Amdor Residence

Structural Calculations

Engineer's seal applies to this entire calculation packet.

This engineering report is valid only for the aforementioned building located at 1601 Sailing Hawks Drive #118, Lake Havasu City, Arizona. This report is to be used only once and may not be copied or reproduced without the written consent of LEI Engineers and Surveyors, Inc.

LEI

- A Utah Corporation -

ENGINEERS

SURVEYORS

PLANNERS

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LEI Project #:

2025-2039

Location:

Lake Havasu City, AZ

Date:

2/26/2025

Engineered by:

M. Larrabee

Reviewed by:

Registered Professional Engineer (Civil)
CERTIFICATE No.
66953
ERIC B.
MURRAY
Signed: 03/21/2025
ARIZONA U. S. A.

Applies to pages 1-81

Structural Review for:	**Amdor Residence**
Location:	**Lake Havasu City, AZ**
Job #:	**2025-2039**
Engineered by:	**M. Larrabee**
Code:	**2018 IBC**

Loadings

Risk Category: II

Ground Snow Load:

Elevation = 1000 ft
p_g = 20.0 psf

Roof Snow Load:

C_t = 1.0
Roof Exposure C_e = 1.0 Partially
I_s = 1.0
p_f = 20.0 psf

Roof Dead Load:

DL = 25 psf (tile)
DL = 20 psf (flat roof)

Floor Loadings:

Live Load = 20 psf
Dead Load = 10 psf

Wind Loading:

Roofing Material = Shingle/Tile
Roof Pitch = 2.5 /12
Roof Angle = 11.8 degrees
Exposure Category = **C**
Mean Roof Height = 20
Wind Speed V = 99
Height & Exposure Factor λ = 1.29

p_{s30} Horizontal Pressures				p_{net30}	
zone A	zone B	zone C	zone D	zone 4	zone 5
18.24	-6.98	12.15	-4.04	16.48	22.06

p_s Horizontal Pressures				p_{net}	
zone A	zone B	zone C	zone D	zone 4	zone 5
23.5	**0.0**	**15.7**	**0.0**	**21.3**	**28.5**

Seismic Loading:

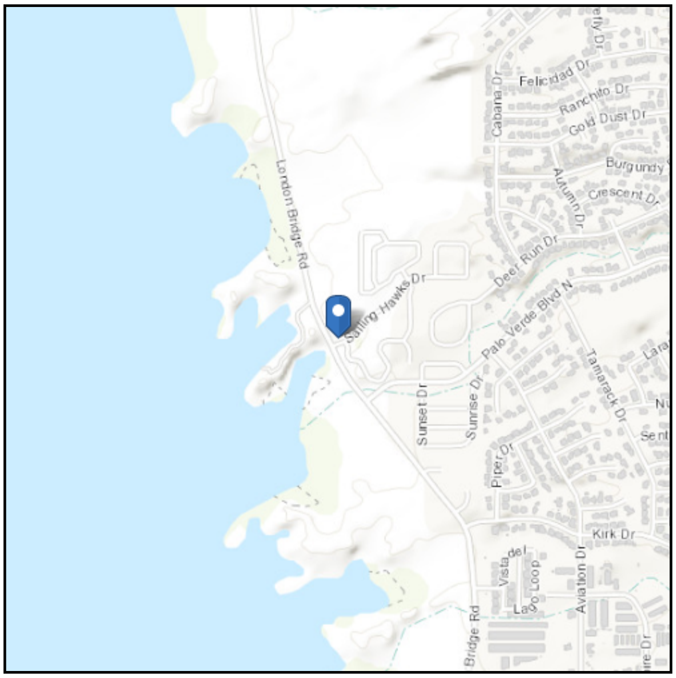
Number of Stories = 1
Roof diaphragm height h_r = 20 ft
I_e = 1.00
Fundamental Period T_a = 0.189 sec.
F = 1
Site Class = D **(Assumed)**
R factor = 6.5 Structural Sheathing
R factor = 6.5 Simpson Strong Wall
R factor = 6.5 Portal Frame
R factor = 2 Gypsum Sheathing
R factor = 5 Masonry Shear Wall
R factor = 4 Concrete Shear Wall
R factor = 2.5 Cantilever Steel Post
R factor = 4.5 Steel Moment Frame
S_S = 0.19
S_1 = 0.113
F_a = 1.6
F_v = 2.374
S_{MS} = 0.304
S_{M1} = 0.268262
S_{DS} = 0.203
S_{D1} = 0.179
T_o = 0.176488 sec.
T_s = 0.882441 sec.
Seismic Design Category = **C**

Soil Bearing Capacity: 1500 psf **(Assumed)**

ASCE Hazards Report

Address:
1601 Sailing Hawks Drive
Lake Havasu City, Arizona
86404

Standard:	ASCE/SEI 7-16
Risk Category:	II
Soil Class:	D - Default (see Section 11.4.3)
Latitude:	34.514198
Longitude:	-114.365425
Elevation:	455.5771940929054 ft (NAVD 88)



Wind

Results:

Wind Speed	99 Vmph
10-year MRI	69 Vmph
25-year MRI	75 Vmph
50-year MRI	80 Vmph
100-year MRI	85 Vmph

Data Source:	ASCE/SEI 7-16, Fig. 26.5-1B and Figs. CC.2-1–CC.2-4, and Section 26.5.2
Date Accessed:	Mon Feb 24 2025

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

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

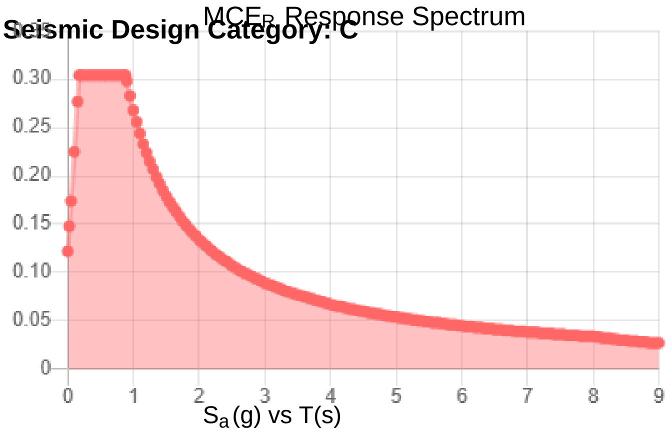
Seismic

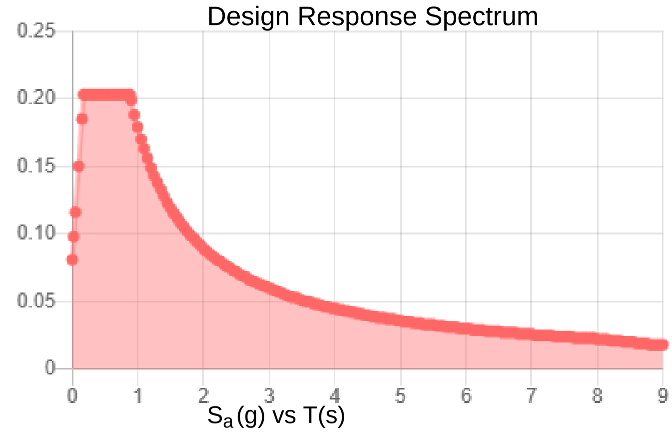
Site Soil Class: D - Default (see Section 11.4.3)

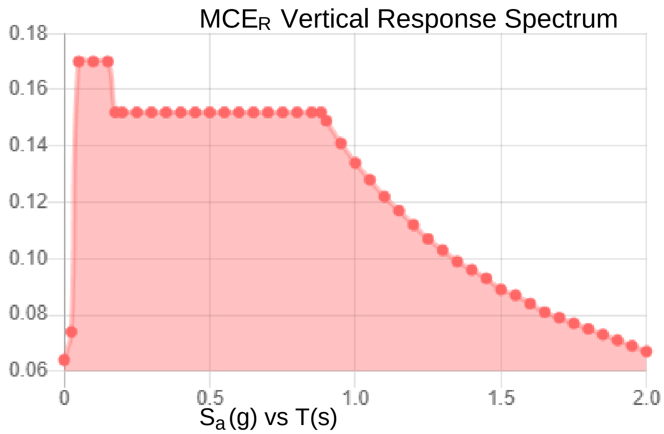
Results:

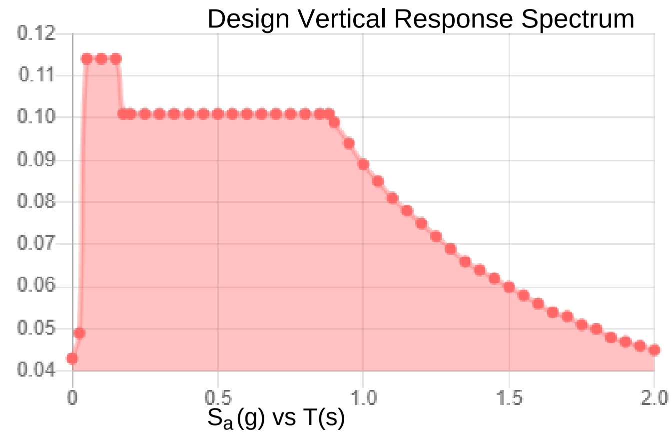
S_S :	0.19	S_{D1} :	0.179
S_1 :	0.113	T_L :	8
F_a :	1.6	PGA :	0.085
F_v :	2.374	PGA_M :	0.137
S_{MS} :	0.304	F_{PGA} :	1.6
S_{M1} :	0.268	I_e :	1
S_{DS} :	0.203	C_v :	0.7

Seismic Design Category: C









Data Accessed: **Mon Feb 24 2025**

Date Source:

USGS Seismic Design Maps based on ASCE/SEI 7-16 and ASCE/SEI 7-16 Table 1.5-2. Additional data for site-specific ground motion procedures in accordance with ASCE/SEI 7-16 Ch. 21 are available from USGS.

Non-Snow Roof Live Load Reduction
RB12

L_r=20R_1R_2 (ASCE 4.8.2)

A_T=	288	ft^2 (Tributary Arear sup
R_1=	0.91	
F=	0.25	in. rise/ft (roof slope)
R_2=	1	
Lr=	**18.24**	**psf**

Preface & Structural Notes

This engineering report is valid only for the following plan and location:

Amdor Residence

1601 Sailing Hawks Drive #118, Lake Havasu City, Arizona

NOTE TO PLAN CHECKER AND BUILDING INSPECTOR:

If the above address does not match the intended building address, notify LEI immediately @ 801-798-0555. This engineering packet is to be used only once for the above mentioned location and is not to be copied or reproduced without written consent of LEI Consulting Engineers and Surveyors, Inc.

Structural Notes:

General Notes

1 If values and assumptions stated in this report are incorrect, or if changes in the field are noticed which are different from those stated in this report, the engineer must be notified in order for the necessary corrections to be made.
2 If there are any discrepancies between the calculations and the drawings, these calculations shall govern.
3 This engineering report deals only with the structural parts of the building and does not apply to the non-structural parts.
4 If drawings are stamped in conjunction with this engineering report, certification pertains only to the structural elements of the drawings.
5 The general contractor is responsible for the method, means, and sequence of all structural erection except when specifically noted otherwise on the drawings. General contractor shall provide temporary shoring and bracing as his method of erection requires to provide adequate vertical and lateral support during erection. This shoring and bracing shall remain in place until all permanent members are placed and all final connections are completed including all roof and floor attachments.

Site Preparation

1 Do not place footings or foundations on disturbed soils, undocumented fill, debris, frozen soil, or in ponded water.
2 All slabs on grade shall be underlain by 4 in. of free-draining granular material such as "pea" gravel or 3/4 - 1 in. minus clean gravel.
3 Footings, foundations, excavations, grading and fill shall comply with the geotechnical report.

Concrete

1 All concrete footings and slabs on grade shall have a 28 day minimum strength = 2500 psi.
2 All concrete foundation walls and retaining walls shall have a 28 day minimum strength = 3000 psi.
3 Concrete shall be thoroughly consolidated by suitable means during placement.
4 Footings shall be centered below the wall and/or column above, typical unless noted otherwise.
5 Exterior footings shall bear below the effects of frost.
6 Stagger footing construction joints from wall construction joints above by at least 6 feet.
7 Reinforcing in continuous footings shall be continuous at corners and/or intersections by providing proper lap lengths and/or corner bars.
8 Interior slabs on grade shall be a min. of 4" thick.
9 Place vertical reinforcing in the center of the wall (except for retaining walls or when each face is specified).
10 Vertical reinforcing shall be dowelled to footing or structure below and to structure above with the same size bar and spacing, typical U.N.O.
11 Provide corner bars at all intersections and corners. Use same size bar and spacing as the horizontal reinforcing.
12 Horizontal reinforcing shall terminate at the ends of the walls and at openings with a standard hook.
13 Provide drainage at the base of retaining walls.

Reinforcing Steel

1 Reinforcing steel shall be new stock deformed bars and shall conform to ASTM A615, grade 60, with a design yield strength = 60 ksi.
2 Reinforcing steel shall be free of loose, flaky rust, scale, grease, oil, dirt, and other materials which might affect or impair bond.
3 Splices in continuous reinforcing shall be made on areas of compression and/or at points of minimum stress, typical U.N.O.
4 Lap splices shall be 40 bar diameters or 24" long in concrete. Dowels shall have a minimum of 30 bar diameters embedment.
5 Bends shall be made cold; do not use heat. Do not un-bend or re-bend a previously bent bar.
6 Reinforcing steel in concrete shall be securely anchored and tied in place prior to placing concrete and shall be positioned with the following minimum cover:
concrete cast against and permanently exposed to earth = 3"
concrete exposed to earth or weather = 1 1/2"
slabs on grade = center of slab

Structural Steel

1 Structural steel W-shapes shall conform to ASTM A992 grade 50 enhanced steel. Structural steel plates shall conform to ASTM A36.
2 Structural steel HSS-shapes shall conform to ASTM A500, grade B, with a min. yield strength Fy = 46 ksi (rectangular) or Fy = 42 ksi (round).
3 Structural pipe shall conform to ASTM A53, with a min. yield strength Fy = 36 ksi.
4 High strength bolts shall conform to ASTM A325, all other bolts shall conform to ASTM A307 or better.
5 Welded anchor studs and deformed bar anchors shall conform to the manufacturer's specs.
6 Fabrication shall be done in an approved fabricator's shop.
7 Use high strength (8000 psi min. at 28 days), non shrink, liquid epoxy grout beneath all steel base plates and bearing plates.
8 Bolt shall be bearing type connections U.N.O.
9 Steel to steel bolted connections shall be made with ASTM A325 high strength bolts and nuts, U.N.O.
10 All other bolted connections shall be made with bolts and nuts conforming to ASTM A307 U.N.O., including anchor bolts.
11 Bolted connections shall be tightened and shall have washers as required by AISC U.N.O.
12 Enlarging of holes shall be accomplished by means of reaming. Do not use a torch on any bolt holes.
13 Welded connections shall be made using low hydrogen matching filler material electrodes, U.N.O.
14 Welders shall be currently certified according to AWS within the last year. All welding procedures shall be pre-qualified. Welders shall follow welding procedures.
15 Welding and gas cutting shall be done per AWS.
16 Welds shall have the slag removed.

Structural Notes (cont):

Masonry Veneer Anchor Ties

1 Masonry veneer ties shall be one of the following:
   a. Dovetail anchors
   b. DX-10 seismic clip interlock system by Hohmann & Barnard
   c. Engineer approved 2 piece adjustable hot-dipped galvanized ties.
2 Maximum spacing shall be 16" o.c. horizontal and vertical.
3 Provide continuous horizontal galvanized #9 wire in center third of mortar joints at 16" o.c. Engage #9 wire with all anchor ties in seismic zone category E.

Wood Truss

1 Bottom chords of trusses, acting as ceiling members must be able to support a 10 psf live load per IBC requirements.
2 The truss manufacturer shall be responsible for the design and fabrication of the pre-engineered trusses.
3 The trusses shall be designed as per the attached engineering specs.
4 The trusses shall be designed to carry any additional loads due to mechanical units, overhead doors, roof overbuilds, etc.
5 The trusses shall be designed per the IBC and local ordinances.
6 All members shall be designed for combined stresses based on the worst loading condition.
7 The truss manufacturer shall indicate proper bracing of compression chord members @ 6' long (or longer), as well as bracing for truss erection.
8 All dimensions shall be field verified prior to fabrication.
9 General contractor shall be responsible for the installation of the trusses per the truss manufacturer's recommendations and specs.
10 No web or chord members shall be modified in the field without approval from the truss engineer.
11 The engineer is not responsible for the pre-engineered trusses, nor for the installation of the trusses.
12 General contractor is to verify truss layout is consistent with these plans and notify engineer of any deviations.

General Framing

1 All joists, rafters, posts and headers shall be DF-L #2 or equal U.N.O. If TJI's or equal are used, they must be installed per manufacturer's specs.
2 All joists and rafters shall have solid blocking at their bearing points.
3 All wood/lumber placed onto concrete shall be pressure treated or redwood.
4 Verify all beam sizes on the drawings with this report.
5 All beams and headers over 6'-0" shall be supported by double trimmer studs U.N.O.
6 All headers over 8'-0" shall shall have double king studs at each end U.N.O.
7 All over frame areas are to have full roof sheathing below.
8 Provide solid blocking and continuous bearing to foundation at all bearing point loads from above.
9 Provide double floor joists below all parallel bearing walls above.
10 Glulam beams shall be 24F-V4 DF/DF for single spans and 24F-V8 DF/DF for multiple spans and cantilevered spans.
11 Microllam beams shall be Laminated Veneer Lumber (LVL) with the following minimum design values: E=1,900,000 psi, Fb=2,600 psi, Fv=285 psi.
12 Parallam beams shall be Parallel Strand Lumber (PSL) with the following minimum design values: E=2,000,000 psi, Fb=2,900 psi, Fv=290 psi.
13 TimberStrand beams shall be Laminated Strand Lumber (LSL) w/ the following minimum design values:
   - 1-1/4" wide (rim board): E=1,300,000 psi, Fb=1,700 psi, Fv=425 psi.
   - 1-3/4" wide: E=1,550,000 psi, Fb=2,325 psi, Fv=310 psi.
14 All rafters and joists over 3 ft long shall be hangered if not supported by bottom bearing.
15 All hangers and other wood connections must be designed to carry the capacity of the member that they are supporting.
16 No structural member shall be cut or notched unless specifically shown, noted or approved by engineer.
17 Lag screws shall be inserted in a drilled pilot hole 60 - 75% of the shank diameter by turning with a wrench, not by driving with a hammer.
18 Nails are to be common wire U.N.O.
19 All bolt holes shall be drilled with a bit 1/32" to 1/16" larger than the nominal bolt diameter.
20 All joints in wall sheathing shall occur in the middle of a plate or block and nailed on each side of the joint w/ edge nailing per the shearwall schedul
21 All over built roof rafters shall be braced vertically to the trusses below at 4' o.c. max.
22 Double top plates are to have a minimum 48" lap splice w/ (8) 16d nails U.N.O.
23 All fasteners and connectors in contact with treated lumber shall be galvanized G90 or better.

Summary

Ceiling Joists:

CJ1: 2x12 DF-L#2 @ 16" o.c. as noted on plans
CJ2: 2x8 DF-L#2 @ 16" o.c. as noted on plans
3/4" APA rated T&G flooring to be nailed with 10d nails @ 6" o.c. edge, 12" o.c. field

Roof:

Trusses by others
Use 7/16" APA rated OSB sheathing w/ 8d nails @ 6" o.c. edge, 12" o.c. field
Overbuild to be 2" x 6" Timber @ 24" o.c.

Other:

All bearing headers to be (2) 2x10 (DF L #2 or better) unless noted otherwise
All exterior sheathing to be Shear Wall #1 unless noted otherwise
All glulam beams are to be 24F-V4 unless noted otherwise
Strap end lengths for shear walls (see also Simpson Coiled strap specs.):
CS16 = 14" CMST14 = 34" CMSTC16 = 25"

Beam Schedule

Desig.	Qty.	Size	Type
RB1	1	6 x 12	DF-L#2
RB2	1	4 x 12	DF-L#2
RB3	1	4 x 6	DF-L#2
RB4	1	6 x 8	DF-L#2
RB5	1	4 x 6	DF-L#2
RB6	1	6 x 8	DF-L#2
RB7	1	6 x 10	DF-L#2
RB8	1	4 x 8	DF-L#2
RB9	1	6 x 12	DF-L#2
RB10	1	5 1/8" x 12"	Glulam
RB11	1	6 x 12	DF-L#2
RB12	1	6 3/4" x 30"	Glulam
RB13	1	5 1/8" x 16 1/2"	Glulam
RB14	1	5 1/8" x 16 1/2"	Glulam
RB15	1	5 1/8" x 13 1/2"	Glulam
RB16	1	5 1/8" x 24"	Glulam
RB17	1	5 1/8" x 21"	Glulam
RB18	1	5 1/8" x 10 1/2"	Glulam
RB19	1	6 x 8	DF-L#2
RB20	1	4 x 12	DF-L#2
RB21	1	6 x 10	DF-L#2

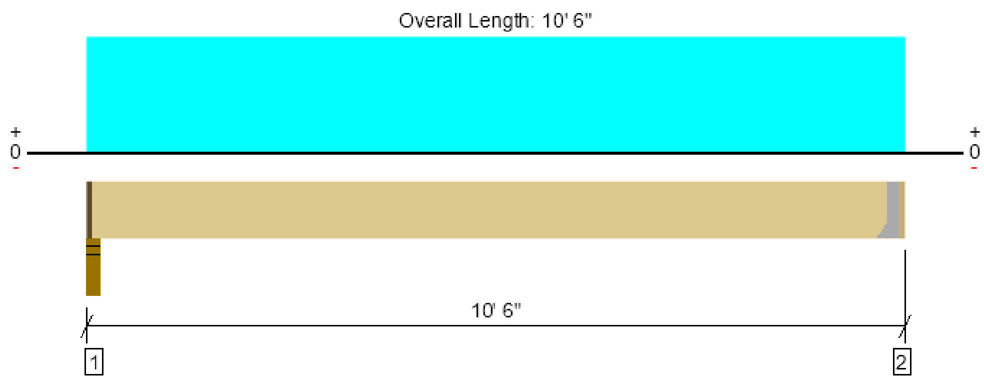
FORTEWEB

MEMBER REPORT

PASSED

Main, CJ1

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)	203 @ 10' 4 1/2"	1406 (1.50")	Passed (14%)	--	1.0 D + 1.0 L (All Spans)
Shear (lbs)	166 @ 9' 5 1/4"	2025	Passed (8%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	517 @ 5' 3 1/2"	2729	Passed (19%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.023 @ 5' 3 1/2"	0.254	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.034 @ 5' 3 1/2"	0.339	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)
TJ-Pro™ Rating	N/A	N/A	N/A	--	N/A

Member Length : 10' 3 1/4"
System : Floor
Member Type : Joist
Building Use : Residential
Building Code : IBC 2018
Design Methodology : ASD

- Deflection criteria: LL (L/480) and TL (L/360).
- 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 - DF	3.50"	2.25"	1.50"	71	141	212	1 1/4" Rim Board
2 - Hanger on 11 1/4" DF Ledger	1.50"	Hanger[1]	1.50"	69	139	208	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)	10' 3" o/c	
Bottom Edge (Lu)	10' 3" o/c	

•Maximum allowable bracing intervals based on applied load.

Connector: Simpson Strong-Tie

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

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

Vertical Load	Location (Side)	Spacing	Dead (0.90)	Floor Live (1.00)	Comments
1 - Uniform (PSF)	0 to 10' 6"	16"	10.0	20.0	Storage

Weyerhaeuser Notes

ForteWEB Software Operator	Job Notes
Michael B. Larrabee LEI Engineers & Surveyors Inc. (801) 798-0555 mikel@lei-eng.com	

Weyerhaeuser

2/24/2025 10:04:40 PM UTC
ForteWEB v3.8, Engine: V8.4.1.24, Data: V8.1.6.3
File Name: 2025-2039 ZG Amdor

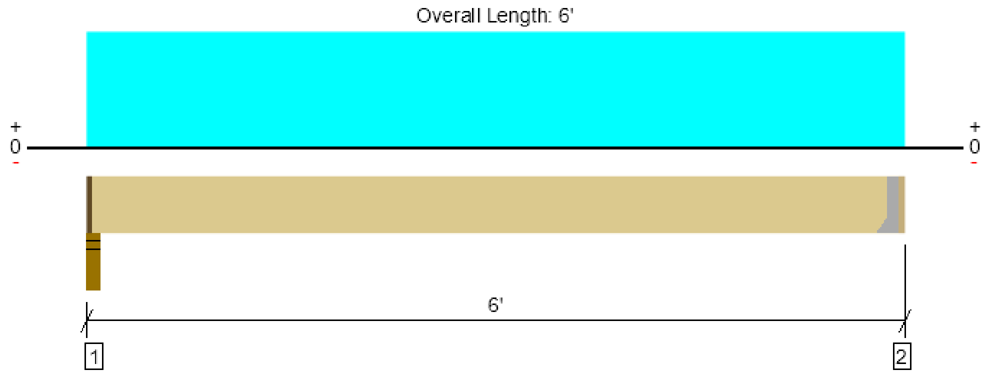
FORTEWEB

MEMBER REPORT

PASSED

Main, CJ2

1 piece(s) 2 x 8 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)	113 @ 5' 10 1/2"	1406 (1.50")	Passed (8%)	--	1.0 D + 1.0 L (All Spans)
Shear (lbs)	89 @ 5' 3 1/4"	1305	Passed (7%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	161 @ 3' 1/2"	1360	Passed (12%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.008 @ 3' 1/2"	0.142	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.012 @ 3' 1/2"	0.189	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)
TJ-Pro™ Rating	N/A	N/A	N/A	--	N/A

Member Length : 5' 9 1/4"
System : Floor
Member Type : Joist
Building Use : Residential
Building Code : IBC 2018
Design Methodology : ASD

- Deflection criteria: LL (L/480) and TL (L/360).
- 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 - DF	3.50"	2.25"	1.50"	41	81	122	1 1/4" Rim Board
2 - Hanger on 7 1/4" DF Ledger	1.50"	Hanger[1]	1.50"	39	79	118	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)	5' 9" o/c	
Bottom Edge (Lu)	5' 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
2 - Face Mount Hanger	LU26	1.50"	N/A	6-10dx1.5	4-10dx1.5	

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

Vertical Load	Location (Side)	Spacing	Dead (0.90)	Floor Live (1.00)	Comments
1 - Uniform (PSF)	0 to 6'	16"	10.0	20.0	Storage

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
Michael B. Larrabee LEI Engineers & Surveyors Inc. (801) 798-0555 mikel@lei-eng.com	

Weyerhaeuser

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ForteWEB v3.8, Engine: V8.4.1.24, Data: V8.1.6.3
File Name: 2025-2039 ZG Amdor

Ledgers

	L1		L2		L3		L4	
Roofing material =	Shingle/Tile		Shingle/Tile		Shingle/Tile		Shingle/Tile	
Roof Pitch=	2.5		2.5		2.5		2.5	
Angle=	11.8		11.8		11.8		11.8	
C_S =	1.000		1.000		1.000		1.000	
Increase for Drift/Valley=	1.000		1.000		1.000		1.000	
Effective snow load (psf)=	20		20		20		20	
Roof dead load (psf)=	25		25		25		25	
Floor live load (psf)=	20		20		20		20	
Floor dead load (psf)=	10		10		10		10	
Fastener spacing (ft)=	1.33		1.33		1.33		2	
Trib. Area $_{roof}$=	0		11		0		5.5	
Trib. Area $_{floor}$=	5		0		3		0	
w_S (plf) =	0		220		0		110	
w_L (plf) =	100		0		60		0	
w_D (plf) =	54		277		32		140	
$w_{self\ weight}$ (plf) =	3.7		2.4		2.4		2.4	
Point Load: Snow (lb)=								
Live (lb)=								
Dead (lb)=								
a (ft)=	0.665		0.665		0.665		1	
b (ft)=	0.665		0.665		0.665		1	
Add. uniform load (plf)=								
Allowable Live Deflection =	L/360		L/240		L/360		L/240	
Allowable Total Deflection =	L/240		L/180		L/240		L/180	
Left/Right Reaction: Factored (lb)=	**204**	**204**	**661**	**661**	**123**	**123**	**500**	**500**
Snow (lb)=	0	0	293	293	0	0	220	220
Live (lb)=	133	133	0	0	80	80	0	0
Dead (lb)=	71	71	369	369	43	43	280	280
V_{max} (lb)=	102		331		61		250	
M_{max} (ftlb)=	34		110		20		125	
Size Factor (C_F)=	1.00		1.20		1.20		1.20	
Volume Factor (C_v)=	1.00		1.00		1.00		1.00	
Duration Factor (C_d) =	1.00		1.15		1.00		1.15	
Beam Type (t,t1,tss,g,m,p,ts,rb)	t		t		t		t	
d (in)=	11.25		7.25		7.25		7.25	
b (in)=	1.5		1.5		1.5		1.5	
	Section OK		**Section OK**		**Section OK**		**Section OK**	
	Shear OK		**Shear OK**		**Shear OK**		**Shear OK**	
	Deflection OK		**Deflection OK**		**Deflection OK**		**Deflection OK**	

Fastener Options:

	L1		L2		L3		L4	
Nails:	16d		16d		16d		16d	
Nails into rim (1.25" min.) =	2	2	6	6	2	2	4	4
Nails into studs / beam >10D =	2	2	5	5	**1**	**1**	4	4
Lag Bolts:								
3/8" lag bolts into rim (1.25" min.) =	1	1	2	2	1	1	2	2
1/2" lag bolts into rim (1.25" min.) =	1	1	2	2	1	1	1	1
SDWS Screws:								
Through sheathing? (1/2" max.) =	No		Yes		No		No	
Through 5/8" gypsum? (# of layers) =	0		0		0		0	
Screws into rim (1.125" min.) =	1	1	3	3	1	1	2	2
	SDWS22400DB		SDWS22400DB		SDWS22400DB		SDWS22400DB	
Screws into studs =	**2**	**2**	**2**	**2**	2	2	**2**	**2**
	SDWS22400DB		**SDWS22500DB**		SDWS22400DB		**SDWS22400DB**	
Screws into beam / blocking =	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
	N/A		N/A		N/A		N/A	
Custom:								
	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A

Beams

	RB1	RB2	RB3	RB4	RB5	RB6	RB7	RB8	RB9	RB10-RB12
Roofing material =	Shingle/Tile	Shingle/Tile	Shingle/Tile	Shingle/Tile	Shingle/Tile	Shingle/Tile	Shingle/Tile	Shingle/Tile	Shingle/Tile	
Roof Pitch=	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	
Angle=	11.8	11.8	11.8	11.8	11.8	11.8	11.8	11.8	11.8	
C_S =	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	
Increase for Drift/Valley=	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	
Effective snow load (psf)=	20	20	20	20	20	20	20	20	20	
Roof dead load (psf)=	25	25	25	25	25	25	25	25	25	
Floor live load (psf)=	20	20	20	20	20	20	20	20	20	
Floor dead load (psf)=	10	10	10	10	10	10	10	10	10	
Length (ft)=	16	18	4	8	6	12	7	5	10	
Trib. Area $_{roof}$=	6	3	15.5	11	5	3	21	16.5	14.5	
Trib. Area $_{floor}$=	0	0	5	0	0	0	0	0	0	
w_S (plf) =	120	60	310	220	100	60	420	330	290	
w_L (plf) =	0	0	100	0	0	0	0	0	0	
w_D (plf) =	164	84	502	324	189	84	536	418	376	
$w_{self\ weight}$ (plf) =	13.7	8.5	4.2	8.9	4.2	8.9	11.3	5.5	13.7	
Point Load: Snow (lb)=										
Live (lb)=										
Dead (lb)=										
a (ft)=	8	9	2	4	3	6	3.5	2.5	5	
b (ft)=	8	9	2	4	3	6	3.5	2.5	5	
Add. uniform load (plf)=			60	40	60					
Allowable Live Deflection =	L/240	L/240	L/480	L/240	L/240	L/240	L/240	L/240	L/240	
Allowable Total Deflection =	L/180	L/180	L/360	L/180	L/180	L/180	L/180	L/180	L/180	
Left/Right Reaction: Factored (lb)=	**2270 2270**	**1292 1292**	**1623 1623**	**2176 2176**	**868 868**	**864 864**	**3347 3347**	**1870 1870**	**3331 3331**	
Snow (lb)=	960 960	540 540	620 620	880 880	300 300	360 360	1470 1470	825 825	1450 1450	
Live (lb)=	0 0	0 0	200 200	0 0	0 0	0 0	0 0	0 0	0 0	
Dead (lb)=	1310 1310	752 752	1003 1003	1296 1296	568 568	504 504	1877 1877	1045 1045	1881 1881	
V Redution Allowed (uniform load)=	No	No	No	No	No	No	No	No	No	
V Redution Allowed (point load)=	No	No	No	No	No	No	No	No	No	
V_{max} (lb)=	2270	1292	1623	2176	868	864	3347	1870	3331	
Location of M_{max} (ft)=	8.00	9.00	2.00	4.00	3.00	6.00	3.50	2.50	5.00	
M_{max} (ftlb)=	9079	5813	1623	4352	1301	2591	5857	2337	8328	
Size Factor (C_F)=	1.00	1.10	1.30	1.00	1.30	1.00	1.00	1.30	1.00	
Volume Factor (C_v)=	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	
Duration Factor (C_d) =	1.15	1.15	1.15	1.15	1.15	1.15	1.15	1.15	1.15	
Beam Type (t,t1,tss,g,m,p,ts,rb)	t	t	t	t	t	t	t	t	t	
d (in)=	11.5	11.25	5.5	7.5	5.5	7.5	9.5	7.25	11.5	
b (in)=	5.5	3.5	3.5	5.5	3.5	5.5	5.5	3.5	5.5	
$F'_{c\perp}$ =	625	625	625	625	625	625	625	625	625	
L/R Bearing Width (in)=	5.5 5.5	3.5 3.5	3.5 3.5	5.5 5.5	3.5 3.5	5.5 5.5	5.5 5.5	3.5 3.5	5.5 5.5	
L/R Req'd Bearing Length (in)=	0.66 0.66	0.59 0.59	0.74 0.74	0.63 0.63	0.40 0.40	0.25 0.25	0.97 0.97	0.85 0.85	0.97 0.97	
I (in^4)=	697.1	415.3	48.5	193.4	48.5	193.4	393.0	111.1	697.1	
F_b	900	900	900	900	900	900	900	900	900	
F'_b=	1035	1139	1346	1035	1346	1035	1035	1346	1035	
S (in^3)=	121.2	73.8	17.6	51.6	17.6	51.6	82.7	30.7	121.2	
S_{req}=	105	61	14	50	12	30	68	21	97	
	Section OK	**Section OK**	**Section OK**	**Section OK**	**Section OK**	**Section OK**	**Section OK**	**Section OK**	**Section OK**	
E (psi)=	1600000	1600000	1600000	1600000	1600000	1600000	1600000	1600000	1600000	
F'_v (psi)=	207	207	207	207	207	207	207	207	207	
f_v (psi)=	54	49	126	79	68	31	96	111	79	
	Shear OK	**Shear OK**	**Shear OK**	**Shear OK**	**Shear OK**	**Shear OK**	**Shear OK**	**Shear OK**	**Shear OK**	
Location of Max Deflection=	8.00	9.00	2.00	4.00	3.00	6.00	3.50	2.50	5.00	
Allowable Live Deflection (in)=	0.800	0.900	0.100	0.400	0.300	0.600	0.350	0.250	0.500	
Live Deflection (in)=	0.159	0.213	0.023	0.066	0.038	0.090	0.036	0.026	0.059	
Allowable Total Deflection (in)=	1.067	1.200	0.133	0.533	0.400	0.800	0.467	0.333	0.667	
Total Deflection (in)=	0.375	0.510	0.060	0.162	0.109	0.217	0.082	0.059	0.134	
	Deflection OK	**Deflection OK**	**Deflection OK**	**Deflection OK**	**Deflection OK**	**Deflection OK**	**Deflection OK**	**Deflection OK**	**Deflection OK**	

See attached calculations

Wood Beam

Project File: 2025-2039.ec6

LIC# : KW-06014416, Build:20.23.08.30 LEI CONSULTING ENGINEERS

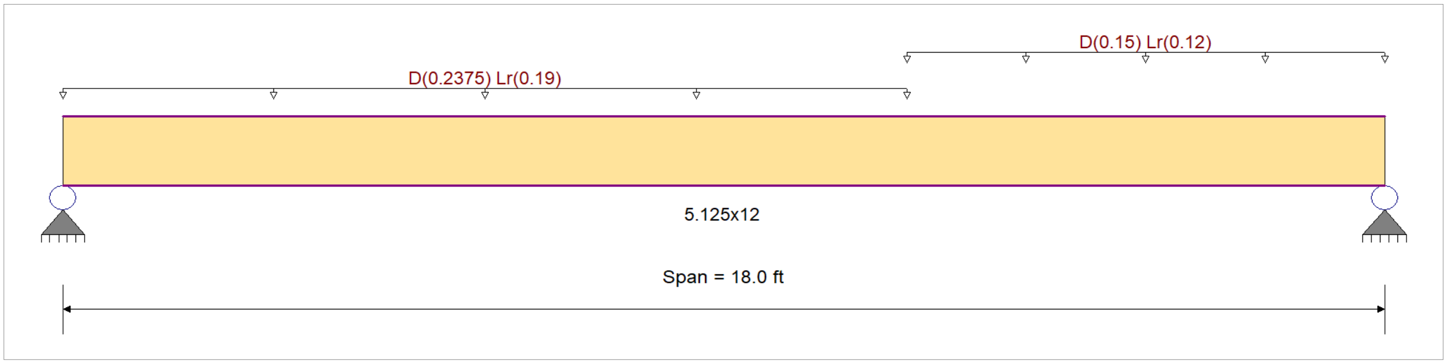
DESCRIPTION: RB10

CODE REFERENCES

Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16
Load Combination Set : ASCE 7-16

Material Properties

Analysis Method :	Allowable Stress Design	Fb +	2,400.0 psi	*E : Modulus of Elasticity*	
Load Combination :	ASCE 7-16	Fb -	1,850.0 psi	Ebend- xx	1,800.0 ksi
		Fc - Prll	1,650.0 psi	Eminbend - xx	950.0 ksi
Wood Species :	DF/DF	Fc - Perp	650.0 psi	Ebend- yy	1,600.0 ksi
Wood Grade :	24F-V4	Fv	265.0 psi	Eminbend - yy	850.0 ksi
		Ft	1,100.0 psi	Density	31.210 pcf
Beam Bracing :	Beam is Fully Braced against lateral-torsional buckling				



Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Beam self weight calculated and added to loading
Load for Span Number 1
Uniform Load : D = 0.0250, Lr = 0.020 ksf, Extent = 0.0 -->> 11.50 ft, Tributary Width = 9.50 ft, (Roof)
Uniform Load : D = 0.0250, Lr = 0.020 ksf, Extent = 11.50 -->> 18.0 ft, Tributary Width = 6.0 ft, (Roof)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.528** : 1	Maximum Shear Stress Ratio	=	**0.247** : 1
Section used for this span		**5.125x12**	Section used for this span		**5.125x12**
fb: Actual	=	1,583.28 psi	fv: Actual	=	81.66 psi
F'b	=	3,000.00 psi	F'v	=	331.25 psi
Load Combination		+D+Lr	Load Combination		+D+Lr
Location of maximum on span	=	8.606 ft	Location of maximum on span	=	0.000 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.304 in	Ratio =	710	>=360	Span: 1 : Lr Only
Max Upward Transient Deflection	0 in	Ratio =	0	<360	n/a
Max Downward Total Deflection	0.708 in	Ratio =	305	>=240	Span: 1 : +D+Lr
Max Upward Total Deflection	0 in	Ratio =	0	<240	n/a

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+Lr	1	0.7081	8.934		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	3.783	3.129
Max Upward from Load Combinations	3.783	3.129
Max Upward from Load Cases	2.155	1.791
D Only	2.155	1.791
+D+Lr	3.783	3.129
+D+0.750Lr	3.376	2.794
+0.60D	1.293	1.075
Lr Only	1.628	1.337

Wood Beam

Project File: 2025-2039.ec6

LIC# : KW-06014416, Build:20.23.08.30 | LEI CONSULTING ENGINEERS | (c) ENERCALC INC 1983-2023

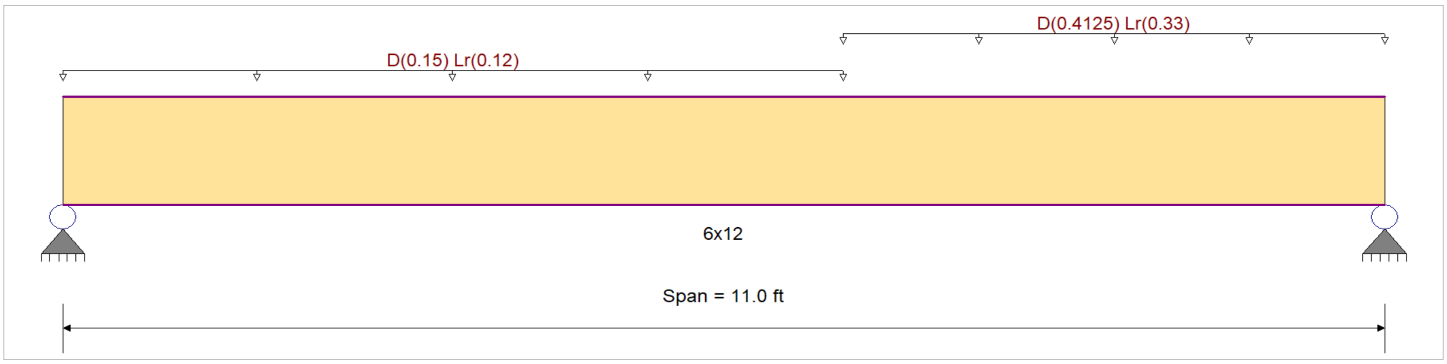
DESCRIPTION: RB11

CODE REFERENCES

Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16
Load Combination Set : ASCE 7-16

Material Properties

Analysis Method :	Allowable Stress Design	Fb +	750 psi	*E : Modulus of Elasticity*	
Load Combination :	ASCE 7-16	Fb -	750 psi	Ebend- xx	1300 ksi
		Fc - Prll	700 psi	Eminbend - xx	470 ksi
Wood Species :	Douglas Fir-Larch	Fc - Perp	625 psi		
Wood Grade :	No.2	Fv	170 psi		
		Ft	475 psi	Density	31.21 pcf
Beam Bracing :	Beam is Fully Braced against lateral-torsional buckling				



Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Beam self weight calculated and added to loading
Load for Span Number 1
Uniform Load : D = 0.0250, Lr = 0.020 ksf, Extent = 0.0 -->> 6.50 ft, Tributary Width = 6.0 ft
Uniform Load : D = 0.0250, Lr = 0.020 ksf, Extent = 6.50 -->> 11.0 ft, Tributary Width = 16.50 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.738** : 1	Maximum Shear Stress Ratio	=	**0.285** : 1
Section used for this span		**6x12**	Section used for this span		**6x12**
fb: Actual	=	692.04 psi	fv: Actual	=	60.56 psi
F'b	=	937.50 psi	F'v	=	212.50 psi
Load Combination		+D+Lr	Load Combination		+D+Lr
Location of maximum on span	=	6.704 ft	Location of maximum on span	=	10.077 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.071 in	Ratio =	1848 >=360	Span: 1 : Lr Only
Max Upward Transient Deflection	0 in	Ratio =	0 <360	n/a
Max Downward Total Deflection	0.166 in	Ratio =	796 >=180	Span: 1 : +D+Lr
Max Upward Total Deflection	0 in	Ratio =	0 <180	n/a

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+Lr	1	0.1656	5.741		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	1.995	3.252
Max Upward from Load Combinations	1.995	3.252
Max Upward from Load Cases	1.142	1.840
D Only	1.142	1.840
+D+Lr	1.995	3.252
+D+0.750Lr	1.782	2.899
+0.60D	0.685	1.104
Lr Only	0.853	1.412

Wood Beam

Project File: 2025-2039.ec6

LIC# : KW-06014416, Build:20.23.08.30 | LEI CONSULTING ENGINEERS |

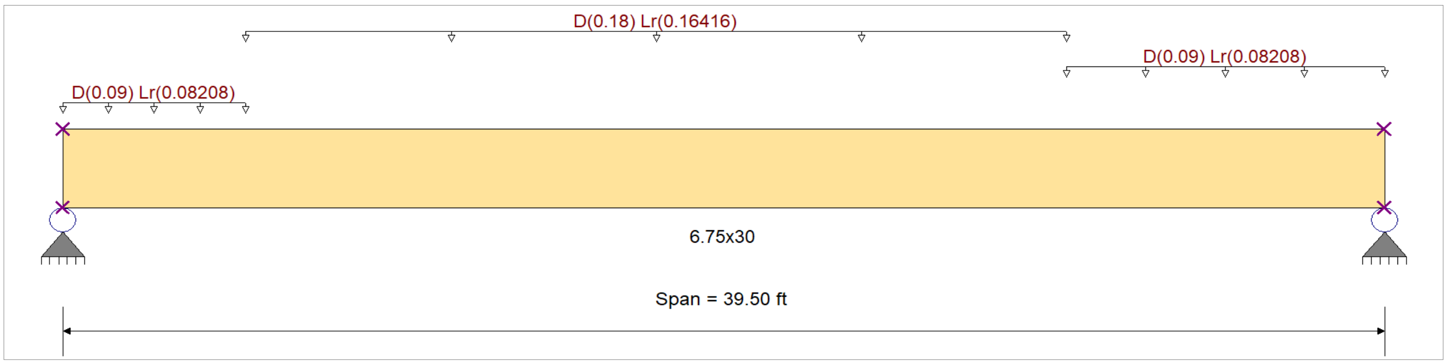
DESCRIPTION: RB12

CODE REFERENCES

Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16
Load Combination Set : ASCE 7-16

Material Properties

Analysis Method :	Allowable Stress Design	Fb +	2,400.0 psi	*E : Modulus of Elasticity*	
Load Combination :	ASCE 7-16	Fb -	1,850.0 psi	Ebend- xx	1,800.0 ksi
		Fc - Prll	1,650.0 psi	Eminbend - xx	950.0 ksi
Wood Species :	DF/DF	Fc - Perp	650.0 psi	Ebend- yy	1,600.0 ksi
Wood Grade :	24F-V4	Fv	265.0 psi	Eminbend - yy	850.0 ksi
Beam Bracing :	Completely Unbraced	Ft	1,100.0 psi	Density	31.210 pcf



Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Beam self weight calculated and added to loading
Load for Span Number 1
Uniform Load : D = 0.020, Lr = 0.01824 ksf, Extent = 0.0 -->> 5.50 ft, Tributary Width = 4.50 ft, (Roof)
Uniform Load : D = 0.020, Lr = 0.01824 ksf, Extent = 30.0 -->> 39.50 ft, Tributary Width = 4.50 ft, (Roof)
Uniform Load : D = 0.020, Lr = 0.01824 ksf, Extent = 5.50 -->> 30.0 ft, Tributary Width = 9.0 ft, (Roof)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.373** : 1	Maximum Shear Stress Ratio	=	**0.135** : 1
Section used for this span		**6.75x30**	Section used for this span		**6.75x30**
fb: Actual	=	835.78 psi	fv: Actual	=	44.87 psi
F'b	=	2,243.02 psi	F'v	=	331.25 psi
Load Combination		+D+Lr	Load Combination		+D+Lr
Location of maximum on span	=	19.462 ft	Location of maximum on span	=	0.000 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.301 in	Ratio =	1573	>=360	Span: 1 : Lr Only
Max Upward Transient Deflection	0 in	Ratio =	0	<360	n/a
Max Downward Total Deflection	0.720 in	Ratio =	658	>=240	Span: 1 : +D+Lr
Max Upward Total Deflection	0 in	Ratio =	0	<240	n/a

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+Lr	1	0.7199	19.750		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1 | Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	6.587	6.160
Max Upward from Load Combinations	6.587	6.160
Max Upward from Load Cases	3.858	3.635
D Only	3.858	3.635
+D+Lr	6.587	6.160
+D+0.750Lr	5.905	5.529
+0.60D	2.315	2.181

Wood Beam

Project File: 2025-2039.ec6

LIC# : KW-06014416, Build:20.23.08.30 LEI CONSULTING ENGINEERS

DESCRIPTION: RB12

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Lr Only	2.728	2.525

Beams

	RB13	RB14	RB15	RB16-RB17	RB18	RB19	RB20	RB21
Roofing material =	Shingle/Tile	Shingle/Tile	Shingle/Tile	See attached calculations	Shingle/Tile	Shingle/Tile	Shingle/Tile	Shingle/Tile
Roof Pitch=	2.5	2.5	2.5		2.5	2.5	2.5	2.5
Angle=	11.8	11.8	11.8		11.8	11.8	11.8	11.8
C_S =	1.000	1.000	1.000		1.000	1.000	1.000	1.000
Increase for Drift/Valley=	1.000	1.000	1.000		1.000	1.000	1.000	1.000
Effective snow load (psf)=	20	20	20		20	20	20	20
Roof dead load (psf)=	25	25	25		25	25	25	25
Floor live load (psf)=	20	20	20		20	20	20	20
Floor dead load (psf)=	10	10	10		10	10	10	10
Length (ft)=	16	16	15		13	6	5	8
Trib. Area $_{roof}$=	9	12.5	17		15	13.5	0	14
Trib. Area $_{floor}$=	0	0	0		0	0	5	0
w_S (plf) =	180	250	340		300	270	0	280
w_L (plf) =	0	0	0		0	0	100	0
w_D (plf) =	275	333	441		388	346	59	361
$w_{self\ weight}$ (plf) =	20.2	20.2	16.5		12.8	8.9	8.5	11.3
Point Load: Snow (lb)=			570			570		
Live (lb)=			0			0		
Dead (lb)=			713			713		
a (ft)=	8	8	13.5		6.5	2	2.5	4
b (ft)=	8	8	1.5		6.5	4	2.5	4
Add. uniform load (plf)=	30							
Allowable Live Deflection =	L/240	L/240	L/240		L/240	L/240	L/480	L/240
Allowable Total Deflection =	L/180	L/180	L/180		L/180	L/180	L/360	L/180
Left/Right Reaction: Factored (lb)=	**3641 3641**	**4661 4661**	**5989 7015**		**4471 4471**	**2704 2277**	**396 396**	**2565 2565**
Snow (lb)=	1440 1440	2000 2000	2607 3063		1950 1950	1190 1000	0 0	1120 1120
Live (lb)=	0 0	0 0	0 0		0 0	0 0	250 250	0 0
Dead (lb)=	2201 2201	2661 2661	3382 3952		2521 2521	1514 1277	146 146	1445 1445
V Redution Allowed (uniform load)=	No	No	No		No	No	No	No
V Redution Allowed (point load)=	No	No	No		No	No	No	No
V_{max} (lb)=	3641	4661	7015		4471	2704	396	2565
Location of M_{max} (ft)=	8.00	8.00	7.66		6.50	2.31	2.50	4.00
M_{max} (ftlb)=	14565	18645	22952		14530	4205	495	5131
Size Factor (C_F)=	1.00	1.00	1.00		1.00	1.00	1.10	1.00
Volume Factor (C_v)=	1.00	1.00	1.00		1.00	1.00	1.00	1.00
Duration Factor (C_d) =	1.15	1.15	1.15		1.15	1.15	1.00	1.15
Beam Type (t,t1,tss,g,m,p,ts,rb)	g	g	g		g	t	t	t
d (in)=	16.5	16.5	13.5		10.5	7.5	11.25	9.5
b (in)=	5.125	5.125	5.125		5.125	5.5	3.5	5.5
$F'_{c\perp}$ =	650	650	650		650	625	625	625
L/R Bearing Width (in)=	5.125 5.125	5.125 5.125	5.125 5.125		5.125 5.125	5.5 5.5	3.5 3.5	5.5 5.5
L/R Req'd Bearing Length (in)=	1.09 1.09	1.40 1.40	**1.80 2.11**		1.34 1.34	0.79 0.66	0.18 0.18	0.75 0.75
I (in^4)=	1918.5	1918.5	1050.8		494.4	193.4	415.3	393.0
F_b	2400	2400	2400		2400	900	900	900
F'_b=	2747	2747	2760		2760	1035	990	1035
S (in^3)=	232.5	232.5	155.7		94.2	51.6	73.8	82.7
S_{req}=	64	81	100		63	49	6	59
	Section OK	**Section OK**	**Section OK**		**Section OK**	**Section OK**	**Section OK**	**Section OK**
E (psi)=	1800000	1800000	1800000		1800000	1600000	1600000	1600000
F'_v (psi)=	304.75	304.75	304.75		304.75	207	180	207
f_v (psi)=	65	83	152		125	98	15	74
	Shear OK	**Shear OK**	**Shear OK**		**Shear OK**	**Shear OK**	**Shear OK**	**Shear OK**
Location of Max Deflection=	8.00	8.00	7.50		6.50	3.00	2.50	4.00
Allowable Live Deflection (in)=	0.800	0.800	0.750		0.650	0.300	0.125	0.400
Live Deflection (in)=	0.077	0.107	0.216		0.217	0.038	0.002	0.041
Allowable Total Deflection (in)=	1.067	1.067	1.000		0.867	0.400	0.167	0.533
Total Deflection (in)=	0.194	0.249	0.495		0.497	0.086	0.003	0.094
	Deflection OK	**Deflection OK**	**Deflection OK**		**Deflection OK**	**Deflection OK**	**Deflection OK**	**Deflection OK**

Wood Beam

Project File: 2025-2039.ec6

LIC# : KW-06014416, Build:20.23.08.30 LEI CONSULTING ENGINEERS (c) ENERCALC INC 1983-2023

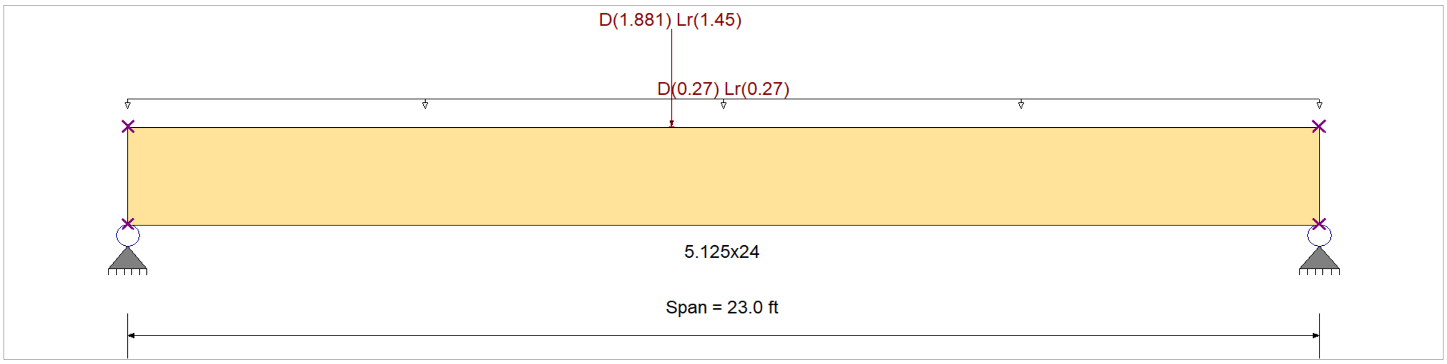
DESCRIPTION: RB16

CODE REFERENCES

Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16
Load Combination Set : ASCE 7-16

Material Properties

Analysis Method :	Allowable Stress Design	Fb +	2400 psi	*E : Modulus of Elasticity*	
Load Combination :	ASCE 7-16	Fb -	1850 psi	Ebend- xx	1800 ksi
		Fc - Prll	1650 psi	Eminbend - xx	950 ksi
Wood Species :	DF/DF	Fc - Perp	650 psi	Ebend- yy	1600 ksi
Wood Grade :	24F-V4	Fv	265 psi	Eminbend - yy	850 ksi
		Ft	1100 psi	Density	31.21 pcf
Beam Bracing :	Completely Unbraced				



Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Beam self weight calculated and added to loading
Uniform Load : D = 0.020, Lr = 0.020 ksf, Tributary Width = 13.50 ft, (Roof)
Point Load : D = 1.881, Lr = 1.450 k @ 10.50 ft, (RB9)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.550**: 1	Maximum Shear Stress Ratio	=	**0.266** : 1
Section used for this span		**5.125x24**	Section used for this span		**5.125x24**
fb: Actual	=	1,370.20 psi	fv: Actual	=	88.21 psi
F'b	=	2,491.30 psi	F'v	=	331.25 psi
Load Combination		+D+Lr	Load Combination		+D+Lr
Location of maximum on span	=	10.493 ft	Location of maximum on span	=	0.000 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.220 in	Ratio =	1252	>=360	Span: 1 : Lr Only
Max Upward Transient Deflection	0 in	Ratio =	0	<360	n/a
Max Downward Total Deflection	0.474 in	Ratio =	581	>=240	Span: 1 : +D+Lr
Max Upward Total Deflection	0 in	Ratio =	0	<240	n/a

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+Lr	1	0.4743	11.416		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	8.327	8.037
Max Upward from Load Combinations	8.327	8.037
Max Upward from Load Cases	4.434	4.270
D Only	4.434	4.270
+D+Lr	8.327	8.037
+D+0.750Lr	7.354	7.096
+0.60D	2.660	2.562
Lr Only	3.893	3.767

Wood Beam

Project File: 2025-2039.ec6

LIC# : KW-06014416, Build:20.23.08.30 LEI CONSULTING ENGINEERS

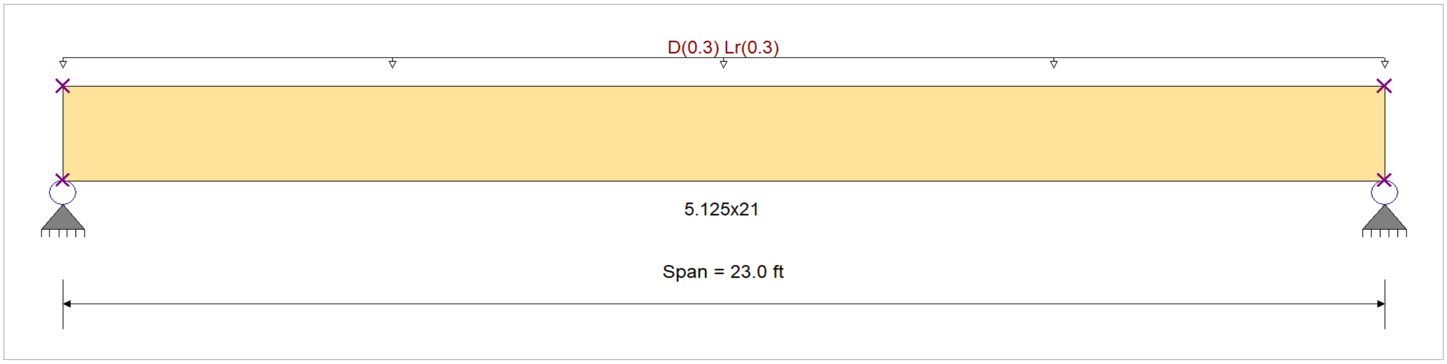
DESCRIPTION: RB17

CODE REFERENCES

Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16
Load Combination Set : ASCE 7-16

Material Properties

Analysis Method :	Allowable Stress Design	Fb +	2400 psi	*E : Modulus of Elasticity*	
Load Combination :	ASCE 7-16	Fb -	1850 psi	Ebend- xx	1800 ksi
		Fc - Prll	1650 psi	Eminbend - xx	950 ksi
Wood Species :	DF/DF	Fc - Perp	650 psi	Ebend- yy	1600 ksi
Wood Grade :	24F-V4	Fv	265 psi	Eminbend - yy	850 ksi
		Ft	1100 psi	Density	31.21 pcf
Beam Bracing :	Completely Unbraced				



Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Beam self weight calculated and added to loading
Uniform Load : D = 0.020, Lr = 0.020 ksf, Tributary Width = 15.0 ft, (Roof)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.497**: 1	Maximum Shear Stress Ratio	=	**0.258** : 1
Section used for this span		**5.125x21**	Section used for this span		**5.125x21**
fb: Actual	=	1,313.05 psi	fv: Actual	=	85.32 psi
F'b	=	2,639.68 psi	F'v	=	331.25 psi
Load Combination		+D+Lr	Load Combination		+D+Lr
Location of maximum on span	=	11.500 ft	Location of maximum on span	=	0.000 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.267 in	Ratio =	1034 >=360	Span: 1 : Lr Only
Max Upward Transient Deflection	0 in	Ratio =	0 <360	n/a
Max Downward Total Deflection	0.554 in	Ratio =	497 >=240	Span: 1 : +D+Lr
Max Upward Total Deflection	0 in	Ratio =	0 <240	n/a

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+Lr	1	0.5545	11.584		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	7.168	7.168
Max Upward from Load Combinations	7.168	7.168
Max Upward from Load Cases	3.718	3.718
D Only	3.718	3.718
+D+Lr	7.168	7.168
+D+0.750Lr	6.306	6.306
+0.60D	2.231	2.231
Lr Only	3.450	3.450

Shear Walls

Gridline 1

Simpson Strong Wall

No anchor bolts

	Length	Inside	Ratio h:w	Ratio 2w/h	SWS Wind	SWS Seismic	Wind $TA_{Roof-end}$	Wind $TA_{Roof-int.}$	Wind V_s	Seismic A_i	Seismic w_i	Seismic F	Seismic ρ	Seismic R	Seismic F_x	Seismic V_{final}	Check uplift DL	Check uplift h	Check uplift h'	Check uplift $Uplift_w$	Check uplift $Uplift_s$
panel 1							0	0	3795	1820	30430	1.000	1.000	6.5	949	**664**					
panel 2																					
panel 3																					
panel 4																					
panel 5																					
panel 6																					
panel 7																					
panel 8																					
panel 9																					
panel 10																					
$ASW_{1,2}$=	0	**0**	Total=	0.00																	

$TA_{Wall-end}$	$TA_{Wall-int}$	$V_{s\ min}$
71	297	**3795**
	$V_{additional}$ =	

$V_{additional}$ =

SW1	SW2	SW3	SW4	SW1	SW2	SW3	SW4
339 plf	495 plf	637 plf	832 plf	241 plf	350 plf	455 plf	595 plf
Total Resistance$_{wind}$				**Total Resistance$_{seismic}$**			
0	0	0	0	0	0	0	0

See the attached Simpson Strong Wall calculations

Perforated Shearwall 1:		**NOT USED**		$t = v =$	0
Total Length =		V_{wind}	V_{seis}	$Uplift_w$	$Uplift_s$
Height =		0	0	0	0
Max opening height=		**Ratio**			
C_o =		h:w	2w/h		
segment 1					
segment 2					
segment 3					
segment 4					
segment 5					
		Total=	0.00		

Perforated Shearwall 2:		**NOT USED**		$t = v =$	0
Total Length =		V_{wind}	V_{seis}	$Uplift_w$	$Uplift_s$
Height =		0	0	0	0
Max opening height=		**Ratio**			
C_o =		h:w	2w/h		
segment 1					
segment 2					
segment 3					
		Total=	0.00		

Perforated Shearwall 3:		**NOT USED**		$t = v =$	0
Total Length =		V_{wind}	V_{seis}	$Uplift_w$	$Uplift_s$
Height =		0	0	0	0
Max opening height=		**Ratio**			
C_o =		h:w	2w/h		
segment 1					
segment 2					
segment 3					
		Total=	0.00		

Gridline 2

Structural Sheathing

1/2" dia. anchor bolts

	Length	Inside	Ratio h:w	Ratio 2w/h	SWS Wind	SWS Seismic	Wind $TA_{Roof-end}$	Wind $TA_{Roof-int.}$	Wind V_s	Seismic A_i	Seismic w_i	Seismic F	Seismic ρ	Seismic R	Seismic F_x	Seismic V_{final}	Check uplift DL	Check uplift h	Check uplift h'	Check uplift $Uplift_w$	Check uplift $Uplift_s$
panel 1							0	0	3800	1820	30430	1.000	1.000	6.5	949	**664**					
panel 2																					
panel 3																					
panel 4																					
panel 5																					
panel 6																					
panel 7																					
panel 8																					
panel 9																					
panel 10																					
$ASW_{1,2}$=	0	**0**	Total=	0.00																	

$TA_{Wall-end}$	$TA_{Wall-int}$	$V_{s\ min}$
80	284	**3800**
	$V_{additional}$ =	

$V_{additional}$ =

SW1	SW2	SW3	SW4	SW1	SW2	SW3	SW4
339 plf	495 plf	637 plf	832 plf	241 plf	350 plf	455 plf	595 plf
Total Resistance$_{wind}$				**Total Resistance$_{seismic}$**			
9880	14426	18565	24248	7024	10200	13261	17341
Use SW1				Use SW1			

Use SW1
Use LSTHD8 holdowns each side of panel as noted on plans
Use 1/2" dia. anchor bolts @ 32" o.c.

Perforated Shearwall 1:				$t = v =$	130
Total Length =	33.5	V_{wind}	V_{seis}	$Uplift_w$	$Uplift_s$
Height =	15.5	3800	664	2021	353
Max opening height=	8	**Ratio**			
C_o =	0.953	h:w	2w/h		
segment 1	12.83	1.2 :1	1.00		
segment 2	17.75	0.9 :1	1.00		
segment 3					
segment 4					
segment 5					
		Total=	29.14		

Perforated Shearwall 2:		**NOT USED**		$t = v =$	0
Total Length =		V_{wind}	V_{seis}	$Uplift_w$	$Uplift_s$
Height =		0	0	0	0
Max opening height=		**Ratio**			
C_o =		h:w	2w/h		
segment 1					
segment 2					
segment 3					
		Total=	0.00		

Perforated Shearwall 3:		**NOT USED**		$t = v =$	0
Total Length =		V_{wind}	V_{seis}	$Uplift_w$	$Uplift_s$
Height =		0	0	0	0
Max opening height=		**Ratio**			
C_o =		h:w	2w/h		
segment 1					
segment 2					
segment 3					
		Total=	0.00		

SIMPSON STRONG-TIE COMPANY INC.
(800) 999-5099
5956 W. Las Positas Blvd., Pleasanton, CA 94588.
www.strongtie.com

Job Name: 2025-2039
Wall Name: Gridline #1 - Front RV Garage
Application: Balloon Framed

Design Criteria:

* 2018 International Bldg Code
* Wind
* 2500 psi concrete
* ASD Design Shear = 3795 lbs
* Nominal wall height = 14 ft

Selected Strong-Wall® Panel Solution:

Model	Type	W (in)	H (in)	T (in)	Sill Anchor	End Anchor Bolts	Total Axial Load (lbs)	Actual Uplift (lbs)
WSWH24x14	Wood	24	168	3.5	N/A	2 - 1"	2389	17022 lb
WSWH24x14	Wood	24	168	3.5	N/A	2 - 1"	2389	17022 lb

Actual Shear & Drift Distribution:

Model	RR Relative Rigidity	Actual Shear (lbs)		Allowable Shear (lbs)	Actual / Allow Shear	Actual Drift (in)	Drift Limit (in)
WSWH24x14	0.50	1898	≤	3365 OK	0.56	0.52	0.93
WSWH24x14	0.50	1898	≤	3365 OK	0.56	0.52	0.93

Notes:

1. Strong-Wall High-Strength Wood Shearwalls have been evaluated to the 2021 IBC/IRC. See www.strongtie.com for additional design and installation information.

2. Anchor templates are recommended for proper anchor bolt placement, and are required in some jurisdictions.

3. The applied vertical load shall be a concentric point load or a uniformly distributed load not exceeding the allowable vertical load. Alternatively, the load may be applied anywhere along the width of the panel if imposed by a continuous bearing vertical load transfer element such as a rimboard or beam. For eccentric axial loads applied directly to the panel, the allowable vertical load shall be divided by two.

4. Panels may be trimmed to a minimum height of 74½".

Disclaimer:

1.Project information

Project description: Amdor Residence
Location: WSWH24x14 Anchors (GL #1)
Design name: Design

Comment:

2. Input Data & Anchor Parameters

General
Design method:ACI 318-19
Units: Imperial units

Anchor Information:
Anchor type: Cast-in-place
Material: AB_H
Diameter (inch): 1.000
Effective Embedment depth, h_{ef} (inch): 10.000
Anchor category: -
Anchor ductility: Yes
h_{min} (inch): 12.63
C_{min} (inch): 6.00
S_{min} (inch): 6.00

Base Material
Concrete: Normal-weight
Concrete thickness, h (inch): 13.78
State: Cracked
Compressive strength, f'_c (psi): 2500
$\Psi_{c,V}$: 1.0
Reinforcement condition: B tension, B shear
Supplemental edge reinforcement: Not applicable
Reinforcement provided at corners: No
Ignore concrete breakout in tension: No
Ignore concrete breakout in shear: No
Ignore 6do requirement: No
Build-up grout pad: No

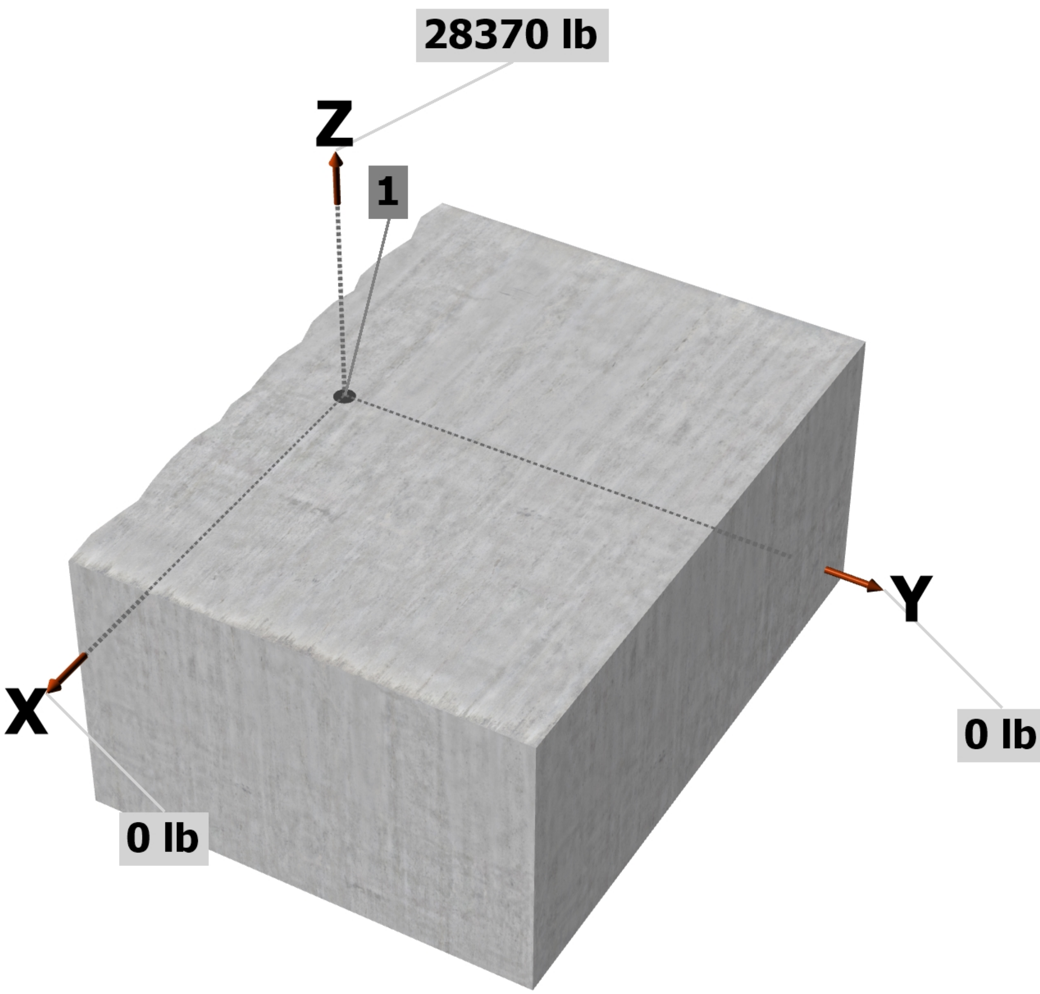
Recommended Anchor
Anchor Name: PAB Pre-Assembled Anchor Bolt - PAB8H (1"Ø)

Load and Geometry
Load factor source: ACI 318 Section 5.3
Load combination: not set
Seismic design: No
Anchors subjected to sustained tension: Not applicable
Apply entire shear load at front row: No
Anchors only resisting wind and/or seismic loads: Yes

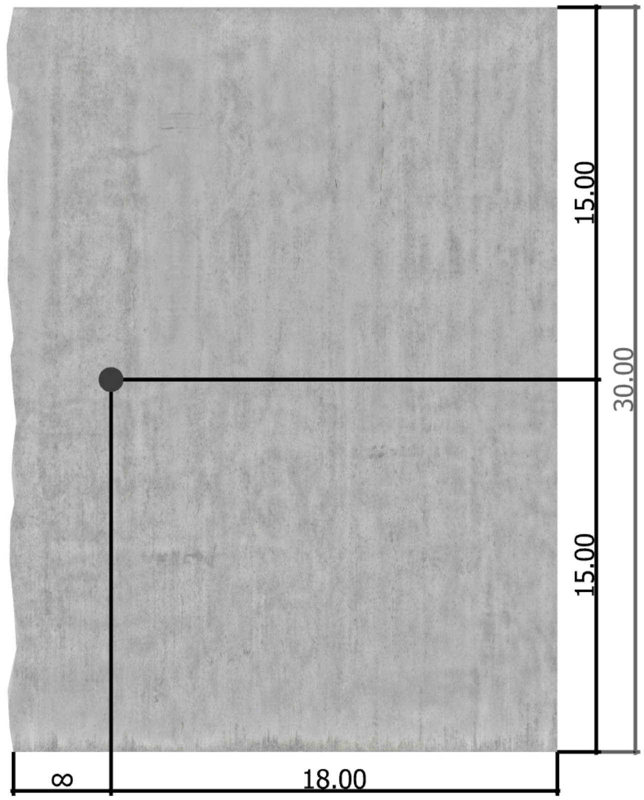
Strength level loads:

N_{ua} [lb]:	28370
V_{uax} [lb]:	0
V_{uay} [lb]:	0

<Figure 1>



<Figure 2>



3. Resulting Anchor Forces

Anchor	Tension load, N_{ua} (lb)	Shear load x, V_{uax} (lb)	Shear load y, V_{uay} (lb)	Shear load combined, $\sqrt{(V_{uax})^2+(V_{uay})^2}$ (lb)
1	28370.0	0.0	0.0	0.0
Sum	28370.0	0.0	0.0	0.0

Maximum concrete compression strain (‰): 0.00
Maximum concrete compression stress (psi): 0
Resultant tension force (lb): 28370
Resultant compression force (lb): 0
Eccentricity of resultant tension forces in x-axis, e'_{Nx} (inch): 0.00
Eccentricity of resultant tension forces in y-axis, e'_{Ny} (inch): 0.00

4. Steel Strength of Anchor in Tension (Sec. 17.6.1)

N_{sa} (lb)	ϕ	ϕN_{sa} (lb)
72720	0.75	54540

5. Concrete Breakout Strength of Anchor in Tension (Sec. 17.6.2)

$N_b = k_c \lambda_a \sqrt{f'_c} h_{ef}^{1.5}$ (Eq. 17.6.2.2.1)

k_c	λ_a	f'_c (psi)	h_{ef} (in)	N_b (lb)
24.0	1.00	2500	10.000	37947

$\phi N_{cb} = \phi\,(A_{Nc} / A_{Nco})\,\Psi_{ed,N}\Psi_{c,N}\Psi_{cp,N}N_b$ (Sec. 17.5.1.2 & Eq. 17.6.2.1a)

A_{Nc} (in^2)	A_{Nco} (in^2)	$c_{a,min}$ (in)	$\Psi_{ed,N}$	$\Psi_{c,N}$	$\Psi_{cp,N}$	N_b (lb)	ϕ	ϕN_{cb} (lb)
982.50	900.00	15.00	1.000	1.00	1.000	37947	0.70	28998

6. Pullout Strength of Anchor in Tension (Sec. 17.6.3)

$\phi N_{pn} = \phi \Psi_{c,P} N_p = \phi \Psi_{c,P} 8 A_{brg} f'_c$ (Sec. 17.5.1.2, Eq. 17.6.3.1 & 17.6.3.2.2a)

$\Psi_{c,P}$	A_{brg} (in^2)	f'_c (psi)	ϕ	ϕN_{pn} (lb)
1.0	5.15	2500	0.70	72156

11. Results

Interaction of Tensile and Shear Forces (Sec. 17.8)

Tension	Factored Load, N_{ua} (lb)	Design Strength, $øN_n$ (lb)	Ratio	Status
Steel	28370	54540	0.52	Pass
Concrete breakout	**28370**	**28998**	**0.98**	**Pass (Governs)**
Pullout	28370	72156	0.39	Pass

PAB8H (1"Ø) with hef = 10.000 inch meets the selected design criteria.

12. Warnings

- Designer must exercise own judgement to determine if this design is suitable.

Shear Walls

				Ratio		SWS		Wind			Seismic							Check uplift				
		Length	Inside	h:w	2w/h	Wind	Seismic	$TA_{Roof-end}$	$TA_{Roof-int.}$	V_s	A_i	w_i	F	ρ	R	F_x	V_{final}	DL	h	h'	$Uplift_w$	$Uplift_s$
Gridline 3	panel 1							0	0	1087	805	12962.5	1.000	1.000	6.5	404	**283**					
	panel 2							$TA_{Wall-end}$	$TA_{Wall-int}$	$V_{s\ min}$						$V_{additional}$ =						
	panel 3							77	0	**1087**												
Simpson Strong Wall	panel 4								$V_{additional}$ =													
	panel 5																					
No anchor bolts	panel 6																					
	panel 7																					
	panel 8																					
	panel 9																					
	panel 10																					
	$ASW_{1,2}$=	0	**0**	Total=	0.00																	

SW1	SW2	SW3	SW4	SW1	SW2	SW3	SW4
339 plf	495 plf	637 plf	832 plf	241 plf	350 plf	455 plf	595 plf
Total Resistance$_{wind}$				**Total Resistance**$_{seismic}$			
0	0	0	0	0	0	0	0

See the attached Simpson Strong Wall calculations

Perforated Shearwall 1:		**NOT USED**		$t = v =$	0
Total Length =		V_{wind}	V_{seis}	$Uplift_w$	$Uplift_s$
Height =		0	0	0	0
Max opening height=		**Ratio**			
C_o =		h:w	2w/h		
segment 1					
segment 2					
segment 3					
segment 4					
segment 5					
		Total=	0.00		

Perforated Shearwall 2:		**NOT USED**		$t = v =$	0
Total Length =		V_{wind}	V_{seis}	$Uplift_w$	$Uplift_s$
Height =		0	0	0	0
Max opening height=		**Ratio**			
C_o =		h:w	2w/h		
segment 1					
segment 2					
segment 3					
		Total=	0.00		

Perforated Shearwall 3:		**NOT USED**		$t = v =$	0
Total Length =		V_{wind}	V_{seis}	$Uplift_w$	$Uplift_s$
Height =		0	0	0	0
Max opening height=		**Ratio**			
C_o =		h:w	2w/h		
segment 1					
segment 2					
segment 3					
		Total=	0.00		

				Ratio		SWS		Wind			Seismic							Check uplift				
		Length	Inside	h:w	2w/h	Wind	Seismic	$TA_{Roof-end}$	$TA_{Roof-int.}$	V_s	A_i	w_i	F	ρ	R	F_x	V_{final}	DL	h	h'	$Uplift_w$	$Uplift_s$
Gridline 4	panel 1	22.5		0.5 :1	1.00	0	94	0	0	0	1824	29800	1.000	1.000	2	3020	**2114**	150	11	11	-1688	-654
	panel 2							$TA_{Wall-end}$	$TA_{Wall-int}$	$V_{s\ min}$						$V_{additional}$ =						
	panel 3							0	0	**0**												
Gypsum Sheathing	panel 4								$V_{additional}$ =													
	panel 5																					
No anchor bolts	panel 6																					
	panel 7																					
	panel 8																					
	panel 9																					
	panel 10																					
	$ASW_{1,2}$=	22.5	**22.5**	Total=	22.50																	

SW5	SW6			SW5	SW6		
90 plf	155 plf			90 plf	155 plf		
Total Resistance$_{wind}$				**Total Resistance**$_{seismic}$			
2025	3488			2025	3488		
Use SW5				Use SW6			

Use SW6
No Uplift

Perforated Shearwall 1:		**NOT USED**		$t = v =$	0
Total Length =		V_{wind}	V_{seis}	$Uplift_w$	$Uplift_s$
Height =		0	0	0	0
Max opening height=		**Ratio**			
C_o =		h:w	2w/h		
segment 1					
segment 2					
segment 3					
segment 4					
segment 5					
		Total=	0.00		

Perforated Shearwall 2:		**NOT USED**		$t = v =$	0
Total Length =		V_{wind}	V_{seis}	$Uplift_w$	$Uplift_s$
Height =		0	0	0	0
Max opening height=		**Ratio**			
C_o =		h:w	2w/h		
segment 1					
segment 2					
segment 3					
		Total=	0.00		

Perforated Shearwall 3:		**NOT USED**		$t = v =$	0
Total Length =		V_{wind}	V_{seis}	$Uplift_w$	$Uplift_s$
Height =		0	0	0	0
Max opening height=		**Ratio**			
C_o =		h:w	2w/h		
segment 1					
segment 2					
segment 3					
		Total=	0.00		

SIMPSON STRONG-TIE COMPANY INC.
(800) 999-5099
5956 W. Las Positas Blvd., Pleasanton, CA 94588.
www.strongtie.com

Job Name: 2025-2039
Wall Name: Gridline #3 - Front 2-Car Garage
Application: Garage Front

Design Criteria:

* 2018 International Bldg Code
* Wind
* 2500 psi concrete
* ASD Design Shear = 1087 lbs
* Shearwall Height = 10' to underside of top plates

Selected Strong-Wall® Panel Solution:

Model	Type	W (in)	H (in)	T (in)	Sill Anchor	End Anchor Bolts	Total Axial Load (lbs)	Actual Uplift (lbs)
WSWH12x10	Wood	12	117.25	3.5	N/A	2 - 1"	887	7914 lb
WSWH12x10	Wood	12	117.25	3.5	N/A	2 - 1"	887	7914 lb

Actual Shear & Drift Distribution:

Model	RR Relative Rigidity	Actual Shear (lbs)		Allowable Shear (lbs)	Actual / Allow Shear	Actual Drift (in)	Drift Limit (in)
WSWH12x10	0.50	543	≤	900 OK	0.60	0.40	0.67
WSWH12x10	0.50	543	≤	900 OK	0.60	0.40	0.67

Notes:

1. Strong-Wall High-Strength Wood Shearwalls have been evaluated to the 2021 IBC/IRC. See www.strongtie.com for additional design and installation information.
2. Anchor templates are recommended for proper anchor bolt placement, and are required in some jurisdictions.
3. Check that wall height "H" plus curb height (above slab) will attain overall rough header opening height (top of driveway slab to bottom of header).
4. WSWH Portal Connection Kit WSWH-PK is included with panels less than 100 inches in height and must be ordered separately for panels over 100 inches tall.
5. The applied vertical load shall be a concentric point load or a uniformly distributed load not exceeding the allowable vertical load. Alternatively, the load may be applied anywhere along the width of the panel if imposed by a continuous bearing vertical load transfer element such as a rimboard or beam. For eccentric axial loads applied directly to the panel, the allowable vertical load shall be divided by two.
6. Panels may be trimmed to a minimum height of 74½".
7. 2-ply headers may be used with Strong-Wall High-Strength Wood Shearwall panels. Minimum 11¼ inch deep nominal header is required with header design by others.

Disclaimer:

SIMPSON Strong-Tie

Anchor Designer™ for Concrete Software
Version 3.3.2410.2

1.Project information

Project description: Amdor Residence
Location: WSWH12x10 Anchors (GL #3)
Design name: Design

Comment:

2. Input Data & Anchor Parameters

General
Design method:ACI 318-19
Units: Imperial units

Anchor Information:
Anchor type: Cast-in-place
Material: AB
Diameter (inch): 1.000
Effective Embedment depth, h_{ef} (inch): 8.000
Anchor category: -
Anchor ductility: Yes
h_{min} (inch): 10.63
C_{min} (inch): 6.00
S_{min} (inch): 6.00

Base Material
Concrete: Normal-weight
Concrete thickness, h (inch): 13.78
State: Cracked
Compressive strength, f'_c (psi): 2500
$\Psi_{c,V}$: 1.0
Reinforcement condition: B tension, B shear
Supplemental edge reinforcement: Not applicable
Reinforcement provided at corners: No
Ignore concrete breakout in tension: No
Ignore concrete breakout in shear: No
Ignore 6do requirement: No
Build-up grout pad: No

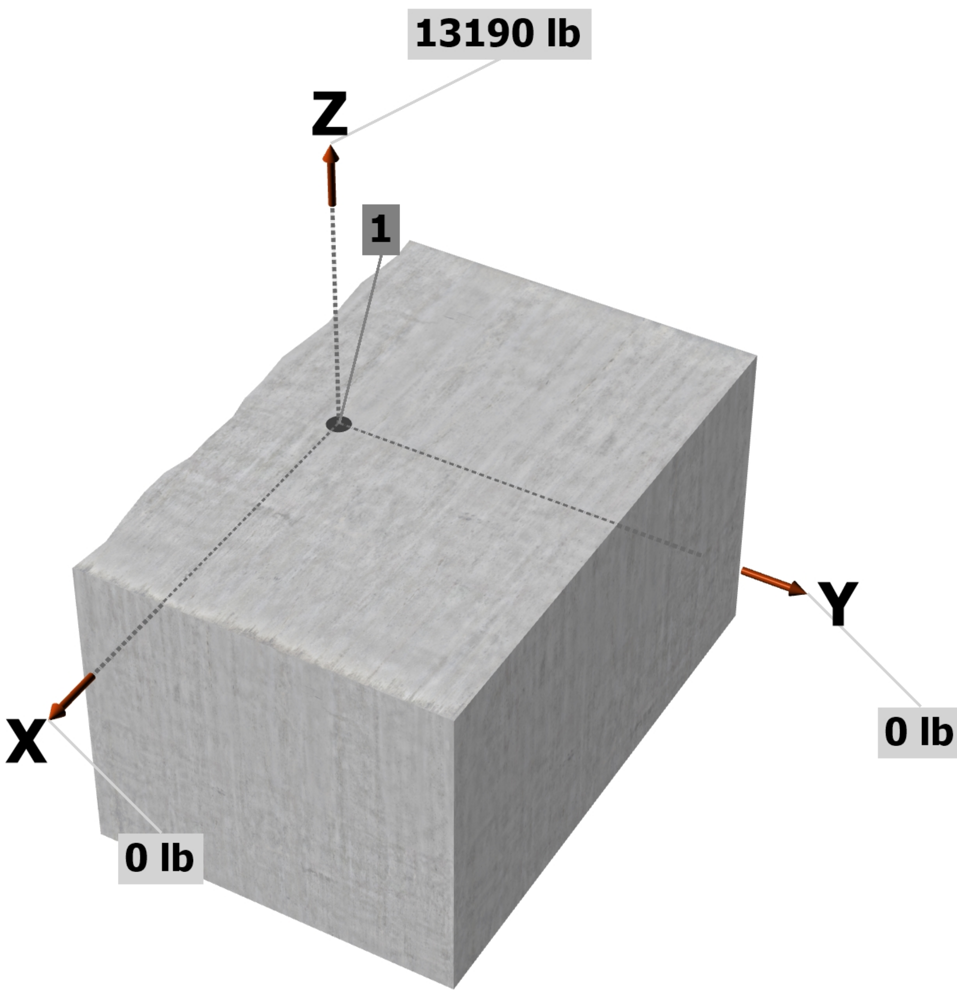
Recommended Anchor
Anchor Name: PAB Pre-Assembled Anchor Bolt - PAB8 (1"Ø)

Load and Geometry
Load factor source: ACI 318 Section 5.3
Load combination: not set
Seismic design: No
Anchors subjected to sustained tension: Not applicable
Apply entire shear load at front row: No
Anchors only resisting wind and/or seismic loads: Yes

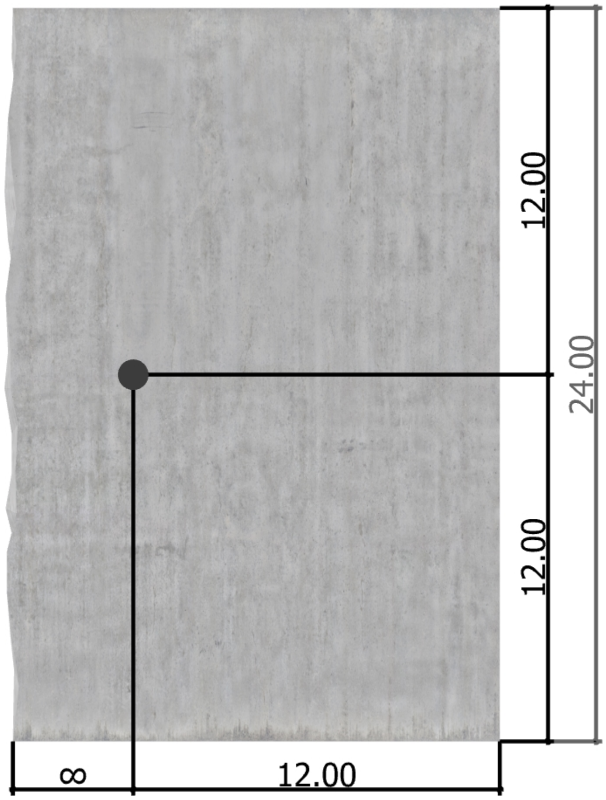
Strength level loads:

N_{ua} [lb]:	13190
V_{uax} [lb]:	0
V_{uay} [lb]:	0

<Figure 1>



<Figure 2>



3. Resulting Anchor Forces

Anchor	Tension load, N_{ua} (lb)	Shear load x, V_{uax} (lb)	Shear load y, V_{uay} (lb)	Shear load combined, $\sqrt{(V_{uax})^2+(V_{uay})^2}$ (lb)
1	13190.0	0.0	0.0	0.0
Sum	13190.0	0.0	0.0	0.0

Maximum concrete compression strain (‰): 0.00
Maximum concrete compression stress (psi): 0
Resultant tension force (lb): 13190
Resultant compression force (lb): 0
Eccentricity of resultant tension forces in x-axis, e'_{Nx} (inch): 0.00
Eccentricity of resultant tension forces in y-axis, e'_{Ny} (inch): 0.00

4. Steel Strength of Anchor in Tension (Sec. 17.6.1)

N_{sa} (lb)	ϕ	ϕN_{sa} (lb)
35150	0.75	26363

5. Concrete Breakout Strength of Anchor in Tension (Sec. 17.6.2)

$N_b = k_c \lambda_a \sqrt{f'_c} h_{ef}^{1.5}$ (Eq. 17.6.2.2.1)

k_c	λ_a	f'_c (psi)	h_{ef} (in)	N_b (lb)
24.0	1.00	2500	8.000	27153

$\phi N_{cb} = \phi\, (A_{Nc} / A_{Nco})\, \Psi_{ed,N} \Psi_{c,N} \Psi_{cp,N} N_b$ (Sec. 17.5.1.2 & Eq. 17.6.2.1a)

A_{Nc} (in^2)	A_{Nco} (in^2)	$c_{a,min}$ (in)	$\Psi_{ed,N}$	$\Psi_{c,N}$	$\Psi_{cp,N}$	N_b (lb)	ϕ	ϕN_{cb} (lb)
609.00	576.00	12.00	1.000	1.00	1.000	27153	0.70	20096

6. Pullout Strength of Anchor in Tension (Sec. 17.6.3)

$\phi N_{pn} = \phi \Psi_{c,P} N_p = \phi \Psi_{c,P} 8 A_{brg} f'_c$ (Sec. 17.5.1.2, Eq. 17.6.3.1 & 17.6.3.2.2a)

$\Psi_{c,P}$	A_{brg} (in^2)	f'_c (psi)	ϕ	ϕN_{pn} (lb)
1.0	5.15	2500	0.70	72156

11. Results

Interaction of Tensile and Shear Forces (Sec. 17.8)

Tension	Factored Load, N_{ua} (lb)	Design Strength, $øN_n$ (lb)	Ratio	Status
Steel	13190	26363	0.50	Pass
Concrete breakout	**13190**	**20096**	**0.66**	**Pass (Governs)**
Pullout	13190	72156	0.18	Pass

PAB8 (1"Ø) with hef = 8.000 inch meets the selected design criteria.

12. Warnings

- Designer must exercise own judgement to determine if this design is suitable.

Shear Walls

Gridline 5

		Length	Inside	Ratio h:w	Ratio 2w/h	SWS Wind	SWS Seismic	Wind $TA_{Roof\text{-}end}$	Wind $TA_{Roof\text{-}int.}$	Wind V_s	Seismic A_i	Seismic w_i	Seismic F	Seismic ρ	Seismic R	Seismic F_x	Seismic V_{final}	Check uplift DL	Check uplift h	Check uplift h'	Check uplift $Uplift_w$	Check uplift $Uplift_s$
Gridline 5	panel 1	6		1.5 :1	1.00	123	45	0	0	2437	1800	26750	1.000	1.000	6.5	834	**584**	100	9	9	805	104
	panel 2	7		1.3 :1	1.00	123	45	$TA_{Wall\text{-}end}$	$TA_{Wall\text{-}int}$	$V_{s\ min}$						$V_{additional}$ =		200	9	9	405	-296
	panel 3							90	124	**1597**												
Structural Sheathing	panel 4								$V_{additional}$ =	-840	<-- To FTAO											
	panel 5																					
1/2" dia. anchor bolts	panel 6																					
	panel 7																					
	panel 8																					
	panel 9																					
	panel 10																					
	$ASW_{1,2}$=	13	**13**	Total=	13.00																	

SW1	SW2	SW3	SW4	SW1	SW2	SW3	SW4
339 plf	495 plf	637 plf	832 plf	241 plf	350 plf	455 plf	595 plf
Total Resistance$_{wind}$				**Total Resistance**$_{seismic}$			
4407	6435	8281	10816	3133	4550	5915	7735
Use SW1				Use SW1			

Use SW1
Use LSTHD8 holdowns each side of panel as noted on plans
Use 1/2" dia. anchor bolts @ 32" o.c.

Perforated Shearwall 1:		NOT USED		*t* = *v* =	0
Total Length =		V_{wind}	V_{seis}	$Uplift_w$	$Uplift_s$
Height =		0	0	0	0
Max opening height=		Ratio			
C_o =		h:w	2w/h		
segment 1					
segment 2					
segment 3					
segment 4					
segment 5					
		Total=	0.00		

Perforated Shearwall 2:		NOT USED		*t* = *v* =	0
Total Length =		V_{wind}	V_{seis}	$Uplift_w$	$Uplift_s$
Height =		0	0	0	0
Max opening height=		Ratio			
C_o =		h:w	2w/h		
segment 1					
segment 2					
segment 3					
		Total=	0.00		

Perforated Shearwall 3:		NOT USED		*t* = *v* =	0
Total Length =		V_{wind}	V_{seis}	$Uplift_w$	$Uplift_s$
Height =		0	0	0	0
Max opening height=		Ratio			
C_o =		h:w	2w/h		
segment 1					
segment 2					
segment 3					
		Total=	0.00		

Gridline 6

		Length	Inside	Ratio h:w	Ratio 2w/h	SWS Wind	SWS Seismic	Wind $TA_{Roof\text{-}end}$	Wind $TA_{Roof\text{-}int.}$	Wind V_s	Seismic A_i	Seismic w_i	Seismic F	Seismic ρ	Seismic R	Seismic F_x	Seismic V_{final}	Check uplift DL	Check uplift h	Check uplift h'	Check uplift $Uplift_w$	Check uplift $Uplift_s$
Gridline 6	panel 1	14.42		1.0 :1	1.00	196	67	0	0	2821	2925	44262.5	1.000	1.000	6.5	1380	**966**	180	14	14	1441	-360
	panel 2							$TA_{Wall\text{-}end}$	$TA_{Wall\text{-}int}$	$V_{s\ min}$						$V_{additional}$ =						
	panel 3							40	240	**2821**												
Structural Sheathing	panel 4								$V_{additional}$ =													
	panel 5																					
1/2" dia. anchor bolts	panel 6																					
	panel 7																					
	panel 8																					
	panel 9																					
	panel 10																					
	$ASW_{1,2}$=	14.42	**14.42**	Total=	14.42																	

SW1	SW2	SW3	SW4	SW1	SW2	SW3	SW4
339 plf	495 plf	637 plf	832 plf	241 plf	350 plf	455 plf	595 plf
Total Resistance$_{wind}$				**Total Resistance**$_{seismic}$			
4888	7138	9186	11997	3475	5047	6561	8580
Use SW1				Use SW1			

Use SW1
Use HDU2-SDS2.5 holdowns each side of panel as noted on plans
Use 1/2" dia. anchor bolts @ 32" o.c.

Perforated Shearwall 1:		NOT USED		*t* = *v* =	0
Total Length =		V_{wind}	V_{seis}	$Uplift_w$	$Uplift_s$
Height =		0	0	0	0
Max opening height=		Ratio			
C_o =		h:w	2w/h		
segment 1					
segment 2					
segment 3					
segment 4					
segment 5					
		Total=	0.00		

Perforated Shearwall 2:		NOT USED		*t* = *v* =	0
Total Length =		V_{wind}	V_{seis}	$Uplift_w$	$Uplift_s$
Height =		0	0	0	0
Max opening height=		Ratio			
C_o =		h:w	2w/h		
segment 1					
segment 2					
segment 3					
		Total=	0.00		

Perforated Shearwall 3:		NOT USED		*t* = *v* =	0
Total Length =		V_{wind}	V_{seis}	$Uplift_w$	$Uplift_s$
Height =		0	0	0	0
Max opening height=		Ratio			
C_o =		h:w	2w/h		
segment 1					
segment 2					
segment 3					
		Total=	0.00		

Force-Transfer Around Openings

Gridline #5

V_{seis} (lb) =	**0**
V_{wind} (lb) =	**840**
	Wall #1
V_{seis} (lb) =	0
V_{wind} (lb) =	840
Shear Wall Required =	SW1
SWS_{seis} (plf) =	0
SWS_{wind} (plf) =	162
L (ft) =	13
h (ft) =	10
L1 (ft) =	3
L2 (ft) =	2.5
L3 (ft) =	0
h_a (ft) =	2
h_o (ft) =	6
h_b (ft) =	2
L_o1 (ft) =	7.5
L_o2 (ft) =	0
h:w1 =	2.0 :1
h:w2 =	2.4 :1
h:w3 =	0.0 :1
2w/h1 =	1.00
2w/h2 =	0.95
2w/h3 =	1.00
DL (lb) =	100
H_{seis} =	0
H_{wind} =	646
$Uplift_{seis}$ =	-650
$Uplift_{wind}$ =	-4
Holdown Required =	Not Req'd
Corner $Force_{seis}$ =	0.0
Corner $Force_{wind}$ =	660.8
Horizontal Strap Required =	CS16
Number of Duplicate Walls =	1
Total V_{seis} (lb) =	0
Total V_{wind} (lb) =	840

Use SW1	Holdowns:	Horizontal Straps:
Use 1/2" anchor bolts @ 32" o.c.	Not Req'd	CS16

1.Project information

Project description: Amdor Residence
Location: Retrfit HDU2 Anchors
Design name: Design

Comment:

2. Input Data & Anchor Parameters

General
Design method:ACI 318-19
Units: Imperial units

Anchor Information:
Anchor type: Bonded anchor
Material: F1554 Grade 36
Diameter (inch): 0.625
Effective Embedment depth, h_{ef} (inch): 6.000
Code report: ESR-5334
Anchor category: -
Anchor ductility: Yes
h_{min} (inch): 9.75
c_{ac} (inch): 9.44
C_{min} (inch): 1.75
S_{min} (inch): 3.00

Base Material
Concrete: Normal-weight
Concrete thickness, h (inch): 13.78
State: Cracked
Compressive strength, f'_c (psi): 2500
$\Psi_{c,V}$: 1.0
Reinforcement condition: B tension, B shear
Supplemental edge reinforcement: Not applicable
Reinforcement provided at corners: No
Ignore concrete breakout in tension: No
Ignore concrete breakout in shear: No
Hole condition: Dry concrete
Inspection: Continuous
Temperature range, Short/Long: 150/110°F
Reduced installation torque (for AT-3G): Not applicable
Ignore 6do requirement: Not applicable
Build-up grout pad: No

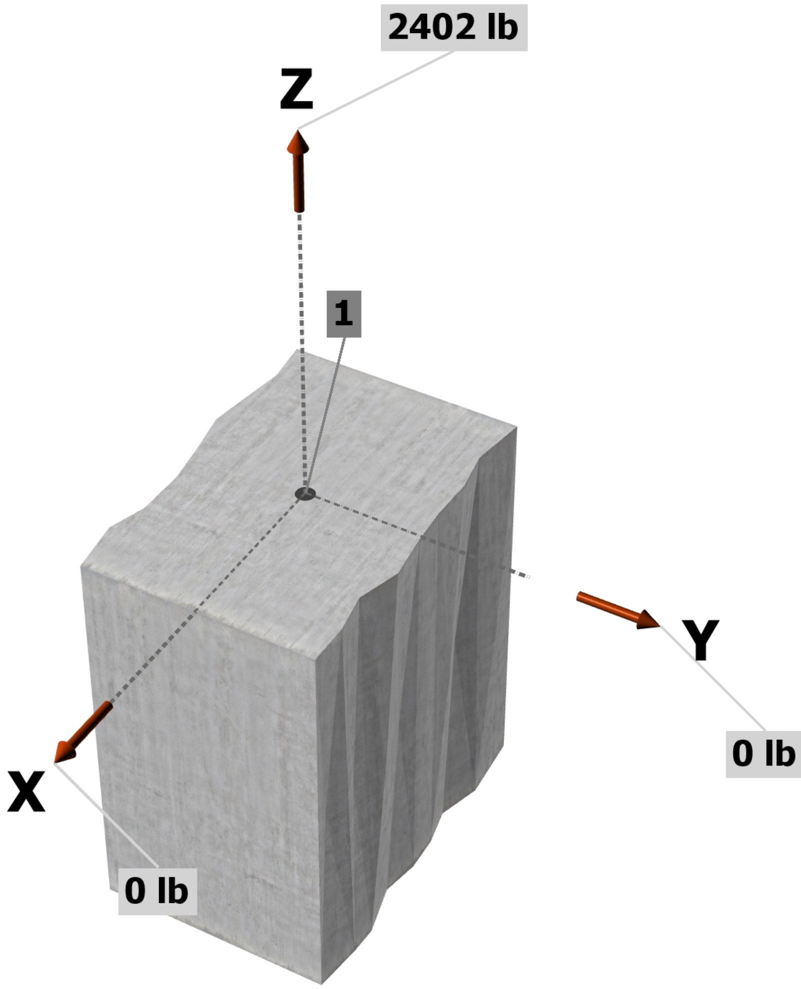
Recommended Anchor
Anchor Name: ET-3G™ - ET-3G w/ 5/8"Ø F1554 Gr. 36
Code Report: ESR-5334

Load and Geometry
Load factor source: ACI 318 Section 5.3
Load combination: not set
Seismic design: No
Anchors subjected to sustained tension: No
Apply entire shear load at front row: Yes
Anchors only resisting wind and/or seismic loads: Yes

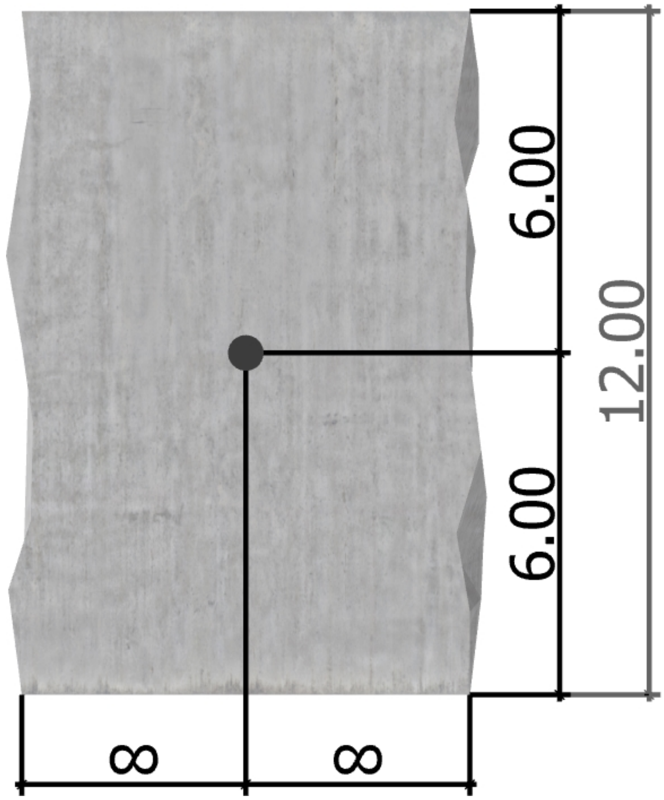
Strength level loads:

N_{ua} [lb]:	2402
V_{uax} [lb]:	0
V_{uay} [lb]:	0

<Figure 1>



<Figure 2>



3. Resulting Anchor Forces

Anchor	Tension load, N_{ua} (lb)	Shear load x, V_{uax} (lb)	Shear load y, V_{uay} (lb)	Shear load combined, $\sqrt{(V_{uax})^2+(V_{uay})^2}$ (lb)
1	2402.0	0.0	0.0	0.0
Sum	2402.0	0.0	0.0	0.0

Maximum concrete compression strain (‰): 0.00
Maximum concrete compression stress (psi): 0
Resultant tension force (lb): 2402
Resultant compression force (lb): 0
Eccentricity of resultant tension forces in x-axis, e'_{Nx} (inch): 0.00
Eccentricity of resultant tension forces in y-axis, e'_{Ny} (inch): 0.00

4. Steel Strength of Anchor in Tension (Sec. 17.6.1)

N_{sa} (lb)	ϕ	ϕN_{sa} (lb)
13110	0.75	9833

5. Concrete Breakout Strength of Anchor in Tension (Sec. 17.6.2)

$N_b = k_c \lambda_a \sqrt{f'_c} h_{ef}^{1.5}$ (Eq. 17.6.2.2.1)

k_c	λ_a	f'_c (psi)	h_{ef} (in)	N_b (lb)
17.0	1.00	2500	6.000	12492

$\phi N_{cb} = \phi\,(A_{Nc} / A_{Nco})\,\Psi_{ed,N} \Psi_{c,N} \Psi_{cp,N} N_b$ (Sec. 17.5.1.2 & Eq. 17.6.2.1a)

A_{Nc} (in^2)	A_{Nco} (in^2)	$c_{a,min}$ (in)	$\Psi_{ed,N}$	$\Psi_{c,N}$	$\Psi_{cp,N}$	N_b (lb)	ϕ	ϕN_{cb} (lb)
216.00	324.00	6.00	0.900	1.00	1.000	12492	0.65	4872

6. Adhesive Strength of Anchor in Tension (Sec. 17.6.5)

$\tau_{k,cr} = \tau_{k,cr} f_{short\text{-}term} K_{sat}$

$\tau_{k,cr}$ (psi)	$f_{short\text{-}term}$	K_{sat}	$\tau_{k,cr}$ (psi)
430	1.72	1.00	740

$N_{ba} = \lambda_a \tau_{cr} \pi d_a h_{ef}$ (Eq. 17.6.5.2.1)

λ_a	τ_{cr} (psi)	d_a (in)	h_{ef} (in)	N_{ba} (lb)
1.00	740	0.63	6.000	8713

$\phi N_a = \phi\,(A_{Na} / A_{Na0})\,\Psi_{ed,Na} \Psi_{cp,Na} N_{ba}$ (Sec. 17.5.1.2 & Eq. 17.6.5.1a)

A_{Na} (in^2)	A_{Na0} (in^2)	c_{Na} (in)	$c_{a,min}$ (in)	$\Psi_{ed,Na}$	$\Psi_{cp,Na}$	N_{ba} (lb)	ϕ	ϕN_a (lb)
192.20	256.53	8.01	6.00	0.925	1.000	8713	0.65	3924

11. Results

Interaction of Tensile and Shear Forces (Sec. 17.8)

Tension	Factored Load, N_{ua} (lb)	Design Strength, ϕN_n (lb)	Ratio	Status
Steel	2402	9833	0.24	Pass
Concrete breakout	2402	4872	0.49	Pass
Adhesive	**2402**	**3924**	**0.61**	**Pass (Governs)**

ET-3G w/ 5/8"Ø F1554 Gr. 36 with hef = 6.000 inch meets the selected design criteria.

12. Warnings

- When cracked concrete is selected, concrete compressive strength used in concrete breakout strength in tension, adhesive strength in tension and concrete pryout strength in shear for ET-3G adhesive anchor is limited to 2,500 psi per ICC-ES ESR-5334 Section 5.3.

- Designer must exercise own judgement to determine if this design is suitable.

- Refer to manufacturer's product literature for hole cleaning and installation instructions.

Shear Walls

Gridline 7

Simpson Strong Wall

No anchor bolts

	Length	Inside	Ratio h:w	Ratio 2w/h	SWS Wind	SWS Seismic
panel 1						
panel 2						
panel 3						
panel 4						
panel 5						
panel 6						
panel 7						
panel 8						
panel 9						
panel 10						
$ASW_{1,2}$=	0	**0**	Total=	0.00		

Wind

$TA_{Roof\text{-}end}$	$TA_{Roof\text{-}int.}$	V_s
0	0	1340
$TA_{Wall\text{-}end}$	$TA_{Wall\text{-}int}$	$V_{s\ min}$
37	87	**1340**
	$V_{additional}$ =	

Seismic

A_i	w_i	F	ρ	R	F_x	V_{final}
1500	24350	1.000	1.000	6.5	759	**531**
					$V_{additional}$ =	

SW1	SW2	SW3	SW4	SW1	SW2	SW3	SW4
339 plf	495 plf	637 plf	832 plf	241 plf	350 plf	455 plf	595 plf
Total Resistance$_{wind}$				**Total Resistance**$_{seismic}$			
0	0	0	0	0	0	0	0

Check uplift

DL	h	h'	Uplift $_w$	Uplift $_s$

See the attached Simpson Strong Wall calculations

Perforated Shearwall 1: **NOT USED**

		V_{wind}	V_{seis}	Uplift $_w$	Uplift $_s$
				t = v =	0
Total Length =					
Height =		0	0	0	0
Max opening height=		**Ratio**			
C_o =		h:w	2w/h		
segment 1					
segment 2					
segment 3					
segment 4					
segment 5					
		Total=	0.00		

Perforated Shearwall 2: **NOT USED**

		V_{wind}	V_{seis}	Uplift $_w$	Uplift $_s$
				t = v =	0
Total Length =					
Height =		0	0	0	0
Max opening height=		**Ratio**			
C_o =		h:w	2w/h		
segment 1					
segment 2					
segment 3					
		Total=	0.00		

Perforated Shearwall 3: **NOT USED**

		V_{wind}	V_{seis}	Uplift $_w$	Uplift $_s$
				t = v =	0
Total Length =					
Height =		0	0	0	0
Max opening height=		**Ratio**			
C_o =		h:w	2w/h		
segment 1					
segment 2					
segment 3					
		Total=	0.00		

Gridline 8

Structural Sheathing

1/2" dia. anchor bolts

	Length	Inside	Ratio h:w	Ratio 2w/h	SWS Wind	SWS Seismic
panel 1	4.75		1.9 :1	1.00	282	67
panel 2						
panel 3						
panel 4						
panel 5						
panel 6						
panel 7						
panel 8						
panel 9						
panel 10						
$ASW_{1,2}$=	4.75	**4.75**	Total=	4.75		

Wind

$TA_{Roof\text{-}end}$	$TA_{Roof\text{-}int.}$	V_s
0	0	1341
$TA_{Wall\text{-}end}$	$TA_{Wall\text{-}int}$	$V_{s\ min}$
95	0	**1341**
	$V_{additional}$ =	

Seismic

A_i	w_i	F	ρ	R	F_x	V_{final}
900	14500	1.000	1.000	6.5	452	**316**
					$V_{additional}$ =	

SW1	SW2	SW3	SW4	SW1	SW2	SW3	SW4
339 plf	495 plf	637 plf	832 plf	241 plf	350 plf	455 plf	595 plf
Total Resistance$_{wind}$				**Total Resistance**$_{seismic}$			
1610	2351	3026	3952	1145	1663	2161	2826
Use SW1				Use SW1			

Check uplift

DL	h	h'	Uplift $_w$	Uplift $_s$
100	9	9	2304	362

Use SW1
Use STHD10 holdowns each side of panel as noted on plans
Use 1/2" dia. anchor bolts @ 32" o.c.

Perforated Shearwall 1: **NOT USED**

		V_{wind}	V_{seis}	Uplift $_w$	Uplift $_s$
				t = v =	0
Total Length =					
Height =		0	0	0	0
Max opening height=		**Ratio**			
C_o =		h:w	2w/h		
segment 1					
segment 2					
segment 3					
segment 4					
segment 5					
		Total=	0.00		

Perforated Shearwall 2: **NOT USED**

		V_{wind}	V_{seis}	Uplift $_w$	Uplift $_s$
				t = v =	0
Total Length =					
Height =		0	0	0	0
Max opening height=		**Ratio**			
C_o =		h:w	2w/h		
segment 1					
segment 2					
segment 3					
		Total=	0.00		

Perforated Shearwall 3: **NOT USED**

		V_{wind}	V_{seis}	Uplift $_w$	Uplift $_s$
				t = v =	0
Total Length =					
Height =		0	0	0	0
Max opening height=		**Ratio**			
C_o =		h:w	2w/h		
segment 1					
segment 2					
segment 3					
		Total=	0.00		

SIMPSON STRONG-TIE COMPANY INC.
(800) 999-5099
5956 W. Las Positas Blvd., Pleasanton, CA 94588.
www.strongtie.com

Job Name: 2025-2039
Wall Name: Gridline #7 - Rear Great Room
Application: Garage Front

Design Criteria:

* 2018 International Bldg Code
* Wind
* 2500 psi concrete
* ASD Design Shear = 1340 lbs
* Shearwall Height = 13' to underside of top plates

Selected Strong-Wall® Panel Solution:

Model	Type	W (in)	H (in)	T (in)	Sill Anchor	End Anchor Bolts	Total Axial Load (lbs)	Actual Uplift (lbs)
WSWH18x13	Wood	18	156	3.5	N/A	2 - 1"	0	8362 lb
WSWH18x13	Wood	18	156	3.5	N/A	2 - 1"	0	8362 lb

Actual Shear & Drift Distribution:

Model	RR Relative Rigidity	Actual Shear (lbs)		Allowable Shear (lbs)	Actual / Allow Shear	Actual Drift (in)	Drift Limit (in)
WSWH18x13	0.50	670	≤	1910 OK	0.35	0.31	0.87
WSWH18x13	0.50	670	≤	1910 OK	0.35	0.31	0.87

Notes:

1. Strong-Wall High-Strength Wood Shearwalls have been evaluated to the 2021 IBC/IRC. See www.strongtie.com for additional design and installation information.

2. Anchor templates are recommended for proper anchor bolt placement, and are required in some jurisdictions.

3. Check that wall height "H" plus curb height (above slab) will attain overall rough header opening height (top of driveway slab to bottom of header).

4. WSWH Portal Connection Kit WSWH-PK is included with panels less than 100 inches in height and must be ordered separately for panels over 100 inches tall.

5. The applied vertical load shall be a concentric point load or a uniformly distributed load not exceeding the allowable vertical load. Alternatively, the load may be applied anywhere along the width of the panel if imposed by a continuous bearing vertical load transfer element such as a rimboard or beam. For eccentric axial loads applied directly to the panel, the allowable vertical load shall be divided by two.

6. Panels may be trimmed to a minimum height of 74½".

7. 2-ply headers may be used with Strong-Wall High-Strength Wood Shearwall panels. Minimum 11¼ inch deep nominal header is required with header design by others.

Disclaimer:

It is the Designer's responsibility to verify product suitability under applicable building codes. In order to verify code listed applications please refer to the appropriate product code reports at www.strongtie.com or contact Simpson Strong-Tie Company Inc. at 1-800-999-5099.

SIMPSON Strong-Tie

Anchor Designer™ for
Concrete Software
Version 3.3.2410.2

Company:		Date:	12/9/2024
Engineer:		Page:	1
Project:	2025-2039		
Address:			
Phone:			
E-mail:			

1.Project information

Project description: Amdor Residence
Location: WSWH18x14 Anchors (GL #7)
Design name: Design

Comment:

2. Input Data & Anchor Parameters

General
Design method:ACI 318-19
Units: Imperial units

Anchor Information:
Anchor type: Cast-in-place
Material: AB
Diameter (inch): 1.000
Effective Embedment depth, h_{ef} (inch): 8.000
Anchor category: -
Anchor ductility: Yes
h_{min} (inch): 10.63
C_{min} (inch): 6.00
S_{min} (inch): 6.00

Base Material
Concrete: Normal-weight
Concrete thickness, h (inch): 13.78
State: Cracked
Compressive strength, f'_c (psi): 2500
$\Psi_{c,V}$: 1.0
Reinforcement condition: B tension, B shear
Supplemental edge reinforcement: Not applicable
Reinforcement provided at corners: No
Ignore concrete breakout in tension: No
Ignore concrete breakout in shear: No
Ignore 6do requirement: No
Build-up grout pad: No

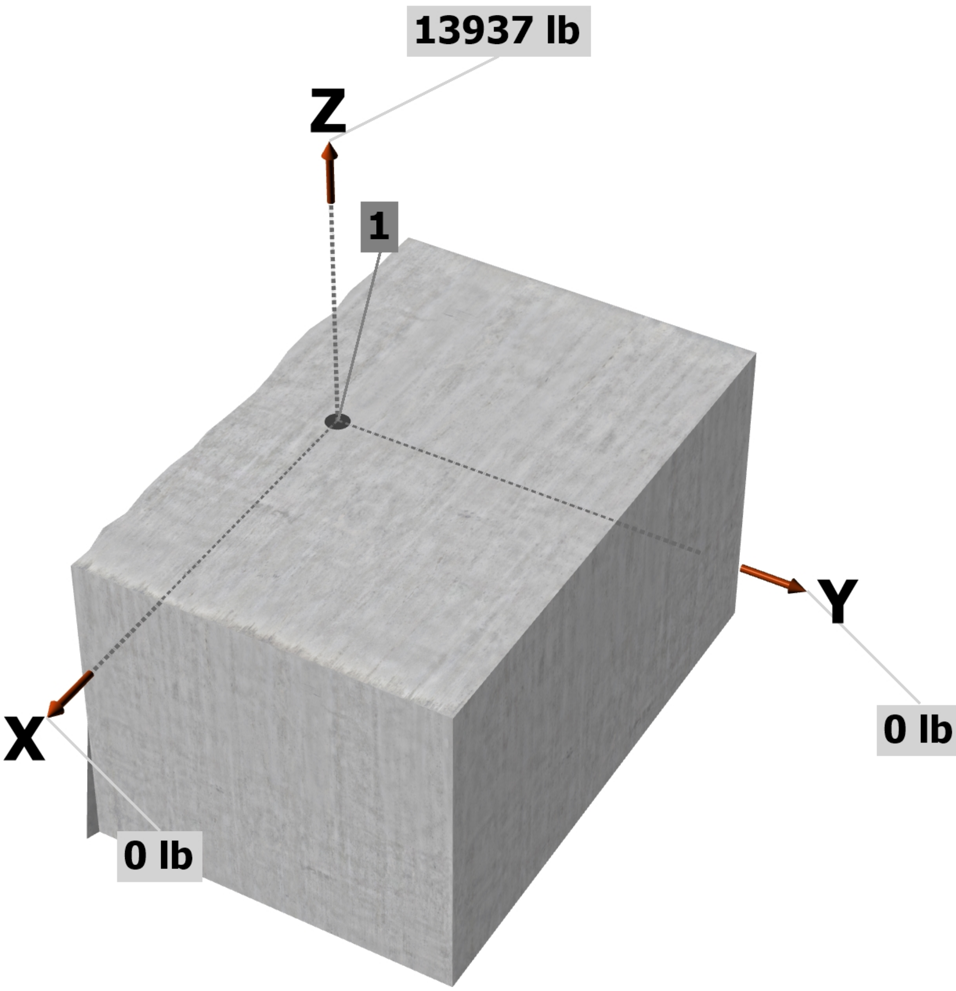
Recommended Anchor
Anchor Name: PAB Pre-Assembled Anchor Bolt - PAB8 (1"Ø)

Load and Geometry
Load factor source: ACI 318 Section 5.3
Load combination: not set
Seismic design: No
Anchors subjected to sustained tension: Not applicable
Apply entire shear load at front row: No
Anchors only resisting wind and/or seismic loads: Yes

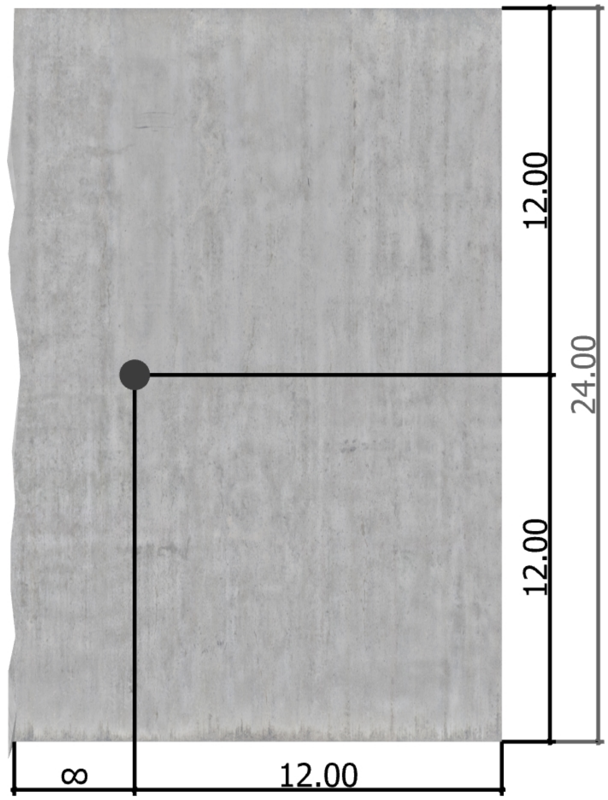
Strength level loads:

N_{ua} [lb]:	13937
V_{uax} [lb]:	0
V_{uay} [lb]:	0

<Figure 1>



<Figure 2>



3. Resulting Anchor Forces

Anchor	Tension load, N_{ua} (lb)	Shear load x, V_{uax} (lb)	Shear load y, V_{uay} (lb)	Shear load combined, $\sqrt{(V_{uax})^2+(V_{uay})^2}$ (lb)
1	13937.0	0.0	0.0	0.0
Sum	13937.0	0.0	0.0	0.0

Maximum concrete compression strain (‰): 0.00
Maximum concrete compression stress (psi): 0
Resultant tension force (lb): 13937
Resultant compression force (lb): 0
Eccentricity of resultant tension forces in x-axis, e'_{Nx} (inch): 0.00
Eccentricity of resultant tension forces in y-axis, e'_{Ny} (inch): 0.00

4. Steel Strength of Anchor in Tension (Sec. 17.6.1)

N_{sa} (lb)	ϕ	ϕN_{sa} (lb)
35150	0.75	26363

5. Concrete Breakout Strength of Anchor in Tension (Sec. 17.6.2)

$N_b = k_c \lambda_a \sqrt{f'_c} h_{ef}^{1.5}$ (Eq. 17.6.2.2.1)

k_c	λ_a	f'_c (psi)	h_{ef} (in)	N_b (lb)
24.0	1.00	2500	8.000	27153

$\phi N_{cb} = \phi\ (A_{Nc} / A_{Nco}) \Psi_{ed,N} \Psi_{c,N} \Psi_{cp,N} N_b$ (Sec. 17.5.1.2 & Eq. 17.6.2.1a)

A_{Nc} (in^2)	A_{Nco} (in^2)	$c_{a,min}$ (in)	$\Psi_{ed,N}$	$\Psi_{c,N}$	$\Psi_{cp,N}$	N_b (lb)	ϕ	ϕN_{cb} (lb)
609.00	576.00	12.00	1.000	1.00	1.000	27153	0.70	20096

6. Pullout Strength of Anchor in Tension (Sec. 17.6.3)

$\phi N_{pn} = \phi \Psi_{c,P} N_p = \phi \Psi_{c,P} 8 A_{brg} f'_c$ (Sec. 17.5.1.2, Eq. 17.6.3.1 & 17.6.3.2.2a)

$\Psi_{c,P}$	A_{brg} (in^2)	f'_c (psi)	ϕ	ϕN_{pn} (lb)
1.0	5.15	2500	0.70	72156

11. Results

Interaction of Tensile and Shear Forces (Sec. 17.8)

Tension	Factored Load, N_{ua} (lb)	Design Strength, ϕN_n (lb)	Ratio	Status
Steel	13937	26363	0.53	Pass
Concrete breakout	**13937**	**20096**	**0.69**	**Pass (Governs)**
Pullout	13937	72156	0.19	Pass

PAB8 (1"Ø) with hef = 8.000 inch meets the selected design criteria.

12. Warnings

- Designer must exercise own judgement to determine if this design is suitable.

Shear Walls

Gridline 9

		Length	Inside	h:w	2w/h	SWS Wind	SWS Seismic	$TA_{Roof\text{-}end}$	$TA_{Roof\text{-}int.}$	V_s	A_i	w_i	F	ρ	R	F_x	V_{final}	DL	h	h'	Uplift $_w$	Uplift $_s$
Gridline 9	panel 1	10.5		1.5 :1	1.00	91	37	0	0	1661	1820	30430	1.000	1.000	6.5	949	**664**	210	15.5	15.5	315	-536
	panel 2							$TA_{Wall\text{-}end}$	$TA_{Wall\text{-}int}$	$V_{s\ min}$						$V_{additional}$ =						
	panel 3							87	46	**1661**												
Structural Sheathing	panel 4								$V_{additional}$ =													
	panel 5																					
1/2" dia. anchor bolts	panel 6																					
	panel 7																					
	panel 8																					
	panel 9																					
	panel 10																					
	$ASW_{1,2}$=	10.5	**10.5**	Total=	10.50																	

SW1	SW2	SW3	SW4	SW1	SW2	SW3	SW4
339 plf	495 plf	637 plf	832 plf	241 plf	350 plf	455 plf	595 plf
Total Resistance$_{wind}$				**Total Resistance$_{seismic}$**			
6155	8987	11565	15106	4376	6355	8261	10803
Use SW1				Use SW1			

Use SW1
Use LSTHD8 holdowns each side of panel as noted on plans
Use 1/2" dia. anchor bolts @ 32" o.c.

Perforated Shearwall 1:		V_{wind}	V_{seis}	*t = v =*	72
Total Length =	15			Uplift $_w$	Uplift $_s$
Height =	15.5	700	280	1111	444
Max opening height=	11	**Ratio**			
C_o =	0.814	h:w	2w/h		
segment 1	6.667	2.3 :1	0.86		
segment 2	5.33	2.9 :1	0.69		
segment 3					
segment 4					
segment 5					
		Total=	7.66		

Perforated Shearwall 2:		**NOT USED**		*t = v =*	0
Total Length =		V_{wind}	V_{seis}	Uplift $_w$	Uplift $_s$
Height =		0	0	0	0
Max opening height=		**Ratio**			
C_o =		h:w	2w/h		
segment 1					
segment 2					
segment 3					
		Total=	0.00		

Perforated Shearwall 3:		**NOT USED**		*t = v =*	0
Total Length =		V_{wind}	V_{seis}	Uplift $_w$	Uplift $_s$
Height =		0	0	0	0
Max opening height=		**Ratio**			
C_o =		h:w	2w/h		
segment 1					
segment 2					
segment 3					
		Total=	0.00		

Gridline 10

		Length	Inside	h:w	2w/h	SWS Wind	SWS Seismic	$TA_{Roof\text{-}end}$	$TA_{Roof\text{-}int.}$	V_s	A_i	w_i	F	ρ	R	F_x	V_{final}	DL	h	h'	Uplift $_w$	Uplift $_s$
Gridline 10	panel 1	28		0.6 :1	1.00	64	31	0	0	2257	3290	50485	1.000	1.000	6.5	1574	**1102**	200	15.5	15.5	-1808	-2326
	panel 2	8		1.4 :1	1.00	64	31	$TA_{Wall\text{-}end}$	$TA_{Wall\text{-}int}$	$V_{s\ min}$						$V_{additional}$ =		200	11	11	-96	-463
	panel 3							0	240	**2304**												
Structural Sheathing	panel 4								$V_{additional}$ =													
	panel 5																					
1/2" dia. anchor bolts	panel 6																					
	panel 7																					
	panel 8																					
	panel 9																					
	panel 10																					
	$ASW_{1,2}$=	36	**36**	Total=	36.00																	

SW1	SW2	SW3	SW4	SW1	SW2	SW3	SW4
339 plf	495 plf	637 plf	832 plf	241 plf	350 plf	455 plf	595 plf
Total Resistance$_{wind}$				**Total Resistance$_{seismic}$**			
12204	17820	22932	29952	8676	12600	16380	21420
Use SW1				Use SW1			

Use SW1
No Uplift
Use 1/2" dia. anchor bolts @ 32" o.c.

Perforated Shearwall 1:		**NOT USED**		*t = v =*	0
Total Length =		V_{wind}	V_{seis}	Uplift $_w$	Uplift $_s$
Height =		0	0	0	0
Max opening height=		**Ratio**			
C_o =		h:w	2w/h		
segment 1					
segment 2					
segment 3					
segment 4					
segment 5					
		Total=	0.00		

Perforated Shearwall 2:		**NOT USED**		*t = v =*	0
Total Length =		V_{wind}	V_{seis}	Uplift $_w$	Uplift $_s$
Height =		0	0	0	0
Max opening height=		**Ratio**			
C_o =		h:w	2w/h		
segment 1					
segment 2					
segment 3					
		Total=	0.00		

Perforated Shearwall 3:		**NOT USED**		*t = v =*	0
Total Length =		V_{wind}	V_{seis}	Uplift $_w$	Uplift $_s$
Height =		0	0	0	0
Max opening height=		**Ratio**			
C_o =		h:w	2w/h		
segment 1					
segment 2					
segment 3					
		Total=	0.00		

Shear Walls

Gridline 11

		Length	Inside	Ratio h:w	Ratio 2w/h	SWS Wind	SWS Seismic	Wind $TA_{Roof\text{-}end}$	Wind $TA_{Roof\text{-}int.}$	Wind V_s	Seismic A_i	Seismic w_i	F	ρ	R	F_x	V_{final}	Check uplift DL	h	h'	Uplift $_w$	Uplift $_s$
Gridline 11	panel 1	20		0.6 :1	1.00	85	22	0	0	1693	1188	20370	1.000	1.000	6.5	635	**445**	190	11	11	-969	-1655
	panel 2																					
	panel 3																					
Structural Sheathing	panel 4																					
	panel 5																					
1/2" dia. anchor bolts	panel 6																					
	panel 7																					
	panel 8																					
	panel 9																					
	panel 10																					
	$ASW_{1,2}$=	20	**20**	Total=	20.00																	

$TA_{Wall\text{-}end}$	$TA_{Wall\text{-}int}$	$V_{s\ min}$
62	87	**1693**

$V_{additional}$ =

$V_{additional}$ =

SW1	SW2	SW3	SW4	SW1	SW2	SW3	SW4
339 plf	495 plf	637 plf	832 plf	241 plf	350 plf	455 plf	595 plf
Total Resistance$_{wind}$				**Total Resistance$_{seismic}$**			
6780	9900	12740	16640	4820	7000	9100	11900
Use SW1				Use SW1			

Use SW1
No Uplift
Use 1/2" dia. anchor bolts @ 32" o.c.

Perforated Shearwall 1:		**NOT USED**		*t* = *v* =	0
Total Length =		V_{wind}	V_{seis}	Uplift $_w$	Uplift $_s$
Height =		0	0	0	0
Max opening height=		**Ratio**			
C_o =		h:w	2w/h		
segment 1					
segment 2					
segment 3					
segment 4					
segment 5					
		Total=	0.00		

Perforated Shearwall 2:		**NOT USED**		*t* = *v* =	0
Total Length =		V_{wind}	V_{seis}	Uplift $_w$	Uplift $_s$
Height =		0	0	0	0
Max opening height=		**Ratio**			
C_o =		h:w	2w/h		
segment 1					
segment 2					
segment 3					
		Total=	0.00		

Perforated Shearwall 3:		**NOT USED**		*t* = *v* =	0
Total Length =		V_{wind}	V_{seis}	Uplift $_w$	Uplift $_s$
Height =		0	0	0	0
Max opening height=		**Ratio**			
C_o =		h:w	2w/h		
segment 1					
segment 2					
segment 3					
		Total=	0.00		

Gridline 12

		Length	Inside	Ratio h:w	Ratio 2w/h	SWS Wind	SWS Seismic	Wind $TA_{Roof\text{-}end}$	Wind $TA_{Roof\text{-}int.}$	Wind V_s	Seismic A_i	Seismic w_i	F	ρ	R	F_x	V_{final}	Check uplift DL	h	h'	Uplift $_w$	Uplift $_s$
Gridline 12	panel 1	16		0.6 :1	1.00	139	45	0	0	2229	2250	32875	1.000	1.000	6.5	1025	**718**	200	10	10	-207	-1152
	panel 2																					
	panel 3																					
Structural Sheathing	panel 4																					
	panel 5																					
1/2" dia. anchor bolts	panel 6																					
	panel 7																					
	panel 8																					
	panel 9																					
	panel 10																					
	$ASW_{1,2}$=	16	**16**	Total=	16.00																	

$TA_{Wall\text{-}end}$	$TA_{Wall\text{-}int}$	$V_{s\ min}$
54	156	**2229**

$V_{additional}$ =

$V_{additional}$ =

SW1	SW2	SW3	SW4	SW1	SW2	SW3	SW4
339 plf	495 plf	637 plf	832 plf	241 plf	350 plf	455 plf	595 plf
Total Resistance$_{wind}$				**Total Resistance$_{seismic}$**			
5424	7920	10192	13312	3856	5600	7280	9520
Use SW1				Use SW1			

Use SW1
No Uplift
Use 1/2" dia. anchor bolts @ 32" o.c.

Perforated Shearwall 1:		**NOT USED**		*t* = *v* =	0
Total Length =		V_{wind}	V_{seis}	Uplift $_w$	Uplift $_s$
Height =		0	0	0	0
Max opening height=		**Ratio**			
C_o =		h:w	2w/h		
segment 1					
segment 2					
segment 3					
segment 4					
segment 5					
		Total=	0.00		

Perforated Shearwall 2:		**NOT USED**		*t* = *v* =	0
Total Length =		V_{wind}	V_{seis}	Uplift $_w$	Uplift $_s$
Height =		0	0	0	0
Max opening height=		**Ratio**			
C_o =		h:w	2w/h		
segment 1					
segment 2					
segment 3					
		Total=	0.00		

Perforated Shearwall 3:		**NOT USED**		*t* = *v* =	0
Total Length =		V_{wind}	V_{seis}	Uplift $_w$	Uplift $_s$
Height =		0	0	0	0
Max opening height=		**Ratio**			
C_o =		h:w	2w/h		
segment 1					
segment 2					
segment 3					
		Total=	0.00		

Shear Walls

Gridline 13

Structural Sheathing

1/2" dia. anchor bolts

		Ratio		SWS		Wind			Seismic							Check uplift				
	Length	Inside	h:w	2w/h	Wind	Seismic	$TA_{Roof-end}$	$TA_{Roof-int.}$	V_s	A_i	w_i	F	ρ	R	F_x	V_{final}	DL	h	h'	$Uplift_w$	$Uplift_s$
panel 1	6		2.3 :1	0.86	109	38	0	0	3500	4000	56500	1.000	1.000	6.5	1762	**1233**	100	14	14	1228	238
panel 2	10.5		1.3 :1	1.00	127	45											100	14	14	1258	103
panel 3	6		1.5 :1	1.00	127	45											100	9	9	846	104
panel 4																					
panel 5																					
panel 6																					
panel 7																					
panel 8																					
panel 9																					
panel 10																					
$ASW_{1,2}$=	22.5	**22.5**	Total=	21.64																	

$TA_{Wall-end}$	$TA_{Wall-int}$	$V_{s\ min}$
128	180	**3500**

$V_{additional}$ =

$V_{additional}$ =

SW1	SW2	SW3	SW4	SW1	SW2	SW3	SW4
339 plf	495 plf	637 plf	832 plf	241 plf	350 plf	455 plf	595 plf
Total $Resistance_{wind}$				Total $Resistance_{seismic}$			
9317	13604	17507	22866	6623	9619	12505	16352
Use SW1				Use SW1			

Use SW1
Use LSTHD8 holdowns each side of panel as noted on plans
Use 1/2" dia. anchor bolts @ 32" o.c.

Perforated Shearwall 1:

				$t = v =$	127
Total Length =	13.5	V_{wind}	V_{seis}	$Uplift_w$	$Uplift_s$
Height =	9	744	262	1146	404
Max opening height=	8	Ratio			
C_o =	0.649	h:w	2w/h		
segment 1	4.5	2.0 :1	1.00		
segment 2	4.5	2.0 :1	1.00		
segment 3					
segment 4					
segment 5					
		Total=	5.84		

Perforated Shearwall 2: NOT USED

				$t = v =$	0
Total Length =		V_{wind}	V_{seis}	$Uplift_w$	$Uplift_s$
Height =		0	0	0	0
Max opening height=		Ratio			
C_o =		h:w	2w/h		
segment 1					
segment 2					
segment 3					
		Total=	0.00		

Perforated Shearwall 3: NOT USED

				$t = v =$	0
Total Length =		V_{wind}	V_{seis}	$Uplift_w$	$Uplift_s$
Height =		0	0	0	0
Max opening height=		Ratio			
C_o =		h:w	2w/h		
segment 1					
segment 2					
segment 3					
		Total=	0.00		

Gridline 14

Structural Sheathing

1/2" dia. anchor bolts

		Ratio		SWS		Wind			Seismic							Check uplift				
	Length	Inside	h:w	2w/h	Wind	Seismic	$TA_{Roof-end}$	$TA_{Roof-int.}$	V_s	A_i	w_i	F	ρ	R	F_x	V_{final}	DL	h	h'	$Uplift_w$	$Uplift_s$
panel 1	7.5		1.3 :1	1.00	325	50	0	0	2436	1107	17237.5	1.000	1.000	6.5	537	**376**	100	10	10	2873	127
panel 2																					
panel 3																					
panel 4																					
panel 5																					
panel 6																					
panel 7																					
panel 8																					
panel 9																					
panel 10																					
$ASW_{1,2}$=	7.5	**7.5**	Total=	7.50																	

$TA_{Wall-end}$	$TA_{Wall-int}$	$V_{s\ min}$
68	157	**2436**

$V_{additional}$ =

$V_{additional}$ =

SW1	SW2	SW3	SW4	SW1	SW2	SW3	SW4
339 plf	495 plf	637 plf	832 plf	241 plf	350 plf	455 plf	595 plf
Total $Resistance_{wind}$				Total $Resistance_{seismic}$			
2543	3713	4778	6240	1808	2625	3413	4463
Use SW1				Use SW1			

Use SW1
Use HDU5-SDS2.5 holdowns each side of panel as noted on plans
Use 1/2" dia. anchor bolts @ 32" o.c.

Perforated Shearwall 1: NOT USED

				$t = v =$	0
Total Length =		V_{wind}	V_{seis}	$Uplift_w$	$Uplift_s$
Height =		0	0	0	0
Max opening height=		Ratio			
C_o =		h:w	2w/h		
segment 1					
segment 2					
segment 3					
segment 4					
segment 5					
		Total=	0.00		

Perforated Shearwall 2: NOT USED

				$t = v =$	0
Total Length =		V_{wind}	V_{seis}	$Uplift_w$	$Uplift_s$
Height =		0	0	0	0
Max opening height=		Ratio			
C_o =		h:w	2w/h		
segment 1					
segment 2					
segment 3					
		Total=	0.00		

Perforated Shearwall 3: NOT USED

				$t = v =$	0
Total Length =		V_{wind}	V_{seis}	$Uplift_w$	$Uplift_s$
Height =		0	0	0	0
Max opening height=		Ratio			
C_o =		h:w	2w/h		
segment 1					
segment 2					
segment 3					
		Total=	0.00		

Company:		Date:	12/12/2024
Engineer:		Page:	1
Project:	2025-2039		
Address:			
Phone:			
E-mail:			

SIMPSON Strong-Tie

Anchor Designer™ for Concrete Software
Version 3.3.2410.2

1.Project information

Project description: Amdor Residence
Location: Retrfit HDU5 Anchors
Design name: Design

Comment:

2. Input Data & Anchor Parameters

General
Design method:ACI 318-19
Units: Imperial units

Anchor Information:
Anchor type: Bonded anchor
Material: F1554 Grade 36
Diameter (inch): 0.625
Effective Embedment depth, h_{ef} (inch): 8.000
Code report: ESR-5334
Anchor category: -
Anchor ductility: Yes
h_{min} (inch): 11.75
c_{ac} (inch): 15.95
C_{min} (inch): 1.75
S_{min} (inch): 3.00

Base Material
Concrete: Normal-weight
Concrete thickness, h (inch): 13.78
State: Cracked
Compressive strength, f'_c (psi): 2500
$\Psi_{c,V}$: 1.0
Reinforcement condition: B tension, B shear
Supplemental edge reinforcement: Not applicable
Reinforcement provided at corners: No
Ignore concrete breakout in tension: No
Ignore concrete breakout in shear: No
Hole condition: Dry concrete
Inspection: Continuous
Temperature range, Short/Long: 150/110°F
Reduced installation torque (for AT-3G): Not applicable
Ignore 6do requirement: Not applicable
Build-up grout pad: No

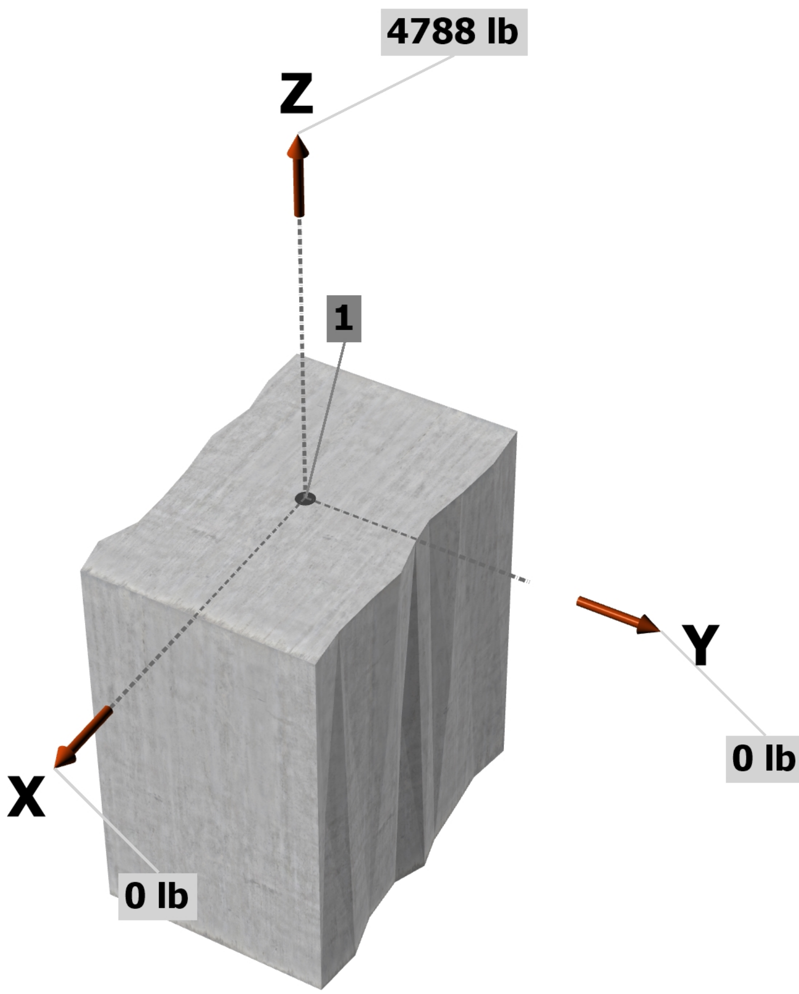
Recommended Anchor
Anchor Name: ET-3G™ - ET-3G w/ 5/8"Ø F1554 Gr. 36
Code Report: ESR-5334

Load and Geometry
Load factor source: ACI 318 Section 5.3
Load combination: not set
Seismic design: No
Anchors subjected to sustained tension: No
Apply entire shear load at front row: Yes
Anchors only resisting wind and/or seismic loads: Yes

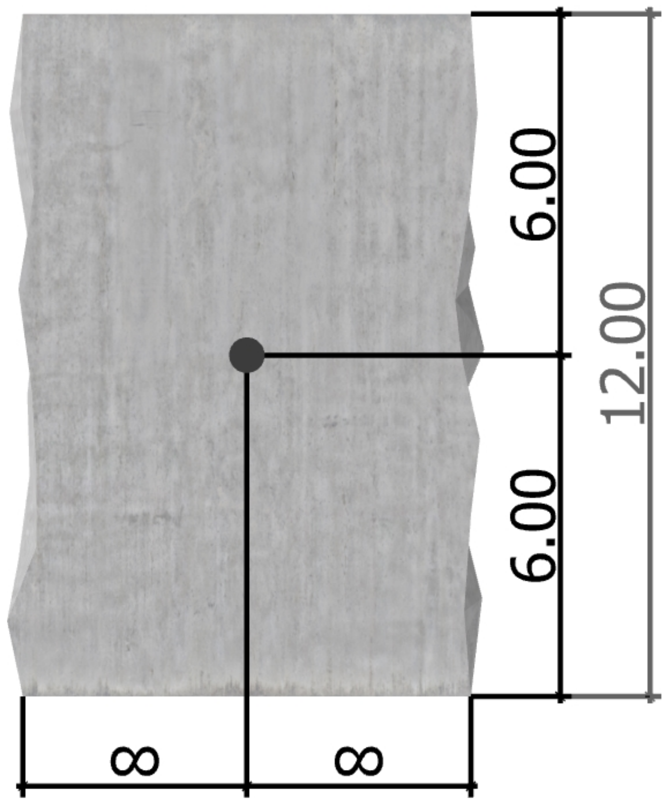
Strength level loads:

N_{ua} [lb]:	4788
V_{uax} [lb]:	0
V_{uay} [lb]:	0

<Figure 1>



<Figure 2>



3. Resulting Anchor Forces

Anchor	Tension load, N_{ua} (lb)	Shear load x, V_{uax} (lb)	Shear load y, V_{uay} (lb)	Shear load combined, $\sqrt{(V_{uax})^2+(V_{uay})^2}$ (lb)
1	4788.0	0.0	0.0	0.0
Sum	4788.0	0.0	0.0	0.0

Maximum concrete compression strain (‰): 0.00
Maximum concrete compression stress (psi): 0
Resultant tension force (lb): 4788
Resultant compression force (lb): 0
Eccentricity of resultant tension forces in x-axis, e'_{Nx} (inch): 0.00
Eccentricity of resultant tension forces in y-axis, e'_{Ny} (inch): 0.00

4. Steel Strength of Anchor in Tension (Sec. 17.6.1)

N_{sa} (lb)	ϕ	ϕN_{sa} (lb)
13110	0.75	9833

5. Concrete Breakout Strength of Anchor in Tension (Sec. 17.6.2)

$N_b = k_c \lambda_a \sqrt{f'_c} h_{ef}^{1.5}$ (Eq. 17.6.2.2.1)

k_c	λ_a	f'_c (psi)	h_{ef} (in)	N_b (lb)
17.0	1.00	2500	8.000	19233

$\phi N_{cb} = \phi\,(A_{Nc} / A_{Nco})\,\Psi_{ed,N}\,\Psi_{c,N}\,\Psi_{cp,N} N_b$ (Sec. 17.5.1.2 & Eq. 17.6.2.1a)

A_{Nc} (in^2)	A_{Nco} (in^2)	$c_{a,min}$ (in)	$\Psi_{ed,N}$	$\Psi_{c,N}$	$\Psi_{cp,N}$	N_b (lb)	ϕ	ϕN_{cb} (lb)
288.00	576.00	6.00	0.850	1.00	1.000	19233	0.65	5313

6. Adhesive Strength of Anchor in Tension (Sec. 17.6.5)

$\tau_{k,cr} = \tau_{k,cr} f_{short\text{-}term} K_{sat}$

$\tau_{k,cr}$ (psi)	$f_{short\text{-}term}$	K_{sat}	$\tau_{k,cr}$ (psi)
430	1.72	1.00	740

$N_{ba} = \lambda_a \tau_{cr} \pi d_a h_{ef}$ (Eq. 17.6.5.2.1)

λ_a	τ_{cr} (psi)	d_a (in)	h_{ef} (in)	N_{ba} (lb)
1.00	740	0.63	8.000	11618

$\phi N_a = \phi\,(A_{Na} / A_{Na0})\,\Psi_{ed,Na}\,\Psi_{cp,Na} N_{ba}$ (Sec. 17.5.1.2 & Eq. 17.6.5.1a)

A_{Na} (in^2)	A_{Na0} (in^2)	c_{Na} (in)	$c_{a,min}$ (in)	$\Psi_{ed,Na}$	$\Psi_{cp,Na}$	N_{ba} (lb)	ϕ	ϕN_a (lb)
192.20	256.53	8.01	6.00	0.925	1.000	11618	0.65	5232

11. Results

Interaction of Tensile and Shear Forces (Sec. 17.8)

Tension	Factored Load, N_{ua} (lb)	Design Strength, $øN_n$ (lb)	Ratio	Status
Steel	4788	9833	0.49	Pass
Concrete breakout	4788	5313	0.90	Pass
Adhesive	**4788**	**5232**	**0.92**	**Pass (Governs)**

ET-3G w/ 5/8"Ø F1554 Gr. 36 with hef = 8.000 inch meets the selected design criteria.

12. Warnings

- When cracked concrete is selected, concrete compressive strength used in concrete breakout strength in tension, adhesive strength in tension and concrete pryout strength in shear for ET-3G adhesive anchor is limited to 2,500 psi per ICC-ES ESR-5334 Section 5.3.

- Designer must exercise own judgement to determine if this design is suitable.

- Refer to manufacturer's product literature for hole cleaning and installation instructions.

Shear Walls

Gridline 15

Structural Sheathing

1/2" dia. anchor bolts

	Length	Inside	Ratio h:w	Ratio 2w/h	SWS Wind	SWS Seismic
panel 1	16		0.6 :1	1.00	62	24
panel 2						
panel 3						
panel 4						
panel 5						
panel 6						
panel 7						
panel 8						
panel 9						
panel 10						
$ASW_{1,2}$=	16	**16**	Total=	16.00		

Wind

$TA_{Roof-end}$	$TA_{Roof-int.}$	V_s
30	29	856
$TA_{Wall-end}$	$TA_{Wall-int}$	$V_{s\ min}$
34	40	**994**
	$V_{additional}$ =	

Seismic

A_i	w_i	F	ρ	R	F_x	V_{final}
1152	17460	1.000	1.000	6.5	544	**381**
					$V_{additional}$ =	

SW1	SW2	SW3	SW4	SW1	SW2	SW3	SW4
339 plf	495 plf	637 plf	832 plf	241 plf	350 plf	455 plf	595 plf
Total Resistance$_{wind}$				Total Resistance$_{seismic}$			
5424	7920	10192	13312	3856	5600	7280	9520
Use SW1				Use SW1			

Check uplift

DL	h	h'	Uplift $_w$	Uplift $_s$
100	9	9	-241	-586

Use SW1
No Uplift
Use 1/2" dia. anchor bolts @ 32" o.c.

Perforated Shearwall 1:		**NOT USED**		*t = v =*	0
Total Length =		V_{wind}	V_{seis}	Uplift $_w$	Uplift $_s$
Height =		0	0	0	0
Max opening height=		**Ratio**			
C_o =		h:w	2w/h		
segment 1					
segment 2					
segment 3					
segment 4					
segment 5					
		Total=	0.00		

Perforated Shearwall 2:		**NOT USED**		*t = v =*	0
Total Length =		V_{wind}	V_{seis}	Uplift $_w$	Uplift $_s$
Height =		0	0	0	0
Max opening height=		**Ratio**			
C_o =		h:w	2w/h		
segment 1					
segment 2					
segment 3					
		Total=	0.00		

Perforated Shearwall 3:		**NOT USED**		*t = v =*	0
Total Length =		V_{wind}	V_{seis}	Uplift $_w$	Uplift $_s$
Height =		0	0	0	0
Max opening height=		**Ratio**			
C_o =		h:w	2w/h		
segment 1					
segment 2					
segment 3					
		Total=	0.00		

Gridline 16

Structural Sheathing

1/2" dia. anchor bolts

	Length	Inside	Ratio h:w	Ratio 2w/h	SWS Wind	SWS Seismic
panel 1	2.5		3.2 :1	0.63	164	62
panel 2	3		2.7 :1	0.75	197	75
panel 3						
panel 4						
panel 5						
panel 6						
panel 7						
panel 8						
panel 9						
panel 10						
$ASW_{1,2}$=	5.5	**5.5**	Total=	3.81		

Wind

$TA_{Roof-end}$	$TA_{Roof-int.}$	V_s
18	29	955
$TA_{Wall-end}$	$TA_{Wall-int}$	$V_{s\ min}$
41	40	**1003**
	$V_{additional}$ =	

Seismic

A_i	w_i	F	ρ	R	F_x	V_{final}
1152	17460	1.000	1.000	6.5	544	**381**
					$V_{additional}$ =	

SW1	SW2	SW3	SW4	SW1	SW2	SW3	SW4
339 plf	495 plf	637 plf	832 plf	241 plf	350 plf	455 plf	595 plf
Total Resistance$_{wind}$				Total Resistance$_{seismic}$			
1292	1887	2429	3172	919	1334	1735	2268
Use SW1				Use SW1			

Check uplift

DL	h	h'	Uplift $_w$	Uplift $_s$
100	9	8	1191	375
100	9	8	1429	450

Use SW1
Use LSTHD8 holdowns each side of panel as noted on plans
Use 1/2" dia. anchor bolts @ 32" o.c.

Perforated Shearwall 1:		**NOT USED**		*t = v =*	0
Total Length =		V_{wind}	V_{seis}	Uplift $_w$	Uplift $_s$
Height =		0	0	0	0
Max opening height=		**Ratio**			
C_o =		h:w	2w/h		
segment 1					
segment 2					
segment 3					
segment 4					
segment 5					
		Total=	0.00		

Perforated Shearwall 2:		**NOT USED**		*t = v =*	0
Total Length =		V_{wind}	V_{seis}	Uplift $_w$	Uplift $_s$
Height =		0	0	0	0
Max opening height=		**Ratio**			
C_o =		h:w	2w/h		
segment 1					
segment 2					
segment 3					
		Total=	0.00		

Perforated Shearwall 3:		**NOT USED**		*t = v =*	0
Total Length =		V_{wind}	V_{seis}	Uplift $_w$	Uplift $_s$
Height =		0	0	0	0
Max opening height=		**Ratio**			
C_o =		h:w	2w/h		
segment 1					
segment 2					
segment 3					
		Total=	0.00		

Shear Walls

				Ratio		SWS		Wind			Seismic							Check uplift				
		Length	Inside	h:w	2w/h	Wind	Seismic	$TA_{Roof-end}$	$TA_{Roof-int.}$	V_s	A_i	w_i	F	ρ	R	F_x	V_{final}	DL	h	h'	Uplift $_w$	Uplift $_s$
Gridline 17	panel 1	16		0.6 :1	1.00	97	24	0	0	1552	1152	17460	1.000	1.000	6.5	544	**381**	160	9	9	-407	-1066
	panel 2																					
	panel 3																					
Structural Sheathing	panel 4																					
	panel 5																					
No anchor bolts	panel 6																					
	panel 7																					
	panel 8																					
	panel 9																					
	panel 10																					
	$ASW_{1,2}$=	16	**16**	Total=	16.00																	

$TA_{Wall-end}$	$TA_{Wall-int}$	$V_{s\ min}$
70	60	**1552**

$V_{additional}$ =

$V_{additional}$ =

SW1	SW2	SW3	SW4	SW1	SW2	SW3	SW4
339 plf	495 plf	637 plf	832 plf	241 plf	350 plf	455 plf	595 plf
Total Resistance$_{wind}$				**Total Resistance$_{seismic}$**			
5424	7920	10192	13312	3856	5600	7280	9520
Use SW1				Use SW1			

Use SW1
No Uplift

Perforated Shearwall 1:		**NOT USED**		*t = v =*	0
Total Length =		V_{wind}	V_{seis}	Uplift $_w$	Uplift $_s$
Height =		0	0	0	0
Max opening height=		**Ratio**			
C_o =		h:w	2w/h		
segment 1					
segment 2					
segment 3					
segment 4					
segment 5					
		Total=	0.00		

Perforated Shearwall 2:		**NOT USED**		*t = v =*	0
Total Length =		V_{wind}	V_{seis}	Uplift $_w$	Uplift $_s$
Height =		0	0	0	0
Max opening height=		**Ratio**			
C_o =		h:w	2w/h		
segment 1					
segment 2					
segment 3					
		Total=	0.00		

Perforated Shearwall 3:		**NOT USED**		*t = v =*	0
Total Length =		V_{wind}	V_{seis}	Uplift $_w$	Uplift $_s$
Height =		0	0	0	0
Max opening height=		**Ratio**			
C_o =		h:w	2w/h		
segment 1					
segment 2					
segment 3					
		Total=	0.00		

				Ratio		SWS		Wind			Seismic							Check uplift				
		Length	Inside	h:w	2w/h	Wind	Seismic	$TA_{Roof-end}$	$TA_{Roof-int.}$	V_s	A_i	w_i	F	ρ	R	F_x	V_{final}	DL	h	h'	Uplift $_w$	Uplift $_s$
Gridline 18	panel 1	6.67		1.3 :1	1.00	155	57	0	0	1035	1152	17460	1.000	1.000	6.5	544	**381**	100	9	9	1063	181
	panel 2																					
	panel 3																					
Structural Sheathing	panel 4																					
	panel 5																					
1/2" dia. anchor bolts	panel 6																					
	panel 7																					
	panel 8																					
	panel 9																					
	panel 10																					
	$ASW_{1,2}$=	6.67	**6.67**	Total=	6.67																	

$TA_{Wall-end}$	$TA_{Wall-int}$	$V_{s\ min}$
42	47	**1035**

$V_{additional}$ =

$V_{additional}$ =

SW1	SW2	SW3	SW4	SW1	SW2	SW3	SW4
339 plf	495 plf	637 plf	832 plf	241 plf	350 plf	455 plf	595 plf
Total Resistance$_{wind}$				**Total Resistance$_{seismic}$**			
2261	3302	4249	5549	1607	2335	3035	3969
Use SW1				Use SW1			

Use SW1
Use LSTHD8 holdowns each side of panel as noted on plans
Use 1/2" dia. anchor bolts @ 32" o.c.

Perforated Shearwall 1:		**NOT USED**		*t = v =*	0
Total Length =		V_{wind}	V_{seis}	Uplift $_w$	Uplift $_s$
Height =		0	0	0	0
Max opening height=		**Ratio**			
C_o =		h:w	2w/h		
segment 1					
segment 2					
segment 3					
segment 4					
segment 5					
		Total=	0.00		

Perforated Shearwall 2:		**NOT USED**		*t = v =*	0
Total Length =		V_{wind}	V_{seis}	Uplift $_w$	Uplift $_s$
Height =		0	0	0	0
Max opening height=		**Ratio**			
C_o =		h:w	2w/h		
segment 1					
segment 2					
segment 3					
		Total=	0.00		

Perforated Shearwall 3:		**NOT USED**		*t = v =*	0
Total Length =		V_{wind}	V_{seis}	Uplift $_w$	Uplift $_s$
Height =		0	0	0	0
Max opening height=		**Ratio**			
C_o =		h:w	2w/h		
segment 1					
segment 2					
segment 3					
		Total=	0.00		

Horiz. Diaphragm

		Wind			Seismic							Diaphragm				
	Length	$TA_{Roof\text{-}end}$	$TA_{Roof\text{-}int.}$	V_s	A_i	w_i	F	ρ	R	F_x	V_{final}	V	$v_{diaphragm}$	v_{allow}	Check	
Gridline 1 L =	68	0	0	3795	1820	30430	1.000	1.000	6.5	949	**664**	3795	**172**	238	**OK**	**Wind**
b =	26.5	$TA_{Wall\text{-}end}$	$TA_{Wall\text{-}int}$	$V_{s\ min}$						$V_{additional}$ =		664	**30**	170	**OK**	**Seismic**
$b_{Collector}$=	22	71	297	**3795**												
Simpson Strong Wall O =	16		$V_{additional}$ =													
$O_{Perf.}$ =	16															

Deflection
OK

Z =	141 lb
Z' =	248 lb

Roof
(Unblocked)
Load Case 3

Diaphragm Chord Force	
M =	64514 ft-lb
T =	2434 lb

Diaphragm Collector Force	
T =	2760 lb
$T_{Perf.}$ =	0 lb

Top Plate Splice		
N =	11.1	nails
Use	**12**	**16d nails between splice points**

Use (4) 16d common toenails at full height truss blocking
Use 7/16" APA rated OSB sheathing w/ 8d nails @ 6" o.c. edge, 12" o.c. field (Unblocked)

		Wind			Seismic							Diaphragm				
	Length	$TA_{Roof\text{-}end}$	$TA_{Roof\text{-}int.}$	V_s	A_i	w_i	F	ρ	R	F_x	V_{final}	V	$v_{diaphragm}$	v_{allow}	Check	
Gridline 2 L =	68	0	0	3800	1820	30430	1.000	1.000	6.5	949	**664**	3800	**173**	238	**OK**	**Wind**
b =	26.5	$TA_{Wall\text{-}end}$	$TA_{Wall\text{-}int}$	$V_{s\ min}$						$V_{additional}$ =		664	**30**	170	**OK**	**Seismic**
$b_{Collector}$=	22	80	284	**3800**												
Structural Sheathing O =	3		$V_{additional}$ =													
$O_{Perf.}$ =	3															

Deflection
OK

Z =	141 lb
Z' =	248 lb

Roof
(Unblocked)
Load Case 3

Diaphragm Chord Force	
M =	64596 ft-lb
T =	2438 lb

Diaphragm Collector Force	
T =	518 lb
$T_{Perf.}$ =	391 lb

Top Plate Splice		
N =	9.8	nails
Use	**10**	**16d nails between splice points**

Use (4) 16d common toenails at full height truss blocking
Use 7/16" APA rated OSB sheathing w/ 8d nails @ 6" o.c. edge, 12" o.c. field (Unblocked)

		Wind			Seismic							Diaphragm				
	Length	$TA_{Roof\text{-}end}$	$TA_{Roof\text{-}int.}$	V_s	A_i	w_i	F	ρ	R	F_x	V_{final}	V	$v_{diaphragm}$	v_{allow}	Check	
Gridline 3 L =	35	0	0	1087	805	12963	1.000	1.000	6.5	404	**283**	1087	**47**	238	**OK**	**Wind**
b =	23	$TA_{Wall\text{-}end}$	$TA_{Wall\text{-}int}$	$V_{s\ min}$						$V_{additional}$ =		283	**12**	170	**OK**	**Seismic**
$b_{Collector}$=	23	77	0	**1087**												
Simpson Strong Wall O =	18		$V_{additional}$ =													
$O_{Perf.}$ =	18															

Deflection
OK

Z =	141 lb
Z' =	248 lb

Roof
(Unblocked)
Load Case 3

Diaphragm Chord Force	
M =	9512 ft-lb
T =	414 lb

Diaphragm Collector Force	
T =	851 lb
$T_{Perf.}$ =	0 lb

Top Plate Splice		
N =	3.4	nails
Use	**8**	**16d nails between splice points**

Use (4) 16d common toenails at full height truss blocking
Use 7/16" APA rated OSB sheathing w/ 8d nails @ 6" o.c. edge, 12" o.c. field (Unblocked)

Horiz. Diaphragm

Gridline 4 — Gypsum Sheathing — Roof (Unblocked) Load Case 3

	Length	Wind $TA_{Roof-end}$	Wind $TA_{Roof-int.}$	Wind V_s	Seismic A_i	w_i	F	ρ	R	F_x	V_{final}	Diaphragm V	$v_{diaphragm}$	v_{allow}	Check	
L =	41	0	0	0	1824	29800	1.000	1.000	2	3020	**2114**	0	**0**	238	**OK**	**Wind**
b =	22	$TA_{Wall-end}$	$TA_{Wall-int}$	$V_{s\ min}$						$V_{additional}$ =		2114	**96**	170	**OK**	**Seismic**
$b_{Collector}$=	22	0	0	**0**												
O =	0		$V_{additional}$ =													
$O_{Perf.}$ =	0															

Deflection
OK

Z = 141 lb
Z' = 248 lb

Diaphragm Chord Force	
M =	21667 ft-lb
T =	985 lb

Diaphragm Collector Force	
T =	0 lb
$T_{Perf.}$ =	0 lb

Top Plate Splice		
N =	4.0	nails
Use	**8**	**16d nails between splice points**

Use (4) 16d common toenails at full height truss blocking
Use 7/16" APA rated OSB sheathing w/ 8d nails @ 6" o.c. edge, 12" o.c. field (Unblocked)

Gridline 5 — Structural Sheathing — Roof (Unblocked) Load Case 3

	Length	Wind $TA_{Roof-end}$	Wind $TA_{Roof-int.}$	Wind V_s	Seismic A_i	w_i	F	ρ	R	F_x	V_{final}	Diaphragm V	$v_{diaphragm}$	v_{allow}	Check	
L =	44	0	0	2437	1800	26750	1.000	1.000	6.5	834	**584**	1597	**40**	238	**OK**	**Wind**
b =	40	$TA_{Wall-end}$	$TA_{Wall-int}$	$V_{s\ min}$						$V_{additional}$ =		584	**15**	170	**OK**	**Seismic**
$b_{Collector}$=	40	90	124	**1597**												
O =	3		$V_{additional}$ =	-840												
$O_{Perf.}$ =	3															

Deflection
OK

Z = 141 lb
Z' = 248 lb

Diaphragm Chord Force	
M =	17562 ft-lb
T =	439 lb

Diaphragm Collector Force	
T =	120 lb
$T_{Perf.}$ =	0 lb

Top Plate Splice		
N =	1.8	nails
Use	**8**	**16d nails between splice points**

Use (4) 16d common toenails at full height truss blocking
Use 7/16" APA rated OSB sheathing w/ 8d nails @ 6" o.c. edge, 12" o.c. field (Unblocked)

Gridline 6 — Structural Sheathing — Roof (Unblocked) Load Case 3

	Length	Wind $TA_{Roof-end}$	Wind $TA_{Roof-int.}$	Wind V_s	Seismic A_i	w_i	F	ρ	R	F_x	V_{final}	Diaphragm V	$v_{diaphragm}$	v_{allow}	Check	
L =	44	0	0	2821	2925	44263	1.000	1.000	6.5	1380	**966**	2821	**66**	238	**OK**	**Wind**
b =	43	$TA_{Wall-end}$	$TA_{Wall-int}$	$V_{s\ min}$						$V_{additional}$ =		966	**22**	170	**OK**	**Seismic**
$b_{Collector}$=	43	40	240	**2821**												
O =	23		$V_{additional}$ =													
$O_{Perf.}$ =	23															

Deflection
OK

Z = 141 lb
Z' = 248 lb

Diaphragm Chord Force	
M =	31035 ft-lb
T =	722 lb

Diaphragm Collector Force	
T =	1509 lb
$T_{Perf.}$ =	0 lb

Top Plate Splice		
N =	6.1	nails
Use	**8**	**16d nails between splice points**

Use (4) 16d common toenails at full height truss blocking
Use 7/16" APA rated OSB sheathing w/ 8d nails @ 6" o.c. edge, 12" o.c. field (Unblocked)

Horiz. Diaphragm

			Wind			Seismic							Diaphragm				
		Length	$TA_{Roof-end}$	$TA_{Roof-int.}$	V_s	A_i	w_i	F	ρ	R	F_x	V_{final}	V	$v_{diaphragm}$	v_{allow}	Check	
Gridline 7	L =	19	0	0	1340	1500	24350	1.000	1.000	6.5	759	**531**	1340	**36**	238	**OK**	**Wind**
	b =	37	$TA_{Wall-end}$	$TA_{Wall-int}$	$V_{s\ min}$						$V_{additional}$ =		531	**14**	170	**OK**	**Seismic**
	$b_{Collector}$=	37	37	87	**1340**									**Deflection**			
Simpson Strong Wall	O =	15		$V_{additional}$ =						Z =	141	lb		**OK**			
	$O_{Perf.}$ =	15								Z' =	248	lb					

Roof	**Diaphragm Chord Force**			**Diaphragm Collector Force**			**Top Plate Splice**		
(Unblocked)	M =	6366.8	ft-lb	T =	543	lb	N =	2.2	nails
Load Case 3	T =	172	lb	$T_{Perf.}$ =	0	lb	**Use**	**8**	**16d nails between splice points**

Use (4) 16d common toenails at full height truss blocking
Use 7/16" APA rated OSB sheathing w/ 8d nails @ 6" o.c. edge, 12" o.c. field (Unblocked)

			Wind			Seismic							Diaphragm				
		Length	$TA_{Roof-end}$	$TA_{Roof-int.}$	V_s	A_i	w_i	F	ρ	R	F_x	V_{final}	V	$v_{diaphragm}$	v_{allow}	Check	
Gridline 8	L =	18	0	0	1341	900	14500	1.000	1.000	6.5	452	**316**	1341	**103**	238	**OK**	**Wind**
	b =	13	$TA_{Wall-end}$	$TA_{Wall-int}$	$V_{s\ min}$						$V_{additional}$ =		316	**24**	170	**OK**	**Seismic**
	$b_{Collector}$=	13	95	0	**1341**									**Deflection**			
Structural Sheathing	O =	8		$V_{additional}$ =						Z =	141	lb		**OK**			
	$O_{Perf.}$ =	8								Z' =	248	lb					

Roof	**Diaphragm Chord Force**			**Diaphragm Collector Force**			**Top Plate Splice**		
(Unblocked)	M =	6035.4	ft-lb	T =	825	lb	N =	3.3	nails
Load Case 3	T =	464	lb	$T_{Perf.}$ =	0	lb	**Use**	**8**	**16d nails between splice points**

Use (4) 16d common toenails at full height truss blocking
Use 7/16" APA rated OSB sheathing w/ 8d nails @ 6" o.c. edge, 12" o.c. field (Unblocked)

			Wind			Seismic							Diaphragm				
		Length	$TA_{Roof-end}$	$TA_{Roof-int.}$	V_s	A_i	w_i	F	ρ	R	F_x	V_{final}	V	$v_{diaphragm}$	v_{allow}	Check	
Gridline 9	L =	22	0	0	1661	1820	30430	1.000	1.000	6.5	949	**664**	1661	**24**	238	**OK**	**Wind**
	b =	68	$TA_{Wall-end}$	$TA_{Wall-int}$	$V_{s\ min}$						$V_{additional}$ =		664	**10**	170	**OK**	**Seismic**
	$b_{Collector}$=	68	87	46	**1661**									**Deflection**			
Structural Sheathing	O =	23		$V_{additional}$ =						Z =	141	lb		**OK**			
	$O_{Perf.}$ =	23								Z' =	248	lb					

Roof	**Diaphragm Chord Force**			**Diaphragm Collector Force**			**Top Plate Splice**		
(Unblocked)	M =	9134.3	ft-lb	T =	562	lb	N =	6.6	nails
Load Case 3	T =	134	lb	$T_{Perf.}$ =	1649	lb	**Use**	**8**	**16d nails between splice points**

Use (4) 16d common toenails at full height truss blocking
Use 7/16" APA rated OSB sheathing w/ 8d nails @ 6" o.c. edge, 12" o.c. field (Unblocked)

Horiz. Diaphragm

Gridline 10

		Wind			Seismic							Diaphragm				
	Length	$TA_{Roof-end}$	$TA_{Roof-int.}$	V_s	A_i	w_i	F	ρ	R	F_x	V_{final}	V	$v_{diaphragm}$	v_{allow}	Check	
L =	22	0	0	2257	3290	50485	1.000	1.000	6.5	1574	**1102**	2304	**30**	238	**OK**	**Wind**
b =	76	$TA_{Wall-end}$	$TA_{Wall-int}$	$V_{s\ min}$						$V_{additional}$ =		1102	**14**	170	**OK**	**Seismic**
$b_{Collector}$=	76	0	240	**2304**								**Deflection**				
O =	39		$V_{additional}$ =						Z =	141	lb	**OK**				
$O_{Perf.}$ =	39								Z' =	248	lb					

Structural Sheathing

Roof (Unblocked) Load Case 3

Diaphragm Chord Force	
M =	12672 ft-lb
T =	167 lb

Diaphragm Collector Force	
T =	1182 lb
$T_{Perf.}$ =	0 lb

Top Plate Splice		
N =	4.8	nails
Use	**8**	**16d nails between splice points**

Use (4) 16d common toenails at full height truss blocking
Use 7/16" APA rated OSB sheathing w/ 8d nails @ 6" o.c. edge, 12" o.c. field (Unblocked)

Gridline 11

		Wind			Seismic							Diaphragm				
	Length	$TA_{Roof-end}$	$TA_{Roof-int.}$	V_s	A_i	w_i	F	ρ	R	F_x	V_{final}	V	$v_{diaphragm}$	v_{allow}	Check	
L =	22	0	0	1693	1188	20370	1.000	1.000	6.5	635	**445**	1693	**50**	238	**OK**	**Wind**
b =	34	$TA_{Wall-end}$	$TA_{Wall-int}$	$V_{s\ min}$						$V_{additional}$ =		445	**13**	170	**OK**	**Seismic**
$b_{Collector}$=	34	62	87	**1693**								**Deflection**				
O =	3		$V_{additional}$ =						Z =	141	lb	**OK**				
$O_{Perf.}$ =	3								Z' =	248	lb					

Structural Sheathing

Roof (Unblocked) Load Case 3

Diaphragm Chord Force	
M =	9313.4 ft-lb
T =	274 lb

Diaphragm Collector Force	
T =	149 lb
$T_{Perf.}$ =	0 lb

Top Plate Splice		
N =	1.1	nails
Use	**8**	**16d nails between splice points**

Use (4) 16d common toenails at full height truss blocking
Use 7/16" APA rated OSB sheathing w/ 8d nails @ 6" o.c. edge, 12" o.c. field (Unblocked)

Gridline 12

		Wind			Seismic							Diaphragm				
	Length	$TA_{Roof-end}$	$TA_{Roof-int.}$	V_s	A_i	w_i	F	ρ	R	F_x	V_{final}	V	$v_{diaphragm}$	v_{allow}	Check	
L =	40	0	0	2229	2250	32875	1.000	1.000	6.5	1025	**718**	2229	**57**	238	**OK**	**Wind**
b =	39	$TA_{Wall-end}$	$TA_{Wall-int}$	$V_{s\ min}$						$V_{additional}$ =		718	**18**	170	**OK**	**Seismic**
$b_{Collector}$=	39	54	156	**2229**								**Deflection**				
O =	10		$V_{additional}$ =						Z =	141	lb	**OK**				
$O_{Perf.}$ =	10								Z' =	248	lb					

Structural Sheathing

Roof (Unblocked) Load Case 3

Diaphragm Chord Force	
M =	22292 ft-lb
T =	572 lb

Diaphragm Collector Force	
T =	572 lb
$T_{Perf.}$ =	0 lb

Top Plate Splice		
N =	2.3	nails
Use	**8**	**16d nails between splice points**

Use (4) 16d common toenails at full height truss blocking
Use 7/16" APA rated OSB sheathing w/ 8d nails @ 6" o.c. edge, 12" o.c. field (Unblocked)

Horiz. Diaphragm

			Wind			Seismic							Diaphragm				
		Length	$TA_{Roof\text{-}end}$	$TA_{Roof\text{-}int.}$	V_s	A_i	w_i	F	ρ	R	F_x	V_{final}	V	$v_{diaphragm}$	v_{allow}	Check	
Gridline 13	L =	40	0	0	3500	4000	56500	1.000	1.000	6.5	1762	**1233**	3500	**56**	238	**OK**	**Wind**
	b =	62	$TA_{Wall\text{-}end}$	$TA_{Wall\text{-}int}$	$V_{s\ min}$						$V_{additional}$ =		1233	**20**	170	**OK**	**Seismic**
	$b_{Collector}$=	62	128	180	**3500**									**Deflection**			
Structural Sheathing	O =	3		$V_{additional}$ =						Z =	141 lb			**OK**			
	$O_{Perf.}$ =	3								Z' =	248 lb						

Roof	**Diaphragm Chord Force**	**Diaphragm Collector Force**	**Top Plate Splice**
(Unblocked)	M = 34996 ft-lb	T = 169 lb	N = 2.3 nails
Load Case 3	T = 564 lb	$T_{Perf.}$ = 382 lb	**Use 8 16d nails between splice points**

Use (4) 16d common toenails at full height truss blocking
Use 7/16" APA rated OSB sheathing w/ 8d nails @ 6" o.c. edge, 12" o.c. field (Unblocked)

			Wind			Seismic							Diaphragm				
		Length	$TA_{Roof\text{-}end}$	$TA_{Roof\text{-}int.}$	V_s	A_i	w_i	F	ρ	R	F_x	V_{final}	V	$v_{diaphragm}$	v_{allow}	Check	
Gridline 14	L =	48	0	0	2436	1107	17238	1.000	1.000	6.5	537	**376**	2436	**128**	238	**OK**	**Wind**
	b =	19	$TA_{Wall\text{-}end}$	$TA_{Wall\text{-}int}$	$V_{s\ min}$						$V_{additional}$ =		376	**20**	170	**OK**	**Seismic**
	$b_{Collector}$=	19	68	157	**2436**									**Deflection**			
Structural Sheathing	O =	6		$V_{additional}$ =						Z =	141 lb			**OK**			
	$O_{Perf.}$ =	6								Z' =	248 lb						

Roof	**Diaphragm Chord Force**	**Diaphragm Collector Force**	**Top Plate Splice**
(Unblocked)	M = 29235 ft-lb	T = 769 lb	N = 6.2 nails
Load Case 3	T = 1539 lb	$T_{Perf.}$ = 0 lb	**Use 8 16d nails between splice points**

Use (4) 16d common toenails at full height truss blocking
Use 7/16" APA rated OSB sheathing w/ 8d nails @ 6" o.c. edge, 12" o.c. field (Unblocked)

			Wind			Seismic							Diaphragm				
		Length	$TA_{Roof\text{-}end}$	$TA_{Roof\text{-}int.}$	V_s	A_i	w_i	F	ρ	R	F_x	V_{final}	V	$v_{diaphragm}$	v_{allow}	Check	
Gridline 15	L =	31.5	30	29	856	1152	17460	1.000	1.000	6.5	544	**381**	994	**45**	238	**OK**	**Wind**
	b =	22	$TA_{Wall\text{-}end}$	$TA_{Wall\text{-}int}$	$V_{s\ min}$						$V_{additional}$ =		381	**17**	170	**OK**	**Seismic**
	$b_{Collector}$=	22	34	40	**994**									**Deflection**			
Structural Sheathing	O =	0		$V_{additional}$ =						Z =	141 lb			**OK**			
	$O_{Perf.}$ =	0								Z' =	248 lb						

Roof	**Diaphragm Chord Force**	**Diaphragm Collector Force**	**Top Plate Splice**
(Unblocked)	M = 7824.6 ft-lb	T = 0 lb	N = 1.4 nails
Load Case 3	T = 356 lb	$T_{Perf.}$ = 0 lb	**Use 8 16d nails between splice points**

Use (4) 16d common toenails at full height truss blocking
Use 7/16" APA rated OSB sheathing w/ 8d nails @ 6" o.c. edge, 12" o.c. field (Unblocked)

Horiz. Diaphragm

			Wind			Seismic							Diaphragm				
		Length	$TA_{Roof\text{-}end}$	$TA_{Roof\text{-}int.}$	V_s	A_i	w_i	F	ρ	R	F_x	V_{final}	V	$v_{diaphragm}$	v_{allow}	Check	
Gridline 16	L =	31.5	18	29	955	1152	17460	1.000	1.000	6.5	544	**381**	1003	**67**	238	**OK**	**Wind**
	b =	22	$TA_{Wall\text{-}end}$	$TA_{Wall\text{-}int}$	$V_{s\ min}$						$V_{additional}$ =		381	**25**	170	**OK**	**Seismic**
	$b_{Collector}$=	15	41	40	**1003**									**Deflection**			
Structural Sheathing	O =	9		$V_{additional}$ =						Z =	141	lb		**OK**			
	$O_{Perf.}$ =	9								Z' =	248	lb					

Roof	**Diaphragm Chord Force**		**Diaphragm Collector Force**		**Top Plate Splice**		
(Unblocked)	M =	7900.2 ft-lb	T =	602 lb	N =	2.4	nails
Load Case 3	T =	359 lb	$T_{Perf.}$ =	0 lb	**Use**	**8**	**16d nails between splice points**

Use (4) 16d common toenails at full height truss blocking
Use 7/16" APA rated OSB sheathing w/ 8d nails @ 6" o.c. edge, 12" o.c. field (Unblocked)

			Wind			Seismic							Diaphragm				
		Length	$TA_{Roof\text{-}end}$	$TA_{Roof\text{-}int.}$	V_s	A_i	w_i	F	ρ	R	F_x	V_{final}	V	$v_{diaphragm}$	v_{allow}	Check	
Gridline 17	L =	22	0	0	1552	1152	17460	1.000	1.000	6.5	544	**381**	1552	**49**	238	**OK**	**Wind**
	b =	31.5	$TA_{Wall\text{-}end}$	$TA_{Wall\text{-}int}$	$V_{s\ min}$						$V_{additional}$ =		381	**12**	170	**OK**	**Seismic**
	$b_{Collector}$=	31.5	70	60	**1552**									**Deflection**			
Structural Sheathing	O =	0		$V_{additional}$ =						Z =	141	lb		**OK**			
	$O_{Perf.}$ =	0								Z' =	248	lb					

Roof	**Diaphragm Chord Force**		**Diaphragm Collector Force**		**Top Plate Splice**		
(Unblocked)	M =	8538.3 ft-lb	T =	0 lb	N =	1.1	nails
Load Case 3	T =	271 lb	$T_{Perf.}$ =	0 lb	**Use**	**8**	**16d nails between splice points**

Use (4) 16d common toenails at full height truss blocking
Use 7/16" APA rated OSB sheathing w/ 8d nails @ 6" o.c. edge, 12" o.c. field (Unblocked)

			Wind			Seismic							Diaphragm				
		Length	$TA_{Roof\text{-}end}$	$TA_{Roof\text{-}int.}$	V_s	A_i	w_i	F	ρ	R	F_x	V_{final}	V	$v_{diaphragm}$	v_{allow}	Check	
Gridline 18	L =	22	0	0	1035	1152	17460	1.000	1.000	6.5	544	**381**	1035	**67**	238	**OK**	**Wind**
	b =	31.5	$TA_{Wall\text{-}end}$	$TA_{Wall\text{-}int}$	$V_{s\ min}$						$V_{additional}$ =		381	**25**	170	**OK**	**Seismic**
	$b_{Collector}$=	15.5	42	47	**1035**									**Deflection**			
Structural Sheathing	O =	4		$V_{additional}$ =						Z =	141	lb		**OK**			
	$O_{Perf.}$ =	4								Z' =	248	lb					

Roof	**Diaphragm Chord Force**		**Diaphragm Collector Force**		**Top Plate Splice**		
(Unblocked)	M =	5691.8 ft-lb	T =	267 lb	N =	1.1	nails
Load Case 3	T =	181 lb	$T_{Perf.}$ =	0 lb	**Use**	**8**	**16d nails between splice points**

Use (4) 16d common toenails at full height truss blocking
Use 7/16" APA rated OSB sheathing w/ 8d nails @ 6" o.c. edge, 12" o.c. field (Unblocked)

STUD WALL CALCULATION

Wall Location =	Exterior		**Loadings**		
Species =	HF Stud		Roofing Material =	Shingle/Tile	
Stud Width =	1.5	in	Roof Pitch =	2.5	
Stud Depth (d_x) =	5.5	in	Angle =	11.8	
L =	15.5	ft	C_S =	1.000	
stud spacing =	1.33	ft	Increase for Drift=	1.000	
F_b =	675	psi	Effective snow load =	20	psf
F_c =	800	psi	Roof dead load =	25	psf
$F_{c\perp}$ =	405	psi	Floor live load =	20	psf
E =	1200000	psi	Floor dead load =	10	psf
E_{min} =	440000	psi	Trib. Area $_{roof}$=	18	ft
C_F =	1.00	for bending	Trib. Area $_{floor}$=	0	ft
C_F =	1.00	for comp. II to grain	Add. Uniform Load =	80	plf
A =	8.25	in^2			
S =	7.56	in^3	Lateral Load =	12.76	psf
I =	20.80	in^4			

Dead Loads:

Use: 2x6 HF Stud Grade @ 16" o.c.

Roof DL =	450	plf
Floor DL =	0	plf
w_{DL} =	530	plf
Live Loads:		
Roof LL =	360.0	plf
Floor LL =	0	plf
W_{LL}=	360.00	

Deflection:

Allowable Deflection =	L/360	
Allowable Deflection =	0.52	in
Deflection =	0.51	in
Check =	**OK**	

Load Case 1: Gravity Loads Only

Load Combinations:

D =	705	lbs
D+L =	705	lbs
D+S =	1184	lbs
D+0.75(L)+0.75(S) =	1064	lbs
C_D (D) =	0.9	
C_D (D+L) =	1	
C_D (D+S) =	1.15	
C_D (D+0.75(L)+0.75(S)) =	1.15	
$f_c = f_{c\perp}$ =	143.5	psi
$(l_e/d)_x$ =	33.8	in
E'_{min} =	440000	psi
c =	0.8	
F_{cE} =	316.2	
F^*_c =	920	psi
F_{cE}/F^*_c =	0.344	psi
$(1+F_{cE}/F^*_c)/2c$ =	0.840	
C_p =	0.315	
F'_c =	289.6	
Check =	**OK**	psi

Bearing of stud on wall plates:

C_b =	1.25	
$F'_{c\perp}$ =	506	
Check =	**OK**	psi

Load Case 2: Gravity Loads + Lateral Loads

C_D =	1.6	
C_r =	1.35	
w =	17.0	plf
M =	6113.7	in.lb
f_b =	808.4	psi
F'_b =	1458.00	psi
Check =	**OK**	
Axial:		
(l_e/d_x) =	33.8	in
E'_{min} =	440000	psi
c =	0.8	
F_{cE} =	316.2	psi
F^*_c =	1280	psi
F_{cE}/F^*_c =	0.247	
$(1+F_{cE}/F^*_c)/2c$ =	0.779	
C_p =	0.233	
F'_c =	298.1	psi

	D+0.75(W)+0.75(L)+0.75(S)	**D+W**	
f_c =	129.0	85.4	psi
Check =	**OK**	**OK**	
Combined Stress:			
F_{cEx} =	316.2	316.2	psi
Interaction Formula =	0.89	0.84	
Check =	**OK**	**OK**	

KING STUD CALCULATION

Species =	HF Stud	
Stud Width =	6	in
Stud Depth (d_x) =	5.5	in
L =	9	ft
opening width (OOP) =	9	ft
max. gravity span	9	ft
F_b =	675	psi
F_c =	800	psi
$F_{c\perp}$ =	405	psi
E =	1200000	psi
E_{min} =	440000	psi
C_F =	1.00	for bending
C_F =	1.00	for comp. II to grain
A =	33	in^2
S =	30.25	in^3
I =	83.19	in^4
Dead Loads:		
Roof DL =	450	plf
Floor DL =	0	plf
w_{DL} =	530	plf
Live Loads:		
Roof LL =	360.0	plf
Floor LL =	0	plf
W_{LL}=	360.00	
Load Case 1: Gravity Loads Only		
Load Combinations:		
D =	2740	lbs
D+L =	2740	lbs
D+S =	4601	lbs
D+0.75(L)+0.75(S) =	4136	lbs
C_D (D) =	0.9	
C_D (D+L) =	1	
C_D (D+S) =	1.15	
C_D (D+0.75(L)+0.75(S)) =	1.15	
$f_c = f_{c\perp}$ =	139.4	psi
$(l_e/d)_x$ =	19.6	in
E'_{min} =	440000	psi
c =	0.8	
F_{cE} =	938.0	
F^*_c =	920	psi
F_{cE}/F^*_c =	1.020	psi
$(1+F_{cE}/F^*_c)/2c$ =	1.262	
C_p =	0.698	
F'_c =	641.8	
Check =	**OK**	psi
Bearing of stud on wall plates:		
C_b =	1.06	
$F'_{c\perp}$ =	430	
Check =	**OK**	psi

Loadings

Roofing Material =	Shingle/Tile	
Roof Pitch =	2.5	
Angle =	11.8	
C_S =	1.000	
Increase for Drift=	1.000	
Effective snow load =	20	psf
Roof dead load =	25	psf
Floor live load =	20	psf
Floor dead load =	10	psf
Trib. Area $_{roof}$=	18	ft
Trib. Area $_{floor}$=	0	ft
Add. Uniform Load =	80	plf
Lateral Load =	12.76	psf

Use: (2) 2x6 Full Height King Studs

Deflection:		
Allowable Deflection =	L/360	
Allowable Deflection =	0.30	in
Deflection =	0.06	in
Check =	**OK**	
Load Case 2: Gravity Loads + Lateral Loads		
C_D =	1.6	
C_r =	1.15	
w =	65.9	plf
M =	8012.4	in.lb
f_b =	264.9	psi
F'_b =	1242.00	psi
Check =	**OK**	
Axial:		
(l_e/d_x) =	19.6	in
E'_{min} =	440000	psi
c =	0.8	
F_{cE} =	938.0	psi
F^*_c =	1280	psi
F_{cE}/F^*_c =	0.733	
$(1+F_{cE}/F^*_c)/2c$ =	1.083	
C_p =	0.576	
F'_c =	737.5	psi

	D+0.75(W)+0.75(L)+0.75(S)	D+W	
f_c =	125.3	83.0	psi
Check =	**OK**	**OK**	
Combined Stress:			
F_{cEx} =	938.0	938.0	psi
Interaction Formula =	0.21	0.25	
Check =	**OK**	**OK**	

Footing(s)

	FT1A	FT1B	FT1C	FT1D	FT1E	FT1F-FT1H	FT2
Width of footing (in)=	12	12	12	18	30	See attached Calculations	12
Depth of footing (in)=	20	14	30	20	20		16
Height of wall (in)=	0	16	0	0	0		0
Width of wall (in)=	6	7	8	8	8		6
Roofing Material =	Shingle/Tile	Shingle/Tile	Shingle/Tile	Shingle/Tile	Shingle/Tile		Shingle/Tile
Roof Pitch=	2.5	2.5	2.5	2.5	2.5		2.5
Angle=	11.8	11.8	11.8	11.8	11.8		11.8
C_S=	1.000	1.000	1.000	1.000	1.000		1.000
Increase for Drift/Valley=	1.000	1.000	1.000	1.000	1.000		1.000
Effective snow load (psf)=	20	20	20	20	20		20
Roof dead load (psf)=	25	25	25	25	25		25
Floor live load (psf)=	20	20	20	20	20		20
Floor dead load (psf)=	10	10	10	10	10		10
Trib. $Area_{ROOF}$=	14	16.5	16.5	4.5	2		20.5
Trib. $Area_{FLOOR2}$=	0	0	0	0	0		0
Trib. $Area_{FLOOR1}$=	0	0	0	0	0		5
w_S (plf)=	280	330	330	90	40		410
w_L (plf)=	0	0	0	0	0		100
w_D (plf)=	350	412.5	412.5	112.5	50		562.5
$w_{CONC.}$ (plf)=	250	292	375	375	625		200
$w_{ADDITIONAL}$ (plf)=	140	160	160	150	150		160
w_{TOTAL} (plf)=	1020	1194	1278	728	865		1333
ecc.	2.92						
	O.K.						
Req.Soil Bearing (psf)=	**1020**	**1194**	**1278**	**485**	**346**		**1333**
Footing Reinforcement:	**(2) #4 bars cont.**	**(2) #4 bars cont.**	**(4) #4 bars cont.**	**(4) #4 bars cont.**	**(6) #4 bars cont.**		**(2) #4 bars cont.**
Crosswise Reinforcement:	**None**	**None**	**None**	**None**	**#4 bars @ 12" o.c.**		**None**

Combined Footing

Project File: 2025-2039.ec6

DESCRIPTION: FT1F (WSWH FOOTING - GL 3)

Code References

Calculations per ACI 318-14, IBC 2018, CBC 2019, ASCE 7-16
Load Combinations Used : ASCE 7-16

General Information

Material Properties

f'c : Concrete 28 day strength		2.50	ksi
fy : Rebar Yield		60.0	ksi
Ec : Concrete Elastic Modulus		3,122.0	ksi
Concrete Density		145.0	pcf
ϕ : Phi Values	Flexure :	0.90	
	Shear :	0.750	

Analysis/Design Settings

Calculate footing weight as dead load ?	Yes
Calculate Pedestal weight as dead load ?	No
Min Steel % Bending Reinf (based on 'd')	
Min Allow % Temp Reinf (based on thick)	0.00180
Min. Overturning Safety Factor	1.0: 1
Min. Sliding Safety Factor	1.0: 1

Soil Information

Allowable Soil Bearing	1.50	ksf
Increase Bearing By Footing Weight	No	
Soil Passive Sliding Resistance	250.0	pcf
(Uses entry for "Footing base depth below soil surface" for force)		
Coefficient of Soil/Concrete Friction	0.30	

Soil Bearing Increase

Footing base depth below soil surface		ft
Increases based on footing Depth		
Allowable pressure increase per foot		ksf
when base of footing is below		ft
Increases based on footing Width . . .		
Allowable pressure increase per foot		ksf
when maximum length or width is greater tha		ft
Maximum Allowed Bearing Pressure	10.0	ksf
(A value of zero implies no limit)		
Adjusted Allowable Soil Bearing	1.50	ksf

(Allowable Soil Bearing adjusted for footing weight and depth & width increases as specified by user.)

Dimensions & Reinforcing

Distance Left of Column #1	=	1.0	ft
Between Columns	=	20.0	ft
Distance Right of Column #2	=	1.0	ft
Total Footing Length	=	22.0	ft
Footing Width	=	2.0	ft
Footing Thickness	=	14.0	in
Rebar Center to Concrete Edge @ Top	=	3.0	in
Rebar Center to Concrete Edge @ Bottom	=	3.0	in

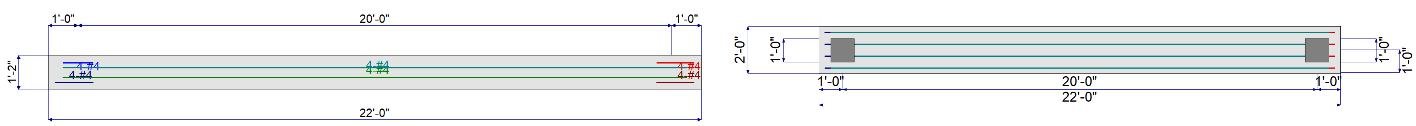
Pedestal dimensions...

		Col #1	Col #2	
Sq. Dim.	=	12.0	12.0	in
Height	=			in

	Count	Size #	As Provided	As Req'd
Bars left of Col #1				
Bottom Bars	4.0	4	0.80	0.6048 in^2
Top Bars	4.0	4	0.80	0.6048 in^2
Bars Btwn Cols				
Bottom Bars	4.0	4	0.80	0.6048 in^2
Top Bars	4.0	4	0.80	0.6048 in^2
Bars Right of Col #2				
Bottom Bars	4.0	4	0.80	0.6048 in^2
Top Bars	4.0	4	0.80	0.6048 in^2

Applied Loads

		D	Lr	L	S	W	E	H	
Applied @ Left Column									
Axial Load Downward	=	0.7520	0.540						k
Moment (+CW)	=					9.058	2.021		k-ft
Shear (+X)	=								k
Applied @ Right Column									
Axial Load Downward	=	0.7510	0.540						k
Moment (+CW)	=					9.058	2.021		k-ft
Shear (+X)	=								k
Overburden	=	0.050							



Combined Footing

Project File: 2025-2039.ec6

LIC# : KW-06014416, Build:20.23.08.30 LEI CONSULTING ENGINEERS

DESCRIPTION: FT1F (WSWH FOOTING - GL 3)

DESIGN SUMMARY

Design OK

	Factor of Safety	Item	Applied	Capacity	Governing Load Combination
PASS	6.769	Overturning	10.870 k-ft	73.572 k-ft	+0.60D+0.60W
PASS	No Sliding	Sliding	0.0 k	3.314 k	No Sliding
PASS	No Uplift	Uplift	0.0 k	0.0 k	No Uplift

	Utilization Ratio	Item	Applied	Capacity	Governing Load Combination
PASS	0.2147	Soil Bearing	0.3221 ksf	1.50 ksf	+D+0.750Lr+0.450W
PASS	0.07867	1-way Shear - Col #1	5.90 psi	75.0 psi	+1.20D+1.60Lr+0.50W
PASS	0.06643	1-way Shear - Col #2	4.982 psi	75.0 psi	+1.20D+1.60Lr
PASS	0.01046	2-way Punching - Col #1	1.568 psi	150.0 psi	+1.20D+1.60Lr+0.50W
PASS	0.009212	2-way Punching - Col #2	1.382 psi	150.0 psi	+1.20D+1.60Lr
PASS	0.001233	Flexure - Left of Col #1 - Top	-0.04674 k-ft	37.906 k-ft	+1.20D+W
PASS	No Bending	Flexure - Left of Col #1 - Bottom	0.0 k-ft	0.0 k-ft	N/A
PASS	0.2691	Flexure - Between Cols - Top	-10.202 k-ft	37.906 k-ft	+1.20D+0.50Lr+W
PASS	0.2234	Flexure - Between Cols - Bottom	8.467 k-ft	37.906 k-ft	+0.90D+W
PASS	0.000606	Flexure - Right of Col #2 - Top	-0.02295 k-ft	37.906 k-ft	+1.40D
PASS	0.000326	Flexure - Right of Col #2 - Bottom	0.01235 k-ft	37.906 k-ft	+0.90D+W

Combined Footing

Project File: 2025-2039.ec6

LIC# : KW-06014416, Build:20.23.08.30 LEI CONSULTING ENGINEERS

DESCRIPTION: FT1G (WSWH FOOTING - GL 7)

Code References

Calculations per ACI 318-14, IBC 2018, CBC 2019, ASCE 7-16
Load Combinations Used : ASCE 7-16

General Information

Material Properties

f'c : Concrete 28 day strength		2.50	ksi
fy : Rebar Yield		60.0	ksi
Ec : Concrete Elastic Modulus		3,122.0	ksi
Concrete Density		145.0	pcf
ϕ : Phi Values	Flexure :	0.90	
	Shear :	0.750	

Analysis/Design Settings

Calculate footing weight as dead load ?	Yes
Calculate Pedestal weight as dead load ?	No
Min Steel % Bending Reinf (based on 'd')	
Min Allow % Temp Reinf (based on thick)	0.00180
Min. Overturning Safety Factor	1.0: 1
Min. Sliding Safety Factor	1.0: 1

Soil Information

Allowable Soil Bearing	1.50	ksf
Increase Bearing By Footing Weight	No	
Soil Passive Sliding Resistance	250.0	pcf
(Uses entry for "Footing base depth below soil surface" for force)		
Coefficient of Soil/Concrete Friction	0.30	

Soil Bearing Increase

Footing base depth below soil surface		ft
Increases based on footing Depth		
Allowable pressure increase per foot		ksf
when base of footing is below		ft
Increases based on footing Width . . .		
Allowable pressure increase per foot		ksf
when maximum length or width is greater tha		ft
Maximum Allowed Bearing Pressure	10.0	ksf
(A value of zero implies no limit)		
Adjusted Allowable Soil Bearing	1.50	ksf

(Allowable Soil Bearing adjusted for footing weight and depth & width increases as specified by user.)

Dimensions & Reinforcing

Distance Left of Column #1	=	1.0	ft
Between Columns	=	18.0	ft
Distance Right of Column #2	=	2.0	ft
Total Footing Length	=	21.0	ft
Footing Width	=	2.0	ft
Footing Thickness	=	14.0	in
Rebar Center to Concrete Edge @ Top	=	3.0	in
Rebar Center to Concrete Edge @ Bottom	=	3.0	in

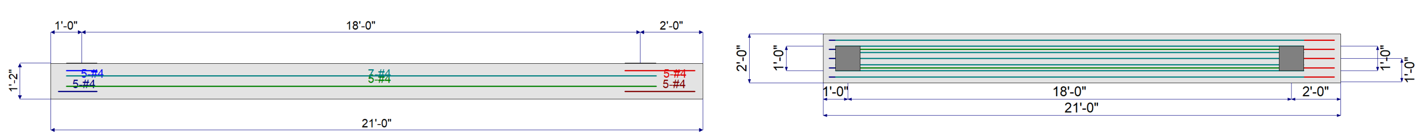
Pedestal dimensions...

		Col #1	Col #2	
Sq. Dim.	=	12.0	12.0	in
Height	=			in

	Count	Size #	As Provided	As Req'd
Bars left of Col #1				
Bottom Bars	5.0	4	1.0	0.6048 in^2
Top Bars	5.0	4	1.0	0.0 in^2
Bars Btwn Cols				
Bottom Bars	5.0	4	1.0	0.6048 in^2
Top Bars	7.0	4	1.40	1.220 in^2
Bars Right of Col #2				
Bottom Bars	5.0	4	1.0	0.6048 in^2
Top Bars	5.0	4	1.0	0.0 in^2

Applied Loads

		D	Lr	L	S	W	E	H	
Applied @ Left Column									
Axial Load Downward	=	4.862	3.440						k
Moment (+CW)	=					15.633	5.310		k-ft
Shear (+X)	=								k
Applied @ Right Column									
Axial Load Downward	=	8.244	6.047						k
Moment (+CW)	=					15.633	5.310		k-ft
Shear (+X)	=								k
Overburden	=	0.050							



Combined Footing

Project File: 2025-2039.ec6

LIC# : KW-06014416, Build:20.23.08.30 LEI CONSULTING ENGINEERS

DESCRIPTION: FT1G (WSWH FOOTING - GL 7)

DESIGN SUMMARY

Design OK

	Factor of Safety	Item	Applied	Capacity	Governing Load Combination
PASS	6.729	Overturning	18.760 k-ft	126.228 k-ft	+0.60D+0.60W
PASS	No Sliding	Sliding	0.0 k	6.663 k	No Sliding
PASS	No Uplift	Uplift	0.0 k	0.0 k	No Uplift

	Utilization Ratio	Item	Applied	Capacity	Governing Load Combination
PASS	0.7023	Soil Bearing	1.053 ksf	1.50 ksf	+D+0.750Lr+0.450W
PASS	0.4992	1-way Shear - Col #1	37.439 psi	75.0 psi	+1.20D+1.60Lr+0.50W
PASS	0.6166	1-way Shear - Col #2	46.245 psi	75.0 psi	+1.20D+1.60Lr
PASS	0.06775	2-way Punching - Col #1	10.162 psi	150.0 psi	+1.20D+1.60Lr+0.50W
PASS	0.04916	2-way Punching - Col #2	7.374 psi	150.0 psi	+1.20D+1.60Lr
PASS	0.000822	Flexure - Left of Col #1 - Top	-0.03851 k-ft	46.853 k-ft	+0.90D+W
PASS	0.001001	Flexure - Left of Col #1 - Bottom	0.04690 k-ft	46.853 k-ft	+1.20D+1.60Lr
PASS	0.8806	Flexure - Between Cols - Top	-56.456 k-ft	64.112 k-ft	+1.20D+1.60Lr+0.50W
PASS	0.2808	Flexure - Between Cols - Bottom	13.158 k-ft	46.853 k-ft	+0.90D+W
PASS	No Bending	Flexure - Right of Col #2 - Top	0.0 k-ft	0.0 k-ft	N/A
PASS	0.05435	Flexure - Right of Col #2 - Bottom	2.547 k-ft	46.853 k-ft	+1.20D+1.60Lr+0.50W

Combined Footing

Project File: 2025-2039.ec6

LIC# : KW-06014416, Build:20.23.08.30 LEI CONSULTING ENGINEERS (c) ENERCALC INC 1983-2023

DESCRIPTION: FT1H (WSWH FOOTING - GL 1)

Code References

Calculations per ACI 318-14, IBC 2018, CBC 2019, ASCE 7-16
Load Combinations Used : ASCE 7-16

General Information

Material Properties

f'c : Concrete 28 day strength		2.50	ksi
fy : Rebar Yield		60.0	ksi
Ec : Concrete Elastic Modulus		3,122.0	ksi
Concrete Density		145.0	pcf
ϕ : Phi Values	Flexure :	0.90	
	Shear :	0.750	

Analysis/Design Settings

Calculate footing weight as dead load ?	Yes
Calculate Pedestal weight as dead load ?	No
Min Steel % Bending Reinf (based on 'd')	
Min Allow % Temp Reinf (based on thick)	0.00180
Min. Overturning Safety Factor	1.0: 1
Min. Sliding Safety Factor	1.0: 1

Soil Information

Allowable Soil Bearing	1.50	ksf
Increase Bearing By Footing Weight	No	
Soil Passive Sliding Resistance	250.0	pcf
(Uses entry for "Footing base depth below soil surface" for force)		
Coefficient of Soil/Concrete Friction	0.30	

Soil Bearing Increase

Footing base depth below soil surface		ft
Increases based on footing Depth		
Allowable pressure increase per foot		ksf
when base of footing is below		ft
Increases based on footing Width . . .		
Allowable pressure increase per foot		ksf
when maximum length or width is greater tha		ft
Maximum Allowed Bearing Pressure	10.0	ksf
(A value of zero implies no limit)		
Adjusted Allowable Soil Bearing	1.50	ksf

(Allowable Soil Bearing adjusted for footing weight and depth & width increases as specified by user.)

Dimensions & Reinforcing

Distance Left of Column #1	=	1.50	ft
Between Columns	=	18.0	ft
Distance Right of Column #2	=	1.50	ft
Total Footing Length	=	21.0	ft
Footing Width	=	2.50	ft
Footing Thickness	=	16.0	in
Rebar Center to Concrete Edge @ Top	=	3.0	in
Rebar Center to Concrete Edge @ Bottom	=	3.0	in

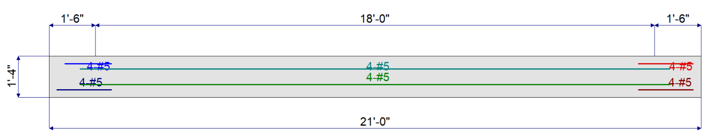
Pedestal dimensions...

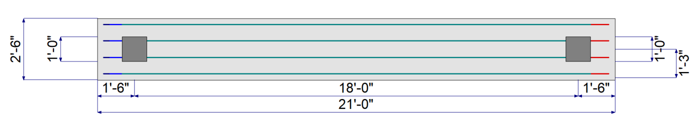
		Col #1	Col #2	
Sq. Dim.	=	12.0	12.0	in
Height	=			in

	Count	Size #	As Provided	As Req'd
Bars left of Col #1				
Bottom Bars	4.0	5	1.240	0.8640 in^2
Top Bars	4.0	5	1.240	0.8640 in^2
Bars Btwn Cols				
Bottom Bars	4.0	5	1.240	0.9921 in^2
Top Bars	4.0	5	1.240	1.0 in^2
Bars Right of Col #2				
Bottom Bars	4.0	5	1.240	0.8640 in^2
Top Bars	4.0	5	1.240	0.8640 in^2

Applied Loads

		D	Lr	L	S	W	E	H	
Applied @ Left Column									
Axial Load Downward	=	1.429	0.960						k
Moment (+CW)	=					44.275	6.640		k-ft
Shear (+X)	=								k
Applied @ Right Column									
Axial Load Downward	=	1.429	0.960						k
Moment (+CW)	=					44.275	6.640		k-ft
Shear (+X)	=								k
Overburden	=	0.050							





Combined Footing

Project File: 2025-2039.ec6

LIC# : KW-06014416, Build:20.23.08.30 LEI CONSULTING ENGINEERS (c) ENERCALC INC 1983-2023

DESCRIPTION: FT1H (WSWH FOOTING - GL 1)

DESIGN SUMMARY

Design OK

	Factor of Safety	Item	Applied	Capacity	Governing Load Combination
PASS	1.854	Overturning	53.130 k-ft	98.488 k-ft	+0.60D+0.60W
PASS	No Sliding	Sliding	0.0 k	4.660 k	No Sliding
PASS	No Uplift	Uplift	0.0 k	0.0 k	No Uplift

	Utilization Ratio	Item	Applied	Capacity	Governing Load Combination
PASS	0.3908	Soil Bearing	0.5862 ksf	1.50 ksf	+D+0.60W
PASS	0.1499	1-way Shear - Col #1	11.243 psi	75.0 psi	+1.20D+0.50Lr+W
PASS	0.1070	1-way Shear - Col #2	8.026 psi	75.0 psi	+0.90D+W
PASS	0.01831	2-way Punching - Col #1	2.747 psi	150.0 psi	+1.20D+1.60Lr+0.50W
PASS	0.01382	2-way Punching - Col #2	2.073 psi	150.0 psi	+1.20D+1.60Lr
PASS	0.006106	Flexure - Left of Col #1 - Top	-0.4230 k-ft	69.284 k-ft	+1.20D+W
PASS	0.001359	Flexure - Left of Col #1 - Bottom	0.09418 k-ft	69.284 k-ft	+1.20D+1.60Lr
PASS	0.6161	Flexure - Between Cols - Top	-42.685 k-ft	69.284 k-ft	+1.20D+0.50Lr+W
PASS	0.6113	Flexure - Between Cols - Bottom	42.356 k-ft	69.284 k-ft	+0.90D+W
PASS	0.000186	Flexure - Right of Col #2 - Top	-0.01286 k-ft	69.284 k-ft	+1.40D
PASS	0.01101	Flexure - Right of Col #2 - Bottom	0.7629 k-ft	69.284 k-ft	+0.90D+W

FT3 calcs:

Properties:

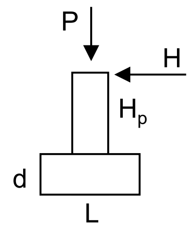
f_c=	2500	psi
f_y=	40000	psi
q_u=	1500	psf
H_p=	0.001	in
d=	12	in

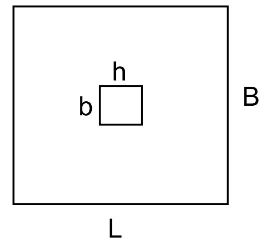
Loadings:

P=	6	kip
H=	0	kip
M=	0.1	kft
Uplift=	0.1	kip

Factored Loadings:

P_u=	9.6	kip
H_u=	0	kip





Moment:

M_1=	0.1	kft
M_u=	0.16	kft

Footing Design:

B=	2	
L=	2	ft

Footing Weight:

$w_{footing}$=	0.60	kip
w_{soil}=	0.26	kip
w_{other}=	0	kip
w_{total}=	0.86	kip
P_{total}=	6.86	kip

Stability Ratios:

M_{OT}=	0.1	kft
M_{resist}=	6.9	kft
SF=	68.6	**OKAY**
Uplift SF=	8.6	**OKAY**

Eccentricity:

e=	0.01	**Inside Third**
L/6=	0.333	

Inside Third:

d_{eff}=	8.5	in	
a=	0.25	ft	
q_{min}=	1.43	ksf	
q_{max}=	1.57	ksf	
q_{allow}=	2.00	ksf	**Soil Bearing Okay**

One Way Shear:

q_{crit}=	1.55	ksf	
V_{u1}=	0.78	kip	
ϕV_{c1}=	17.3	kip	**One Way Shear Okay**

Two Way Shear:

q_1=	1.451	ksf	
V_{u2}=	2.63	kip	
b_o=	6.00	ft	
ϕV_{c2}=	104.04	kip	**Two Way Shear Okay**

Footing Reinforcement:

q_f=	1.50273183	ksf
M_{max}=	0.79	kft
m=	18.82	
R_n=	10.9079826	
ρ =	0.0033	
A_s=	0.337	in^2

Use (3) #4 bars each way

FT4 calcs:

Properties:

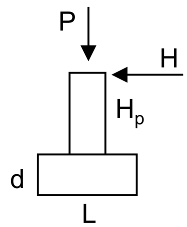
f_c=	2500	psi
f_y=	40000	psi
q_u=	1500	psf
H_p=	0.001	in
d=	12	in

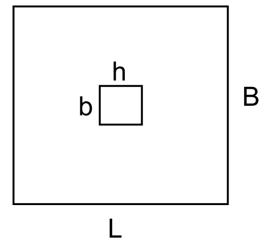
Loadings:

P=	9.375	kip
H=	0	kip
M=	0.1	kft
Uplift=	0.1	kip

Factored Loadings:

P_u=	15	kip
H_u=	0	kip





Moment:

M_1=	0.1	kft
M_u=	0.16	kft

Footing Design:

B=	2.5	
L=	2.5	ft

Footing Weight:

$w_{footing}$=	0.94	kip
w_{soil}=	0.41	kip
w_{other}=	0	kip
w_{total}=	1.35	kip
P_{total}=	10.72	kip

Stability Ratios:

M_{OT}=	0.1	kft
M_{resist}=	13.4	kft
SF=	134.1	**OKAY**
Uplift SF=	13.5	**OKAY**

Eccentricity:

e=	0.01	**Inside Third**
L/6=	0.417	

Inside Third:

d_{eff}=	8.5	in	
a=	0.50	ft	
q_{min}=	1.47	ksf	
q_{max}=	1.53	ksf	
q_{allow}=	2.00	ksf	**Soil Bearing Okay**

One Way Shear:

q_{crit}=	1.52	ksf	
V_{u1}=	1.91	kip	
ϕV_{c1}=	21.7	kip	**One Way Shear Okay**

Two Way Shear:

q_1=	1.480	ksf	
V_{u2}=	6.00	kip	
b_o=	6.00	ft	
ϕV_{c2}=	104.04	kip	**Two Way Shear Okay**

Footing Reinforcement:

q_f=	1.50111893	ksf
M_{max}=	1.24	kft
m=	18.82	
R_n=	17.0959094	
ρ =	0.0033	
A_s=	0.337	in^2

Use (4) #4 bars each way

FT5 calcs:

Properties:

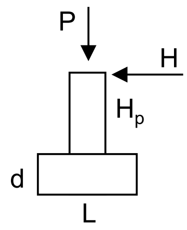
f_c=	2500	psi
f_y=	40000	psi
q_u=	1500	psf
H_p=	0.001	in
d=	12	in

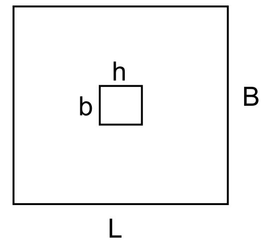
Loadings:

P=	13.5	kip
H=	0	kip
M=	0.1	kft
Uplift=	0.1	kip

Factored Loadings:

P_u=	21.6	kip
H_u=	0	kip





Moment:

M_1=	0.1	kft
M_u=	0.16	kft

Footing Design:

B=	3	
L=	3	ft

Footing Weight:

$w_{footing}$=	1.35	kip
w_{soil}=	0.59	kip
w_{other}=	0	kip
w_{total}=	1.94	kip
P_{total}=	15.44	kip

Stability Ratios:

M_{OT}=	0.1	kft
M_{resist}=	23.2	kft
SF=	231.7	**OKAY**
Uplift SF=	19.4	**OKAY**

Eccentricity:

e=	0.01	**Inside Third**
L/6=	0.500	

Inside Third:

d_{eff}=	8.5	in	
a=	0.75	ft	
q_{min}=	1.48	ksf	
q_{max}=	1.52	ksf	
q_{allow}=	2.00	ksf	**Soil Bearing Okay**

One Way Shear:

q_{crit}=	1.51	ksf	
V_{u1}=	3.41	kip	
ϕV_{c1}=	26.0	kip	**One Way Shear Okay**

Two Way Shear:

q_1=	1.490	ksf	
V_{u2}=	10.13	kip	
b_o=	6.00	ft	
ϕV_{c2}=	104.04	kip	**Two Way Shear Okay**

Footing Reinforcement:

q_f=	1.5005396	ksf
M_{max}=	1.79	kft
m=	18.82	
R_n=	24.7445503	
ρ =	0.0033	
A_s=	0.337	in^2

Use (4) #4 bars each way

FT6 calcs:

Properties:

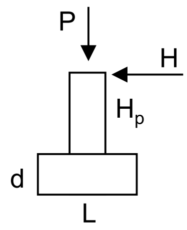
f_c=	2500	psi
f_y=	40000	psi
q_u=	1500	psf
H_p=	0.001	in
d=	12	in

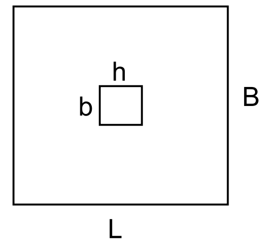
Loadings:

P=	18.375	kip
H=	0	kip
M=	0.1	kft
Uplift=	0.1	kip

Factored Loadings:

P_u=	29.4	kip
H_u=	0	kip





Moment:

M_1=	0.1	kft
M_u=	0.16	kft

Footing Design:

B=	3.5	
L=	3.5	ft

Footing Weight:

$w_{footing}$=	1.84	kip
w_{soil}=	0.81	kip
w_{other}=	0	kip
w_{total}=	2.65	kip
P_{total}=	21.02	kip

Stability Ratios:

M_{OT}=	0.1	kft
M_{resist}=	36.8	kft
SF=	367.9	**OKAY**
Uplift SF=	26.5	**OKAY**

Eccentricity:

e=	0.00	**Inside Third**
L/6=	0.583	

Inside Third:

d_{eff}=	8.5	in	
a=	1.00	ft	
q_{min}=	1.49	ksf	
q_{max}=	1.51	ksf	
q_{allow}=	2.00	ksf	**Soil Bearing Okay**

One Way Shear:

q_{crit}=	1.51	ksf	
V_{u1}=	5.28	kip	
ϕV_{c1}=	30.3	kip	**One Way Shear Okay**

Two Way Shear:

q_1=	1.495	ksf	
V_{u2}=	15.00	kip	
b_o=	6.00	ft	
ϕV_{c2}=	104.04	kip	**Two Way Shear Okay**

Footing Reinforcement:

q_f=	1.50029126	ksf
M_{max}=	2.45	kft
m=	18.82	
R_n=	33.8461201	
ρ =	0.0033	
A_s=	0.337	in^2

Use (5) #4 bars each way

FT7 calcs:

Properties:

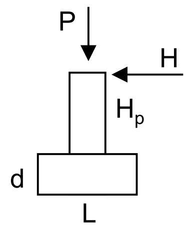
f_c=	2500	psi
f_y=	40000	psi
q_u=	1500	psf
H_p=	0.001	in
d=	12	in

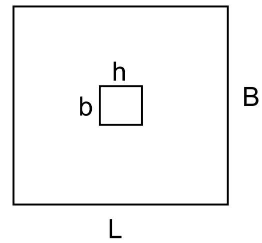
Loadings:

P=	24	kip
H=	0	kip
M=	0.1	kft
Uplift=	0.1	kip

Factored Loadings:

P_u=	38.4	kip
H_u=	0	kip





Moment:

M_1=	0.1	kft
M_u=	0.16	kft

Footing Design:

B=	4	
L=	4	ft

Footing Weight:

$w_{footing}$=	2.40	kip
w_{soil}=	1.06	kip
w_{other}=	0	kip
w_{total}=	3.46	kip
P_{total}=	27.46	kip

Stability Ratios:

M_{OT}=	0.1	kft
M_{resist}=	54.9	kft
SF=	549.1	**OKAY**
Uplift SF=	34.6	**OKAY**

Eccentricity:

e=	0.00	**Inside Third**
L/6=	0.667	

Inside Third:

d_{eff}=	8.5	in	
a=	1.25	ft	
q_{min}=	1.49	ksf	
q_{max}=	1.51	ksf	
q_{allow}=	2.00	ksf	**Soil Bearing Okay**

One Way Shear:

q_{crit}=	1.50	ksf	
V_{u1}=	7.53	kip	
ϕV_{c1}=	34.7	kip	**One Way Shear Okay**

Two Way Shear:

q_1=	1.497	ksf	
V_{u2}=	20.63	kip	
b_o=	6.00	ft	
ϕV_{c2}=	104.04	kip	**Two Way Shear Okay**

Footing Reinforcement:

q_f=	1.50017073	ksf
M_{max}=	3.21	kft
m=	18.82	
R_n=	44.3965771	
ρ =	0.0033	
A_s=	0.337	in^2

Use (6) #4 bars each way

General Footing

Project File: 2025-2039.ec6

LIC# : KW-06014416, Build:20.23.08.30 LEI CONSULTING ENGINEERS

DESCRIPTION: FT8

Code References

Calculations per ACI 318-14, IBC 2018, CBC 2019, ASCE 7-16
Load Combinations Used : ASCE 7-16

General Information

Material Properties

f'c : Concrete 28 day strength	=	2.50	ksi
fy : Rebar Yield	=	60.0	ksi
Ec : Concrete Elastic Modulus	=	3,122.0	ksi
Concrete Density	=	145.0	pcf
φ Values Flexure	=	0.90	
Shear	=	0.750	

Analysis Settings

Min Steel % Bending Reinf.	=		
Min Allow % Temp Reinf.	=	0.00180	
Min. Overturning Safety Factor	=	1.0	: 1
Min. Sliding Safety Factor	=	1.0	: 1
Add Ftg Wt for Soil Pressure	:	Yes	
Use ftg wt for stability, moments & shears	:	Yes	
Add Pedestal Wt for Soil Pressure	:	No	
Use Pedestal wt for stability, mom & shear	:	No	

Soil Design Values

Allowable Soil Bearing	=	1.50	ksf
Soil Density	=	110.0	pcf
Increase Bearing By Footing Weight	=	No	
Soil Passive Resistance (for Sliding)	=	250.0	pcf
Soil/Concrete Friction Coeff.	=	0.30	

Increases based on footing Depth

Footing base depth below soil surface	=		ft
Allow press. increase per foot of depth	=		ksf
when footing base is below	=		ft

Increases based on footing plan dimension

Allowable pressure increase per foot of depth	=		ksf
when max. length or width is greater than	=		ft

Dimensions

Width parallel to X-X Axis	=	3.0	ft
Length parallel to Z-Z Axis	=	5.0	ft
Footing Thickness	=	12.0	in

Load location offset from footing center...

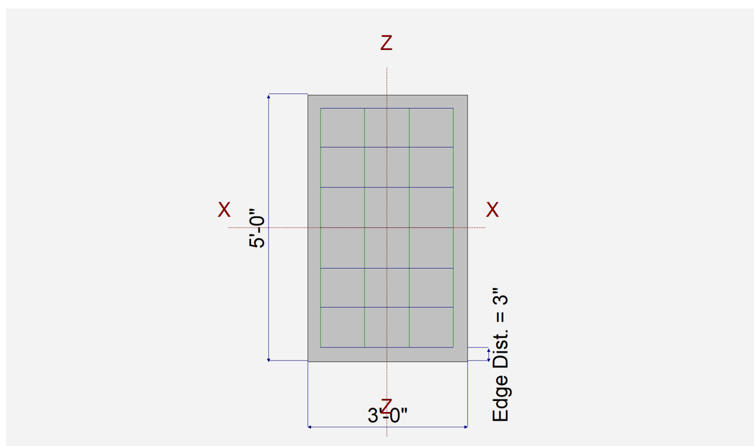
ex : Prll to X-X Axis	=	11	in
	=		in

Pedestal dimensions...

px : parallel to X-X Axis	=		in
pz : parallel to Z-Z Axis	=		in
Height	=		in

Rebar Centerline to Edge of Concrete...

at Bottom of footing	=	3.0	in



Reinforcing

Bars parallel to X-X Axis

Number of Bars	=		7
Reinforcing Bar Size	=	#	4

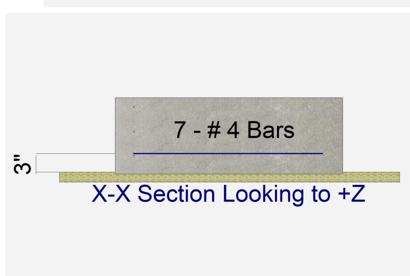
Bars parallel to Z-Z Axis

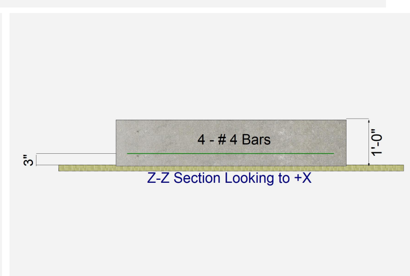
Number of Bars	=		4
Reinforcing Bar Size	=	#	4

Bandwidth Distribution Check (ACI 15.4.4.2)

Direction Requiring Closer Separation

	Bars along X-X Axis
# Bars required within zone	75.0 %
# Bars required on each side of zone	25.0 %





Applied Loads

		D	Lr	L	S	W	E	H	
P : Column Load	=	3.858	2.728						k
OB : Overburden	=								ksf
M-xx	=								k-ft
M-zz	=								k-ft
V-x	=								k
V-z	=								k

General Footing

Project File: 2025-2039.ec6

LIC# : KW-06014416, Build:20.23.08.30 LEI CONSULTING ENGINEERS

DESCRIPTION: FT8

DESIGN SUMMARY

Design OK

	Min. Ratio	Item	Applied		Capacity		Governing Load Combination
PASS	0.9547	Soil Bearing	1.432	ksf	1.50	ksf	+D+Lr about Z-Z axis
PASS	n/a	Overturning - X-X	0.0	k-ft	0.0	k-ft	No Overturning
PASS	n/a	Overturning - Z-Z	0.0	k-ft	0.0	k-ft	No Overturning
PASS	n/a	Sliding - X-X	0.0	k	0.0	k	No Sliding
PASS	n/a	Sliding - Z-Z	0.0	k	0.0	k	No Sliding
PASS	n/a	Uplift	0.0	k	0.0	k	No Uplift
PASS	0.02529	Z Flexure (+X)	0.2763	k-ft/ft	10.925	k-ft/ft	+1.20D+1.60Lr
PASS	0.02530	Z Flexure (-X)	0.2765	k-ft/ft	10.925	k-ft/ft	+1.20D+1.60Lr
PASS	0.1798	X Flexure (+Z)	1.874	k-ft/ft	10.424	k-ft/ft	+1.20D+1.60Lr
PASS	0.1798	X Flexure (-Z)	1.874	k-ft/ft	10.424	k-ft/ft	+1.20D+1.60Lr
PASS	n/a	1-way Shear (+X)	0.0	psi	75.0	psi	n/a
PASS	0.02006	1-way Shear (-X)	1.505	psi	75.0	psi	+1.20D+1.60Lr
PASS	0.1295	1-way Shear (+Z)	9.716	psi	75.0	psi	+1.20D+1.60Lr
PASS	0.1295	1-way Shear (-Z)	9.716	psi	75.0	psi	+1.20D+1.60Lr
PASS	0.1789	2-way Punching	26.836	psi	150.0	psi	+1.20D+1.60Lr

Post Calculations

	2"-4" Thick Timber DF-L#2	5"x5"and Larger Timber DF-L#2	Parallam	Glulam Comb #4	
F_c	1350	700	2500	2100	psi
F_{bx}	900	750	2400	1900	psi
F_{by}	900	750	2400	2200	psi
E_x	1600000	1300000	1800000	1900000	psi
E_y	1600000	1300000	1800000	1900000	psi

Example Calculations:

Post	lb Max P	ft l	ft le_x	ft le_y	in e_x	in e_y	C_d	$(le/d)_x$	$(le/d)_y$	A	S_x	S_y	f_c	F'c	F'_{bx}	F'_{by}	Comb.	Check
(2) 2x4	3725	8	8	1	0.61	0.00	1.15	27.4	4.0	10.5	6	5	355	582	1551	1708	0.6	**OK**
(2) 2x6	8990	8	8	1	0.96	0.00	1.15	17.5	4.0	16.5	15	8	545	1013	1344	1547	0.7	**OK**
(3) 2x4	5805	8	8	1	0.61	0.00	1.15	27.4	2.7	15.75	9	12	369	582	1785	1964	0.6	**OK**
(3) 2x6	14295	8	8	1	0.96	0.00	1.15	17.5	2.7	24.75	23	19	578	1019	1547	1779	0.7	**OK**
(4) 2x4	7745	8	8	1	0.61	0.00	1.15	27.4	2.0	21	12	21	369	582	1785	1964	0.6	**OK**
(4) 2x6	19080	8	8	1	0.96	0.00	1.15	17.5	2.0	33	30	33	578	1022	1547	1779	0.7	**OK**
(5) 2x4	9680	8	8	1	0.61	0.00	1.15	27.4	1.6	26.25	15	33	369	582	1785	1964	0.6	**OK**
(5) 2x6	23860	8	8	1	0.96	0.00	1.15	17.5	1.6	41.25	38	52	578	1023	1547	1779	0.7	**OK**
4x4	4340	8	8	1	0.61	0.00	1.15	27.4	3.4	12.25	7	7	354	571	1034	1035	0.7	**OK**
6x6	11200	8	8	1	0.96	0.00	1.15	17.5	2.2	30.25	28	28	370	663	862	863	0.8	**OK**
3 1/2" x 3 1/2" PLP	7440	8	1	8	0.00	0.61	1.15	3.4	27.4	12.25	7	7	607	953	3171	3174	1.0	**OK**
3 1/2" x 5 1/4" PLP	11035	8	1	8	0.00	0.61	1.15	2.3	27.4	18.375	16	11	601	953	3032	3036	1.0	**OK**
5 1/4" x 5 1/4" PLP	27915	8	1	8	0.00	0.92	1.15	2.3	18.3	27.563	24	24	1013	1889	3034	3036	1.0	**OK**
3 1/8" x 7 1/2" GLP	11495	8	1	8	0.00	0.55	1.15	1.6	30.7	23.438	29	12	490	802	2181	2935	0.9	**OK**
3 1/8" x 9" GLP	13790	8	1	8	0.00	0.55	1.15	1.3	30.7	28.125	42	15	490	802	2180	2935	0.9	**OK**
5 1/8" x 6" GLP	26595	8	1	8	0.00	0.90	1.15	2.0	18.7	30.75	31	26	865	1773	2184	2783	0.8	**OK**
5 1/8" x 7 1/2" GLP	33240	8	1	8	0.00	0.90	1.15	1.6	18.7	38.438	48	33	865	1773	2184	2783	0.8	**OK**
5 1/8" x 9" GLP	39890	8	1	8	0.00	0.90	1.15	1.3	18.7	46.125	69	39	865	1773	2183	2783	0.8	**OK**

Additional Post Calculations:

	0	8	8	8	0.61	0.61	1.15	27.4	27.4	12.25	7	7	0	571	1031	1035	0.0	**OK**
	0	8	8	8	0.61	0.61	1.15	27.4	27.4	12.25	7	7	0	571	1031	1035	0.0	**OK**
	0	8	8	8	0.61	0.61	1.15	27.4	27.4	12.25	7	7	0	571	1031	1035	0.0	**OK**

Load Charts:

Braced in One Direction

	Roof Loads 7 ft	Roof Loads 8 ft	Roof Loads 9 ft	Roof Loads 10 ft	Floor Loads 7 ft	Floor Loads 8 ft	Floor Loads 9 ft	Floor Loads 10 ft
(1) 2 x 4	2215	1855	1570	1340	2100	1775	1505	1290
(1) 2 x 6	5150	4630	4140	3695	4695	4270	3855	3470
(2) 2 x 4	4450	3725	3150	2690	4215	3560	3025	2595
(2) 2 x 6	9535	8990	8325	7430	8500	8080	7615	6970
(3) 2 x 4	6960	5805	4890	4160	6620	5560	4710	4025
(3) 2 x 6	15165	14295	13180	11720	13510	12845	12105	11020
(4) 2 x 4	9290	7745	6520	5550	8830	7415	6280	5365
(4) 2 x 6	20245	19080	17580	15630	18035	17145	16155	14700
(5) 2 x 4	11615	9680	8150	6935	11035	9265	7850	6710
(5) 2 x 6	25320	23860	21980	19535	22555	21440	20200	18375

Unbraced in Both Directions

	Roof Loads 7 ft	Roof Loads 8 ft	Roof Loads 9 ft	Roof Loads 10 ft	Floor Loads 7 ft	Floor Loads 8 ft	Floor Loads 9 ft	Floor Loads 10 ft
(2) 2 x 4	2905	2350	1930	1605	2800	2285	1885	1575
(2) 2 x 6	4670	3775	3095	2570	4500	3670	3025	2525
(3) 2 x 4	6605	5590	4750	4065	6205	5310	4550	3915
(3) 2 x 6	11575	9985	8575	7380	10745	9405	8170	7090
(4) 2 x 4	9290	7745	6520	5550	8830	7415	6280	5365
(4) 2 x 6	18155	16500	14830	13245	16425	15120	13760	12425
(5) 2 x 4	11615	9680	8150	6935	11035	9265	7850	6710
(5) 2 x 6	23935	22215	20425	18635	21465	20125	18695	17235
4 x 4	5185	4340	3670	3135	4915	4145	3525	3025
6 x 6	12040	11200	10330	9460	10790	10130	9430	8720
3 1/2" x 3 1/2" PLP	9000	7440	6225	5270	8595	7155	6015	5115
3 1/2" x 5 1/4" PLP	13330	11035	9245	7840	12720	10600	8930	7600
5 1/4" x 5 1/4" PLP	31850	27915	24355	21295	29340	26080	23000	20250
3 1/8" x 7 1/2"	13795	11495	9680	8245	13115	11005	9320	7970
3 1/8" x 9"	16555	13790	11620	9895	15735	13205	11185	9565
5 1/8" x 6"	29565	26595	23720	21095	26900	24510	22110	19840
5 1/8" x 7 1/2"	36955	33240	29650	26370	33625	30640	27640	24805
5 1/8" x 9"	44350	39890	35580	31645	40350	36765	33170	29765

Notes:
1. Example calculations show posts braced in one direction.
2. Loads have been adjusted to accommodate for the worst case of the following eccentric conditions: .175 of column thickness or .175 of column width.

POST / SHEAR WALL / FOOTING / FOUNDATION WALL SCHEDULE

(not all are necessarily used)

Post Schedule

Designation	Post Size
P1	(1) 2x
P2	(2) 2x
P3	(3) 2x
P4	(4) 2x
P5	(5) 2x
P6	6x6
P7	5 1/4"x 5 1/4" Parallam Post

Notes:
1. Posts indicate number of trimmer studs when specified at headers. All other post designations refer to full height king studs U.N.O.
2. Install (1) trimmer stud and (1) king stud each side of each opening U.N.O.
3. Install (2) trimmer studs each side of openings greater than 6'-0" U.N.O.
4. Install (2) king studs each side of openings greater than 8'-0" U.N.O.
5. 2x built-up posts shall be the same width of the wall in which they are framed U.N.O.
6. Nail each ply of 2x built-up posts w/ 16d nails @ 6" o.c. staggered U.N.O.
7. Posts that are not framed within a stud wall shall be braced with BC or AC post cap and PB or ABA post base U.N.O.

Shear Wall Schedule[1,3]

Designation	Material	1 1/2" 16 Gage Staples		8d Nails		Capacity		1/2" Anchor Bolt Spacing	Note
		Edge	Field	Edge	Field	Wind	Seismic		
1	3/8" OSB or CDX plywood	3 1/2"	12"	6"	12"	339	241	32" o.c.	2,4,5
2	3/8" OSB or CDX plywood	2" [6]	12"	4"	12"	495	350	24" o.c.	2,4,5
3	3/8" OSB or CDX plywood	-	-	3"	12"	637	455	16" o.c.	2,4,5,6
4	3/8" OSB or CDX plywood	-	-	2"	12"	832	595	12" o.c.	2,4,5,6
5	1/2" Gypsum or better	-	-	6"	12"	90	90	32" o.c.	4,7
6	1/2" Gypsum or better	-	-	4"	12"	155	155	32" o.c.	4,7
S1	WSWH24x14 Simpson Strong-Wall High-Strength Wood Shearwall. See details 16/S1.2, 17/S1.2, 17/S2.2, & 19/S2.2								
S2	WSWH12x10 Simpson Strong-Wall High-Strength Wood Shearwall. See details 17/S1.2, 21/S1.3, 16/S2.2, & 19/S2.2								
S3	WSWH18x13 Simpson Strong-Wall High-Strength Wood Shearwall. See details 17/S1.2, 19/S1.2, 18/S2.2, & 19/S2.2								

Notes:
1. Wall studs are to be spaced at 16" o.c. U.N.O.
2. Sheath above and below openings in perforated shear walls as per the adjacent shear wall designation on each side of the opening.
3. Use (2) king studs at each end of shear panels (Shear Wall Chords) U.N.O.
4. All panel edges shall be blocked with 2x or wider framing with edge nailing at all supports and panel edges U.N.O.
5. Where panels are applied on both faces of a wall and nail spacing is less than 6" o.c. on either side, panel joints shall be offset to fall on different framing members.
6. Framing at adjoining panel edges and sill plates shall be 3x or wider for edge nailing 3" o.c. or less. Nails at adjoining panel edges and into sill plates shall be staggered. (Double 2x framing stitch-nailed with staggered 16d nails with spacing equal to the shear wall edge nailing is an adequate substitute for 3x framing.)
7. Fasteners for sheet rock shear walls shall be No. 6 Type S or W drywall screws 1-1/4" long in lieu of 8d nails.

Footing Schedule

Designation	Length	Width	Depth	Lengthwise Reinforcement				Crosswise Reinforcement				Capacity	Note
				Qty.	Size	Length	Spacing	Qty.	Size	Length	Spacing		
FT1A	Cont.	12"	20"	2	#4	Cont.	EQ.	-	-	-	-	1250 PLF	(1) top, (1) bottom
FT1B	Cont.	12"	14"	2	#4	Cont.	EQ.	-	-	-	-	1288 PLF	(1) top, (1) bottom
FT1C	Cont.	12"	30"	4	#4	Cont.	EQ.	-	-	-	-	1125 PLF	(2) top, (2) bottom
FT1D	Cont.	18"	20"	4	#4	Cont.	EQ.	-	-	-	-	1875 PLF	(2) top, (2) bottom
FT1E	Cont.	30"	20"	6	#4	Cont.	EQ.	-	#4	24"	12" o.c.	3125 PLF	(3) top, (3) bottom
FT1F	Cont.	24"	14"	8	#4	Cont.	EQ.	-	-	-	-	-	(4) top, (4) bottom
FT1G	Cont.	24"	14"	12	#4	Cont.	EQ.	-	-	-	-	-	(7) top, (5) bottom
FT1H	Cont.	30"	16"	8	#5	Cont.	EQ.	-	-	-	-	-	(4) top, (4) bottom
FT2	Cont.	12"	16"	2	#4	Cont.	EQ.	-	-	-	-	1300 PLF	(1) top, (1) bottom
FT3	24"	24"	12"	3	#4	18"	EQ.	3	#4	18"	EQ.	5400 LBS	
FT4	30"	30"	12"	4	#4	24"	EQ.	4	#4	24"	EQ.	8438 LBS	
FT5	36"	36"	12"	4	#4	30"	EQ.	4	#4	30"	EQ.	12150 LBS	
FT6	42"	42"	12"	5	#4	36"	EQ.	5	#4	36"	EQ.	16538 LBS	
FT7	48"	48"	12"	6	#4	42"	EQ.	6	#4	42"	EQ.	21600 LBS	
FT8	60"	36"	12"	4	#4	48"	EQ.	7	#4	30"	EQ.	21750 LBS	

Notes:
1. f'c= 2,500 psi, fy= 60,000 psi. No special inspection required.
2. Footings shall bear on undisturbed native soils or structural compacted fill (95% compaction), specified and tested by a registered geotechnical engineer.
3. All footings shall bear a minimum of 12" below grade or below the frost line of the locality, whichever is deeper.

Framing Notes

1. Plans are not complete without the structural calculations.
2. Refer to sheet S0.0 for the general structural notes.
3. Roof sheathing to be APA rated 7/16" OSB or CDX plywood with 8d nails at 6" o.c. edge, 12" o.c. field.
4. Floor sheathing to be APA rated 3/4" T&G with 10d nails or Simpson WSNTL2LS #8 wood screws at 6" o.c. edge, 12" o.c. field.
5. Exterior stud walls (including garage walls) to be 2x6 at 16" o.c. U.N.O.
6. Use (12) 16d nails between top plate splice points on all exterior and shear walls. Provide a 4'-0" minimum lap splice.
7. Install all Simpson hardware per manufacturer's specifications.
8. Holdowns shall be installed on (2) full height king studs (minimum).
9. Ceiling joists to be 2x12 DF-L#2 at 16" o.c. U.N.O.
10. Overbuild roof rafters to be 2x6 DF-L #2 at 24" o.c. U.N.O.
11. Provide 2x squash blocking at floor framing to match dimensions of post above.
12. All details shall apply in all similar situations.
13. All lumber not permanently protected from the elements shall be preservative treated or of a decay resistant species. Contact LEI Engineers and Surveyors, Inc. if a different species is to be used.

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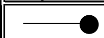
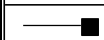
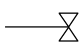
Holdown Schedule

Symbol	Holdown/Strap
	LSTHD8 Holdown See detail 9/S1.2
	STHD10 Holdown See detail 9/S1.2
	HDU2-SDS2.5 Retrofit holdown w/ 5/8" dia. A36 threadad rod anchor embedded 6" into footing w/ Simpson ET-3G or AT-3G epoxy. Special inspection required. See detail 7/S1.2
	HDU5-SDS2.5 Retrofit holdown w/ 5/8" dia. A36 threadad rod anchor embedded 8" into footing w/ Simpson ET-3G or AT-3G epoxy. Special inspection required. See detail 7/S1.2

Simpson WSWH Notes

1. Strong Wall High Strength Wood Shear Walls shall be installed as per Simpson Specifications.
2. WSWH may be field trimmed to a minimum height of 74 1/2". (Trim top of wall only - Do not trim from sides or bottom)
3. Drilling holdes in WSWH is not allowed except as specifically allowed by the manufacturer (Refer to Simpson Specifications)
4. Anchor bolt nuts should be finger tight plus 1/2 turn.
5. Top connection installs with a combination of SDS25600 Heavy-Duty connector screws & SWS16150 Strong Wall Screws
6. Take precaution when installing cast-in-place bolts at concrete foundation (No retrofit option is available)
7. Contact Simpson representative Gary Pugmire (801-244-7430) with questions regarding the installation of Simpson Strong Walls.