 = 0.5 in² < 0.8 in² OK — ACI 8.6.1.1

X-As max for fy/Es + 0.003 tension strain = 1.81 in² > 0.80 in² OK — ACI 21.2.2

Z-As max for fy/Es + 0.003 tension strain = 1.81 in² > 0.80 in² OK — ACI 21.2.2

X-Cover factor = *Min (2.5, (Cover + db /2, Spacing / 2) / db)* = Min (2.5, (3.0 + 0.50 / 2, 10.0 / 2) / 0.50) = 2.5

Straight *X-Ld = Max (12.0, 3 / 40 * fy / (f'c)½ * Grade * Size * Casting / Cover * db * ratio)* — ACI Eq. (25.4.2.4a)

X-Ld = Max (12.0, 3 / 40 * 60.0 * 1000 / (2500)½ * 1.0 * 0.8 * 1.0 / 2.5 * 0.50 * 0.26) = 12.0 in

Hooked *X-Ldh = Max (8 db, 6, 1 / 55 * fy / (f'c)½ * Confining * Location * Concrete * db^1.5) =* — ACI 25.4.3

X-Ldh = Max (8 db, 6, 1 / 55 * 60.0 * 1000 / (2500)½ * 1.0 * 1.0 * 0.8 * 0.50^1.5) = 6.0 in

-X Ld provided = *(Length - Col) / 2 + Offset - Cover =* 3.00 * 12 / 2 + 0.0 - 6.0 / 2 - 2.5 = 12.5 in > 12.0 in OK

+X Ld provided = *(Length - Col) / 2 - Offset - Cover =* 3.00 * 12 / 2 - 0.0 - 6.0 / 2 - 2.5 = 12.5 in > 12.0 in OK

Z-Cover factor = *Min (2.5, (Cover + db /2, Spacing / 2) / db) =* Min (2.5, (3.0 + 0.50 / 2, 10.0 / 2) / 0.50) = 2.5

Straight *Z-Ld = Max (12.0, 3 / 40 * fy / (f'c)½ * Grade * Size * Casting / Cover * db * ratio)* ACI Eq. (25.4.2.4a)

Z-Ld = Max (12.0, 3 / 40 * 60.0 * 1000 / (2500)½ * 1.0 * 0.8 * 1.0 / 2.5 * 0.50 * 0.26) = 12.0 in

Hooked *Z-Ldh = Max (8 db, 6, 1 / 55 * fy / (f'c)½ * Confining * Location * Concrete * db^1.5) =* ACI 25.4.3

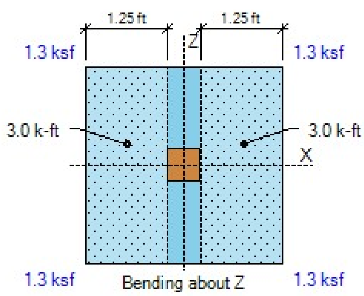
Z-Ldh = Max (8 db, 6, 1 / 55 * 60.0 * 1000 / (2500)½ * 1.0 * 1.0 * 0.8 * 0.50^1.5) = 6.0 in

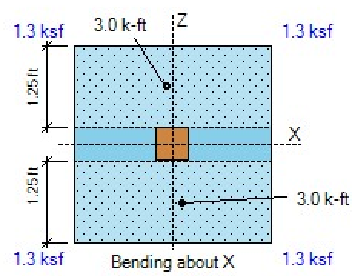
-Z Ld provided = *(Width - Col) / 2 + Offset - Cover =* 3.00 * 12 / 2 + 0.0 - 6.0 / 2 - 2.5 = 12.5 in > 12.0 in OK

+Z Ld provided =*(Width - Col) / 2 - Offset - Cover =* 3.00 * 12 / 2 - 0.0 - 6.0 / 2 - 2.5 = 12.5 in > 12.0 in OK

X-bar spacing = 10.0 in < Min (3 * t, 18.0) = 18.0 in OK ACI 7.7.2.3

Z-bar spacing = 10.0 in < Min (3 * t, 18.0) = 18.0 in OK





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

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

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

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

Bearing *Pbu = P / A1 + Mz / Sx + Mx / Sz =* 15.8 / 36.0 + 0.0 * 12 / 36.0 + 0.0 * 12 / 36.0 = 0.4 ksi

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

Min edge = Min (3.00 * 12 / 2 - 0.0 - 6.0 / 2, 3.00 * 12 / 2 - 0.0 - 6.0 / 2 = 15.0 in

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

A2 = Min [3.00 * 12 * 3.0 * 12, (6.0 + 2 * 15.0) * (6.0 + 2 * 15.0)] = 1296.0 in²

Footing *ϕPnc = ϕ * 0.85 * f'c * Min [2, √(A2 / A1)] =* 0.65 * 0.85 * 2.5 * Min [2, √(1296.0 / 36.0)] = 2.8 ksi

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

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

Hooked *Ldh = Max (8 db, 6, 1 / 55 * fy / (f'c)½ * Confining * Location * Concrete * db^1.5)* ACI 25.4.3

Ldh = Max (8 db, 6, 1 / 55 * 60.0 * 1000 / (2500)½ * 1.6 * 1.0 * 0.8 * 0.75^1.5) = 17.4 in

Ld provided = *Dowel length =* 3.00 * 12 = 36.0 in > 13.6 in OK

Ldh provided = *Footing thickness - Cover =* 8.00 - 3.0 = 5.0 in < 17.4 in NG

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

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

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

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

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

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

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

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

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

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

F = 15.8 + 0.07 * 110.3 / 144 - 1.4 = 14.4 kip

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

$$\gamma vx \text{ factor} = 1 - \frac{1}{1 + (2/3)\sqrt{b2 / b1}} = 1 - \frac{1}{1 + (2/3)\sqrt{10.5 / 10.5}} = 0.40$$ 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 in X2x = *b2 / 2 =* 10.5 / 2 = 5.3 in

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

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

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

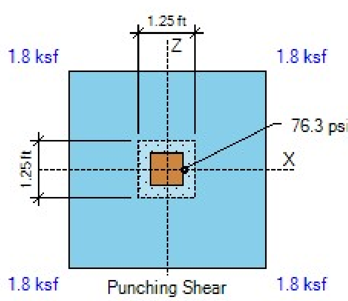
Jcx = 10.5 * 4.5³ / 6 + 10.5³ * 4.5 / 6 + 10.5² * 10.5 * 4.5 / 2 = 3632 in⁴

Stress due to P = *F / (bo * d) * 1000 =* 14.4 / (42.0 * 4.5) * 1000 = 76.3 psi

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

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

Punching stress = *P-stress + Mx-stress + Mz-stress =* 76.3 + 0.0 + 0.0 = 76.3 psi < 150.0 psi OK



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

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

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

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

Bearing *Pbu = P / A1 + Mz / Sx + Mx / Sz* = 15.8 / 36.0 + 0.0 * 12 / 36.0 + 0.0 * 12 / 36.0 = 0.4 ksi

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

Min edge = Min (3.00 * 12 / 2 - 0.0 - 6.0 / 2, 3.00 * 12 / 2 - 0.0 - 6.0 / 2 = 15.0 in

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

A2 = Min [3.00 * 12 * 3.0 * 12, (6.0 + 2 * 15.0) * (6.0 + 2 * 15.0)] = 1296.0 in²

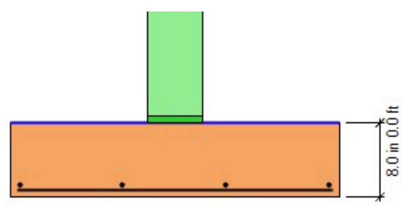
Footing *ϕPnc = ϕ * 0.85 * f'c * Min [2, √(A2 / A1)]* = 0.65 * 0.85 * 2.5 * Min [2, √(1296.0 / 36.0)] = 2.8 ksi

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

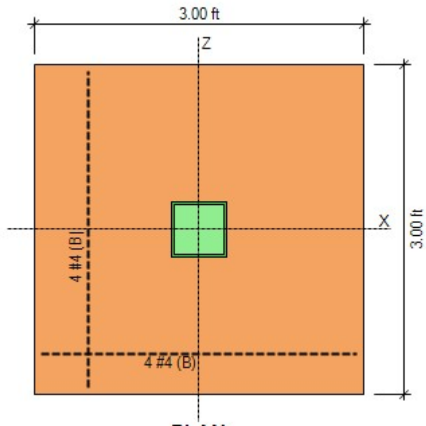
Footing bearing *ϕPn = ϕPnc + ϕPns* = 2.8 + 0.0 = 2.8 ksi > 0.4 psi OK

DESIGN CODES

Concrete Design	ACI 318-19
Load Combinations	ASCE 7-22



ELEVATION



PLAN

WIND V_{ASD} = 85 MPH V_{ULT} = 110 MPH EXP. B K_{ZT} = 1.0 SLOPE = 34°

ZONE A = ~~12.9 PSF~~ 16.0 PSF MIN

ZONE B = 8.8 PSF

ZONE C = ~~10.2 PSF~~ 16.0 PSF MIN

ZONE D = ~~7.0 PSF~~ 8.0 PSF MIN

SEISMIC S_{DS} = 0.831 R = 6.5 I_e = 1.0

$$C_s = (0.831/(6.5/1.0))/1.4 = 0.091$$

W_{ROOF} = (30 PSF × 1,954 SF) = 58,620# h = 9' h_R = 21'

W_{LEVEL2} = (45 PSF × 4,022 SF) = 180,990# h = 11' h_2 = 12'

W_{TOTAL} = 239,610#

$$V_S = 239,610\# \times 0.091 = 21,805\#$$

$$F_{ROOF} = \left[\frac{(58,620\# \times 21')}{(58,620\# \times 21') + (180,990\# \times 12')}\right] \times 21,805\# = 7,944\#$$

$$F_{LEVEL2} = \left[\frac{(180,990\# \times 12')}{(58,620\# \times 21') + (180,990\# \times 12')}\right] \times 21,805\# = 13,861\#$$

GRIDS (1 & 2)

$F_{2W} = (16psf \times 100sf) + (8.8psf \times 33sf) + (8.0psf \times 160sf) = 3{,}170\#$

$F_{2E} = 7{,}944\# \times (1{,}023sf / 1{,}954sf) = 4{,}159\#$

$F_{1W} = 3{,}170\# + (16psf \times 87sf) = 4{,}562\#$

$F_{1E} = 4{,}159\# + 13{,}861\# \times (389sf / 4{,}022sf) = 5{,}500\#$

GRIDS (3-5)

$F_{1W} = (16psf \times 265sf) = 4{,}240\#$

$F_{1E} = 13{,}861\# \times (1{,}869sf / 4{,}022sf) = 6{,}441\#$

GRIDS (7-9)

$F_{2W} = (16psf \times 86sf) + (8.8psf \times 66sf) + (8.0psf \times 140sf) = 3{,}077\#$

$F_{2E} = 7{,}944\# \times (931sf / 1{,}954sf) = 3{,}785\#$

$F_{1W} = 3{,}077\# + (16psf \times 192sf) + (8.8psf \times 24sf) = 6{,}360\#$

$F_{1E} = 3{,}785\# + 13{,}861\# \times (1{,}764sf / 4{,}022sf) = 9{,}864\#$

GRID A

F_{1W} = (16 PSF x 105 SF) + (8.8 PSF x 90 SF) + (8.0 PSF x 195 SF) = 2,594#

F_{1E} = 13,861# x (424 SF / 4,022 SF) = 1,461#

GRID C

F_{1W} = (16 PSF x 200 SF) + (8.0 PSF x 166 SF) = 4,528#

F_{1E} = 13,861# x (713 SF / 4,022 SF) = 2,457#

GRID E

F_{2W} = (16 PSF x 180 SF) + (8.0 PSF x 53 SF) = 3,304#

F_{2E} = 7,944# x (1042 SF / 1,954 SF) = 4,236#

GRID D

78% GRID E

F_{1W} = (16 PSF x 193 SF) + (8.0 PSF x 174 SF) + 2,577# = 7,057#

F_{1E} = 13,961# x (962 SF / 4,022 SF) + 3,304# = 6,619#

GRID F

22% GRID E

F_{1W} = (16 PSF x 268 SF) + 727# = 5,015#

22% GRID E

F_{1E} = 13,861# x (1,214 SF / 4,022 SF) + 932# = 5,176#

GRID H

F_{2W} = (16 PSF x 85 SF) + (8.8 PSF x 33 SF) + (8.0 PSF x 115 SF) = 2,570#

F_{2E} = 7,944# x (912 SF / 1,959 SF) = 3,708#

GRID I

F_{1W} = (16 PSF x 123 SF) + 2,570# = 9,912#

F_{1E} = 3,708# + 13,861# x (709 SF / 4,022 SF) = 6,151#

GRID 2/3 (LEVEL 2) FE = 4,159# 2 SEGMENT L=5'-7" h=9'
L=10'-7"
L=16'-2"

VE = 4,159#/16.16' = 257 PLF

USE W2 VEALLOW = 353 PLF

HOLD DOWNS

L=5'-7" TE = 257 PLF x 9' x 1.25 - 1/2(20 PSF x 3.7' x 2.79') - 1/2(12 PSF x 4.5' x 2.79') = 2,713#

USE MST48 W/ 2 STUDS TEALLOW = 3,425# x 1.4/1.6 = 2,997#

L=10'-7" TE = 257 PLF x 9' x 1.25 - 1/2(20 PSF x 18.85' x 1.25') - 1/2(12 PSF x 4.5' x 5.29') = 2,513#

USE MST48 W/ 2 STUDS TEALLOW = 3,425# x 1.4/1.6 = 2,997#

GRID 1/2 (LEVEL 1) FE = 5,500# 2 SEGMENTS L=5'-0" h=11'
L=4'-0"
L=9'-0"

VE = 5,500#/9.0' = 611 PLF

USE W8 VEALLOW = 770 PLF x (1.25 - 0.125 x 11'/4') = 693 PLF

HOLD DOWNS

TE = 611 PLF x 11' x 1.25 - 1/2(20 PSF x 7.2' x 2') - 1/2(12 PSF x 5.5' x 2') = 8,191#

USE HDU11-SDS2.5 W/ 3 DF #2 STUDS TEALLOW = 9,535# x 1.4/1.6 = 8,343#

GRID 3/5 (LEVEL 1) FE = 6,441# 10'-7" SEGMENT h=11'
13'-0" FTAO h=12'
7'-8" FTAO h=12'
4'-0" SEGMENT h=12'
44'-3"

L=10'-7" FE = 1,540#

VE = 1,540#/10.58' = 145 PLF

USE W2 VEALLOW = 353 PLF

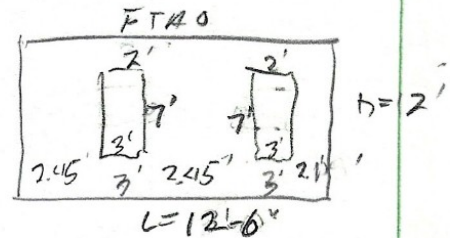
HOLD DOWNS

TE = 145 PLF x 11' x 1.25 + 2,513# - 1/2(20 PSF x 5.25' x 5.29') - 1/2(30 PSF x 8.4' x 5.29') = 3,570#

USE HDU5-SDS2.5 W/ 2 STUDS TE = 4,340# x 1.4/1.6 = 3,798

FTAO
L=13'-0"

GRID 5 (LEVEL 1) FE = 1,892#



VE = 503 PLF

USE $\boxed{W4}$ VE allow = 595 PLF

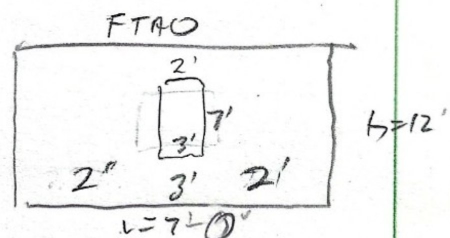
HOLD DOWNS

TE = 1,809# x 1.25 - 1/2(20 PSF x 15.25' x 6.25') - 1/2(12 PSF x 6' x 6.25') = 1,093#

$\boxed{\text{USE HDU2-SDS2.5 w/ 2 STUDS}}$ TE allow = 2,215# x 1.4/1.6 = 1,938#

FTAO
L=7'-0"

GRID 5 (LEVEL 1) FE = 1,116#



VE = 488 PLF

USE $\boxed{W4}$ VE allow = 595 PLF

HOLD DOWNS

TE = 1,913# x 1.25 - 1/2(20 PSF x 15.25' x 3.5') - 1/2(12 PSF x 6' x 3.5') = 1,732#

$\boxed{\text{USE HDU2-SDS2.5 w/ 2 STUDS}}$ TE allow = 2,215# x 1.4/1.6 = 1,938#

SEGMENT
L=4'-0"

GRID 5 (LEVEL 1) FE = 582#   L=4' h=12'

VE = 582#/4' = 146 PLF

USE $\boxed{W1}$ VE allow = 242 PLF x (1.25 - 0.125 x 12'/4') = 212 PLF

HOLD DOWNS

TE = 146 PLF x 12' x 1.25 - 1/2(20 PSF x 15' x 2') - 1/2(12 PSF x 6' x 2') = 1,746#

$\boxed{\text{USE HDU2-SDS2.5 W/ 2 STUDS}}$ TE allow = 2,215# x 1.4/1.6 = 1,938#

GRID 7/9 (LEVEL 2) FE = 3,785#   2 FTAO L = 18'-3" h = 9'

L = 21'-9" h = 9'

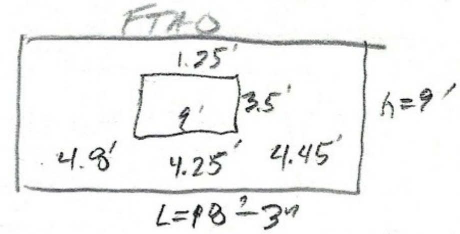
LT = 40'-0"

FTAO 18'-3"

FE = 1,727#

VE = 187 plf

USE W1 VEallow = 242 plf



HOLD DOWNS

TE = 852# x 1.25 - 1/2 (20 psf x 2.5' x 9.13') - 1/2 (12 psf x 4.5' x 9.13') = 590#

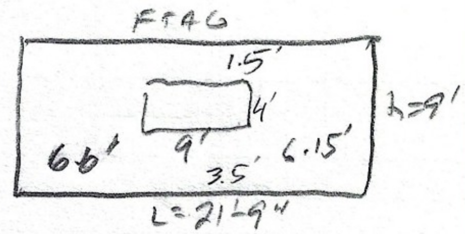
USE MST48 33 w/ 2 STUDS   TEallow = 3,900# x 0.85 x 1.4/1.6 = 2,900#

FTAO 21'-9"

FE = 2,058#

VE = 170 plf

USE W1 VEallow = 242 plf



HOLD DOWNS

TE = 852# x 1.25 - 1/2 (20 psf x 2.5' x 10.88') - 1/2 (12 psf x 4.5' x 10.88') = 499#

USE MST37 w/ 2 STUDS   TEallow = 2,140# x 1.4/1.6 = 1,873#

GRID 7-9 (LEVEL 1) FE = 9,864#   3 SEGMENTS L = 5'-0" h = 11'

L = 17'-2" h = 12'

L = 4'-0" h = 12'

LT = 26'-2"

VE = 9,864# / 26.16' = 377 plf

USE W3 VEallow = 456 plf x (1.25 - 0.125 x 12'/4') = 399 plf

HOLD DOWNS

L = 5'   TE = 377 plf x 11' x 1.25 - 1/2 (30 psf x 9' x 2.5') - 1/2 (12 psf x 9.5' x 2.5') = 4,764#

USE HDU8-SDS2.5 w/ 2 STUDS   TEallow = 5,820# x 1.4/1.6 = 5,093#

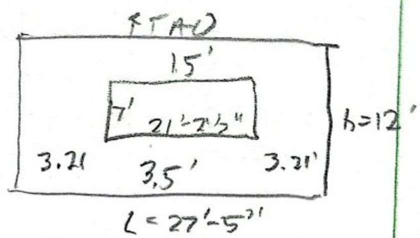
L = 17'-2"   TE = 377 plf x 12' x 1.25 - 1/2 (20 psf x 2.7' x 8.6') - 1/2 (12 psf x 6' x 8.6') = 5,113#

USE HDU8-SDS2.5 w/ 3 STUDS   TEallow = 6,580# x 1.4/1.6 = 5,758#

L = 4'-0"   TE = 377 plf x 12' x 1.25 - 1/2 (20 psf x 15.25' x 2') - 1/2 (12 psf x 6' x 2') = 5,278#

USE HDU8-SDS2.5 w/ 3 STUDS   TEallow = 6,580# x 1.4/1.6 = 5,758#

GRID A (LEVEL 1) $F_W = 3{,}584$ $F_E = 1{,}461^{\#}$



$V_W = 439$ plf
$V_E = 248$ plf

USE W2 $V_{W\,ALLOW} = 495$ plf
$V_{E\,ALLOW} = 353$ plf

HOLD DOWNS

$T_W = 1{,}122^{\#} - \tfrac{1}{2}(20\text{psf} \times 2.5' \times 13.71') - \tfrac{1}{2}(12\text{psf} \times 6' \times 13.71') = 286^{\#}$
$T_E = 635^{\#} \times 1.25 - \tfrac{1}{2}(20\text{psf} \times 2.5' \times 13.71') - \tfrac{1}{2}(12\text{psf} \times 6' \times 13.71') = -43^{\#}$

$\boxed{\text{USE HDU2-SDS2.5}}$ w/ 2 STUDS $T_{W\,ALLOW} = 2{,}215^{\#}$

GRID C (LEVEL 1) $F_W = 4{,}528^{\#}$ $F_E = 2{,}457^{\#}$ 1 SEGMENT L=17'-9"
h=12'-0"

$V_W = 4528^{\#}/17.75' = 255$ plf
$V_E = 2{,}457^{\#}/17.75' = 138$ plf

USE W1 $V_{W\,ALLOW} = 339$ plf
$V_{E\,ALLOW} = 242$ plf

HOLD DOWNS

$T_W = 255\text{plf} \times 12' - \tfrac{1}{2}(8\text{psf} \times 6' \times 8.88') = 2{,}847^{\#}$
$T_E = 138\text{plf} \times 12' \times 1.25 - \tfrac{1}{2}(8\text{psf} \times 6' \times 8.88') = 1{,}857^{\#}$

$\boxed{\text{USE HDU4-SDS2.5 W/ 2 STUDS}}$ $T_{W\,ALLOW} = 3{,}285^{\#}$
$T_{E\,ALLOW} = 3{,}285^{\#} \times 1.4/1.6 = 2{,}874^{\#}$

GRID D (LEVEL 1) $F_W = 7{,}057^{\#}$ $F_E = 6{,}619^{\#}$ 1 SEGMENT L=19'-11"
h=12'-0"

$V_W = 7{,}057^{\#}/19.92' = 354$ plf
$V_E = 6{,}619^{\#}/19.92' = 332$ plf

USE W2 $V_{W\,ALLOW} = 495$ plf
$V_{E\,ALLOW} = 353$ plf

HOLD DOWNS

$T_W = 354\text{plf} \times 12' - \tfrac{1}{2}(8\text{psf} \times 6' \times 9.94') = 4{,}009^{\#}$
$T_E = 332\text{plf} \times 12' \times 1.25 - \tfrac{1}{2}(8\text{psf} \times 6' \times 9.94') = 4{,}741^{\#}$

$\boxed{\text{USE HDU8-SDS2.5 W/ 2 STUDS}}$ $T_{W\,ALLOW} = 5{,}820^{\#}$
$T_{E\,ALLOW} = 5{,}820^{\#} \times 1.4/1.6 = 5{,}093^{\#}$

GRID E (LEVEL 2) FE=4,236# 1 SEGMENT L=31'-7" h=9'

VE = 4,236# / 31.58' = 134 PLF

USE W1 VEALLOW=242 PLF

HOLD DOWNS

TE = 134 PLF x 9' x 1.25 - 1/2 (20 PSF x 16.25' x 15.79') - 1/2 (12 PSF x 4.5' x 15.79') = -1,493#

∴ NO HDS REQ'D

GRID F (LEVEL 1) FE = 5,116# 1 SEGMENT L=19'-11" h=11'

VE = 5,116# / 19.92' = 257 PLF

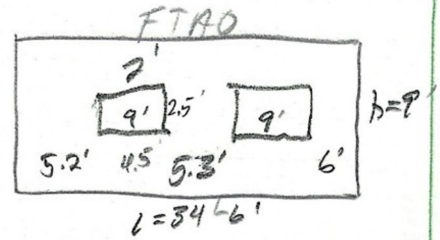
USE W2 VEALLOW = 353 PLF

HOLD DOWNS

TE = 257 PLF x 11' x 1.25 - 1/2 (30 PSF x 5.5' x 9.96') - 1/2 (8 PSF x 5.5' x 9.96') = 2,491#

$\boxed{\text{USE HDU4-SDS 2.5 w/ 2 STUDS}}$ TEALLOW = 3,285 x 1.4/1.6 = 2,874#

GRID H (LEVEL 2) $F_E = 3{,}708^{\#}$



$V_E = 285$ plf

USE = W2 $V_{E\,ALLOW} = 353$ plf

HOLD DOWNS

$T_E = 967^{\#} \times 1.25 - \tfrac{1}{2}(20\text{psf} \times 37' \times 17.25') - \tfrac{1}{2}(12\text{psf} \times 4.5' \times 17.25') = 105^{\#}$

USE MSTC48B3 w/2STUDS
OR MST48 w/2STUDS

$T_{E\,ALLOW} = 0.85 \times 3{,}900^{\#} \times 1.4/1.6 = 2{,}900^{\#}$
$T_{E\,ALLOW} = 3{,}425^{\#} \times 1.4/1.6 = 2{,}997^{\#}$

GRID I (LEVEL 1) $F_W = 9{,}912^{\#}$ $F_E = 6{,}151^{\#}$

L=9'-8"
L=5'-4"
L=5'-2"
L=20'-2"

h=11'

$V_W = 9{,}912^{\#}/20.16' = 492$ plf
$V_E = 6{,}151^{\#}/20.16' = 305$ plf

USE W3

$V_{W\,ALLOW} = 627\text{plf} \times (1.25 - 0.125 \times 11'/5.16') = 617$ plf
$V_{E\,ALLOW} = 456\text{plf} \times (1.25 - 0.125 \times 11'/5.16') = 448$ plf

HOLD DOWNS

$T_W = 492\text{plf} \times 11' - \tfrac{1}{2}(12\text{psf} \times 5.5' \times 2.58') = 5{,}327^{\#}$
$T_E = 305\text{plf} \times 11' \times 1.25 - \tfrac{1}{2}(12\text{psf} \times 5.5' \times 2.58') = 4{,}109^{\#}$

USE HDU8-SDS2.5 w/2STUDS

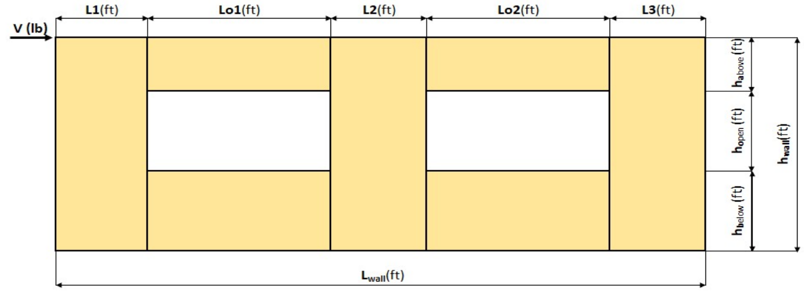
$T_{W\,ALLOW} = 5{,}920^{\#}$
$T_{E\,ALLOW} = 5{,}920^{\#} \times 1.4/1.6 = 5{,}093^{\#}$

APA

This version of the Force Transfer Around Openings calculator has expired.
Please go to **www.apawood.org** to download the latest version.

Project Information

Code:	IBC 2021	**Date:**	2/26/2025
Designer:	Chon Pieruccioni, PE		
Client:			
Project:	East Town Crossing - Clubhouse		
Wall Line:	Grid 5 (12'-6" Section) - (Level 1 Seismic)		



Shear Wall Calculation Variables

V	1892 lbf		Opening 1		Opening 2	Adj. Factor Method =	2bs/h
L1	2.00 ft	h_a1	2.00 ft	h_a2	2.00 ft	Wall Pier Aspect Ratio	Adj. Factor
L2	2.45 ft	h_o1	7.00 ft	h_o2	7.00 ft	$P1=h_o/L1=$ 3.50	0.571
L3	2.10 ft	h_b1	3.00 ft	h_b2	3.00 ft	$P2=h_o/L2=$ 2.86	0.700
h_{wall}	12.00 ft	Lo1	3.00 ft	Lo2	3.00 ft	$P3=h_o/L3=$ 3.33	0.600
L_{wall}	12.55 ft						

1. Hold-down forces: $H = Vh_{wall}/L_{wall}$ 1809 lbf

2. Unit shear above + below opening

First opening: $va1 = vb1 = H/(h_a1+h_b1) =$	362 plf
Second opening: $va2 = vb2 = H/(h_a2+h_b2) =$	362 plf

3. Total boundary force above + below openings

First opening: O1 = va1 x (Lo1) =	1085 lbf
Second opening: O2 = va2 x (Lo2) =	1085 lbf

4. Corner forces

F1 = O1(L1)/(L1+L2) =	488 lbf
F2 = O1(L2)/(L1+L2) =	598 lbf
F3 = O2(L2)/(L2+L3) =	584 lbf
F4 = O2(L3)/(L2+L3) =	501 lbf

5. Tributary length of openings

T1 = (L1*Lo1)/(L1+L2) =	1.35 ft
T2 = (L2*Lo1)/(L1+L2) =	1.65 ft
T3 = (L2*Lo2)/(L2+L3) =	1.62 ft
T4 = (L3*Lo2)/(L2+L3) =	1.38 ft

6. Unit shear beside opening

v1 = (V/L)(L1+T1)/L1 =	252 plf
v2 = (V/L)(T2+L2+T3)/L2 =	352 plf
v3 = (V/L)(T4+L3)/L3 =	250 plf
Check v1*L1+v2*L2+v3*L3=V?	1892 lbf **OK**

7. Resistance to corner forces

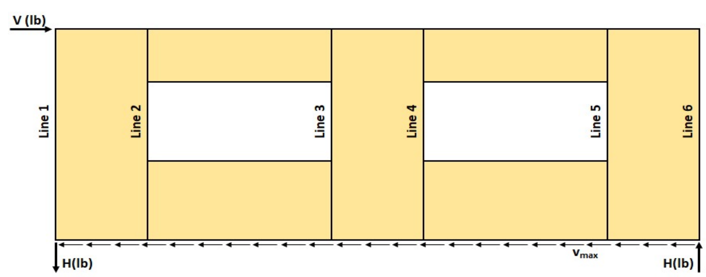
R1 = v1*L1 =	505 lbf
R2 = v2*L2 =	862 lbf
R3 = v3*L3 =	525 lbf

8. Difference corner force + resistance

R1-F1 =	17 lbf
R2-F2-F3 =	-320 lbf
R3-F4 =	24 lbf

9. Unit shear in corner zones

vc1 = (R1-F1)/L1 =	8 plf
vc2 = (R2-F2-F3)/L2 =	-131 plf
vc3 = (R3-F4)/L3 =	12 plf



Check Summary of Shear Values for Two Openings

Line 1: $vc1(h_a1+h_b1)+v1(h_o1)=H?$		42	1767	1809 lbf
Line 2: $va1(h_a1+h_b1)-vc1(h_a1+h_b1)-v1(h_o1)=0?$	1809	42	1767	0
Line 3: $vc2(h_a1+h_b1)+v2(h_o1)-va1(h_a1+h_b1)=0?$	-653	2463	1809	0
Line 4: $va2(h_a2+h_b2)-v2(h_o2)-vc2(h_a2+h_b2)=0?$	1809	2463	-653	0
Line 5: $va2(h_a2+h_b2)-vc3(h_a2+h_b2)-v3(h_o2)=0?$	1809	58	1751	0
Line 6: $vc3(h_a2+h_b2)+ v3(h_o2) = H?$		58	1751	1809 lbf

Design Summary*

Req. Sheathing Capacity	503 plf	**	**4-Term Deflection**	0.682 in.	**3-Term Deflection**	0.711 in.
Req. Strap Force	598 lbf		**4-Term Story Drift %**	0.019 %	**3-Term Story Drift %**	0.020 %
Req. HD Force	1809 lbf					
Req. Shear Wall Anchorage Force	151 plf					

****Req. Sheathing Capacity has been adjusted per the Aspect Ratio Adjustment Factor**

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

Project Information

Code:	IBC 2021	**Date:**	2/26/2025
Designer:	Chon Pieruccioni, PE		
Client:			
Project:	East Town Crossing - Clubhouse		
Wall Line:	Grid 5 (12'-6" Section) - (Level 1 Seismic)		

Shear Wall Deflection Calculation Variables

Unfactored Shear Load $V_{unfactored}$: 1892 (lbf)

Sheathing Type: 7/16 OSB
Grade: APA Rated Sheathing

G_t Override:
G_a Override:

Wood End Post Values:
Species: HF#2
E: 1.30E+06 (psi)

Enter individual post sizes below.

C_d: 4.00

Nail Type: 8d common (penny weight)

	Pier 1	Pier 3	
Nail Spacing:	2	2	(in.)
HD Capacity:	1938	1938	(lbf)
HD Deflection:	0.088	0.088	(in.)

Four-Term Equation Deflection Check

$$\Delta = \frac{8vh^3}{EAb} + \frac{vh}{Gt} + 0.75he_n + d_a\frac{h}{b} \qquad \text{(Equation 23-2)}$$

	Pier 1-L	Pier 1-R	Pier 2-L	Pier 2-R	Pier 3-L	Pier 3-R	
$v_{unfactored}$:	252	252	352	352	250	250	(plf)
E:	1.30E+06	1.30E+06	1.30E+06	1.30E+06	1.30E+06	1.30E+06	(psi)
h:	12.00	9.00	9.00	9.00	9.00	12.00	(ft)
Qty:	2	2	2	2	2	2	
Stud Size:	2x6	2x6	2x6	2x6	2x6	2x6	
A Override:							(in.^{2})
A:	16.5	16.5	16.5	16.5	16.5	16.5	(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:	42	42	59	59	42	42	(plf)
e_n:	0.0004	0.0004	0.0010	0.0010	0.0004	0.0004	(in.)
b:	2.00	2.00	2.45	2.45	2.10	2.10	(ft)
HD Capacity:	1938	1938	1938	1938	1938	1938	(lbf)
HD Defl:	0.088	0.088	0.088	0.088	0.088	0.088	(in.)

Sheathing Type: 7/16 OSB APA Rated Sheathing
Nail Type: 8d common

Check Total Deflection of Wall System

Pier 1 (left)				Pier 1 (right)			
Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 4 HD-1	Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 4 HD-2
0.081	0.036	0.003	0.825	0.034	0.027	0.002	0.464
		Sum	0.946			Sum	0.528
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
0.039	0.038	0.007	0.528	0.039	0.038	0.007	0.528
		Sum	0.612			Sum	0.612
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
0.032	0.027	0.002	0.438	0.077	0.036	0.003	0.779
		Sum	0.500			Sum	0.895

Total Defl.	
0.682	(in.)
0.0189	%drift

Project Information

Code:	IBC 2021	**Date:**	2/26/2025
Designer:	Chon Pieruccioni, PE		
Client:			
Project:	East Town Crossing - Clubhouse		
Wall Line:	Grid 5 (12'-6" Section) - (Level 1 Seismic)		

Shear Wall Deflection Calculation Variables

Unfactored Shear Load $V_{unfactored}$: 1892 (lbf)

Wood End Post Values:

Sheathing Type:	7/16 OSB	Species:	HF#2	Nail Type:	8d common	(penny weight)
Grade:	APA Rated Sheathing	E:	1.30E+06 (psi)			

					Pier 1	Pier 3	
G_t Override:				Nail Spacing:	2	2	(in.)
G_a Override:		C_d:	4.00	HD Capacity:	1938	1938	(lbf)
				HD Deflection:	0.088	0.088	(in.)

Three-Term Equation Deflection Check

$$\delta_{sw} = \frac{8vh^3}{EAb} + \frac{vh}{1000G_a} + \frac{h\Delta_a}{b} \qquad (4.3\text{-}1)$$

Sheathing Type: 7/16 OSB APA Rated Sheathing

Nail Type: 8d common

	Pier 1-L	Pier 1-R	Pier 2-L	Pier 2-R	Pier 3-L	Pier 3-R	
$v_{unfactored}$:	252	252	352	352	250	250	(plf)
E:	1.30E+06	1.30E+06	1.30E+06	1.30E+06	1.30E+06	1.30E+06	(psi)
h:	12.00	9.00	9.00	9.00	9.00	12.00	(ft)
Qty:	2	2	2	2	2	2	
Stud Size:	2x6	2x6	2x6	2x6	2x6	2x6	
A Override:							(in.^{2})
A:	16.5	16.5	16.5	16.5	16.5	16.5	(in.^{2})
G_a:	42.0	42.0	42.0	42.0	42.0	42.0	(kips/in.)
b:	2.00	2.00	2.45	2.45	2.10	2.10	(ft)
HD Capacity:	1938	1938	1938	1938	1938	1938	(lbf)
HD Defl:	0.088	0.088	0.088	0.088	0.088	0.088	(in.)

Check Total Deflection of Wall System

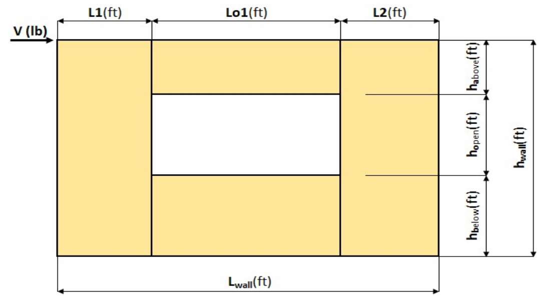
Pier 1 (left)			Pier 1 (right)		
Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 1 Bending	Term 2 Shear	Term 3 Fastener
0.081	0.072	0.825	0.034	0.054	0.464
	Sum	0.979		Sum	0.553
Pier 2 (left)			**Pier 2 (right)**		
Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 1 Bending	Term 2 Shear	Term 3 Fastener
0.039	0.075	0.528	0.039	0.075	0.528
	Sum	0.643		Sum	0.643
Pier 3 (left)			**Pier 3 (right)**		
Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 1 Bending	Term 2 Shear	Term 3 Fastener
0.032	0.054	0.438	0.077	0.071	0.779
	Sum	0.524		Sum	0.927

Total Defl.	
0.711	(in.)
0.0198	%drift

Comment: The 3-term equation is calibrated to be approximately equal to 4-term equation at 1.4*ASD capacity.

Project Information

Code:	2021 IBC	**Date:**	2/26/2025
Designer:	Chon Pieruccioni, PE		
Client:			
Project:	East Town Crossing - Clubhouse		
Wall Line:	Grid 5 (7'-0" Section) - (Level 1 Seismic)		



Shear Wall Calculation Variables

V	1116 lbf		Opening 1	Adj. Factor Method =	2bs/h	
L1	2.00 ft	h_a	2.00 ft	Wall Pier Aspect Ratio		Adj. Factor
L2	2.00 ft	h_o	7.00 ft	$P1=h_o/L1=$	3.50	0.571
h_{wall}	12.00 ft	h_b	3.00 ft	$P2=h_o/L2=$	3.50	0.571
L_{wall}	7.00 ft	Lo1	3.00 ft			

1. Hold-down forces: $H = Vh_{wall}/L_{wall}$ 1913 lbf

2. Unit shear above + below opening

First opening: $va1 = vb1 = H/(h_a+h_b) =$ 383 plf

3. Total boundary force above + below openings

First opening: O1 = va1 x (Lo1) = 1148 lbf

4. Corner forces

F1 = O1(L1)/(L1+L2) = 574 lbf

F2 = O1(L2)/(L1+L2) = 574 lbf

5. Tributary length of openings

T1 = (L1*Lo1)/(L1+L2) = 1.50 ft

T2 = (L2*Lo1)/(L1+L2) = 1.50 ft

6. Unit shear beside opening

v1 = (V/L)(L1+T1)/L1 = 279 plf

v2 = (V/L)(T2+L2)/L2 = 279 plf

Check v1*L1+v2*L2=V? 1116 lbf **OK**

7. Resistance to corner forces

R1 = v1*L1 = 558 lbf

R2 = v2*L2 = 558 lbf

8. Difference corner force + resistance

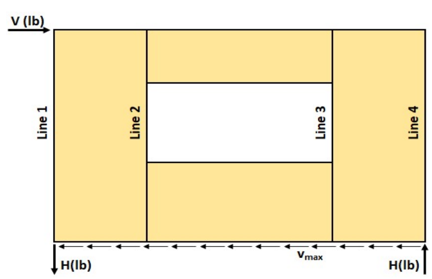
R1-F1 = -16 lbf

R2-F2 = -16 lbf

9. Unit shear in corner zones

vc1 = (R1-F1)/L1 = -8 plf

vc2 = (R2-F2)/L2 = -8 plf



Check Summary of Shear Values for One Opening

Line 1: $vc1(h_a+h_b)+v1(h_o)=H$?		-40	1953	1913 lbf
Line 2: $va1(h_a+h_b)-vc1(h_a+h_b)-v1(h_o)=0$?	1913	-40	1953	0
Line 3: $va1(h_a+h_b)-vc2(h_a+h_b)-v1(h_o)=0$?	1913	-40	1953	0
Line 4: $vc2(h_a+h_b)+v2(h_o)=H$?		-40	1953	1913 lbf

Design Summary*

Req. Sheathing Capacity	488 plf **	**4-Term Deflection**	0.442 in.	**3-Term Deflection**	0.473 in.
Req. Strap Force	574 lbf	**4-Term Story Drift %**	0.012 %	**3-Term Story Drift %**	0.013 %
Req. HD Force (H)	1913 lbf				
Req. Shear Wall Anchorage Force (v_{max})	159 plf				

****Req. Sheathing Capacity has been adjusted per the Aspect Ratio Adjustment Factor**

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

Project Information

Code:	2021 IBC	**Date:**	2/26/2025
Designer:	Chon Pieruccioni, PE		
Client:			
Project:	East Town Crossing - Clubhouse		
Wall Line:	Grid 5 (7'-0" Section) - (Level 1 Seismic)		

Shear Wall Deflection Calculation Variables

Unfactored Shear Load $V_{unfactored}$: 1116 (lbf)

Sheathing Type: 7/16 OSB
Grade: APA Rated Sheathing

G_t Override:
G_a Override:

Wood End Post Values:
Species: HF#2
E: 1.30E+06 (psi)

Enter individual post sizes below.

C_d: 4.00

Nail Type: 8d common (penny weight)

	Pier 1	Pier 2	
Nail Spacing:	2	2	(in.)
HD Capacity:	5093	5093	(lbf)
HD Deflection:	0.11	0.11	(in.)

Four-Term Equation Deflection Check

$$\Delta = \frac{8vh^3}{EAb} + \frac{vh}{Gt} + 0.75he_n + d_a\frac{h}{b} \quad \text{(Equation 23-2)}$$

Sheathing Type: 7/16 OSB APA Rated Sheathing
Nail Type: 8d common

	Pier 1-L	Pier 1-R	Pier 2-L	Pier 2-R	
$v_{unfactored}$:	279	279	279	279	(plf)
E:	1.30E+06	1.30E+06	1.30E+06	1.30E+06	(psi)
h:	12.00	9.00	9.00	12.00	(ft)
Qty:	2.00E+00	2.00E+00	2.00E+00	2.00E+00	
Stud Size:	2x6	2x6	2x6	2x6	
A Override:					(in.^{2})
A:	16.5	16.5	16.5	16.5	(in.^{2})
G_t:	83,500	83,500	83,500	83,500	(lbf/in.)
Nail Spacing:	2	2	2	2	(in.)
V_n:	47	47	47	47	(plf)
e_n:	0.0005	0.0005	0.0005	0.0005	(in.)
b:	2.00	2.00	2.00	2.00	(ft)
HD Capacity:	5093	5093	5093	5093	(lbf)
HD Defl:	0.11	0.11	0.11	0.11	(in.)

Check Total Deflection of Wall System

Pier 1 (left)				Pier 1 (right)			
Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 4 HD-1	Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 4 HD-2
0.090	0.040	0.004	0.434	0.038	0.030	0.003	0.244
		Sum	0.568			Sum	0.315
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
0.038	0.030	0.003	0.244	0.090	0.040	0.004	0.434
		Sum	0.315			Sum	0.568

Total Defl.	
0.442	(in.)
0.0123	%drift

Project Information

Code:	2021 IBC	**Date:**	2/26/2025
Designer:	Chon Pieruccioni, PE		
Client:			
Project:	East Town Crossing - Clubhouse		
Wall Line:	Grid 5 (7'-0" Section) - (Level 1 Seismic)		

Shear Wall Deflection Calculation Variables

Unfactored Shear Load $V_{unfactored}$: 1116 (lbf)

Sheathing Type: 7/16 OSB
Grade: APA Rated Sheathing

Wood End Post Values:
Species: HF#2
E: 1.30E+06 (psi)

Nail Type: 8d common (penny weight)

G_t Override:
G_a Override:

C_d: 4.00

	Pier 1	Pier 2	
Nail Spacing:	2	2	(in.)
HD Capacity:	5093	5093	(lbf)
HD Deflection:	0.11	0.11	(in.)

Three-Term Equation Deflection Check

$$\delta_{sw} = \frac{8vh^3}{EAb} + \frac{vh}{1000G_a} + \frac{h\Delta_a}{b} \qquad (4.3\text{-}1)$$

	Pier 1-L	Pier 1-R	Pier 2-L	Pier 2-R	
$v_{unfactored}$:	279	279	279	279	(plf)
E:	1.30E+06	1.30E+06	1.30E+06	1.30E+06	(psi)
h:	12.00	9.00	9.00	12.00	(ft)
Qty:	2.00E+00	2.00E+00	2.00E+00	2.00E+00	
Stud Size:	2x6	2x6	2x6	2x6	
A Override:					(in.^{2})
A:	16.5	16.5	16.5	16.5	(in.^{2})
G_a:	42.0	42.0	42.0	42.0	(kips/in.)
b:	2.00	2.00	2.00	2.00	(ft)
HD Capacity:	5093	5093	5093	5093	(lbf)
HD Defl:	0.11	0.11	0.11	0.11	(in.)

Sheathing Type: 7/16 OSB APA Rated Sheathing
Nail Type: 8d common

Check Total Deflection of Wall System

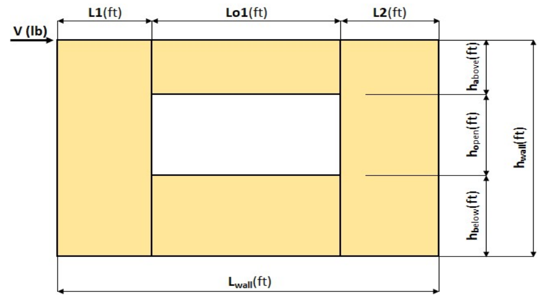
Pier 1 (left)			Pier 1 (right)		
Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 1 Bending	Term 2 Shear	Term 3 Fastener
0.090	0.080	0.434	0.038	0.060	0.244
	Sum	0.603		Sum	0.342
Pier 2 (left)			**Pier 2 (right)**		
Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 1 Bending	Term 2 Shear	Term 3 Fastener
0.038	0.060	0.244	0.090	0.080	0.434
	Sum	0.342		Sum	0.603

Total Defl.	
0.473	(in.)
0.0131	%drift

Comment: The 3-term equation is calibrated to be approximately equal to 4-term equation at 1.4*ASD capacity.

Project Information

Code:	2021 IBC	**Date:**	2/26/2025
Designer:	Chon Pieruccioni, PE		
Client:			
Project:	East Town Crossing - Clubhouse		
Wall Line:	Grid 7 (18'-3" Section) - (Level 2 Seismic)		



Shear Wall Calculation Variables

V	1727 lbf		Opening 1	Adj. Factor Method =		2bs/h
L1	4.80 ft	h_a	1.25 ft	Wall Pier Aspect Ratio		Adj. Factor
L2	4.45 ft	h_o	3.50 ft	$P1=h_o/L1=$	0.73	N/A
h_{wall}	9.00 ft	h_b	4.25 ft	$P2=h_o/L2=$	0.79	N/A
L_{wall}	18.25 ft	Lo1	9.00 ft			

Note to Designer: The width-to-height ratio of sheathing above or below the openings exceeds 6.5:1. Exercise caution when assuming fixity at corner regions, as assumed in this calculator.

1. Hold-down forces: $H = Vh_{wall}/L_{wall}$ 852 lbf

2. Unit shear above + below opening

First opening: $va1 = vb1 = H/(h_a+h_b) =$ 155 plf

3. Total boundary force above + below openings

First opening: O1 = va1 x (Lo1) = 1394 lbf

4. Corner forces

F1 = O1(L1)/(L1+L2) = 723 lbf
F2 = O1(L2)/(L1+L2) = 670 lbf

5. Tributary length of openings

T1 = (L1*Lo1)/(L1+L2) = 4.67 ft
T2 = (L2*Lo1)/(L1+L2) = 4.33 ft

6. Unit shear beside opening

v1 = (V/L)(L1+T1)/L1 = 187 plf
v2 = (V/L)(T2+L2)/L2 = 187 plf
Check v1*L1+v2*L2=V? 1727 lbf **OK**

7. Resistance to corner forces

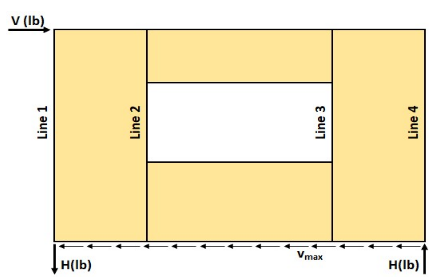
R1 = v1*L1 = 896 lbf
R2 = v2*L2 = 831 lbf

8. Difference corner force + resistance

R1-F1 = 173 lbf
R2-F2 = 160 lbf

9. Unit shear in corner zones

vc1 = (R1-F1)/L1 = 36 plf
vc2 = (R2-F2)/L2 = 36 plf



Check Summary of Shear Values for One Opening

Line 1: $vc1(h_a+h_b)+v1(h_o)=H?$		198	653	852 lbf
Line 2: $va1(h_a+h_b)-vc1(h_a+h_b)-v1(h_o)=0?$	852	198	653	0
Line 3: $va1(h_a+h_b)-vc2(h_a+h_b)-v1(h_o)=0?$	852	198	653	0
Line 4: $vc2(h_a+h_b)+v2(h_o)=H?$		198	653	852 lbf

Design Summary*

Req. Sheathing Capacity	187 plf	**4-Term Deflection**	0.088 in.	**3-Term Deflection**	0.137 in.
Req. Strap Force	723 lbf	**4-Term Story Drift %**	0.003 %	**3-Term Story Drift %**	0.005 %
Req. HD Force (H)	852 lbf				
Req. Shear Wall Anchorage Force (v_{max})	95 plf				

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

Project Information

Code:	2021 IBC	**Date:**	2/26/2025
Designer:	Chon Pieruccioni, PE		
Client:			
Project:	East Town Crossing - Clubhouse		
Wall Line:	Grid 7 (18'-3" Section) - (Level 2 Seismic)		

Shear Wall Deflection Calculation Variables

Unfactored Shear Load $V_{unfactored}$: 1727 (lbf)

Sheathing Type: 7/16 OSB
Grade: APA Rated Sheathing

G_t Override:
G_a Override:

Wood End Post Values:
Species: HF#2
E: 1.30E+06 (psi)

Enter individual post sizes below.

C_d: 4.00

Nail Type: 8d common (penny weight)

	Pier 1	Pier 2	
Nail Spacing:	6	6	(in.)
HD Capacity:	5093	5093	(lbf)
HD Deflection:	0.11	0.11	(in.)

Four-Term Equation Deflection Check

$$\Delta = \frac{8vh^3}{EAb} + \frac{vh}{Gt} + 0.75he_n + d_a\frac{h}{b} \qquad \text{(Equation 23-2)}$$

	Pier 1-L	Pier 1-R	Pier 2-L	Pier 2-R	
$v_{unfactored}$:	187	187	187	187	(plf)
E:	1.30E+06	1.30E+06	1.30E+06	1.30E+06	(psi)
h:	9.00	4.75	4.75	9.00	(ft)
Qty:	2.00E+00	2.00E+00	2.00E+00	2.00E+00	
Stud Size:	2x6	2x6	2x6	2x6	
A Override:					(in.^{2})
A:	16.5	16.5	16.5	16.5	(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:	4.80	4.80	4.45	4.45	(ft)
HD Capacity:	5093	5093	5093	5093	(lbf)
HD Defl:	0.11	0.11	0.11	0.11	(in.)

Sheathing Type: 7/16 OSB APA Rated Sheathing
Nail Type: 8d common

Check Total Deflection of Wall System

Pier 1 (left)				Pier 1 (right)			
Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 4 HD-1	Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 4 HD-2
0.011	0.020	0.027	0.068	0.002	0.011	0.014	0.019
		Sum	0.126			Sum	0.046
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
0.002	0.011	0.014	0.020	0.011	0.020	0.027	0.073
		Sum	0.047			Sum	0.132

Total Defl.	
0.088	(in.)
0.0032	%drift

Project Information

Code:	2021 IBC	**Date:**	2/26/2025
Designer:	Chon Pieruccioni, PE		
Client:			
Project:	East Town Crossing - Clubhouse		
Wall Line:	Grid 7 (18'-3" Section) - (Level 2 Seismic)		

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:	6	6	(in.)
HD Capacity:	5093	5093	(lbf)
HD Deflection:	0.11	0.11	(in.)

Three-Term Equation Deflection Check

$$\delta_{sw} = \frac{8vh^3}{EAb} + \frac{vh}{1000G_a} + \frac{h\Delta_a}{b} \qquad (4.3\text{-}1)$$

	Pier 1-L	Pier 1-R	Pier 2-L	Pier 2-R	
$v_{unfactored}$:	187	187	187	187	(plf)
E:	1.30E+06	1.30E+06	1.30E+06	1.30E+06	(psi)
h:	9.00	4.75	4.75	9.00	(ft)
Qty:	2.00E+00	2.00E+00	2.00E+00	2.00E+00	
Stud Size:	2x6	2x6	2x6	2x6	
A Override:					(in.^{2})
A:	16.5	16.5	16.5	16.5	(in.^{2})
G_a:	15.0	15.0	15.0	15.0	(kips/in.)
b:	4.80	4.80	4.45	4.45	(ft)
HD Capacity:	5093	5093	5093	5093	(lbf)
HD Defl:	0.11	0.11	0.11	0.11	(in.)

Sheathing Type: 7/16 OSB APA Rated Sheathing
Nail Type: 8d common

Check Total Deflection of Wall System

Pier 1 (left)			Pier 1 (right)		
Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 1 Bending	Term 2 Shear	Term 3 Fastener
0.011	0.112	0.068	0.002	0.059	0.019
	Sum	0.191		Sum	0.080
Pier 2 (left)			**Pier 2 (right)**		
Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 1 Bending	Term 2 Shear	Term 3 Fastener
0.002	0.059	0.020	0.011	0.112	0.073
	Sum	0.081		Sum	0.197

Total Defl.	
0.137	(in.)
0.0051	%drift

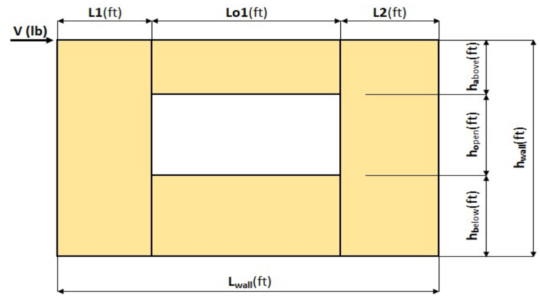
Comment: The 3-term equation is calibrated to be approximately equal to 4-term equation at 1.4*ASD capacity.

APA

This version of the Force Transfer Around Openings calculator has expired.
Please go to **www.apawood.org** to download the latest version.

Project Information

Code:	2021 IBC	**Date:**	2/26/2025
Designer:	Chon Pieruccioni, PE		
Client:			
Project:	East Town Crossing - Clubhouse		
Wall Line:	Grid 7 (21'-9" Section) - (Level 2 Seismic)		



Shear Wall Calculation Variables

V	2058 lbf	Opening 1		Adj. Factor Method =		2bs/h
L1	6.60 ft	h_a	1.50 ft	Wall Pier Aspect Ratio		Adj. Factor
L2	6.15 ft	h_o	4.00 ft	$P1=h_o/L1=$	0.61	N/A
h_{wall}	9.00 ft	h_b	3.50 ft	$P2=h_o/L2=$	0.65	N/A
L_{wall}	21.75 ft	Lo1	9.00 ft			

1. Hold-down forces: $H = Vh_{wall}/L_{wall}$ 852 lbf

2. Unit shear above + below opening

First opening: $va1 = vb1 = H/(h_a+h_b) =$ 170 plf

3. Total boundary force above + below openings

First opening: O1 = va1 x (Lo1) = 1533 lbf

4. Corner forces

F1 = O1(L1)/(L1+L2) = 793 lbf

F2 = O1(L2)/(L1+L2) = 739 lbf

5. Tributary length of openings

T1 = (L1*Lo1)/(L1+L2) = 4.66 ft

T2 = (L2*Lo1)/(L1+L2) = 4.34 ft

6. Unit shear beside opening

v1 = (V/L)(L1+T1)/L1 = 161 plf

v2 = (V/L)(T2+L2)/L2 = 161 plf

Check v1*L1+v2*L2=V? 2058 lbf **OK**

7. Resistance to corner forces

R1 = v1*L1 = 1065 lbf

R2 = v2*L2 = 993 lbf

8. Difference corner force + resistance

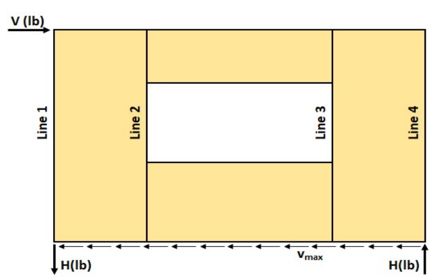
R1-F1 = 272 lbf

R2-F2 = 253 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: $vc1(h_a+h_b)+v1(h_o)=H?$		206	646	852 lbf
Line 2: $va1(h_a+h_b)-vc1(h_a+h_b)-v1(h_o)=0?$	852	206	646	0
Line 3: $va1(h_a+h_b)-vc2(h_a+h_b)-v1(h_o)=0?$	852	206	646	0
Line 4: $vc2(h_a+h_b)+v2(h_o)=H?$		206	646	852 lbf

Design Summary*

Req. Sheathing Capacity	170 plf	**4-Term Deflection**	0.063 in.	**3-Term Deflection**	0.113 in.
Req. Strap Force	793 lbf	**4-Term Story Drift %**	0.002 %	**3-Term Story Drift %**	0.004 %
Req. HD Force (H)	852 lbf				
Req. Shear Wall Anchorage Force (v_{max})	95 plf				

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

Project Information

Code:	2021 IBC	**Date:**	2/26/2025
Designer:	Chon Pieruccioni, PE		
Client:			
Project:	East Town Crossing - Clubhouse		
Wall Line:	Grid 7 (21'-9" Section) - (Level 2 Seismic)		

Shear Wall Deflection Calculation Variables

Unfactored Shear Load $V_{unfactored}$: 2058 (lbf)

Sheathing Type: 7/16 OSB
Grade: APA Rated Sheathing

G_t Override:
G_a Override:

Wood End Post Values:
Species: HF#2
E: 1.30E+06 (psi)

Enter individual post sizes below.

C_d: 4.00

Nail Type: 8d common (penny weight)

	Pier 1	Pier 2	
Nail Spacing:	6	6	(in.)
HD Capacity:	5093	5093	(lbf)
HD Deflection:	0.11	0.11	(in.)

Four-Term Equation Deflection Check

$$\Delta = \frac{8vh^3}{EAb} + \frac{vh}{Gt} + 0.75he_n + d_a\frac{h}{b} \qquad \text{(Equation 23-2)}$$

Sheathing Type: 7/16 OSB APA Rated Sheathing
Nail Type: 8d common

	Pier 1-L	Pier 1-R	Pier 2-L	Pier 2-R	
$v_{unfactored}$:	161	161	161	161	(plf)
E:	1.30E+06	1.30E+06	1.30E+06	1.30E+06	(psi)
h:	9.00	5.50	5.50	9.00	(ft)
Qty:	2.00E+00	2.00E+00	2.00E+00	2.00E+00	
Stud Size:	2x6	2x6	2x6	2x6	
A Override:					(in.^{2})
A:	16.5	16.5	16.5	16.5	(in.^{2})
G_t:	83,500	83,500	83,500	83,500	(lbf/in.)
Nail Spacing:	6	6	6	6	(in.)
V_n:	81	81	81	81	(plf)
e_n:	0.0026	0.0026	0.0026	0.0026	(in.)
b:	6.60	6.60	6.15	6.15	(ft)
HD Capacity:	5093	5093	5093	5093	(lbf)
HD Defl:	0.11	0.11	0.11	0.11	(in.)

Check Total Deflection of Wall System

Pier 1 (left)				Pier 1 (right)			
Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 4 HD-1	Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 4 HD-2
0.007	0.017	0.018	0.043	0.002	0.011	0.011	0.016
		Sum	0.084			Sum	0.039
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
0.002	0.011	0.011	0.017	0.007	0.017	0.018	0.046
		Sum	0.040			Sum	0.088

Total Defl.	
0.063	(in.)
0.0023	%drift

Project Information

Code:	2021 IBC	**Date:**	2/26/2025
Designer:	Chon Pieruccioni, PE		
Client:			
Project:	East Town Crossing - Clubhouse		
Wall Line:	Grid 7 (21'-9" Section) - (Level 2 Seismic)		

Shear Wall Deflection Calculation Variables

Unfactored Shear Load $V_{unfactored}$: 2058 (lbf)

Sheathing Type:	7/16 OSB	Wood End Post Values: Species:	HF#2	Nail Type:	8d common	(penny weight)
Grade:	APA Rated Sheathing	E:	1.30E+06 (psi)			

					Pier 1	Pier 2	
G_t Override:				Nail Spacing:	6	6	(in.)
G_a Override:		C_d:	4.00	HD Capacity:	5093	5093	(lbf)
				HD Deflection:	0.11	0.11	(in.)

Three-Term Equation Deflection Check

$$\delta_{sw} = \frac{8vh^3}{EAb} + \frac{vh}{1000G_a} + \frac{h\Delta_a}{b} \quad (4.3\text{-}1)$$

Sheathing Type: 7/16 OSB APA Rated Sheathing

Nail Type: 8d common

	Pier 1-L	Pier 1-R	Pier 2-L	Pier 2-R	
$v_{unfactored}$:	161	161	161	161	(plf)
E:	1.30E+06	1.30E+06	1.30E+06	1.30E+06	(psi)
h:	9.00	5.50	5.50	9.00	(ft)
Qty:	2.00E+00	2.00E+00	2.00E+00	2.00E+00	
Stud Size:	2x6	2x6	2x6	2x6	
A Override:					(in.^{2})
A:	16.5	16.5	16.5	16.5	(in.^{2})
G_a:	15.0	15.0	15.0	15.0	(kips/in.)
b:	6.60	6.60	6.15	6.15	(ft)
HD Capacity:	5093	5093	5093	5093	(lbf)
HD Defl:	0.11	0.11	0.11	0.11	(in.)

Check Total Deflection of Wall System

Pier 1 (left)			Pier 1 (right)		
Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 1 Bending	Term 2 Shear	Term 3 Fastener
0.007	0.097	0.043	0.002	0.059	0.016
	Sum	0.146		Sum	0.077
Pier 2 (left)			**Pier 2 (right)**		
Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 1 Bending	Term 2 Shear	Term 3 Fastener
0.002	0.059	0.017	0.007	0.097	0.046
	Sum	0.078		Sum	0.150

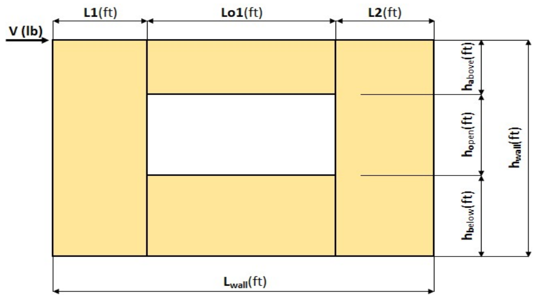
Total Defl.	
0.113	(in.)
0.0042	%drift

Comment: The 3-term equation is calibrated to be approximately equal to 4-term equation at 1.4*ASD capacity.

This version of the Force Transfer Around Openings calculator has expired.
Please go to **www.apawood.org** to download the latest version.

Project Information

Code:	2021 IBC	**Date:**	2/26/2025
Designer:	Chon Pieruccioni, PE		
Client:			
Project:	East Town Crossing - Clubhouse		
Wall Line:	Grid A (27'-5" Section) - (Level 1 Wind)		



Shear Wall Calculation Variables

V	2584 lbf	Opening 1		Adj. Factor Method =	2bs/h
L1	3.21 ft	h_a	1.50 ft	Wall Pier Aspect Ratio	Adj. Factor
L2	3.21 ft	h_o	7.00 ft	$P1=h_o/L1=$ 2.18	0.917
h_{wall}	12.00 ft	h_b	3.50 ft	$P2=h_o/L2=$ 2.18	0.917
L_{wall}	27.63 ft	Lo1	21.21 ft		

Note to Designer: The width-to-height ratio of sheathing above or below the openings exceeds 6.5:1. Exercise caution when assuming fixity at corner regions, as assumed in this calculator.

1. Hold-down forces: $H = Vh_{wall}/L_{wall}$ 1122 lbf

2. Unit shear above + below opening

First opening: $va1 = vb1 = H/(h_a+h_b) =$ 224 plf

3. Total boundary force above + below openings

First opening: O1 = va1 x (Lo1) = 4761 lbf

4. Corner forces

F1 = O1(L1)/(L1+L2) = 2380 lbf
F2 = O1(L2)/(L1+L2) = 2380 lbf

5. Tributary length of openings

T1 = (L1*Lo1)/(L1+L2) = 10.61 ft
T2 = (L2*Lo1)/(L1+L2) = 10.61 ft

6. Unit shear beside opening

v1 = (V/L)(L1+T1)/L1 = 402 plf
v2 = (V/L)(T2+L2)/L2 = 402 plf
Check v1*L1+v2*L2=V? 2584 lbf **OK**

7. Resistance to corner forces

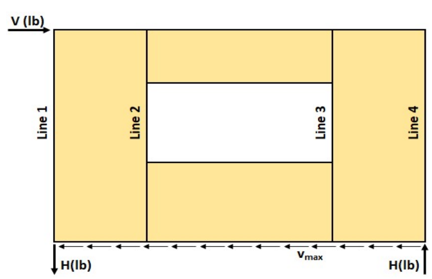
R1 = v1*L1 = 1292 lbf
R2 = v2*L2 = 1292 lbf

8. Difference corner force + resistance

R1-F1 = -1088 lbf
R2-F2 = -1088 lbf

9. Unit shear in corner zones

vc1 = (R1-F1)/L1 = -339 plf
vc2 = (R2-F2)/L2 = -339 plf



Check Summary of Shear Values for One Opening

Line 1: $vc1(h_a+h_b)+v1(h_o)=H$?		-1695	2817	1122 lbf
Line 2: $va1(h_a+h_b)-vc1(h_a+h_b)-v1(h_o)=0$?	1122	-1695	2817	0
Line 3: $va1(h_a+h_b)-vc2(h_a+h_b)-v1(h_o)=0$?	1122	-1695	2817	0
Line 4: $vc2(h_a+h_b)+v2(h_o)=H$?		-1695	2817	1122 lbf

Design Summary*

Req. Sheathing Capacity	439 plf **	**4-Term Deflection**	0.490 in.	**3-Term Deflection**	0.535 in.
Req. Strap Force	2380 lbf	**4-Term Story Drift %**	0.000 %	**3-Term Story Drift %**	0.000 %
Req. HD Force (H)	1122 lbf				
Req. Shear Wall Anchorage Force (v_{max})	94 plf				

****Req. Sheathing Capacity has been adjusted per the Aspect Ratio Adjustment Factor**

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

Project Information

Code:	2021 IBC	**Date:**	2/26/2025
Designer:	Chon Pieruccioni, PE		
Client:			
Project:	East Town Crossing - Clubhouse		
Wall Line:	Grid A (27'-5" Section) - (Level 1 Wind)		

Shear Wall Deflection Calculation Variables

Unfactored Shear Load $V_{unfactored}$: 2584 (lbf)

Sheathing Type: 7/16 OSB
Grade: APA Rated Sheathing

G_t Override:
G_a Override:

Wood End Post Values:
Species: HF#2
E: 1.30E+06 (psi)

Enter individual post sizes below.

C_d: 0.00

Nail Type: 8d common (penny weight)

	Pier 1	Pier 2	
Nail Spacing:	4	4	(in.)
HD Capacity:	5093	5093	(lbf)
HD Deflection:	0.11	0.11	(in.)

Four-Term Equation Deflection Check

$$\Delta = \frac{8vh^3}{EAb} + \frac{vh}{Gt} + 0.75he_n + d_a\frac{h}{b} \qquad \text{(Equation 23-2)}$$

Sheathing Type: 7/16 OSB APA Rated Sheathing
Nail Type: 8d common

	Pier 1-L	Pier 1-R	Pier 2-L	Pier 2-R	
$v_{unfactored}$:	402	402	402	402	(plf)
E:	1.30E+06	1.30E+06	1.30E+06	1.30E+06	(psi)
h:	12.00	8.50	8.50	12.00	(ft)
Qty:	2.00E+00	2.00E+00	2.00E+00	2.00E+00	
Stud Size:	2x6	2x6	2x6	2x6	
A Override:					(in.^{2})
A:	16.5	16.5	16.5	16.5	(in.^{2})
G_t:	83,500	83,500	83,500	83,500	(lbf/in.)
Nail Spacing:	4	4	4	4	(in.)
V_n:	134	134	134	134	(plf)
e_n:	0.0121	0.0121	0.0121	0.0121	(in.)
b:	3.21	3.21	3.21	3.21	(ft)
HD Capacity:	5093	5093	5093	5093	(lbf)
HD Defl:	0.11	0.11	0.11	0.11	(in.)

Check Total Deflection of Wall System

Pier 1 (left)				Pier 1 (right)			
Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 4 HD-1	Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 4 HD-2
0.081	0.058	0.109	0.390	0.029	0.041	0.077	0.196
		Sum	0.637			Sum	0.342
Pier 2 (left)				**Pier 2 (right)**			
Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 4 HD-1	Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 4 HD-2
0.029	0.041	0.077	0.196	0.081	0.058	0.109	0.390
		Sum	0.342			Sum	0.637

Total Defl.	
0.490	(in.)
0.0000	%drift

Project Information

Code:	2021 IBC	**Date:**	2/26/2025
Designer:	Chon Pieruccioni, PE		
Client:			
Project:	East Town Crossing - Clubhouse		
Wall Line:	Grid A (27'-5" Section) - (Level 1 Wind)		

Shear Wall Deflection Calculation Variables

Unfactored Shear Load $V_{unfactored}$: 2584 (lbf)

Wood End Post Values:

Sheathing Type:	7/16 OSB	Species:	HF#2	Nail Type:	8d common (penny weight)
Grade:	APA Rated Sheathing	E:	1.30E+06 (psi)		

				Pier 1	Pier 2	
G_t Override:			Nail Spacing:	4	4	(in.)
G_a Override:		C_d: 0.00	HD Capacity:	5093	5093	(lbf)
			HD Deflection:	0.11	0.11	(in.)

Three-Term Equation Deflection Check

$$\delta_{sw} = \frac{8vh^3}{EAb} + \frac{vh}{1000G_a} + \frac{h\Delta_a}{b} \qquad (4.3\text{-}1)$$

Sheathing Type: 7/16 OSB APA Rated Sheathing

Nail Type: 8d common

	Pier 1-L	Pier 1-R	Pier 2-L	Pier 2-R	
$v_{unfactored}$:	402	402	402	402	(plf)
E:	1.30E+06	1.30E+06	1.30E+06	1.30E+06	(psi)
h:	12.00	8.50	8.50	12.00	(ft)
Qty:	2.00E+00	2.00E+00	2.00E+00	2.00E+00	
Stud Size:	2x6	2x6	2x6	2x6	
A Override:					($in.^2$)
A:	16.5	16.5	16.5	16.5	($in.^2$)
G_a:	22.0	22.0	22.0	22.0	(kips/in.)
b:	3.21	3.21	3.21	3.21	(ft)
HD Capacity:	5093	5093	5093	5093	(lbf)
HD Defl:	0.11	0.11	0.11	0.11	(in.)

Check Total Deflection of Wall System

Pier 1 (left)			Pier 1 (right)		
Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 1 Bending	Term 2 Shear	Term 3 Fastener
0.081	0.220	0.390	0.029	0.156	0.196
	Sum	0.690		Sum	0.380
Pier 2 (left)			**Pier 2 (right)**		
Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 1 Bending	Term 2 Shear	Term 3 Fastener
0.029	0.156	0.196	0.081	0.220	0.390
	Sum	0.380		Sum	0.690

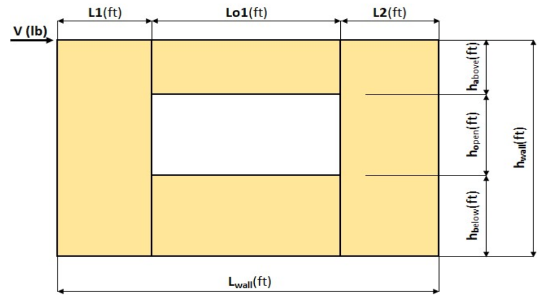
Total Defl.	
0.535	(in.)
0.0000	%drift

Comment: The 3-term equation is calibrated to be approximately equal to 4-term equation at 1.4*ASD capacity.

This version of the Force Transfer Around Openings calculator has expired.
Please go to **www.apawood.org** to download the latest version.

Project Information

Code:	2021 IBC	**Date:**	2/26/2025
Designer:	Chon Pieruccioni, PE		
Client:			
Project:	East Town Crossing - Clubhouse		
Wall Line:	Grid A (27'-5" Section) - (Level 1 Seismic)		



Shear Wall Calculation Variables

V	1461 lbf	Opening 1		
L1	3.21 ft	h_a	1.50 ft	
L2	3.21 ft	h_o	7.00 ft	
h_{wall}	12.00 ft	h_b	3.50 ft	
L_{wall}	27.63 ft	Lo1	21.21 ft	

Adj. Factor Method =		2bs/h
Wall Pier Aspect Ratio		Adj. Factor
$P1=h_o/L1=$	2.18	0.917
$P2=h_o/L2=$	2.18	0.917

Note to Designer: The width-to-height ratio of sheathing above or below the openings exceeds 6.5:1. Exercise caution when assuming fixity at corner regions, as assumed in this calculator.

1. Hold-down forces: $H = Vh_{wall}/L_{wall}$ 635 lbf

2. Unit shear above + below opening

First opening: $va1 = vb1 = H/(h_a+h_b) =$ 127 plf

3. Total boundary force above + below openings

First opening: O1 = va1 x (Lo1) = 2692 lbf

4. Corner forces

F1 = O1(L1)/(L1+L2) =	1346 lbf
F2 = O1(L2)/(L1+L2) =	1346 lbf

5. Tributary length of openings

T1 = (L1*Lo1)/(L1+L2) =	10.61 ft
T2 = (L2*Lo1)/(L1+L2) =	10.61 ft

6. Unit shear beside opening

v1 = (V/L)(L1+T1)/L1 =	228 plf	
v2 = (V/L)(T2+L2)/L2 =	228 plf	
Check v1*L1+v2*L2=V?	1461 lbf	**OK**

7. Resistance to corner forces

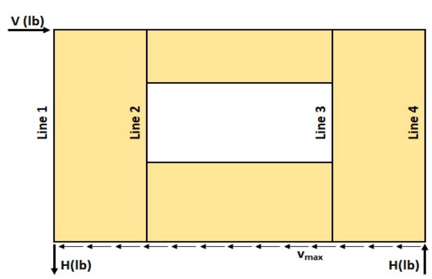
R1 = v1*L1 =	731 lbf
R2 = v2*L2 =	731 lbf

8. Difference corner force + resistance

R1-F1 =	-615 lbf
R2-F2 =	-615 lbf

9. Unit shear in corner zones

vc1 = (R1-F1)/L1 =	-192 plf
vc2 = (R2-F2)/L2 =	-192 plf



Check Summary of Shear Values for One Opening

Line 1: $vc1(h_a+h_b)+v1(h_o)=H$?		-958	1593	635 lbf
Line 2: $va1(h_a+h_b)-vc1(h_a+h_b)-v1(h_o)=0$?	635	-958	1593	0
Line 3: $va1(h_a+h_b)-vc2(h_a+h_b)-v1(h_o)=0$?	635	-958	1593	0
Line 4: $vc2(h_a+h_b)+v2(h_o)=H$?		-958	1593	635 lbf

Design Summary*

Req. Sheathing Capacity	248 plf	**	**4-Term Deflection**	0.241 in.	**3-Term Deflection**	0.303 in.
Req. Strap Force	1346 lbf		**4-Term Story Drift %**	0.007 %	**3-Term Story Drift %**	0.008 %
Req. HD Force (H)	635 lbf					
Req. Shear Wall Anchorage Force (v_{max})	53 plf					

****Req. Sheathing Capacity has been adjusted per the Aspect Ratio Adjustment Factor**

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

Project Information

Code:	2021 IBC	**Date:**	2/26/2025
Designer:	Chon Pieruccioni, PE		
Client:			
Project:	East Town Crossing - Clubhouse		
Wall Line:	Grid A (27'-5" Section) - (Level 1 Seismic)		

Shear Wall Deflection Calculation Variables

Unfactored Shear Load $V_{unfactored}$: 1461 (lbf)

Sheathing Type: 7/16 OSB
Grade: APA Rated Sheathing

G_t Override:
G_a Override:

Wood End Post Values:
Species: HF#2
E: 1.30E+06 (psi)

Enter individual post sizes below.

C_d: 4.00

Nail Type: 8d common (penny weight)

	Pier 1	Pier 2	
Nail Spacing:	4	4	(in.)
HD Capacity:	5093	5093	(lbf)
HD Deflection:	0.11	0.11	(in.)

Four-Term Equation Deflection Check

$$\Delta = \frac{8vh^3}{EAb} + \frac{vh}{Gt} + 0.75he_n + d_a\frac{h}{b} \qquad \text{(Equation 23-2)}$$

Sheathing Type: 7/16 OSB APA Rated Sheathing
Nail Type: 8d common

	Pier 1-L	Pier 1-R	Pier 2-L	Pier 2-R	
$v_{unfactored}$:	228	228	228	228	(plf)
E:	1.30E+06	1.30E+06	1.30E+06	1.30E+06	(psi)
h:	12.00	8.50	8.50	12.00	(ft)
Qty:	2.00E+00	2.00E+00	2.00E+00	2.00E+00	
Stud Size:	2x6	2x6	2x6	2x6	
A Override:					($in.^2$)
A:	16.5	16.5	16.5	16.5	($in.^2$)
G_t:	83,500	83,500	83,500	83,500	(lbf/in.)
Nail Spacing:	4	4	4	4	(in.)
V_n:	76	76	76	76	(plf)
e_n:	0.0022	0.0022	0.0022	0.0022	(in.)
b:	3.21	3.21	3.21	3.21	(ft)
HD Capacity:	5093	5093	5093	5093	(lbf)
HD Defl:	0.11	0.11	0.11	0.11	(in.)

Check Total Deflection of Wall System

Pier 1 (left)				Pier 1 (right)			
Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 4 HD-1	Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 4 HD-2
0.046	0.033	0.019	0.220	0.016	0.023	0.014	0.111
		Sum	0.318			Sum	0.164
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
0.016	0.023	0.014	0.111	0.046	0.033	0.019	0.220
		Sum	0.164			Sum	0.318

Total Defl.	
0.241	(in.)
0.0067	%drift

Project Information

Code:	2021 IBC	**Date:**	2/26/2025
Designer:	Chon Pieruccioni, PE		
Client:			
Project:	East Town Crossing - Clubhouse		
Wall Line:	Grid A (27'-5" Section) - (Level 1 Seismic)		

Shear Wall Deflection Calculation Variables

Unfactored Shear Load $V_{unfactored}$: 1461 (lbf)

Wood End Post Values:

Sheathing Type:	7/16 OSB	Species:	HF#2	Nail Type:	8d common (penny weight)
Grade:	APA Rated Sheathing	E:	1.30E+06 (psi)		

					Pier 1	Pier 2	
G_t Override:				Nail Spacing:	4	4	(in.)
G_a Override:		C_d:	4.00	HD Capacity:	5093	5093	(lbf)
				HD Deflection:	0.11	0.11	(in.)

Three-Term Equation Deflection Check

$$\delta_{sw} = \frac{8vh^3}{EAb} + \frac{vh}{1000G_a} + \frac{h\Delta_a}{b} \quad (4.3\text{-}1)$$

Sheathing Type: 7/16 OSB APA Rated Sheathing

Nail Type: 8d common

	Pier 1-L	Pier 1-R	Pier 2-L	Pier 2-R	
$v_{unfactored}$:	228	228	228	228	(plf)
E:	1.30E+06	1.30E+06	1.30E+06	1.30E+06	(psi)
h:	12.00	8.50	8.50	12.00	(ft)
Qty:	2.00E+00	2.00E+00	2.00E+00	2.00E+00	
Stud Size:	2x6	2x6	2x6	2x6	
A Override:					(in.^{2})
A:	16.5	16.5	16.5	16.5	(in.^{2})
G_a:	22.0	22.0	22.0	22.0	(kips/in.)
b:	3.21	3.21	3.21	3.21	(ft)
HD Capacity:	5093	5093	5093	5093	(lbf)
HD Defl:	0.11	0.11	0.11	0.11	(in.)

Check Total Deflection of Wall System

Pier 1 (left)			Pier 1 (right)		
Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 1 Bending	Term 2 Shear	Term 3 Fastener
0.046	0.124	0.220	0.016	0.088	0.111
	Sum	0.390		Sum	0.215
Pier 2 (left)			**Pier 2 (right)**		
Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 1 Bending	Term 2 Shear	Term 3 Fastener
0.016	0.088	0.111	0.046	0.124	0.220
	Sum	0.215		Sum	0.390

Total Defl.	
0.303	(in.)
0.0084	%drift

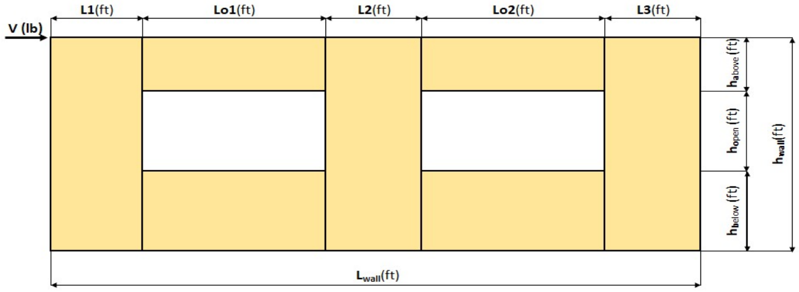
Comment: The 3-term equation is calibrated to be approximately equal to 4-term equation at 1.4*ASD capacity.

APA

This version of the Force Transfer Around Openings calculator has expired.
Please go to **www.apawood.org** to download the latest version.

Project Information

Code:	IBC 2021	**Date:**	2/26/2025
Designer:	Chon Pieruccioni, PE		
Client:			
Project:	East Town Crossing - Clubhouse		
Wall Line:	Grid H (34'-6" Section) - (Level 2 Seismic)		



Shear Wall Calculation Variables

V	3708 lbf		Opening 1		Opening 2	Adj. Factor Method =		2bs/h
L1	5.20 ft	h_a1	2.00 ft	h_a2	2.00 ft	Wall Pier Aspect Ratio		Adj. Factor
L2	5.30 ft	h_o1	2.50 ft	h_o2	2.50 ft	$P1=h_o/L1=$	0.48	N/A
L3	6.00 ft	h_b1	4.50 ft	h_b2	4.50 ft	$P2=h_o/L2=$	0.47	N/A
h_{wall}	9.00 ft	Lo1	9.00 ft	Lo2	9.00 ft	$P3=h_o/L3=$	0.42	N/A
L_{wall}	34.50 ft							

1. Hold-down forces: $H = Vh_{wall}/L_{wall}$ 967 lbf

2. Unit shear above + below opening

First opening: $va1 = vb1 = H/(h_a1+h_b1) =$	149 plf
Second opening: $va2 = vb2 = H/(h_a2+h_b2) =$	149 plf

3. Total boundary force above + below openings

First opening: O1 = va1 x (Lo1) =	1339 lbf
Second opening: O2 = va2 x (Lo2) =	1339 lbf

4. Corner forces

F1 = O1(L1)/(L1+L2) =	663 lbf
F2 = O1(L2)/(L1+L2) =	676 lbf
F3 = O2(L2)/(L2+L3) =	628 lbf
F4 = O2(L3)/(L2+L3) =	711 lbf

5. Tributary length of openings

T1 = (L1*Lo1)/(L1+L2) =	4.46 ft
T2 = (L2*Lo1)/(L1+L2) =	4.54 ft
T3 = (L2*Lo2)/(L2+L3) =	4.22 ft
T4 = (L3*Lo2)/(L2+L3) =	4.78 ft

6. Unit shear beside opening

v1 = (V/L)(L1+T1)/L1 =	200 plf	
v2 = (V/L)(T2+L2+T3)/L2 =	285 plf	
v3 = (V/L)(T4+L3)/L3 =	193 plf	
Check v1*L1+v2*L2+v3*L3=V?	3708 lbf	**OK**

7. Resistance to corner forces

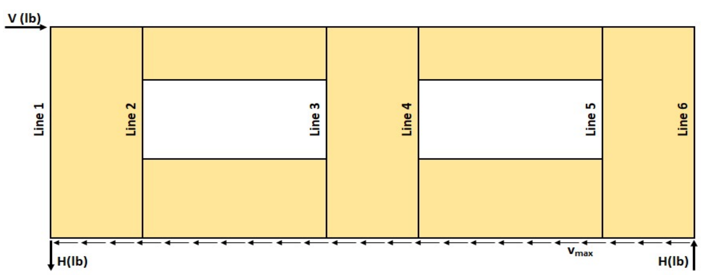
R1 = v1*L1 =	1038 lbf
R2 = v2*L2 =	1512 lbf
R3 = v3*L3 =	1158 lbf

8. Difference corner force + resistance

R1-F1 =	375 lbf
R2-F2-F3 =	207 lbf
R3-F4 =	447 lbf

9. Unit shear in corner zones

vc1 = (R1-F1)/L1 =	72 plf
vc2 = (R2-F2-F3)/L2 =	39 plf
vc3 = (R3-F4)/L3 =	75 plf



Check Summary of Shear Values for Two Openings

Line 1: $vc1(h_a1+h_b1)+v1(h_o1)=H?$		468	499	967 lbf
Line 2: $va1(h_a1+h_b1)-vc1(h_a1+h_b1)-v1(h_o1)=0?$	967	468	499	0
Line 3: $vc2(h_a1+h_b1)+v2(h_o1)-va1(h_a1+h_b1)=0?$	254	713	967	0
Line 4: $va2(h_a2+h_b2)-v2(h_o2)-vc2(h_a2+h_b2)=0?$	967	713	254	0
Line 5: $va2(h_a2+h_b2)-vc3(h_a2+h_b2)-v3(h_o2)=0?$	967	485	483	0
Line 6: $vc3(h_a2+h_b2)+ v3(h_o2) = H?$		485	483	967 lbf

Design Summary*

Req. Sheathing Capacity	285 plf	**4-Term Deflection**	0.100 in.	**3-Term Deflection**	0.134 in.
Req. Strap Force	711 lbf	**4-Term Story Drift %**	0.004 %	**3-Term Story Drift %**	0.005 %
Req. HD Force	967 lbf				
Req. Shear Wall Anchorage Force	107 plf				

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

Project Information

Code:	IBC 2021	**Date:**	2/26/2025
Designer:	Chon Pieruccioni, PE		
Client:			
Project:	East Town Crossing - Clubhouse		
Wall Line:	Grid H (34'-6" Section) - (Level 2 Seismic)		

Shear Wall Deflection Calculation Variables

Unfactored Shear Load $V_{unfactored}$: 3708 (lbf)

Sheathing Type: 7/16 OSB
Grade: APA Rated Sheathing

G_t Override:
G_a Override:

Wood End Post Values:
Species: HF#2
E: 1.30E+06 (psi)

Enter individual post sizes below.

C_d: 4.00

Nail Type: 8d common (penny weight)

	Pier 1	Pier 3	
Nail Spacing:	4	4	(in.)
HD Capacity:	1938	1938	(lbf)
HD Deflection:	0.088	0.088	(in.)

Four-Term Equation Deflection Check

$$\Delta = \frac{8vh^3}{EAb} + \frac{vh}{Gt} + 0.75he_n + d_a\frac{h}{b} \qquad \text{(Equation 23-2)}$$

	Pier 1-L	Pier 1-R	Pier 2-L	Pier 2-R	Pier 3-L	Pier 3-R	
$v_{unfactored}$:	200	200	285	285	193	193	(plf)
E:	1.30E+06	1.30E+06	1.30E+06	1.30E+06	1.30E+06	1.30E+06	(psi)
h:	9.00	4.50	4.50	4.50	4.50	9.00	(ft)
Qty:	2	2	2	2	2	2	
Stud Size:	2x6	2x6	2x6	2x6	2x6	2x6	
A Override:							(in.^{2})
A:	16.5	16.5	16.5	16.5	16.5	16.5	(in.^{2})
G_t:	83,500	83,500	83,500	83,500	83,500	83,500	(lbf/in.)
Nail Spacing:	4	4	4	4	4	4	(in.)
V_n:	67	67	95	95	64	64	(plf)
e_n:	0.0015	0.0015	0.0043	0.0043	0.0013	0.0013	(in.)
b:	5.20	5.20	5.30	5.30	6.00	6.00	(ft)
HD Capacity:	1938	1938	1938	1938	1938	1938	(lbf)
HD Defl:	0.088	0.088	0.088	0.088	0.088	0.088	(in.)

Sheathing Type: 7/16 OSB APA Rated Sheathing
Nail Type: 8d common

Check Total Deflection of Wall System

Pier 1 (left)				Pier 1 (right)			
Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 4 HD-1	Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 4 HD-2
0.010	0.022	0.010	0.141	0.001	0.011	0.005	0.035
		Sum	0.183			Sum	0.052
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
0.002	0.015	0.014	0.049	0.002	0.015	0.014	0.049
		Sum	0.081			Sum	0.081
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
0.001	0.010	0.004	0.030	0.009	0.021	0.009	0.118
		Sum	0.046			Sum	0.157

Total Defl.	
0.100	(in.)
0.0037	%drift

Project Information

Code:	IBC 2021	**Date:**	2/26/2025
Designer:	Chon Pierucccioni, PE		
Client:			
Project:	East Town Crossing - Clubhouse		
Wall Line:	Grid H (34'-6" Section) - (Level 2 Seismic)		

Shear Wall Deflection Calculation Variables

Unfactored Shear Load $V_{unfactored}$: 3708 (lbf)

Wood End Post Values:

Sheathing Type:	7/16 OSB	Species:	HF#2	Nail Type:	8d common	(penny weight)
Grade:	APA Rated Sheathing	E:	1.30E+06 (psi)			

					Pier 1	Pier 3	
G_t Override:				Nail Spacing:	4	4	(in.)
G_a Override:		C_d:	4.00	HD Capacity:	1938	1938	(lbf)
				HD Deflection:	0.088	0.088	(in.)

Three-Term Equation Deflection Check

$$\delta_{sw} = \frac{8vh^3}{EAb} + \frac{vh}{1000G_a} + \frac{h\Delta_a}{b} \qquad (4.3\text{-}1)$$

	Pier 1-L	Pier 1-R	Pier 2-L	Pier 2-R	Pier 3-L	Pier 3-R	
$v_{unfactored}$:	200	200	285	285	193	193	(plf)
E:	1.30E+06	1.30E+06	1.30E+06	1.30E+06	1.30E+06	1.30E+06	(psi)
h:	9.00	4.50	4.50	4.50	4.50	9.00	(ft)
Qty:	2	2	2	2	2	2	
Stud Size:	2x6	2x6	2x6	2x6	2x6	2x6	
A Override:							(in.^{2})
A:	16.5	16.5	16.5	16.5	16.5	16.5	(in.^{2})
G_a:	22.0	22.0	22.0	22.0	22.0	22.0	(kips/in.)
b:	5.20	5.20	5.30	5.30	6.00	6.00	(ft)
HD Capacity:	1938	1938	1938	1938	1938	1938	(lbf)
HD Defl:	0.088	0.088	0.088	0.088	0.088	0.088	(in.)

Sheathing Type: 7/16 OSB APA Rated Sheathing

Nail Type: 8d common

Check Total Deflection of Wall System

Pier 1 (left)			Pier 1 (right)		
Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 1 Bending	Term 2 Shear	Term 3 Fastener
0.010	0.082	0.141	0.001	0.041	0.035
	Sum	0.233		Sum	0.077
Pier 2 (left)			**Pier 2 (right)**		
Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 1 Bending	Term 2 Shear	Term 3 Fastener
0.002	0.058	0.049	0.002	0.058	0.049
	Sum	0.110		Sum	0.110
Pier 3 (left)			**Pier 3 (right)**		
Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 1 Bending	Term 2 Shear	Term 3 Fastener
0.001	0.039	0.030	0.009	0.079	0.118
	Sum	0.070		Sum	0.206

Total Defl.	
0.134	(in.)
0.0050	%drift

Comment: The 3-term equation is calibrated to be approximately equal to 4-term equation at 1.4*ASD capacity.