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BOULDER | GOLDEN | STEAMBOAT SPRINGS

Structural Calculations

Basecamp Steamboat Townhomes – TH1

Anthem Job # 22-048

1950 Curce Court
Steamboat Springs, CO 80487

Sept. 6th, 2022

COLORADO LICENSED PROFESSIONAL ENGINEER
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09/07/2022

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DESIGN CRITERIA

STRUCTURAL CALCULATIONS

FOR

BASECAMP STEAMBOAT TOWNHOMES

Code Search

Code: International Building Code 2018

Occupancy:

Occupancy Group = R Residential

Risk Category & Importance Factors:

Risk Category =	II
Wind factor =	1.00
Snow factor =	1.00
Seismic factor =	1.00

Type of Construction:

Fire Rating:

Roof =	0.0 hr
Floor =	0.0 hr

Building Geometry:

Roof angle (θ)	3.00 / 12	14.0 deg
Building length (L)	40.0 ft	
Least width (B)	120.0 ft	
Mean Roof Ht (h)	48.5 ft	
Parapet ht above grd	0.0 ft	
Minimum parapet ht	0.0 ft	

Live Loads:

Roof

0 to 200 sf:	20 psf
200 to 600 sf:	24 - 0.02Area, but not less than 12 psf
over 600 sf:	12 psf

Floor:

Typical Floor	40 psf
Partitions	15 psf
Lobbies & first floor corridors	100 psf
Corridors above first floor	80 psf
Balconies (1.5 times live load)	60 psf

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JOB TITLE BASECAMP STEAMBOAT TOWNHOMES

JOB NO. ______ **SHEET NO.** ______
CALCULATED BY ______ **DATE** ______
CHECKED BY ______ **DATE** ______

Wind Loads - MWFRS h≤60' (Low-rise Buildings) except for open buildings

Kz = Kh (case 1) = 1.09
Base pressure (qh) = **24.6 psf**
GCpi = +/-0.18

Edge Strip (a) = 4.0 ft
End Zone (2a) = 8.0 ft
Zone 2 length = 20.0 ft

Wind Pressure Coefficients

	CASE A	θ = 14 deg		CASE B		
Surface	GCpf	w/-GCpi	w/+GCpi	GCpf	w/-GCpi	w/+GCpi
1	0.48	0.66	0.30	-0.45	-0.27	-0.63
2	-0.69	-0.51	-0.87	-0.69	-0.51	-0.87
3	-0.44	-0.26	-0.62	-0.37	-0.19	-0.55
4	-0.37	-0.19	-0.55	-0.45	-0.27	-0.63
5				0.40	0.58	0.22
6				-0.29	-0.11	-0.47
1E	0.72	0.90	0.54	-0.48	-0.30	-0.66
2E	-1.07	-0.89	-1.25	-1.07	-0.89	-1.25
3E	-0.63	-0.45	-0.81	-0.53	-0.35	-0.71
4E	-0.56	-0.38	-0.74	-0.48	-0.30	-0.66
5E				0.61	0.79	0.43
6E				-0.43	-0.25	-0.61

Ultimate Wind Surface Pressures (psf)

1		16.2	7.3		-6.6	-15.5
2		-12.5	-21.4		-12.5	-21.4
3		-6.3	-15.1		-4.7	-13.5
4		-4.8	-13.6		-6.6	-15.5
5					14.2	5.4
6					-2.7	-11.5
1E		22.2	13.4		-7.4	-16.2
2E		-21.9	-30.7		-21.9	-30.7
3E		-11.0	-19.8		-8.6	-17.4
4E		-9.2	-18.1		-7.4	-16.2
5E					19.4	10.6
6E					-6.1	-15.0

Parapet

Windward parapet = 0.0 psf (GCpn = +1.5)
Leeward parapet = 0.0 psf (GCpn = -1.0)

Windward roof overhangs = 17.2 psf (upward) add to windward roof pressure

Horizontal MWFRS Simple Diaphragm Pressures (psf)

Transverse direction (normal to L)

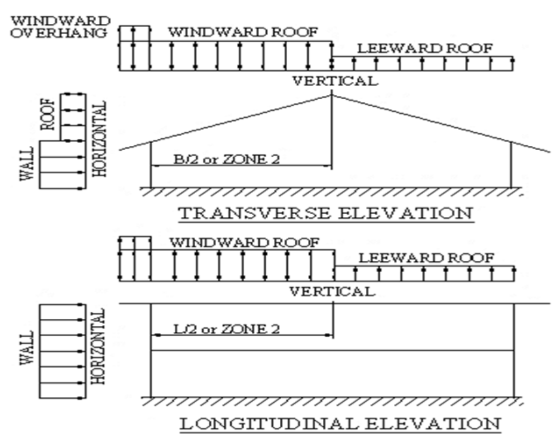
Interior Zone: Wall 20.9 psf
Roof -6.2 psf **
End Zone: Wall 31.5 psf
Roof -10.9 psf **

Longitudinal direction (parallel to L)

Interior Zone: Wall 17.0 psf
End Zone: Wall 25.6 psf

** NOTE: Total horiz force shall not be less than that determined by neglecting roof forces (except for MWFRS moment frames).

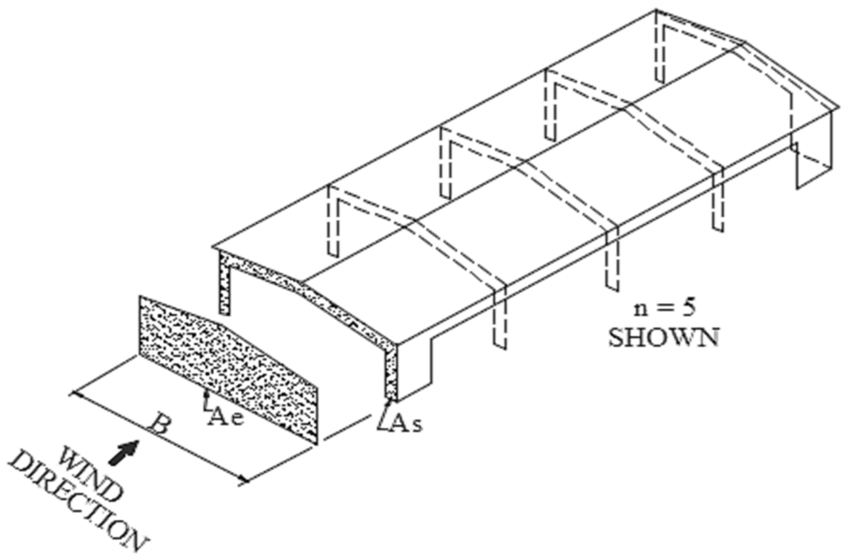
The code requires the MWFRS be designed for a min ultimate force of 16 psf multiplied by the wall area plus an 8 psf force applied to the vertical projection of the roof.



Wind Loads - h≤60' Longitudinal Direction MWFRS On Open or Partially Enclosed Buildings with Transverse Frames and Pitched Roofs

ASCE 7-16 procedure

Base pressure (qh) = **24.6 psf**
GCpi = +/-0.18 Enclosed bldg, procdure doesn't apply
Roof Angle (θ) = 14.0 deg



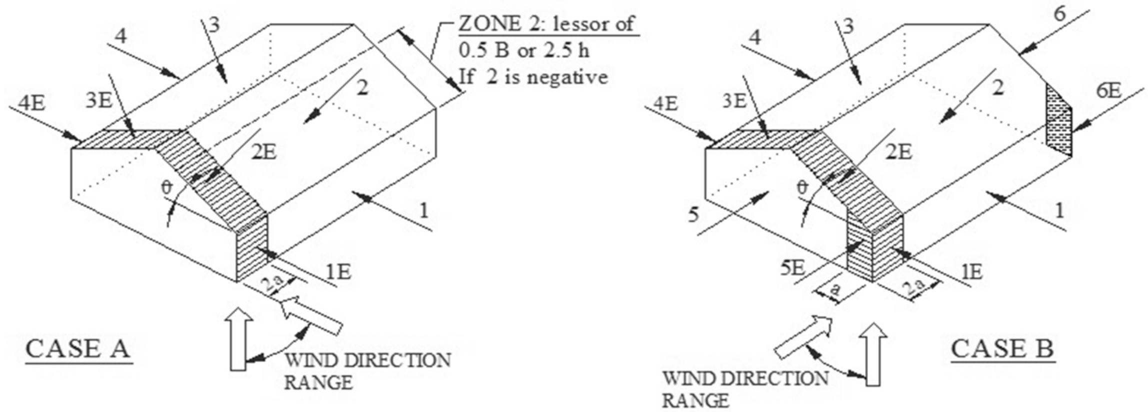
B=	120.0	ft
# of frames (n) =	5	
Solid are of end wall including fascia (As) =	1,500.0	sf
Roof ridge height =	56.0	ft
Roof eave height =	41.0	ft
Total end wall area if soild (Ae) =	5,820.0	sf

Longidinal Directional Force (F) = pAe
p= qh [(GCpf)windward -(GCpf)leeward] $K_B K_S$

Solidarity ratio (Φ) =	0.258	
n =	5	
KB =	0.6	
KS =	0.855	
Zones 5 & 6 area =	5,641	sf
5E & 6E area =	179	sf
(GCpf) windward - (GCpf) leeward] =	0.701	
p =	8.8	psf

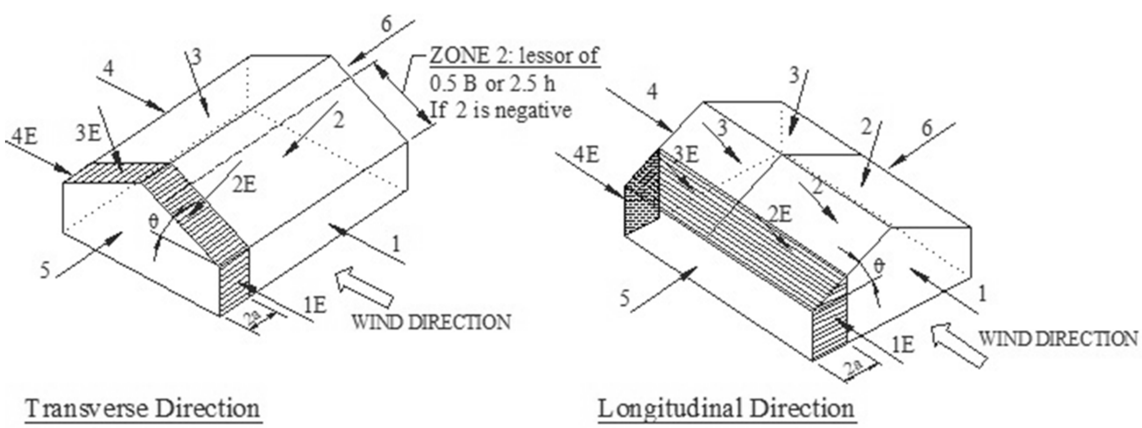
Total force to be resisted by MWFRS (F) = **51.4 kips** applied at the centroid of the end wall area Ae

Note: The longidudinal force acts in combination with roof loads calculated elsewhere for an open or partially enclosed building.



NOTE: Torsional loads are 25% of zones 1 - 6. See code for loading diagram.
Exception: One story buildings h<30' and 1 to 2 storybuildings framed with light-frame construction or with flexible diaphragms need not be designed for the torsional load case.

ASCE 7-98 & ASCE 7-10 (& later) - MWFRS wind pressure zones



NOTE: Torsional loads are 25% of zones 1 - 4. See code for loading diagram.
Exception: One story buildings h<30' and 1 to 2 storybuildings framed with light-frame construction or with flexible diaphragms need not be designed for the torsional load case.

ASCE 7-02 and ASCE 7-05 - MWFRS wind pressure zones

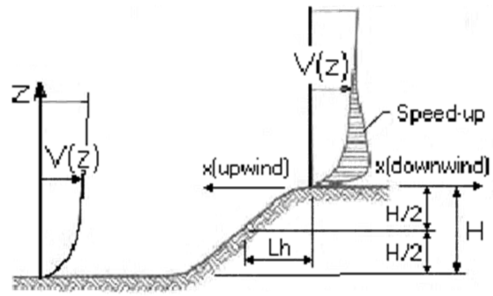
Wind Loads : ASCE 7- 16

Ultimate Wind Speed	115 mph
Nominal Wind Speed	89.1 mph
Risk Category	II
Exposure Category	C
Enclosure Classif.	Enclosed Building
Internal pressure	+/-0.18
Directionality (Kd)	0.85
Kh case 1	1.087
Kh case 2	1.087
Type of roof	Sawtooth

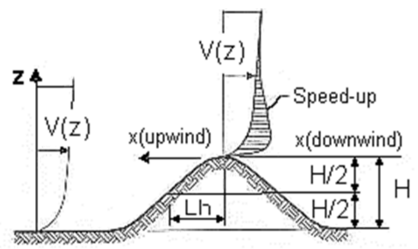
Topographic Factor (Kzt)

Topography	Flat
Hill Height (H)	0.0 ft
Half Hill Length (Lh)	0.0 ft
Actual H/Lh =	0.00
Use H/Lh =	0.00
Modified Lh =	0.0 ft
From top of crest: x =	0.0 ft
Bldg up/down wind?	downwind

H< 15ft;exp C
∴ Kzt=1.0



ESCARPMENT



2D RIDGE or 3D AXISYMMETRICAL HILL

H/Lh= 0.00 K_1 = 0.000
x/Lh = 0.00 K_2 = 0.000
z/Lh = 0.00 K_3 = 1.000
At Mean Roof Ht:
$Kzt = (1+K_1K_2K_3)^2$ = 1.00

Gust Effect Factor

h = 48.5 ft
B = 120.0 ft
/z (0.6h) = 29.1 ft

Flexible structure if natural frequency < 1 Hz (T > 1 second).
If building h/B>4 then may be flexible and should be investigated.
h/B = 0.40 Rigid structure (low rise bldg)

G = 0.85 Using rigid structure default

Rigid Structure

ē =	0.20
ℓ =	500 ft
z_{min} =	15 ft
c =	0.20
g_Q, g_v =	3.4
L_z =	487.6 ft
Q =	0.87
I_z =	0.20
G =	0.86 use G = 0.85

Flexible or Dynamically Sensitive Structure

34 ıcy (η_1) =	0.0 Hz				
Damping ratio (β) =	0				
/b =	0.65				
/α =	0.15				
Vz =	107.5				
N_1 =	0.00				
R_n =	0.000				
R_h =	28.282	η =	0.000	h =	48.5 ft
R_B =	28.282	η =	0.000		
R_L =	28.282	η =	0.000		
g_R =	0.000				
R =	0.000				
Gf =	0.000				

Enclosure Classification

Test for Enclosed Building: Ao < 0.01Ag or 4 sf, whichever is smaller

Test for Open Building: All walls are at least 80% open.
$A_o \geq 0.8A_g$

Test for Partially Enclosed Building: Predominately open on one side only

	Input			Test	
Ao	500.0	sf	Ao ≥ 1.1Aoi	NO	
Ag	600.0	sf	Ao > 4' or 0.01Ag	YES	
Aoi	1000.0	sf	Aoi / Agi ≤ 0.20	YES	Building is NOT
Agi	10000.0	sf			Partially Enclosed

Conditions to qualify as Partially Enclosed Building. Must satisfy all of the following:

$A_o \geq 1.1A_{oi}$

$A_o >$ smaller of 4' or 0.01 A_g

$A_{oi} / A_{gi} \leq 0.20$

Where:
Ao = the total area of openings in a wall that receives positive external pressure.
Ag = the gross area of that wall in which Ao is identified.
Aoi = the sum of the areas of openings in the building envelope (walls and roof) not including Ao.
Agi = the sum of the gross surface areas of the building envelope (walls and roof) not including Ag.

Test for Partially Open Building: A building that does not qualify as open, enclosed or partially enclosed.
(This type building will have same wind pressures as an enclosed building.

Reduction Factor for large volume partially enclosed buildings (Ri) :

If the partially enclosed building contains a single room that is unpartitioned , the internal pressure coefficient may be multiplied by the reduction factor Ri.

Total area of all wall & roof openings (Aog):		0 sf
Unpartitioned internal volume (Vi) :		0 cf
	Ri =	1.00

Ground Elevation Factor (Ke)

Grd level above sea level =	6668.0 ft			Ke =	0.7855
Constant =	0.00256	Adj Constant =	0.00201		

Ultimate Wind Pressures

Wind Loads - Components & Cladding : h ≤ 60'

Kh (case 1) =	1.09	h =	48.5 ft
Base pressure (qh) =	**24.6 psf**	a =	4.0 ft
Minimum parapet ht =	0.0 ft	GCpi =	+/-0.18
Roof Angle (θ) =	14.0 deg	qi = qh =	24.6 psf
Type of roof =	Sawtooth		

Roof

	GCp +/- Gcpi				Surface Pressure (psf)				User input	
Area	10 sf	50 sf	100 sf	500 sf	10 sf	50 sf	100 sf	500 sf	75 sf	300 sf
Negative Zone 1	-2.38	-1.93	-1.73	-1.28	-58.5	-47.4	-42.6	-31.4	-44.6	-35.0
Negative Zone 2	-3.38	-2.72	-2.44	-1.78	-83.0	-66.9	-59.9	-43.7	-62.8	-48.9
Span A Negative Zone 3	-4.28	-4	-3.88	-2.28	-105.2	-98.3	-95.3	-56.0	-96.6	-68.5
Span B,C&D Neg. Zone 3	-2.78	-2.78	-2.78	-2.08	-68.3	-68.3	-68.3	-51.1	-68.3	-56.6
Positive Zone 1	0.88	0.76	0.7	0.58	21.6	18.6	17.3	16.0	17.8	16.0
Positive Zone 2	1.28	1.07	0.98	0.98	31.4	26.3	24.1	24.1	25.0	24.1
'Positive Zone 3	0.98	0.91	0.88	0.88	24.1	22.4	21.6	21.6	21.9	21.6

Parapet

qp = 0.0 psf

	Surface Pressure (psf)						User input
Solid Parapet Pressure	10 sf	20 sf	50 sf	100 sf	200 sf	500 sf	40 sf
CASE A: Zone 2 :	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Span A Zone 3 :	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Span B,C&D Zone 3 :	0.0	0.0	0.0	0.0	0.0	0.0	0.0
CASE B : Interior zone :	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Corner zone :	0.0	0.0	0.0	0.0	0.0	0.0	0.0

Walls

	GCp +/- GCpi				Surface Pressure at h				User input	
Area	10 sf	100 sf	200 sf	500 sf	10 sf	100 sf	200 sf	500 sf	46 sf	83 sf
Negative Zone 4	-1.28	-1.10	-1.05	-0.98	-31.4	-27.1	-25.8	-24.1	-28.6	-27.5
Negative Zone 5	-1.58	-1.23	-1.12	-0.98	-38.8	-30.1	-27.5	-24.1	-33.0	-30.9
Positive Zone 4 & 5	1.18	1.00	0.95	0.88	29.0	24.7	23.3	21.6	26.1	25.0

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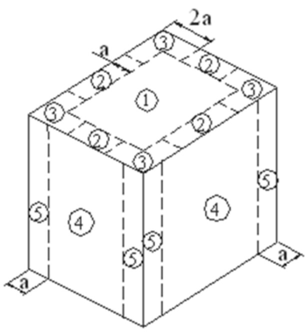
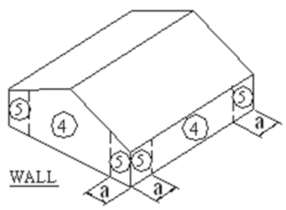
JOB TITLE BASECAMP STEAMBOAT TOWNHOMES

JOB NO. SHEET NO.
CALCULATED BY DATE
CHECKED BY DATE

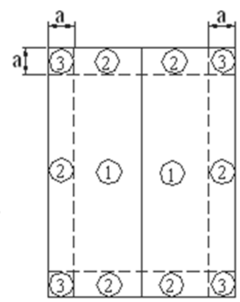
Location of C&C Wind Pressure Zones - ASCE 7-10 & earlier



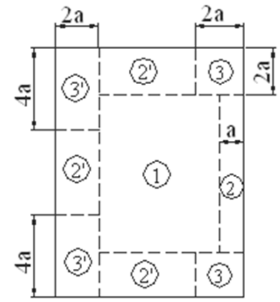
Roofs w/ θ ≤ 10°
and all walls
h > 60'



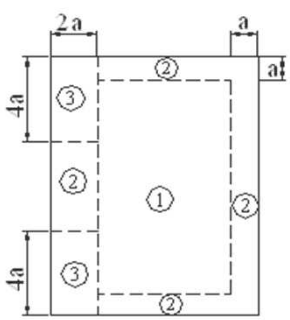
Walls h ≤ 60'
& alt design h<90'



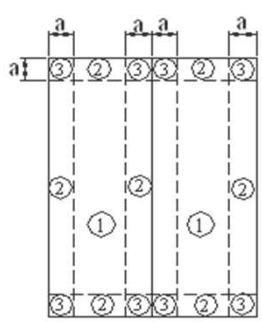
Gable, Sawtooth and
Multispan Gable θ ≤ 7 degrees &
Monoslope ≤ 3 degrees
h ≤ 60' & alt design h<90'



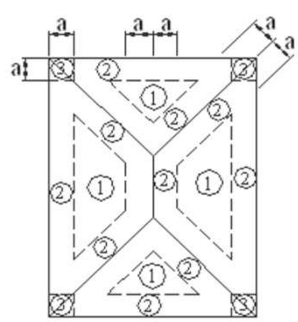
Monoslope roofs
3° < θ ≤ 10°
h ≤ 60' & alt design h<90'



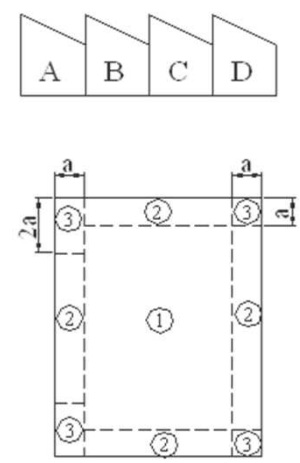
Monoslope roofs
10° < θ ≤ 30°
h ≤ 60' & alt design h<90'



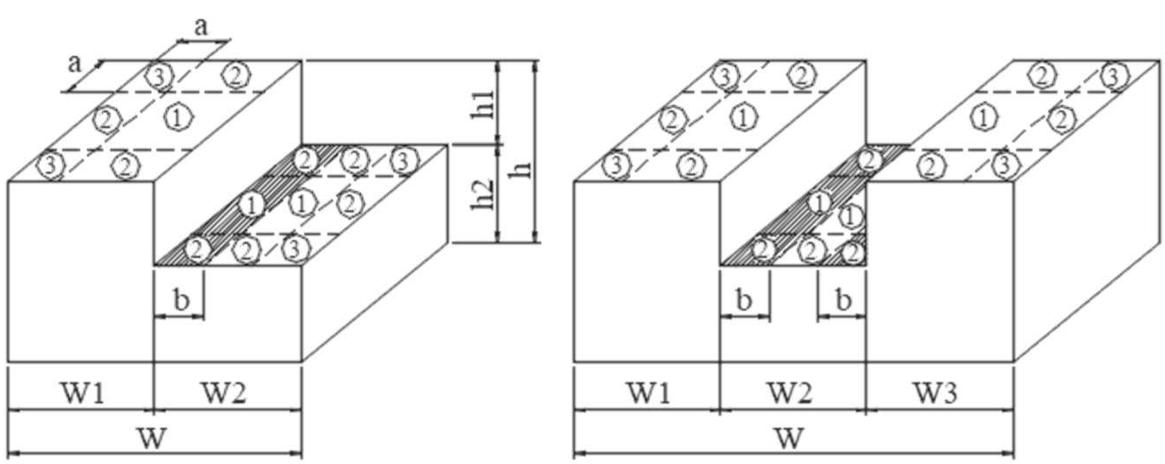
Multispan Gable &
Gable 7° < θ ≤ 45°



Hip 7° < θ ≤ 27°



Sawtooth 10° < θ ≤ 45°
h ≤ 60' & alt design h<90'



Stepped roofs θ ≤ 3°
h ≤ 60' & alt design h<90'

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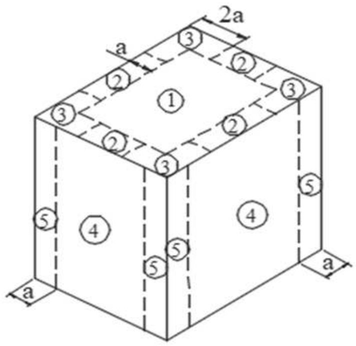
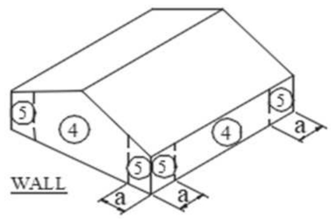
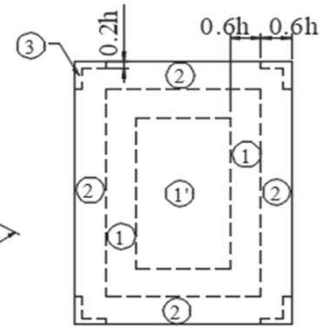
Location of C&C Wind Pressure Zones - ASCE 7-16



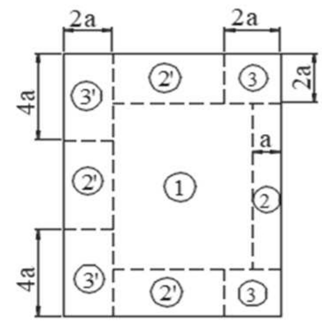
Roofs w/ θ ≤ 10°
and all walls
h > 60'



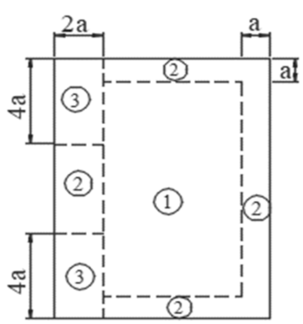
Walls h ≤ 60'
& alt design h<90'



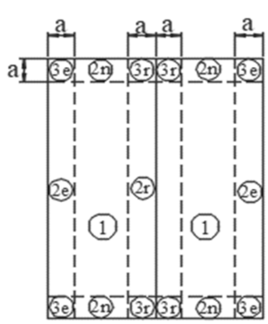
Gable, Sawtooth and
Multispan Gable θ ≤ 7 degrees &
Monoslope ≤ 3 degrees
h ≤ 60' & alt design h<90'



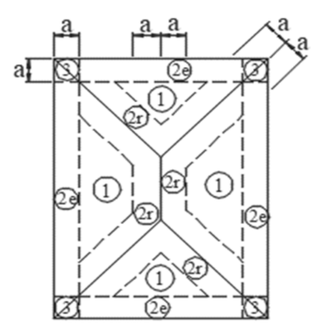
Monoslope roofs
3° < θ ≤ 10°
h ≤ 60' & alt design h<90'



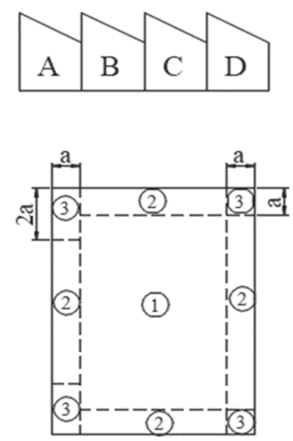
Monoslope roofs
10° < θ ≤ 30°
h ≤ 60' & alt design h<90'



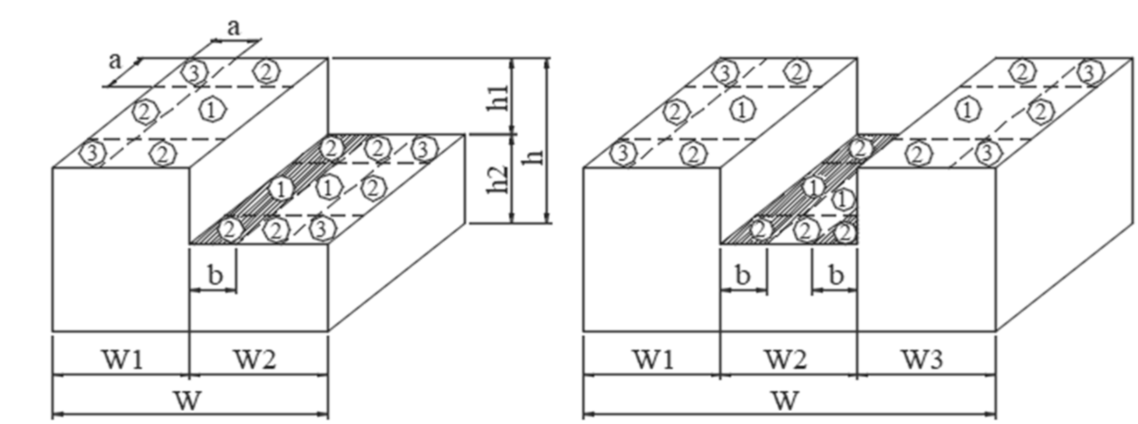
Multispan Gable &
Gable 7° < θ ≤ 45°



Hip 7° < θ ≤ 27°



Sawtooth 10° < θ ≤ 45°
h ≤ 60' & alt design h<90'



Stepped roofs θ ≤ 3°
h ≤ 60' & alt design h<90'

Wind Loads - Other Structures:

ASCE 7- 16 | Ultimate Wind Pressures

Wind Factor =	1.00		
Gust Effect Factor (G) =	0.85	Ultimate Wind Speed =	115 mph
Kzt =	1.00	Exposure =	C

A. Solid Freestanding Walls & Solid Signs (& open signs with less than 30% open)

		s/h =	1.00	**Case A & B**	
Dist to sign top (h)	8.0 ft	B/s =	25.00	C_f =	1.30
Height (s)	8.0 ft	Lr/s =	0.00	F = qz G Cf As =	**21.2 As**
Width (B)	200.0 ft	Kz =	0.849	As =	10.0 sf
Wall Return (Lr) =		qz =	19.2 psf	F =	212 lbs
Directionality (Kd)	0.85				
Percent of open area to gross area	0.0%	Open reduction factor =	1.00		

Case C reduction factors

Factor if s/h>0.8 =	0.80
Wall return factor for Cf at 0 to s =	1.00

CaseC

Horiz dist from windward edge	Cf	F=qzGCfAs (psf)
0 to s	3.29	53.7 **As**
s to 2s	2.07	33.7 **As**
2s to 3s	1.59	25.9 **As**
3s to 4s	1.31	21.3 **As**
4s to 5s	1.23	20.1 **As**
5s to 10s	0.78	16.0 **As**
>10s	0.44	16.0 **As**

B. Open Signs & Single-Plane Open Frames (openings 30% or more of gross area)

Height to centroid of Af (z)	15.0 ft			Kz =	0.849
				Base pressure (qz) =	19.2 psf
Width (zero if round)	0.0 ft				
Diameter (zero if rect)	2.0 ft	D(qz)^.5 =	8.76	$F = q_z G C_f A_f$ =	**17.9 Af**
Percent of open area		I =	0.65	Solid Area: A_f =	10.0 sf
to gross area	35.0%	C_f =	1.1	F =	179 lbs
Directionality (Kd)	0.85				

C. Chimneys, Tanks, & Similar Structures

Height to centroid of Af (z)	15.0 ft	Kz =	0.849
Cross-Section	Square	Base pressure (qz) =	20.3 psf
Directionality (Kd)	0.90	h/D =	15.00
Height (h)	15.0 ft		
Width (D)	1.0 ft		
Type of Surface	N/A		

Square (wind along diagonal)		Square (wind normal to face)	
Cf =	1.28	C_f =	1.67
F = qz G Cf Af =	**22.1 Af**	$F = q_z G C_f A_f$ =	**28.8 Af**
Af =	sf	A_f =	10.0 sf
F =	0 lbs	F =	288 lbs

D. Trussed Towers

Height to centroid of Af (z)	15.0 ft	Kz =	0.849
$\in$ =	0.27	Base pressure (qz) =	21.4 psf
Tower Cross Section	triangle		
Member Shape	flat	Diagonal wind factor =	1
Directionality (Kd)	0.95	Round member factor =	1.000

Triangular Cross Section

C_f =	2.38
$F = q_z G C_f A_f$ =	**43.4 Af**
Solid Area: A_f =	10.0 sf
F =	434 lbs

~~10.0~~

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JOB TITLE BASECAMP STEAMBOAT TOWNHOMES

JOB NO. ______ **SHEET NO.** ______
CALCULATED BY ______ **DATE** ______
CHECKED BY ______ **DATE** ______

Snow Loads : ASCE 7- 16

Nominal Snow Forces

Roof slope		=	14.0 deg
Horiz. eave to ridge dist (W)		=	60.0 ft
Roof length parallel to ridge (L)		=	40.0 ft
Type of Roof			Sawtooth, etc.
Ground Snow Load	Pg	=	104.0 psf
Risk Category		=	II
Importance Factor	I	=	1.0
Thermal Factor	Ct	=	1.00
Exposure Factor	Ce	=	1.1
Pf = 0.7*Ce*Ct*I*Pg		=	80.1 psf
Unobstructed Slippery Surface			no
Sloped-roof Factor	Cs	=	1.00
Balanced Snow Load		**=**	**80.1 psf**
Rain on Snow Surcharge Angle			1.20 deg
Code Maximum Rain Surcharge			5.0 psf
Rain on Snow Surcharge		=	0.0 psf
Ps plus rain surcharge		=	80.1 psf
Minimum Snow Load	Pm	=	20.0 psf
Uniform Roof Design Snow Load		=	**80.1 psf**

Near ground level surface balanced snow load = **104.0 psf**

NOTE: Alternate spans of continuous beams shall be loaded with half the design roof snow load so as to produce the greatest possible effect - see code for loading diagrams and exceptions for gable roofs..

Unbalanced Snow Loads - for Sawtooth, Multiple Folded Plates & Barrel Vault roofs only

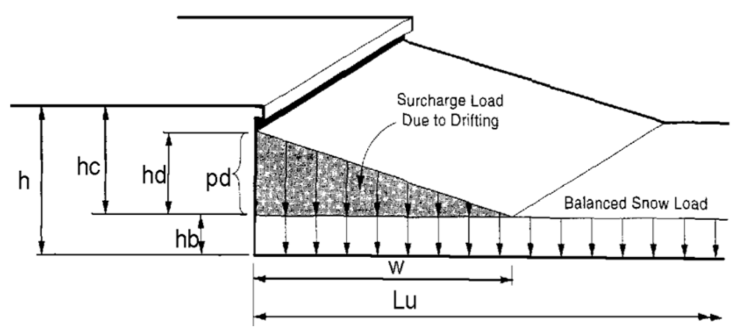
Unbalanced snow loads must be applied

Ridge or Crown snow load =	40.0 psf	= 0.5*Pf	1.45 ft of snow
Valley snow load =	145.6 psf	= 2*Pf/Ce	5.29 ft of snow

NOTE: Valley snow shall not be higher than snow at the ridge.

Windward Snow Drifts 1 - Against walls, parapets, etc

Upwind fetch	lu	=	220.0 ft
Projection height	h	=	5.2 ft
Snow density	g	=	27.5 pcf
Balanced snow height	hb	=	2.91 ft
	hd	=	5.24 ft
	hc	=	2.29 ft
hc/hb >0.2 = 0.8	**Therefore, design for drift**		
Drift height (hc)		=	2.29 ft
Drift width	w	=	18.32 ft
Surcharge load:	pd = γ*hd	=	**63.0 psf**
Balanced Snow load:		=	80.1 psf
			143.1 psf



Note: If bottom of projection is at least 2 feet above hb then snow drift is not required.

Windward Snow Drifts 2 - Against walls, parapets, etc

Upwind fetch	lu	=	160.0 ft
Projection height	h	=	4.0 ft
Snow density	g	=	27.5 pcf
Balanced snow height	hb	=	2.91 ft
	hd	=	4.60 ft
	hc	=	1.09 ft
hc/hb >0.2 = 0.4	**Therefore, design for drift**		
Drift height (hc)		=	1.09 ft
Drift width	w	=	8.72 ft
Surcharge load:	pd = γ*hd	=	**30.0 psf**
Balanced Snow load:		=	80.1 psf
			110.1 psf

Snow Loads - from adjacent building or roof:

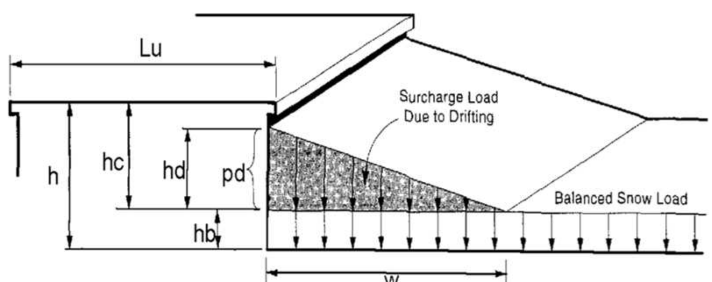
ASCE 7- 16 — Nominal Snow Forces

			Higher Roof	Lower Roof
Roof slope		=	14.0 deg	0.25 / 12 = 1.2 deg
Horiz. eave to ridge dist (W)		=	60.0 ft	24.0 ft
Roof length parallel to ridge (L)		=	40.0 ft	4.5 ft
Projection height (roof step)	h	=		30.0 ft
Building separation	s	=		0.0 ft
Type of Roof			Sawtooth, etc.	Monoslope
Ground Snow Load	Pg	=	104.0 psf	104.0 psf
Risk Category		=	II	II
Importance Factor	I	=	1.0	1.0
Thermal Factor	Ct	=	1.10	1.20
Exposure Factor	Ce	=	1.0	1.1
Pf = 0.7*Ce*Ct*I*Pg		=	80.1 psf	96.1 psf
Unobstructed Slippery Surface			no	no
Sloped-roof Factor	Cs	=	1.00	1.00
Balanced Snow Load	**Ps**	**=**	**80.1 psf**	**96.1 psf**
Rain on Snow Surcharge Angle			1.20 deg	0.48 deg
Code Maximum Rain Surcharge			5.0 psf	5.0 psf
Rain on Snow Surcharge		=	0.0 psf	0.0 psf
Ps plus rain surcharge		=	80.1 psf	96.1 psf
Minimum Snow Load	Pm	=	20.0 psf	20.0 psf
Uniform Roof Design Snow Load		=	**80.1 psf**	**96.1 psf**
Building Official Minimum		=		

NOTE: Alternate spans of continuous beams and other areas shall be loaded with half the design roof snow load so as to produce the greatest possible effect - see code.

Leeward Snow Drifts - from adjacent higher roof

Upper roof length	lu	=	38.7 ft
Snow density	ɣ	=	27.5 pcf
Balanced snow height	hb	=	3.49 ft
	hc	=	26.51 ft
hc/hb >0.2 = 7.6	**Therefore, design for drift**		
Adj structure factor		=	1.00
Drift height (hd)		=	3.25 ft
Drift width	w	=	13.00 ft
Surcharge load:	pd = γ*hd	=	**89.5 psf**
Balanced Snow load:		=	96.1 psf
			185.6 psf **Leeward drift controls**

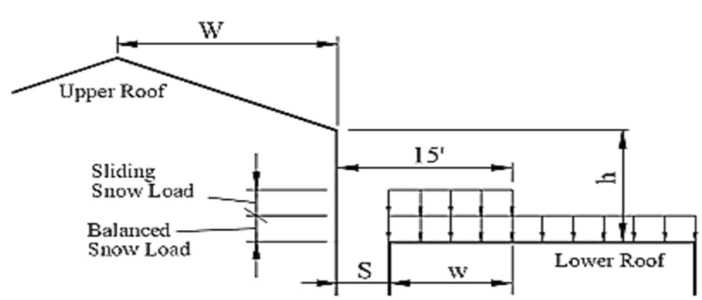


Windward Snow Drifts - from low roof against high roof

Lower roof length	lu	=	24.0 ft
Adj structure factor		=	1.00
Drift height	hd	=	1.91 ft
Drift width	w	=	7.66 ft
Surcharge load:	pd = γ*hd	=	**52.7 psf**
Balanced Snow load:		=	96.1 psf
			148.8 psf

Sliding Snow - onto lower roof

Sliding snow = 0.4 Pf W	=	1921.9 plf
Distributed over 15 feet	=	128.1 psf
hd + hb	=	8.15 ft
hd + hb < =h therefore sliding snow	=	**128.1 psf**
Balanced snow load	=	96.1 psf
Uniform snow load within 15' of higher roof	=	224.2 psf
	w =	15.00 ft



Seismic Loads: IBC 2018 Strength Level Forces

Risk Category : II
Importance Factor (I) : 1.00
Site Class : C

Ss (0.2 sec) = 59.70 %g
S1 (1.0 sec) = 10.30 %g

Fa =	1.261	use 0.84	Sms =	0.500	S_{DS} =	0.333	Design Category =	C
Fv =	1.500		Sm1 =	0.155	S_{D1} =	0.103	Design Category =	B

Seismic Design Category = **C**

Redundancy Coefficient ρ = 1.30
Number of Stories 4
Structure Type: All other building system:
Horizontal Struct Irregularities: No plan Irregularity
Vertical Structural Irregularities: 1b) Stiffness Irregularity—Extreme Soft Story

Flexible Diaphragms: Yes
Building System: **Bearing Wall Systems**
Seismic resisting system: **Light frame (wood) walls with structural wood shear panels**
System Structural Height Limit: **Height not limited**
Actual Structural Height (hn) = 48.5 ft

DESIGN COEFFICIENTS AND FACTORS

Response Modification Coefficient (R) = 6.5
Over-Strength Factor (Ωo) = 2.5
Deflection Amplification Factor (Cd) = 4
S_{DS} = 0.333
S_{D1} = 0.103

Seismic Load Effect (E) = Eh +/-Ev = ρQ_E +/- $0.2S_{DS}$ D = 1.3Qe +/- 0.067D Q_E = horizontal seismic force
Special Seismic Load Effect (Em) = Emh +/- Ev = $\Omega o\, Q_E$ +/- $0.2S_{DS}$ D = 2.5Qe +/- 0.067D D = dead load

PERMITTED ANALYTICAL PROCEDURES

Simplified Analysis - Use Equivalent Lateral Force Analysis

Equivalent Lateral-Force Analysis - Permitted

Building period coef. (C_T) =	0.020				Cu = 1.69
Approx fundamental period (Ta) =	$C_T h_n^x$ =	0.368	sec	x= 0.75	Tmax = CuTa = 0.623
User calculated fundamental period (T)			sec		Use T = 0.368
Long Period Transition Period (TL) =	ASCE7 map =	4			
Seismic response coef. (Cs) =	$S_{DS}I/R$ =	0.051			
need not exceed Cs =	Sd1 I /RT =	0.043			
but not less than Cs =	0.044SdsI =	0.015			
USE Cs =		0.043			

Design Base Shear V = 0.043W

Model & Seismic Response Analysis - Permitted (see code for procedure

ALLOWABLE STORY DRIFT

Structure Type: All other structures

Allowable story drift Δa = 0.020hsx where hsx is the story height below level

Seismic Loads - cont. :

Strength Level Forces

Seismic Design Category (SDC)= C
Ie = 1.00
Sds = 0.333

CONNECTIONS

Force to connect smaller portions of structure to remainder of structure

$F_p = 0.133Sdsw_p =$ 0.044 w_p
or $F_p = 0.05w_p =$ 0.05 w_p Use Fp = 0.05 w_p w_p = weight of smaller portion

Beam, girder or truss connection for resisting horizontal force parallel to member

F_P = no less than 0.05 times dead plus live load vertical reaction

Anchorage of Structural Walls to elements providing lateral support

Fp = not less than 0.2KaIeWp
Flexible diaphragm span Lf =
Enter Lf to calculate Fp for flexible diaphragm
Fp =0.4SdskaIeWp = 0.133 Wp, but not less than 0.2Wp (rigid diaphragm) ka= 1 Fp = 0.200 Wp
but Fp shall not be less than 5 psf

MEMBER DESIGN

Bearing Walls and Shear Walls (out of plane force)

$F_p = 0.4SdsIeWw =$ 0.133 w_w
but not less than 0.10 w_w Use Fp = 0.13 w_w

Diaphragms

Fp = 0.2ISdsWp + Vpx = 0.067 Wp + Vpx

ARCHITECTURAL COMPONENTS SEISMIC COEFFICIENTS

Architectural Component : Cantilever Elements (Unbraced or Braced to Structural Frame Below Its Center of Mass): Parapets and cantilever interior nonstructural walls

Importance Factor (Ip) : 1.0
Component Amplification Factor (a_p) = 2.5 h= 48.5 feet
Comp Response Modification Factor (R_p) = 2.5 z= 50.0 feet z/h = 1.00
Over-Strength Factor (Ωo) = 2

$F_p = 0.4a_pSdsIpWp(1+2z/h)/Rp =$ 0.400 Wp
not greater than Fp = 1.6SdsIpWp = 0.533 Wp
but not less than Fp = 0.3SdsIpWp = 0.100 Wp use Fp = 0.400 Wp

MECH AND ELEC COMPONENTS SEISMIC COEFFICIENTS

Seismic Design Category C & Ip=)1.0, therefore not required

Mech or Electrical Component : Wet-side HVAC, boilers, furnaces, atmospheric tanks and bins, chillers, water heaters, etc plus other mechanical components constructed of high-deformability materials.

Importance Factor (Ip) : 1.0
Component Amplification Factor (a_p) = 1 h= 48.5 feet
Comp Response Modification Factor (R_p) = 2.5 z= 50.0 feet z/h = 1.00
Over-Strength Factor (Ωo) = 2

$F_p = 0.4a_pSdsIpWp(1+2z/h)/Rp =$ 0.160 Wp
not greater than Fp = 1.6SdsIpWp = 0.533 Wp
but not less than Fp = 0.3SdsIpWp = 0.100 Wp use Fp = 0.160 Wp

Roof Design Loads

Items	Description	Multiple	psf (max)	psf (min)
Roofing	Asphalt Shingles w/roll roofing		3.0	2.0
Decking	5/8" plywood/OSB		2.2	1.8
Framing	Wood Trusses @ 24"		3.0	2.5
Insulation	R-30 Fiberglass insul.		0.9	0.9
Ceiling	5/8" gypsum		2.8	2.5
Mech & Elec	Mech. & Elec.		2.0	0.0
Misc.	Misc.		0.5	0.0
			0.0	0.0
		Actual Dead Load	14.4	9.7
		Use this DL instead	16.0	9.0
		Live Load	20.0	0.0
		Snow Load	80.1	0.0
		Ultimate Wind (zone 2 - 100sf)	-68.3	-59.9
ASD Loading		D + S	96.1	-
		D + 0.75(0.6*W + S)	45.3	-
		0.6*D + 0.6*W	-	-30.5
LRFD Loading		1.2D + 1.6 S + 0.5W	113.2	-
		1.2D + 1.0W + 0.5S	-9.1	-
		0.9D + 1.0W	-	-51.8

Roof Live Load Reduction Roof angle 3.00 / 12 14.0 deg

0 to 200 sf: 20.0 psf
200 to 600 sf: 24 - 0.02Area, but not less than 12 psf
over 600 sf: 12.0 psf

	300 sf	18.0 psf
	400 sf	16.0 psf
	500 sf	14.0 psf
User Input:	450 sf	15.0 psf

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Floor Design Loads

Items	Description	Multiple	psf (max)	psf (min)
Flooring	Hardwood (Nominal 1")		4.0	3.0
	None	x 1.5	0.0	0.0
Decking	3/4" plywood/OSB		2.7	2.3
Framing	TJI @ 24"	x 1.5	3.0	1.5
Insulation	R-11 Fiberglass insul.		0.4	0.4
Ceiling	5/8" gypsum		2.8	2.5
Mech & Elec	Mech. & Elec.		2.0	0.0
	None		0.0	0.0
Misc.	Misc.		0.5	0.0
		Actual Dead Load	15.4	9.7
		Use this DL instead	16.0	65.0
		Partitions	15.0	0.0
		Live Load	40.0	0.0
		Total Live Load	55.0	0.0
		Total Load	71.0	65.0

FLOOR LIVE LOAD REDUCTION (not including partitions)

NOTE: Not allowed for assembly occupancy or LL>100psf or passenger car garages, except may reduce members supporting 2 or more floors & non-assembly 20%.

$L=Lo(0.25+15/\sqrt{K_{LL}A_T})$

Unreduced design live load: Lo =	40 psf
Floor member & 1 floor cols K_{LL} =	2
Tributary Area A_T =	300 sf
Reduced live load: L =	34.5 psf
Columns (2 or more floors) K_{LL} =	4
Tributary Area A_T =	500 sf
Reduced live load: L =	23.4 psf

IBC alternate procedure

Smallest of:

R= .08%(SF - 150)

R= 23.1(1+D/L) = 32.3%

R= 40% member supports 1 floor

R= 60% member supports ≥2 floors

R =	12.0%
Reduced live load: L =	35.2 psf
R =	28.0%
Reduced live load: L =	28.8 psf

Wall Design Loads

Items	Description	Multiple	psf (max)	psf (min)
Sheathing	7/16" plywood/OSB		1.6	1.4
Sheathing	5/8" gypsum		2.8	2.5
Framing	2x6 wood stud @ 16"		2.0	1.1
veneer			0.0	0.0
Wall Covering	1" Wood Paneling	x 0.38	0.9	0.9
Insulation	R-11 Fiberglass insul.		0.4	0.4
Mech & Elec	Mech. & Elec.		1.0	0.0
Misc.	Misc.		0.5	0.0
		Actual Dead Load	9.2	6.2
		Use this DL instead	10.0	40.0

Wall Design Loads

Items	Description	Multiple	psf (max)	psf (min)
Sheathing	7/16" plywood/OSB		1.6	1.4
Sheathing	5/8" gypsum		2.8	2.5
Framing	6" metal studs @16"		2.5	0.9
veneer	7/8" Stucco		10.0	10.0
		x 0.38	0.0	0.0
Insulation	R-11 Fiberglass insul.		0.4	0.4
Mech & Elec	Mech. & Elec.		1.0	0.0
Misc.	Misc.		0.5	0.0
		Actual Dead Load	18.8	15.2
		Use this DL instead	20.0	40.0

CODE SUMMARY

Code:		International Building Code 2018	
Live Loads:			
Roof	0 to 200 sf:	20 psf	
	200 to 600 sf:	24 - 0.02Area, but not less than 12 psf	
	over 600 sf:	12 psf	
Typical Floor		40 psf	
Partitions		15 psf	
Lobbies & first floor corridors		100 psf	
Corridors above first floor		80 psf	
Balconies (1.5 times live load)		60 psf	
Dead Loads:			
Floor		16.0 psf	
Roof		16.0 psf	
Wind Design Data:			
Ultimate Design Wind Speed		115	mph
Nominal Design Wind Speed		89.08	mph
Risk Category		II	
Mean Roof Ht (h)		48.5 ft	
Exposure Category		C	
Enclosure Classif.		Enclosed Building	
Internal pressure Coef.		+/-0.18	
Directionality (Kd)		0.85	

Roof Snow Loads:			
Design Uniform Roof Snow load		=	80.1 psf
Flat Roof Snow Load	Pf	=	80.1 psf
Balanced Snow Load	Ps	=	80.1 psf
Ground Snow Load	Pg	=	104.0 psf
Importance Factor	I	=	1.00
Snow Exposure Factor	Ce	=	1.10
Thermal Factor	Ct	=	1.00
Sloped-roof Factor	Cs	=	1.00
Drift Surcharge load	Pd	=	
Width of Snow Drift	w	=	
Earthquake Design Data:			
Risk Category		=	II
Importance Factor	I	=	1.00
Mapped spectral response acceleratio	Ss	=	59.70
	S1	=	10.30
Site Class		=	C
Spectral Response Coef.	Sds	=	0.333
	Sd1	=	0.103
Seismic Design Category		=	C
Basic Structural System		=	Bearing Wall Systems
Seismic Resisting System		=	Light frame (wood) walls with structural wood shear panels
Design Base Shear	V	=	0.043W
Seismic Response Coef.	Cs	=	0.043
Response Modification Factor	R	=	6.5
Analysis Procedure		=	Equivalent Lateral-Force Analysis

CODE SUMMARY- continued

Component and cladding ultimate wind pressures

Roof

	Surface Pressure (psf)			
Area	10 sf	50 sf	100 sf	500 sf
Negative Zone 1	-58.5	-47.4	-42.6	-31.4
Negative Zone 2	-83.0	-66.9	-59.9	-43.7
Span A Negative Zone 3	-105.2	-98.3	-95.3	-56.0
Span B,C&D Neg. Zone 3	-68.3	-68.3	-68.3	-51.1
Positive Zone 1	21.6	18.6	17.3	16.0
Positive Zone 2	31.4	26.3	24.1	24.1
'Positive Zone 3	24.1	22.4	21.6	21.6

Parapet

		Solid Parapet Pressure (psf)					
	Area	10 sf	20 sf	50 sf	100 sf	200 sf	500 sf
CASE A:	Zone 2 :	0.0	0.0	0.0	0.0	0.0	0.0
	Span A Zone 3 :	0.0	0.0	0.0	0.0	0.0	0.0
	Span B,C&D Zone 3 :	0.0	0.0	0.0	0.0	0.0	0.0
CASE B :	Interior zone :	0.0	0.0	0.0	0.0	0.0	0.0
	Corner zone :	0.0	0.0	0.0	0.0	0.0	0.0

Wall

	Surface Pressure (psf)			
Area	10 sf	100 sf	200 sf	500 sf
Negative Zone 4	-31.4	-27.1	-25.8	-24.1
Negative Zone 5	-38.8	-30.1	-27.5	-24.1
Positive Zone 4 & 5	29.0	24.7	23.3	21.6

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JOB TITLE BASECAMP STEAMBOAT TOWNHOMES

JOB NO. 　 SHEET NO.
CALCULATED BY 　 DATE
CHECKED BY 　 DATE

Wind Loads - MWFRS all h (Except for Open Buildings)

Kh (case 2) =	1.09	h =	48.5 ft	GCpi =	+/-0.18
Base pressure (q_h) =	**24.6 psf**	ridge ht =	48.5 ft	G =	0.85
Roof Angle (θ) =	14.0 deg	L =	40.0 ft	qi = qh	
Roof tributary area - (h/2)*L:	970 sf	B =	120.0 ft		
(h/2)*B:	2910 sf				

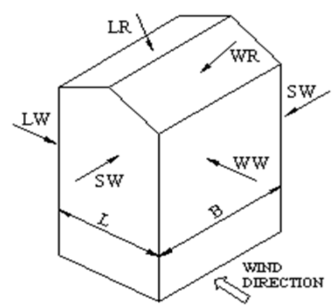
Ultimate Wind Surface Pressures (psf)

Surface	Wind Normal to Ridge				Wind Parallel to Ridge				
	B/L = 3.00		h/L = 0.40		L/B = 0.33			h/L = 1.21	
	Cp	q_hGC_p	w/+q_iGC_{pi}	w/-q_hGC_{pi}	Dist.*	Cp	q_hGC_p	w/ +q_iGC_{pi}	w/ -q_hGC_{pi}
Windward Wall (WW)	0.80	16.7	see table below			0.80	16.7	see table below	
Leeward Wall (LW)	-0.25	-5.2	-9.6	-0.8		-0.50	-10.4	-14.9	-6.0
Side Wall (SW)	-0.70	-14.6	-19.0	-10.2		-0.70	-14.6	-19.0	-10.2
Leeward Roof (LR)	-0.49	-10.1	-14.6	-5.7		Included in windward roof			
Neg Windward Roof pressure	-0.66	-13.8	-18.2	-9.4	0 to h/2*	-1.04	-21.7	-26.1	-17.3
Pos/min Windward Roof press.	-0.12	-2.6	-7.0	1.8	> h/2*	-0.70	-14.6	-19.0	-10.2
					Min press.	-0.18	-3.8	-8.2	0.7

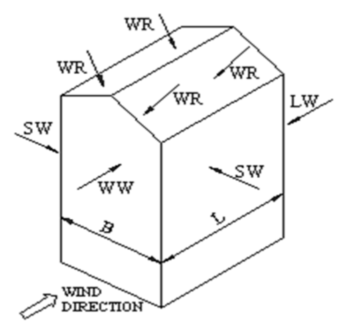
*Horizontal distance from windward edge

Windward Wall Pressures at "z" (psf)

	z	Kz	Kzt	Windward Wall q_zGC_p	w/+q_iGC_{pi}	w/-q_hGC_{pi}	Combined WW + LW Normal to Ridge	Parallel to Ridge
	0 to 15'	0.85	1.00	13.0	8.6	17.5	18.3	23.5
	20.5 ft	0.91	1.00	13.9	9.5	18.4	19.2	24.4
	31.0 ft	0.99	1.00	15.2	10.8	19.6	20.4	25.6
	40.0 ft	1.04	1.00	16.0	11.6	20.5	21.3	26.5
	40.0 ft	1.04	1.00	16.0	11.6	20.5	21.3	26.5
h=	48.5 ft	1.09	1.00	16.7	12.3	21.1	21.9	27.1



WIND NORMAL TO RIDGE



WIND PARALLEL TO RIDGE

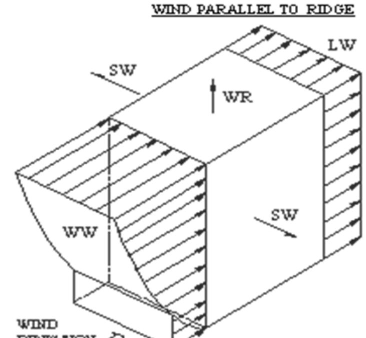
NOTE:
See figure in ASCE7 for the application of full and partial loading of the above wind pressures. There are 4 different loading cases.

Parapet

z	Kz	Kzt	qp (psf)
0.0 ft	0.85	1.00	0.0

Windward parapet: 0.0 psf (GCpn = +1.5)
Leeward parapet: 0.0 psf (GCpn = -1.0)

Windward roof overhangs (add to windward roof pressure) : 16.7 psf (upward)



TYPICAL WIND LOADING

Policy: 2018 ICC Building Code Adoption Seismic Category C

Date: 02/23/2021

The Routt County Regional Building Department has composed a Seismic Design Category C Policy to provide our Professionals with clear information on the adoption of the 2018 ICC International Residential Building Code and the 2018 ICC International Building Code respectively.

Through our Code Adoption Processes within each Jurisdiction including; Routt County, Town of Hayden, Town of Yampa, Town of Oak Creek, and City of Steamboat Springs it was voted and approved that all of Routt County will be considered a Seismic Design Category C in respect to both the IRC and IBC Design standards. No Jurisdictions in Routt County have Adopted nor Accepted the Seismic Design Category D designation that is showing in the 2018 IRC in Figure R301.2(2), nor have we adopted or accepted ASCE 7-16 Design Code Reference Document that will display properties as Seismic Category D designation in certain areas. Please note, both the IRC Map and ASCE 7-16 will display certain properties as a Seismic Design Category D, the map in the 2018 IRC has a dark circle displaying this Seismic Category D designation that is centered over the City of Steamboat Springs, and extends outward into Rural Routt County approximately 17 mile total radius. When working within ASCE 7-16 using web link https://seismicmaps.org/ You will find that properties located in the Town of Oak as an example, would be identified as a Seismic Design Category D designation, this would also be the case for any property you pulled up within the City of Steamboat Springs as well as an example. However, the Routt County Regional Building Department and all Jurisdictions we serve voted this down, and we refused to accept the Seismic Design Category D designation throughout all of Routt County.

Routt County Regional Building Department 2018 IBC Policy Amendment to Section 1613:

2018 IBC Section 1613 Earthquake Loads is hereby amended to read as follows:
1613.1 Scope. Every structure, and portion thereof, including nonstructural components that are permanently attached to structures and their supports and attachments, shall be designed and constructed to resist the effects of earthquake motion and accordance with ASCE 7, excluding Chapter 14 and Appendix 11A. The seismic design category for a structure is permitted to be determined in accordance with Section 1613 or ASCE 7.
Exceptions:

1. Detached one- and two-family dwellings, assigned to Seismic Design Category A, B or C, or located where the mapped short-period spectral response acceleration, SS, is less than 0.4 g.
2. The seismic force-resisting system of wood-frame buildings that conform to the provisions of Section 2308 are not required to be analyzed as specified in this section.

3. Agricultural storage structures intended only for incidental human occupancy.
4. Structures that require special consideration of their response characteristics and environment that are not addressed by this code or ASCE 7 and for which other regulations provide seismic criteria, such as vehicular bridges, electrical transmission towers, hydraulic structures, buried utility lines and their appurtenances and nuclear reactors.

Routt County Building Department Local Policy Amendment to Section 1613 Earth quake Loads: All properties within Routt County Incorporated and Unincorporated Jurisdictions have been adopted and approved to be a Seismic Design Category C designation through our Building Code Adoption Approval Processes. Structures shall be designed in accordance with our local amendment policy using a Seismic Design Category C designation as the base level design standard. When approved by the Structural Engineer of Record through review of the Geotechnical Soils Report and Soils Site Class, the Seismic Category may be reduced by the Engineer of Record based on the known Soils Site Class and in accordance with ASCE-7 and Chapter 16 of the IBC.

Structural Engineers Acceptable Design Parameters Local Routt County Building Department Policy: The Routt County Building Department has developed these design parameters to align with our Local Code Adoptions that were approved designating all of Routt County a Seismic Design Category C. This Policy has been created to provide maximum values for SDS and SD1 respectively to be used in the mapped areas throughout Routt County that have been designated Seismic Category D in accordance ASCE 7-16 USGS Seismic Design Data Map found at https://seismicmaps.org/. The parameters below may be used by Structural Engineers based on the Risk Factor of the Building to perform calculations to determine structural designs. The below parameters may be used with Site Class D- Default (See Section 11.4.3) being set on the ASCE 7-16 USGS Seismic Design Data Map found at https://seismicmaps.org/. Lower values may be used if justified by soil Site Class and resulting site-specific ground motion parameters set forth in ASCE 7-16 and USGS Seismic Design Data Map and approved by the Code Official.

- **Risk Category I, II, and II Building: SDS = 0.333 and SD1 = 0.133**
- **Risk Category IV Building: SDS = 0.499 and SD1 = 0.199**

The intent of setting these parameters and values is to help support Structural Engineers in designing buildings within the spirt of our Locally Approved Code Adoptions designating a standard Seismic Design Category C throughout all of Routt County, to avoid conflicts in what data would otherwise be provided through ASCE 7-16 USGS Seismic Design Data Map found at https://seismicmaps.org/.

Routt County Regional Building Department 2018 IRC Code Adoption

Table R301.2(1) CLIMATIC AND GEOGRAPHIC DESIGN CRITERIA, is completed as follows:

- Ground Snow Load – Case Study Area contact the Building Department for Ground Snow Load Valuations per site.
- Climate Zone 7
- Wind Speed – 115 MPH (ultimate design wind speed)
- Topographic Effects – No
- Seismic Design Category – C Note: When approved by the Structural Engineer of Record through review of the Geotechnical Soils Report and Soils Site Class, the Seismic Category may be reduced

by the Engineer of Record based on the known Soils Site Class and in accordance with ASCE-7 and Chapter 16 of the IBC.

- Subject to Damage by Weathering – Severe
- Subject to Damage by Frost line Depth – 48 inches (1220mm)
- Subject to Damage by Termite – None to slight
- Subject to Damage by Decay – None to slight
- Winter Design:
    - Outdoor Winter Design Dry-Bulb Temperature – -15ºF (-26ºC)
    - Indoor Winter Design Dry-Bulb Temperature: 70° F (21° C)
    - Coincident Wet Bulb: 56° F (13° C)
    - Heating temperature Difference: 85° F (29° C)
- Summer Design:
    - Outdoor Summer Design Dry-Bulb Temperature: 85° F (29° C)
    - Indoor Summer Design Dry-Bulb Temperature: 75° F (24° C)
    - Design Grains: Varies based on weather data Range: -35 to -55
    - Cooling Temperature Difference: 10° F (-12° C)
- Elevation: Varies   Elevation by address can be found at: https://elevation.maplogs.com/poi/routt_county_co_usa.12879.html
- Altitude Correction: Varies
    - 7,000’ 0.77
    - 8,000’ 0.75
    - 9,000’ 0.72
    - 10,000’ 0.69
    - 12,000’ 0.63
- Latitude : 40° North
- Ice Shield Underlayment Required – Yes
- Flood Hazards – FIRM, February 4, 2005
- Air Freezing Index – Steamboat 2239
- Mean Annual Temperature – 40-45ºF (4.5-7.2ºC)
- Ground Snow Load Values are Governed by Routt County Regional Building Department based on geographic location. Please visit our home page and click on Ground Snow Load Values for site-specific information.

Sincerely,

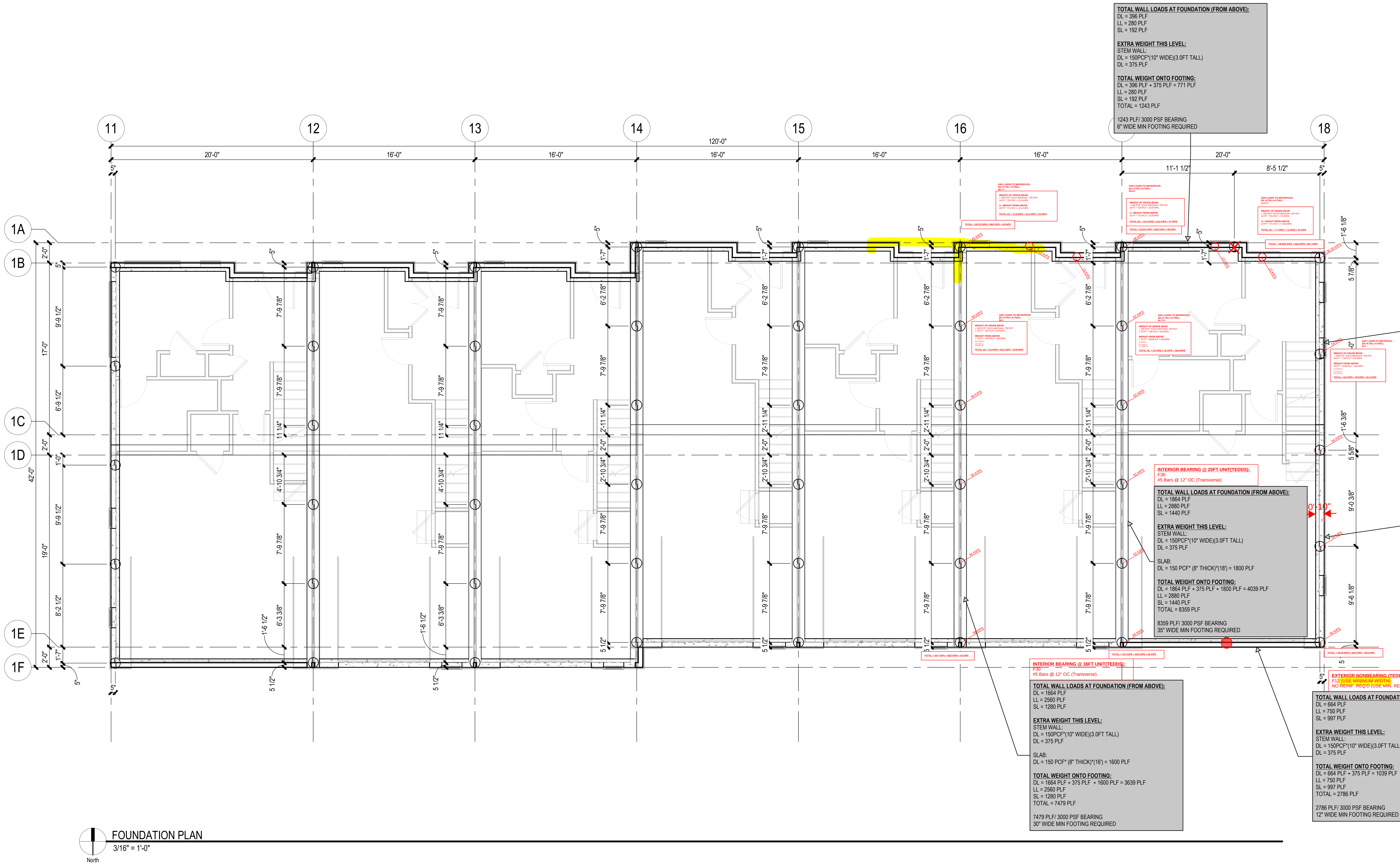
Todd Carr

Todd Carr, Building Official
Routt County Building Department

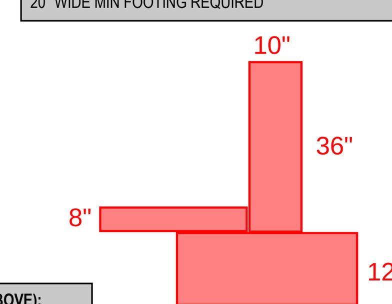
GRAVITY DESIGN

GRAVITY LOAD TRACE

APPROVAL STAMPS:



FOUNDATION PLAN
3/16" = 1'-0"
North



CONCRETE FOOTING SCHEDULE (ISO PADS)

TAG	LENGTH	WIDTH	THICKNESS	REINFORCEMENT

CONCRETE FOOTING SCHEDULE (CONT)

TAG	WIDTH	THICKNESS	REINFORCEMENT

8/3/2022 12:26:10 PM

FOUNDATION PLAN NOTES:

1. SEE S0.01 FOR GENERAL STRUCTURAL NOTES, ABBREVIATIONS KEY AND LEGEND.
2. SEE S5.01 FOR TYPICAL FOUNDATION DETAILS
3. SEE S6.01 FOR SCHEDULES
4. VIF (E) CONDITIONS PRIOR TO NEW CONSTRUCTION AND NOTIFY ANTHEM OF ANY DISCREPANCIES
5. FOOTINGS TO BEAR ON APPROVED SUBGRADE PER GEOTECH REPORT. FOUNDATION DESIGNED BASED ON X,XXX PSF ALLOWABLE BEARING PRESSURE, XXX PSF DEAD LOAD MINIMUM BEARING PRESSURE
6. FLOOR FRAMING AND SHEATHING TO BE INSTALLED PRIOR TO FULL FOUNDATION BACKFILLING.
7. DO NOT BACKFILL BASEMENT WALLS UNTIL 7 DAYS AFTER BASEMENT SLAB INSTALL
8. SEE ARCHITECTURAL DRAWINGS FOR LOCATIONS OF RAMPS, SLAB SLOPES, STEPPED SLABS, AND PARTITION WALLS. SLAB ELEVATIONS SHOWN ARE APPROXIMATE.
9. REFER TO ARCHITECTURAL DRAWINGS FOR ADDITIONAL INFORMATION AND DIMENSIONS.
10. **TYPICAL CONCRETE WALL (UNO):** X" WALLS: #X @ X" EACH WAY, CENTERED IN WALL; (2) #X BARS TOP AND BOT HORIZ. FOOTING DOWELS TO MATCH SIZE AND SPACING OF VERTICAL WALL REINFORCING. 1/2" Øx12" ANCHOR BOLTS AND WASHERS TO WOOD WALL ABOVE W/ 5" PROJECTION @32" AND 8" MAX FROM CORNERS AND STEPS IN WALL.
11. **CONCRETE SLAB ON GRADE (UNO):**
    A. X" THICK CONCRETE SLAB ON PREPARED SUB-GRADE PER SOILS REPORT. REINFORCE WITH #X @XX" EACH WAY AT MID-DEPTH. SAWCUT OR TOOLED 1/8" x X" CONTROL JOINTS @XX-X" MAX EACH WAY. INSTALL (3) #4 x 5'-0 DIAGONAL BARS AT MID-DEPTH OF SLAB AT ALL RE-ENTRANT CORNERS
    B. X" THICK CONCRETE SLAB ON PREPARED SUB-GRADE PER SOILS REPORT. REINFORCE WITH XxX - WX.X x WX.X WWF PLACED AT MID DEPTH. SAWCUT OR TOOLED 1/8" x X" CONTROL JOINTS @XX-X" MAX EACH WAY. INSTALL (3) #4 x 5'-0 DIAGONAL BARS AT MID-DEPTH OF SLAB AT ALL RE-ENTRANT CORNERS
    C. X" THICK CONCRETE SLAB ON PREPARED SUB-GRADE PER SOILS REPORT. REINFORCE WITH MICRO FIBER REINFORCEMENT IN SLAB MIX (MIN 1.5 LBS PER CUBIC YARD). INSTALL (3) #4 x 5'-0 DIAGONAL BARS AT MID-DEPTH OF SLAB AT ALL RE-ENTRANT CORNERS
12. FLOAT INTERIOR PARTITION WALLS W/ MIN 1 1/2" AIR GAP, SEE ARCH PLANS FOR DETAIL
13. HDX INDICATES HOLDOWNS LOCATED AT THE TOP OF THE ... ND
14. ...): X" ... H X" @ X" ... (3) #4 x ... LL RE- ... JOINTS

No.	Date	Description

SUBMISSIONS & REVISIONS

OWNER

ARCHITECT

K A S A

KEVIN & ASAKO SPERRY ARCHITECTURE
3318 N. Columbus Street
Arlington, VA 22207
T.312.636.3248 / 312.636.4252
www.kasa-arch.com

GENERAL CONTRACTOR

CIVIL ENGINEER

LANDSCAPE ARCHITECT

anthem structural engineers
303-848-8497 970-300-3338
anthemstructural.com Job #22-048

M.E.P. & F.P. ENGINEERS

INTERIOR DESIGNER:

PROJECT LOCATION

BASECAMP TOWNHOME
1950 CURVE COURT
STEAMBOAT SPRINGS, CO 80487

DRAWING TITLE

TH1 FOUNDATION PLAN

SEAL

DATE: 05/31/19
DRAWN BY:
CHECKED BY:
PROJECT NO: 22-048

DRAWING NO:
S0200-TH1

GRAVITY LOADS

WALLS:
DL = 10 PSF

LOADS TO WALL LINE:
LEVEL 4:
DECK:
DL = 16 PSF * (2 FT) = 32 PLF
LL = 60 PSF * (2 FT) = 120 PLF
SL = 96 PSF * (2 FT) = 192 PLF

LEVEL 3:
FLOOR:
DL = 16 PSF * (2 FT) = 32 PLF
LL = 40 PSF * (2 FT) = 80 PLF

WALLS:
DL = 10 PSF * (2 FT) = 20 PLF

LEVEL 2:
FLOOR:
DL = 16 PSF * (2 FT) = 32 PLF
LL = 40 PSF * (2 FT) = 80 PLF

WALLS:
DL = 10 PSF * (2 FT) = 20 PL

LEVEL 1:
WALLS:
DL = 10 PSF * (2 FT) = 20 PL

LOADS TO WALL LINE:

LEVEL 4:
DECK:
DL = 16 PSF * (10 FT) = 160 PLF
LL = 60 PSF * (10 FT) = 600 PLF
SL = 170 PSF * (10 FT) = 1700 PLF

WALLS:
DL = 10 PSF * (10 FT) = 100 PLF

LEVEL 3:
FLOOR:
DL = 16 PSF * (10 FT) = 160 PLF
LL = 40 PSF * (10 FT) = 400 PLF

WALLS:
DL = 10 PSF * (10 FT) = 100 PL

LEVEL 2:
FLOOR:
DL = 16 PSF * (10 FT) = 160 PLF
LL = 40 PSF * (10 FT) = 400 PLF

WALLS:
DL = 10 PSF * (10 FT) = 100 PL

LEVEL 1:
WALLS:
DL = 10 PSF * (10 FT) = 100 PL

LOADS TO WALL LINE:

ROOF:
DL = 16 PSF*(10 FT) = 160 PLF
SL = 80 PSF * (10 FT) = 800 PLF

LEVEL 4:
FLOOR:
DL = 16 PSF * (10 FT) = 160 PLF
LL = 40 PSF * (10 FT) = 400 PLF

WALLS:
DL = 10 PSF * (10 FT) = 100 PLF

LEVEL 3:
FLOOR:
DL = 16 PSF * (10 FT) = 160 PLF
LL = 40 PSF * (10 FT) = 400 PLF

WALLS:
DL = 10 PSF * (10 FT) = 100 PL

LEVEL 2:
FLOOR:
DL = 16 PSF * (10 FT) = 160 PLF
LL = 40 PSF * (10 FT) = 400 PLF

WALLS:
DL = 10 PSF * (10 FT) = 100 PL

LEVEL 1:
WALLS:
DL = 10 PSF * (10 FT) = 100 PL

LOADS TO WALL LINE:

ROOF:
DL = 16 PSF*(2 FT) = 32 PLF
SL = 80 PSF * (2 FT) = 160 PLF

LEVEL 4:
FLOOR:
DL = 16 PSF * (2 FT) = 32 PLF
LL = 40 PSF * (2 FT) = 80 PLF

WALLS:
DL = 10 PSF * (2 FT) = 20 PLF

LEVEL 3:
FLOOR:
DL = 16 PSF * (2 FT) = 32 PLF
LL = 40 PSF * (2 FT) = 80 PLF

WALLS:
DL = 10 PSF * (2 FT) = 20 PLF

LEVEL 2:
FLOOR:
DL = 16 PSF * (2 FT) = 32 PLF
LL = 40 PSF * (2 FT) = 80 PLF

BALCONY:
DL = 16 PSF * (8.5 FT) = 136 PLF
LL = 60 PSF * (8.5 FT) = 510 PLF
SL = 186 PSF * (4.5 FT) = 837 PLF

WALLS:
DL = 10 PSF * (2 FT) = 20 PLF

LEVEL 1:
WALLS:
DL = 10 PSF * (2 FT) = 20 PL

LOADS TO WALL LINE:

ROOF:
DL = 16 PSF*(16 FT) = 256 PLF
SL = 80 PSF * (16 FT) = 1280 PLF

LEVEL 4:
FLOOR:
DL = 16 PSF * (16 FT) = 256 PLF
LL = 40 PSF * (16 FT) = 640 PLF

WALLS:
DL = 10 PSF * (14 FT) = 160 PLF

LEVEL 3:
FLOOR:
DL = 16 PSF * (16 FT) = 256 PLF
LL = 40 PSF * (16 FT) = 640 PLF

WALLS:
DL = 10 PSF * (16 FT) = 160 PLF

LEVEL 2:
FLOOR:
DL = 16 PSF * (16 FT) = 256 PLF
LL = 40 PSF * (16 FT) = 640 PLF

WALLS:
DL = 10 PSF * (16 FT) = 160 PLF

LEVEL 1:
WALLS:
DL = 10 PSF * (16 FT) = 160 PL

LOADS TO WALL LINE:

ROOF:
DL = 16 PSF*(18 FT) = 288 PLF
SL = 80 PSF * (18 FT) = 1440 PLF

LEVEL 4:
FLOOR:
DL = 16 PSF * (18 FT) = 288 PLF
LL = 40 PSF * (18 FT) =720 PLF

WALLS:
DL = 10 PSF * (18 FT) = 180 PLF

LEVEL 3:
FLOOR:
DL = 16 PSF * (18 FT) = 288 PLF
LL = 40 PSF * (18 FT) =720 PLF

WALLS:
DL = 10 PSF * (18 FT) = 180 PLF

LEVEL 2:
FLOOR:
DL = 16 PSF * (18 FT) = 288 PLF
LL = 40 PSF * (18 FT) =720 PLF

WALLS:
DL = 10 PSF * (18 FT) = 180 PLF

LEVEL 1:
WALLS:
DL = 10 PSF * (18 FT) = 180 PL

1 TH1 LEVEL 1
A0300A0200 3/16" = 1'-0"

5/28/2022 12:25:33 AM

APPROVAL STAMPS:

No.	Date	Description

SUBMISSIONS & REVISIONS

OWNER

MAY RIEGLER PROPERTIES
2201 WISCONSIN AVE NW
SUITE 200
WASHINGTON, DC 20007

ARCHITECT

K A S A

KEVIN & ASAKO SPERRY ARCHITECTURE
3318 N. Columbus Street
Arlington, VA 22207
T.312.636.3248 / 312.636.4252
www.kasa-arch.com

GENERAL CONTRACTOR

CIVIL ENGINEER

LANDMARK CONSULTANTS, INC.
141 9TH STREET
PO BOX 774943
STEAMBOAT SPRINGS, CO 80477

LANDSCAPE ARCHITECT

STRUCTURAL ENGINEER

ANTHEM STRUCTURAL ENGINEERS
430 YAMPA ST
STEAMBOAT SPRINGS, CO 80487

M.E.P. & F.P. ENGINEERS

BOULDER ENGINEERING
1717 15TH STREET
BOULDER, CO 80302

INTERIOR DESIGNER:

PROJECT LOCATION

BASECAMP TOWNHOME
1950 CURVE COURT
STEAMBOAT SPRINGS, CO 80487

DRAWING TITLE

TH1 LEVEL 1 PLAN

SEAL

DATE:
05/27/22

DRAWN BY:

CHECKED BY:

PROJECT NO:

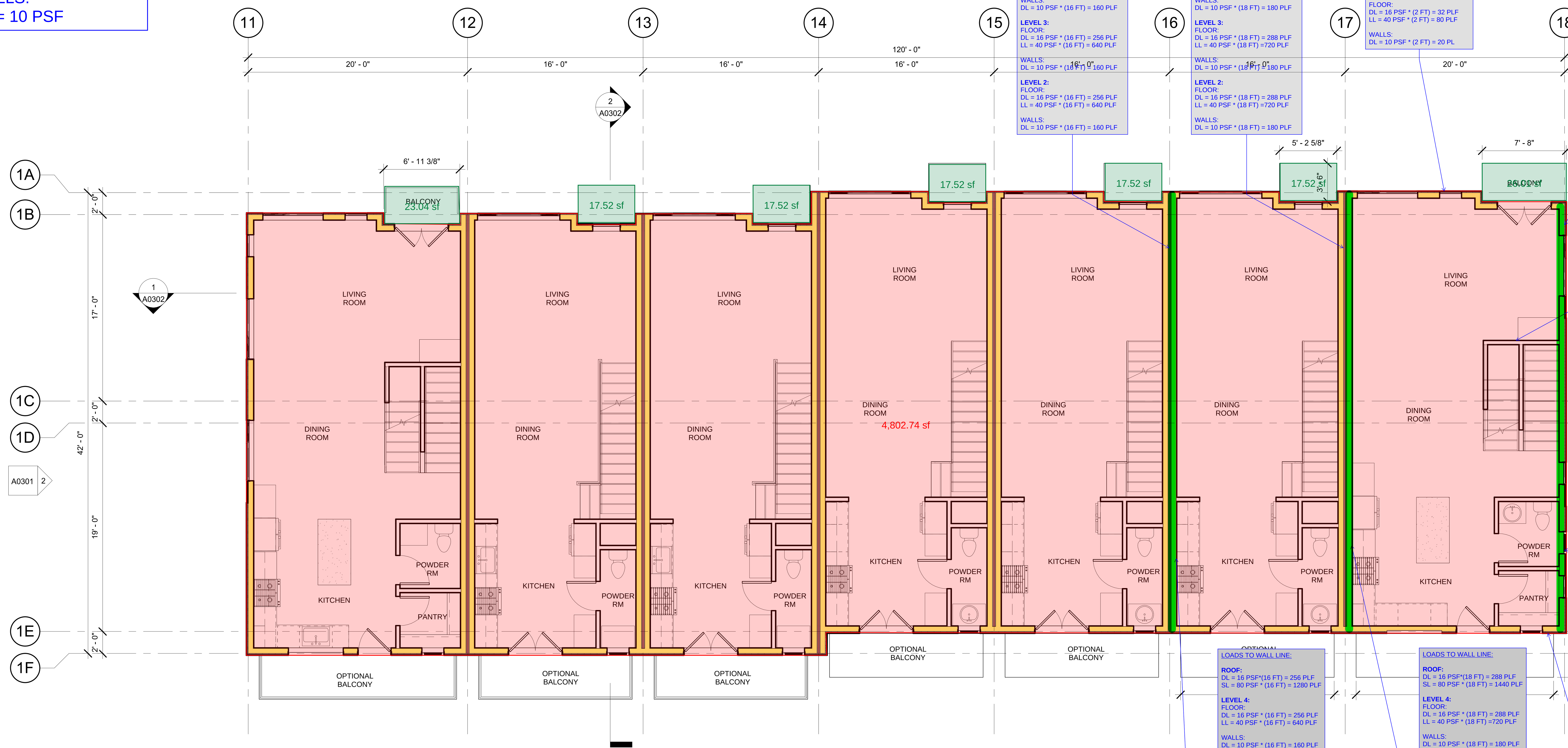
DRAWING NO:
A0200

GRAVITY LOADS

BALCONY:
DL = 16 PSF
LL = 60 PSF
SL = 186 PSF

FLOOR:
DL = 16 PSF
LL = 40 PSF

WALLS:
DL = 10 PSF



LOADS TO WALL LINE:
LEVEL 4:
DECK:
DL = 16 PSF * (16 FT) = 256 PLF
LL = 60 PSF * (16 FT) = 960 PLF
SL = 170 PSF * (16 FT) = 2720 PLF

WALLS:
DL = 10 PSF * (16 FT) = 160 PLF

LEVEL 3:
FLOOR:
DL = 16 PSF * (16 FT) = 256 PLF
LL = 40 PSF * (16 FT) = 640 PLF

WALLS:
DL = 10 PSF * (16 FT) = 160 PLF

LEVEL 2:
FLOOR:
DL = 16 PSF * (16 FT) = 256 PLF
LL = 40 PSF * (16 FT) = 640 PLF

WALLS:
DL = 10 PSF * (16 FT) = 160 PLF

LOADS TO WALL LINE:
LEVEL 4:
DECK:
DL = 16 PSF * (18 FT) = 288 PLF
LL = 60 PSF * (18 FT) = 1080 PLF
SL = 170 PSF * (18 FT) = 3060 PLF

WALLS:
DL = 10 PSF * (18 FT) = 180 PLF

LEVEL 3:
FLOOR:
DL = 16 PSF * (18 FT) = 288 PLF
LL = 40 PSF * (18 FT) =720 PLF

WALLS:
DL = 10 PSF * (18 FT) = 180 PLF

LEVEL 2:
FLOOR:
DL = 16 PSF * (18 FT) = 288 PLF
LL = 40 PSF * (18 FT) =720 PLF

WALLS:
DL = 10 PSF * (18 FT) = 180 PLF

LOADS TO WALL LINE:
LEVEL 4:
DECK:
DL = 16 PSF * (2 FT) = 32 PLF
LL = 60 PSF * (2 FT) = 120 PLF
SL = 96 PSF * (2 FT) = 192 PLF

LEVEL 3:
FLOOR:
DL = 16 PSF * (2 FT) = 32 PLF
LL = 40 PSF * (2 FT) = 80 PLF

WALLS:
DL = 10 PSF * (2 FT) = 20 PLF

LEVEL 2:
FLOOR:
DL = 16 PSF * (2 FT) = 32 PLF
LL = 40 PSF * (2 FT) = 80 PLF

WALLS:
DL = 10 PSF * (2 FT) = 20 PL

LOADS TO WALL LINE:

LEVEL 4:
DECK:
DL = 16 PSF * (10 FT) = 360 PLF
LL = 60 PSF * (10 FT) = 600 PLF
SL = 170 PSF * (10 FT) = 1700 PLF

WALLS:
DL = 10 PSF * (10 FT) = 100 PLF

LEVEL 3:
FLOOR:
DL = 16 PSF * (10 FT) = 160 PLF
LL = 40 PSF * (10 FT) = 400 PLF

WALLS:
DL = 10 PSF * (10 FT) = 100 PL

LEVEL 2:
FLOOR:
DL = 16 PSF * (10 FT) = 160 PLF
LL = 40 PSF * (10 FT) = 400 PLF

WALLS:
DL = 10 PSF * (10 FT) = 100 PL

LOADS TO WALL LINE:

ROOF:
DL = 16 PSF*(2 FT) = 32 PLF
SL = 80 PSF * (2 FT) = 160 PLF

LEVEL 4:
FLOOR:
DL = 16 PSF * (2 FT) = 32 PLF
LL = 40 PSF * (2 FT) = 80 PLF

DECK:
DL = 16 PSF * (2 FT) = 32 PLF
LL = 60 PSF * (2 FT) = 120 PLF
SL = 170 PSF * (2 FT) = 340 PLF

WALLS:
DL = 10 PSF * (2 FT) = 20 PLF

LEVEL 3:
FLOOR:
DL = 16 PSF * (2 FT) = 32 PLF
LL = 40 PSF * (2 FT) = 80 PLF

WALLS:
DL = 10 PSF * (2 FT) = 20 PLF

LEVEL 2:
FLOOR:
DL = 16 PSF * (2 FT) = 32 PLF
LL = 40 PSF * (2 FT) = 80 PLF

WALLS:
DL = 10 PSF * (2 FT) = 20 PLF

LOADS TO WALL LINE:

ROOF:
DL = 16 PSF*(10 FT) = 160 PLF
SL = 80 PSF * (10 FT) = 800 PLF

LEVEL 4:
FLOOR:
DL = 16 PSF * (10 FT) = 160 PLF
LL = 40 PSF * (10 FT) = 400 PLF

WALLS:
DL = 10 PSF * (10 FT) = 100 PLF

LEVEL 3:
FLOOR:
DL = 16 PSF * (10 FT) = 160 PLF
LL = 40 PSF * (10 FT) = 400 PLF

WALLS:
DL = 10 PSF * (10 FT) = 100 PL

LEVEL 2:
FLOOR:
DL = 16 PSF * (10 FT) = 160 PLF
LL = 40 PSF * (10 FT) = 400 PLF

WALLS:
DL = 10 PSF * (10 FT) = 100 PL

LOADS TO WALL LINE:

ROOF:
DL = 16 PSF*(16 FT) = 256 PLF
SL = 80 PSF * (16 FT) = 1280 PLF

LEVEL 4:
FLOOR:
DL = 16 PSF * (16 FT) = 256 PLF
LL = 40 PSF * (16 FT) = 640 PLF

WALLS:
DL = 10 PSF * (16 FT) = 160 PLF

LEVEL 3:
FLOOR:
DL = 16 PSF * (16 FT) = 256 PLF
LL = 40 PSF * (16 FT) = 640 PLF

WALLS:
DL = 10 PSF * (16 FT) = 160 PLF

LEVEL 2:
FLOOR:
DL = 16 PSF * (16 FT) = 256 PLF
LL = 40 PSF * (16 FT) = 640 PLF

WALLS:
DL = 10 PSF * (16 FT) = 160 PLF

LOADS TO WALL LINE:

ROOF:
DL = 16 PSF*(18 FT) = 288 PLF
SL = 80 PSF * (18 FT) = 1440 PLF

LEVEL 4:
FLOOR:
DL = 16 PSF * (18 FT) = 288 PLF
LL = 40 PSF * (18 FT) =720 PLF

WALLS:
DL = 10 PSF * (18 FT) = 180 PLF

LEVEL 3:
FLOOR:
DL = 16 PSF * (18 FT) = 288 PLF
LL = 40 PSF * (18 FT) =720 PLF

WALLS:
DL = 10 PSF * (18 FT) = 180 PLF

LEVEL 2:
FLOOR:
DL = 16 PSF * (18 FT) = 288 PLF
LL = 40 PSF * (18 FT) =720 PLF

WALLS:
DL = 10 PSF * (18 FT) = 180 PLF

LOADS TO WALL LINE:

ROOF:
DL = 16 PSF*(2 FT) = 32 PLF
SL = 80 PSF * (2 FT) = 160 PLF

LEVEL 4:
FLOOR:
DL = 16 PSF * (2 FT) = 32 PLF
LL = 40 PSF * (2 FT) = 80 PLF

WALLS:
DL = 10 PSF * (2 FT) = 20 PLF

LEVEL 3:
FLOOR:
DL = 16 PSF * (2 FT) = 32 PLF
LL = 40 PSF * (2 FT) = 80 PLF

WALLS:
DL = 10 PSF * (2 FT) = 20 PLF

LEVEL 2:
FLOOR:
DL = 16 PSF * (2 FT) = 32 PLF
LL = 40 PSF * (2 FT) = 80 PLF

BALCONY:
DL = 16 PSF * (8.5 FT) = 136 PLF
LL = 60 PSF * (8.5 FT) = 510 PLF
SL = 186 PSF * (4.5 FT) = 837 PLF

WALLS:
DL = 10 PSF * (2 FT) = 20 PLF

1 TH1 LEVEL 2
A0300 A0201 3/16" = 1'-0"

5/28/2022 12:25:33 AM

APPROVAL STAMPS:

No.	Date	Description

SUBMISSIONS & REVISIONS

OWNER
MAY RIEGLER PROPERTIES
2201 WISCONSIN AVE NW
SUITE 200
WASHINGTON, DC 20007

ARCHITECT
K A S A
KEVIN & ASAKO SPERRY ARCHITECTURE
3318 N. Columbus Street
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GENERAL CONTRACTOR

CIVIL ENGINEER
LANDMARK CONSULTANTS, INC.
141 9TH STREET
PO BOX 774943
STEAMBOAT SPRINGS, CO 80477

LANDSCAPE ARCHITECT

STRUCTURAL ENGINEER
ANTHEM STRUCTURAL ENGINEERS
430 YAMPA ST
STEAMBOAT SPRINGS, CO 80487

M.E.P. & F.P. ENGINEERS
BOULDER ENGINEERING
1717 15TH STREET
BOULDER, CO 80302

INTERIOR DESIGNER:

PROJECT LOCATION
BASECAMP TOWNHOME
1950 CURVE COURT
STEAMBOAT SPRINGS, CO 80487

DRAWING TITLE
TH1 LEVEL 2 PLAN

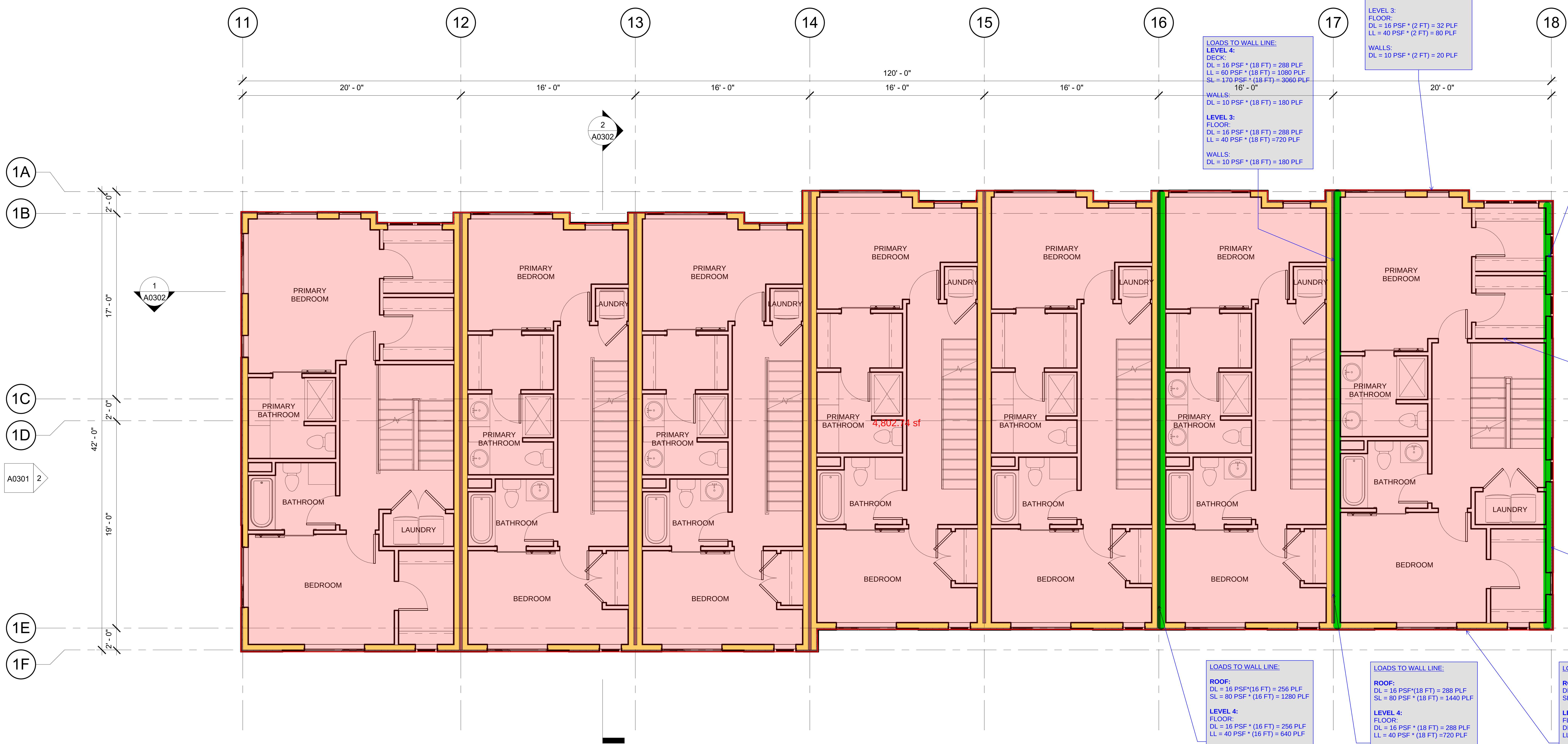
SEAL

DATE: 05/27/22
DRAWN BY:
CHECKED BY:
PROJECT NO:

DRAWING NO:
A0201

GRAVITY LOADS
FLOOR:
DL = 16 PSF
LL = 40 PSF

WALLS:
DL = 10 PSF



LOADS TO WALL LINE:
LEVEL 4:
DECK:
DL = 16 PSF * (2 FT) = 32 PLF
LL = 60 PSF * (2 FT) = 120 PLF
SL = 96 PSF * (2 FT) = 192 PLF

LEVEL 3:
FLOOR:
DL = 16 PSF * (2 FT) = 32 PLF
LL = 40 PSF * (2 FT) = 80 PLF

WALLS:
DL = 10 PSF * (2 FT) = 20 PLF

LOADS TO WALL LINE:
LEVEL 4:
DECK:
DL = 16 PSF * (18 FT) = 288 PLF
LL = 60 PSF * (18 FT) = 1080 PLF
SL = 170 PSF * (18 FT) = 3060 PLF

WALLS:
DL = 10 PSF * (18 FT) = 180 PLF

LEVEL 3:
FLOOR:
DL = 16 PSF * (18 FT) = 288 PLF
LL = 40 PSF * (18 FT) =720 PLF

WALLS:
DL = 10 PSF * (18 FT) = 180 PLF

LOADS TO WALL LINE:

LEVEL 4:
DECK:
DL = 16 PSF * (10 FT) = 360 PLF
LL = 60 PSF * (10 FT) = 600 PLF
SL = 170 PSF * (10 FT) = 1700 PLF

WALLS:
DL = 10 PSF * (10 FT) = 100 PLF

LEVEL 3:
FLOOR:
DL = 16 PSF * (10 FT) = 160 PLF
LL = 40 PSF * (10 FT) = 400 PLF

WALLS:
DL = 10 PSF * (10 FT) = 100 PL

LOADS TO WALL LINE:

ROOF:
DL = 16 PSF*(2 FT) = 32 PLF
SL = 80 PSF * (2 FT) = 160 PLF

LEVEL 4:
FLOOR:
DL = 16 PSF * (2 FT) = 32 PLF
LL = 40 PSF * (2 FT) = 80 PLF

DECK:
DL = 16 PSF * (2 FT) = 32 PLF
LL = 60 PSF * (2 FT) = 120 PLF
SL = 170 PSF * (2 FT) = 340 PLF

WALLS:
DL = 10 PSF * (2 FT) = 20 PLF

LEVEL 3:
FLOOR:
DL = 16 PSF * (2 FT) = 32 PLF
LL = 40 PSF * (2 FT) = 80 PLF

WALLS:
DL = 10 PSF * (2 FT) = 20 PLF

LOADS TO WALL LINE:

ROOF:
DL = 16 PSF*(10 FT) = 160 PLF
SL = 80 PSF * (10 FT) = 800 PLF

LEVEL 4:
FLOOR:
DL = 16 PSF * (10 FT) = 160 PLF
LL = 40 PSF * (10 FT) = 400 PLF

WALLS:
DL = 10 PSF * (10 FT) = 100 PLF

LEVEL 3:
FLOOR:
DL = 16 PSF * (10 FT) = 160 PLF
LL = 40 PSF * (10 FT) = 400 PLF

WALLS:
DL = 10 PSF * (10 FT) = 100 PL

LOADS TO WALL LINE:

ROOF:
DL = 16 PSF*(16 FT) = 256 PLF
SL = 80 PSF * (16 FT) = 1280 PLF

LEVEL 4:
FLOOR:
DL = 16 PSF * (16 FT) = 256 PLF
LL = 40 PSF * (16 FT) = 640 PLF

WALLS:
DL = 10 PSF * (16 FT) = 160 PLF

LEVEL 3:
FLOOR:
DL = 16 PSF * (16 FT) = 256 PLF
LL = 40 PSF * (16 FT) = 640 PLF

WALLS:
DL = 10 PSF * (16 FT) = 160 PLF

LOADS TO WALL LINE:

ROOF:
DL = 16 PSF*(18 FT) = 288 PLF
SL = 80 PSF * (18 FT) = 1440 PLF

LEVEL 4:
FLOOR:
DL = 16 PSF * (18 FT) = 288 PLF
LL = 40 PSF * (18 FT) =720 PLF

WALLS:
DL = 10 PSF * (18 FT) = 180 PLF

LEVEL 3:
FLOOR:
DL = 16 PSF * (18 FT) = 288 PLF
LL = 40 PSF * (18 FT) =720 PLF

WALLS:
DL = 10 PSF * (18 FT) = 180 PLF

LOADS TO WALL LINE:

ROOF:
DL = 16 PSF*(2 FT) = 32 PLF
SL = 80 PSF * (2 FT) = 160 PLF

LEVEL 4:
FLOOR:
DL = 16 PSF * (2 FT) = 32 PLF
LL = 40 PSF * (2 FT) = 80 PLF

WALLS:
DL = 10 PSF * (2 FT) = 20 PLF

LEVEL 3:
FLOOR:
DL = 16 PSF * (2 FT) = 32 PLF
LL = 40 PSF * (2 FT) = 80 PLF

WALLS:
DL = 10 PSF * (2 FT) = 20 PLF

1 TH1 LEVEL 3
A0300A0202 3/16" = 1'-0"

5/28/2022 12:25:34 AM

APPROVAL STAMPS:

No.	Date	Description

SUBMISSIONS & REVISIONS

OWNER
MAY RIEGLER PROPERTIES
2201 WISCONSIN AVE NW
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M.E.P. & F.P. ENGINEERS
BOULDER ENGINEERING
1717 15TH STREET
BOULDER, CO 80302

INTERIOR DESIGNER:

PROJECT LOCATION
BASECAMP TOWNHOME
1950 CURVE COURT
STEAMBOAT SPRINGS, CO 80487

DRAWING TITLE
TH1 LEVEL 3 PLAN

SEAL

DATE: 05/27/22
DRAWN BY:
CHECKED BY:
PROJECT NO:

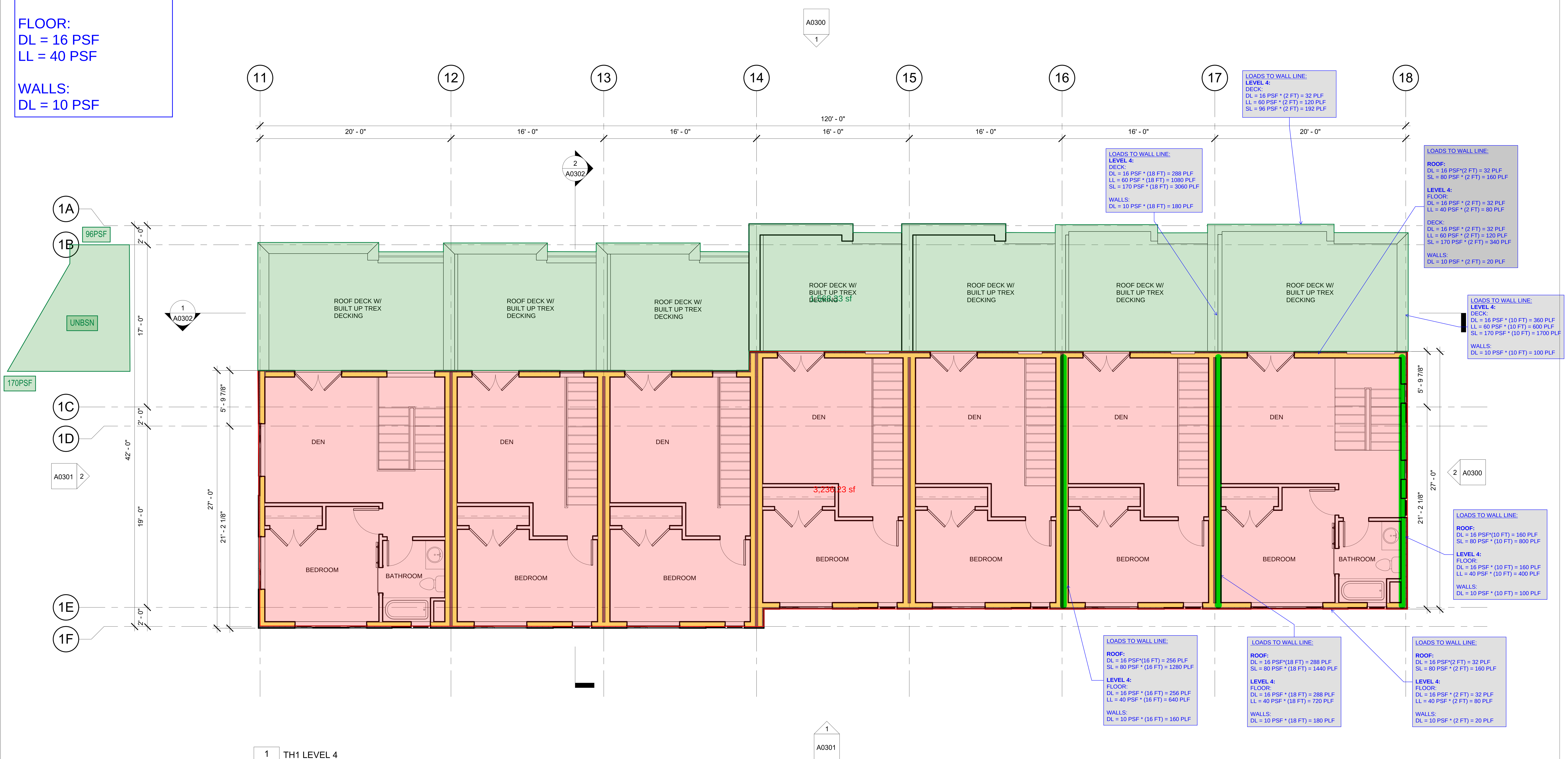
DRAWING NO:
A0202

GRAVITY LOADS

DECK:
DL = 16 PSF
LL = 60 PSF
UNBALANCED SNOW
SL_{LOW} = 96.0 PSF
SL_{HIGH} = 170 PSF

FLOOR:
DL = 16 PSF
LL = 40 PSF

WALLS:
DL = 10 PSF



1 TH1 LEVEL 4
A0300A0203 3/16" = 1'-0"

APPROVAL STAMPS:

No.	Date	Description

SUBMISSIONS & REVISIONS

OWNER

MAY RIEGLER PROPERTIES
2201 WISCONSIN AVE NW
SUITE 200
WASHINGTON, DC 20007

ARCHITECT

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1717 15TH STREET
BOULDER, CO 80302

INTERIOR DESIGNER:

PROJECT LOCATION

BASECAMP TOWNHOME
1950 CURVE COURT
STEAMBOAT SPRINGS, CO 80487

DRAWING TITLE

TH1 LEVEL 4 PLAN

SEAL

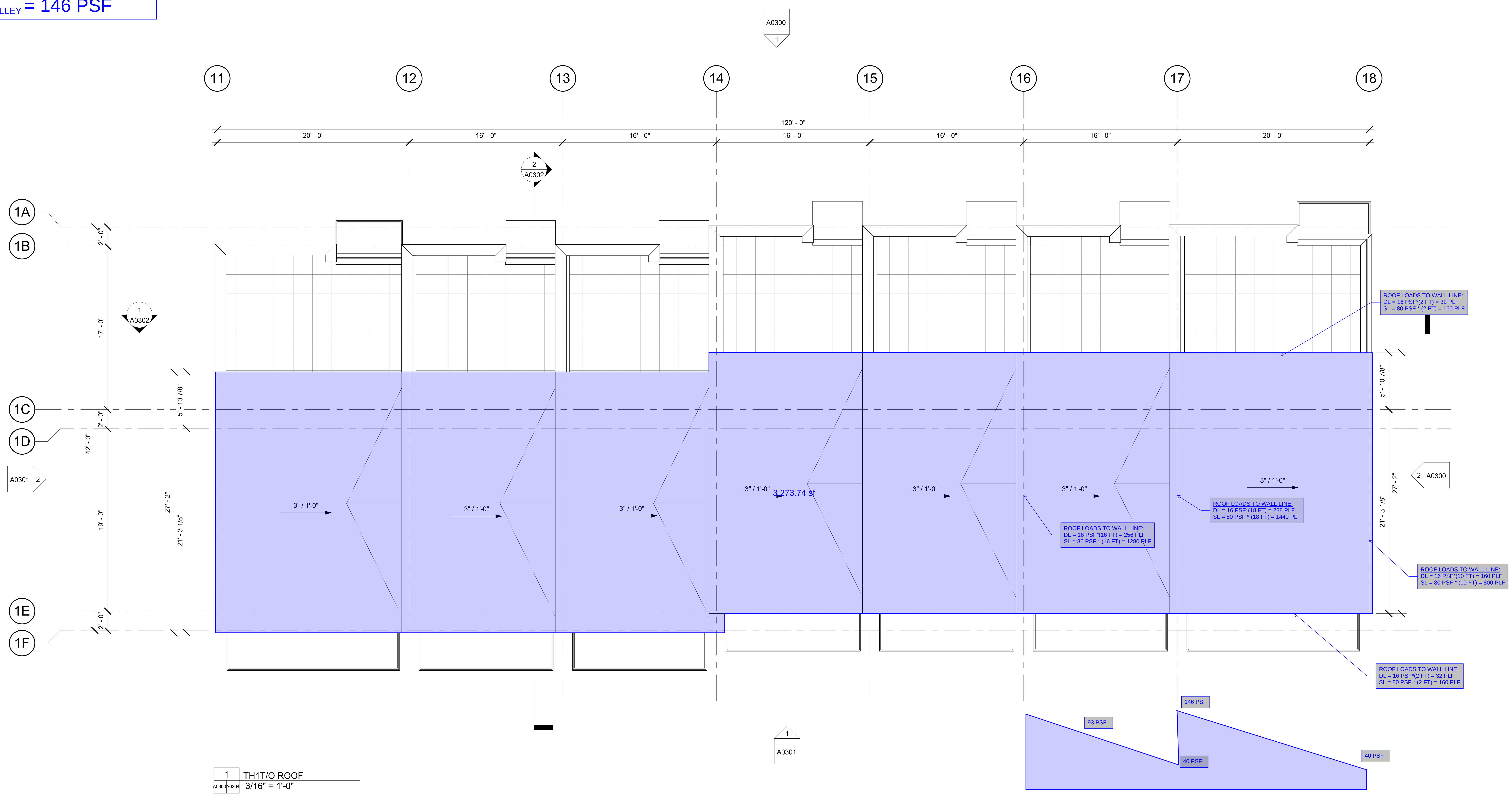
DATE:
05/27/22
DRAWN BY:
CHECKED BY:
PROJECT NO:

DRAWING NO:
A0203

5/28/2022 12:25:34 AM

GRAVITY LOADS
ROOF:
DL = 16 PSF

UNBALANCED SNOW
$SL_{RIDGE/CROWN}$ = 40.0 PSF
SL_{VALLEY} = 146 PSF



1 TH1T/O ROOF
A0300A0204 3/16" = 1'-0"

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APPROVAL STAMPS:

No.	Date	Description

SUBMISSIONS & REVISIONS

OWNER
MAY RIEGLER PROPERTIES
2201 WISCONSIN AVE NW
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BOULDER ENGINEERING
1717 15TH STREET
BOULDER, CO 80302

INTERIOR DESIGNER:

PROJECT LOCATION
BASECAMP TOWNHOME
1950 CURVE COURT
STEAMBOAT SPRINGS, CO 80487

DRAWING TITLE
TH1 ROOF PLAN

SEAL

DATE:
05/27/22
DRAWN BY:
CHECKED BY:
PROJECT NO:

DRAWING NO:
A0204

UNIT TYPE:

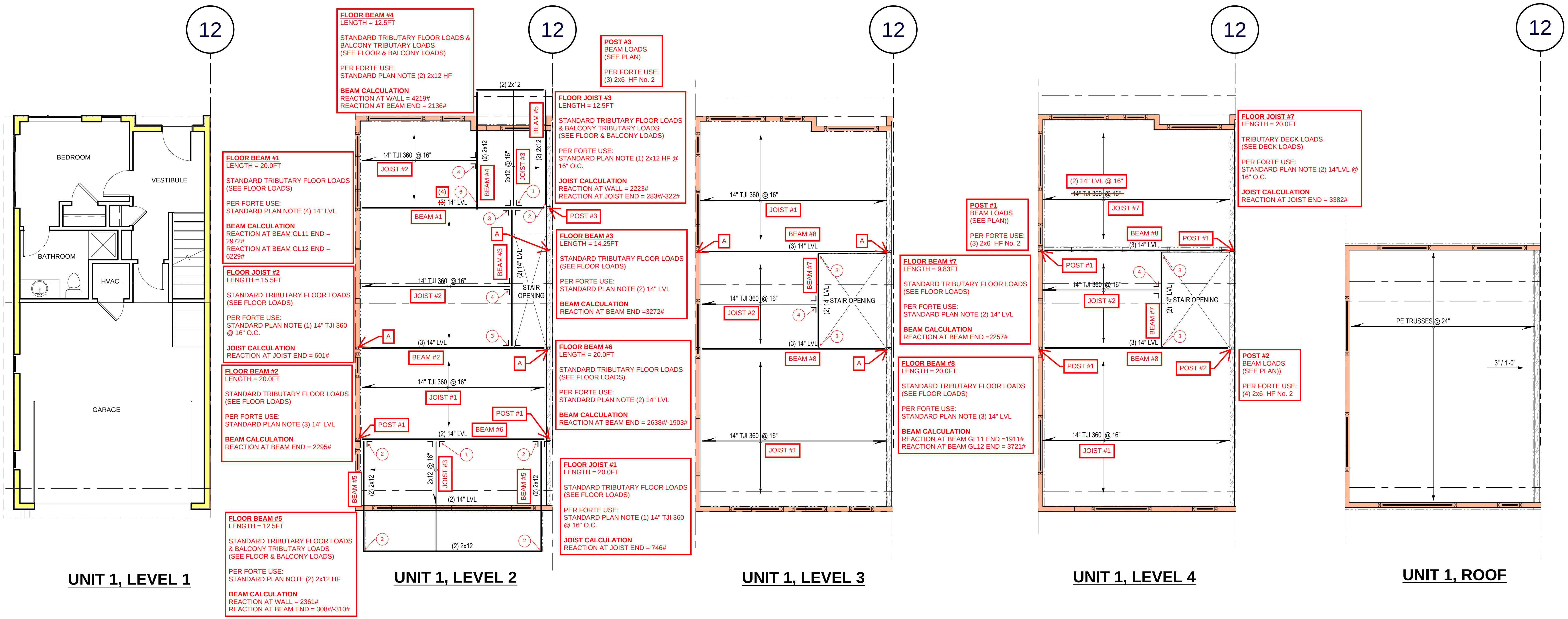
1

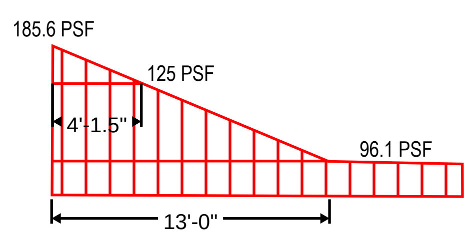
GRAVITY LOADS
BALCONY:
DL = 16 PSF
LL = 60 PSF
SL = 186 PSF

DECK:
DL = 16 PSF + SLAB
LL = 60 PSF
SL = 96 PSF + DRIFT

FLOOR:
DL = 16 PSF
LL = 40 PSF

WALLS:
DL = 10 PSF





BALCONY SNOW LOAD

HANGER SCHEDULE

1. ALL HANGERS NOTED TO BE INSTALLED WITH NUMBER AND SIZE FASTENERS SPECIFIED BY MNFR. ANY SUBSTITUTIONS SHALL BE REVIEWED AND APPROVED BY ANTHEM

2. INSTALL HANGERS NOTED OR APPROVED EQUIVALENT

MARK	DESCRIPTION	FACE FASTENERS	JOIST FASTENERS
1	LUS28	(6) 10d X 3" NAILS	(4) 10d X 1.5" NAILS
2	LUS28-2	(6) 10d X 3" NAILS	(4) 10d X 3" NAILS
3	HHUS410	(30) 10d X 3" NAILS	(10) 10d X 3" NAILS
4	ISU2.37/14	(12) 10d X 3" NAILS	-
5	IUS1.81/14	(12) 10d X 1.5" NAILS	-
6	U210-2	(14) 16d X 3.5" NAILS	(6) 10d X 3" Nails

UNIT TYPE:

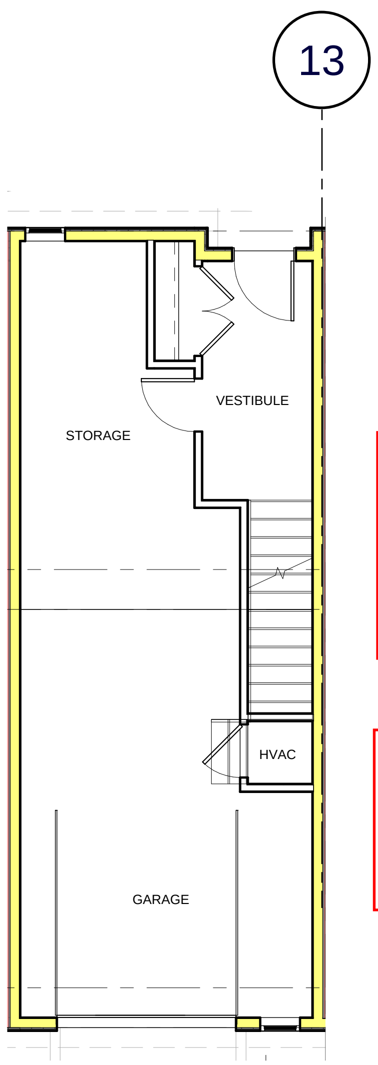
2

GRAVITY LOADS
BALCONY:
DL = 16 PSF
LL = 60 PSF
SL = 176 PSF

DECK:
DL = 16 PSF + SLAB
LL = 60 PSF
SL = 96 PSF + DRIFT

FLOOR:
DL = 16 PSF
LL = 40 PSF

WALLS:
DL = 10 PSF



UNIT 2, LEVEL 1

FLOOR JOIST #5
LENGTH = 9.5FT

STANDARD TRIBUTARY FLOOR LOADS
(SEE FLOOR LOADS)

PER FORTE USE:
STANDARD PLAN NOTE (1) 14" TJI 110 @ 16" O.C.

JOIST CALCULATION
REACTION AT JOIST END = 356#

FLOOR BEAM #9
LENGTH = 16.0FT

STANDARD TRIBUTARY FLOOR LOADS
(SEE FLOOR LOADS)

PER FORTE USE:
STANDARD PLAN NOTE (2) 14" LVL

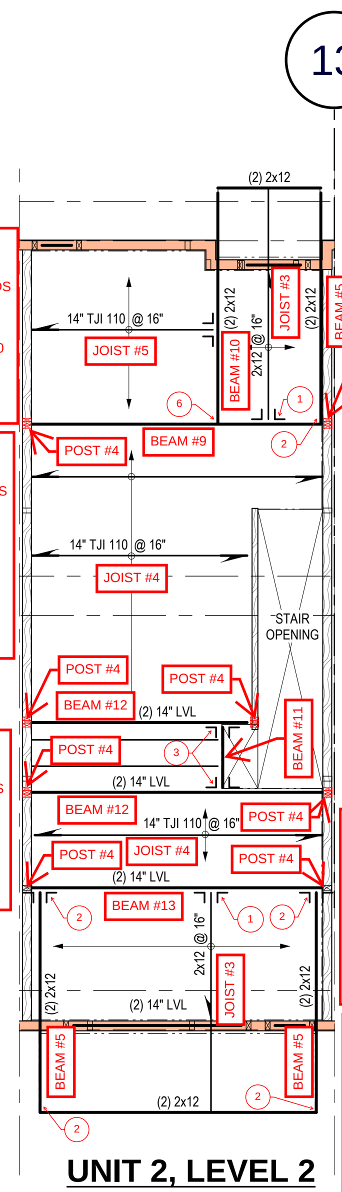
BEAM CALCULATION
REACTION AT BEAM GL12 END = 1276#
REACTION AT BEAM GL13 END = 2381#/-834#

FLOOR BEAM #11
LENGTH = 3.5FT

STANDARD TRIBUTARY FLOOR LOADS
(SEE FLOOR LOADS)

PER FORTE USE:
STANDARD PLAN NOTE (2) 14" LVL

BEAM CALCULATION
REACTION AT BEAM END = 585#



UNIT 2, LEVEL 2

POST #4
BEAM LOADS
(SEE PLAN)

PER FORTE USE:
(3) 2x6 HF No. 2

FLOOR BEAM #10
LENGTH = 12.5FT

STANDARD TRIBUTARY FLOOR LOADS
(SEE FLOOR LOADS)

PER FORTE USE:
STANDARD PLAN NOTE (2) 14" LVL

BEAM CALCULATION
REACTION AT WALL = 3474#
REACTION AT BEAM END = 1437#

FLOOR BEAM #12
LENGTH = 16.0FT MAX

STANDARD TRIBUTARY FLOOR LOADS
(SEE FLOOR LOADS)

PER FORTE USE:
STANDARD PLAN NOTE (2) 14" LVL

BEAM CALCULATION
REACTION AT BEAM END = 2547#

FLOOR JOIST #4
LENGTH = 16.0FT

STANDARD TRIBUTARY FLOOR LOADS
(SEE FLOOR LOADS)

PER FORTE USE:
STANDARD PLAN NOTE (1) 14" TJI 110 @ 16" O.C.

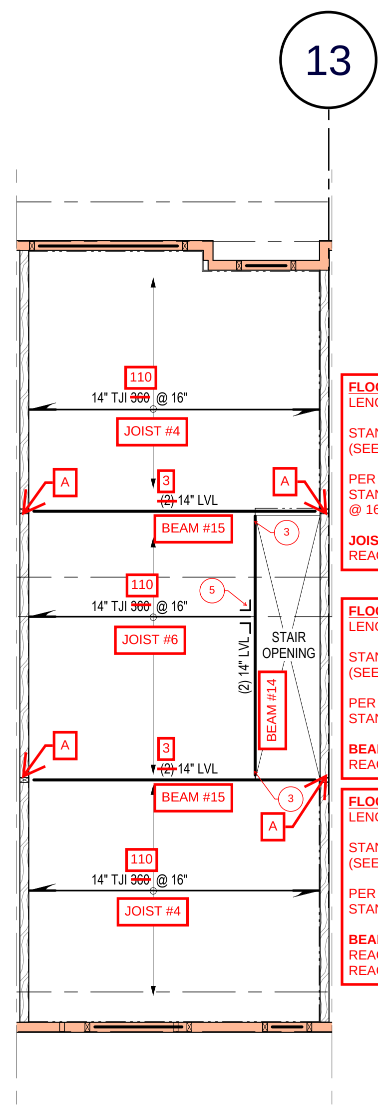
JOIST CALCULATION
REACTION AT JOIST END = 598#

FLOOR BEAM #13
LENGTH = 16.0FT

STANDARD TRIBUTARY FLOOR LOADS
(SEE FLOOR LOADS)

PER FORTE USE:
STANDARD PLAN NOTE (2) 14" LVL

BEAM CALCULATION
REACTION AT BEAM END = 2109#/-1523#



UNIT 2, LEVEL 3

FLOOR JOIST #6
LENGTH = 11.5FT

STANDARD TRIBUTARY FLOOR LOADS
(SEE FLOOR LOADS)

PER FORTE USE:
STANDARD PLAN NOTE (1) 14" TJI 110 @ 16" O.C.

JOIST CALCULATION
REACTION AT JOIST END = 431#

FLOOR BEAM #14
LENGTH = 16.0FT

STANDARD TRIBUTARY FLOOR LOADS
(SEE FLOOR LOADS)

PER FORTE USE:
STANDARD PLAN NOTE (2) 14" LVL

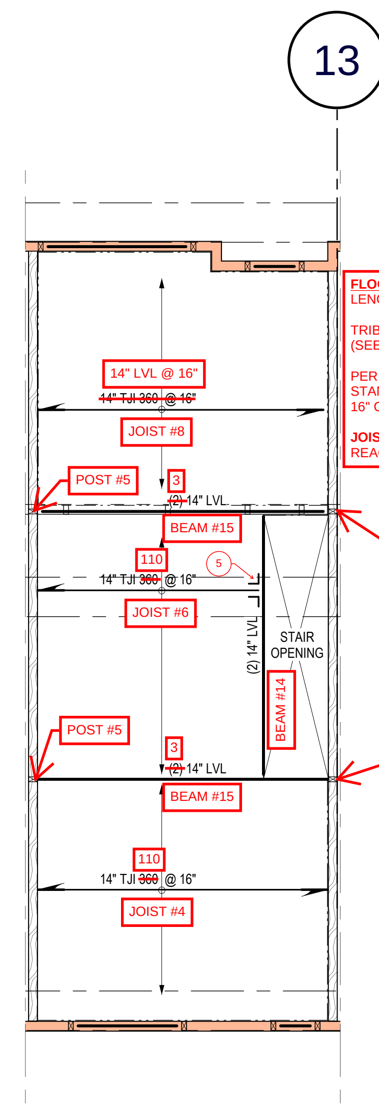
BEAM CALCULATION
REACTION AT BEAM END = 2291#

FLOOR BEAM #15
LENGTH = 16.0FT

STANDARD TRIBUTARY FLOOR LOADS
(SEE FLOOR LOADS)

PER FORTE USE:
STANDARD PLAN NOTE (3) 14" LVL

BEAM CALCULATION
REACTION AT BEAM END = 1622#
REACTION AT BEAM GL13 END = 3908#



UNIT 2, LEVEL 4

FLOOR JOIST #8
LENGTH = 16.0FT

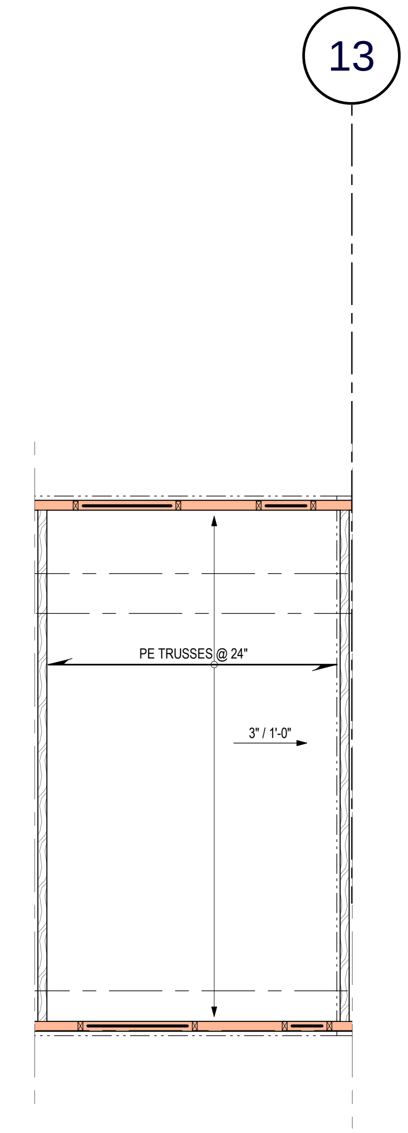
TRIBUTARY DECK LOADS
(SEE DECK LOADS)

PER FORTE USE:
STANDARD PLAN NOTE (1) 14" LVL @ 16" O.C.

JOIST CALCULATION
REACTION AT JOIST END = 2721#

POST #5
BEAM LOADS
(SEE PLAN)

PER FORTE USE:
(3) 2x6 HF No. 2



UNIT 2, ROOF

UNIT TYPE:

3

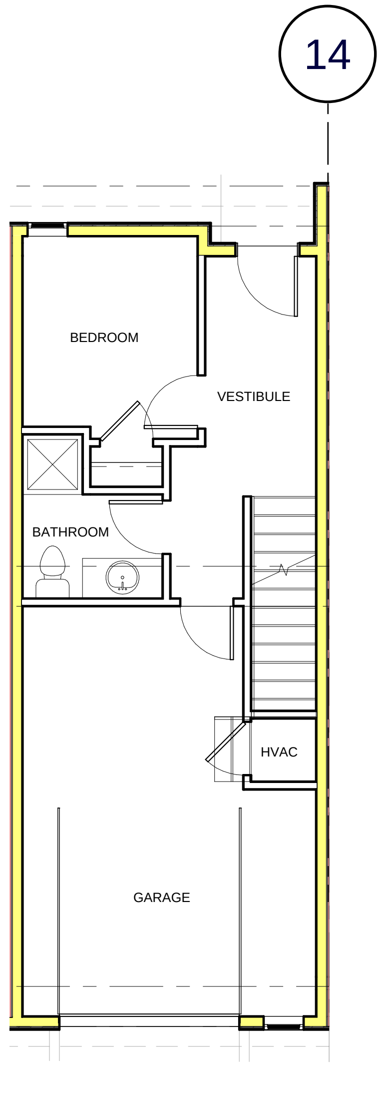
GRAVITY LOADS

BALCONY:
DL = 16 PSF
LL = 60 PSF
SL = 176 PSF

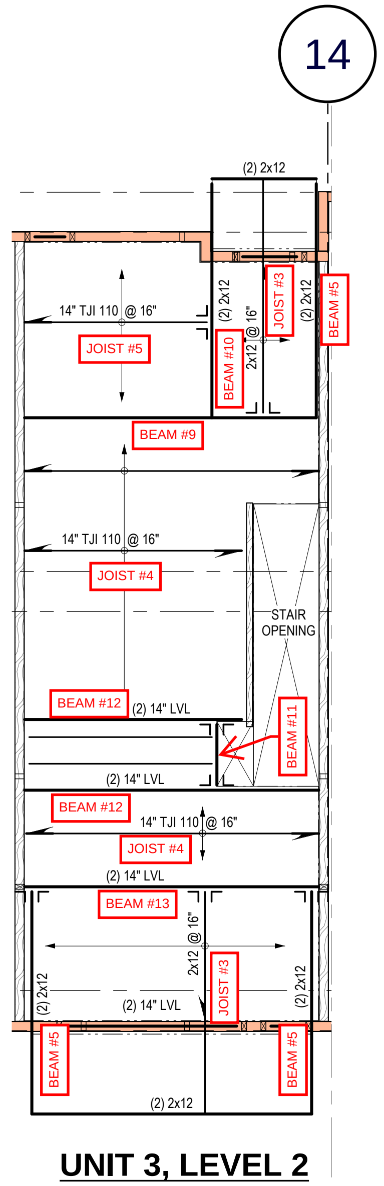
DECK:
DL = 16 PSF + SLAB
LL = 60 PSF
SL = 96 PSF + DRIFT

FLOOR:
DL = 16 PSF
LL = 40 PSF

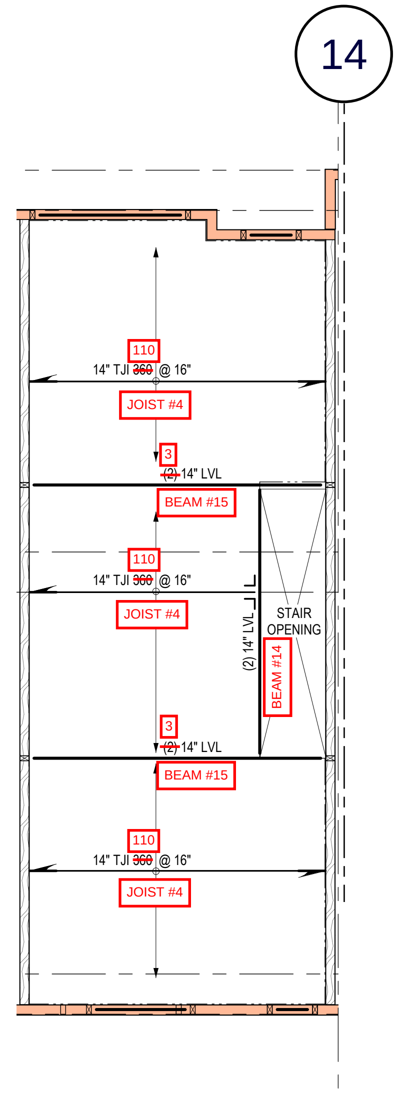
WALLS:
DL = 10 PSF



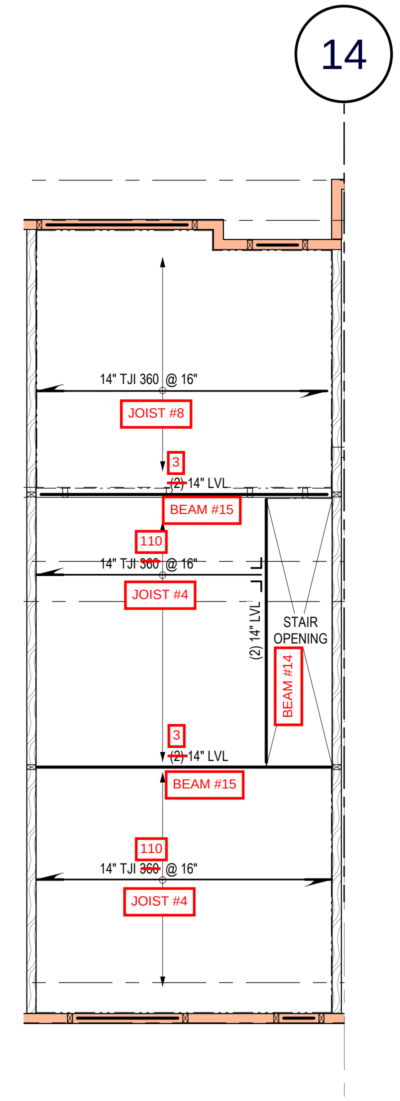
UNIT 3, LEVEL 1



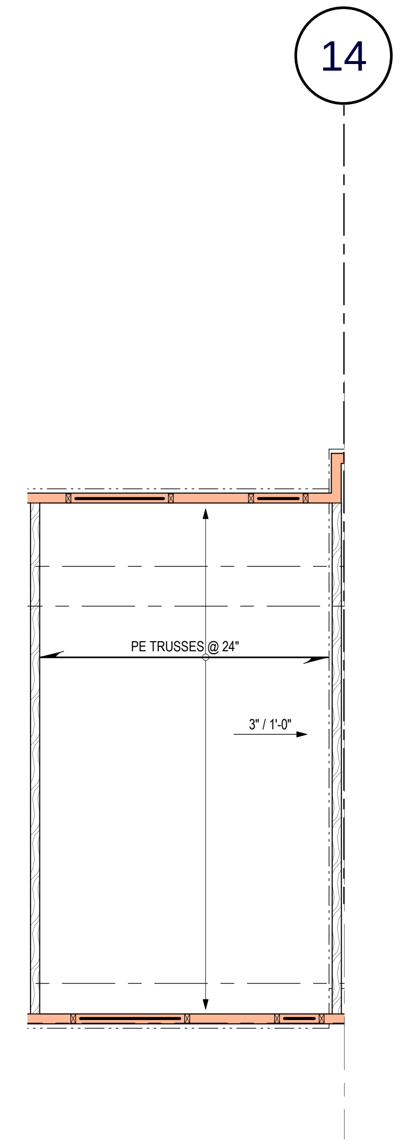
UNIT 3, LEVEL 2



UNIT 3, LEVEL 3



UNIT 3, LEVEL 4



UNIT 3, ROOF

UNIT TYPE:

4

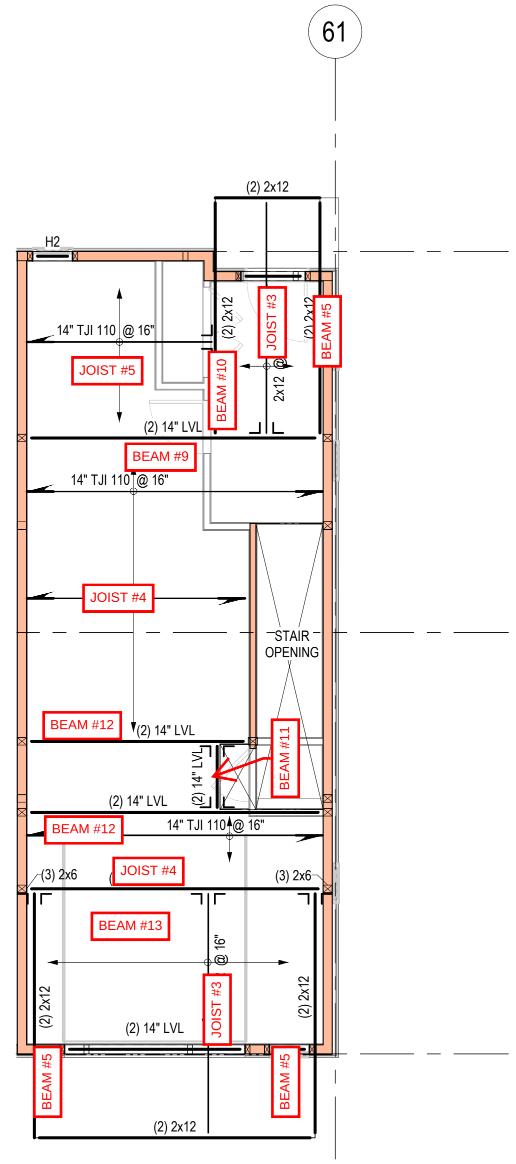
GRAVITY LOADS
BALCONY:
DL = 16 PSF
LL = 60 PSF
SL = 176 PSF

DECK:
DL = 16 PSF + SLAB
LL = 60 PSF
SL = 96 PSF + DRIFT

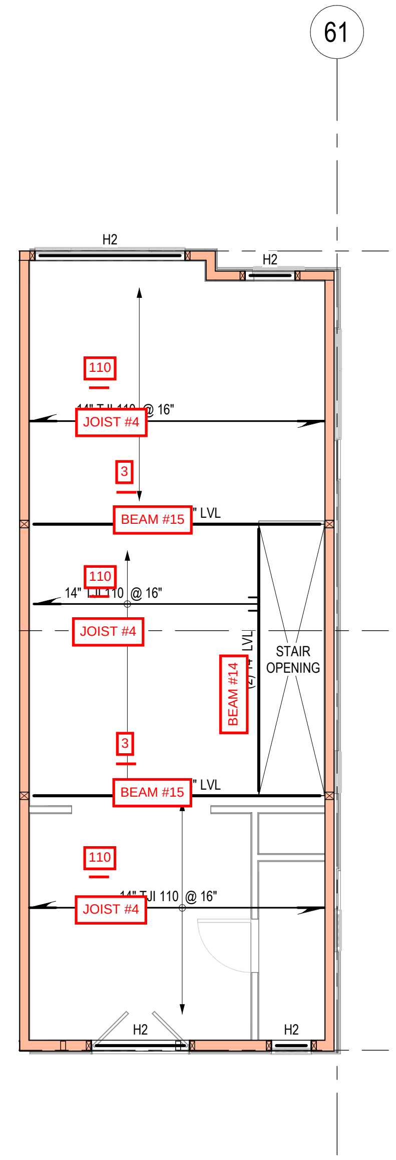
FLOOR:
DL = 16 PSF
LL = 40 PSF

WALLS:
DL = 10 PSF

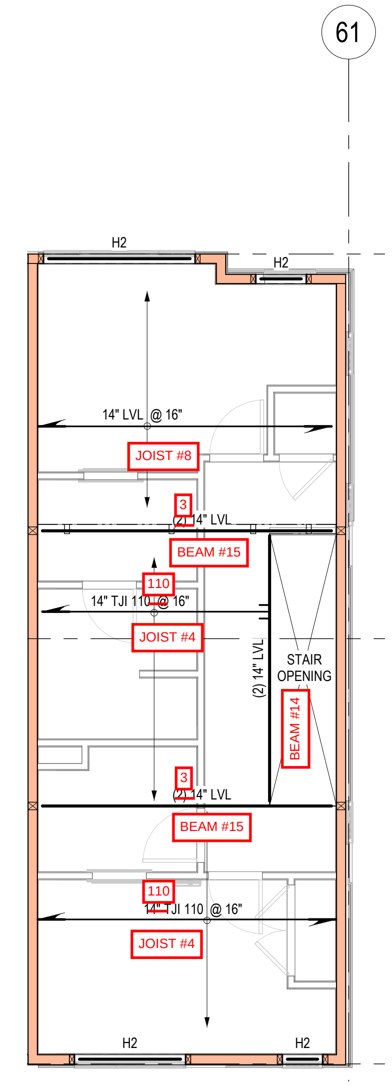
UNIT 4, LEVEL 1



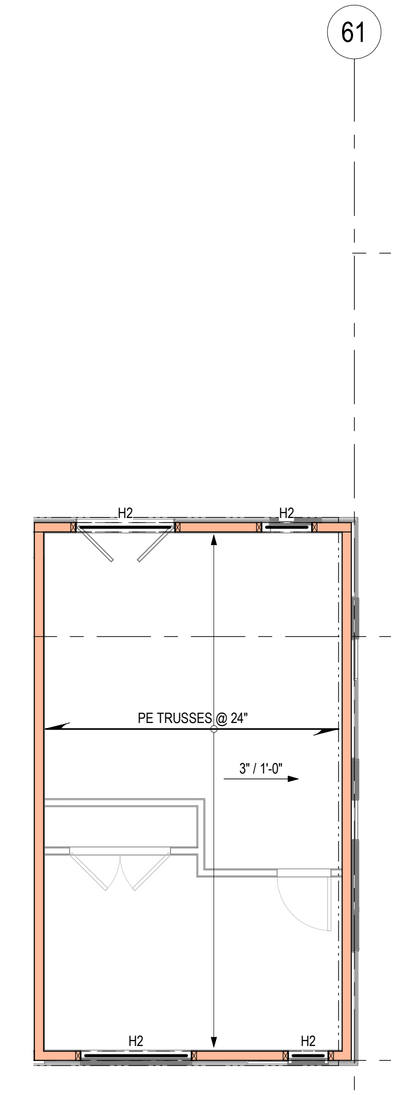
UNIT 4, LEVEL 2



UNIT 4, LEVEL 3



UNIT 4, LEVEL 4



UNIT 4, ROOF

LEVEL 2 FRAMING PLAN

3/16" = 1'-0"

North

LOADS TO WALL LINE:

ROOF:
DL = 16 PSF*(1 FT) = 16 PLF
SL = 80 PSF * (1 FT) = 81 PLF

LEVEL 4:
FLOOR:
DL = 16 PSF * (1 FT) = 16 PLF
LL = 40 PSF * (1 FT) = 40 PLF

WALLS:
DL = 10 PSF * (10 FT) = 100 PLF

LEVEL 3:
FLOOR:
DL = 16 PSF * (1 FT) = 16 PLF
LL = 40 PSF * (1 FT) = 40 PLF

WALLS:
DL = 10 PSF * (10 FT) = 100 PLF

LEVEL 2:
FLOOR:
DL = 16 PSF * (1 FT) = 16 PLF
LL = 40 PSF * (1 FT) = 40 PLF

BALCONY:
DL = 16 PSF * (8.5 FT) = 136 PLF
LL = 60 PSF * (8.5 FT) = 510 PLF
SL = 186 PSF * (4.5 FT) = 837 PLF

WALLS:
DL = 10 PSF * (10 FT) = 100 PLF

TOTAL:
DL = 500 PLF
LL = 630 PLF
SL = 917 PLF

6/23/2022 2:05:29 PM

FRAMING LEVEL PLAN NOTES:

1. SEE S0.01 FOR GENERAL STRUCTURAL NOTES, ABBREVIATIONS KEY AND LEGEND
2. SEE S5.01 FOR TYPICAL DETAILS
3. SEE S6.01 FOR SCHEDULES
4. REFER TO ARCHITECTURAL DRAWINGS FOR ADDITIONAL INFORMATION AND DIMENSIONS.
5. VIF (E) CONDITIONS PRIOR TO NEW CONSTRUCTION AND NOTIFY ANTHEM OF ANY DISCREPANCIES
6. TOP OF SUB-FLOOR SHEATHING =XXX'-X" UNO
7. TOP OF PLATE HEIGHT = XXX'-X" UNO
8. WALL FRAMING AND COLUMNS SHOWN SUPPORT THE FRAMING ON THIS LEVEL
9. **FLOOR CONSTRUCTION (UNO):** X" LIGHTWEIGHT CONCRETE TOPPING OVER 3/4" STURD-I-FLOOR APA RATED @ 24" OC TONGUE AND GROOVE SHEATHING, OVER WOOD I-JOISTS / TRUSSES W/ 1 1/4" LSL RIM, SEE PLAN. GLUE AND FASTEN SHEATHING TO JOISTS, RIM, FLUSH BEAMS, AND LEDGERS WITH 8d GUN NAILS (0.113"Ø x 2 3/8") @ 4" O.C. AND @ 8" O.C. ALONG INTERMEDIATE FRAMING MEMBERS. LAY PANELS PERPENDICULAR TO FRAMING MEMBERS AND STAGGER PANEL JOINTS.
10. **EXTERIOR WALL CONSTRUCTION (UNO):** 2x STUDS @ 16". SHEATHED WITH 1 1/2" ZIP-R SHEATHING ON EXTERIOR FACE. NAIL WALL SHEATHING W/ 10d GUN NAILS (0.131"Ø x 3") @ 3" PANEL EDGES AND BOUNDARIES AND @ 12" IN FIELD OF PANEL. BLOCK AND NAIL ALL EDGES BETWEEN STUDS.
11. **EXTERIOR WALL CONSTRUCTION (UNO):** 2x STUDS @16" SHEATHED WITH 7/16" CDX PLYWOOD OR OSB, APA 24/16 ON EXTERIOR FACE. NAIL WALL SHEATHING WTH 8d GUN NAILS (0.131"Ø x 2 3/8") @ 4" AT PANEL EDGES AND BOUNDARIES AND @ 12" IN FIELD OF PANEL. BLOCK AND NAIL ALL EDGES BETWEEN STUDS.
12. **INTERIOR BEARING WALL CONSTRUCTION (UNO):** 2x STUDS @ 16". SHEATHED WITH 1/2" MIN. GYPSUM WALLBOARD ON EACH FACE. ATTACH W/ NO. 6 x 1 1/4" DRYWALL SCREWS @ 8" AT PANEL EDGES AND BOUNDARIES AND @ 12" IN FIELD OF PANEL.
13. **WALL OPENING CONSTRUCTION (UNO):** (2) 2x8 HEADER W/ MINIMUM (1) 2x6 TRIM AND (1) 2x6 KING STUD EACH END. HEADERS ARE DROPPED UNO.
14. **WOOD BEAM / JOIST FACE MOUNT HANGER TO FLUSH STEEL BEAM (UNO):** AT FLUSH STEEL BEAMS, PACK OUT WEB WITH 2x AND PLYWOOD/OSB TIGHT TO TOP AND BOTTOM FLANGES AS REQUIRED (MIN 2'-0" LENGTH) FOR FACE MOUNT HANGERS, SECURE WITH 1/2"Ø THRU BOLTS AND CONSTRUCTION ADHESIVE @ 24", STAGGERED.
15. **TYPICAL DECK TIE (UNO)**: MIN (2) DTT2Z PER DECK INSTALLED TO SIDE OF DECK JOIST, W/ 1/2" Ø THREADED ROD PENETRATING THROUGH RIM TO EITHER BLOCKING OR FIRST JOIST OTHER SIDE. AT JOIST, PACK WEB PER MANUFACTURER AS REQUIRED & CONNECT W/ NUT AND WASHER. SEE PLAN FOR ADDITIONAL LOCATIONS AND INFORMATION.
16. **TYPICAL DECK CONSTRUCTION (UNO)** Xx EXTERIOR DECKING OVER WOOD JOISTS PER PLAN. LAY DECKING PERPENDICULAR TO FRAMING AND FASTEN DECKING TO JOIST W/ (2) #8x3" EXTERIOR DECK SCREWS PER BOARD. FLASH TOP OF MULTI-PLY JOISTS / BEAMS.
17. HDX INDICATES HOLDOWN THROUGH LEVEL SHOWN, SEE SX.XX. CONTRACTOR TO VERIFY LOCATIONS AND LAYOUT WITH FRAMING ABOVE
18. SWX INDICATES SHEAR WALL TO BE SHEATHED ON SIDE INDICATED BY ARROW (UNO) WITH SHEATHING PER SHEAR WALL SCHEDULE. SEE S6.01

LEVEL 2 KEYNOTE SCHEDULE

X	Description

HANGER SCHEDULE

1. ALL HANGERS NOTED TO BE INSTALLED WITH NUMBER AND SIZE FASTENERS SPECIFIED BY MNFR. ANY SUBSTITUTIONS SHALL BE REVIEWED AND APPROVED BY ANTHEM

2. INSTALL HANGERS NOTED OR APPROVED EQUIVALENT

X	DESCRIPTION

APPROVAL STAMPS:

No.	Date	Description

SUBMISSIONS & REVISIONS

OWNER

ARCHITECT

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GENERAL CONTRACTOR

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303-848-8497 970-300-3338
anthemstructural.com Job #22-048

M.E.P. & F.P. ENGINEERS

INTERIOR DESIGNER:

PROJECT LOCATION

BASECAMP TOWNHOME

1950 CURVE COURT
STEAMBOAT SPRINGS, CO 80487

DRAWING TITLE

LEVEL 2 FRAMING PLAN

SEAL

DATE:
06/17/22

DRAWN BY:

CHECKED BY:

PROJECT NO:
22-048

DRAWING NO:

S1.03

LEVEL 3 FRAMING PLAN
3/16" = 1'-0"

North

FRAMING LEVEL PLAN NOTES:

1. SEE S0.01 FOR GENERAL STRUCTURAL NOTES, ABBREVIATIONS KEY AND LEGEND
2. SEE S5.01 FOR TYPICAL DETAILS
3. SEE S6.01 FOR SCHEDULES
4. REFER TO ARCHITECTURAL DRAWINGS FOR ADDITIONAL INFORMATION AND DIMENSIONS.
5. VIF (E) CONDITIONS PRIOR TO NEW CONSTRUCTION AND NOTIFY ANTHEM OF ANY DISCREPANCIES
6. TOP OF SUB-FLOOR SHEATHING =XXX'-X" UNO
7. TOP OF PLATE HEIGHT = XXX'-X" UNO
8. WALL FRAMING AND COLUMNS SHOWN SUPPORT THE FRAMING ON THIS LEVEL
9. **FLOOR CONSTRUCTION (UNO):** X" LIGHTWEIGHT CONCRETE TOPPING OVER 3/4" STURD-I-FLOOR APA RATED @ 24" OC TONGUE AND GROOVE SHEATHING, OVER WOOD I-JOISTS / TRUSSES W/ 1 1/4" LSL RIM, SEE PLAN. GLUE AND FASTEN SHEATHING TO JOISTS, RIM, FLUSH BEAMS, AND LEDGERS WITH 8d GUN NAILS (0.113"Ø x 2 3/8") @ 4" O.C. AND @ 8" O.C. ALONG INTERMEDIATE FRAMING MEMBERS. LAY PANELS PERPENDICULAR TO FRAMING MEMBERS AND STAGGER PANEL JOINTS.
10. **EXTERIOR WALL CONSTRUCTION (UNO):** 2x STUDS @ 16". SHEATHED WITH 1 1/2" ZIP-R SHEATHING ON EXTERIOR FACE. NAIL WALL SHEATHING W/ 10d GUN NAILS (0.131"Ø x 3") @ 3" PANEL EDGES AND BOUNDARIES AND @ 12" IN FIELD OF PANEL. BLOCK AND NAIL ALL EDGES BETWEEN STUDS.
11. **EXTERIOR WALL CONSTRUCTION (UNO):** 2x STUDS @16" SHEATHED WITH 7/16" CDX PLYWOOD OR OSB, APA 24/16 ON EXTERIOR FACE. NAIL WALL SHEATHING WTH 8d GUN NAILS (0.131"Ø x 2 3/8") @ 4" AT PANEL EDGES AND BOUNDARIES AND @ 12" IN FIELD OF PANEL. BLOCK AND NAIL ALL EDGES BETWEEN STUDS.
12. **INTERIOR BEARING WALL CONSTRUCTION (UNO):** 2x STUDS @ 16". SHEATHED WITH 1/2" MIN. GYPSUM WALLBOARD ON EACH FACE. ATTACH W/ NO. 6 x 1 1/4" DRYWALL SCREWS @ 8" AT PANEL EDGES AND BOUNDARIES AND @ 12" IN FIELD OF PANEL.
13. **WALL OPENING CONSTRUCTION (UNO):** (2) 2x8 HEADER W/ MINIMUM (1) 2x6 TRIM AND (1) 2x6 KING STUD EACH END. HEADERS ARE DROPPED UNO.
14. **WOOD BEAM / JOIST FACE MOUNT HANGER TO FLUSH STEEL BEAM (UNO):** AT FLUSH STEEL BEAMS, PACK OUT WEB WITH 2x AND PLYWOOD/OSB TIGHT TO TOP AND BOTTOM FLANGES AS REQUIRED (MIN 2'-0" LENGTH) FOR FACE MOUNT HANGERS, SECURE WITH 1/2"Ø THRU BOLTS AND CONSTRUCTION ADHESIVE @ 24", STAGGERED.
15. **TYPICAL DECK TIE (UNO)**: MIN (2) DTT2Z PER DECK INSTALLED TO SIDE OF DECK JOIST, W/ 1/2" Ø THREADED ROD PENETRATING THROUGH RIM TO EITHER BLOCKING OR FIRST JOIST OTHER SIDE. AT JOIST, PACK WEB PER MANUFACTURER AS REQUIRED & CONNECT W/ NUT AND WASHER. SEE PLAN FOR ADDITIONAL LOCATIONS AND INFORMATION.
16. **TYPICAL DECK CONSTRUCTION (UNO)** Xx EXTERIOR DECKING OVER WOOD JOISTS PER PLAN. LAY DECKING PERPENDICULAR TO FRAMING AND FASTEN DECKING TO JOIST W/ (2) #8x3" EXTERIOR DECK SCREWS PER BOARD. FLASH TOP OF MULTI-PLY JOISTS / BEAMS.
17. HDX INDICATES HOLDOWN THROUGH LEVEL SHOWN, SEE SX.XX. CONTRACTOR TO VERIFY LOCATIONS AND LAYOUT WITH FRAMING ABOVE
18. SWX INDICATES SHEAR WALL TO BE SHEATHED ON SIDE INDICATED BY ARROW (UNO) WITH SHEATHING PER SHEAR WALL SCHEDULE. SEE S6.01

LEVEL 3 KEYNOTE SCHEDULE

X	DESCRIPTION

HANGER SCHEDULE

1. ALL HANGERS NOTED TO BE INSTALLED WITH NUMBER AND SIZE FASTENERS SPECIFIED BY MNFR. ANY SUBSTITUTIONS SHALL BE REVIEWED AND APPROVED BY ANTHEM
2. INSTALL HANGERS NOTED OR APPROVED EQUIVALENT

X	DESCRIPTION

APPROVAL STAMPS:

No.	Date	Description

SUBMISSIONS & REVISIONS

OWNER

ARCHITECT

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GENERAL CONTRACTOR

CIVIL ENGINEER

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303-848-8497 970-300-3338
anthemstructural.com Job #22-048

M.E.P. & F.P. ENGINEERS

INTERIOR DESIGNER:

PROJECT LOCATION

BASECAMP TOWNHOME

1950 CURVE COURT
STEAMBOAT SPRINGS, CO 80487

DRAWING TITLE

LEVEL 3 FRAMING PLAN

SEAL

DATE: 06/17/22

DRAWN BY:

CHECKED BY:

PROJECT NO: 22-048

DRAWING NO: **S1.04**

6/23/2022 2:05:29 PM

LEVEL 4 FRAMING PLAN

3/16" = 1'-0"

North

FRAMING LEVEL PLAN NOTES:

1. SEE S0.01 FOR GENERAL STRUCTURAL NOTES, ABBREVIATIONS KEY AND LEGEND
2. SEE S5.01 FOR TYPICAL DETAILS
3. SEE S6.01 FOR SCHEDULES
4. REFER TO ARCHITECTURAL DRAWINGS FOR ADDITIONAL INFORMATION AND DIMENSIONS.
5. VIF (E) CONDITIONS PRIOR TO NEW CONSTRUCTION AND NOTIFY ANTHEM OF ANY DISCREPANCIES
6. TOP OF SUB-FLOOR SHEATHING =XXX'-X" UNO
7. TOP OF PLATE HEIGHT = XXX'-X" UNO
8. WALL FRAMING AND COLUMNS SHOWN SUPPORT THE FRAMING ON THIS LEVEL
9. **FLOOR CONSTRUCTION (UNO):** X" LIGHTWEIGHT CONCRETE TOPPING OVER 3/4" STURD-I-FLOOR APA RATED @ 24" OC TONGUE AND GROOVE SHEATHING, OVER WOOD I-JOISTS / TRUSSES W/ 1 1/4" LSL RIM, SEE PLAN. GLUE AND FASTEN SHEATHING TO JOISTS, RIM, FLUSH BEAMS, AND LEDGERS WITH 8d GUN NAILS (0.113"Ø x 2 3/8") @ 4" O.C. AND @ 8" O.C. ALONG INTERMEDIATE FRAMING MEMBERS. LAY PANELS PERPENDICULAR TO FRAMING MEMBERS AND STAGGER PANEL JOINTS.
10. **EXTERIOR WALL CONSTRUCTION (UNO):** 2x STUDS @ 16". SHEATHED WITH 1 1/2" ZIP-R SHEATHING ON EXTERIOR FACE. NAIL WALL SHEATHING W/ 10d GUN NAILS (0.131"Ø x 3") @ 3" PANEL EDGES AND BOUNDARIES AND @ 12" IN FIELD OF PANEL. BLOCK AND NAIL ALL EDGES BETWEEN STUDS.
11. **EXTERIOR WALL CONSTRUCTION (UNO):** 2x STUDS @16" SHEATHED WITH 7/16" CDX PLYWOOD OR OSB, APA 24/16 ON EXTERIOR FACE. NAIL WALL SHEATHING WTH 8d GUN NAILS (0.131"Ø x 2 3/8") @ 4" AT PANEL EDGES AND BOUNDARIES AND @ 12" IN FIELD OF PANEL. BLOCK AND NAIL ALL EDGES BETWEEN STUDS.
12. **INTERIOR BEARING WALL CONSTRUCTION (UNO):** 2x STUDS @ 16". SHEATHED WITH 1/2" MIN. GYPSUM WALLBOARD ON EACH FACE. ATTACH W/ NO. 6 x 1 1/4" DRYWALL SCREWS @ 8" AT PANEL EDGES AND BOUNDARIES AND @ 12" IN FIELD OF PANEL.
13. **WALL OPENING CONSTRUCTION (UNO):** (2) 2x8 HEADER W/ MINIMUM (1) 2x6 TRIM AND (1) 2x6 KING STUD EACH END. HEADERS ARE DROPPED UNO.
14. **WOOD BEAM / JOIST FACE MOUNT HANGER TO FLUSH STEEL BEAM (UNO):** AT FLUSH STEEL BEAMS, PACK OUT WEB WITH 2x AND PLYWOOD/OSB TIGHT TO TOP AND BOTTOM FLANGES AS REQUIRED (MIN 2'-0" LENGTH) FOR FACE MOUNT HANGERS, SECURE WITH 1/2"Ø THRU BOLTS AND CONSTRUCTION ADHESIVE @ 24", STAGGERED.
15. **TYPICAL DECK TIE (UNO)**: MIN (2) DTT2Z PER DECK INSTALLED TO SIDE OF DECK JOIST, W/ 1/2" Ø THREADED ROD PENETRATING THROUGH RIM TO EITHER BLOCKING OR FIRST JOIST OTHER SIDE. AT JOIST, PACK WEB PER MANUFACTURER AS REQUIRED & CONNECT W/ NUT AND WASHER. SEE PLAN FOR ADDITIONAL LOCATIONS AND INFORMATION.
16. **TYPICAL DECK CONSTRUCTION (UNO)** Xx EXTERIOR DECKING OVER WOOD JOISTS PER PLAN. LAY DECKING PERPENDICULAR TO FRAMING AND FASTEN DECKING TO JOIST W/ (2) #8x3" EXTERIOR DECK SCREWS PER BOARD. FLASH TOP OF MULTI-PLY JOISTS / BEAMS.
17. HDX INDICATES HOLDOWN THROUGH LEVEL SHOWN, SEE SX.XX. CONTRACTOR TO VERIFY LOCATIONS AND LAYOUT WITH FRAMING ABOVE
18. SWX INDICATES SHEAR WALL TO BE SHEATHED ON SIDE INDICATED BY ARROW (UNO) WITH SHEATHING PER SHEAR WALL SCHEDULE. SEE S6.01

LEVEL 4 KEYNOTE SCHEDULE

X	DESCRIPTION

HANGER SCHEDULE

1. ALL HANGERS NOTED TO BE INSTALLED WITH NUMBER AND SIZE FASTENERS SPECIFIED BY MNFR. ANY SUBSTITUTIONS SHALL BE REVIEWED AND APPROVED BY ANTHEM
2. INSTALL HANGERS NOTED OR APPROVED EQUIVALENT

X	DESCRIPTION

APPROVAL STAMPS:

No.	Date	Description

SUBMISSIONS & REVISIONS

OWNER

ARCHITECT

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GENERAL CONTRACTOR

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anthemstructural.com Job #22-048

M.E.P. & F.P. ENGINEERS

INTERIOR DESIGNER:

PROJECT LOCATION

BASECAMP TOWNHOME

1950 CURVE COURT
STEAMBOAT SPRINGS, CO 80487

DRAWING TITLE

LEVEL 4 FRAMING PLAN

SEAL

DATE: 06/17/22

DRAWN BY:

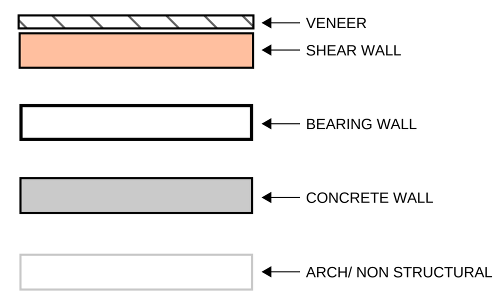
CHECKED BY:

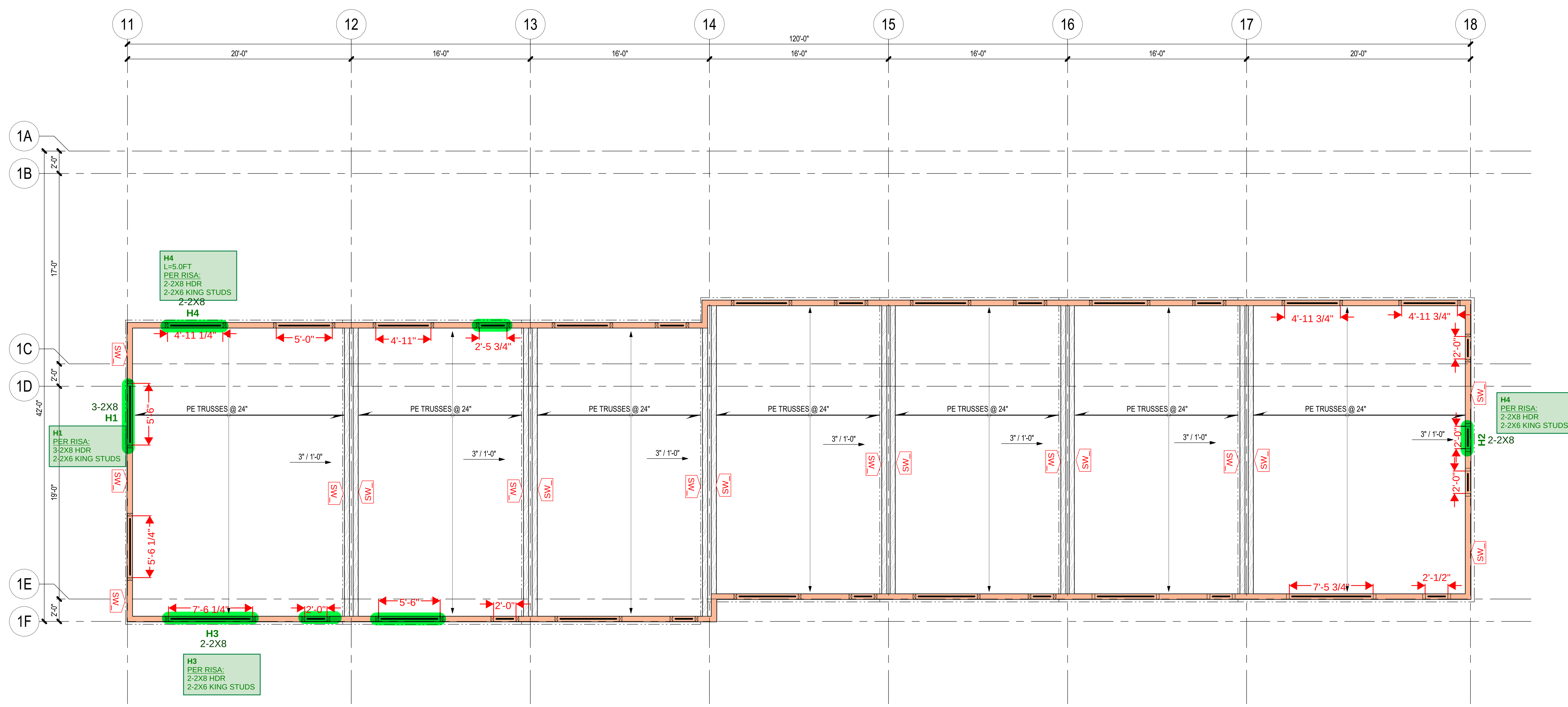
PROJECT NO: 22-048

DRAWING NO:

S1.05

6/23/2022 2:05:29 PM





ROOF FRAMING PLAN
3/16" = 1'-0"
North

6/23/2022 2:05:29 PM

ROOF LEVEL PLAN NOTES:

1. SEE S0.01 FOR GENERAL STRUCTURAL NOTES, ABBREVIATIONS KEY AND LEGEND
2. SEE S5.01 FOR TYPICAL DETAILS
3. SEE S6.01 FOR SCHEDULES
4. TOP OF PLATE HEIGHT = XXX'-X" UNO
5. INSTALL LOOSE LINTEL PER SX.XX AT ALL OPENINGS W/ VENEER ABOVE
6. WALL FRAMING AND COLUMNS SHOWN SUPPORT THE FRAMING ON THIS LEVEL
7. REFER TO ARCHITECTURAL DRAWINGS FOR ADDITIONAL INFORMATION AND DIMENSIONS.
8. **ROOF CONSTRUCTION (UNO)** 15/32" NOMINAL APA 32/16 OR 5/8" NOMINAL APA 40/20 RATED SHEATHING OVER PE TRUSSES / RAFTERS SEE PLAN. FASTEN SHEATHING TO TRUSSES / RAFTERS, RIMS, AND LEDGERS WITH 8d GUN NAILS (0.113" x 2 3/8") @ 4" ALONG PANEL EDGES AND @ 8" ALONG INTERMEDIATE FRAMING MEMBERS. LAY PANELS PERPENDICULAR TO FRAMING MEMBERS AND STAGGER PANEL JOINTS.
9. **TYPICAL TRUSS / RAFTER TIE DOWN:** H2.5T AT BEARING, (2) CLIPS WITHIN 8'-0" OF CORNERS (UNO)
10. **TYPICAL GIRDER TRUSS / ROOF BEAM TIE DOWN:** (2) HTS20 (OR ST22 STRAP) EACH END AT BEARING (UNO)
11. **EXTERIOR WALL CONSTRUCTION (UNO):** 2x STUDS @ 16". SHEATH WITH 1 1/2" ZIP-R SHEATHING ON EXTERIOR FACE. NAIL WALL SHEATHING W/ 10d GUN NAILS (0.131"Ø x 3") @ 3" PANEL EDGES AND BOUNDARIES AND @ 12" IN FIELD OF PANEL. BLOCK AND NAIL ALL EDGES BETWEEN STUDS.
12. **EXTERIOR WALL CONSTRUCTION (UNO):** 2x STUDS @ 16" SHEATHED WITH 7/16" CDX PLYWOOD OR OSB, APA 24/16 ON EXTERIOR FACE. NAIL WALL SHEATHING WTH 8d GUN NAILS (0.131"Ø x 2 3/8") @ 4" AT PANEL EDGES AND BOUNDARIES AND @ 12" IN FIELD OF PANEL. BLOCK AND NAIL ALL EDGES BETWEEN STUDS.
13. **WALL OPENING CONSTRUCTION (UNO):** (2) 2x8 HEADER W/ MINIMUM (1) 2x6 TRIM AND (1) 2x6 KING STUD EACH END.
14. SWX **INDICATES SHEAR WALL:** TO BE SHEATHED ON SIDE INDICATED BY ARROW (UNO) WITH WOOD SHEATHING PER SHEAR WALL SCHEDULE. SEE SHEET SS6.01
15. **INDICATES OVERFRAMING:** CONTINUE PRIMARY ROOF SHEATHING BELOW OVERFRAMED AREAS. OVERFRAME WITH RAFTERS/TRUSSES SPACED 24" MAX, PLACE RAFTERS/TRUSSES PERPINDICULAR TO FRAMING BELOW.

ROOF KEYNOTE SCHEDULE

X	DESCRIPTION

HANGER SCHEDULE

1. ALL HANGERS NOTED TO BE INSTALLED WITH NUMBER AND SIZE FASTENERS SPECIFIED BY MNFR. ANY SUBSTITUTIONS SHALL BE REVIEWED AND APPROVED BY ANTHEM
2. INSTALL HANGERS NOTED OR APPROVED EQUIVALENT

X	DESCRIPTION

APPROVAL STAMPS:

No.	Date	Description

SUBMISSIONS & REVISIONS

OWNER

ARCHITECT

KASA

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GENERAL CONTRACTOR

CIVIL ENGINEER

LANDSCAPE ARCHITECT

anthem structural engineers
303-848-8497 970-300-3338
anthemstructural.com Job #22-048

M.E.P. & F.P. ENGINEERS

INTERIOR DESIGNER:

PROJECT LOCATION

BASECAMP TOWNHOME
1950 CURVE COURT
STEAMBOAT SPRINGS, CO 80487

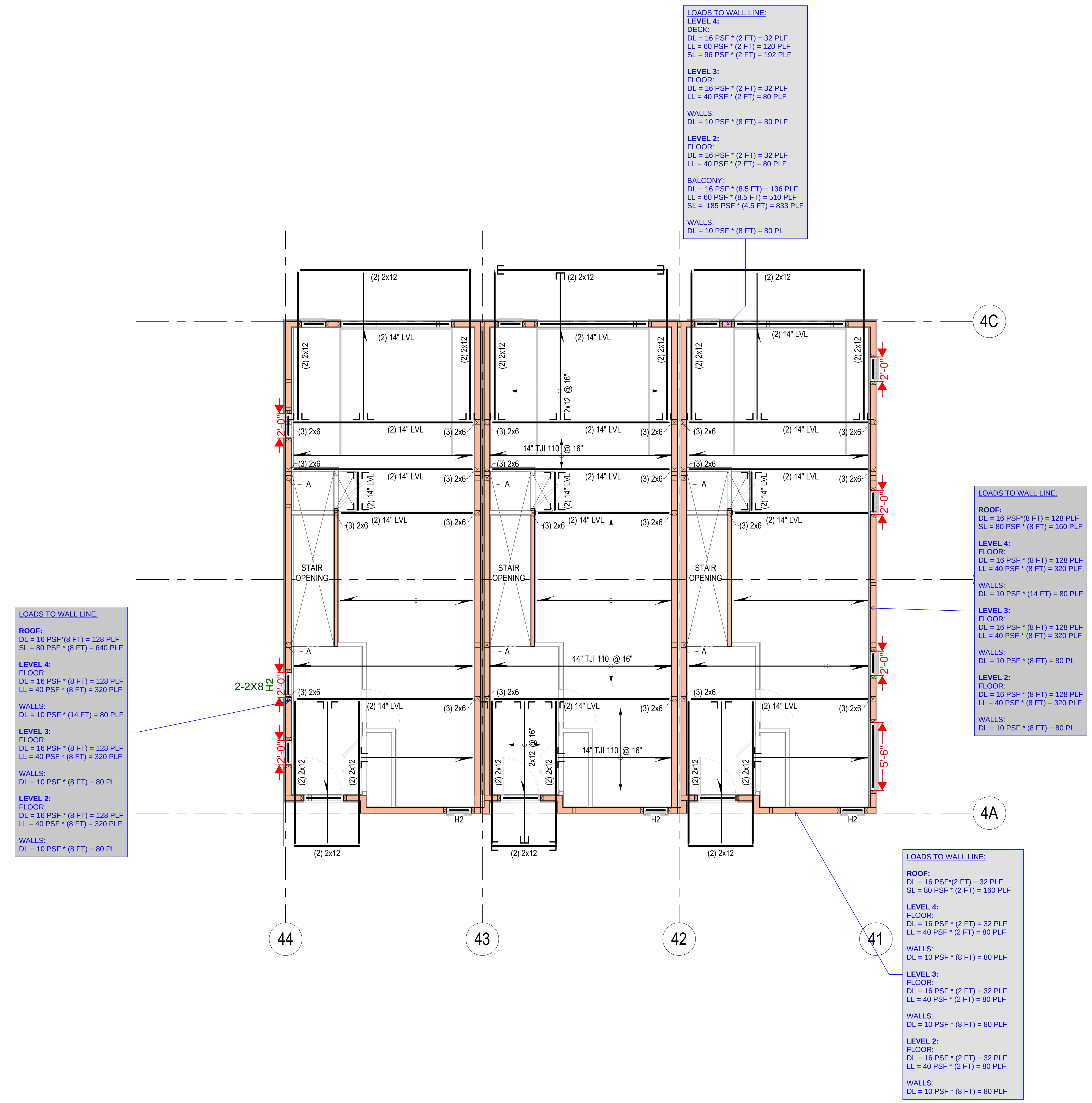
DRAWING TITLE

ROOF FRAMING PLAN

SEAL

DATE: 01/04/21
DRAWN BY:
CHECKED BY:
PROJECT NO: 22-048

DRAWING NO:
S1.06



TH4 LEVEL 2 FRAMING PLAN
3/16" = 1'-0"
North

FRAMING LEVEL PLAN NOTES:

1. SEE S0.01 FOR GENERAL STRUCTURAL NOTES, ABBREVIATIONS KEY AND LEGEND
2. SEE S5.01 FOR TYPICAL DETAILS
3. SEE S6.01 FOR SCHEDULES
4. REFER TO ARCHITECTURAL DRAWINGS FOR ADDITIONAL INFORMATION AND DIMENSIONS.
5. VIF (E) CONDITIONS PRIOR TO NEW CONSTRUCTION AND NOTIFY ANTHEM OF ANY DISCREPANCIES
6. TOP OF SUB-FLOOR SHEATHING =XXX'-X" UNO
7. TOP OF PLATE HEIGHT = XXX'-X" UNO
8. WALL FRAMING AND COLUMNS SHOWN SUPPORT THE FRAMING ON THIS LEVEL
9. **FLOOR CONSTRUCTION (UNO):** X" LIGHTWEIGHT CONCRETE TOPPING OVER 3/4" STURD-I-FLOOR APA RATED @ 24" OC TONGUE AND GROOVE SHEATHING, OVER WOOD I-JOISTS / TRUSSES W/ 1 1/4" LSL RIM, SEE PLAN. GLUE AND FASTEN SHEATHING TO JOISTS, RIM, FLUSH BEAMS, AND LEDGERS WITH 8d GUN NAILS (0.113"Ø x 2 3/8") @ 4" O.C. AND @ 8" O.C. ALONG INTERMEDIATE FRAMING MEMBERS. LAY PANELS PERPENDICULAR TO FRAMING MEMBERS AND STAGGER PANEL JOINTS.
10. **EXTERIOR WALL CONSTRUCTION (UNO):** 2x STUDS @ 16". SHEATHED WITH 1 1/2" ZIP-R SHEATHING ON EXTERIOR FACE. NAIL WALL SHEATHING W/ 10d GUN NAILS (0.131"Ø x 3") @ 3" PANEL EDGES AND BOUNDARIES AND @ 12" IN FIELD OF PANEL. BLOCK AND NAIL ALL EDGES BETWEEN STUDS.
11. **EXTERIOR WALL CONSTRUCTION (UNO):** 2x STUDS @16" SHEATHED WITH 7/16" CDX PLYWOOD OR OSB, APA 24/16 ON EXTERIOR FACE. NAIL WALL SHEATHING WTH 8d GUN NAILS (0.131"Ø x 2 3/8") @ 4" AT PANEL EDGES AND BOUNDARIES AND @ 12" IN FIELD OF PANEL. BLOCK AND NAIL ALL EDGES BETWEEN STUDS.
12. **INTERIOR BEARING WALL CONSTRUCTION (UNO):** 2x STUDS @ 16". SHEATHED WITH 1/2" MIN. GYPSUM WALLBOARD ON EACH FACE. ATTACH W/ NO. 6 x 1 1/4" DRYWALL SCREWS @ 8" AT PANEL EDGES AND BOUNDARIES AND @ 12" IN FIELD OF PANEL.
13. **WALL OPENING CONSTRUCTION (UNO):** (2) 2x8 HEADER W/ MINIMUM (1) 2x6 TRIM AND (1) 2x6 KING STUD EACH END. HEADERS ARE DROPPED UNO.
14. **WOOD BEAM / JOIST FACE MOUNT HANGER TO FLUSH STEEL BEAM (UNO):** AT FLUSH STEEL BEAMS, PACK OUT WEB WITH 2x AND PLYWOOD/OSB TIGHT TO TOP AND BOTTOM FLANGES AS REQUIRED (MIN 2'-0" LENGTH) FOR FACE MOUNT HANGERS, SECURE WITH 1/2"Ø THRU BOLTS AND CONSTRUCTION ADHESIVE @ 24", STAGGERED.
15. **TYPICAL DECK TIE (UNO)**: MIN (2) DTT2Z PER DECK INSTALLED TO SIDE OF DECK JOIST, W/ 1/2" Ø THREADED ROD PENETRATING THROUGH RIM TO EITHER BLOCKING OR FIRST JOIST OTHER SIDE. AT JOIST, PACK WEB PER MANUFACTURER AS REQUIRED & CONNECT W/ NUT AND WASHER. SEE PLAN FOR ADDITIONAL LOCATIONS AND INFORMATION.
16. **TYPICAL DECK CONSTRUCTION (UNO)** Xx EXTERIOR DECKING OVER WOOD JOISTS PER PLAN. LAY DECKING PERPENDICULAR TO FRAMING AND FASTEN DECKING TO JOIST W/ (2) #8x3" EXTERIOR DECK SCREWS PER BOARD. FLASH TOP OF MULTI-PLY JOISTS / BEAMS.
17. HDX INDICATES HOLDOWN THROUGH LEVEL SHOWN, SEE SX.XX. CONTRACTOR TO VERIFY LOCATIONS AND LAYOUT WITH FRAMING ABOVE
18. SWX INDICATES SHEAR WALL TO BE SHEATHED ON SIDE INDICATED BY ARROW (UNO) WITH SHEATHING PER SHEAR WALL SCHEDULE. SEE S6.01

LEVEL 1 KEYNOTE SCHEDULE

	DESCRIPTION

HANGER SCHEDULE

1. ALL HANGERS NOTED TO BE INSTALLED WITH NUMBER AND SIZE FASTENERS SPECIFIED BY MNFR. ANY SUBSTITUTIONS SHALL BE REVIEWED AND APPROVED BY ANTHEM

2. INSTALL HANGERS NOTED OR APPROVED EQUIVALENT

	DESCRIPTION	Face Fasteners	Joist Fasteners
1	LUS28	(6) 10d x 3" NAILS	(4) 10d x 3" NAILS
2	LUS28-2	(6) 10d x 3" NAILS	(4) 10d x 3" NAILS
3	HHUS410	(30) 10d x 3" NAILS	(10) 10d x 3" NAILS
4	ISU2.37/14	(12) 10d x 3" NAILS	-
5	IUS1.81/14	(12) 10d x 3" NAILS	-
6	U210-2	(14) 16d x 3 1/2" NAILS	(6) 10d x 3" NAILS
7	HUC212-2	(14) 10d x 3" NAILS	(6) 10d x 3" NAILS
8	LUC210Z	(10) 10d x 3" NAILS	(4) 10d x 3" NAILS

APPROVAL STAMPS:

No.	Date	Description

SUBMISSIONS & REVISIONS

OWNER

ARCHITECT

K A S A

KEVIN & ASAKO SPERRY ARCHITECTURE
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GENERAL CONTRACTOR

CIVIL ENGINEER

LANDSCAPE ARCHITECT

anthem structural engineers
303-848-8497 970-300-3338
anthemstructural.com Job #22-048

M.E.P. & F.P. ENGINEERS

INTERIOR DESIGNER:

PROJECT LOCATION

BASECAMP TOWNHOME

1950 CURVE COURT
STEAMBOAT SPRINGS, CO 80487

DRAWING TITLE

TH4 LEVEL 2 FRAMING PLAN

SEAL

DATE: 07/11/22

DRAWN BY:

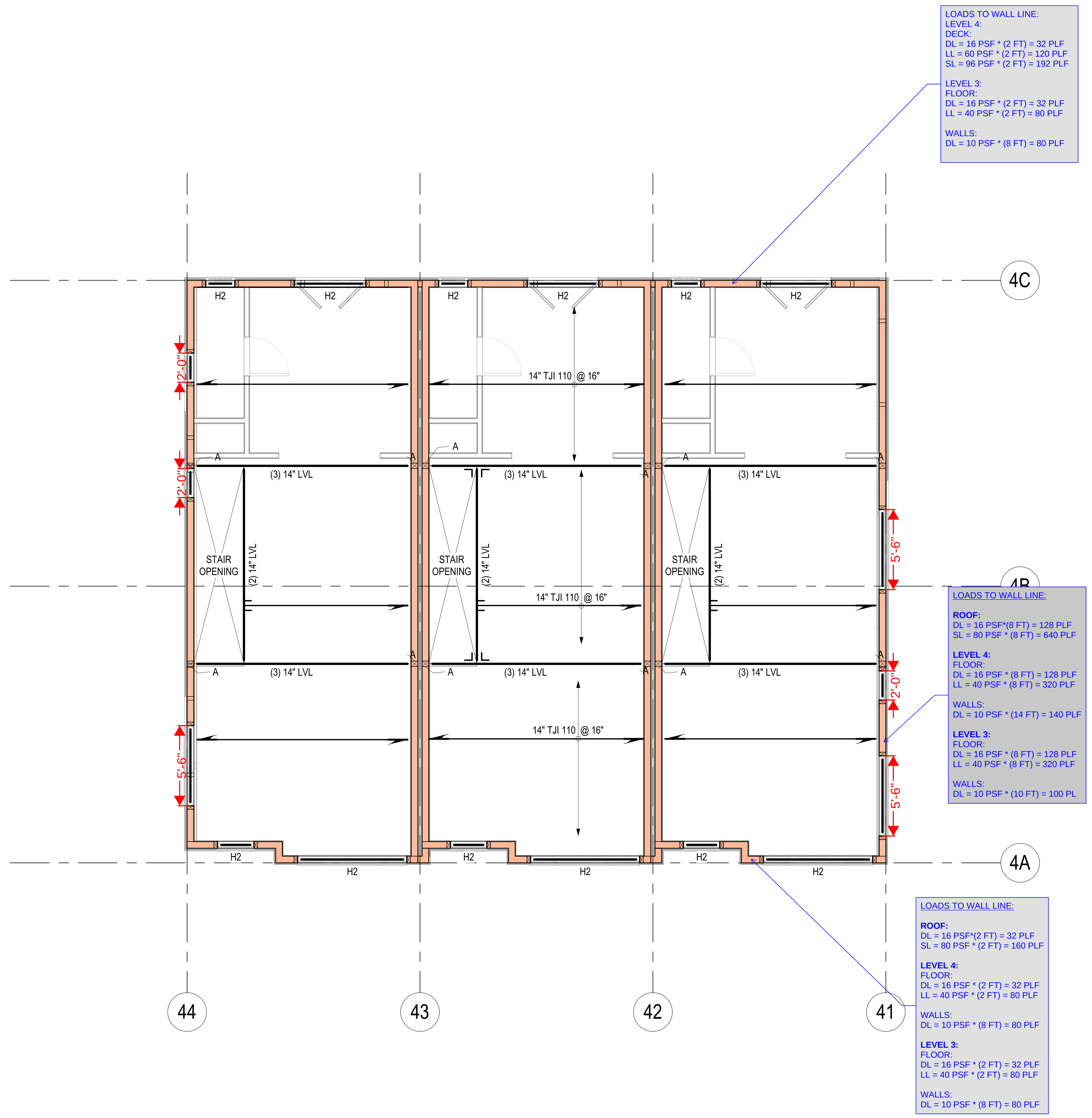
CHECKED BY:

PROJECT NO: 22-048

DRAWING NO:

S1.21

7/15/2022 8:53:09 AM



North

TH4 LEVEL 3 FRAMING PLAN

3/16" = 1'-0"

7/15/2022 8:53:11 AM

FRAMING LEVEL PLAN NOTES:

1. SEE S0.01 FOR GENERAL STRUCTURAL NOTES, ABBREVIATIONS KEY AND LEGEND
2. SEE S5.01 FOR TYPICAL DETAILS
3. SEE S6.01 FOR SCHEDULES
4. REFER TO ARCHITECTURAL DRAWINGS FOR ADDITIONAL INFORMATION AND DIMENSIONS.
5. VIF (E) CONDITIONS PRIOR TO NEW CONSTRUCTION AND NOTIFY ANTHEM OF ANY DISCREPANCIES
6. TOP OF SUB-FLOOR SHEATHING =XXX'-X" UNO
7. TOP OF PLATE HEIGHT = XXX'-X" UNO
8. WALL FRAMING AND COLUMNS SHOWN SUPPORT THE FRAMING ON THIS LEVEL
9. **FLOOR CONSTRUCTION (UNO):** X" LIGHTWEIGHT CONCRETE TOPPING OVER 3/4" STURD-I-FLOOR APA RATED @ 24" OC TONGUE AND GROOVE SHEATHING, OVER WOOD I-JOISTS / TRUSSES W/ 1 1/4" LSL RIM, SEE PLAN. GLUE AND FASTEN SHEATHING TO JOISTS, RIM, FLUSH BEAMS, AND LEDGERS WITH 8d GUN NAILS (0.113"Ø x 2 3/8") @ 4" O.C. AND @ 8" O.C. ALONG INTERMEDIATE FRAMING MEMBERS. LAY PANELS PERPENDICULAR TO FRAMING MEMBERS AND STAGGER PANEL JOINTS.
10. **EXTERIOR WALL CONSTRUCTION (UNO):** 2x STUDS @ 16". SHEATHED WITH 1 1/2" ZIP-R SHEATHING ON EXTERIOR FACE. NAIL WALL SHEATHING W/ 10d GUN NAILS (0.131"Ø x 3") @ 3" PANEL EDGES AND BOUNDARIES AND @ 12" IN FIELD OF PANEL. BLOCK AND NAIL ALL EDGES BETWEEN STUDS.
11. **EXTERIOR WALL CONSTRUCTION (UNO):** 2x STUDS @16" SHEATHED WITH 7/16" CDX PLYWOOD OR OSB, APA 24/16 ON EXTERIOR FACE. NAIL WALL SHEATHING WTH 8d GUN NAILS (0.131"Ø x 2 3/8") @ 4" AT PANEL EDGES AND BOUNDARIES AND @ 12" IN FIELD OF PANEL. BLOCK AND NAIL ALL EDGES BETWEEN STUDS.
12. **INTERIOR BEARING WALL CONSTRUCTION (UNO):** 2x STUDS @ 16". SHEATHED WITH 1/2" MIN. GYPSUM WALLBOARD ON EACH FACE. ATTACH W/ NO. 6 x 1 1/4" DRYWALL SCREWS @ 8" AT PANEL EDGES AND BOUNDARIES AND @ 12" IN FIELD OF PANEL.
13. **WALL OPENING CONSTRUCTION (UNO):** (2) 2x8 HEADER W/ MINIMUM (1) 2x6 TRIM AND (1) 2x6 KING STUD EACH END. HEADERS ARE DROPPED UNO.
14. **WOOD BEAM / JOIST FACE MOUNT HANGER TO FLUSH STEEL BEAM (UNO):** AT FLUSH STEEL BEAMS, PACK OUT WEB WITH 2x AND PLYWOOD/OSB TIGHT TO TOP AND BOTTOM FLANGES AS REQUIRED (MIN 2'-0" LENGTH) FOR FACE MOUNT HANGERS, SECURE WITH 1/2"Ø THRU BOLTS AND CONSTRUCTION ADHESIVE @ 24", STAGGERED.
15. **TYPICAL DECK TIE (UNO)**: MIN (2) DTT2Z PER DECK INSTALLED TO SIDE OF DECK JOIST, W/ 1/2" Ø THREADED ROD PENETRATING THROUGH RIM TO EITHER BLOCKING OR FIRST JOIST OTHER SIDE. AT JOIST, PACK WEB PER MANUFACTURER AS REQUIRED & CONNECT W/ NUT AND WASHER. SEE PLAN FOR ADDITIONAL LOCATIONS AND INFORMATION.
16. **TYPICAL DECK CONSTRUCTION (UNO)** Xx EXTERIOR DECKING OVER WOOD JOISTS PER PLAN. LAY DECKING PERPENDICULAR TO FRAMING AND FASTEN DECKING TO JOIST W/ (2) #8x3" EXTERIOR DECK SCREWS PER BOARD. FLASH TOP OF MULTI-PLY JOISTS / BEAMS.
17. HDX — INDICATES HOLDOWN THROUGH LEVEL SHOWN, SEE SX.XX. CONTRACTOR TO VERIFY LOCATIONS AND LAYOUT WITH FRAMING ABOVE
18. SWX — INDICATES SHEAR WALL TO BE SHEATHED ON SIDE INDICATED BY ARROW (UNO) WITH SHEATHING PER SHEAR WALL SCHEDULE. SEE S6.01

LEVEL 1 KEYNOTE SCHEDULE

	DESCRIPTION

HANGER SCHEDULE

1. ALL HANGERS NOTED TO BE INSTALLED WITH NUMBER AND SIZE FASTENERS SPECIFIED BY MNFR. ANY SUBSTITUTIONS SHALL BE REVIEWED AND APPROVED BY ANTHEM

2. INSTALL HANGERS NOTED OR APPROVED EQUIVALENT

	DESCRIPTION	Face Fasteners	Joist Fasteners
1	LUS28	(6) 10d x 3" NAILS	(4) 10d x 3" NAILS
2	LUS28-2	(6) 10d x 3" NAILS	(4) 10d x 3" NAILS
3	HHUS410	(30) 10d x 3" NAILS	(10) 10d x 3" NAILS
4	ISU2.37/14	(12) 10d x 3" NAILS	-
5	IUS1.81/14	(12) 10d x 3" NAILS	-
6	U210-2	(14) 16d x 3 1/2" NAILS	(6) 10d x 3" NAILS
7	HUC212-2	(14) 10d x 3" NAILS	(6) 10d x 3" NAILS
8	LUC210Z	(10) 10d x 3" NAILS	(4) 10d x 3" NAILS

APPROVAL STAMPS:

No.	Date	Description

SUBMISSIONS & REVISIONS

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GENERAL CONTRACTOR

CIVIL ENGINEER

LANDSCAPE ARCHITECT

anthem structural engineers
303-848-8497 970-300-3338
anthemstructural.com Job #22-048

M.E.P. & F.P. ENGINEERS

INTERIOR DESIGNER:

PROJECT LOCATION

BASECAMP TOWNHOME

1950 CURVE COURT
STEAMBOAT SPRINGS, CO 80487

DRAWING TITLE

TH4 LEVEL 3 FRAMING PLAN

SEAL

DATE: 07/11/22

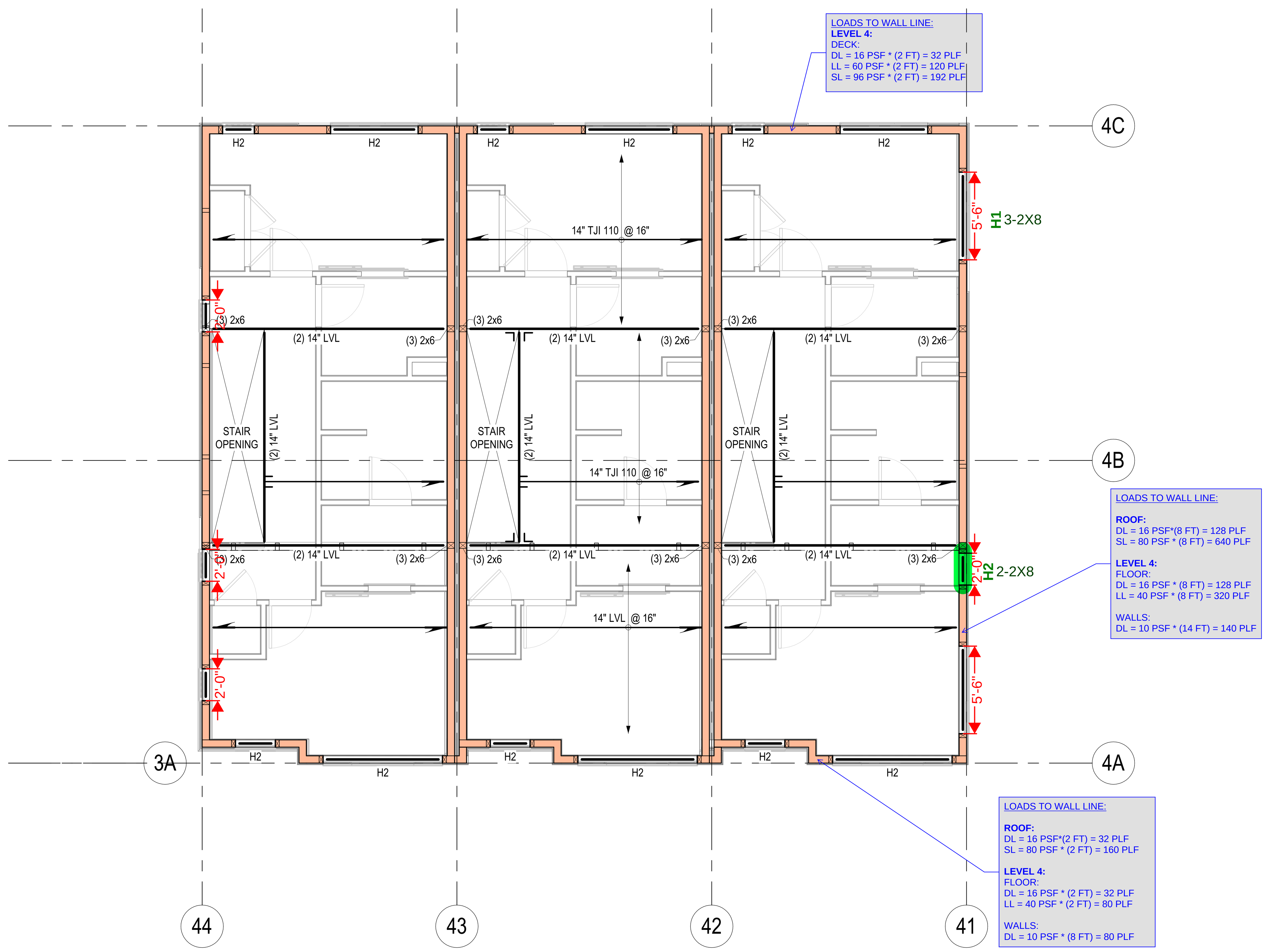
DRAWN BY:

CHECKED BY:

PROJECT NO: 22-048

DRAWING NO:

S1.22



North

TH4 LEVEL 4 FRAMING PLAN

3/16" = 1'-0"

FRAMING LEVEL PLAN NOTES:

1. SEE S0.01 FOR GENERAL STRUCTURAL NOTES, ABBREVIATIONS KEY AND LEGEND
2. SEE S5.01 FOR TYPICAL DETAILS
3. SEE S6.01 FOR SCHEDULES
4. REFER TO ARCHITECTURAL DRAWINGS FOR ADDITIONAL INFORMATION AND DIMENSIONS.
5. VIF (E) CONDITIONS PRIOR TO NEW CONSTRUCTION AND NOTIFY ANTHEM OF ANY DISCREPANCIES
6. TOP OF SUB-FLOOR SHEATHING =XXX'-X" UNO
7. TOP OF PLATE HEIGHT = XXX'-X" UNO
8. WALL FRAMING AND COLUMNS SHOWN SUPPORT THE FRAMING ON THIS LEVEL
9. **FLOOR CONSTRUCTION (UNO):** X" LIGHTWEIGHT CONCRETE TOPPING OVER 3/4" STURD-I-FLOOR APA RATED @ 24" OC TONGUE AND GROOVE SHEATHING, OVER WOOD I-JOISTS / TRUSSES W/ 1 1/4" LSL RIM, SEE PLAN. GLUE AND FASTEN SHEATHING TO JOISTS, RIM, FLUSH BEAMS, AND LEDGERS WITH 8d GUN NAILS (0.113"Ø x 2 3/8") @ 4" O.C. AND @ 8" O.C. ALONG INTERMEDIATE FRAMING MEMBERS. LAY PANELS PERPENDICULAR TO FRAMING MEMBERS AND STAGGER PANEL JOINTS.
10. **EXTERIOR WALL CONSTRUCTION (UNO):** 2x STUDS @ 16". SHEATHED WITH 1 1/2" ZIP-R SHEATHING ON EXTERIOR FACE. NAIL WALL SHEATHING W/ 10d GUN NAILS (0.131"Ø x 3") @ 3" PANEL EDGES AND BOUNDARIES AND @ 12" IN FIELD OF PANEL. BLOCK AND NAIL ALL EDGES BETWEEN STUDS.
11. **EXTERIOR WALL CONSTRUCTION (UNO):** 2x STUDS @16" SHEATHED WITH 7/16" CDX PLYWOOD OR OSB, APA 24/16 ON EXTERIOR FACE. NAIL WALL SHEATHING WTH 8d GUN NAILS (0.131"Ø x 2 3/8") @ 4" AT PANEL EDGES AND BOUNDARIES AND @ 12" IN FIELD OF PANEL. BLOCK AND NAIL ALL EDGES BETWEEN STUDS.
12. **INTERIOR BEARING WALL CONSTRUCTION (UNO):** 2x STUDS @ 16". SHEATHED WITH 1/2" MIN. GYPSUM WALLBOARD ON EACH FACE. ATTACH W/ NO. 6 x 1 1/4" DRYWALL SCREWS @ 8" AT PANEL EDGES AND BOUNDARIES AND @ 12" IN FIELD OF PANEL.
13. **WALL OPENING CONSTRUCTION (UNO):** (2) 2x8 HEADER W/ MINIMUM (1) 2x6 TRIM AND (1) 2x6 KING STUD EACH END. HEADERS ARE DROPPED UNO.
14. **WOOD BEAM / JOIST FACE MOUNT HANGER TO FLUSH STEEL BEAM (UNO):** AT FLUSH STEEL BEAMS, PACK OUT WEB WITH 2x AND PLYWOOD/OSB TIGHT TO TOP AND BOTTOM FLANGES AS REQUIRED (MIN 2'-0" LENGTH) FOR FACE MOUNT HANGERS, SECURE WITH 1/2"Ø THRU BOLTS AND CONSTRUCTION ADHESIVE @ 24", STAGGERED.
15. **TYPICAL DECK TIE (UNO)**: MIN (2) DTT2Z PER DECK INSTALLED TO SIDE OF DECK JOIST, W/ 1/2" Ø THREADED ROD PENETRATING THROUGH RIM TO EITHER BLOCKING OR FIRST JOIST OTHER SIDE. AT JOIST, PACK WEB PER MANUFACTURER AS REQUIRED & CONNECT W/ NUT AND WASHER. SEE PLAN FOR ADDITIONAL LOCATIONS AND INFORMATION.
16. **TYPICAL DECK CONSTRUCTION (UNO)** Xx EXTERIOR DECKING OVER WOOD JOISTS PER PLAN. LAY DECKING PERPENDICULAR TO FRAMING AND FASTEN DECKING TO JOIST W/ (2) #8x3" EXTERIOR DECK SCREWS PER BOARD. FLASH TOP OF MULTI-PLY JOISTS / BEAMS.
17. HDX INDICATES HOLDOWN THROUGH LEVEL SHOWN, SEE SX.XX. CONTRACTOR TO VERIFY LOCATIONS AND LAYOUT WITH FRAMING ABOVE
18. SWX INDICATES SHEAR WALL TO BE SHEATHED ON SIDE INDICATED BY ARROW (UNO) WITH SHEATHING PER SHEAR WALL SCHEDULE. SEE S6.01

LEVEL 1 KEYNOTE SCHEDULE

	DESCRIPTION

HANGER SCHEDULE

1. ALL HANGERS NOTED TO BE INSTALLED WITH NUMBER AND SIZE FASTENERS SPECIFIED BY MNFR. ANY SUBSTITUTIONS SHALL BE REVIEWED AND APPROVED BY ANTHEM

2. INSTALL HANGERS NOTED OR APPROVED EQUIVALENT

	DESCRIPTION	Face Fasteners	Joist Fasteners
1	LUS28	(6) 10d x 3" NAILS	(4) 10d x 3" NAILS
2	LUS28-2	(6) 10d x 3" NAILS	(4) 10d x 3" NAILS
3	HHUS410	(30) 10d x 3" NAILS	(10) 10d x 3" NAILS
4	ISU2.37/14	(12) 10d x 3" NAILS	-
5	IUS1.81/14	(12) 10d x 3" NAILS	-
6	U210-2	(14) 16d x 3 1/2" NAILS	(6) 10d x 3" NAILS
7	HUC212-2	(14) 10d x 3" NAILS	(6) 10d x 3" NAILS
8	LUC210Z	(10) 10d x 3" NAILS	(4) 10d x 3" NAILS

APPROVAL STAMPS:

No.	Date	Description

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ARCHITECT

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M.E.P. & F.P. ENGINEERS

INTERIOR DESIGNER:

PROJECT LOCATION

BASECAMP TOWNHOME

1950 CURVE COURT
STEAMBOAT SPRINGS, CO 80487

DRAWING TITLE

TH4 LEVEL 4 FRAMING PLAN

SEAL

DATE: 07/11/22

DRAWN BY:

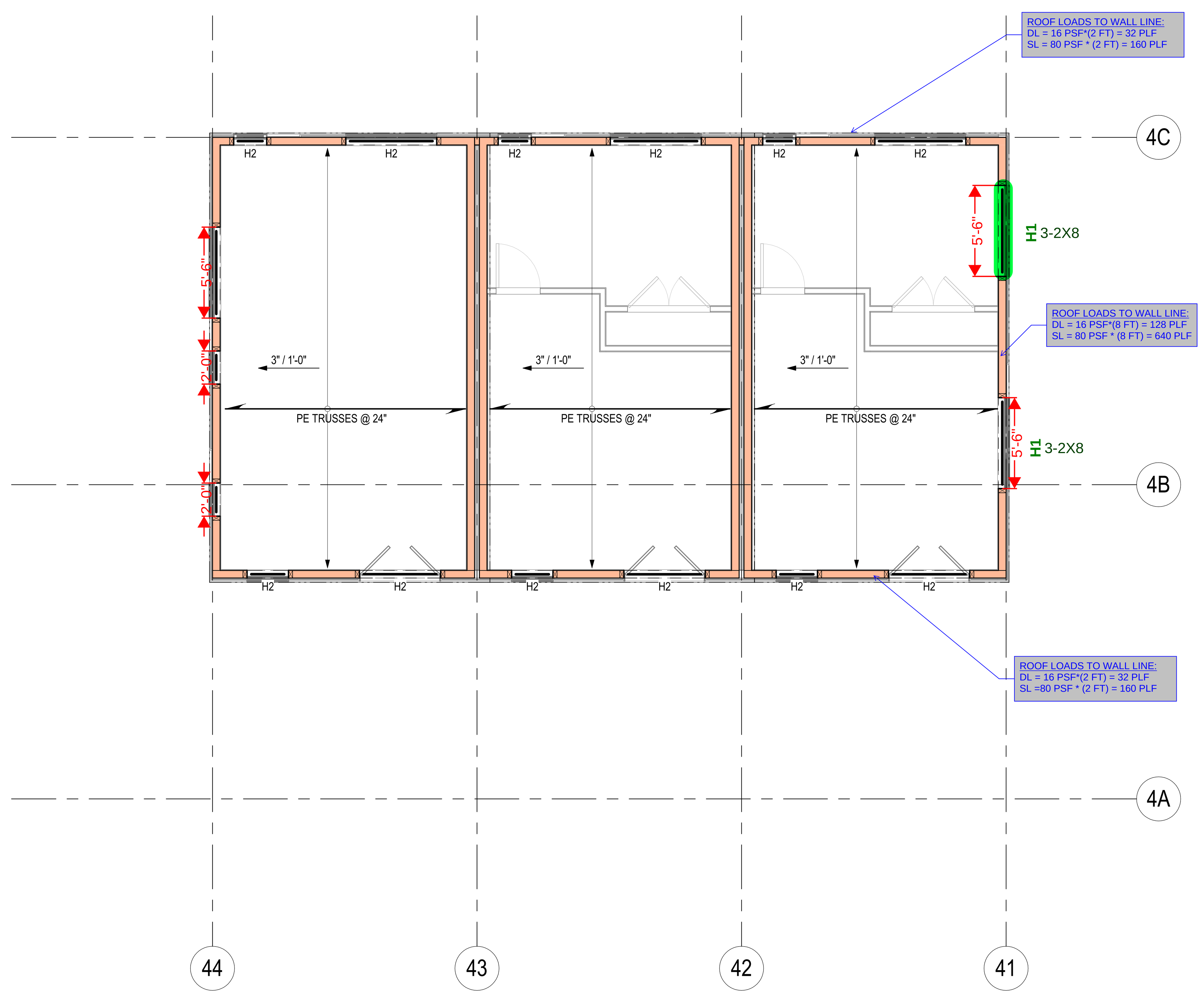
CHECKED BY:

PROJECT NO: 22-048

DRAWING NO:

S1.23

7/15/2022 8:53:13 AM



TH4 ROOF FRAMING PLAN

3/16" = 1'-0"

North

ROOF LEVEL PLAN NOTES:

1. SEE S0.01 FOR GENERAL STRUCTURAL NOTES, ABBREVIATIONS KEY AND LEGEND
2. SEE S5.01 FOR TYPICAL DETAILS
3. SEE S6.01 FOR SCHEDULES
4. TOP OF PLATE HEIGHT = XXX'-X" UNO
5. INSTALL LOOSE LINTEL PER SX.XX AT ALL OPENINGS W/ VENEER ABOVE
6. WALL FRAMING AND COLUMNS SHOWN SUPPORT THE FRAMING ON THIS LEVEL
7. REFER TO ARCHITECTURAL DRAWINGS FOR ADDITIONAL INFORMATION AND DIMENSIONS.
8. **ROOF CONSTRUCTION (UNO)** 15/32" NOMINAL APA 32/16 OR 5/8" NOMINAL APA 40/20 RATED SHEATHING OVER PE TRUSSES / RAFTERS SEE PLAN. FASTEN SHEATHING TO TRUSSES / RAFTERS, RIMS, AND LEDGERS WITH 8d GUN NAILS (0.113" x 2 3/8") @ 4" ALONG PANEL EDGES AND @ 8" ALONG INTERMEDIATE FRAMING MEMBERS. LAY PANELS PERPENDICULAR TO FRAMING MEMBERS AND STAGGER PANEL JOINTS.
9. **TYPICAL TRUSS / RAFTER TIE DOWN:** H2.5T AT BEARING, (2) CLIPS WITHIN 8'-0" OF CORNERS (UNO)
10. **TYPICAL GIRDER TRUSS / ROOF BEAM TIE DOWN:** (2) HTS20 (OR ST22 STRAP) EACH END AT BEARING (UNO)
11. **EXTERIOR WALL CONSTRUCTION (UNO):** 2x STUDS @ 16". SHEATH WITH 1 1/2" ZIP-R SHEATHING ON EXTERIOR FACE. NAIL WALL SHEATHING W/ 10d GUN NAILS (0.131"Ø x 3") @ 3" PANEL EDGES AND BOUNDARIES AND @ 12" IN FIELD OF PANEL. BLOCK AND NAIL ALL EDGES BETWEEN STUDS.
12. **EXTERIOR WALL CONSTRUCTION (UNO):** 2x STUDS @ 16" SHEATHED WITH 7/16" CDX PLYWOOD OR OSB, APA 24/16 ON EXTERIOR FACE. NAIL WALL SHEATHING WTH 8d GUN NAILS (0.131"Ø x 2 3/8") @ 4" AT PANEL EDGES AND BOUNDARIES AND @ 12" IN FIELD OF PANEL. BLOCK AND NAIL ALL EDGES BETWEEN STUDS.
13. **WALL OPENING CONSTRUCTION (UNO):** (2) 2x8 HEADER W/ MINIMUM (1) 2x6 TRIM AND (1) 2x6 KING STUD EACH END.
14. SWX **INDICATES SHEAR WALL:** TO BE SHEATHED ON SIDE INDICATED BY ARROW (UNO) WITH WOOD SHEATHING PER SHEAR WALL SCHEDULE. SEE SHEET SS6.01
15. **INDICATES OVERFRAMING:** CONTINUE PRIMARY ROOF SHEATHING BELOW OVERFRAMED AREAS. OVERFRAME WITH RAFTERS/TRUSSES SPACED 24" MAX, PLACE RAFTERS/TRUSSES PERPINDICULAR TO FRAMING BELOW.

ROOF KEYNOTE SCHEDULE

	DESCRIPTION

HANGER SCHEDULE

1. ALL HANGERS NOTED TO BE INSTALLED WITH NUMBER AND SIZE FASTENERS SPECIFIED BY MNFR. ANY SUBSTITUTIONS SHALL BE REVIEWED AND APPROVED BY ANTHEM

2. INSTALL HANGERS NOTED OR APPROVED EQUIVALENT

	DESCRIPTION	Face Fasteners	Joist Fasteners
1	LUS28	(6) 10d x 3" NAILS	(4) 10d x 3" NAILS
2	LUS28-2	(6) 10d x 3" NAILS	(4) 10d x 3" NAILS
3	HHUS410	(30) 10d x 3" NAILS	(10) 10d x 3" NAILS
4	ISU2.37/14	(12) 10d x 3" NAILS	-
5	IUS1.81/14	(12) 10d x 3" NAILS	-
6	U210-2	(14) 16d x 3 1/2" NAILS	(6) 10d x 3" NAILS
7	HUC212-2	(14) 10d x 3" NAILS	(6) 10d x 3" NAILS
8	LUC210Z	(10) 10d x 3" NAILS	(4) 10d x 3" NAILS

APPROVAL STAMPS:

No.	Date	Description

SUBMISSIONS & REVISIONS

OWNER

ARCHITECT

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M.E.P. & F.P. ENGINEERS

INTERIOR DESIGNER:

PROJECT LOCATION

BASECAMP TOWNHOME

1950 CURVE COURT
STEAMBOAT SPRINGS, CO 80487

DRAWING TITLE

TH4 ROOF FRAMING PLAN

SEAL

DATE: 07/11/22

DRAWN BY:

CHECKED BY:

PROJECT NO: 22-048

DRAWING NO:

S1.24

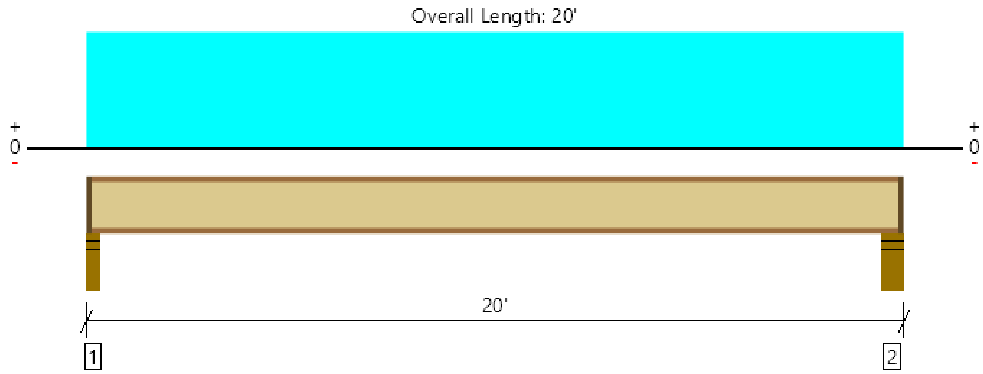
7/15/2022 8:53:16 AM

MEMBER REPORT

PASSED

4 bed unit typ joist, Floor Joist #1 - 20' span

1 piece(s) 14" TJI® 360 @ 16" OC



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

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	732 @ 2 1/4"	1141 (2.00")	Passed (64%)	1.00	1.0 D + 1.0 L (All Spans)
Shear (lbs)	719 @ 3 1/4"	1955	Passed (37%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	3526 @ 9' 10 7/8"	7335	Passed (48%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.277 @ 9' 10 7/8"	0.486	Passed (L/843)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.387 @ 9' 10 7/8"	0.972	Passed (L/602)	--	1.0 D + 1.0 L (All Spans)
TJ-Pro™ Rating	46	40	Passed	--	--

System : Floor
Member Type : Joist
Building Use : Residential
Building Code : IBC 2018
Design Methodology : ASD

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

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Floor Live	Factored	
1 - Stud wall - HF	3.25"	2.00"	1.75"	211	528	740	1 1/4" Rim Board
2 - Stud wall - HF	5.50"	4.25"	1.75"	215	538	754	1 1/4" Rim Board

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

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	5' 6" o/c	
Bottom Edge (Lu)	19' 10" o/c	

- TJI joists are only analyzed using Maximum Allowable bracing solutions.
- Maximum allowable bracing intervals based on applied load.

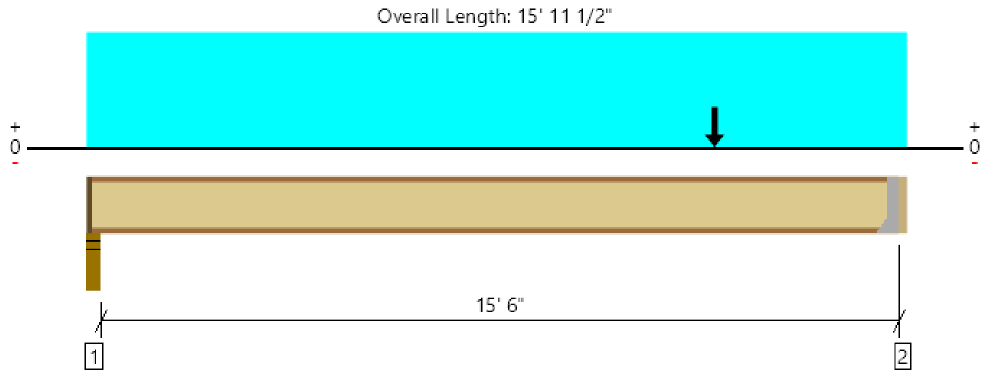
Vertical Load	Location	Spacing	Dead (0.90)	Floor Live (1.00)	Comments
1 - Uniform (PSF)	0 to 20'	16"	16.0	40.0	Default Load

Weyerhaeuser Notes

ForteWEB Software Operator	Job Notes
Samantha Taylor Anthem Structural Engineers (303) 848-8497 staylor@anthemstructural.com	

4 bed unit typ joist, Floor Joist #2 - 15'-6" span

1 piece(s) 14" TJI® 360 @ 16" OC



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

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	726 @ 15' 9 1/2"	1080 (1.75")	Passed (67%)	1.00	1.0 D + 1.0 L (All Spans)
Shear (lbs)	726 @ 15' 9 1/2"	1955	Passed (37%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	2609 @ 8' 6 13/16"	7335	Passed (36%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.139 @ 8' 1 1/4"	0.390	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.200 @ 8' 1 1/2"	0.779	Passed (L/936)	--	1.0 D + 1.0 L (All Spans)
TJ-Pro™ Rating	53	40	Passed	--	--

System : Floor
Member Type : Joist
Building Use : Residential
Building Code : IBC 2018
Design Methodology : ASD

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

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Floor Live	Factored	
1 - Stud wall - HF	3.50"	2.25"	1.75"	189	451	640	1 1/4" Rim Board
2 - Hanger on 14" HF beam	2.00"	Hanger[1]	1.75" / - [2]	232	507	738	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.
- [2] Required Bearing Length / Required Bearing Length with Web Stiffeners

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	6' 5" o/c	
Bottom Edge (Lu)	15' 8" o/c	

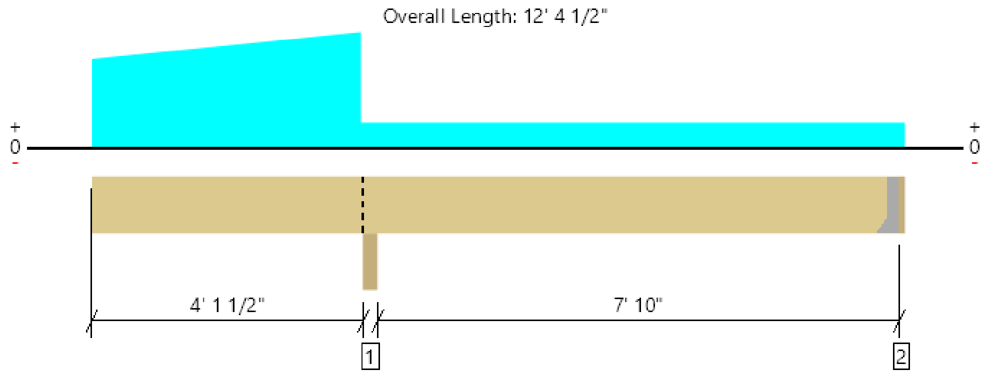
- TJI joists are only analyzed using Maximum Allowable bracing solutions.
- Maximum allowable bracing intervals based on applied load.

Connector: Simpson Strong-Tie						
Support	Model	Seat Length	Top Fasteners	Face Fasteners	Member Fasteners	Accessories
2 - Face Mount Hanger	IUS2.37/14	2.00"	N/A	12-10dx1.5	2-Strong-Grip	

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

Vertical Loads	Location	Spacing	Dead (0.90)	Floor Live (1.00)	Comments
1 - Uniform (PSF)	0 to 15' 11 1/2"	16"	16.0	40.0	Default Load
2 - Point (PLF)	12' 3"	16"	60.0	80.0	HALF WALL ABOVE

4 bed unit typ joist, Floor Joist #3 - 12'-6" span
1 piece(s) 2 x 12 HF No.2 @ 16" OC



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

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	1287 @ 4' 3 1/4"	2126 (3.50")	Passed (61%)	--	1.0 D + 0.75 L + 0.75 S (All Spans)
Shear (lbs)	614 @ 3' 2 1/4"	1941	Passed (32%)	1.15	1.0 D + 0.75 L + 0.75 S (All Spans)
Moment (Ft-lbs)	-1738 @ 4' 3 1/4"	2964	Passed (59%)	1.15	1.0 D + 0.75 L + 0.75 S (All Spans)
Live Load Defl. (in)	0.183 @ 0	0.285	Passed (2L/562)	--	1.0 D + 0.75 L + 0.75 S (Alt Spans)
Total Load Defl. (in)	0.191 @ 0	0.427	Passed (2L/536)	--	1.0 D + 0.75 L + 0.75 S (Alt Spans)
TJ-Pro™ Rating	N/A	N/A	N/A	--	N/A

System : Floor
Member Type : Joist
Building Use : Residential
Building Code : IBC 2018
Design Methodology : ASD

- Deflection criteria: LL (L/360) and TL (L/240).
- Overhang deflection criteria: LL (2L/360) and TL (2L/240).
- Left cantilever length exceeds 1/3 member length or 1/2 back span length. Additional bracing should be considered.
- 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	Snow	Factored	
1 - Beam - HF	3.50"	3.50"	2.12"	201	642	807	1287	Blocking
2 - Hanger on 11 1/4" HF beam	1.50"	Hanger[1]	1.50"	63	219/-85	-167	283/-125	See note [1]

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

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	12' 3" o/c	
Bottom Edge (Lu)	6' 6" o/c	

•Maximum allowable bracing intervals based on applied load.

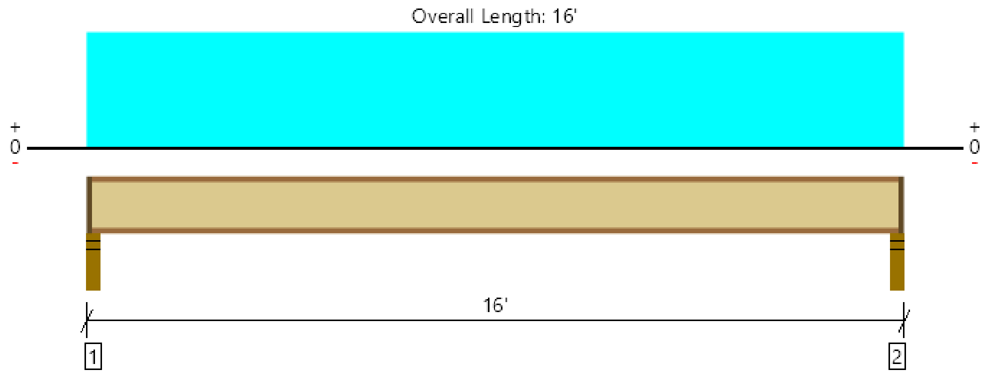
Connector: Simpson Strong-Tie						
Support	Model	Seat Length	Top Fasteners	Face Fasteners	Member Fasteners	Accessories
2 - Face Mount Hanger	LUS28	1.75"	N/A	6-10dx1.5	3-10d	

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

Vertical Loads	Location (Side)	Spacing	Dead (0.90)	Floor Live (1.00)	Snow (1.15)	Wind (1.60)	Comments
1 - Uniform (PSF)	0 to 12' 4 1/2"	16"	16.0	40.0	-	-	Floor Load
2 - Uniform (PSF)	0 to 4' 1 1/2"	16"	-	20.0	-	21.9	Balcony Load
3 - Uniform (PSF)	0 to 4' 1 1/2"	16"	-	-	-	-21.9	Wind Uplift
4 - Tapered (PLF)	0 to 4' 1 1/2"	N/A	-	-	125.0 to 185.6	-	Balcony Snow Load

ForteWEB Software Operator	Job Notes
Samantha Taylor Anthem Structural Engineers (303) 848-8497 staylor@anthemstructural.com	

4 bed unit typ joist, Floor Joist #4 - 16' span
1 piece(s) 14" TJI® 110 @ 16" OC



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

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	590 @ 2 1/4"	976 (2.00")	Passed (60%)	1.00	1.0 D + 1.0 L (All Spans)
Shear (lbs)	577 @ 3 1/4"	1860	Passed (31%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	2279 @ 8'	3740	Passed (61%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.171 @ 8'	0.391	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.240 @ 8'	0.781	Passed (L/781)	--	1.0 D + 1.0 L (All Spans)
TJ-Pro™ Rating	51	40	Passed	--	--

System : Floor
Member Type : Joist
Building Use : Residential
Building Code : IBC 2018
Design Methodology : ASD

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

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Floor Live	Factored	
1 - Stud wall - HF	3.25"	2.00"	1.75"	171	427	597	1 1/4" Rim Board
2 - Stud wall - HF	3.25"	2.00"	1.75"	171	427	597	1 1/4" Rim Board

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

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	4' 1" o/c	
Bottom Edge (Lu)	15' 10" o/c	

•TJI joists are only analyzed using Maximum Allowable bracing solutions.
•Maximum allowable bracing intervals based on applied load.

Vertical Load	Location	Spacing	Dead (0.90)	Floor Live (1.00)	Comments
1 - Uniform (PSF)	0 to 16'	16"	16.0	40.0	Floor Load

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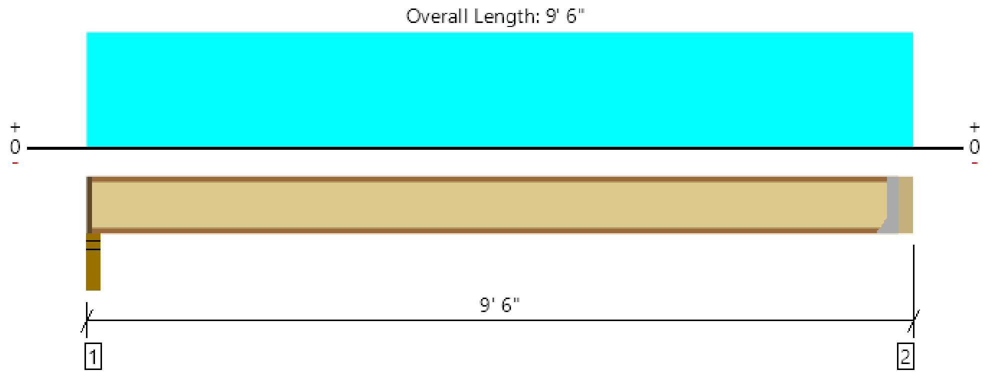
The product application, input design loads, dimensions and support information have been provided by ForteWEB Software Operator

ForteWEB Software Operator	Job Notes
Samantha Taylor Anthem Structural Engineers (303) 848-8497 staylor@anthemstructural.com	

MEMBER REPORT

PASSED

4 bed unit typ joist, Floor Joist #5 - 9.5' span
1 piece(s) 14" TJI® 110 @ 16" OC



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

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	337 @ 9' 2 1/2"	910 (1.75")	Passed (37%)	1.00	1.0 D + 1.0 L (All Spans)
Shear (lbs)	337 @ 9' 2 1/2"	1860	Passed (18%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	760 @ 4' 8 3/8"	3740	Passed (20%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.025 @ 4' 8 3/8"	0.226	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.034 @ 4' 8 3/8"	0.451	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)
TJ-Pro™ Rating	65	40	Passed	--	--

System : Floor
Member Type : Joist
Building Use : Residential
Building Code : IBC 2018
Design Methodology : ASD

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

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Floor Live	Factored	
1 - Stud wall - HF	3.25"	2.00"	1.75"	100	251	351	1 1/4" Rim Board
2 - Hanger on 14" HF beam	3.50"	Hanger¹	1.75" / - ²	102	256	359	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.
- ² Required Bearing Length / Required Bearing Length with Web Stiffeners

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	7' 3" o/c	
Bottom Edge (Lu)	9' 1" o/c	

- TJI joists are only analyzed using Maximum Allowable bracing solutions.
- Maximum allowable bracing intervals based on applied load.

Connector: Simpson Strong-Tie

Support	Model	Seat Length	Top Fasteners	Face Fasteners	Member Fasteners	Accessories
2 - Face Mount Hanger	IUS1.81/14	2.00"	N/A	12-10dx1.5	2-Strong-Grip	

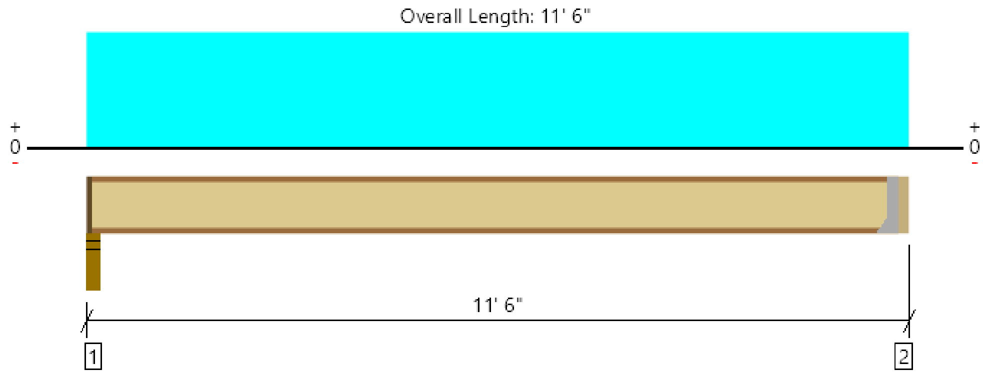
- Refer to manufacturer notes and instructions for proper installation and use of all connectors.

Vertical Load	Location	Spacing	Dead (0.90)	Floor Live (1.00)	Comments
1 - Uniform (PSF)	0 to 9' 6"	16"	16.0	40.0	Floor Load

Weyerhaeuser Notes

ForteWEB Software Operator	Job Notes
Samantha Taylor Anthem Structural Engineers (303) 848-8497 staylor@anthemstructural.com	

4 bed unit typ joist, Floor Joist #6 - 11'-6" span
1 piece(s) 14" TJI® 110 @ 16" OC



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

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	415 @ 11' 3 1/2"	910 (1.75")	Passed (46%)	1.00	1.0 D + 1.0 L (All Spans)
Shear (lbs)	415 @ 11' 3 1/2"	1860	Passed (22%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	1151 @ 5' 8 7/8"	3740	Passed (31%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.050 @ 5' 8 7/8"	0.278	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.070 @ 5' 8 7/8"	0.555	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)
TJ-Pro™ Rating	61	40	Passed	--	--

System : Floor
Member Type : Joist
Building Use : Residential
Building Code : IBC 2018
Design Methodology : ASD

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

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Floor Live	Factored	
1 - Stud wall - HF	3.25"	2.00"	1.75"	122	306	429	1 1/4" Rim Board
2 - Hanger on 14" HF beam	2.50"	Hanger[1]	1.75" / - [2]	123	307	430	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.
- [2] Required Bearing Length / Required Bearing Length with Web Stiffeners

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	5' 10" o/c	
Bottom Edge (Lu)	11' 2" o/c	

- TJI joists are only analyzed using Maximum Allowable bracing solutions.
- Maximum allowable bracing intervals based on applied load.

Connector: Simpson Strong-Tie

Support	Model	Seat Length	Top Fasteners	Face Fasteners	Member Fasteners	Accessories
2 - Face Mount Hanger	IUS1.81/14	2.00"	N/A	12-10dx1.5	2-Strong-Grip	

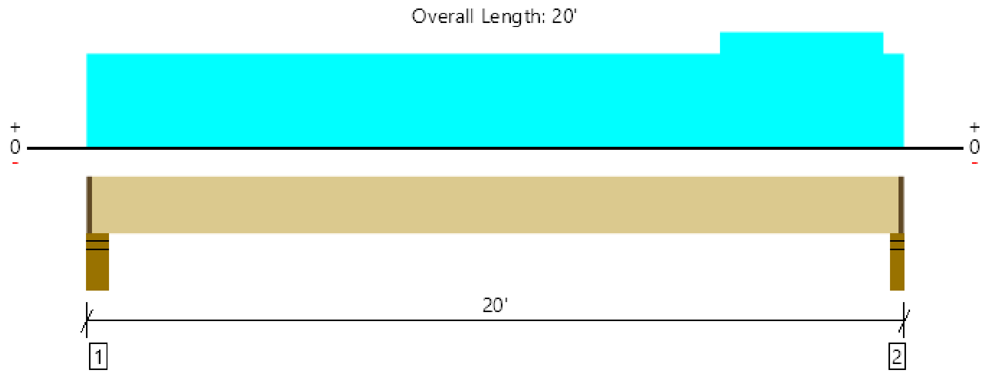
- Refer to manufacturer notes and instructions for proper installation and use of all connectors.

Vertical Load	Location	Spacing	Dead (0.90)	Floor Live (1.00)	Comments
1 - Uniform (PSF)	0 to 11' 6"	16"	16.0	40.0	Floor Load

Weyerhaeuser Notes

ForteWEB Software Operator	Job Notes
Samantha Taylor Anthem Structural Engineers (303) 848-8497 staylor@anthemstructural.com	

4 bed unit typ joist, Floor Joist #7 - 20' span
2 piece(s) 1 3/4" x 14" 2.0E Microllam® LVL @ 16" OC



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

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	2404 @ 19' 9 3/4"	2835 (2.00")	Passed (85%)	--	1.0 D + 0.75 L + 0.75 S (All Spans)
Shear (lbs)	2047 @ 18' 6 3/4"	10707	Passed (19%)	1.15	1.0 D + 0.75 L + 0.75 S (All Spans)
Moment (Ft-lbs)	10764 @ 10' 2 7/8"	29013	Passed (37%)	1.15	1.0 D + 0.75 L + 0.75 S (All Spans)
Live Load Defl. (in)	0.405 @ 10' 1 1/8"	0.648	Passed (L/576)	--	1.0 D + 0.75 L + 0.75 S (All Spans)
Total Load Defl. (in)	0.464 @ 10' 1 5/8"	0.972	Passed (L/503)	--	1.0 D + 0.75 L + 0.75 S (All Spans)
TJ-Pro™ Rating	60	40	Passed	--	--

System : Floor
Member Type : Joist
Building Use : Residential
Building Code : IBC 2018
Design Methodology : ASD

- Deflection criteria: LL (L/360) and TL (L/240).
- Allowed moment does not reflect the adjustment for the beam stability factor.
- A 4% increase in the moment capacity has been added to account for repetitive member usage.
- A structural analysis of the deck has not been performed.
- Deflection analysis is based on composite action with a single layer of 23/32" Weyerhaeuser Edge™ Panel (24" Span Rating) that is glued and nailed down.
- Additional considerations for the TJ-Pro™ Rating include: None.

Supports	Bearing Length			Loads to Supports (lbs)				Accessories
	Total	Available	Required	Dead	Floor Live	Snow	Factored	
1 - Stud wall - HF	5.50"	4.25"	1.58"	247	808	1884	2266	1 1/4" Rim Board
2 - Stud wall - HF	3.25"	2.00"	1.70"	446	792	1849	2428	1 1/4" Rim Board

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

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	16' 1" o/c	
Bottom Edge (Lu)	19' 10" o/c	

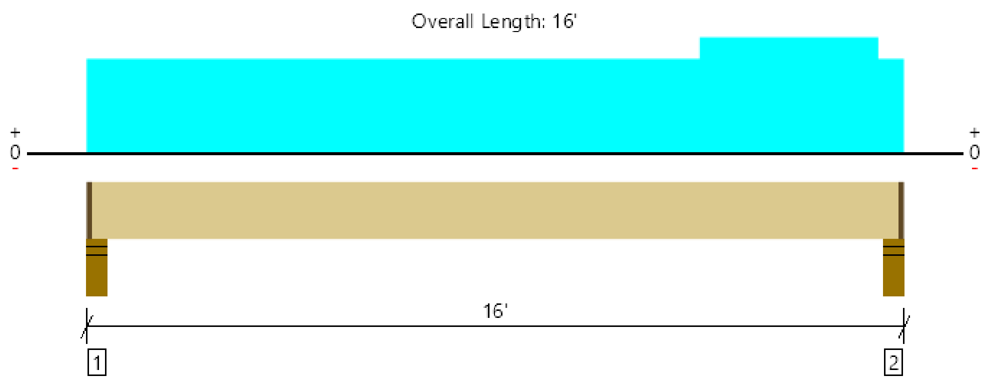
•Maximum allowable bracing intervals based on applied load.

Vertical Loads	Location (Side)	Spacing	Dead (0.90)	Floor Live (1.00)	Snow (1.15)	Comments
1 - Uniform (PSF)	0 to 20'	16"	16.0	60.0	140.0	Deck Loads
2 - Uniform (PSF)	15' 6" to 19' 6"	16"	50.0	-	-	Housekeeping Slab

Weyerhaeuser Notes

ForteWEB Software Operator	Job Notes
Samantha Taylor Anthem Structural Engineers (303) 848-8497 staylor@anthemstructural.com	

4 bed unit typ joist, Floor Joist #8 - 16' span
1 piece(s) 1 3/4" x 14" 2.0E Microllam® LVL @ 16" OC



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

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	1952 @ 15' 8"	2658 (3.75")	Passed (73%)	--	1.0 D + 0.75 L + 0.75 S (All Spans)
Shear (lbs)	1552 @ 14' 5"	5353	Passed (29%)	1.15	1.0 D + 0.75 L + 0.75 S (All Spans)
Moment (Ft-lbs)	6730 @ 8' 1 9/16"	14506	Passed (46%)	1.15	1.0 D + 0.75 L + 0.75 S (All Spans)
Live Load Defl. (in)	0.313 @ 8'	0.383	Passed (L/589)	--	1.0 D + 0.75 L + 0.75 S (All Spans)
Total Load Defl. (in)	0.360 @ 8' 7/16"	0.767	Passed (L/512)	--	1.0 D + 0.75 L + 0.75 S (All Spans)
TJ-Pro™ Rating	61	40	Passed	--	--

System : Floor
Member Type : Joist
Building Use : Residential
Building Code : IBC 2018
Design Methodology : ASD

- Deflection criteria: LL (L/480) and TL (L/240).
- Allowed moment does not reflect the adjustment for the beam stability factor.
- A 4% increase in the moment capacity has been added to account for repetitive member usage.
- A structural analysis of the deck has not been performed.
- Deflection analysis is based on composite action with a single layer of 23/32" Weyerhaeuser Edge™ Panel (24" Span Rating) that is glued and nailed down.
- Additional considerations for the TJ-Pro™ Rating include: None.

Supports	Bearing Length			Loads to Supports (lbs)				Accessories
	Total	Available	Required	Dead	Floor Live	Snow	Factored	
1 - Stud wall - HF	5.00"	3.75"	2.51"	200	640	1493	1800	1 1/4" Rim Board
2 - Stud wall - HF	5.00"	3.75"	2.75"	375	640	1493	1975	1 1/4" Rim Board

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

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	7' 2" o/c	
Bottom Edge (Lu)	15' 10" o/c	

•Maximum allowable bracing intervals based on applied load.

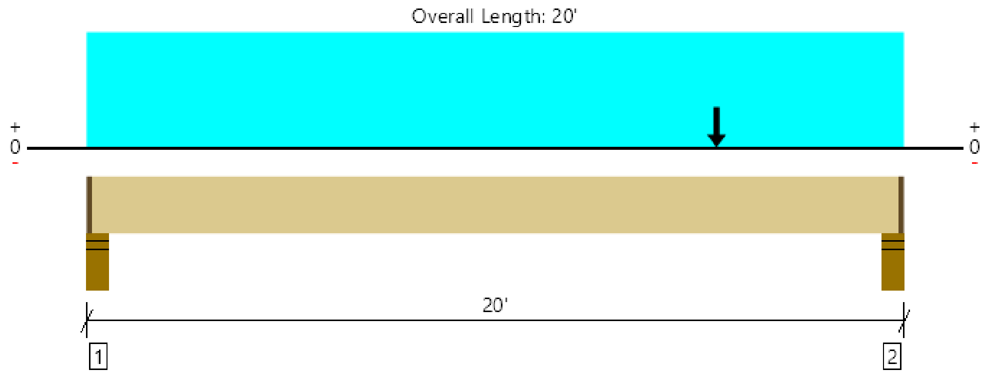
Vertical Loads	Location (Side)	Spacing	Dead (0.90)	Floor Live (1.00)	Snow (1.15)	Comments
1 - Uniform (PSF)	0 to 16'	16"	16.0	60.0	140.0	Deck Load
2 - Uniform (PSF)	12' to 15' 6"	16"	50.0	-	-	Housekeeping Slab

Weyerhaeuser Notes

ForteWEB Software Operator	Job Notes
Samantha Taylor Anthem Structural Engineers (303) 848-8497 staylor@anthemstructural.com	

4 bed unit typ joist, Floor Beam #1

3 piece(s) 1 3/4" x 14" 2.0E Microllam® LVL



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

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	3610 @ 19' 8"	9037 (4.25")	Passed (40%)	--	1.0 D + 1.0 L (All Spans)
Shear (lbs)	3464 @ 18' 4 1/2"	13965	Passed (25%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	14380 @ 15' 5"	36387	Passed (40%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.236 @ 10' 9 13/16"	0.483	Passed (L/981)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.376 @ 10' 9 1/4"	0.967	Passed (L/618)	--	1.0 D + 1.0 L (All Spans)

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

System : Floor
Member Type : Flush Beam
Building Use : Residential
Building Code : IBC 2018
Design Methodology : ASD

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Floor Live	Factored	
1 - Stud wall - HF	5.50"	4.25"	1.50"	673	1035	1708	1 1/4" Rim Board
2 - Stud wall - HF	5.50"	4.25"	1.70"	1304	2314	3618	1 1/4" Rim Board

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

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	19' 10" o/c	
Bottom Edge (Lu)	19' 10" o/c	

•Maximum allowable bracing intervals based on applied load.

Vertical Loads	Location (Side)	Tributary Width	Dead (0.90)	Floor Live (1.00)	Comments
0 - Self Weight (PLF)	1 1/4" to 19' 10 3/4"	N/A	21.5	--	
1 - Uniform (PSF)	0 to 20' (Top)	1' 4"	16.0	40.0	Floor Load
2 - Point (lb)	15' 5" (Back)	N/A	1126	2282	Linked from: Floor Beam #3, Support 1

Member Notes
Stair Framing Beam

Weyerhaeuser Notes

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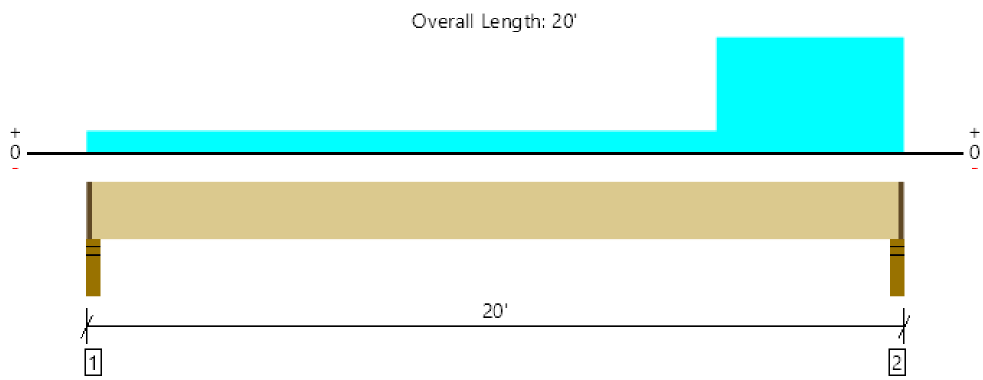
The product application, input design loads, dimensions and support information have been provided by ForteWEB Software Operator

4 bed unit typ joist, Floor Beam #2

3 piece(s) 1 3/4" x 14" 2.0E Microllam® LVL



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

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	2250 @ 19' 10 1/4"	4253 (2.00")	Passed (53%)	--	1.0 D + 1.0 L (All Spans)
Shear (lbs)	1686 @ 18' 6 3/4"	13965	Passed (12%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	6411 @ 11' 8 3/8"	36387	Passed (18%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.120 @ 10' 5 3/8"	0.493	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.200 @ 10' 4 1/2"	0.985	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)

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

System : Floor
Member Type : Flush Beam
Building Use : Residential
Building Code : IBC 2018
Design Methodology : ASD

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Floor Live	Factored	
1 - Stud wall - HF	3.25"	2.00"	1.50"	472	650	1122	1 1/4" Rim Board
2 - Stud wall - HF	3.25"	2.00"	1.50"	806	1485	2291	1 1/4" Rim Board

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

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	19' 10" o/c	
Bottom Edge (Lu)	19' 10" o/c	

•Maximum allowable bracing intervals based on applied load.

Vertical Loads	Location (Side)	Tributary Width	Dead (0.90)	Floor Live (1.00)	Comments
0 - Self Weight (PLF)	1 1/4" to 19' 10 3/4"	N/A	21.5	--	
1 - Uniform (PSF)	0 to 20' (Top)	1' 4"	16.0	40.0	Floor Load
2 - Uniform (PLF)	15' 5" to 20' (Front)	N/A	93.3	233.0	Stair Load

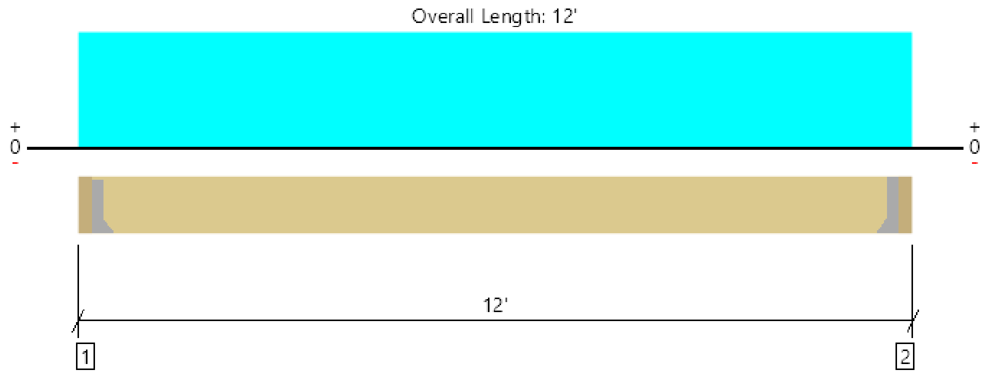
Member Notes

Stair Framing Beam

Weyerhaeuser Notes

ForteWEB Software Operator	Job Notes
Samantha Taylor Anthem Structural Engineers (303) 848-8497 staylor@anthemstructural.com	

4 bed unit typ joist, Floor Beam #3
2 piece(s) 1 3/4" x 14" 2.0E Microllam® LVL



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

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	3257 @ 3 1/4"	3938 (1.50")	Passed (83%)	--	1.0 D + 1.0 L (All Spans)
Shear (lbs)	2594 @ 1' 5 1/4"	9310	Passed (28%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	9331 @ 6'	24258	Passed (38%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.107 @ 6'	0.286	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.160 @ 6'	0.573	Passed (L/861)	--	1.0 D + 1.0 L (All Spans)

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

System : Floor
Member Type : Flush Beam
Building Use : Residential
Building Code : IBC 2018
Design Methodology : ASD

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Floor Live	Factored	
1 - Hanger on 14" HF beam	3.25"	Hanger¹	1.50"	1126	2282	3407	See note ¹
2 - Hanger on 14" HF beam	3.25"	Hanger¹	1.50"	1126	2282	3407	See note ¹

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

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	11' 6" o/c	
Bottom Edge (Lu)	11' 6" o/c	

•Maximum allowable bracing intervals based on applied load.

Connector: Simpson Strong-Tie						
Support	Model	Seat Length	Top Fasteners	Face Fasteners	Member Fasteners	Accessories
1 - Face Mount Hanger	HHUS410	3.00"	N/A	30-10d	10-10d	
2 - Face Mount Hanger	HHUS410	3.00"	N/A	30-10d	10-10d	

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

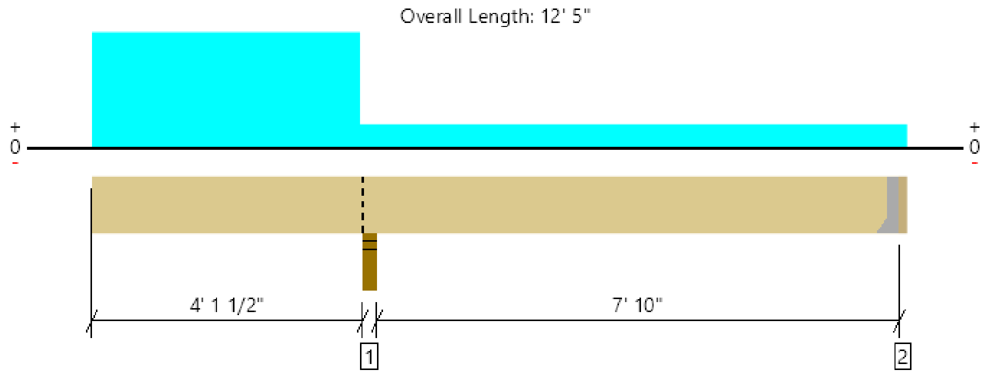
Vertical Loads	Location (Side)	Tributary Width	Dead (0.90)	Floor Live (1.00)	Comments
0 - Self Weight (PLF)	3 1/4" to 11' 8 3/4"	N/A	14.3	--	
1 - Uniform (PLF)	0 to 12' (Front)	N/A	174.0	380.3	Linked from: Floor Joist #2 - 15'-6" span, Support 2

Member Notes
Stair Framing Beam

ForteWEB Software Operator	Job Notes
Samantha Taylor Anthem Structural Engineers (303) 848-8497 staylor@anthemstructural.com	

4 bed unit typ joist, Floor Beam #5

2 piece(s) 2 x 12 HF No.2



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

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	1837 @ 4' 3 1/4"	4253 (3.50")	Passed (43%)	--	1.0 D + 0.75 L + 0.75 S (All Spans)
Shear (lbs)	963 @ 3' 2 1/4"	3881	Passed (25%)	1.15	1.0 D + 1.0 S (All Spans)
Moment (Ft-lbs)	-2752 @ 4' 3 1/4"	5155	Passed (53%)	1.15	1.0 D + 1.0 S (All Spans)
Live Load Defl. (in)	0.147 @ 0	0.214	Passed (2L/696)	--	1.0 D + 1.0 S (All Spans)
Total Load Defl. (in)	0.154 @ 0	0.427	Passed (2L/668)	--	1.0 D + 1.0 S (All Spans)

System : Floor
Member Type : Flush Beam
Building Use : Residential
Building Code : IBC 2018
Design Methodology : ASD

- Deflection criteria: LL (L/480) and TL (L/240).
- Overhang deflection criteria: LL (2L/480) and TL (2L/240).
- Left cantilever length exceeds 1/3 member length or 1/2 back span length. Additional bracing should be considered.
- Allowed moment does not reflect the adjustment for the beam stability factor.
- -222 lbs uplift at support located at 12' 3". Strapping or other restraint may be required.
- Applicable calculations are based on NDS.

Supports	Bearing Length			Loads to Supports (lbs)				Accessories
	Total	Available	Required	Dead	Floor Live	Snow	Factored	
1 - Stud wall - HF	3.50"	3.50"	1.51"	281	642	1433	1837	Blocking
2 - Hanger on 11 1/4" HF beam	2.00"	Hanger[1]	1.50"	89	222/-83	-311	310/-222	See note [1]

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

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	12' 3" o/c	
Bottom Edge (Lu)	12' 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	LUS28-2	2.00"	N/A	6-10dx1.5	3-10d	

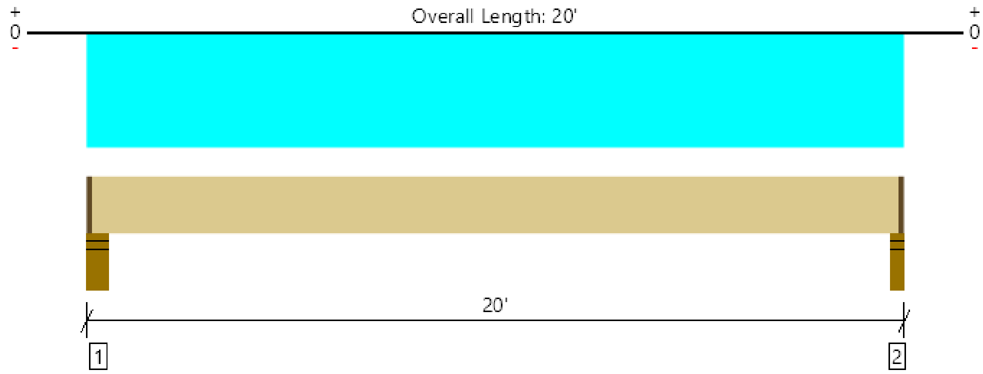
- Refer to manufacturer notes and instructions for proper installation and use of all connectors.

Vertical Loads	Location (Side)	Tributary Width	Dead (0.90)	Floor Live (1.00)	Snow (1.15)	Wind (1.60)	Comments
0 - Self Weight (PLF)	0 to 12' 3"	N/A	8.6	--	--	--	
1 - Uniform (PSF)	0 to 12' 5" (Top)	1' 4"	16.0	40.0	-	-	Floor Load
2 - Uniform (PSF)	0 to 4' 1 1/2" (Top)	1' 4"	-	20.0	204.1	21.9	Balcony Load
3 - Uniform (PSF)	0 to 4' 1 1/2" (Top)	1' 4"	-	-	-	-21.9	Balcony Wind Uplift

Member Notes
Stair Framing Beam

ForteWEB Software Operator	Job Notes
Samantha Taylor Anthem Structural Engineers (303) 848-8497 staylor@anthemstructural.com	

4 bed unit typ joist, Floor Beam #6
2 piece(s) 1 3/4" x 14" 2.0E Microllam® LVL



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

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern) [Group]
Member Reaction (lbs)	2579 @ 19' 10 1/4"	2835 (2.00")	Passed (91%)	--	1.0 D + 1.0 L (All Spans) [1]
Shear (lbs)	2228 @ 1' 7 1/2"	9310	Passed (24%)	1.00	1.0 D + 1.0 L (All Spans) [1]
Moment (Ft-lbs)	12533 @ 10' 1 1/8"	24258	Passed (52%)	1.00	1.0 D + 1.0 L (All Spans) [1]
Live Load Defl. (in)	0.411 @ 10' 1 1/8"	0.651	Passed (L/570)	--	1.0 D + 1.0 L (All Spans) [1]
Total Load Defl. (in)	0.567 @ 10' 1 1/8"	0.976	Passed (L/413)	--	1.0 D + 1.0 L (All Spans) [1]

System : Floor
Member Type : Flush Beam
Building Use : Residential
Building Code : IBC 2018
Design Methodology : ASD

- Deflection criteria: LL (L/360) and TL (L/240).
- Allowed moment does not reflect the adjustment for the beam stability factor.
- -537 lbs uplift at support located at 4". Strapping or other restraint may be required.
- -527 lbs uplift at support located at 19' 10 1/4". Strapping or other restraint may be required.

Supports	Bearing Length			Loads to Supports (lbs)				Accessories
	Total	Available	Required	Dead	Floor Live	Snow	Factored	
1 - Stud wall - HF	5.50"	4.25"	1.85"	727	1927/-377	-1264	2654/-537	1 1/4" Rim Board
2 - Stud wall - HF	3.25"	2.00"	1.82"	714	1891/-370	-1241	2605/-527	1 1/4" Rim Board

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

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	13' 2" o/c	
Bottom Edge (Lu)	19' 10" o/c	

•Maximum allowable bracing intervals based on applied load.

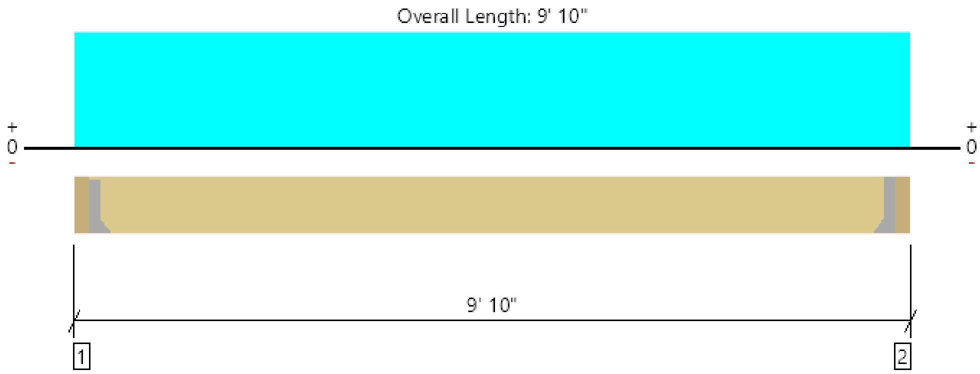
Vertical Loads	Location (Side)	Tributary Width	Dead (0.90)	Floor Live (1.00)	Snow (1.15)	Comments
0 - Self Weight (PLF)	1 1/4" to 19' 10 3/4"	N/A	14.3	--	--	
1 - Uniform (PSF)	0 to 20' (Top)	8"	16.0	40.0	-	Floor Load
2 - Uniform (PLF)	0 to 20' (Front)	N/A	47.3	164.3/-63.8	-125.3	Linked from: Floor Joist #3 - 12'-6" span, Support 2

Weyerhaeuser Notes

ForteWEB Software Operator	Job Notes
Samantha Taylor Anthem Structural Engineers (303) 848-8497 staylor@anthemstructural.com	

Weyerhaeuser

4 bed unit typ joist, Floor Beam #7
2 piece(s) 1 3/4" x 14" 2.0E Microllam® LVL



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

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	2630 @ 3 1/2"	3938 (1.50")	Passed (67%)	--	1.0 D + 1.0 L (All Spans)
Shear (lbs)	1966 @ 1' 5 1/2"	9310	Passed (21%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	6081 @ 4' 11"	24258	Passed (25%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.049 @ 4' 11"	0.231	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.073 @ 4' 11"	0.463	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)

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

System : Floor
Member Type : Flush Beam
Building Use : Residential
Building Code : IBC 2018
Design Methodology : ASD

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Floor Live	Factored	
1 - Hanger on 14" HF beam	3.50"	Hanger[1]	1.50"	922	1870	2791	See note [1]
2 - Hanger on 14" HF beam	3.50"	Hanger[1]	1.50"	922	1870	2791	See note [1]

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

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

•Maximum allowable bracing intervals based on applied load.

Connector: Simpson Strong-Tie						
Support	Model	Seat Length	Top Fasteners	Face Fasteners	Member Fasteners	Accessories
1 - Face Mount Hanger	HHUS410	3.00"	N/A	30-10d	10-10d	
2 - Face Mount Hanger	HHUS410	3.00"	N/A	30-10d	10-10d	

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

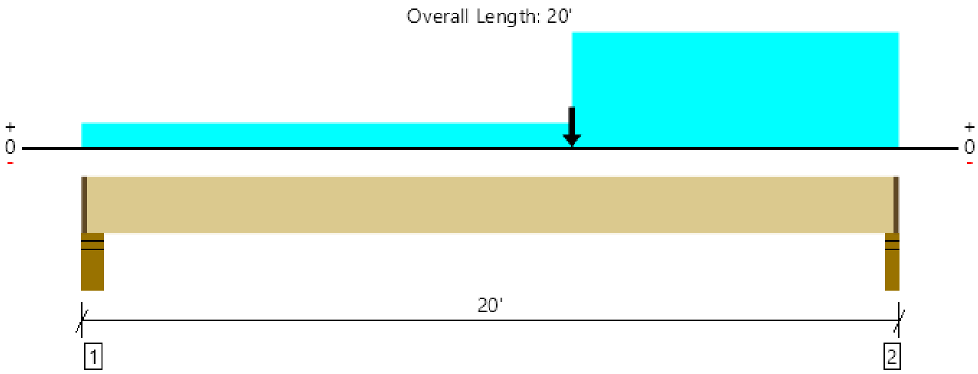
Vertical Loads	Location (Side)	Tributary Width	Dead (0.90)	Floor Live (1.00)	Comments
0 - Self Weight (PLF)	3 1/2" to 9' 6 1/2"	N/A	14.3	--	
1 - Uniform (PLF)	0 to 9' 10" (Front)	N/A	174.0	380.3	Linked from: Floor Joist #2 - 15'-6" span, Support 2

Member Notes
Stair Framing Beam

ForteWEB Software Operator	Job Notes
Samantha Taylor Anthem Structural Engineers (303) 848-8497 staylor@anthemstructural.com	

4 bed unit typ joist, Floor Beam #8

3 piece(s) 1 3/4" x 14" 2.0E Microllam® LVL



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

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	4350 @ 19' 10 1/4"	4253 (2.00")	Passed (102%)	--	1.0 D + 1.0 L (All Spans)
Shear (lbs)	3855 @ 18' 6 3/4"	13965	Passed (28%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	22586 @ 12'	36387	Passed (62%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.385 @ 10' 7 13/16"	0.488	Passed (L/608)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.590 @ 10' 7 7/16"	0.976	Passed (L/397)	--	1.0 D + 1.0 L (All Spans)

System : Floor
Member Type : Flush Beam
Building Use : Residential
Building Code : IBC 2018
Design Methodology : ASD

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

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Floor Live	Factored	
1 - Stud wall - HF	5.50"	4.25"	1.50"	925	1602	2526	1 1/4" Rim Board
2 - Stud wall - HF	3.25"	2.00"	2.05"	1478	2909	4386	1 1/4" Rim Board

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

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	14' 1" o/c	
Bottom Edge (Lu)	19' 10" o/c	

•Maximum allowable bracing intervals based on applied load.

Vertical Loads	Location (Side)	Tributary Width	Dead (0.90)	Floor Live (1.00)	Comments
0 - Self Weight (PLF)	1 1/4" to 19' 10 3/4"	N/A	21.5	--	
1 - Uniform (PSF)	0 to 20' (Top)	1' 4"	16.0	40.0	Floor Load
2 - Uniform (PLF)	12' to 20' (Front)	N/A	78.7	196.7	Stair Load
3 - Point (lb)	12' (Front)	N/A	922	1870	Linked from: Floor Beam #7, Support 2

Member Notes

Stair Framing Beam

Weyerhaeuser Notes

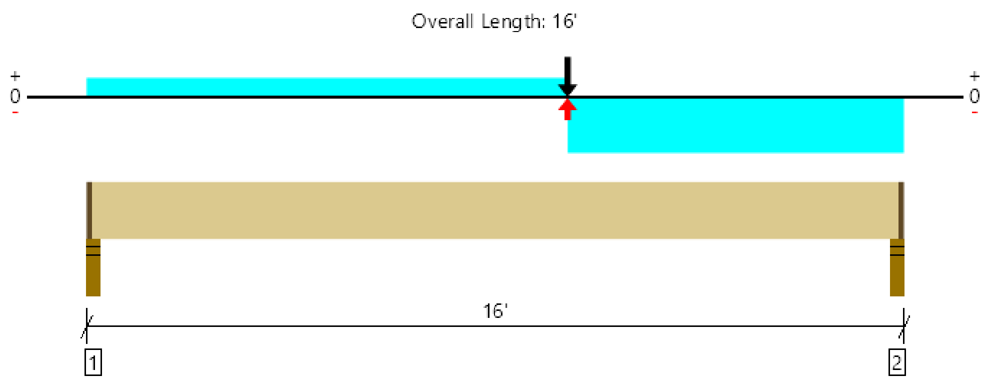
The product application, input design loads, dimensions and support information have been provided by ForteWEB Software Operator

ForteWEB Software Operator	Job Notes
Samantha Taylor Anthem Structural Engineers (303) 848-8497 staylor@anthemstructural.com	

Weyerhaeuser

4 bed unit typ joist, Floor Beam #9

2 piece(s) 1 3/4" x 14" 2.0E Microllam® LVL



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

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern) [Group]
Member Reaction (lbs)	2349 @ 15' 10 1/4"	2835 (2.00")	Passed (83%)	--	1.0 D + 1.0 L (All Spans) [1]
Shear (lbs)	1999 @ 14' 6 3/4"	9310	Passed (21%)	1.00	1.0 D + 1.0 L (All Spans) [1]
Moment (Ft-lbs)	9602 @ 9' 5"	24258	Passed (40%)	1.00	1.0 D + 1.0 L (All Spans) [1]
Live Load Defl. (in)	0.174 @ 8' 5 1/2"	0.393	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans) [1]
Total Load Defl. (in)	0.250 @ 8' 5 1/8"	0.785	Passed (L/753)	--	1.0 D + 1.0 L (All Spans) [1]

System : Floor
Member Type : Flush Beam
Building Use : Residential
Building Code : IBC 2018
Design Methodology : ASD

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

Supports	Bearing Length			Loads to Supports (lbs)				Accessories
	Total	Available	Required	Dead	Floor Live	Snow	Factored	
1 - Stud wall - HF	3.25"	2.00"	1.50"	429	852	-293	1281	1 1/4" Rim Board
2 - Stud wall - HF	3.25"	2.00"	1.66"	689	1686/-149	-843	2375/-153	1 1/4" Rim Board

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

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	15' 10" o/c	
Bottom Edge (Lu)	15' 10" o/c	

•Maximum allowable bracing intervals based on applied load.

Vertical Loads	Location (Side)	Tributary Width	Dead (0.90)	Floor Live (1.00)	Snow (1.15)	Comments
0 - Self Weight (PLF)	1 1/4" to 15' 10 3/4"	N/A	14.3	--	--	
1 - Uniform (PSF)	0 to 16' (Top)	8"	16.0	40.0	-	Floor Load
2 - Uniform (PLF)	9' 5" to 16' (Front)	N/A	47.3	164.3/-63.8	-125.3	Linked from: Floor Joist #3 - 12'-6" span, Support 2
3 - Point (lb)	9' 5" (Front)	N/A	411	1030/-41	-311	Linked from: Floor Beam #10, Support 2

Member Notes

Stair Framing Beam

Weyerhaeuser Notes

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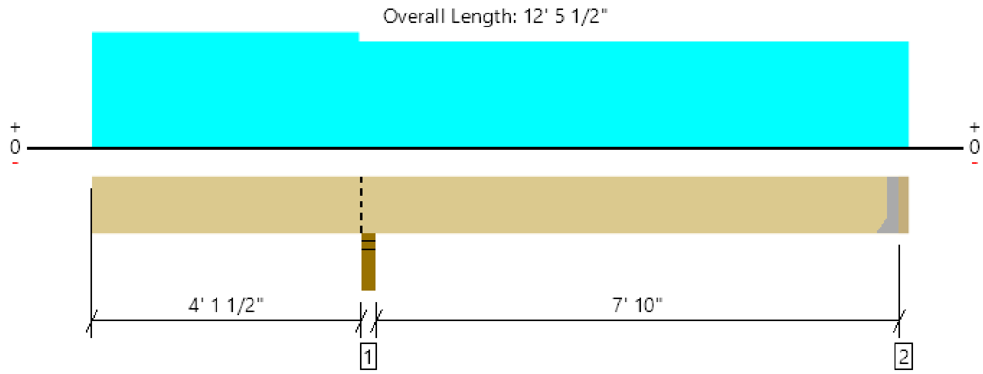
The product application, input design loads, dimensions and support information have been provided by ForteWEB Software Operator

MEMBER REPORT

PASSED

4 bed unit typ joist, Floor Beam #10

2 piece(s) 2 x 12 HF No.2



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

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	1369 @ 12' 3"	1823 (1.50")	Passed (75%)	--	1.0 D + 1.0 L (Alt Spans)
Shear (lbs)	1148 @ 5' 4 1/4"	3375	Passed (34%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	2664 @ 8' 4 5/16"	4482	Passed (59%)	1.00	1.0 D + 1.0 L (Alt Spans)
Live Load Defl. (in)	0.147 @ 0	0.214	Passed (2L/696)	--	1.0 D + 1.0 S (All Spans)
Total Load Defl. (in)	0.128 @ 0	0.427	Passed (2L/802)	--	1.0 D + 1.0 S (All Spans)

System : Floor
Member Type : Flush Beam
Building Use : Residential
Building Code : IBC 2018
Design Methodology : ASD

- Deflection criteria: LL (L/480) and TL (L/240).
- Overhang deflection criteria: LL (2L/480) and TL (2L/240).
- Left cantilever length exceeds 1/3 member length or 1/2 back span length. Additional bracing should be considered.
- Allowed moment does not reflect the adjustment for the beam stability factor.
- Applicable calculations are based on NDS.

Supports	Bearing Length			Loads to Supports (lbs)				Accessories
	Total	Available	Required	Dead	Floor Live	Snow	Factored	
1 - Stud wall - HF	3.50"	3.50"	2.26"	597	1436	1433	2750	Blocking
2 - Hanger on 11 1/4" HF beam	2.50"	Hanger[1]	1.50"	411	1030/-41	-311	1440	See note [1]

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

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	12' 3" o/c	
Bottom Edge (Lu)	12' 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	LUS210-2	2.00"	N/A	8-16d	6-16d	

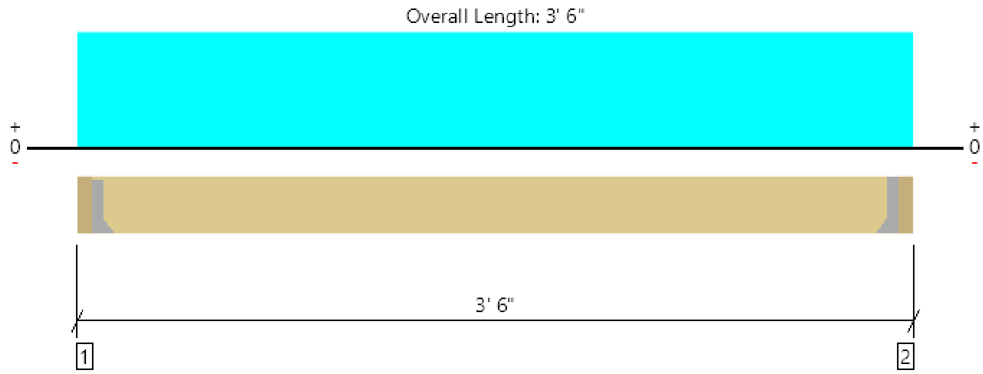
- Refer to manufacturer notes and instructions for proper installation and use of all connectors.

Vertical Loads	Location (Side)	Tributary Width	Dead (0.90)	Floor Live (1.00)	Snow (1.15)	Wind (1.60)	Comments
0 - Self Weight (PLF)	0 to 12' 3"	N/A	8.6	--	--	--	
1 - Uniform (PSF)	0 to 12' 5 1/2" (Top)	1' 4"	16.0	40.0	-	-	Floor Load
2 - Uniform (PSF)	0 to 4' 1 1/2" (Top)	1' 4"	-	20.0	204.1	21.9	Balcony Load
3 - Uniform (PSF)	0 to 4' 1 1/2" (Top)	1' 4"	-	-	-	-21.9	Balcony Wind Uplift
4 - Uniform (PLF)	4' 1 1/2" to 12' 5 1/2" (Front)	N/A	76.5	192.0	-	-	Linked from: Floor Joist #5 - 9.5' span, Support 2

Member Notes
Stair Framing Beam

ForteWEB Software Operator	Job Notes
Samantha Taylor Anthem Structural Engineers (303) 848-8497 staylor@anthemstructural.com	

4 bed unit typ joist, Floor Beam #11
2 piece(s) 1 3/4" x 14" 2.0E Microllam® LVL



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

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	490 @ 3 1/2"	3938 (1.50")	Passed (12%)	--	1.0 D + 1.0 L (All Spans)
Shear (lbs)	98 @ 1' 5 1/2"	9310	Passed (1%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	358 @ 1' 9"	24258	Passed (1%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.001 @ 1' 9"	0.073	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.001 @ 1' 9"	0.146	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)

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

System : Floor
Member Type : Flush Beam
Building Use : Residential
Building Code : IBC 2018
Design Methodology : ASD

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Floor Live	Factored	
1 - Hanger on 14" HF beam	3.50"	Hanger[1]	1.50"	182	403	584	See note [1]
2 - Hanger on 14" HF beam	3.50"	Hanger[1]	1.50"	182	403	584	See note [1]

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

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

•Maximum allowable bracing intervals based on applied load.

Connector: Simpson Strong-Tie						
Support	Model	Seat Length	Top Fasteners	Face Fasteners	Member Fasteners	Accessories
1 - Face Mount Hanger	LUS410	2.00"	N/A	8-10dx1.5	6-10d	
2 - Face Mount Hanger	LUS410	2.00"	N/A	8-10dx1.5	6-10d	

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

Vertical Loads	Location (Side)	Tributary Width	Dead (0.90)	Floor Live (1.00)	Comments
0 - Self Weight (PLF)	3 1/2" to 3' 2 1/2"	N/A	14.3	--	
1 - Uniform (PSF)	0 to 3' 6" (Top)	5' 9"	16.0	40.0	Floor and Stair Load

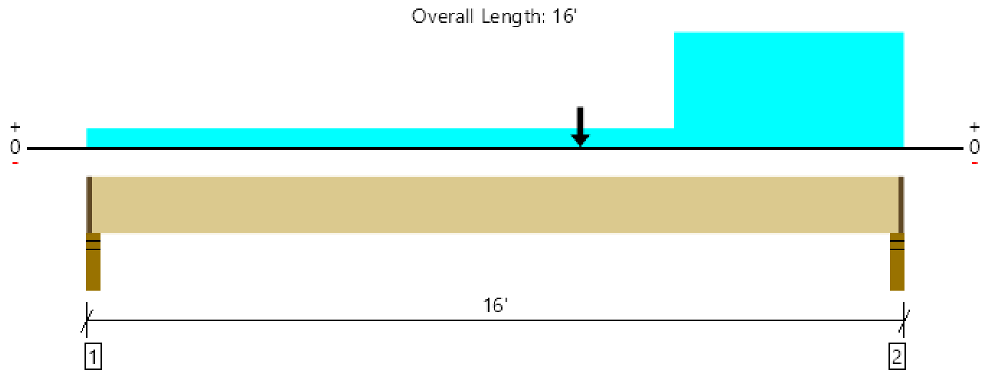
Weyerhaeuser Notes

ForteWEB Software Operator	Job Notes
Samantha Taylor Anthem Structural Engineers (303) 848-8497 staylor@anthemstructural.com	

MEMBER REPORT

PASSED

4 bed unit typ joist, Floor Beam #12
2 piece(s) 1 3/4" x 14" 2.0E Microllam® LVL



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

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	2494 @ 15' 10 1/4"	2835 (2.00")	Passed (88%)	--	1.0 D + 1.0 L (All Spans)
Shear (lbs)	1870 @ 14' 6 3/4"	9310	Passed (20%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	6992 @ 9' 8"	24258	Passed (29%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.130 @ 8' 5 9/16"	0.393	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.197 @ 8' 5 3/16"	0.785	Passed (L/958)	--	1.0 D + 1.0 L (All Spans)

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

System : Floor
Member Type : Flush Beam
Building Use : Residential
Building Code : IBC 2018
Design Methodology : ASD

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Floor Live	Factored	
1 - Stud wall - HF	3.25"	2.00"	1.50"	420	749	1169	1 1/4" Rim Board
2 - Stud wall - HF	3.25"	2.00"	1.76"	815	1727	2542	1 1/4" Rim Board

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

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	15' 10" o/c	
Bottom Edge (Lu)	15' 10" o/c	

•Maximum allowable bracing intervals based on applied load.

Vertical Loads	Location (Side)	Tributary Width	Dead (0.90)	Floor Live (1.00)	Comments
0 - Self Weight (PLF)	1 1/4" to 15' 10 3/4"	N/A	14.3	--	
1 - Uniform (PSF)	0 to 16' (Top)	1' 4"	16.0	40.0	Floor Load
2 - Uniform (PLF)	11' 6" to 16' (Back)	N/A	108.0	271.0	Stair Load
3 - Point (lb)	9' 8" (Front)	N/A	182	403	Linked from: Floor Beam #11, Support 1

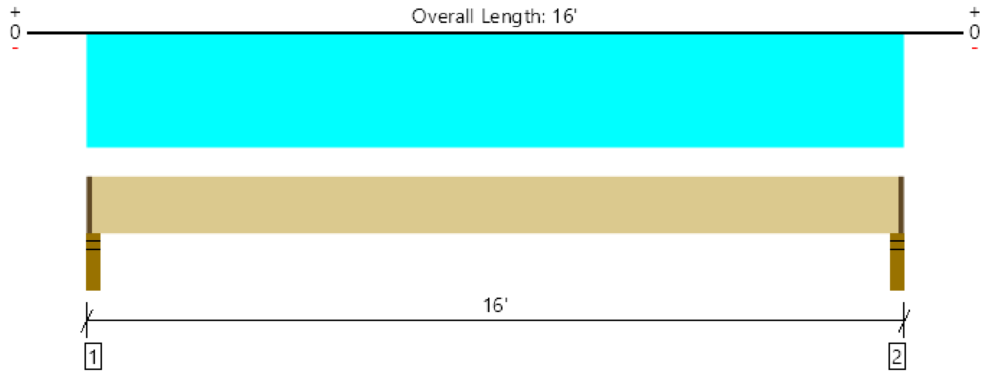
Weyerhaeuser Notes

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

ForteWEB Software Operator	Job Notes
Samantha Taylor Anthem Structural Engineers (303) 848-8497 staylor@anthemstructural.com	

4 bed unit typ joist, Floor Beam #13

2 piece(s) 1 3/4" x 14" 2.0E Microllam® LVL



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

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern) [Group]
Member Reaction (lbs)	2078 @ 1 3/4"	2835 (2.00")	Passed (73%)	--	1.0 D + 1.0 L (All Spans) [1]
Shear (lbs)	1727 @ 1' 5 1/4"	9310	Passed (19%)	1.00	1.0 D + 1.0 L (All Spans) [1]
Moment (Ft-lbs)	8116 @ 8'	24258	Passed (33%)	1.00	1.0 D + 1.0 L (All Spans) [1]
Live Load Defl. (in)	0.177 @ 8'	0.524	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans) [1]
Total Load Defl. (in)	0.244 @ 8'	0.785	Passed (L/772)	--	1.0 D + 1.0 L (All Spans) [1]

System : Floor
Member Type : Flush Beam
Building Use : Residential
Building Code : IBC 2018
Design Methodology : ASD

- Deflection criteria: LL (L/360) and TL (L/240).
- Allowed moment does not reflect the adjustment for the beam stability factor.
- -426 lbs uplift at support located at 1 3/4". Strapping or other restraint may be required.
- -426 lbs uplift at support located at 15' 10 1/4". Strapping or other restraint may be required.

Supports	Bearing Length			Loads to Supports (lbs)				Accessories
	Total	Available	Required	Dead	Floor Live	Snow	Factored	
1 - Stud wall - HF	3.25"	2.00"	1.50"	576	1527/-299	-1002	2104/-426	1 1/4" Rim Board
2 - Stud wall - HF	3.25"	2.00"	1.50"	576	1527/-299	-1002	2104/-426	1 1/4" Rim Board

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

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	15' 10" o/c	
Bottom Edge (Lu)	15' 10" o/c	

- Maximum allowable bracing intervals based on applied load.

Vertical Loads	Location (Side)	Tributary Width	Dead (0.90)	Floor Live (1.00)	Snow (1.15)	Comments
0 - Self Weight (PLF)	1 1/4" to 15' 10 3/4"	N/A	14.3	--	--	
1 - Uniform (PSF)	0 to 16' (Top)	8"	16.0	40.0	-	Floor Load
2 - Uniform (PLF)	0 to 16' (Front)	N/A	47.3	164.3/-63.8	-125.3	Linked from: Floor Joist #3 - 12'-6" span, Support 2

Weyerhaeuser Notes

ForteWEB Software Operator	Job Notes
Samantha Taylor Anthem Structural Engineers (303) 848-8497 staylor@anthemstructural.com	

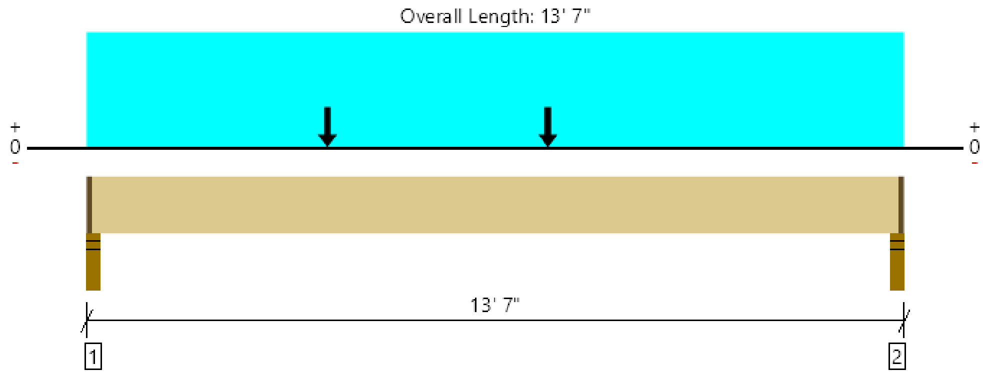
Weyerhaeuser

MEMBER REPORT

PASSED

4 bed unit typ joist, Floor Beam #14

2 piece(s) 1 3/4" x 14" 2.0E Microllam® LVL



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

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	2651 @ 1 3/4"	2835 (2.00")	Passed (94%)	--	1.0 D + 1.0 L (All Spans)
Shear (lbs)	2204 @ 1' 5 1/4"	9310	Passed (24%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	9126 @ 6' 11 5/16"	24258	Passed (38%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.138 @ 6' 9 3/16"	0.443	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.201 @ 6' 9 3/16"	0.665	Passed (L/792)	--	1.0 D + 1.0 L (All Spans)

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

System : Floor
Member Type : Flush Beam
Building Use : Residential
Building Code : IBC 2018
Design Methodology : ASD

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Floor Live	Factored	
1 - Stud wall - HF	3.25"	2.00"	1.87"	843	1841	2684	1 1/4" Rim Board
2 - Stud wall - HF	3.25"	2.00"	1.80"	811	1770	2581	1 1/4" Rim Board

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

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	13' 5" o/c	
Bottom Edge (Lu)	13' 5" o/c	

•Maximum allowable bracing intervals based on applied load.

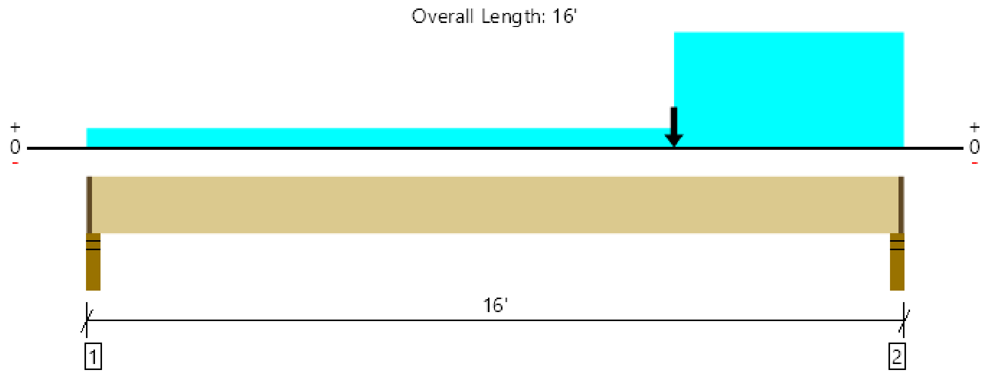
Vertical Loads	Location (Side)	Tributary Width	Dead (0.90)	Floor Live (1.00)	Comments
0 - Self Weight (PLF)	1 1/4" to 13' 5 3/4"	N/A	14.3	--	
1 - Uniform (PLF)	0 to 13' 7" (Front)	N/A	91.5	229.5	Linked from: Floor Joist #6 - 11'-6" span, Support 1
2 - Point (lb)	4' (Front)	N/A	110	247	Linked from: TRANSFER BM @ STAIR POST, Support 2
3 - Point (lb)	7' 8" (Front)	N/A	110	247	Linked from: TRANSFER BM @ STAIR POST, Support 2

Weyerhaeuser Notes

ForteWEB Software Operator	Job Notes
Samantha Taylor Anthem Structural Engineers (303) 848-8497 staylor@anthemstructural.com	

4 bed unit typ joist, Floor Beam #15

3 piece(s) 1 3/4" x 14" 2.0E Microllam® LVL



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

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	4062 @ 15' 10 1/4"	4253 (2.00")	Passed (96%)	--	1.0 D + 1.0 L (All Spans)
Shear (lbs)	3428 @ 14' 6 3/4"	13965	Passed (25%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	13095 @ 11' 6"	36387	Passed (36%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.147 @ 8' 8 3/16"	0.393	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.224 @ 8' 7 3/4"	0.785	Passed (L/843)	--	1.0 D + 1.0 L (All Spans)

System : Floor
Member Type : Flush Beam
Building Use : Residential
Building Code : IBC 2018
Design Methodology : ASD

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

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Floor Live	Factored	
1 - Stud wall - HF	3.25"	2.00"	1.50"	630	1081	1711	1 1/4" Rim Board
2 - Stud wall - HF	3.25"	2.00"	1.91"	1347	2762	4109	1 1/4" Rim Board

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

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	15' 10" o/c	
Bottom Edge (Lu)	15' 10" o/c	

•Maximum allowable bracing intervals based on applied load.

Vertical Loads	Location (Side)	Tributary Width	Dead (0.90)	Floor Live (1.00)	Comments
0 - Self Weight (PLF)	1 1/4" to 15' 10 3/4"	N/A	21.5	--	
1 - Uniform (PSF)	0 to 16' (Top)	1' 4"	16.0	40.0	Floor Load
2 - Uniform (PLF)	11' 6" to 16' (Back)	N/A	108.0	271.0	Stair Load
3 - Point (lb)	11' 6" (Front)	N/A	811	1770	Linked from: Floor Beam #14, Support 2

Weyerhaeuser Notes

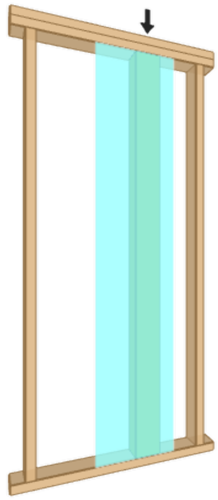
ForteWEB Software Operator	Job Notes
Samantha Taylor Anthem Structural Engineers (303) 848-8497 staylor@anthemstructural.com	

MEMBER REPORT

PASSED

4 bed unit typ joist, Post #1
3 piece(s) 2 x 6 HF No.2

Wall Height: 10' 3 7/8" Member Height: 9' 11 3/8" Tributary Width: 1' 4"



Drawing is Conceptual

Design Results	Actual	Allowed	Result	LDF	Load: Combination
Slenderness	22	50	Passed (43%)	--	--
Compression (lbs)	9214	17100	Passed (54%)	1.00	1.0 D + 1.0 L
Plate Bearing (lbs)	9214	10024	Passed (92%)	--	1.0 D + 1.0 L
Lateral Reaction (lbs)	98	--	--	1.60	1.0 D + 0.6 W
Lateral Shear (lbs)	89	3960	Passed (2%)	1.60	1.0 D + 0.6 W
Lateral Moment (ft-lbs)	244 @ mid-span	3339	Passed (7%)	1.60	1.0 D + 0.6 W
Total Deflection (in)	0.12 @ mid-span	0.50	Passed (L/1017)	--	1.0 D + 0.45 W + 0.75 L + 0.75 S
Bending/Compression	0.97	1	Passed (97%)	1.00	1.0 D + 1.0 L

- Lateral deflection criteria: Wind (L/240)
- Input axial load eccentricity for this design is 16.67% of applicable member side dimension.
- Applicable calculations are based on NDS.
- Bearing shall be on a metal plate or strap, or on other equivalently durable, rigid, homogeneous material with sufficient stiffness to distribute applied load.
- The column stability factor (Kf = 0.6) applied to this design assumes nailed built-up columns per NDS section 15.3.3. For Weyerhaeuser ELP products refer to the U.S. Wall Guide for multiple-member connection requirements.

Supports	Type	Material
Top	Dbl 2X	Hem Fir
Base	2X	Hem Fir

System : Wall
Member Type : Column
Building Code : IBC 2018
Design Methodology : ASD

Max Unbraced Length	Comments
1'	

Lateral Connections

Supports	Connector	Type/Model	Quantity	Connector Nailing
Top	Nails	8d (0.113" x 2 1/2") (Toe)	2	N/A
Base	Nails	8d (0.113" x 2 1/2") (Toe)	2	N/A

- Nailed connection at the top of the member is assumed to be nailed through the bottom 2x plate prior to placement of the top 2x of the double top plate assembly.

Vertical Loads	Tributary Width	Dead (0.90)	Floor Live (1.00)	Snow (1.15)	Comments
1 - Point (lb)	N/A	10	-	-	Wall Dead Load
2 - Point (lb)	N/A	1478	2909	-	Linked from: Floor Beam #8, Support 2
3 - Point (lb)	N/A	3235	1582	730	Linked from: Copy of Floor Beam #8 - L4 ROOF TRANSFER, Support 1

Lateral Load	Location	Tributary Width	Wind (1.60)	Comments
1 - Uniform (PSF)	Full Length	1' 4"	24.6	

- ASCE/SEI 7 Sec. 30.4: Exposure Category (B), Mean Roof Height (33'), Topographic Factor (1.0), Wind Directionality Factor (0.85), Basic Wind Speed (115), Risk Category(II), Effective Wind Area determined using full member span and trib. width.
- IBC Table 1604.3, footnote f: Deflection checks are performed using 42% of this lateral wind load.

Weyerhaeuser Notes

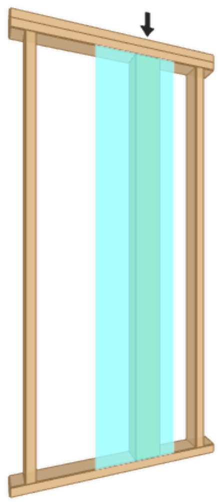
ForteWEB Software Operator	Job Notes
Samantha Taylor Anthem Structural Engineers (303) 848-8497 staylor@anthemstructural.com	

MEMBER REPORT

PASSED

4 bed unit typ joist, Post #2
4 piece(s) 2 x 6 HF No.2

Wall Height: 10' 3 7/8" Member Height: 9' 11 3/8" Tributary Width: 1' 4"



Drawing is Conceptual

Design Results	Actual	Allowed	Result	LDF	Load: Combination
Slenderness	22	50	Passed (43%)	--	--
Compression (lbs)	11075	22800	Passed (49%)	1.00	1.0 D + 1.0 L
Plate Bearing (lbs)	11075	13365	Passed (83%)	--	1.0 D + 1.0 L
Lateral Reaction (lbs)	98	--	--	1.60	1.0 D + 0.6 W
Lateral Shear (lbs)	89	5280	Passed (2%)	1.60	1.0 D + 0.6 W
Lateral Moment (ft-lbs)	244 @ mid-span	4454	Passed (5%)	1.60	1.0 D + 0.6 W
Total Deflection (in)	0.09 @ mid-span	0.50	Passed (L/1287)	--	1.0 D + 0.45 W + 0.75 L + 0.75 Lr
Bending/Compression	0.80	1	Passed (80%)	1.00	1.0 D + 1.0 L

- Lateral deflection criteria: Wind (L/240)
- Input axial load eccentricity for this design is 16.67% of applicable member side dimension.
- Applicable calculations are based on NDS.
- Bearing shall be on a metal plate or strap, or on other equivalently durable, rigid, homogeneous material with sufficient stiffness to distribute applied load.
- The column stability factor (Kf = 0.6) applied to this design assumes nailed built-up columns per NDS section 15.3.3. For Weyerhaeuser ELP products refer to the U.S. Wall Guide for multiple-member connection requirements.

Supports	Type	Material
Top	Dbl 2X	Hem Fir
Base	2X	Hem Fir

System : Wall
Member Type : Column
Building Code : IBC 2018
Design Methodology : ASD

Max Unbraced Length	Comments
1'	

Lateral Connections				
Supports	Connector	Type/Model	Quantity	Connector Nailing
Top	Nails	8d (0.113" x 2 1/2") (Toe)	2	N/A
Base	Nails	8d (0.113" x 2 1/2") (Toe)	2	N/A

- Nailed connection at the top of the member is assumed to be nailed through the bottom 2x plate prior to placement of the top 2x of the double top plate assembly.

Vertical Loads	Tributary Width	Dead (0.90)	Floor Live (1.00)	Comments
1 - Point (lb)	N/A	10	-	Wall Dead Load
2 - Point (lb)	N/A	806	1485	Linked from: Floor Beam #2, Support 2
3 - Point (lb)	N/A	1478	2909	Linked from: Floor Beam #8, Support 2
4 - Point (lb)	N/A	1478	2909	Linked from: Floor Beam #8, Support 2

Lateral Load	Location	Tributary Width	Wind (1.60)	Comments
1 - Uniform (PSF)	Full Length	1' 4"	24.6	

- ASCE/SEI 7 Sec. 30.4: Exposure Category (B), Mean Roof Height (33'), Topographic Factor (1.0), Wind Directionality Factor (0.85), Basic Wind Speed (115), Risk Category(II), Effective Wind Area determined using full member span and trib. width.
- IBC Table 1604.3, footnote f: Deflection checks are performed using 42% of this lateral wind load.

Member Notes
Stud pack at Beam #1

Weyerhaeuser Notes

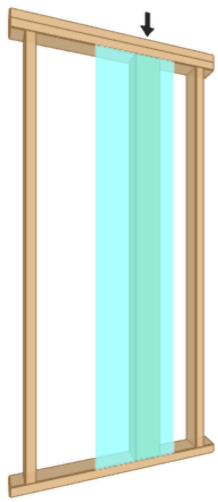
ForteWEB Software Operator	Job Notes
Samantha Taylor Anthem Structural Engineers (303) 848-8497 staylor@anthemstructural.com	

MEMBER REPORT

PASSED

4 bed unit typ joist, Post #3
3 piece(s) 2 x 6 HF No.2

Wall Height: 10' 3 7/8"   Member Height: 9' 11 3/8"   Tributary Width: 1' 4"



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Design Results	Actual	Allowed	Result	LDF	Load: Combination
Slenderness	22	50	Passed (43%)	--	--
Compression (lbs)	3628	17100	Passed (21%)	1.00	1.0 D + 1.0 L
Plate Bearing (lbs)	3628	10024	Passed (36%)	--	1.0 D + 1.0 L
Lateral Reaction (lbs)	98	--	--	1.60	1.0 D + 0.6 W
Lateral Shear (lbs)	89	3960	Passed (2%)	1.60	1.0 D + 0.6 W
Lateral Moment (ft-lbs)	244 @ mid-span	3339	Passed (7%)	1.60	1.0 D + 0.6 W
Total Deflection (in)	0.06 @ mid-span	0.50	Passed (L/2003)	--	1.0 D + 0.45 W + 0.75 L + 0.75 Lr
Bending/Compression	0.21	1	Passed (21%)	1.00	1.0 D + 1.0 L

- Lateral deflection criteria: Wind (L/240)
- Input axial load eccentricity for this design is 16.67% of applicable member side dimension.
- Applicable calculations are based on NDS.
- The column stability factor (Kf = 0.6) applied to this design assumes nailed built-up columns per NDS section 15.3.3. For Weyerhaeuser ELP products refer to the U.S. Wall Guide for multiple-member connection requirements.

Supports	Type	Material
Top	Dbl 2X	Hem Fir
Base	2X	Hem Fir

System : Wall
Member Type : Column
Building Code : IBC 2018
Design Methodology : ASD

Max Unbraced Length	Comments
1'	

Lateral Connections

Supports	Connector	Type/Model	Quantity	Connector Nailing
Top	Nails	8d (0.113" x 2 1/2") (Toe)	2	N/A
Base	Nails	8d (0.113" x 2 1/2") (Toe)	2	N/A

- Nailed connection at the top of the member is assumed to be nailed through the bottom 2x plate prior to placement of the top 2x of the double top plate assembly.

Vertical Loads	Tributary Width	Dead (0.90)	Floor Live (1.00)	Comments
1 - Point (lb)	N/A	10	-	Wall Dead Load
2 - Point (lb)	N/A	1304	2314	Linked from: Floor Beam #1, Support 2

Lateral Load	Location	Tributary Width	Wind (1.60)	Comments
1 - Uniform (PSF)	Full Length	1' 4"	24.6	

- ASCE/SEI 7 Sec. 30.4: Exposure Category (B), Mean Roof Height (33'), Topographic Factor (1.0), Wind Directionality Factor (0.85), Basic Wind Speed (115), Risk Category(II), Effective Wind Area determined using full member span and trib. width.
- IBC Table 1604.3, footnote f: Deflection checks are performed using 42% of this lateral wind load.

Member Notes

Stud pack at Beam #1

Weyerhaeuser Notes

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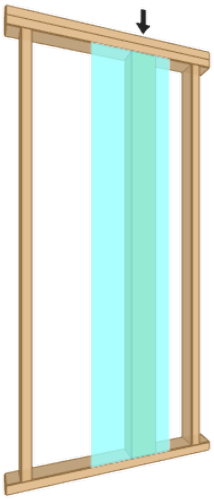
The product application, input design loads, dimensions and support information have been provided by ForteWEB Software Operator

ForteWEB Software Operator	Job Notes
Samantha Taylor Anthem Structural Engineers (303) 848-8497 staylor@anthemstructural.com	

Weyerhaeuser

4 bed unit typ joist, Post #4
3 piece(s) 2 x 4 HF No.2

Wall Height: 10' 3 7/8" | Member Height: 9' 11 3/8" | Tributary Width: 1' 4"



Drawing is Conceptual

Design Results	Actual	Allowed	Result	LDF	Load: Combination
Slenderness	34	50	Passed (68%)	--	--
Compression (lbs)	2552	4965	Passed (51%)	1.00	1.0 D + 1.0 L
Plate Bearing (lbs)	2552	6379	Passed (40%)	--	1.0 D + 1.0 L
Lateral Reaction (lbs)	98	--	--	1.60	1.0 D + 0.6 W
Lateral Shear (lbs)	92	2520	Passed (4%)	1.60	1.0 D + 0.6 W
Lateral Moment (ft-lbs)	244 @ mid-span	1561	Passed (16%)	1.60	1.0 D + 0.6 W
Total Deflection (in)	0.17 @ mid-span	0.50	Passed (L/717)	--	1.0 D + 0.6 W
Bending/Compression	0.54	1	Passed (54%)	1.00	1.0 D + 1.0 L

- Lateral deflection criteria: Wind (L/240)
- Input axial load eccentricity for this design is 16.67% of applicable member side dimension.
- Applicable calculations are based on NDS.
- The column stability factor (Kf = 0.6) applied to this design assumes nailed built-up columns per NDS section 15.3.3. For Weyerhaeuser ELP products refer to the U.S. Wall Guide for multiple-member connection requirements.

Supports	Type	Material
Top	Dbl 2X	Hem Fir
Base	2X	Hem Fir

System : Wall
Member Type : Column
Building Code : IBC 2018
Design Methodology : ASD

Max Unbraced Length	Comments
1'	

Lateral Connections

Supports	Connector	Type/Model	Quantity	Connector Nailing
Top	Nails	8d (0.113" x 2 1/2") (Toe)	2	N/A
Base	Nails	8d (0.113" x 2 1/2") (Toe)	2	N/A

- Nailed connection at the top of the member is assumed to be nailed through the bottom 2x plate prior to placement of the top 2x of the double top plate assembly.

Vertical Loads	Tributary Width	Dead (0.90)	Floor Live (1.00)	Comments
1 - Point (lb)	N/A	10	-	Wall Dead Load
2 - Point (lb)	N/A	815	1727	Linked from: Floor Beam #12, Support 2

Lateral Load	Location	Tributary Width	Wind (1.60)	Comments
1 - Uniform (PSF)	Full Length	1' 4"	24.6	

• ASCE/SEI 7 Sec. 30.4: Exposure Category (B), Mean Roof Height (33'), Topographic Factor (1.0), Wind Directionality Factor (0.85), Basic Wind Speed (115), Risk Category(II), Effective Wind Area determined using full member span and trib. width.
• IBC Table 1604.3, footnote f: Deflection checks are performed using 42% of this lateral wind load.

Member Notes

Stud pack at Beam #1

Weyerhaeuser Notes

ForteWEB Software Operator	Job Notes
Samantha Taylor Anthem Structural Engineers (303) 848-8497 staylor@anthemstructural.com	

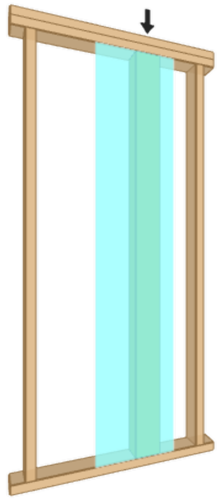
FORTEWEB

MEMBER REPORT

PASSED

4 bed unit typ joist, Post #5
4 piece(s) 2 x 6 HF No.2

Wall Height: 10' 3 7/8" Member Height: 9' 11 3/8" Tributary Width: 1' 4"



Drawing is Conceptual

Design Results	Actual	Allowed	Result	LDF	Load: Combination
Slenderness	22	50	Passed (43%)	--	--
Compression (lbs)	11650	22800	Passed (51%)	1.00	1.0 D + 1.0 L
Plate Bearing (lbs)	11650	13365	Passed (87%)	--	1.0 D + 1.0 L
Lateral Reaction (lbs)	98	--	--	1.60	1.0 D + 0.6 W
Lateral Shear (lbs)	89	5280	Passed (2%)	1.60	1.0 D + 0.6 W
Lateral Moment (ft-lbs)	244 @ mid-span	4454	Passed (5%)	1.60	1.0 D + 0.6 W
Total Deflection (in)	0.10 @ mid-span	0.50	Passed (L/1238)	--	1.0 D + 0.45 W + 0.75 L + 0.75 Lr
Bending/Compression	0.88	1	Passed (88%)	1.00	1.0 D + 1.0 L

- Lateral deflection criteria: Wind (L/240)
- Input axial load eccentricity for this design is 16.67% of applicable member side dimension.
- Applicable calculations are based on NDS.
- Bearing shall be on a metal plate or strap, or on other equivalently durable, rigid, homogeneous material with sufficient stiffness to distribute applied load.
- The column stability factor (Kf = 0.6) applied to this design assumes nailed built-up columns per NDS section 15.3.3. For Weyerhaeuser ELP products refer to the U.S. Wall Guide for multiple-member connection requirements.

Supports	Type	Material
Top	Dbl 2X	Hem Fir
Base	2X	Hem Fir

System : Wall
Member Type : Column
Building Code : IBC 2018
Design Methodology : ASD

Max Unbraced Length	Comments
1'	

Lateral Connections				
Supports	Connector	Type/Model	Quantity	Connector Nailing
Top	Nails	8d (0.113" x 2 1/2") (Toe)	2	N/A
Base	Nails	8d (0.113" x 2 1/2") (Toe)	2	N/A

- Nailed connection at the top of the member is assumed to be nailed through the bottom 2x plate prior to placement of the top 2x of the double top plate assembly.

Vertical Loads	Tributary Width	Dead (0.90)	Floor Live (1.00)	Comments
1 - Point (lb)	N/A	10	-	Wall Dead Load
2 - Point (lb)	N/A	1347	2762	Linked from: Floor Beam #15, Support 2
3 - Point (lb)	N/A	1347	2762	Linked from: Floor Beam #15, Support 2
4 - Point (lb)	N/A	630	1081	Linked from: Floor Beam #15, Support 1
5 - Point (lb)	N/A	630	1081	Linked from: Floor Beam #15, Support 1

Lateral Load	Location	Tributary Width	Wind (1.60)	Comments
1 - Uniform (PSF)	Full Length	1' 4"	24.6	

- ASCE/SEI 7 Sec. 30.4: Exposure Category (B), Mean Roof Height (33'), Topographic Factor (1.0), Wind Directionality Factor (0.85), Basic Wind Speed (115), Risk Category(II), Effective Wind Area determined using full member span and trib. width.
- IBC Table 1604.3, footnote f: Deflection checks are performed using 42% of this lateral wind load.

Member Notes

Stud pack at Beam #1

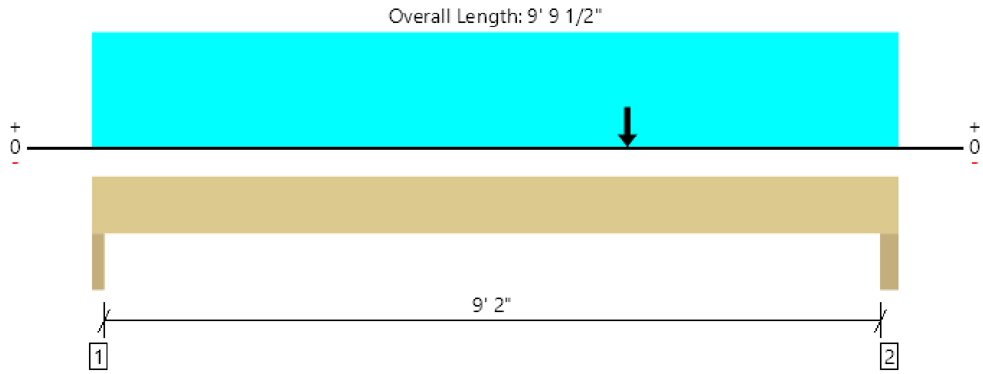
Weyerhaeuser Notes

ForteWEB Software Operator	Job Notes
Samantha Taylor Anthem Structural Engineers (303) 848-8497 staylor@anthemstructural.com	

Weyerhaeuser

4 bed unit typ joist, BM @ GARAGE / HOLD DOWN - C

2 piece(s) 1 3/4" x 14" 2.0E Microllam® LVL



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

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	6830 @ 1 1/2"	7613 (3.00")	Passed (90%)	--	1.0 D + 0.45 W + 0.75 L + 0.75 S (All Spans)
Shear (lbs)	7282 @ 8' 3"	14896	Passed (49%)	1.60	1.0 D + 0.45 W + 0.75 L + 0.75 S (All Spans)
Moment (Ft-lbs)	21419 @ 6' 6"	38813	Passed (55%)	1.60	1.0 D + 0.45 W + 0.75 L + 0.75 S (All Spans)
Live Load Defl. (in)	0.224 @ 5' 7/16"	0.314	Passed (L/505)	--	1.0 D + 0.45 W + 0.75 L + 0.75 S (All Spans)
Total Load Defl. (in)	0.249 @ 5' 3/16"	0.471	Passed (L/453)	--	1.0 D + 0.45 W + 0.75 L + 0.75 S (All Spans)

System : Wall
Member Type : Header
Building Use : Residential
Building Code : IBC 2018
Design Methodology : ASD

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

Supports	Bearing Length			Loads to Supports (lbs)					Accessories
	Total	Available	Required	Dead	Floor Live	Snow	Wind	Factored	
1 - Trimmer - SPF	3.00"	3.00"	2.69"	906	2617	2925	3926	6830	None
2 - Trimmer - HF	4.50"	4.50"	3.51"	930	2685	3001	8228	8897	None

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	7' o/c	
Bottom Edge (Lu)	9' 10" o/c	

•Maximum allowable bracing intervals based on applied load.

Vertical Loads	Location	Tributary Width	Dead (0.90)	Floor Live (1.00)	Snow (1.15)	Wind (1.60)	Comments
0 - Self Weight (PLF)	0 to 9' 9 1/2"	N/A	14.3	--	--	--	
1 - Uniform (PSF)	0 to 9' 9 1/2"	1' 6"	15.0	40.0	-	-	Default Load
2 - Point (lb)	6' 6"	N/A	-	-	-	12154	Holddown
3 - Uniform (PLF)	0 to 9' 9 1/2"	N/A	150.8	481.5	605.3	-	Linked from: Floor Joist #3 - 12'-6" span, Support 1

Weyerhaeuser Notes

ForteWEB Software Operator	Job Notes
Samantha Taylor Anthem Structural Engineers (303) 848-8497 staylor@anthemstructural.com	

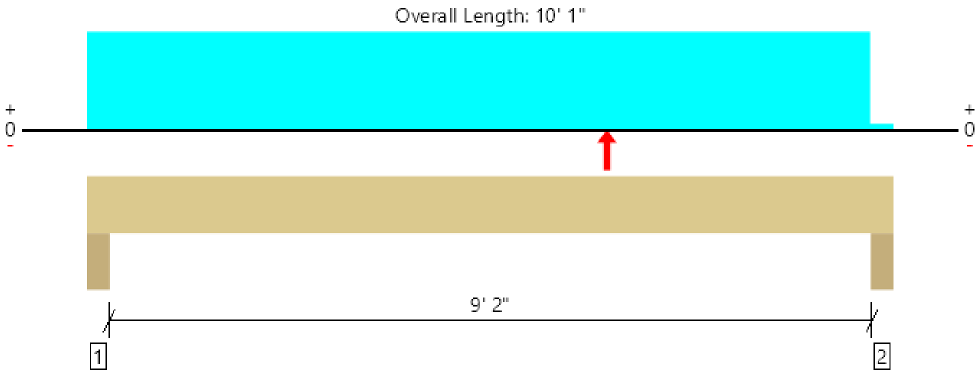
4 bed unit typ joist, BM @ GARAGE / HOLD DOWN - T

2 piece(s) 1 3/4" x 14" 2.0E Microllam® LVL

An excessive uplift of -1400 lbs at support located at 4" failed this product.

An excessive uplift of -3192 lbs at support located at 9' 9" failed this product.

UPLIFT RESOLVED WITH HOLD DOWNS
SEE PLAN LEVEL 1 PLAN



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

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	5282 @ 4"	13956 (5.50")	Passed (38%)	--	1.0 D + 0.75 L + 0.75 S (All Spans)
Shear (lbs)	3579 @ 1' 7 1/2"	10707	Passed (33%)	1.15	1.0 D + 0.75 L + 0.75 S (All Spans)
Moment (Ft-lbs)	11612 @ 5' 1/2"	27897	Passed (42%)	1.15	1.0 D + 0.75 L + 0.75 S (All Spans)
Live Load Defl. (in)	0.117 @ 5' 1/2"	0.314	Passed (L/962)	--	1.0 D + 0.75 L + 0.75 S (All Spans)
Total Load Defl. (in)	0.143 @ 5' 1/2"	0.471	Passed (L/790)	--	1.0 D + 0.75 L + 0.75 S (All Spans)

System : Wall
Member Type : Header
Building Use : Residential
Building Code : IBC 2018
Design Methodology : ASD

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

Supports	Bearing Length			Loads to Supports (lbs)					Accessories
	Total	Available	Required	Dead	Floor Live	Snow	Wind	Factored	
1 - Trimmer - SPF	5.50"	5.50"	2.08"	946	2730	3051	-3279	5282/-1400	None
2 - Trimmer - HF	5.50"	5.50"	1.97"	902	2590	2875	-6221	5000/-3192	None

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	10' 1" o/c	
Bottom Edge (Lu)	10' 1" o/c	

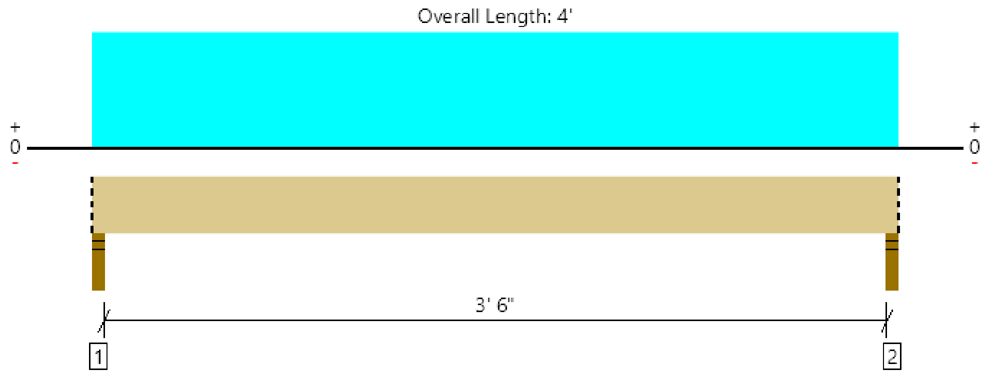
•Maximum allowable bracing intervals based on applied load.

Vertical Loads	Location	Tributary Width	Dead (0.90)	Floor Live (1.00)	Snow (1.15)	Wind (1.60)	Comments
0 - Self Weight (PLF)	0 to 10' 1"	N/A	14.3	--	--	--	
1 - Uniform (PSF)	0 to 10' 1"	1' 6"	15.0	40.0	-	-	Default Load
2 - Point (lb)	6' 6"	N/A	-	-	-	-9500	Holddown Uplift
3 - Uniform (PLF)	0 to 9' 9 1/2"	N/A	150.8	481.5	605.3	-	Linked from: Floor Joist #3 - 12'-6" span, Support 1

ForteWEB Software Operator	Job Notes
Samantha Taylor Anthem Structural Engineers (303) 848-8497 staylor@anthemstructural.com	

4 bed unit typ joist, STAIR LEDGER

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



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

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	398 @ 1 1/2"	1823 (3.00")	Passed (22%)	--	1.0 D + 1.0 L (All Spans)
Shear (lbs)	228 @ 10 1/4"	1088	Passed (21%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	349 @ 2'	1117	Passed (31%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.010 @ 2'	0.094	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.014 @ 2'	0.188	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)

System : Floor
Member Type : Flush Beam
Building Use : Residential
Building Code : IBC 2018
Design Methodology : ASD

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

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Floor Live	Factored	
1 - Stud wall - HF	3.00"	3.00"	1.50"	118	280	398	Blocking
2 - Stud wall - HF	3.00"	3.00"	1.50"	118	280	398	Blocking

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

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	4' o/c	
Bottom Edge (Lu)	4' o/c	

•Maximum allowable bracing intervals based on applied load.

Vertical Loads	Location (Side)	Tributary Width	Dead (0.90)	Floor Live (1.00)	Comments
0 - Self Weight (PLF)	0 to 4'	N/A	2.8	--	
1 - Uniform (PSF)	0 to 4' (Front)	3' 6"	16.0	40.0	Default Load

Weyerhaeuser Notes

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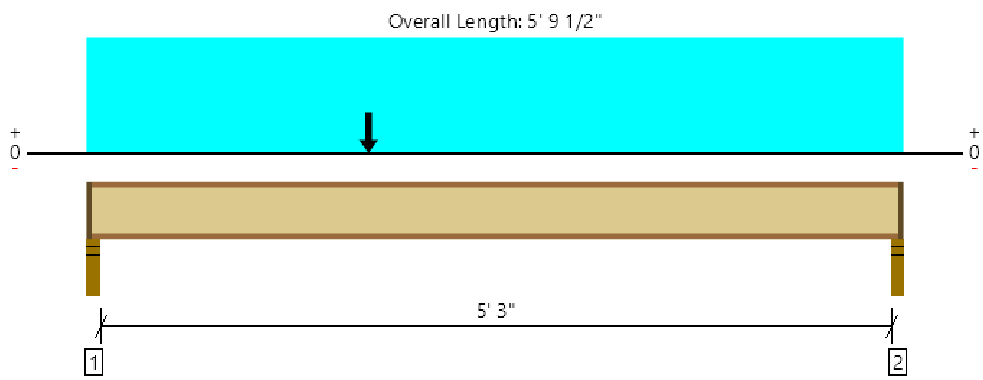
The product application, input design loads, dimensions and support information have been provided by ForteWEB Software Operator

ForteWEB Software Operator	Job Notes
Samantha Taylor Anthem Structural Engineers (303) 848-8497 staylor@anthemstructural.com	

4 bed unit typ joist, TRANSFER BM @ STAIR POST
1 piece(s) 14" TJI® 360



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

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	485 @ 2"	1202 (2.25")	Passed (40%)	1.00	1.0 D + 1.0 L (All Spans)
Shear (lbs)	470 @ 3 1/2"	1955	Passed (24%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	748 @ 2'	7335	Passed (10%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.012 @ 2'	0.138	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.017 @ 2'	0.275	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)

System : Floor
Member Type : Flush Beam
Building Use : Residential
Building Code : IBC 2018
Design Methodology : ASD

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

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Floor Live	Factored	
1 - Stud wall - HF	3.50"	2.25"	1.75"	150	342	492	1 1/4" Rim Board
2 - Stud wall - HF	3.00"	1.75"	1.75"	110	247	356	1 1/4" Rim Board

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

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	5' 7" o/c	
Bottom Edge (Lu)	5' 7" o/c	

- TJI joists are only analyzed using Maximum Allowable bracing solutions.
- Maximum allowable bracing intervals based on applied load.

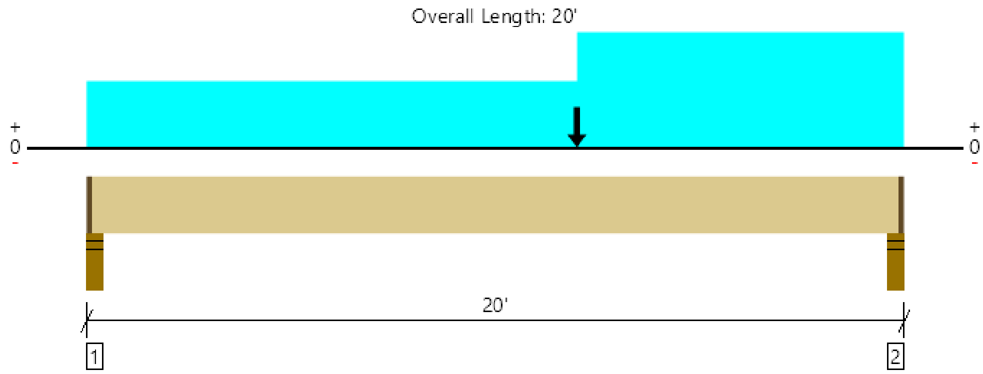
Vertical Loads	Location	Tributary Width	Dead (0.90)	Floor Live (1.00)	Comments
0 - Self Weight (PLF)	1 1/4" to 5' 8 1/4"	N/A	3.3	--	
1 - Uniform (PSF)	0 to 5' 9 1/2"	1' 4"	16.0	40.0	Default Load
2 - Point (lb)	2'	N/A	118	280	Linked from: STAIR LEDGER, Support 1

Weyerhaeuser Notes

ForteWEB Software Operator	Job Notes
Samantha Taylor Anthem Structural Engineers (303) 848-8497 staylor@anthemstructural.com	

4 bed unit typ joist, Copy of Floor Beam #8 - L4 ROOF TRANSFER

4 piece(s) 1 3/4" x 14" 2.0E Microllam® LVL



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

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	6677 @ 19' 9 1/2"	7796 (2.75")	Passed (86%)	--	1.0 D + 1.0 L (All Spans)
Shear (lbs)	5834 @ 18' 6"	18620	Passed (31%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	33212 @ 12'	48517	Passed (68%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.290 @ 10' 6 7/8"	0.490	Passed (L/810)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.696 @ 10' 4 5/16"	0.979	Passed (L/338)	--	1.0 D + 1.0 L (All Spans)

System : Floor
Member Type : Flush Beam
Building Use : Residential
Building Code : IBC 2018
Design Methodology : ASD

- Deflection criteria: LL (L/480) and TL (L/240).
- Allowed moment does not reflect the adjustment for the beam stability factor.
- Member should be side-loaded from both sides of the member or braced to prevent rotation.

Supports	Bearing Length			Loads to Supports (lbs)				Accessories
	Total	Available	Required	Dead	Floor Live	Snow	Factored	
1 - Stud wall - HF	4.00"	2.75"	1.74"	3235	1582	730	4969	1 1/4" Rim Board
2 - Stud wall - HF	4.00"	2.75"	2.36"	3809	2928	730	6737	1 1/4" Rim Board

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

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	15' 3" o/c	
Bottom Edge (Lu)	19' 10" o/c	

•Maximum allowable bracing intervals based on applied load.

Vertical Loads	Location (Side)	Tributary Width	Dead (0.90)	Floor Live (1.00)	Snow (1.15)	Comments
0 - Self Weight (PLF)	1 1/4" to 19' 10 3/4"	N/A	28.6	--	--	
1 - Uniform (PSF)	0 to 20' (Top)	1' 4"	16.0	40.0	-	Floor Load
2 - Uniform (PLF)	12' to 20' (Front)	N/A	78.7	196.7	-	Stair Load
3 - Uniform (PLF)	0 to 20' (Top)	N/A	225.0	-	73.0	
4 - Point (lb)	12' (Front)	N/A	922	1870	-	Linked from: Floor Beam #7, Support 1

Member Notes

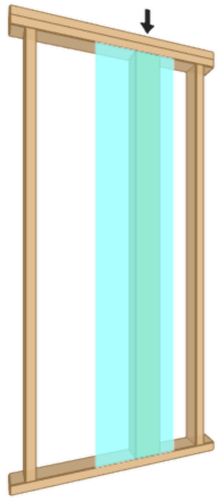
Stair Framing Beam

Weyerhaeuser Notes

ForteWEB Software Operator	Job Notes
Samantha Taylor Anthem Structural Engineers (303) 848-8497 staylor@anthemstructural.com	

4 bed unit typ joist, Copy of Post #1 - CENTER POST
4 piece(s) 2 x 6 HF No.2

Wall Height: 9' | Member Height: 8' 7 1/2" | Tributary Width: 1' 4"



Drawing is Conceptual

Design Results	Actual	Allowed	Result	LDF	Load: Combination
Slenderness	19	50	Passed (38%)	--	--
Compression (lbs)	16558	27916	Passed (59%)	1.00	1.0 D + 1.0 L
Plate Bearing (lbs)	16558	20625	Passed (80%)	--	1.0 D + 1.0 L
Lateral Reaction (lbs)	87	--	--	1.60	1.0 D + 0.6 W
Lateral Shear (lbs)	77	5280	Passed (1%)	1.60	1.0 D + 0.6 W
Lateral Moment (ft-lbs)	187 @ mid-span	4454	Passed (4%)	1.60	1.0 D + 0.6 W
Total Deflection (in)	0.07 @ mid-span	0.43	Passed (L/1542)	--	1.0 D + 0.45 W + 0.75 L + 0.75 S
Bending/Compression	0.91	1	Passed (91%)	1.00	1.0 D + 1.0 L

- Lateral deflection criteria: Wind (L/240)
- Input axial load eccentricity for this design is 10% of applicable member side dimension.
- Applicable calculations are based on NDS.
- Bearing shall be on a metal plate or strap, or on other equivalently durable, rigid, homogeneous material with sufficient stiffness to distribute applied load.
- The column stability factor (Kf = 0.6) applied to this design assumes nailed built-up columns per NDS section 15.3.3. For Weyerhaeuser ELP products refer to the U.S. Wall Guide for multiple-member connection requirements.

Supports	Type	Material
Top	Dbl 2X	Douglas Fir-Larch
Base	2X	Douglas Fir-Larch

System : Wall
Member Type : Column
Building Code : IBC 2018
Design Methodology : ASD

Max Unbraced Length	Comments
1'	

Lateral Connections

Supports	Connector	Type/Model	Quantity	Connector Nailing
Top	Nails	8d (0.113" x 2 1/2") (Toe)	2	N/A
Base	Nails	8d (0.113" x 2 1/2") (Toe)	2	N/A

- Nailed connection at the top of the member is assumed to be nailed through the bottom 2x plate prior to placement of the top 2x of the double top plate assembly.

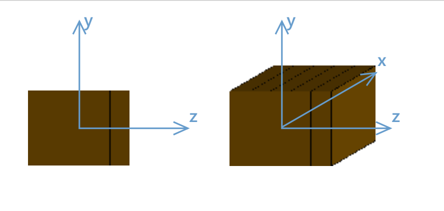
Vertical Loads	Tributary Width	Dead (0.90)	Floor Live (1.00)	Snow (1.15)	Comments
1 - Point (lb)	N/A	10	-	-	Wall Dead Load
2 - Point (lb)	N/A	1478	2909	-	Linked from: Floor Beam #8, Support 2
3 - Point (lb)	N/A	925	1602	-	Linked from: Floor Beam #8, Support 1
4 - Point (lb)	N/A	3235	1582	730	Linked from: Copy of Floor Beam #8 - L4 ROOF TRANSFER, Support 1
5 - Point (lb)	N/A	3235	1582	730	Linked from: Copy of Floor Beam #8 - L4 ROOF TRANSFER, Support 1

Lateral Load	Location	Tributary Width	Wind (1.60)	Comments
1 - Uniform (PSF)	Full Length	1' 4"	25.1	

- ASCE/SEI 7 Sec. 30.4: Exposure Category (B), Mean Roof Height (33'), Topographic Factor (1.0), Wind Directionality Factor (0.85), Basic Wind Speed (115), Risk Category(II), Effective Wind Area determined using full member span and trib. width.
- IBC Table 1604.3, footnote f: Deflection checks are performed using 42% of this lateral wind load.

ForteWEB Software Operator	Job Notes
Samantha Taylor Anthem Structural Engineers (303) 848-8497 staylor@anthemstructural.com	

Detail Report: M24 **Unity Check:** 4.381 (LC 7) **Load Combination:** Envelope



Input Data:

Shape:	5-2X6 (nominal)	I Node:	N45
Member Type:	Column	J Node:	N48
Length (ft):	10.233	I Release:	Fixed
Material Type:	Wood	J Release:	Fixed
Design Rule:	Typical	I Offset (in):	N/A
Number of Internal Sections:	97	J Offset (in):	N/A

Material Properties:

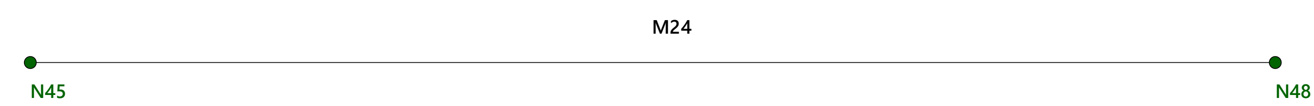
Material:	HF	Grade:	No.2	Nu:	0.3
Type:	Solid Sawn	Cm:	No	Therm. Coeff. ($1e^5 °F^{-1}$):	0.3
Database:	Visually Graded	Ci:	No	Density (k/ft^3):	0.035
Species:	Hem-Fir	Emod:	1		

Shape Properties:

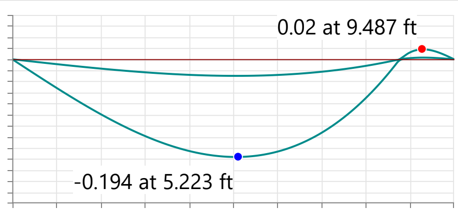
F_b (ksi):	0.85	E (ksi):	1300	b (actual) (in):	7.5
F_t (ksi):	0.525	Emod:	1	d (actual) (in):	5.5
F_v (ksi):	0.15	COV_E (Table F1):	0.25	# of Plies:	5
F_c (ksi):	1.3	E_{min} (ksi):	474.901	K_f:	0.6

Design Properties:

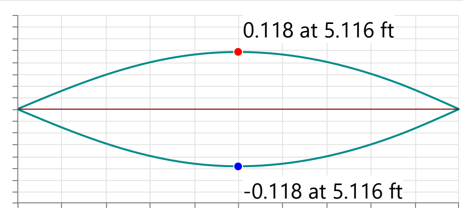
le2 (ft) :	0.5	C_D:	1.15	Max Defl Ratio:	L/0
le1 (ft) :	N/A	R_B:	3.465	Max Defl Location:	0
le-bend top (ft) :	Lbyy	C_L:	0.999	Span:	N/A
le-bend bot (ft) :	N/A	C_r:	1		
K_{y-y}:	1	C_{fu}:	1		
K_{z-z}:	1	C_P:	0.417		
y sway:	No	K_f:	0.6		
z sway:	No				



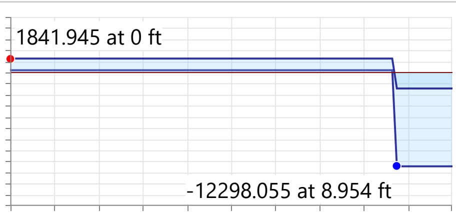
Diagrams:



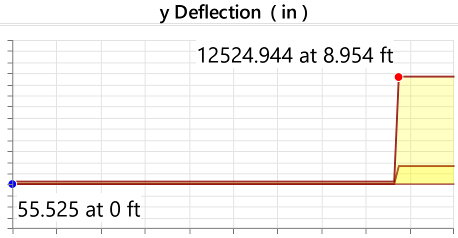
y Deflection (in)



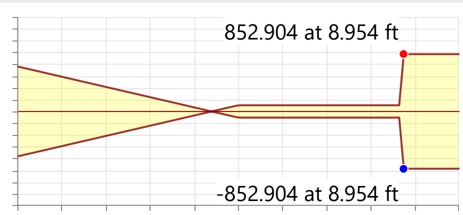
z Deflection (in)



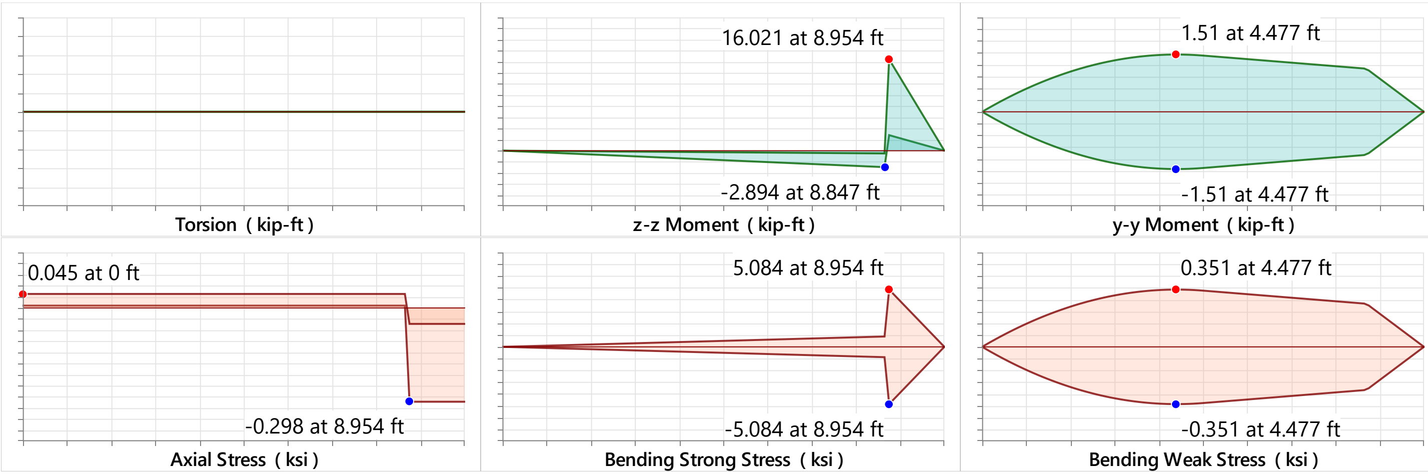
Axial Force (lbs)



y Shear Force (lbs)



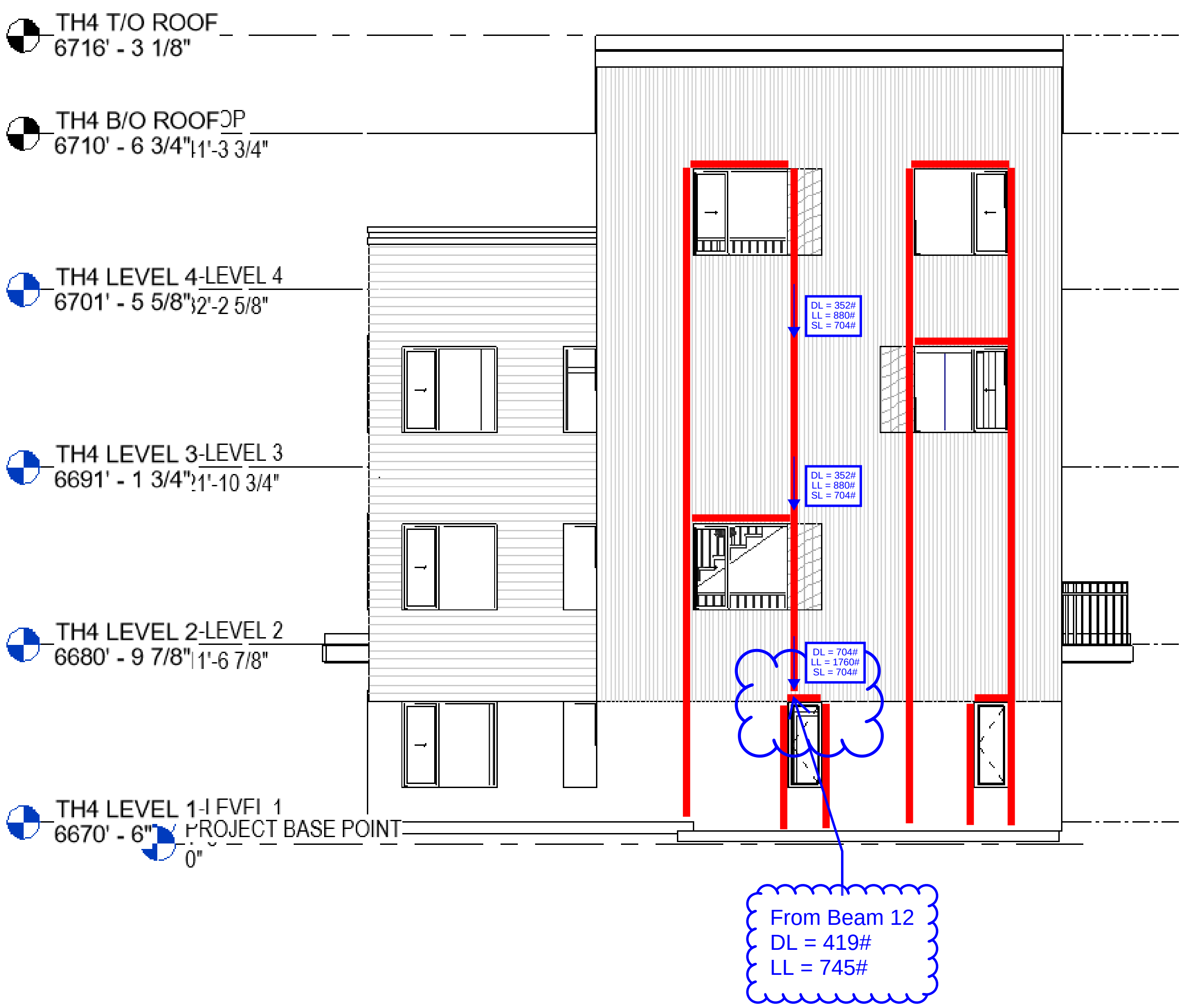
z Shear Force (lbs)



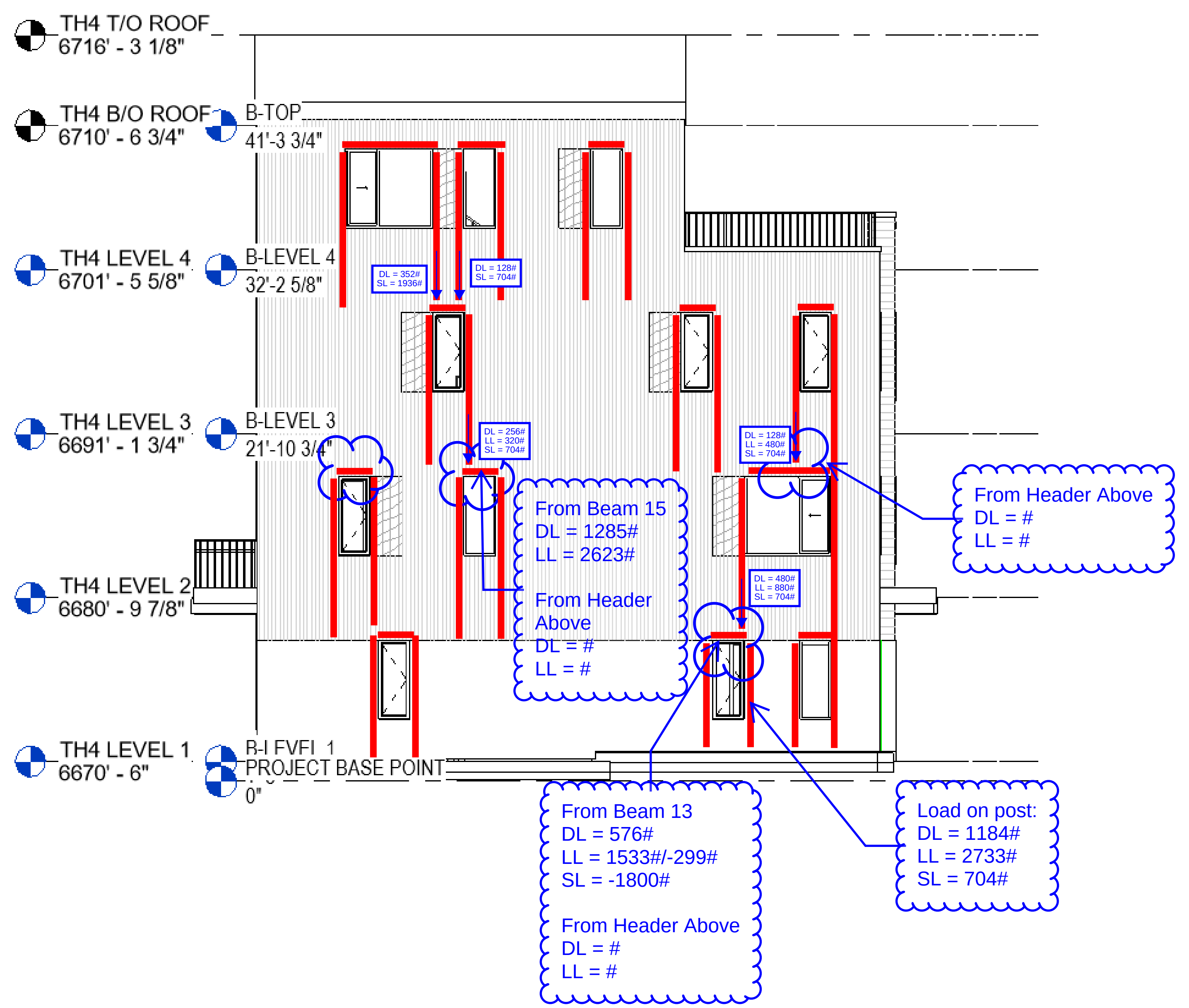
AWC NDS-18: ASD Code Check

Limit State	Gov. LC	Required	Available	Unity Check	Result
Applied Loading - Bending/Axial	7	-	-	-	-
Applied Loading - Shear + Torsion	7	-	-	-	-
Axial Compression Analysis		0.000 ksi	0.685 ksi	-	-
Axial Tension Analysis		-0.298 ksi	0.785 ksi	-	-
Flexural Analysis, Fb1'		5.084 ksi	1.269 ksi	-	-
Flexural Analysis, Fb2'		0.000 ksi	1.271 ksi	-	-
Bending & Axial Compression Analysis		-	-	4.007	Fail
Bending & Axial Tension Analysis		-	-	4.381	Fail
Shear Analysis		0.455 ksi	0.172 ksi	2.64	Fail

TYPE B-WIDE
EAST ELEVATION



TYPE B-WIDE (FLIP)
WEST ELEVATION

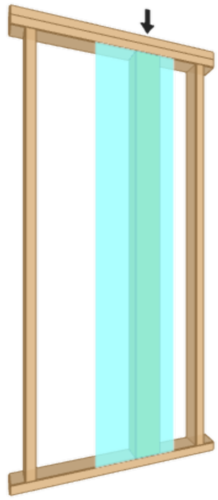


MEMBER REPORT

PASSED

Level, 16' Garage Header Column
3 piece(s) 2 x 6 HF No.2

Wall Height: 10' 3 7/8" Member Height: 9' 11 3/8" Tributary Width: 4' 6"



Drawing is Conceptual

Design Results	Actual	Allowed	Result	LDF	Load: Combination
Slenderness	22	50	Passed (43%)	--	--
Compression (lbs)	7954	17611	Passed (45%)	1.15	1.0 D + 0.75 L + 0.75 S
Plate Bearing (lbs)	7954	10024	Passed (79%)	--	1.0 D + 0.75 L + 0.75 S
Lateral Reaction (lbs)	324	--	--	1.60	1.0 D + 0.6 W
Lateral Shear (lbs)	294	3960	Passed (7%)	1.60	1.0 D + 0.6 W
Lateral Moment (ft-lbs)	806 @ mid-span	3339	Passed (24%)	1.60	1.0 D + 0.6 W
Total Deflection (in)	0.18 @ mid-span	0.33	Passed (L/682)	--	1.0 D + 0.45 W + 0.75 L + 0.75 S
Bending/Compression	0.81	1	Passed (81%)	1.60	1.0 D + 0.45 W + 0.75 L + 0.75 S

- Lateral deflection criteria: Wind (L/360)
- Input axial load eccentricity for this design is 16.67% of applicable member side dimension.
- Applicable calculations are based on NDS.
- Bearing shall be on a metal plate or strap, or on other equivalently durable, rigid, homogeneous material with sufficient stiffness to distribute applied load.
- The column stability factor (Kf = 0.6) applied to this design assumes nailed built-up columns per NDS section 15.3.3. For Weyerhaeuser ELP products refer to the U.S. Wall Guide for multiple-member connection requirements.

Supports	Type	Material
Top	Dbl 2X	Hem Fir
Base	2X	Hem Fir

System : Wall
Member Type : Column
Building Code : IBC 2018
Design Methodology : ASD

Max Unbraced Length	Comments
1'	

Lateral Connections

Supports	Connector	Type/Model	Quantity	Connector Nailing
Top	Nails	10d (0.128" x 3") (End)	4	N/A
Base	Nails	10d (0.128" x 3") (End)	4	N/A

- Nailed connection at the top of the member is assumed to be nailed through the bottom 2x plate prior to placement of the top 2x of the double top plate assembly.

Vertical Load	Tributary Width	Dead (0.90)	Floor Live (1.00)	Snow (1.15)	Comments
1 - Point (lb)	N/A	2250	2835	4770	Default Load

Lateral Load	Location	Tributary Width	Wind (1.60)	Comments
1 - Uniform (PSF)	Full Length	4' 6"	24.1	

- IBC Table 1604.3, footnote f: Deflection checks are performed using 42% of this lateral wind load.

Weyerhaeuser Notes

ForteWEB Software Operator	Job Notes
Samantha Taylor Anthem Structural Engineers (303) 848-8497 staylor@anthemstructural.com	

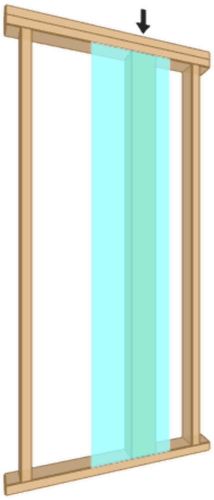
FORTEWEB

MEMBER REPORT

PASSED

Level, 20' Garage Header Column
4 piece(s) 2 x 6 HF No.2

Wall Height: 9' | Member Height: 8' 7 1/2" | Tributary Width: 4' 6"



Drawing is Conceptual

Design Results	Actual	Allowed	Result	LDF	Load: Combination
Slenderness	19	50	Passed (38%)	--	--
Compression (lbs)	14140	29200	Passed (48%)	1.15	1.0 D + 0.75 L + 0.75 S
Plate Bearing (lbs)	14140	20625	Passed (69%)	--	1.0 D + 0.75 L + 0.75 S
Lateral Reaction (lbs)	284	--	--	1.60	1.0 D + 0.6 W
Lateral Shear (lbs)	254	5280	Passed (5%)	1.60	1.0 D + 0.6 W
Lateral Moment (ft-lbs)	612 @ mid-span	4454	Passed (14%)	1.60	1.0 D + 0.6 W
Total Deflection (in)	0.12 @ mid-span	0.29	Passed (L/847)	--	1.0 D + 0.45 W + 0.75 L + 0.75 S
Bending/Compression	0.84	1	Passed (84%)	1.15	1.0 D + 0.75 L + 0.75 S

- Lateral deflection criteria: Wind (L/360)
- Input axial load eccentricity for this design is 16.67% of applicable member side dimension.
- Applicable calculations are based on NDS.
- The column stability factor (Kf = 0.6) applied to this design assumes nailed built-up columns per NDS section 15.3.3. For Weyerhaeuser ELP products refer to the U.S. Wall Guide for multiple-member connection requirements.

Supports	Type	Material
Top	Dbl 2X	Douglas Fir-Larch
Base	2X	Douglas Fir-Larch

System : Wall
Member Type : Column
Building Code : IBC 2018
Design Methodology : ASD

Max Unbraced Length	Comments
1'	

Lateral Connections

Supports	Connector	Type/Model	Quantity	Connector Nailing
Top	Nails	8d (0.113" x 2 1/2") (Toe)	3	N/A
Base	Nails	8d (0.113" x 2 1/2") (Toe)	3	N/A

- Nailed connection at the top of the member is assumed to be nailed through the bottom 2x plate prior to placement of the top 2x of the double top plate assembly.

Vertical Load	Tributary Width	Dead (0.90)	Floor Live (1.00)	Snow (1.15)	Comments
1 - Point (lb)	N/A	4000	5040	8480	Default Load

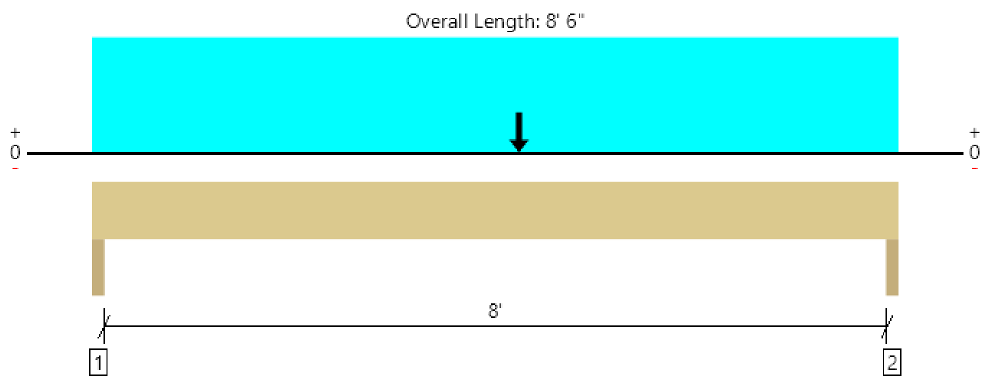
Lateral Load	Location	Tributary Width	Wind (1.60)	Comments
1 - Uniform (PSF)	Full Length	4' 6"	24.4	

- IBC Table 1604.3, footnote f: Deflection checks are performed using 42% of this lateral wind load.

Weyerhaeuser Notes

ForteWEB Software Operator	Job Notes
Samantha Taylor Anthem Structural Engineers (303) 848-8497 staylor@anthemstructural.com	

Level, Wall: Header - 8' TRIB (H1 CHECK)
2 piece(s) 1 3/4" x 9 1/4" 2.0E Microllam® LVL



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

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	2743 @ 8' 4 1/2"	7613 (3.00")	Passed (36%)	--	1.0 D + 1.0 L (All Spans)
Shear (lbs)	2161 @ 7' 5 3/4"	6151	Passed (35%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	6076 @ 4' 6"	11204	Passed (54%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.127 @ 4' 3 1/16"	0.275	Passed (L/782)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.176 @ 4' 3 1/16"	0.412	Passed (L/561)	--	1.0 D + 1.0 L (All Spans)

System : Wall
Member Type : Header
Building Use : Residential
Building Code : IBC 2018
Design Methodology : ASD

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

Supports	Bearing Length			Loads to Supports (lbs)				Accessories
	Total	Available	Required	Dead	Floor Live	Snow	Factored	
1 - Trimmer - HF	3.00"	3.00"	1.50"	780	1925	331	2706	None
2 - Trimmer - HF	3.00"	3.00"	1.50"	788	1955	373	2743	None

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

•Maximum allowable bracing intervals based on applied load.

Vertical Loads	Location	Tributary Width	Dead (0.90)	Floor Live (1.00)	Snow (1.15)	Comments
0 - Self Weight (PLF)	0 to 8' 6"	N/A	9.4	--	--	
1 - Uniform (PSF)	0 to 8' 6"	10'	16.0	40.0	-	Default Load
2 - Point (lb)	4' 6"	N/A	128	480	704	

Weyerhaeuser Notes

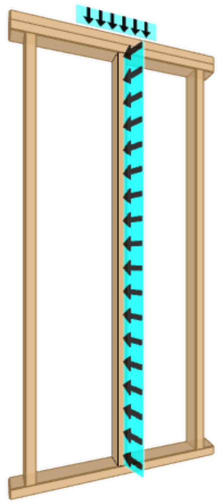
ForteWEB Software Operator	Job Notes
Samantha Taylor Anthem Structural Engineers (303) 848-8497 staylor@anthemstructural.com	

STUD WALLS

EXTERIOR BEARING WALLS, GRID 11, LEVEL 4

1 piece(s) 2 x 6 HF No.2 @ 16" OC

Wall Height: 9' 7 1/8" Member Height: 9' 2 5/8" O. C. Spacing: 16.00"



Drawing is Conceptual

Design Results	Actual	Allowed	Result	LDF	Load: Combination
Slenderness	20	50	Passed (40%)	--	--
Compression (lbs)	1280	6618	Passed (19%)	1.15	1.0 D + 1.0 S
Plate Bearing (lbs)	1280	4177	Passed (31%)	--	1.0 D + 1.0 S
Lateral Reaction (lbs)	140	--	--	1.60	1.0 D + 0.6 W
Lateral Shear (lbs)	126	1320	Passed (10%)	1.60	1.0 D + 0.6 W
Lateral Moment (ft-lbs)	323 @ mid-span	1264	Passed (26%)	1.60	1.0 D + 0.6 W
Total Deflection (in)	0.13 @ mid-span	0.92	Passed (L/829)	--	1.0 D + 0.6 W
Bending/Compression	0.31	1	Passed (31%)	1.60	1.0 D + 0.45 W + 0.75 L + 0.75 S

- Lateral deflection criteria: Wind (L/120)
- Input axial load eccentricity for this design is 16.67% of applicable member side dimension.
- Applicable calculations are based on NDS.
- A bearing area factor of 1.25 has been applied to base plate bearing capacity.
- A 15% increase in the moment capacity has been added to account for repetitive member usage.

Supports	Type	Material
Top	Dbl 2X	Hem Fir
Base	2X	Hem Fir

System : Wall
Member Type : Stud
Building Code : IBC 2018
Design Methodology : ASD

Max Unbraced Length	Comments
1'	

Lateral Connections

Supports	Connector	Type/Model	Quantity	Connector Nailing
Top	Nails	8d (0.113" x 2 1/2") (Toe)	2	N/A
Base	Nails	8d (0.113" x 2 1/2") (Toe)	2	N/A

- Nailed connection at the top of the member is assumed to be nailed through the bottom 2x plate prior to placement of the top 2x of the double top plate assembly.

Vertical Load	Spacing	Dead (0.90)	Snow (1.15)	Comments
1 - Point (PLF)	16.00"	160.0	800.0	ROOF

Lateral Load	Location	Spacing	Wind (1.60)	Comments
1 - Uniform (PLF)	Full Length	N/A	50.6	37.9PSF * 1.33FT

- IBC Table 1604.3, footnote f: Deflection checks are performed using 42% of this lateral wind load.

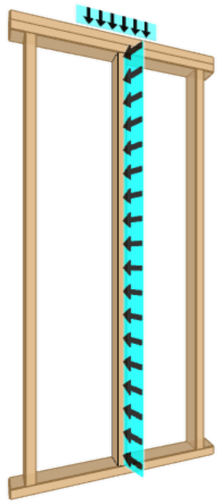
Weyerhaeuser Notes

ForteWEB Software Operator	Job Notes
Samantha Taylor Anthem Structural Engineers (303) 848-8497 staylor@anthemstructural.com	

EXTERIOR BEARING WALLS, GRID 11, LEVEL 3

1 piece(s) 2 x 6 HF No.2 @ 16" OC

Wall Height: 9' 7 1/8" Member Height: 9' 2 5/8" O. C. Spacing: 16.00"



Drawing is Conceptual

Design Results	Actual	Allowed	Result	LDF	Load: Combination
Slenderness	20	50	Passed (40%)	--	--
Compression (lbs)	1760	6618	Passed (27%)	1.15	1.0 D + 0.75 L + 0.75 S
Plate Bearing (lbs)	1760	4177	Passed (42%)	--	1.0 D + 0.75 L + 0.75 S
Lateral Reaction (lbs)	140	--	--	1.60	1.0 D + 0.6 W
Lateral Shear (lbs)	126	1320	Passed (10%)	1.60	1.0 D + 0.6 W
Lateral Moment (ft-lbs)	323 @ mid-span	1264	Passed (26%)	1.60	1.0 D + 0.6 W
Total Deflection (in)	0.14 @ mid-span	0.92	Passed (L/775)	--	1.0 D + 0.45 W + 0.75 L + 0.75 S
Bending/Compression	0.45	1	Passed (45%)	1.60	1.0 D + 0.45 W + 0.75 L + 0.75 S

- Lateral deflection criteria: Wind (L/120)
- Input axial load eccentricity for this design is 16.67% of applicable member side dimension.
- Applicable calculations are based on NDS.
- A bearing area factor of 1.25 has been applied to base plate bearing capacity.
- A 15% increase in the moment capacity has been added to account for repetitive member usage.

Supports	Type	Material
Top	Dbl 2X	Hem Fir
Base	2X	Hem Fir

System : Wall
Member Type : Stud
Building Code : IBC 2018
Design Methodology : ASD

Max Unbraced Length	Comments
1'	

Lateral Connections				
Supports	Connector	Type/Model	Quantity	Connector Nailing
Top	Nails	8d (0.113" x 2 1/2") (Toe)	2	N/A
Base	Nails	8d (0.113" x 2 1/2") (Toe)	2	N/A

- Nailed connection at the top of the member is assumed to be nailed through the bottom 2x plate prior to placement of the top 2x of the double top plate assembly.

Vertical Loads	Spacing	Dead (0.90)	Floor Live (1.00)	Snow (1.15)	Comments
1 - Point (PLF)	16.00"	160.0	-	800.0	ROOF
2 - Point (PLF)	16.00"	260.0	400.0	-	LEVEL 4

Lateral Load	Location	Spacing	Wind (1.60)	Comments
1 - Uniform (PLF)	Full Length	N/A	50.6	37.9PSF * 1.33FT

- IBC Table 1604.3, footnote f: Deflection checks are performed using 42% of this lateral wind load.

Weyerhaeuser Notes

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

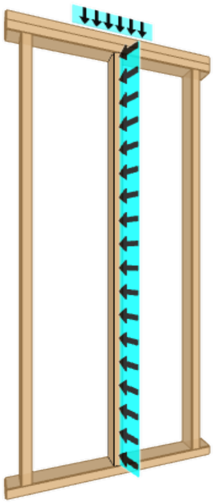
ForteWEB Software Operator	Job Notes
Samantha Taylor Anthem Structural Engineers (303) 848-8497 staylor@anthemstructural.com	

MEMBER REPORT

PASSED

EXTERIOR BEARING WALLS, GRID 11, LEVEL 2
1 piece(s) 2 x 6 HF No.2 @ 16" OC

Wall Height: 9' 7 1/8" Member Height: 9' 2 5/8" O. C. Spacing: 16.00"



Drawing is Conceptual

Design Results	Actual	Allowed	Result	LDF	Load: Combination
Slenderness	20	50	Passed (40%)	--	--
Compression (lbs)	2507	6618	Passed (38%)	1.15	1.0 D + 0.75 L + 0.75 S
Plate Bearing (lbs)	2507	4177	Passed (60%)	--	1.0 D + 0.75 L + 0.75 S
Lateral Reaction (lbs)	140	--	--	1.60	1.0 D + 0.6 W
Lateral Shear (lbs)	126	1320	Passed (10%)	1.60	1.0 D + 0.6 W
Lateral Moment (ft-lbs)	323 @ mid-span	1264	Passed (26%)	1.60	1.0 D + 0.6 W
Total Deflection (in)	0.16 @ mid-span	0.92	Passed (L/681)	--	1.0 D + 0.45 W + 0.75 L + 0.75 S
Bending/Compression	0.65	1	Passed (65%)	1.60	1.0 D + 0.45 W + 0.75 L + 0.75 S

- Lateral deflection criteria: Wind (L/120)
- Input axial load eccentricity for this design is 16.67% of applicable member side dimension.
- Applicable calculations are based on NDS.
- A bearing area factor of 1.25 has been applied to base plate bearing capacity.
- A 15% increase in the moment capacity has been added to account for repetitive member usage.

Supports	Type	Material
Top	Dbl 2X	Hem Fir
Base	2X	Hem Fir

System : Wall
Member Type : Stud
Building Code : IBC 2018
Design Methodology : ASD

Max Unbraced Length	Comments
1'	

Lateral Connections				
Supports	Connector	Type/Model	Quantity	Connector Nailing
Top	Nails	8d (0.113" x 2 1/2") (Toe)	2	N/A
Base	Nails	8d (0.113" x 2 1/2") (Toe)	2	N/A

- Nailed connection at the top of the member is assumed to be nailed through the bottom 2x plate prior to placement of the top 2x of the double top plate assembly.

Vertical Loads	Spacing	Dead (0.90)	Floor Live (1.00)	Snow (1.15)	Comments
1 - Point (PLF)	16.00"	160.0	-	800.0	ROOF
2 - Point (PLF)	16.00"	260.0	400.0	-	LEVEL 4
3 - Point (PLF)	16.00"	260.0	400.0	-	LEVEL 3

Lateral Load	Location	Spacing	Wind (1.60)	Comments
1 - Uniform (PLF)	Full Length	N/A	50.6	37.9PSF * 1.33FT

- IBC Table 1604.3, footnote f: Deflection checks are performed using 42% of this lateral wind load.

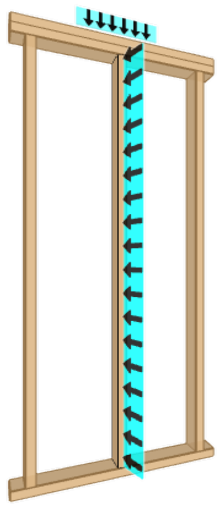
ForteWEB Software Operator	Job Notes
Samantha Taylor Anthem Structural Engineers (303) 848-8497 staylor@anthemstructural.com	

MEMBER REPORT

PASSED

EXTERIOR BEARING WALLS, GRID 11, LEVEL 1
1 piece(s) 2 x 6 HF No.2 @ 16" OC

Wall Height: 9' 7 1/8" Member Height: 9' 2 5/8" O. C. Spacing: 16.00"



Drawing is Conceptual

Design Results	Actual	Allowed	Result	LDF	Load: Combination
Slenderness	20	50	Passed (40%)	--	--
Compression (lbs)	3253	6618	Passed (49%)	1.15	1.0 D + 0.75 L + 0.75 S
Plate Bearing (lbs)	3253	6445	Passed (50%)	--	1.0 D + 0.75 L + 0.75 S
Lateral Reaction (lbs)	140	--	--	1.60	1.0 D + 0.6 W
Lateral Shear (lbs)	126	1320	Passed (10%)	1.60	1.0 D + 0.6 W
Lateral Moment (ft-lbs)	323 @ mid-span	1264	Passed (26%)	1.60	1.0 D + 0.6 W
Total Deflection (in)	0.18 @ mid-span	0.92	Passed (L/606)	--	1.0 D + 0.45 W + 0.75 L + 0.75 S
Bending/Compression	0.91	1	Passed (91%)	1.60	1.0 D + 0.45 W + 0.75 L + 0.75 S

- Lateral deflection criteria: Wind (L/120)
- Input axial load eccentricity for this design is 16.67% of applicable member side dimension.
- Applicable calculations are based on NDS.
- A bearing area factor of 1.25 has been applied to base plate bearing capacity.
- A 15% increase in the moment capacity has been added to account for repetitive member usage.

Supports	Type	Material
Top	Dbl 2X	Douglas Fir-Larch
Base	2X	Douglas Fir-Larch

System : Wall
Member Type : Stud
Building Code : IBC 2018
Design Methodology : ASD

Max Unbraced Length	Comments
1'	

Lateral Connections				
Supports	Connector	Type/Model	Quantity	Connector Nailing
Top	Nails	8d (0.113" x 2 1/2") (Toe)	2	N/A
Base	Nails	8d (0.113" x 2 1/2") (Toe)	2	N/A

- Nailed connection at the top of the member is assumed to be nailed through the bottom 2x plate prior to placement of the top 2x of the double top plate assembly.

Vertical Loads	Spacing	Dead (0.90)	Floor Live (1.00)	Snow (1.15)	Comments
1 - Point (PLF)	16.00"	160.0	-	800.0	ROOF
2 - Point (PLF)	16.00"	260.0	400.0	-	LEVEL 4
3 - Point (PLF)	16.00"	260.0	400.0	-	LEVEL 3
4 - Point (PLF)	16.00"	260.0	400.0	-	LEVEL 2

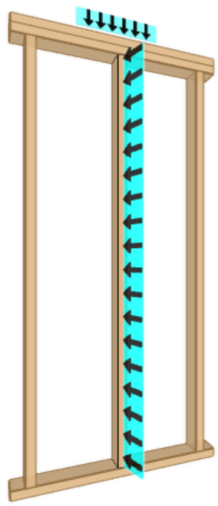
Lateral Load	Location	Spacing	Wind (1.60)	Comments
1 - Uniform (PLF)	Full Length	N/A	50.6	37.9PSF * 1.33FT

- IBC Table 1604.3, footnote f: Deflection checks are performed using 42% of this lateral wind load.

ForteWEB Software Operator	Job Notes
Samantha Taylor Anthem Structural Engineers (303) 848-8497 staylor@anthemstructural.com	

EXTERIOR BEARING WALLS, GRID 11, LEVEL 1 (AT UPPER DECK)
2 piece(s) 2 x 6 HF No.2 @ 16" OC

Wall Height: 10' 3" Member Height: 9' 10 1/2" O. C. Spacing: 16.00"



Drawing is Conceptual

Design Results	Actual	Allowed	Result	LDF	Load: Combination
Slenderness	22	50	Passed (43%)	--	--
Compression (lbs)	4140	11881	Passed (35%)	1.15	1.0 D + 0.75 L + 0.75 S
Plate Bearing (lbs)	4140	11602	Passed (36%)	--	1.0 D + 0.75 L + 0.75 S
Lateral Reaction (lbs)	150	--	--	1.60	1.0 D + 0.6 W
Lateral Shear (lbs)	136	2640	Passed (5%)	1.60	1.0 D + 0.6 W
Lateral Moment (ft-lbs)	370 @ mid-span	2555	Passed (14%)	1.60	1.0 D + 0.6 W
Total Deflection (in)	0.13 @ mid-span	0.99	Passed (L/938)	--	1.0 D + 0.45 W + 0.75 L + 0.75 S
Bending/Compression	0.45	1	Passed (45%)	1.60	1.0 D + 0.45 W + 0.75 L + 0.75 S

- Lateral deflection criteria: Wind (L/120)
- Input axial load eccentricity for this design is 16.67% of applicable member side dimension.
- Applicable calculations are based on NDS.
- A bearing area factor of 1.125 has been applied to base plate bearing capacity.
- The column stability factor (Kf = 0.6) applied to this design assumes nailed built-up columns per NDS section 15.3.3. For Weyerhaeuser ELP products refer to the U.S. Wall Guide for multiple-member connection requirements.
- A 15% increase in the moment capacity has been added to account for repetitive member usage.

Supports	Type	Material
Top	Dbl 2X	Douglas Fir-Larch
Base	2X	Douglas Fir-Larch

System : Wall
Member Type : Stud
Building Code : IBC 2018
Design Methodology : ASD

Max Unbraced Length	Comments
1'	

Lateral Connections

Supports	Connector	Type/Model	Quantity	Connector Nailing
Top	Nails	8d (0.113" x 2 1/2") (Toe)	2	N/A
Base	Nails	8d (0.113" x 2 1/2") (Toe)	2	N/A

- Nailed connection at the top of the member is assumed to be nailed through the bottom 2x plate prior to placement of the top 2x of the double top plate assembly.

Vertical Loads	Spacing	Dead (0.90)	Floor Live (1.00)	Snow (1.15)	Comments
1 - Point (PLF)	16.00"	-	-	-	ROOF
2 - Point (PLF)	16.00"	260.0	600.0	1700.0	LEVEL 4 (DECK)
3 - Point (PLF)	16.00"	260.0	400.0	-	LEVEL 3
4 - Point (PLF)	16.00"	260.0	400.0	-	LEVEL 2

Lateral Load	Location	Spacing	Wind (1.60)	Comments
1 - Uniform (PLF)	Full Length	N/A	50.6	37.9PSF * 1.33FT

- IBC Table 1604.3, footnote f: Deflection checks are performed using 42% of this lateral wind load.

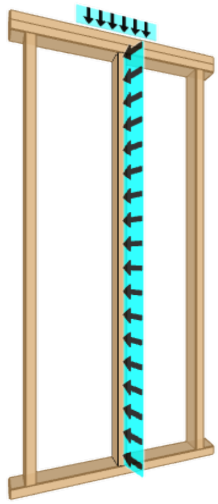
Weyerhaeuser Notes

ForteWEB Software Operator	Job Notes
Samantha Taylor Anthem Structural Engineers (303) 848-8497 staylor@anthemstructural.com	

PASSED

EXTERIOR NON-BEARING, GRID 1A, LEVEL 3 TYP
1 piece(s) 2 x 6 HF No.2 @ 16" OC

Wall Height: 9' 7 1/8" | Member Height: 9' 2 5/8" | O. C. Spacing: 16.00"



Drawing is Conceptual

Design Results	Actual	Allowed	Result	LDF	Load: Combination
Slenderness	20	50	Passed (40%)	--	--
Compression (lbs)	355	6618	Passed (5%)	1.15	1.0 D + 0.75 L + 0.75 S
Plate Bearing (lbs)	355	4177	Passed (8%)	--	1.0 D + 0.75 L + 0.75 S
Lateral Reaction (lbs)	140	--	--	1.60	1.0 D + 0.6 W
Lateral Shear (lbs)	126	1320	Passed (10%)	1.60	1.0 D + 0.6 W
Lateral Moment (ft-lbs)	323 @ mid-span	1264	Passed (26%)	1.60	1.0 D + 0.6 W
Total Deflection (in)	0.13 @ mid-span	0.92	Passed (L/858)	--	1.0 D + 0.6 W
Bending/Compression	0.26	1	Passed (26%)	1.60	1.0 D + 0.6 W

- Lateral deflection criteria: Wind (L/120)
- Input axial load eccentricity for this design is 16.67% of applicable member side dimension.
- Applicable calculations are based on NDS.
- A bearing area factor of 1.25 has been applied to base plate bearing capacity.
- A 15% increase in the moment capacity has been added to account for repetitive member usage.

Supports	Type	Material
Top	Dbl 2X	Hem Fir
Base	2X	Hem Fir

System : Wall
Member Type : Stud
Building Code : IBC 2018
Design Methodology : ASD

Max Unbraced Length	Comments
1'	

Lateral Connections				
Supports	Connector	Type/Model	Quantity	Connector Nailing
Top	Nails	8d (0.113" x 2 1/2") (Toe)	2	N/A
Base	Nails	8d (0.113" x 2 1/2") (Toe)	2	N/A

- Nailed connection at the top of the member is assumed to be nailed through the bottom 2x plate prior to placement of the top 2x of the double top plate assembly.

Vertical Load	Spacing	Dead (0.90)	Floor Live (1.00)	Snow (1.15)	Comments
1 - Point (PLF)	16.00"	32.0	120.0	192.0	LEVEL 4 (DECK)

Lateral Load	Location	Spacing	Wind (1.60)	Comments
1 - Uniform (PLF)	Full Length	N/A	50.6	37.9PSF * 1.33FT

- IBC Table 1604.3, footnote f: Deflection checks are performed using 42% of this lateral wind load.

Weyerhaeuser Notes

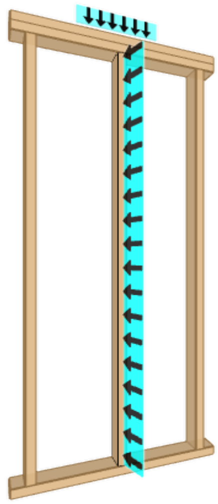
FORTEWEB

MEMBER REPORT

PASSED

EXTERIOR NON-BEARING, GRID 1A, LEVEL 2 TYP
1 piece(s) 2 x 6 HF No.2 @ 16" OC

Wall Height: 9' 7 1/8"   Member Height: 9' 2 5/8"   O. C. Spacing: 16.00"



Drawing is Conceptual

Design Results	Actual	Allowed	Result	LDF	Load: Combination
Slenderness	20	50	Passed (40%)	--	--
Compression (lbs)	504	6618	Passed (8%)	1.15	1.0 D + 0.75 L + 0.75 S
Plate Bearing (lbs)	504	4177	Passed (12%)	--	1.0 D + 0.75 L + 0.75 S
Lateral Reaction (lbs)	140	--	--	1.60	1.0 D + 0.6 W
Lateral Shear (lbs)	126	1320	Passed (10%)	1.60	1.0 D + 0.6 W
Lateral Moment (ft-lbs)	323 @ mid-span	1264	Passed (26%)	1.60	1.0 D + 0.6 W
Total Deflection (in)	0.13 @ mid-span	0.92	Passed (L/846)	--	1.0 D + 0.6 W
Bending/Compression	0.27	1	Passed (27%)	1.60	1.0 D + 0.6 W

- Lateral deflection criteria: Wind (L/120)
- Input axial load eccentricity for this design is 16.67% of applicable member side dimension.
- Applicable calculations are based on NDS.
- A bearing area factor of 1.25 has been applied to base plate bearing capacity.
- A 15% increase in the moment capacity has been added to account for repetitive member usage.

Supports	Type	Material
Top	Dbl 2X	Hem Fir
Base	2X	Hem Fir

System : Wall
Member Type : Stud
Building Code : IBC 2018
Design Methodology : ASD

Max Unbraced Length	Comments
1'	

Lateral Connections				
Supports	Connector	Type/Model	Quantity	Connector Nailing
Top	Nails	8d (0.113" x 2 1/2") (Toe)	2	N/A
Base	Nails	8d (0.113" x 2 1/2") (Toe)	2	N/A

- Nailed connection at the top of the member is assumed to be nailed through the bottom 2x plate prior to placement of the top 2x of the double top plate assembly.

Vertical Loads	Spacing	Dead (0.90)	Floor Live (1.00)	Snow (1.15)	Comments
1 - Point (PLF)	16.00"	32.0	120.0	192.0	LEVEL 4
2 - Point (PLF)	16.00"	52.0	80.0	-	LEVEL 3

Lateral Load	Location	Spacing	Wind (1.60)	Comments
1 - Uniform (PLF)	Full Length	N/A	50.6	37.9PSF * 1.33FT

- IBC Table 1604.3, footnote f: Deflection checks are performed using 42% of this lateral wind load.

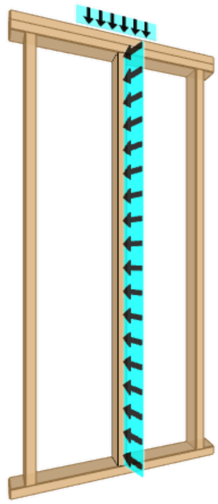
Weyerhaeuser Notes

ForteWEB Software Operator	Job Notes
Samantha Taylor Anthem Structural Engineers (303) 848-8497 staylor@anthemstructural.com	

Weyerhaeuser

EXTERIOR NON-BEARING, GRID 1A, LEVEL 1 TYP
1 piece(s) 2 x 6 HF No.2 @ 16" OC

Wall Height: 9' 7 1/8" | Member Height: 9' 2 5/8" | O. C. Spacing: 16.00"



Drawing is Conceptual

Design Results	Actual	Allowed	Result	LDF	Load: Combination
Slenderness	20	50	Passed (40%)	--	--
Compression (lbs)	653	6618	Passed (10%)	1.15	1.0 D + 0.75 L + 0.75 S
Plate Bearing (lbs)	653	4177	Passed (16%)	--	1.0 D + 0.75 L + 0.75 S
Lateral Reaction (lbs)	140	--	--	1.60	1.0 D + 0.6 W
Lateral Shear (lbs)	126	1320	Passed (10%)	1.60	1.0 D + 0.6 W
Lateral Moment (ft-lbs)	323 @ mid-span	1264	Passed (26%)	1.60	1.0 D + 0.6 W
Total Deflection (in)	0.13 @ mid-span	0.31	Passed (L/834)	--	1.0 D + 0.6 W
Bending/Compression	0.27	1	Passed (27%)	1.60	1.0 D + 0.6 W

- Lateral deflection criteria: Wind (L/360)
- Input axial load eccentricity for this design is 16.67% of applicable member side dimension.
- Applicable calculations are based on NDS.
- A bearing area factor of 1.25 has been applied to base plate bearing capacity.
- A 15% increase in the moment capacity has been added to account for repetitive member usage.

Supports	Type	Material
Top	Dbl 2X	Hem Fir
Base	2X	Hem Fir

System : Wall
Member Type : Stud
Building Code : IBC 2018
Design Methodology : ASD

Max Unbraced Length	Comments
1'	

Lateral Connections				
Supports	Connector	Type/Model	Quantity	Connector Nailing
Top	Nails	8d (0.113" x 2 1/2") (Toe)	2	N/A
Base	Nails	8d (0.113" x 2 1/2") (Toe)	2	N/A

- Nailed connection at the top of the member is assumed to be nailed through the bottom 2x plate prior to placement of the top 2x of the double top plate assembly.

Vertical Loads	Spacing	Dead (0.90)	Floor Live (1.00)	Snow (1.15)	Comments
1 - Point (PLF)	16.00"	32.0	120.0	192.0	LEVEL 4
2 - Point (PLF)	16.00"	52.0	80.0	-	LEVEL 3
3 - Point (PLF)	16.00"	52.0	80.0	-	LEVEL 2

Lateral Load	Location	Spacing	Wind (1.60)	Comments
1 - Uniform (PLF)	Full Length	N/A	50.6	37.9PSF * 1.33FT

- IBC Table 1604.3, footnote f: Deflection checks are performed using 42% of this lateral wind load.

Weyerhaeuser Notes

ForteWEB Software Operator	Job Notes
Samantha Taylor Anthem Structural Engineers (303) 848-8497 staylor@anthemstructural.com	

FORTEWEB

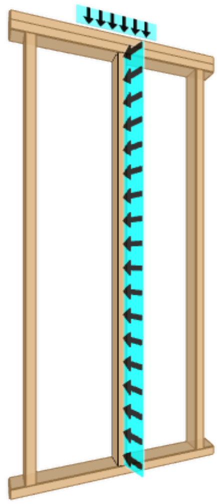
MEMBER REPORT

PASSED

EXTERIOR NON-BEARING, GRID 1E, LEVEL 4 TYP

1 piece(s) 2 x 6 HF No.2 @ 16" OC

Wall Height: 9' 7 1/8" Member Height: 9' 2 5/8" O. C. Spacing: 16.00"



Drawing is Conceptual

Design Results	Actual	Allowed	Result	LDF	Load: Combination
Slenderness	20	50	Passed (40%)	--	--
Compression (lbs)	256	6618	Passed (4%)	1.15	1.0 D + 1.0 S
Plate Bearing (lbs)	256	4177	Passed (6%)	--	1.0 D + 1.0 S
Lateral Reaction (lbs)	140	--	--	1.60	1.0 D + 0.6 W
Lateral Shear (lbs)	126	1320	Passed (10%)	1.60	1.0 D + 0.6 W
Lateral Moment (ft-lbs)	323 @ mid-span	1264	Passed (26%)	1.60	1.0 D + 0.6 W
Total Deflection (in)	0.13 @ mid-span	0.92	Passed (L/858)	--	1.0 D + 0.6 W
Bending/Compression	0.26	1	Passed (26%)	1.60	1.0 D + 0.6 W

- Lateral deflection criteria: Wind (L/120)
- Input axial load eccentricity for this design is 16.67% of applicable member side dimension.
- Applicable calculations are based on NDS.
- A bearing area factor of 1.25 has been applied to base plate bearing capacity.
- A 15% increase in the moment capacity has been added to account for repetitive member usage.

Supports	Type	Material
Top	Dbl 2X	Hem Fir
Base	2X	Hem Fir

System : Wall
Member Type : Stud
Building Code : IBC 2018
Design Methodology : ASD

Max Unbraced Length	Comments
1'	

Lateral Connections

Supports	Connector	Type/Model	Quantity	Connector Nailing
Top	Nails	8d (0.113" x 2 1/2") (Toe)	2	N/A
Base	Nails	8d (0.113" x 2 1/2") (Toe)	2	N/A

- Nailed connection at the top of the member is assumed to be nailed through the bottom 2x plate prior to placement of the top 2x of the double top plate assembly.

Vertical Load	Spacing	Dead (0.90)	Snow (1.15)	Comments
1 - Point (PLF)	16.00"	32.0	160.0	ROOF

Lateral Load	Location	Spacing	Wind (1.60)	Comments
1 - Uniform (PLF)	Full Length	N/A	50.6	37.9PSF * 1.33FT

- IBC Table 1604.3, footnote f: Deflection checks are performed using 42% of this lateral wind load.

Weyerhaeuser Notes

ForteWEB Software Operator	Job Notes
Samantha Taylor Anthem Structural Engineers (303) 848-8497 staylor@anthemstructural.com	

FORTEWEB

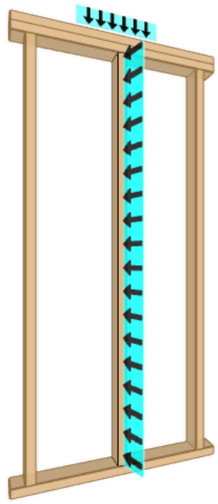
MEMBER REPORT

PASSED

EXTERIOR NON-BEARING, GRID 1E, LEVEL 3 TYP

1 piece(s) 2 x 6 HF No.2 @ 16" OC

Wall Height: 9' 7 1/8" Member Height: 9' 2 5/8" O. C. Spacing: 16.00"



Drawing is Conceptual

Design Results	Actual	Allowed	Result	LDF	Load: Combination
Slenderness	20	50	Passed (40%)	--	--
Compression (lbs)	352	6618	Passed (5%)	1.15	1.0 D + 0.75 L + 0.75 S
Plate Bearing (lbs)	352	4177	Passed (8%)	--	1.0 D + 0.75 L + 0.75 S
Lateral Reaction (lbs)	140	--	--	1.60	1.0 D + 0.6 W
Lateral Shear (lbs)	126	1320	Passed (10%)	1.60	1.0 D + 0.6 W
Lateral Moment (ft-lbs)	323 @ mid-span	1264	Passed (26%)	1.60	1.0 D + 0.6 W
Total Deflection (in)	0.13 @ mid-span	0.92	Passed (L/846)	--	1.0 D + 0.6 W
Bending/Compression	0.27	1	Passed (27%)	1.60	1.0 D + 0.6 W

- Lateral deflection criteria: Wind (L/120)
- Input axial load eccentricity for this design is 16.67% of applicable member side dimension.
- Applicable calculations are based on NDS.
- A bearing area factor of 1.25 has been applied to base plate bearing capacity.
- A 15% increase in the moment capacity has been added to account for repetitive member usage.

Supports	Type	Material
Top	Dbl 2X	Hem Fir
Base	2X	Hem Fir

System : Wall
Member Type : Stud
Building Code : IBC 2018
Design Methodology : ASD

Max Unbraced Length	Comments
1'	

Lateral Connections				
Supports	Connector	Type/Model	Quantity	Connector Nailing
Top	Nails	8d (0.113" x 2 1/2") (Toe)	2	N/A
Base	Nails	8d (0.113" x 2 1/2") (Toe)	2	N/A

- Nailed connection at the top of the member is assumed to be nailed through the bottom 2x plate prior to placement of the top 2x of the double top plate assembly.

Vertical Loads	Spacing	Dead (0.90)	Floor Live (1.00)	Snow (1.15)	Comments
1 - Point (PLF)	16.00"	32.0	-	160.0	ROOF
2 - Point (PLF)	16.00"	52.0	80.0	-	LEVEL 4

Lateral Load	Location	Spacing	Wind (1.60)	Comments
1 - Uniform (PLF)	Full Length	N/A	50.6	37.9PSF * 1.33FT

- IBC Table 1604.3, footnote f: Deflection checks are performed using 42% of this lateral wind load.

Weyerhaeuser Notes

FORTEWEB

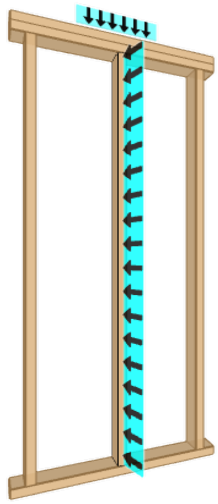
MEMBER REPORT

PASSED

EXTERIOR NON-BEARING, GRID 1E, LEVEL 2 TYP

1 piece(s) 2 x 6 HF No.2 @ 16" OC

Wall Height: 9' 7 1/8"   Member Height: 9' 2 5/8"   O. C. Spacing: 16.00"



Drawing is Conceptual

Design Results	Actual	Allowed	Result	LDF	Load: Combination
Slenderness	20	50	Passed (40%)	--	--
Compression (lbs)	501	6618	Passed (8%)	1.15	1.0 D + 0.75 L + 0.75 S
Plate Bearing (lbs)	501	4177	Passed (12%)	--	1.0 D + 0.75 L + 0.75 S
Lateral Reaction (lbs)	177	--	--	1.60	1.0 D + 0.6 W
Lateral Shear (lbs)	159	1320	Passed (12%)	1.60	1.0 D + 0.6 W
Lateral Moment (ft-lbs)	408 @ mid-span	1264	Passed (32%)	1.60	1.0 D + 0.6 W
Total Deflection (in)	0.17 @ mid-span	0.92	Passed (L/665)	--	1.0 D + 0.6 W
Bending/Compression	0.34	1	Passed (34%)	1.60	1.0 D + 0.6 W

- Lateral deflection criteria: Wind (L/120)
- Input axial load eccentricity for this design is 16.67% of applicable member side dimension.
- Applicable calculations are based on NDS.
- A bearing area factor of 1.25 has been applied to base plate bearing capacity.
- A 15% increase in the moment capacity has been added to account for repetitive member usage.

Supports	Type	Material
Top	Dbl 2X	Hem Fir
Base	2X	Hem Fir

System : Wall
Member Type : Stud
Building Code : IBC 2018
Design Methodology : ASD

Max Unbraced Length	Comments
1'	

Lateral Connections				
Supports	Connector	Type/Model	Quantity	Connector Nailing
Top	Nails	8d (0.113" x 2 1/2") (Toe)	3	N/A
Base	Nails	8d (0.113" x 2 1/2") (Toe)	3	N/A

- Nailed connection at the top of the member is assumed to be nailed through the bottom 2x plate prior to placement of the top 2x of the double top plate assembly.

Vertical Loads	Spacing	Dead (0.90)	Floor Live (1.00)	Snow (1.15)	Comments
1 - Point (PLF)	16.00"	32.0	-	160.0	ROOF
2 - Point (PLF)	16.00"	52.0	80.0	-	LEVEL 4
3 - Point (PLF)	16.00"	52.0	80.0	-	LEVEL 3

Lateral Load	Location	Spacing	Wind (1.60)	Comments
1 - Uniform (PLF)	Full Length	N/A	64.0	37.9PSF * 1.33FT

- IBC Table 1604.3, footnote f: Deflection checks are performed using 42% of this lateral wind load.

Weyerhaeuser Notes

Weyerhaeuser warrants that the sizing of its products will be in accordance with Weyerhaeuser product design criteria and published design values. Weyerhaeuser expressly disclaims any other warranties related to the software. Use of this software is not intended to circumvent the need for a design professional as determined by the authority having jurisdiction. The designer of record, builder or framer is responsible to assure that this calculation is compatible with the overall project. Accessories (Rim Board, Blocking Panels and Squash Blocks) are not designed by this software. Products manufactured at Weyerhaeuser facilities are third-party certified to sustainable forestry standards. Weyerhaeuser Engineered Lumber Products have been evaluated by ICC-ES under evaluation reports ESR-1153 and ESR-1387 and/or tested in accordance with applicable ASTM standards. For current code evaluation reports, Weyerhaeuser product literature and installation details refer to www.weyerhaeuser.com/woodproducts/document-library.

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

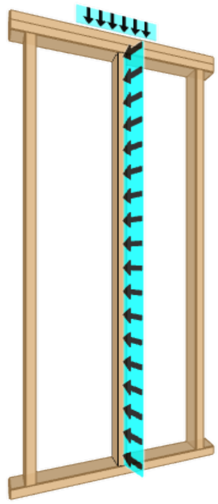
MEMBER REPORT

PASSED

EXTERIOR NON-BEARING, GRID 1E, LEVEL 1 TYP

1 piece(s) 2 x 6 HF No.2 @ 16" OC

Wall Height: 9' 7 1/8" Member Height: 9' 2 5/8" O. C. Spacing: 16.00"



Drawing is Conceptual

Design Results	Actual	Allowed	Result	LDF	Load: Combination
Slenderness	20	50	Passed (40%)	--	--
Compression (lbs)	651	6618	Passed (10%)	1.15	1.0 D + 0.75 L + 0.75 S
Plate Bearing (lbs)	651	4177	Passed (16%)	--	1.0 D + 0.75 L + 0.75 S
Lateral Reaction (lbs)	177	--	--	1.60	1.0 D + 0.6 W
Lateral Shear (lbs)	159	1320	Passed (12%)	1.60	1.0 D + 0.6 W
Lateral Moment (ft-lbs)	408 @ mid-span	1264	Passed (32%)	1.60	1.0 D + 0.6 W
Total Deflection (in)	0.17 @ mid-span	0.31	Passed (L/658)	--	1.0 D + 0.6 W
Bending/Compression	0.35	1	Passed (35%)	1.60	1.0 D + 0.6 W

- Lateral deflection criteria: Wind (L/360)
- Input axial load eccentricity for this design is 16.67% of applicable member side dimension.
- Applicable calculations are based on NDS.
- A bearing area factor of 1.25 has been applied to base plate bearing capacity.
- A 15% increase in the moment capacity has been added to account for repetitive member usage.

Supports	Type	Material
Top	Dbl 2X	Hem Fir
Base	2X	Hem Fir

System : Wall
Member Type : Stud
Building Code : IBC 2018
Design Methodology : ASD

Max Unbraced Length	Comments
1'	

Lateral Connections				
Supports	Connector	Type/Model	Quantity	Connector Nailing
Top	Nails	8d (0.113" x 2 1/2") (Toe)	3	N/A
Base	Nails	8d (0.113" x 2 1/2") (Toe)	3	N/A

- Nailed connection at the top of the member is assumed to be nailed through the bottom 2x plate prior to placement of the top 2x of the double top plate assembly.

Vertical Loads	Spacing	Dead (0.90)	Floor Live (1.00)	Snow (1.15)	Comments
1 - Point (PLF)	16.00"	32.0	-	160.0	ROOF
2 - Point (PLF)	16.00"	52.0	80.0	-	LEVEL 4
3 - Point (PLF)	16.00"	52.0	80.0	-	LEVEL 3
4 - Point (PLF)	16.00"	52.0	80.0	-	LEVEL 2

Lateral Load	Location	Spacing	Wind (1.60)	Comments
1 - Uniform (PLF)	Full Length	N/A	64.0	37.9PSF * 1.33FT

- IBC Table 1604.3, footnote f: Deflection checks are performed using 42% of this lateral wind load.

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

ForteWEB Software Operator	Job Notes
Samantha Taylor Anthem Structural Engineers (303) 848-8497 staylor@anthemstructural.com	

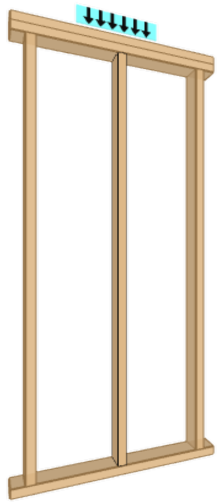
FORTEWEB

MEMBER REPORT

PASSED

INTERIOR BEARING STAGGERED 2X4, GRID 12, LEVEL 4 (FOOTING STEP)
1 piece(s) 2 x 4 HF No.2 @ 16" OC

Wall Height: 9' 7 1/8"   Member Height: 9' 2 5/8"   O. C. Spacing: 16.00"



Drawing is Conceptual

Design Results	Actual	Allowed	Result	LDF	Load: Combination
Slenderness	32	50	Passed (63%)	--	--
Compression (lbs)	1280	1926	Passed (66%)	1.15	1.0 D + 1.0 S
Plate Bearing (lbs)	1280	4102	Passed (31%)	--	1.0 D + 1.0 S
Lateral Reaction (lbs)	0	--	--	--	N/A
Lateral Shear (lbs)	0	N/A	Passed (N/A)	--	N/A
Lateral Moment (ft-lbs)	0 @ mid-span	N/A	Passed (N/A)	--	N/A
Total Deflection (in)	0.08 @ mid-span	0.92	Passed (L/1315)	--	1.0 D + 1.0 S
Bending/Compression	0.89	1	Passed (89%)	1.15	1.0 D + 1.0 S

- Lateral deflection criteria: Wind (L/120)
- Input axial load eccentricity for this design is 16.67% of applicable member side dimension.
- Applicable calculations are based on NDS.
- A bearing area factor of 1.25 has been applied to base plate bearing capacity.
- A 15% increase in the moment capacity has been added to account for repetitive member usage.

Supports	Type	Material
Top	Dbl 2X	Douglas Fir-Larch
Base	2X	Douglas Fir-Larch

System : Wall
Member Type : Stud
Building Code : IBC 2018
Design Methodology : ASD

Max Unbraced Length	Comments
1'	

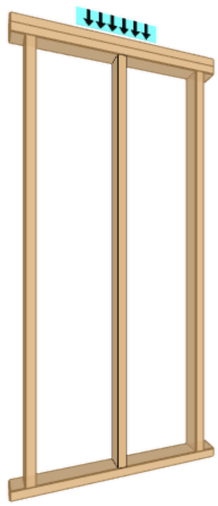
Vertical Load	Spacing	Dead (0.90)	Snow (1.15)	Comments
1 - Point (PLF)	16.00"	160.0	800.0	ROOF

Weyerhaeuser Notes

INTERIOR BEARING STAGGERED 2X4, GRID 12, LEVEL 3 (FOOTING STEP)

2 piece(s) 2 x 4 HF No.2 @ 16" OC

Wall Height: 9' 7 1/8" Member Height: 9' 2 5/8" O. C. Spacing: 16.00"



Drawing is Conceptual

Design Results	Actual	Allowed	Result	LDF	Load: Combination
Slenderness	32	50	Passed (63%)	--	--
Compression (lbs)	1760	3852	Passed (46%)	1.15	1.0 D + 0.75 L + 0.75 S
Plate Bearing (lbs)	1760	7383	Passed (24%)	--	1.0 D + 0.75 L + 0.75 S
Lateral Reaction (lbs)	0	--	--	--	N/A
Lateral Shear (lbs)	0	N/A	Passed (N/A)	--	N/A
Lateral Moment (ft-lbs)	0 @ mid-span	N/A	Passed (N/A)	--	N/A
Total Deflection (in)	0.06 @ mid-span	0.92	Passed (L/1913)	--	1.0 D + 0.75 L + 0.75 S
Bending/Compression	0.40	1	Passed (40%)	1.15	1.0 D + 0.75 L + 0.75 S

- Lateral deflection criteria: Wind (L/120)
- Input axial load eccentricity for this design is 16.67% of applicable member side dimension.
- Applicable calculations are based on NDS.
- A bearing area factor of 1.125 has been applied to base plate bearing capacity.
- The column stability factor (Kf = 0.6) applied to this design assumes nailed built-up columns per NDS section 15.3.3. For Weyerhaeuser ELP products refer to the U.S. Wall Guide for multiple-member connection requirements.
- A 15% increase in the moment capacity has been added to account for repetitive member usage.

Supports	Type	Material
Top	Dbl 2X	Douglas Fir-Larch
Base	2X	Douglas Fir-Larch

System : Wall
Member Type : Stud
Building Code : IBC 2018
Design Methodology : ASD

Max Unbraced Length	Comments
1'	

Vertical Loads	Spacing	Dead (0.90)	Floor Live (1.00)	Snow (1.15)	Comments
1 - Point (PLF)	16.00"	160.0	-	800.0	ROOF
2 - Point (PLF)	16.00"	260.0	400.0	-	LEVEL 4

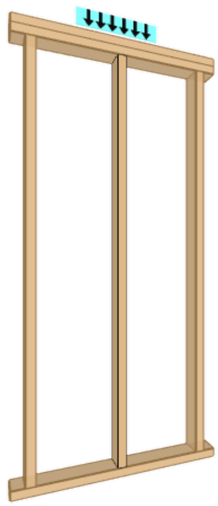
Weyerhaeuser Notes

ForteWEB Software Operator	Job Notes
Samantha Taylor Anthem Structural Engineers (303) 848-8497 staylor@anthemstructural.com	

INTERIOR BEARING STAGGERED 2X4, GRID 12, LEVEL 3 (FOOTING STEP - ROOF DECK)

2 piece(s) 2 x 4 HF No.2 @ 16" OC

Wall Height: 9' 7 1/8" Member Height: 9' 2 5/8" O. C. Spacing: 16.00"



Drawing is Conceptual

Design Results	Actual	Allowed	Result	LDF	Load: Combination
Slenderness	32	50	Passed (63%)	--	--
Compression (lbs)	2647	3852	Passed (69%)	1.15	1.0 D + 0.75 L + 0.75 S
Plate Bearing (lbs)	2647	7383	Passed (36%)	--	1.0 D + 0.75 L + 0.75 S
Lateral Reaction (lbs)	0	--	--	--	N/A
Lateral Shear (lbs)	0	N/A	Passed (N/A)	--	N/A
Lateral Moment (ft-lbs)	0 @ mid-span	N/A	Passed (N/A)	--	N/A
Total Deflection (in)	0.09 @ mid-span	0.92	Passed (L/1272)	--	1.0 D + 0.75 L + 0.75 S
Bending/Compression	0.97	1	Passed (97%)	1.15	1.0 D + 0.75 L + 0.75 S

- Lateral deflection criteria: Wind (L/120)
- Input axial load eccentricity for this design is 16.67% of applicable member side dimension.
- Applicable calculations are based on NDS.
- A bearing area factor of 1.125 has been applied to base plate bearing capacity.
- The column stability factor (Kf = 0.6) applied to this design assumes nailed built-up columns per NDS section 15.3.3. For Weyerhaeuser ELP products refer to the U.S. Wall Guide for multiple-member connection requirements.
- A 15% increase in the moment capacity has been added to account for repetitive member usage.

Supports	Type	Material
Top	Dbl 2X	Douglas Fir-Larch
Base	2X	Douglas Fir-Larch

System : Wall
Member Type : Stud
Building Code : IBC 2018
Design Methodology : ASD

Max Unbraced Length	Comments
1'	

Vertical Loads	Spacing	Dead (0.90)	Floor Live (1.00)	Snow (1.15)	Comments
1 - Point (PLF)	16.00"	-	-	-	ROOF
2 - Point (PLF)	16.00"	260.0	600.0	1700.0	LEVEL 4 (DECK)

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
Samantha Taylor Anthem Structural Engineers (303) 848-8497 staylor@anthemstructural.com	

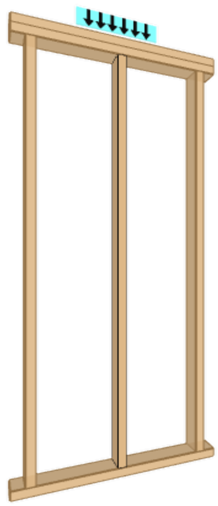
FORTEWEB

MEMBER REPORT

PASSED

INTERIOR BEARING STAGGERED 2X4, GRID 12, LEVEL 2 (FOOTING STEP)
2 piece(s) 2 x 4 HF No.2 @ 16" OC

Wall Height: 9' 7 1/8" Member Height: 9' 2 5/8" O. C. Spacing: 16.00"



Drawing is Conceptual

Design Results	Actual	Allowed	Result	LDF	Load: Combination
Slenderness	32	50	Passed (63%)	--	--
Compression (lbs)	2507	3852	Passed (65%)	1.15	1.0 D + 0.75 L + 0.75 S
Plate Bearing (lbs)	2507	7383	Passed (34%)	--	1.0 D + 0.75 L + 0.75 S
Lateral Reaction (lbs)	0	--	--	--	N/A
Lateral Shear (lbs)	0	N/A	Passed (N/A)	--	N/A
Lateral Moment (ft-lbs)	0 @ mid-span	N/A	Passed (N/A)	--	N/A
Total Deflection (in)	0.08 @ mid-span	0.92	Passed (L/1343)	--	1.0 D + 0.75 L + 0.75 S
Bending/Compression	0.85	1	Passed (85%)	1.15	1.0 D + 0.75 L + 0.75 S

- Lateral deflection criteria: Wind (L/120)
- Input axial load eccentricity for this design is 16.67% of applicable member side dimension.
- Applicable calculations are based on NDS.
- A bearing area factor of 1.125 has been applied to base plate bearing capacity.
- The column stability factor (Kf = 0.6) applied to this design assumes nailed built-up columns per NDS section 15.3.3. For Weyerhaeuser ELP products refer to the U.S. Wall Guide for multiple-member connection requirements.
- A 15% increase in the moment capacity has been added to account for repetitive member usage.

Supports	Type	Material
Top	Dbl 2X	Douglas Fir-Larch
Base	2X	Douglas Fir-Larch

System : Wall
Member Type : Stud
Building Code : IBC 2018
Design Methodology : ASD

Max Unbraced Length	Comments
1'	

Vertical Loads	Spacing	Dead (0.90)	Floor Live (1.00)	Snow (1.15)	Comments
1 - Point (PLF)	16.00"	160.0	-	800.0	ROOF
2 - Point (PLF)	16.00"	260.0	400.0	-	LEVEL 4
3 - Point (PLF)	16.00"	260.0	400.0	-	LEVEL 3

ForteWEB Software Operator	Job Notes
Samantha Taylor Anthem Structural Engineers (303) 848-8497 staylor@anthemstructural.com	

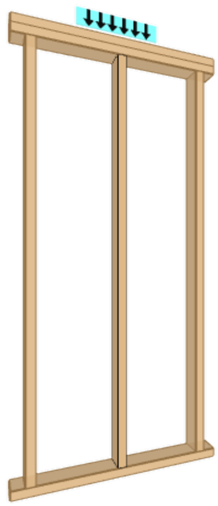
FORTEWEB

MEMBER REPORT

PASSED

INTERIOR BEARING STAGGERED 2X4, GRID 12, LEVEL 1 (FOOTING STEP)
3 piece(s) 2 x 4 HF No.2 @ 16" OC

Wall Height: 9' 7 1/8" | Member Height: 9' 2 5/8" | O. C. Spacing: 16.00"



Drawing is Conceptual

Design Results	Actual	Allowed	Result	LDF	Load: Combination
Slenderness	32	50	Passed (63%)	--	--
Compression (lbs)	3253	5777	Passed (56%)	1.15	1.0 D + 0.75 L + 0.75 S
Plate Bearing (lbs)	3253	10664	Passed (31%)	--	1.0 D + 0.75 L + 0.75 S
Lateral Reaction (lbs)	0	--	--	--	N/A
Lateral Shear (lbs)	0	N/A	Passed (N/A)	--	N/A
Lateral Moment (ft-lbs)	0 @ mid-span	N/A	Passed (N/A)	--	N/A
Total Deflection (in)	0.07 @ mid-span	0.92	Passed (L/1552)	--	1.0 D + 0.75 L + 0.75 S
Bending/Compression	0.61	1	Passed (61%)	1.15	1.0 D + 0.75 L + 0.75 S

- Lateral deflection criteria: Wind (L/120)
- Input axial load eccentricity for this design is 16.67% of applicable member side dimension.
- Applicable calculations are based on NDS.
- A bearing area factor of 1.083333 has been applied to base plate bearing capacity.
- The column stability factor (Kf = 0.6) applied to this design assumes nailed built-up columns per NDS section 15.3.3. For Weyerhaeuser ELP products refer to the U.S. Wall Guide for multiple-member connection requirements.
- A 15% increase in the moment capacity has been added to account for repetitive member usage.

Supports	Type	Material
Top	Dbl 2X	Douglas Fir-Larch
Base	2X	Douglas Fir-Larch

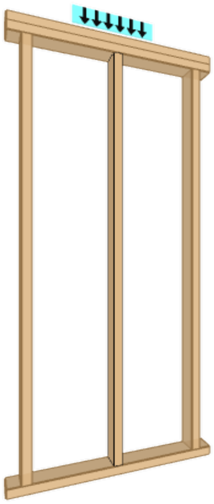
System : Wall
Member Type : Stud
Building Code : IBC 2018
Design Methodology : ASD

Max Unbraced Length	Comments
1'	

Vertical Loads	Spacing	Dead (0.90)	Floor Live (1.00)	Snow (1.15)	Comments
1 - Point (PLF)	16.00"	160.0	-	800.0	ROOF
2 - Point (PLF)	16.00"	260.0	400.0	-	LEVEL 4
3 - Point (PLF)	16.00"	260.0	400.0	-	LEVEL 3
4 - Point (PLF)	16.00"	260.0	400.0	-	LEVEL 2

Weyerhaeuser Notes

INTERIOR BEARING STAGGERED 2X4, GRID 12, LEVEL 1 (FOOTING STEP - ROOF DECK)
3 piece(s) 2 x 4 HF No.2 @ 16" OC



Drawing is Conceptual

Wall Height: 9' 7 1/8" Member Height: 9' 2 5/8" O. C. Spacing: 16.00"

Design Results	Actual	Allowed	Result	LDF	Load: Combination
Slenderness	32	50	Passed (63%)	--	--
Compression (lbs)	4140	5777	Passed (72%)	1.15	1.0 D + 0.75 L + 0.75 S
Plate Bearing (lbs)	4140	10664	Passed (39%)	--	1.0 D + 0.75 L + 0.75 S
Lateral Reaction (lbs)	0	--	--	--	N/A
Lateral Shear (lbs)	0	N/A	Passed (N/A)	--	N/A
Lateral Moment (ft-lbs)	0 @ mid-span	N/A	Passed (N/A)	--	N/A
Total Deflection (in)	0.09 @ mid-span	0.92	Passed (L/1220)	--	1.0 D + 0.75 L + 0.75 S
Bending/Compression	1.08	1	Failed (108%)	1.15	1.0 D + 0.75 L + 0.75 S

- Lateral deflection criteria: Wind (L/120)
- Input axial load eccentricity for this design is 16.67% of applicable member side dimension.
- Applicable calculations are based on NDS.
- A bearing area factor of 1.083333 has been applied to base plate bearing capacity.
- The column stability factor (Kf = 0.6) applied to this design assumes nailed built-up columns per NDS section 15.3.3. For Weyerhaeuser ELP products refer to the U.S. Wall Guide for multiple-member connection requirements.
- A 15% increase in the moment capacity has been added to account for repetitive member usage.

Supports	Type	Material
Top	Dbl 2X	Douglas Fir-Larch
Base	2X	Douglas Fir-Larch

System : Wall
Member Type : Stud
Building Code : IBC 2018
Design Methodology : ASD

Max Unbraced Length	Comments
1'	

Vertical Loads	Spacing	Dead (0.90)	Floor Live (1.00)	Snow (1.15)	Comments
1 - Point (PLF)	16.00"	-	-	-	ROOF
2 - Point (PLF)	16.00"	260.0	600.0	1700.0	LEVEL 4 (DECK)
3 - Point (PLF)	16.00"	260.0	400.0	-	LEVEL 3
4 - Point (PLF)	16.00"	260.0	400.0	-	LEVEL 2

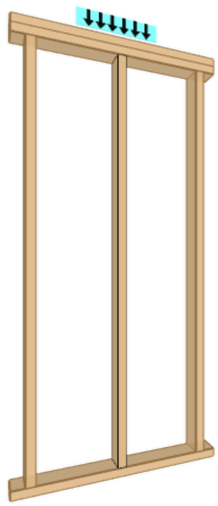
ForteWEB Software Operator	Job Notes
Samantha Taylor Anthem Structural Engineers (303) 848-8497 staylor@anthemstructural.com	

MEMBER REPORT

PASSED

INTERIOR BEARING STAGGERED 2X4, GRID 17, LEVEL 4
1 piece(s) 2 x 4 HF No.2 @ 8" OC

Wall Height: 9' 7 1/8" | Member Height: 9' 2 5/8" | O. C. Spacing: 8.00"



Drawing is Conceptual

Design Results	Actual	Allowed	Result	LDF	Load: Combination
Slenderness	32	50	Passed (63%)	--	--
Compression (lbs)	1152	1926	Passed (60%)	1.15	1.0 D + 1.0 S
Plate Bearing (lbs)	1152	4102	Passed (28%)	--	1.0 D + 1.0 S
Lateral Reaction (lbs)	0	--	--	--	N/A
Lateral Shear (lbs)	0	N/A	Passed (N/A)	--	N/A
Lateral Moment (ft-lbs)	0 @ mid-span	N/A	Passed (N/A)	--	N/A
Total Deflection (in)	0.08 @ mid-span	0.92	Passed (L/1461)	--	1.0 D + 1.0 S
Bending/Compression	0.70	1	Passed (70%)	1.15	1.0 D + 1.0 S

- Lateral deflection criteria: Wind (L/120)
- Input axial load eccentricity for this design is 16.67% of applicable member side dimension.
- Applicable calculations are based on NDS.
- A bearing area factor of 1.25 has been applied to base plate bearing capacity.
- A 15% increase in the moment capacity has been added to account for repetitive member usage.

Supports	Type	Material
Top	Dbl 2X	Douglas Fir-Larch
Base	2X	Douglas Fir-Larch

System : Wall
Member Type : Stud
Building Code : IBC 2018
Design Methodology : ASD

Max Unbraced Length	Comments
1'	

Vertical Load	Spacing	Dead (0.90)	Snow (1.15)	Comments
1 - Point (PLF)	8.00"	288.0	1440.0	ROOF

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

FORTEWEB™

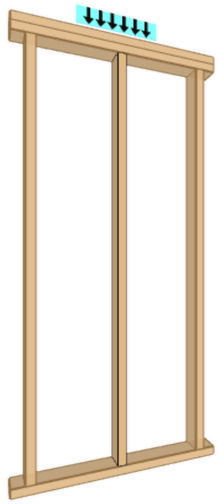
MEMBER REPORT

PASSED

INTERIOR BEARING STAGGERED 2X4, GRID 17, LEVEL 3

2 piece(s) 2 x 4 HF No.2 @ 12" OC

Wall Height: 9' 7 1/8" | Member Height: 9' 2 5/8" | O. C. Spacing: 12.00"



Drawing is Conceptual

Design Results	Actual	Allowed	Result	LDF	Load: Combination
Slenderness	32	50	Passed (63%)	--	--
Compression (lbs)	2376	3852	Passed (62%)	1.15	1.0 D + 0.75 L + 0.75 S
Plate Bearing (lbs)	2376	7383	Passed (32%)	--	1.0 D + 0.75 L + 0.75 S
Lateral Reaction (lbs)	0	--	--	--	N/A
Lateral Shear (lbs)	0	N/A	Passed (N/A)	--	N/A
Lateral Moment (ft-lbs)	0 @ mid-span	N/A	Passed (N/A)	--	N/A
Total Deflection (in)	0.08 @ mid-span	0.92	Passed (L/1417)	--	1.0 D + 0.75 L + 0.75 S
Bending/Compression	0.75	1	Passed (75%)	1.15	1.0 D + 0.75 L + 0.75 S

- Lateral deflection criteria: Wind (L/120)
- Input axial load eccentricity for this design is 16.67% of applicable member side dimension.
- Applicable calculations are based on NDS.
- A bearing area factor of 1.125 has been applied to base plate bearing capacity.
- The column stability factor (Kf = 0.6) applied to this design assumes nailed built-up columns per NDS section 15.3.3. For Weyerhaeuser ELP products refer to the U.S. Wall Guide for multiple-member connection requirements.
- A 15% increase in the moment capacity has been added to account for repetitive member usage.

Supports	Type	Material
Top	Dbl 2X	Douglas Fir-Larch
Base	2X	Douglas Fir-Larch

System : Wall
Member Type : Stud
Building Code : IBC 2018
Design Methodology : ASD

Max Unbraced Length	Comments
1'	

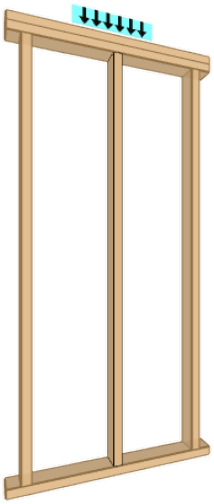
Vertical Loads	Spacing	Dead (0.90)	Floor Live (1.00)	Snow (1.15)	Comments
1 - Point (PLF)	12.00"	288.0	-	1440.0	ROOF
2 - Point (PLF)	12.00"	468.0	720.0	-	LEVEL 4

Weyerhaeuser Notes

ForteWEB Software Operator	Job Notes
Samantha Taylor Anthem Structural Engineers (303) 848-8497 staylor@anthemstructural.com	

INTERIOR BEARING STAGGERED 2X4, GRID 17, LEVEL 2
2 piece(s) 2 x 4 HF No.2 @ 8" OC

Wall Height: 9' 7 1/8" | Member Height: 9' 2 5/8" | O. C. Spacing: 8.00"



Drawing is Conceptual

Design Results	Actual	Allowed	Result	LDF	Load: Combination
Slenderness	32	50	Passed (63%)	--	--
Compression (lbs)	2256	3852	Passed (59%)	1.15	1.0 D + 0.75 L + 0.75 S
Plate Bearing (lbs)	2256	7383	Passed (31%)	--	1.0 D + 0.75 L + 0.75 S
Lateral Reaction (lbs)	0	--	--	--	N/A
Lateral Shear (lbs)	0	N/A	Passed (N/A)	--	N/A
Lateral Moment (ft-lbs)	0 @ mid-span	N/A	Passed (N/A)	--	N/A
Total Deflection (in)	0.07 @ mid-span	0.92	Passed (L/1492)	--	1.0 D + 0.75 L + 0.75 S
Bending/Compression	0.67	1	Passed (67%)	1.15	1.0 D + 0.75 L + 0.75 S

- Lateral deflection criteria: Wind (L/120)
- Input axial load eccentricity for this design is 16.67% of applicable member side dimension.
- Applicable calculations are based on NDS.
- A bearing area factor of 1.125 has been applied to base plate bearing capacity.
- The column stability factor (Kf = 0.6) applied to this design assumes nailed built-up columns per NDS section 15.3.3. For Weyerhaeuser ELP products refer to the U.S. Wall Guide for multiple-member connection requirements.
- A 15% increase in the moment capacity has been added to account for repetitive member usage.

Supports	Type	Material
Top	Dbl 2X	Douglas Fir-Larch
Base	2X	Douglas Fir-Larch

System : Wall
Member Type : Stud
Building Code : IBC 2018
Design Methodology : ASD

Max Unbraced Length	Comments
1'	

Vertical Loads	Spacing	Dead (0.90)	Floor Live (1.00)	Snow (1.15)	Comments
1 - Point (PLF)	8.00"	288.0	-	1440.0	ROOF
2 - Point (PLF)	8.00"	468.0	720.0	-	LEVEL 4
3 - Point (PLF)	8.00"	468.0	720.0	-	LEVEL 3

Weyerhaeuser Notes

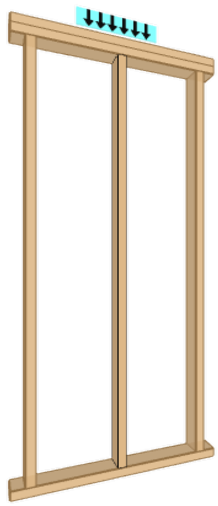
FORTEWEB

MEMBER REPORT

PASSED

INTERIOR BEARING STAGGERED 2X4, GRID 17, LEVEL 1
3 piece(s) 2 x 4 HF No.2 @ 8" OC

Wall Height: 9' 7 1/8" | Member Height: 9' 2 5/8" | O. C. Spacing: 8.00"



Drawing is Conceptual

Design Results	Actual	Allowed	Result	LDF	Load: Combination
Slenderness	32	50	Passed (63%)	--	--
Compression (lbs)	2928	5777	Passed (51%)	1.15	1.0 D + 0.75 L + 0.75 S
Plate Bearing (lbs)	2928	10664	Passed (27%)	--	1.0 D + 0.75 L + 0.75 S
Lateral Reaction (lbs)	0	--	--	--	N/A
Lateral Shear (lbs)	0	N/A	Passed (N/A)	--	N/A
Lateral Moment (ft-lbs)	0 @ mid-span	N/A	Passed (N/A)	--	N/A
Total Deflection (in)	0.06 @ mid-span	0.92	Passed (L/1724)	--	1.0 D + 0.75 L + 0.75 S
Bending/Compression	0.49	1	Passed (49%)	1.15	1.0 D + 0.75 L + 0.75 S

- Lateral deflection criteria: Wind (L/120)
- Input axial load eccentricity for this design is 16.67% of applicable member side dimension.
- Applicable calculations are based on NDS.
- A bearing area factor of 1.083333 has been applied to base plate bearing capacity.
- The column stability factor (Kf = 0.6) applied to this design assumes nailed built-up columns per NDS section 15.3.3. For Weyerhaeuser ELP products refer to the U.S. Wall Guide for multiple-member connection requirements.
- A 15% increase in the moment capacity has been added to account for repetitive member usage.

Supports	Type	Material
Top	Dbl 2X	Douglas Fir-Larch
Base	2X	Douglas Fir-Larch

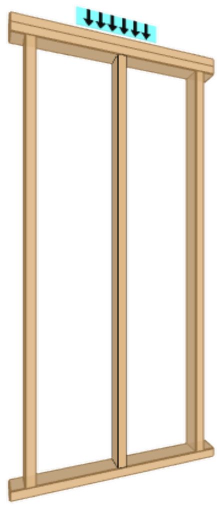
System : Wall
Member Type : Stud
Building Code : IBC 2018
Design Methodology : ASD

Max Unbraced Length	Comments
1'	

Vertical Loads	Spacing	Dead (0.90)	Floor Live (1.00)	Snow (1.15)	Comments
1 - Point (PLF)	8.00"	288.0	-	1440.0	ROOF
2 - Point (PLF)	8.00"	468.0	720.0	-	LEVEL 4
3 - Point (PLF)	8.00"	468.0	720.0	-	LEVEL 3
4 - Point (PLF)	8.00"	468.0	720.0	-	LEVEL 2

INTERIOR BEARING STAGGERED 2X4, GRID 17, LEVEL 1 (ROOF DECK)
3 piece(s) 2 x 4 HF No.2 @ 8" OC

Wall Height: 9' 7 1/8" Member Height: 9' 2 5/8" O. C. Spacing: 8.00"



Drawing is Conceptual

Design Results	Actual	Allowed	Result	LDF	Load: Combination
Slenderness	32	50	Passed (63%)	--	--
Compression (lbs)	3726	5777	Passed (64%)	1.15	1.0 D + 0.75 L + 0.75 S
Plate Bearing (lbs)	3726	10664	Passed (35%)	--	1.0 D + 0.75 L + 0.75 S
Lateral Reaction (lbs)	0	--	--	--	N/A
Lateral Shear (lbs)	0	N/A	Passed (N/A)	--	N/A
Lateral Moment (ft-lbs)	0 @ mid-span	N/A	Passed (N/A)	--	N/A
Total Deflection (in)	0.08 @ mid-span	0.92	Passed (L/1355)	--	1.0 D + 0.75 L + 0.75 S
Bending/Compression	0.83	1	Passed (83%)	1.15	1.0 D + 0.75 L + 0.75 S

- Lateral deflection criteria: Wind (L/120)
- Input axial load eccentricity for this design is 16.67% of applicable member side dimension.
- Applicable calculations are based on NDS.
- A bearing area factor of 1.083333 has been applied to base plate bearing capacity.
- The column stability factor (Kf = 0.6) applied to this design assumes nailed built-up columns per NDS section 15.3.3. For Weyerhaeuser ELP products refer to the U.S. Wall Guide for multiple-member connection requirements.
- A 15% increase in the moment capacity has been added to account for repetitive member usage.

Supports	Type	Material
Top	Dbl 2X	Douglas Fir-Larch
Base	2X	Douglas Fir-Larch

System : Wall
Member Type : Stud
Building Code : IBC 2018
Design Methodology : ASD

Max Unbraced Length	Comments
1'	

Vertical Loads	Spacing	Dead (0.90)	Floor Live (1.00)	Snow (1.15)	Comments
1 - Point (PLF)	8.00"	-	-	-	ROOF
2 - Point (PLF)	8.00"	468.0	1080.0	3060.0	LEVEL 4 (DECK)
3 - Point (PLF)	8.00"	468.0	720.0	-	LEVEL 3
4 - Point (PLF)	8.00"	468.0	720.0	-	LEVEL 2

Weyerhaeuser Notes

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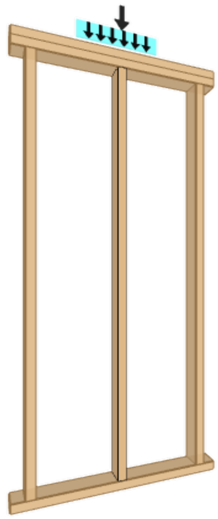
MEMBER REPORT

PASSED

INTERIOR BEARING STAGGERED 2X4, STAIR WALL, LEVEL 1
1 piece(s) 2 x 4 HF No.2 @ 16" OC

Wall Height: 10' 3"   Member Height: 9' 10 1/2"   O. C. Spacing: 16.00"



Drawing is Conceptual

Design Results	Actual	Allowed	Result	LDF	Load: Combination
Slenderness	34	50	Passed (68%)	--	--
Compression (lbs)	1050	1678	Passed (63%)	1.00	1.0 D + 1.0 L
Plate Bearing (lbs)	1050	2658	Passed (40%)	--	1.0 D + 1.0 L
Lateral Reaction (lbs)	0	--	--	--	N/A
Lateral Shear (lbs)	0	N/A	Passed (N/A)	--	N/A
Lateral Moment (ft-lbs)	0 @ mid-span	N/A	Passed (N/A)	--	N/A
Total Deflection (in)	0.08 @ mid-span	0.99	Passed (L/1496)	--	1.0 D + 1.0 L
Bending/Compression	0.78	1	Passed (78%)	1.00	1.0 D + 1.0 L

- Lateral deflection criteria: Wind (L/120)
- Input axial load eccentricity for this design is 16.67% of applicable member side dimension.
- Applicable calculations are based on NDS.
- A bearing area factor of 1.25 has been applied to base plate bearing capacity.
- A 15% increase in the moment capacity has been added to account for repetitive member usage.

Supports	Type	Material
Top	Dbl 2X	Hem Fir
Base	2X	Hem Fir

System : Wall
Member Type : Stud
Building Code : IBC 2018
Design Methodology : ASD

Max Unbraced Length	Comments
1'	

Vertical Loads	Spacing	Dead (0.90)	Floor Live (1.00)	Comments
1 - Point (PLF)	16.00"	130.0	320.0	LEVEL 2
2 - Point (lb)	N/A	130	320	LEVEL 3

Weyerhaeuser Notes

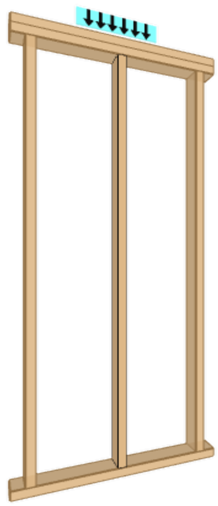
ForteWEB Software Operator	Job Notes
Samantha Taylor Anthem Structural Engineers (303) 848-8497 staylor@anthemstructural.com	

FORTEWEB

MEMBER REPORT

PASSED

INTERIOR BEARING 2X6, GRID 17, LEVEL 4
1 piece(s) 2 x 6 HF No.2 @ 16" OC



Drawing is Conceptual

Wall Height: 9' 7 1/8" | Member Height: 9' 2 5/8" | O. C. Spacing: 16.00"

Design Results	Actual	Allowed	Result	LDF	Load: Combination
Slenderness	20	50	Passed (40%)	--	--
Compression (lbs)	2304	6618	Passed (35%)	1.15	1.0 D + 1.0 S
Plate Bearing (lbs)	2304	6445	Passed (36%)	--	1.0 D + 1.0 S
Lateral Reaction (lbs)	0	--	--	--	N/A
Lateral Shear (lbs)	0	N/A	Passed (N/A)	--	N/A
Lateral Moment (ft-lbs)	0 @ mid-span	N/A	Passed (N/A)	--	N/A
Total Deflection (in)	0.06 @ mid-span	0.92	Passed (L/1804)	--	1.0 D + 1.0 S
Bending/Compression	0.41	1	Passed (41%)	1.15	1.0 D + 1.0 S

- Lateral deflection criteria: Wind (L/120)
- Input axial load eccentricity for this design is 16.67% of applicable member side dimension.
- Applicable calculations are based on NDS.
- A bearing area factor of 1.25 has been applied to base plate bearing capacity.
- A 15% increase in the moment capacity has been added to account for repetitive member usage.

Supports	Type	Material
Top	Dbl 2X	Douglas Fir-Larch
Base	2X	Douglas Fir-Larch

System : Wall
Member Type : Stud
Building Code : IBC 2018
Design Methodology : ASD

Max Unbraced Length	Comments
1'	

Vertical Load	Spacing	Dead (0.90)	Snow (1.15)	Comments
1 - Point (PLF)	16.00"	288.0	1440.0	ROOF

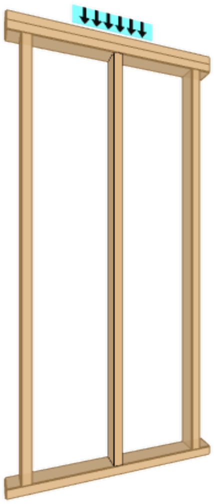
Weyerhaeuser Notes

ForteWEB Software Operator	Job Notes
Samantha Taylor Anthem Structural Engineers (303) 848-8497 staylor@anthemstructural.com	

Weyerhaeuser

INTERIOR BEARING 2X6, GRID 17, LEVEL 3
1 piece(s) 2 x 6 HF No.2 @ 16" OC

Wall Height: 9' 7 1/8" Member Height: 9' 2 5/8" O. C. Spacing: 16.00"



Drawing is Conceptual

Design Results	Actual	Allowed	Result	LDF	Load: Combination
Slenderness	20	50	Passed (40%)	--	--
Compression (lbs)	3168	6618	Passed (48%)	1.15	1.0 D + 0.75 L + 0.75 S
Plate Bearing (lbs)	3168	6445	Passed (49%)	--	1.0 D + 0.75 L + 0.75 S
Lateral Reaction (lbs)	0	--	--	--	N/A
Lateral Shear (lbs)	0	N/A	Passed (N/A)	--	N/A
Lateral Moment (ft-lbs)	0 @ mid-span	N/A	Passed (N/A)	--	N/A
Total Deflection (in)	0.08 @ mid-span	0.92	Passed (L/1312)	--	1.0 D + 0.75 L + 0.75 S
Bending/Compression	0.71	1	Passed (71%)	1.15	1.0 D + 0.75 L + 0.75 S

- Lateral deflection criteria: Wind (L/120)
- Input axial load eccentricity for this design is 16.67% of applicable member side dimension.
- Applicable calculations are based on NDS.
- A bearing area factor of 1.25 has been applied to base plate bearing capacity.
- A 15% increase in the moment capacity has been added to account for repetitive member usage.

Supports	Type	Material
Top	Dbl 2X	Douglas Fir-Larch
Base	2X	Douglas Fir-Larch

System : Wall
Member Type : Stud
Building Code : IBC 2018
Design Methodology : ASD

Max Unbraced Length	Comments
1'	

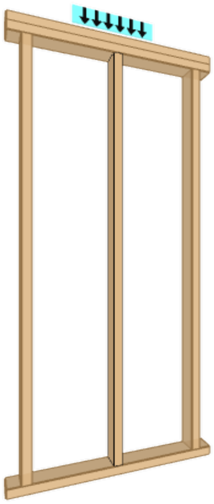
Vertical Loads	Spacing	Dead (0.90)	Floor Live (1.00)	Snow (1.15)	Comments
1 - Point (PLF)	16.00"	288.0	-	1440.0	ROOF
2 - Point (PLF)	16.00"	468.0	720.0	-	LEVEL 4

Weyerhaeuser Notes

ForteWEB Software Operator	Job Notes
Samantha Taylor Anthem Structural Engineers (303) 848-8497 staylor@anthemstructural.com	

INTERIOR BEARING 2X6, GRID 17, LEVEL 3 (ROOF DECK)
2 piece(s) 2 x 6 HF No.2 @ 16" OC

Wall Height: 9' 7 1/8" | Member Height: 9' 2 5/8" | O. C. Spacing: 16.00"



Drawing is Conceptual

Design Results	Actual	Allowed	Result	LDF	Load: Combination
Slenderness	20	50	Passed (40%)	--	--
Compression (lbs)	4764	13236	Passed (36%)	1.15	1.0 D + 0.75 L + 0.75 S
Plate Bearing (lbs)	4764	11602	Passed (41%)	--	1.0 D + 0.75 L + 0.75 S
Lateral Reaction (lbs)	0	--	--	--	N/A
Lateral Shear (lbs)	0	N/A	Passed (N/A)	--	N/A
Lateral Moment (ft-lbs)	0 @ mid-span	N/A	Passed (N/A)	--	N/A
Total Deflection (in)	0.06 @ mid-span	0.92	Passed (L/1745)	--	1.0 D + 0.75 L + 0.75 S
Bending/Compression	0.43	1	Passed (43%)	1.15	1.0 D + 0.75 L + 0.75 S

- Lateral deflection criteria: Wind (L/120)
- Input axial load eccentricity for this design is 16.67% of applicable member side dimension.
- Applicable calculations are based on NDS.
- A bearing area factor of 1.125 has been applied to base plate bearing capacity.
- The column stability factor (Kf = 0.6) applied to this design assumes nailed built-up columns per NDS section 15.3.3. For Weyerhaeuser ELP products refer to the U.S. Wall Guide for multiple-member connection requirements.
- A 15% increase in the moment capacity has been added to account for repetitive member usage.

Supports	Type	Material
Top	Dbl 2X	Douglas Fir-Larch
Base	2X	Douglas Fir-Larch

System : Wall
Member Type : Stud
Building Code : IBC 2018
Design Methodology : ASD

Max Unbraced Length	Comments
1'	

Vertical Loads	Spacing	Dead (0.90)	Floor Live (1.00)	Snow (1.15)	Comments
1 - Point (PLF)	16.00"	-	-	-	ROOF
2 - Point (PLF)	16.00"	468.0	1080.0	3060.0	LEVEL 4 (DECK)

Weyerhaeuser Notes

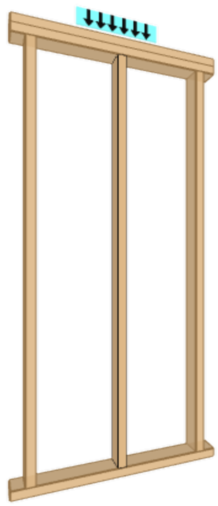
ForteWEB Software Operator	Job Notes
Samantha Taylor Anthem Structural Engineers (303) 848-8497 staylor@anthemstructural.com	

MEMBER REPORT

PASSED

INTERIOR BEARING 2X6, GRID 17, LEVEL 2
2 piece(s) 2 x 6 HF No.2 @ 16" OC

Wall Height: 9' 7 1/8" Member Height: 9' 2 5/8" O. C. Spacing: 16.00"



Drawing is Conceptual

Design Results	Actual	Allowed	Result	LDF	Load: Combination
Slenderness	20	50	Passed (40%)	--	--
Compression (lbs)	4512	13236	Passed (34%)	1.15	1.0 D + 0.75 L + 0.75 S
Plate Bearing (lbs)	4512	11602	Passed (39%)	--	1.0 D + 0.75 L + 0.75 S
Lateral Reaction (lbs)	0	--	--	--	N/A
Lateral Shear (lbs)	0	N/A	Passed (N/A)	--	N/A
Lateral Moment (ft-lbs)	0 @ mid-span	N/A	Passed (N/A)	--	N/A
Total Deflection (in)	0.06 @ mid-span	0.92	Passed (L/1842)	--	1.0 D + 0.75 L + 0.75 S
Bending/Compression	0.40	1	Passed (40%)	1.15	1.0 D + 0.75 L + 0.75 S

- Lateral deflection criteria: Wind (L/120)
- Input axial load eccentricity for this design is 16.67% of applicable member side dimension.
- Applicable calculations are based on NDS.
- A bearing area factor of 1.125 has been applied to base plate bearing capacity.
- The column stability factor (Kf = 0.6) applied to this design assumes nailed built-up columns per NDS section 15.3.3. For Weyerhaeuser ELP products refer to the U.S. Wall Guide for multiple-member connection requirements.
- A 15% increase in the moment capacity has been added to account for repetitive member usage.

Supports	Type	Material
Top	Dbl 2X	Douglas Fir-Larch
Base	2X	Douglas Fir-Larch

System : Wall
Member Type : Stud
Building Code : IBC 2018
Design Methodology : ASD

Max Unbraced Length	Comments
1'	

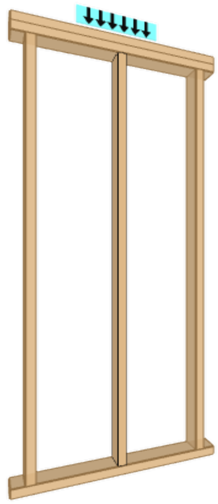
Vertical Loads	Spacing	Dead (0.90)	Floor Live (1.00)	Snow (1.15)	Comments
1 - Point (PLF)	16.00"	288.0	-	1440.0	ROOF
2 - Point (PLF)	16.00"	468.0	720.0	-	LEVEL 4
3 - Point (PLF)	16.00"	468.0	720.0	-	LEVEL 3

Weyerhaeuser Notes

ForteWEB Software Operator	Job Notes
Samantha Taylor Anthem Structural Engineers (303) 848-8497 staylor@anthemstructural.com	

INTERIOR BEARING 2X6, GRID 17, LEVEL 2 (ROOF DECK)
2 piece(s) 2 x 6 HF No.2 @ 16" OC

Wall Height: 9' 7 1/8" | Member Height: 9' 2 5/8" | O. C. Spacing: 16.00"



Drawing is Conceptual

Design Results	Actual	Allowed	Result	LDF	Load: Combination
Slenderness	20	50	Passed (40%)	--	--
Compression (lbs)	7548	13236	Passed (57%)	1.15	1.0 D + 0.75 L + 0.75 S
Plate Bearing (lbs)	7548	11602	Passed (65%)	--	1.0 D + 0.75 L + 0.75 S
Lateral Reaction (lbs)	0	--	--	--	N/A
Lateral Shear (lbs)	0	N/A	Passed (N/A)	--	N/A
Lateral Moment (ft-lbs)	0 @ mid-span	N/A	Passed (N/A)	--	N/A
Total Deflection (in)	0.10 @ mid-span	0.92	Passed (L/1101)	--	1.0 D + 0.75 L + 0.75 S
Bending/Compression	0.99	1	Passed (99%)	1.15	1.0 D + 0.75 L + 0.75 S

- Lateral deflection criteria: Wind (L/120)
- Input axial load eccentricity for this design is 16.67% of applicable member side dimension.
- Applicable calculations are based on NDS.
- A bearing area factor of 1.125 has been applied to base plate bearing capacity.
- The column stability factor (Kf = 0.6) applied to this design assumes nailed built-up columns per NDS section 15.3.3. For Weyerhaeuser ELP products refer to the U.S. Wall Guide for multiple-member connection requirements.
- A 15% increase in the moment capacity has been added to account for repetitive member usage.

Supports	Type	Material
Top	Dbl 2X	Douglas Fir-Larch
Base	2X	Douglas Fir-Larch

System : Wall
Member Type : Stud
Building Code : IBC 2018
Design Methodology : ASD

Max Unbraced Length	Comments
1'	

Vertical Loads	Spacing	Dead (0.90)	Floor Live (1.00)	Snow (1.15)	Comments
1 - Point (PLF)	16.00"	-	-	1440.0	ROOF
2 - Point (PLF)	16.00"	468.0	1080.0	3060.0	LEVEL 4 (ROOF DECK)
3 - Point (PLF)	16.00"	468.0	720.0	-	LEVEL 3

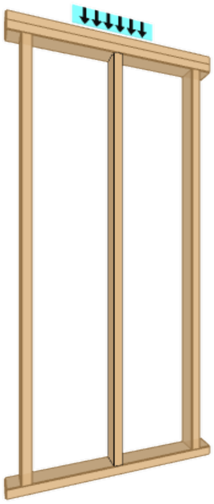
Weyerhaeuser Notes

ForteWEB Software Operator	Job Notes
Samantha Taylor Anthem Structural Engineers (303) 848-8497 staylor@anthemstructural.com	

INTERIOR BEARING 2X6, GRID 17, LEVEL 1

2 piece(s) 2 x 6 HF No.2 @ 16" OC

Wall Height: 9' 7 1/8" Member Height: 9' 2 5/8" O. C. Spacing: 16.00"



Drawing is Conceptual

Design Results	Actual	Allowed	Result	LDF	Load: Combination
Slenderness	20	50	Passed (40%)	--	--
Compression (lbs)	5856	13236	Passed (44%)	1.15	1.0 D + 0.75 L + 0.75 S
Plate Bearing (lbs)	5856	11602	Passed (50%)	--	1.0 D + 0.75 L + 0.75 S
Lateral Reaction (lbs)	0	--	--	--	N/A
Lateral Shear (lbs)	0	N/A	Passed (N/A)	--	N/A
Lateral Moment (ft-lbs)	0 @ mid-span	N/A	Passed (N/A)	--	N/A
Total Deflection (in)	0.08 @ mid-span	0.92	Passed (L/1419)	--	1.0 D + 0.75 L + 0.75 S
Bending/Compression	0.62	1	Passed (62%)	1.15	1.0 D + 0.75 L + 0.75 S

- Lateral deflection criteria: Wind (L/120)
- Input axial load eccentricity for this design is 16.67% of applicable member side dimension.
- Applicable calculations are based on NDS.
- A bearing area factor of 1.125 has been applied to base plate bearing capacity.
- The column stability factor (Kf = 0.6) applied to this design assumes nailed built-up columns per NDS section 15.3.3. For Weyerhaeuser ELP products refer to the U.S. Wall Guide for multiple-member connection requirements.
- A 15% increase in the moment capacity has been added to account for repetitive member usage.

Supports	Type	Material
Top	Dbl 2X	Douglas Fir-Larch
Base	2X	Douglas Fir-Larch

System : Wall
Member Type : Stud
Building Code : IBC 2018
Design Methodology : ASD

Max Unbraced Length	Comments
1'	

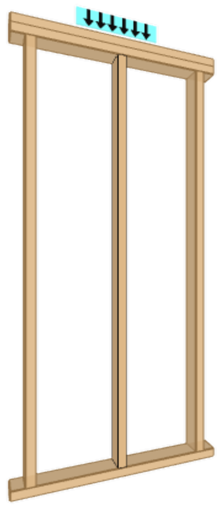
Vertical Loads	Spacing	Dead (0.90)	Floor Live (1.00)	Snow (1.15)	Comments
1 - Point (PLF)	16.00"	288.0	-	1440.0	ROOF
2 - Point (PLF)	16.00"	468.0	720.0	-	LEVEL 4
3 - Point (PLF)	16.00"	468.0	720.0	-	LEVEL 3
4 - Point (PLF)	16.00"	468.0	720.0	-	LEVEL 2

Weyerhaeuser Notes

ForteWEB Software Operator	Job Notes
Samantha Taylor Anthem Structural Engineers (303) 848-8497 staylor@anthemstructural.com	

INTERIOR BEARING 2X6, GRID 17, LEVEL 1 (ROOF DECK)
2 piece(s) 2 x 6 HF No.2 @ 16" OC

Wall Height: 9' 7 1/8" | Member Height: 9' 2 5/8" | O. C. Spacing: 16.00"



Drawing is Conceptual

Design Results	Actual	Allowed	Result	LDF	Load: Combination
Slenderness	20	50	Passed (40%)	--	--
Compression (lbs)	7452	13236	Passed (56%)	1.15	1.0 D + 0.75 L + 0.75 S
Plate Bearing (lbs)	7452	11602	Passed (64%)	--	1.0 D + 0.75 L + 0.75 S
Lateral Reaction (lbs)	0	--	--	--	N/A
Lateral Shear (lbs)	0	N/A	Passed (N/A)	--	N/A
Lateral Moment (ft-lbs)	0 @ mid-span	N/A	Passed (N/A)	--	N/A
Total Deflection (in)	0.10 @ mid-span	0.92	Passed (L/1115)	--	1.0 D + 0.75 L + 0.75 S
Bending/Compression	0.97	1	Passed (97%)	1.15	1.0 D + 0.75 L + 0.75 S

- Lateral deflection criteria: Wind (L/120)
- Input axial load eccentricity for this design is 16.67% of applicable member side dimension.
- Applicable calculations are based on NDS.
- A bearing area factor of 1.125 has been applied to base plate bearing capacity.
- The column stability factor (Kf = 0.6) applied to this design assumes nailed built-up columns per NDS section 15.3.3. For Weyerhaeuser ELP products refer to the U.S. Wall Guide for multiple-member connection requirements.
- A 15% increase in the moment capacity has been added to account for repetitive member usage.

Supports	Type	Material
Top	Dbl 2X	Douglas Fir-Larch
Base	2X	Douglas Fir-Larch

System : Wall
Member Type : Stud
Building Code : IBC 2018
Design Methodology : ASD

Max Unbraced Length	Comments
1'	

Vertical Loads	Spacing	Dead (0.90)	Floor Live (1.00)	Snow (1.15)	Comments
1 - Point (PLF)	16.00"	-	-	-	ROOF
2 - Point (PLF)	16.00"	468.0	1080.0	3060.0	LEVEL 4 (DECK)
3 - Point (PLF)	16.00"	468.0	720.0	-	LEVEL 3
4 - Point (PLF)	16.00"	468.0	720.0	-	LEVEL 2

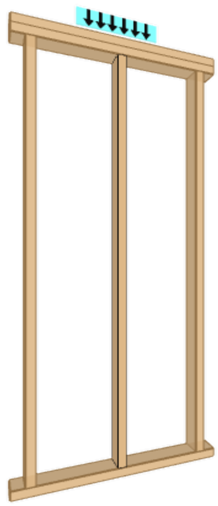
Weyerhaeuser Notes

ForteWEB Software Operator	Job Notes
Samantha Taylor Anthem Structural Engineers (303) 848-8497 staylor@anthemstructural.com	

INTERIOR BEARING 2X6, STAIR WALL, LEVEL 1

1 piece(s) 2 x 4 HF No.2 @ 16" OC

Wall Height: 10' 3" Member Height: 9' 10 1/2" O. C. Spacing: 16.00"



Drawing is Conceptual

Design Results	Actual	Allowed	Result	LDF	Load: Combination
Slenderness	34	50	Passed (68%)	--	--
Compression (lbs)	600	1678	Passed (36%)	1.00	1.0 D + 1.0 L
Plate Bearing (lbs)	600	2658	Passed (23%)	--	1.0 D + 1.0 L
Lateral Reaction (lbs)	0	--	--	--	N/A
Lateral Shear (lbs)	0	N/A	Passed (N/A)	--	N/A
Lateral Moment (ft-lbs)	0 @ mid-span	N/A	Passed (N/A)	--	N/A
Total Deflection (in)	0.05 @ mid-span	0.99	Passed (L/2619)	--	1.0 D + 1.0 L
Bending/Compression	0.26	1	Passed (26%)	1.00	1.0 D + 1.0 L

- Lateral deflection criteria: Wind (L/120)
- Input axial load eccentricity for this design is 16.67% of applicable member side dimension.
- Applicable calculations are based on NDS.
- A bearing area factor of 1.25 has been applied to base plate bearing capacity.
- A 15% increase in the moment capacity has been added to account for repetitive member usage.

Supports	Type	Material
Top	Dbl 2X	Hem Fir
Base	2X	Hem Fir

System : Wall
Member Type : Stud
Building Code : IBC 2018
Design Methodology : ASD

Max Unbraced Length	Comments
1'	

Vertical Load	Spacing	Dead (0.90)	Floor Live (1.00)	Comments
1 - Point (PLF)	16.00"	130.0	320.0	LEVEL 2

Weyerhaeuser Notes

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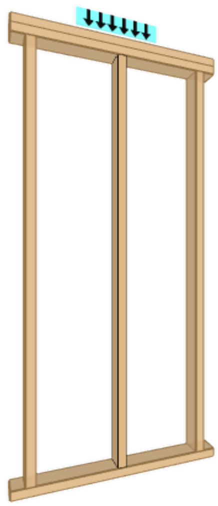
The product application, input design loads, dimensions and support information have been provided by ForteWEB Software Operator

ForteWEB Software Operator	Job Notes
Samantha Taylor Anthem Structural Engineers (303) 848-8497 staylor@anthemstructural.com	

INTERIOR BEARING 2X6, GRID 16, LEVEL 3 (ROOF DECK)

2 piece(s) 2 x 6 HF No.2 @ 16" OC

Wall Height: 9' 7 1/8" Member Height: 9' 2 5/8" O. C. Spacing: 16.00"



Drawing is Conceptual

Design Results	Actual	Allowed	Result	LDF	Load: Combination
Slenderness	20	50	Passed (40%)	--	--
Compression (lbs)	4235	13236	Passed (32%)	1.15	1.0 D + 0.75 L + 0.75 S
Plate Bearing (lbs)	4235	11602	Passed (37%)	--	1.0 D + 0.75 L + 0.75 S
Lateral Reaction (lbs)	0	--	--	--	N/A
Lateral Shear (lbs)	0	N/A	Passed (N/A)	--	N/A
Lateral Moment (ft-lbs)	0 @ mid-span	N/A	Passed (N/A)	--	N/A
Total Deflection (in)	0.06 @ mid-span	0.92	Passed (L/1963)	--	1.0 D + 0.75 L + 0.75 S
Bending/Compression	0.36	1	Passed (36%)	1.15	1.0 D + 0.75 L + 0.75 S

- Lateral deflection criteria: Wind (L/120)
- Input axial load eccentricity for this design is 16.67% of applicable member side dimension.
- Applicable calculations are based on NDS.
- A bearing area factor of 1.125 has been applied to base plate bearing capacity.
- The column stability factor (Kf = 0.6) applied to this design assumes nailed built-up columns per NDS section 15.3.3. For Weyerhaeuser ELP products refer to the U.S. Wall Guide for multiple-member connection requirements.
- A 15% increase in the moment capacity has been added to account for repetitive member usage.

Supports	Type	Material
Top	Dbl 2X	Douglas Fir-Larch
Base	2X	Douglas Fir-Larch

System : Wall
Member Type : Stud
Building Code : IBC 2018
Design Methodology : ASD

Max Unbraced Length	Comments
1'	

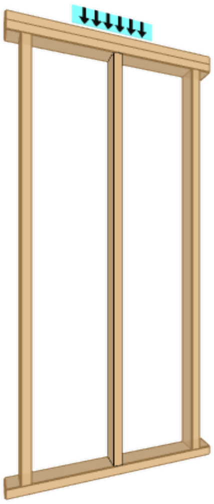
Vertical Loads	Spacing	Dead (0.90)	Floor Live (1.00)	Snow (1.15)	Comments
1 - Point (PLF)	16.00"	-	-	-	ROOF
2 - Point (PLF)	16.00"	416.0	960.0	2720.0	LEVEL 4 (DECK)

Weyerhaeuser Notes

ForteWEB Software Operator	Job Notes
Samantha Taylor Anthem Structural Engineers (303) 848-8497 staylor@anthemstructural.com	

INTERIOR BEARING 2X6, GRID 16, LEVEL 2
2 piece(s) 2 x 6 HF No.2 @ 16" OC

Wall Height: 9' 7 1/8" Member Height: 9' 2 5/8" O. C. Spacing: 16.00"



Drawing is Conceptual

Design Results	Actual	Allowed	Result	LDF	Load: Combination
Slenderness	20	50	Passed (40%)	--	--
Compression (lbs)	4011	13236	Passed (30%)	1.15	1.0 D + 0.75 L + 0.75 S
Plate Bearing (lbs)	4011	11602	Passed (35%)	--	1.0 D + 0.75 L + 0.75 S
Lateral Reaction (lbs)	0	--	--	--	N/A
Lateral Shear (lbs)	0	N/A	Passed (N/A)	--	N/A
Lateral Moment (ft-lbs)	0 @ mid-span	N/A	Passed (N/A)	--	N/A
Total Deflection (in)	0.05 @ mid-span	0.92	Passed (L/2072)	--	1.0 D + 0.75 L + 0.75 S
Bending/Compression	0.33	1	Passed (33%)	1.15	1.0 D + 0.75 L + 0.75 S

- Lateral deflection criteria: Wind (L/120)
- Input axial load eccentricity for this design is 16.67% of applicable member side dimension.
- Applicable calculations are based on NDS.
- A bearing area factor of 1.125 has been applied to base plate bearing capacity.
- The column stability factor (Kf = 0.6) applied to this design assumes nailed built-up columns per NDS section 15.3.3. For Weyerhaeuser ELP products refer to the U.S. Wall Guide for multiple-member connection requirements.
- A 15% increase in the moment capacity has been added to account for repetitive member usage.

Supports	Type	Material
Top	Dbl 2X	Douglas Fir-Larch
Base	2X	Douglas Fir-Larch

System : Wall
Member Type : Stud
Building Code : IBC 2018
Design Methodology : ASD

Max Unbraced Length	Comments
1'	

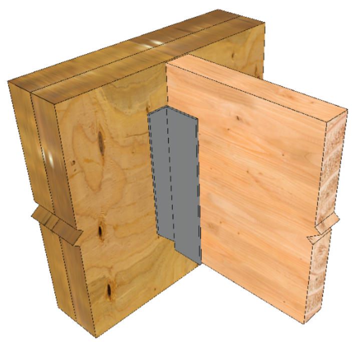
Vertical Loads	Spacing	Dead (0.90)	Floor Live (1.00)	Snow (1.15)	Comments
1 - Point (PLF)	16.00"	256.0	-	1280.0	ROOF
2 - Point (PLF)	16.00"	416.0	640.0	-	LEVEL 4
3 - Point (PLF)	16.00"	416.0	640.0	-	LEVEL 3

ForteWEB Software Operator	Job Notes
Samantha Taylor Anthem Structural Engineers (303) 848-8497 staylor@anthemstructural.com	

HANGERS

HS
Hanger Selector

3D Model



Input

Settings

Country	Connection Type
USA	Joist (Flush Top)

Job Settings

Hanger Type	Fastener Type	Download Duration	Uplift Duration
All Types	All Types	Snow (115)	Quake/Wind (160)

Header

Member Type	Lumber Species	Width	Depth	Number of Plies	Member ID	Lumber Finish
Lamimated Veneer Lumber	DF/SP (Douglas fir or Southern Pine)	1 3/4" x	14"	2	Beam#6, #13	No

Joist

Member Type	Lumber Species	Width	Depth	Number of Plies	Member ID	Lumber Finish	Download (ASD)	Uplift (ASD)
Solid Sawn	HF (Hem Fir)	2x (1 1/2")	12 (11 1/4")	1	Joist #3	No	282	333

Hanger Options

Skew Type	Skew (Degrees)	Slope Type	Slope (Degrees)	Open Closed Type	Top Flange Bend (Degrees)	Sloped Down Type	Top Flange Slope (Degrees)	Offset Direction	High, Low, Center Flush
No Skew	0	No Sloped	0	Normal	0	No Sloped	0	Centered (No Offset)	Center

Output

Results

Show Optimized Models: Yes

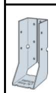
Model	Installed Cost	Width	Height	Bearing	TF Depth	TF Fasteners	Face Fasteners	Joist Fasteners	Download (lbs)	Uplift (lbs)
 LUS28	Lowest	1.563	6.625	1.75	-	-	6-10d	3-10d	1160	870
 LUS28	+2.00%	1.563	6.625	1.75	-	-	6-10d	4-10d	1275	1105
 LUS28	+3.00%	1.563	6.625	1.75	-	-	6-10dx1.5	3-10d	1000	870
 LUS28	+5.00%	1.563	6.625	1.75	-	-	6-10dx1.5	4-10d	1130	1105
 LU210	+9.00%	1.563	7.813	1.5	-	-	10-10d	6-10dx1.5	1325	850
 LU210	+13.00%	1.563	7.813	1.5	-	-	10-16d	6-10dx1.5	1475	850
 LU210	+14.00%	1.563	7.813	1.5	-	-	10-10dx1.5	6-10dx1.5	1095	850
 LUS210	+17.00%	1.563	7.813	1.75	-	-	8-10d	4-10d	1545	1105
 LUS210	+21.00%	1.563	7.813	1.75	-	-	8-10dx1.5	4-10d	1355	1105
 LUC210Z	+71.00%	1.563	7.75	1.75	-	-	10-10d	6-10dx1.5	1345	945

Table Notes

1. All loads are displayed in units of pounds and based on Allowable Stress Design
2. Click on the Models above to be taken to the product page for more information, refer to the current Wood Construction Connectors catalog for **General Notes** and **Installation Instructions**

HS
Hanger Selector

3D Model

Input

Settings

Country	Connection Type
USA	Joist (Flush Top)

Job Settings

Hanger Type	Fastener Type	Download Duration	Uplift Duration
All Types	All Types	Snow (115)	Quake/Wind (160)

Header

Member Type	Lumber Species	Width	Depth	Number of Plies	Member ID	Lumber Finish
Lamimated Veneer Lumber	DF/SP (Douglas fir or Southern Pine)	1 3/4" x	14"	2	Beam #6, Beam #13	No

Joist

Member Type	Lumber Species	Width	Depth	Number of Plies	Member ID	Lumber Finish	Download (ASD)	Uplift (ASD)
Solid Sawn	HF (Hem Fir)	2x (1 1/2")	12 (11 1/4")	2	Beam #5	No	282	333

Hanger Options

Skew Type	Skew (Degrees)	Slope Type	Slope (Degrees)	Open Closed Type	Top Flange Bend (Degrees)	Sloped Down Type	Top Flange Slope (Degrees)	Offset Direction	High, Low, Center Flush
No Skew	0	No Sloped	0	Normal	0	No Sloped	0	Centered (No Offset)	Center

Output

Results

Show Optimized Models: Yes

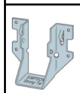
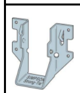
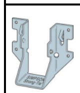
Model	Installed Cost	Width	Height	Bearing	TF Depth	TF Fasteners	Face Fasteners	Joist Fasteners	Download (lbs)	Uplift (lbs)
 LUS28-2	Lowest	3.125	7	2	-	-	6-10d	3-10d	1120	745
 LUS28-2	+2.00%	3.125	7	2	-	-	6-10d	4-10d	1225	890
 LUS28-2	+2.00%	3.125	7	2	-	-	6-10dx1.5	3-10d	975	745
 LUS28-2	+2.00%	3.125	7	2	-	-	6-16d	3-16d	1385	875
 LUS28-2	+4.00%	3.125	7	2	-	-	6-10dx1.5	4-10d	1080	890
 LUS28-2	+4.00%	3.125	7	2	-	-	6-16d	4-16d	1525	1045
 LUS210-2	+12.00%	3.125	8.938	2	-	-	8-10d	6-10d	1705	1230
 LUS210-2	+14.00%	3.125	8.938	2	-	-	8-10dx1.5	6-10d	1515	1230
 LUS210-2	+15.00%	3.125	8.938	2	-	-	8-16d	6-16d	2130	1445
 LUS28-2	+86.00%	3.125	7	2	-	-	6-SD9112	4-SD9212	1570	1100

Table Notes

1. All loads are displayed in units of pounds and based on Allowable Stress Design
2. Click on the Models above to be taken to the product page for more information, refer to the current Wood Construction Connectors catalog for **General Notes** and **Installation Instructions**

HS
Hanger Selector

3D Model

Input

Settings

Country	Connection Type
USA	Joist (Flush Top)

Job Settings

Hanger Type	Fastener Type	Download Duration	Uplift Duration
All Types	All Types	Snow (115)	Quake/Wind (160)

Header

Member Type	Lumber Species	Width	Depth	Number of Plies	Member ID	Lumber Finish
Lamimated Veneer Lumber	DF/SP (Douglas fir or Southern Pine)	1 3/4" x	14"	3	Beam #6, Beam #13	No

Joist

Member Type	Lumber Species	Width	Depth	Number of Plies	Member ID	Lumber Finish	Download (ASD)	Uplift (ASD)
Lamimated Veneer Lumber	DF/SP (Douglas fir or Southern Pine)	1 3/4" x	14"	2	Beam #3, Beam #14	No	3320	0

Hanger Options

Skew Type	Skew (Degrees)	Slope Type	Slope (Degrees)	Open Closed Type	Top Flange Bend (Degrees)	Sloped Down Type	Top Flange Slope (Degrees)	Offset Direction	High, Low, Center Flush
No Skew	0	No Sloped	0	Normal	0	No Sloped	0	Centered (No Offset)	Center

Output

Results

Show Optimized Models: Yes

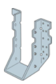
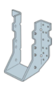
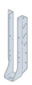
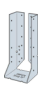
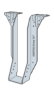
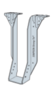
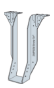
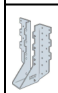
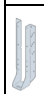
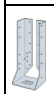
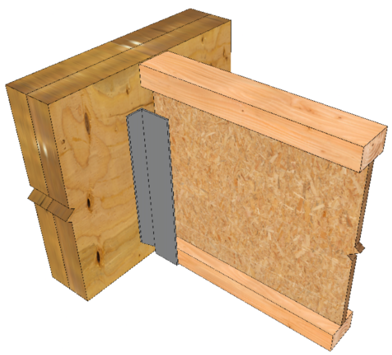
Model	Installed Cost	Width	Height	Bearing	TF Depth	TF Fasteners	Face Fasteners	Joist Fasteners	Download (lbs)	Uplift (lbs)
 HHUS410	Lowest	3.625	9	3	-	-	30-10d	10-10d	5330	2425
 HHUS410	+2.00%	3.625	9	3	-	-	30-16d	10-16d	6440	3550
 HU412	+51.00%	3.563	10.344	2.5	-	-	22-16d	10-10d	3695	1780
 HUC412	+51.00%	3.563	10.344	2.5	-	-	22-16d	10-10d	3695	1780
 BA3.56/14	+57.00%	3.563	13.969	3	2.5	-	10-16d	2-10dx1.5	4015	255
 BA3.56/14	+59.00%	3.563	13.969	3	2.5	-	10-10d	8-10dx1.5	3555	1275
 BA3.56/14	+60.00%	3.563	13.969	3	2.5	-	10-16d	8-10dx1.5	4720	1275
 HUS412	+63.00%	3.563	10.563	2	-	-	10-SD10212	10-SD10212	3820	2830
 HU414	+80.00%	3.563	11.906	2.5	-	-	24-10d	12-10d	3420	1780
 HUC414	+80.00%	3.563	11.906	2.5	-	-	24-10d	12-10d	3420	1780

Table Notes

1. All loads are displayed in units of pounds and based on Allowable Stress Design
2. Click on the Models above to be taken to the product page for more information, refer to the current Wood Construction Connectors catalog for **General Notes** and **Installation Instructions**

HS
Hanger Selector

3D Model



Input

Settings

Country	Connection Type
USA	Joist (Flush Top)

Job Settings

Hanger Type	Fastener Type	Download Duration	Uplift Duration
All Types	All Types	Snow (115)	Quake/Wind (160)

Header

Member Type	Lumber Species	Width	Depth	Number of Plies	Member ID	Lumber Finish
Lamimated Veneer Lumber	DF/SP (Douglas fir or Southern Pine)	1 3/4" x	14"	2	Beam #3, Beam #7	No

Joist

Member Type	Lumber Species	Width	Depth	Number of Plies	Member ID	Lumber Finish	Download (ASD)	Uplift (ASD)
I-Joist	DF (Douglas Fir)	2 5/16"	14"	1	Joist #2	No	603	0

Hanger Options

Skew Type	Skew (Degrees)	Slope Type	Slope (Degrees)	Open Closed Type	Top Flange Bend (Degrees)	Sloped Down Type	Top Flange Slope (Degrees)	Offset Direction	High, Low, Center Flush
No Skew	0	No Sloped	0	Normal	0	No Sloped	0	Centered (No Offset)	Center

Output

Results

Show Optimized Models: Yes

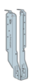
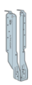
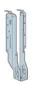
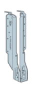
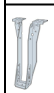
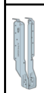
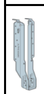
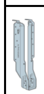
Model	Installed Cost	Width	Height	Bearing	TF Depth	TF Fasteners	Face Fasteners	Joist Fasteners	Download (lbs)	Uplift (lbs)
 IUS2.37/14*	Lowest	2.438	13.969	2	-	-	12-10d	2-Strong-Grip	1615	70
 IUS2.37/14*	+2.00%	2.438	13.969	2	-	-	14-10d	2-Strong-Grip	1805	70
 IUS2.37/14*	+2.00%	2.438	13.969	2	-	-	12-10dx1.5	2-Strong-Grip	1325	70
 IUS2.37/14*	+4.00%	2.438	13.969	2	-	-	14-10dx1.5	2-Strong-Grip	1450	70
 ITS2.37/14	+8.00%	2.438	13.938	2	1.438	-	2-10d	2-Strong-Grip	1550	120
 ITS2.37/14	+8.00%	2.438	13.938	2	1.438	-	2-16d	2-Strong-Grip	1785	120
 ITS2.37/14	+9.00%	2.438	13.938	2	1.438	-	2-10dx1.5	2-Strong-Grip	1395	120
 IUS2.37/9.5*	+9.00%	2.438	9.469	2	-	-	8-10d	2-10dx1.5	1075	345
 IUS2.37/9.5*	+10.00%	2.438	9.469	2	-	-	8-10dx1.5	2-10dx1.5	885	345
 IUS2.37/11.88*	+20.00%	2.438	11.844	2	-	-	10-10d	2-10dx1.5	1345	345

Table Notes

* Web Stiffener Required. Refer to current Wood Construction Connectors catalog for General Notes & Installation Instructions.

1. All loads are displayed in units of pounds and based on Allowable Stress Design
2. Click on the Models above to be taken to the product page for more information, refer to the current Wood Construction Connectors catalog for **General Notes** and **Installation Instructions**

HS
Hanger Selector

3D Model

Input

Settings

Country	Connection Type
USA	Joist (Flush Top)

Job Settings

Hanger Type	Fastener Type	Download Duration	Uplift Duration
All Types	All Types	Snow (115)	Quake/Wind (160)

Header

Member Type	Lumber Species	Width	Depth	Number of Plies	Member ID	Lumber Finish
Lamimated Veneer Lumber	DF/SP (Douglas fir or Southern Pine)	1 3/4" x	14"	2	Beam #12	No

Joist

Member Type	Lumber Species	Width	Depth	Number of Plies	Member ID	Lumber Finish	Download (ASD)	Uplift (ASD)
I-Joist	DF (Douglas Fir)	1 3/4"	14"	1	Joist #6	No	433	0

Hanger Options

Skew Type	Skew (Degrees)	Slope Type	Slope (Degrees)	Open Closed Type	Top Flange Bend (Degrees)	Sloped Down Type	Top Flange Slope (Degrees)	Offset Direction	High, Low, Center Flush
No Skew	0	No Sloped	0	Normal	0	No Sloped	0	Centered (No Offset)	Center

Output

Results

Show Optimized Models: Yes

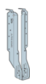
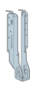
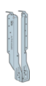
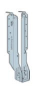
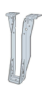
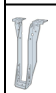
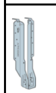
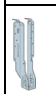
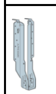
Model	Installed Cost	Width	Height	Bearing	TF Depth	TF Fasteners	Face Fasteners	Joist Fasteners	Download (lbs)	Uplift (lbs)
 IUS1.81/14*	Lowest	1.875	13.969	2	-	-	12-10d	2-Strong-Grip	1615	70
 ITS1.81/14	+2.00%	1.875	13.938	2	1.375	-	2-10d	2-Strong-Grip	1550	120
 IUS1.81/14*	+2.00%	1.875	13.969	2	-	-	12-10dx1.5	2-Strong-Grip	1325	70
 IUS1.81/14*	+2.00%	1.875	13.969	2	-	-	14-10d	2-Strong-Grip	1805	70
 IUS1.81/9.5*	+2.00%	1.875	9.469	2	-	-	8-10d	2-10dx1.5	1075	345
 ITS1.81/14	+3.00%	1.875	13.938	2	1.375	-	2-16d	2-Strong-Grip	1785	120
 ITS1.81/14	+3.00%	1.875	13.938	2	1.375	-	2-10dx1.5	2-Strong-Grip	1395	120
 IUS1.81/9.5*	+3.00%	1.875	9.469	2	-	-	8-10dx1.5	2-10dx1.5	885	345
 IUS1.81/14*	+4.00%	1.875	13.969	2	-	-	14-10dx1.5	2-Strong-Grip	1450	70
 IUS1.81/11.88*	+14.00%	1.875	11.844	2	-	-	10-10d	2-10dx1.5	1345	345

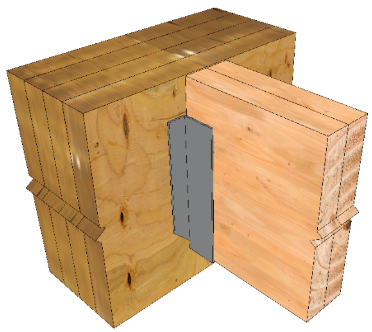
Table Notes

* Web Stiffener Required. Refer to current Wood Construction Connectors catalog for General Notes & Installation Instructions.

1. All loads are displayed in units of pounds and based on Allowable Stress Design
2. Click on the Models above to be taken to the product page for more information, refer to the current Wood Construction Connectors catalog for **General Notes** and **Installation Instructions**

HS
Hanger Selector

3D Model



Input

Settings

Country	Connection Type
USA	Joist (Flush Top)

Job Settings

Hanger Type	Fastener Type	Download Duration	Uplift Duration
All Types	All Types	Snow (115)	Quake/Wind (160)

Header

Member Type	Lumber Species	Width	Depth	Number of Plies	Member ID	Lumber Finish
Lamimated Veneer Lumber	DF/SP (Douglas fir or Southern Pine)	1 3/4" x	14"	4	Beam #1, Beam #9	No

Joist

Member Type	Lumber Species	Width	Depth	Number of Plies	Member ID	Lumber Finish	Download (ASD)	Uplift (ASD)
Solid Sawn	HF (Hem Fir)	2x (1 1/2")	12 (11 1/4")	2	Beam #4, Beam #10	No	2138	0

Hanger Options

Skew Type	Skew (Degrees)	Slope Type	Slope (Degrees)	Open Closed Type	Top Flange Bend (Degrees)	Sloped Down Type	Top Flange Slope (Degrees)	Offset Direction	High, Low, Center Flush
No Skew	0	No Sloped	0	Normal	0	No Sloped	0	Centered (No Offset)	Center

Output

Results

Show Optimized Models: Yes

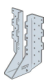
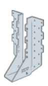
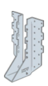
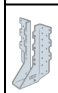
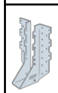
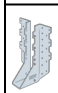
Model	Installed Cost	Width	Height	Bearing	TF Depth	TF Fasteners	Face Fasteners	Joist Fasteners	Download (lbs)	Uplift (lbs)
 LUS210-2	Lowest	3.125	8.938	2	-	-	8-SD9112	6-SD9212	2235	1240
 U210-2	+1.00%	3.125	8.5	2	-	-	14-16d	6-10d	2285	960
 HUS210-2	+50.00%	3.125	9.188	2	-	-	8-16d	8-16d	2465	2820
 LUS214-2	+50.00%	3.125	10.938	2	-	-	10-16d	6-16d	2450	1445
 LUS214-2	+106.00%	3.125	10.938	2	-	-	10-SD9112	6-SD9212	2630	1650
 HUS28-2	+108.00%	3.125	7.188	2	-	-	6-SD10212	6-SD10212	2290	1950
 HUS212-2	+115.00%	3.125	11	2	-	-	10-10d	10-10d	2525	2880
 HUS212-2	+117.00%	3.125	11	2	-	-	10-16d	10-16d	3080	3435
 HUS212-2	+117.00%	3.125	11	2	-	-	10-10dx1.5	10-10d	2250	2880
 HUS210-2	+152.00%	3.125	9.188	2	-	-	8-SD10212	8-SD10212	3055	2440

Table Notes

1. All loads are displayed in units of pounds and based on Allowable Stress Design
2. Click on the Models above to be taken to the product page for more information, refer to the current Wood Construction Connectors catalog for **General Notes** and **Installation Instructions**

L/LS/GA

Reinforcing and Skewable Angles

L — Staggered nail pattern reduces the possibility for splitting.

LS — Field-adjustable 0° to 135° angles.

GA — Gusset angles' embossed bend section provides added strength.

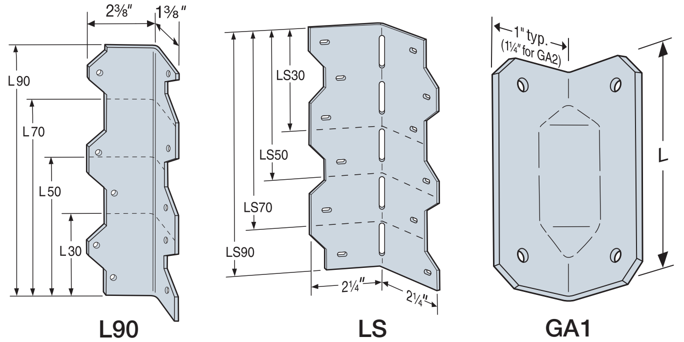
Material: L — 16 gauge; GA and LS — 18 gauge

Finish: Galvanized. Some products available in stainless steel or ZMAX® coating. See Corrosion Information, pp. 12–15.

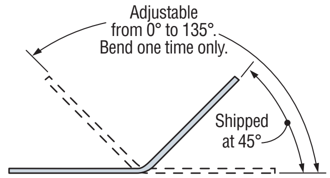
Installation:

- Use all specified fasteners; see General Notes
- LS — Field skewable; bend one time only
- Joist must be constrained against rotation (for example, with solid blocking) when using a single LS per connection
- Nail the L angle's wider leg into the joist to ensure table loads and allow correct nailing

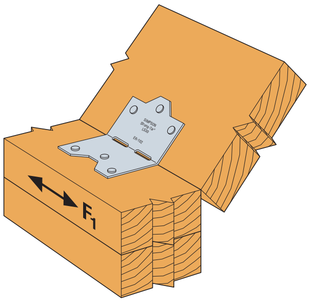
Codes: See p. 11 for Code Reference Key Chart



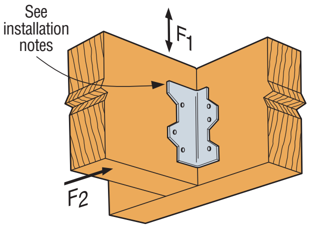
L90 LS GA1



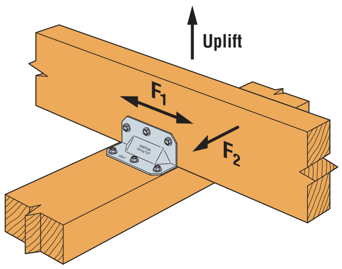
LS Top View



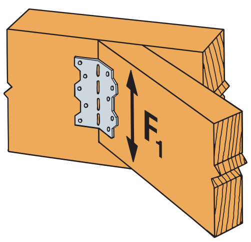
Typical LS30 Installation



Typical L50 Installation



Typical GA Installation



Typical LS70 Installation

These products are available with additional corrosion protection. For more information, see p. 14.

SS For stainless-steel fasteners, see p. 21.

SD Many of these products are approved for installation with Strong-Drive® SD Connector screws. See pp. 348–352 for more information.

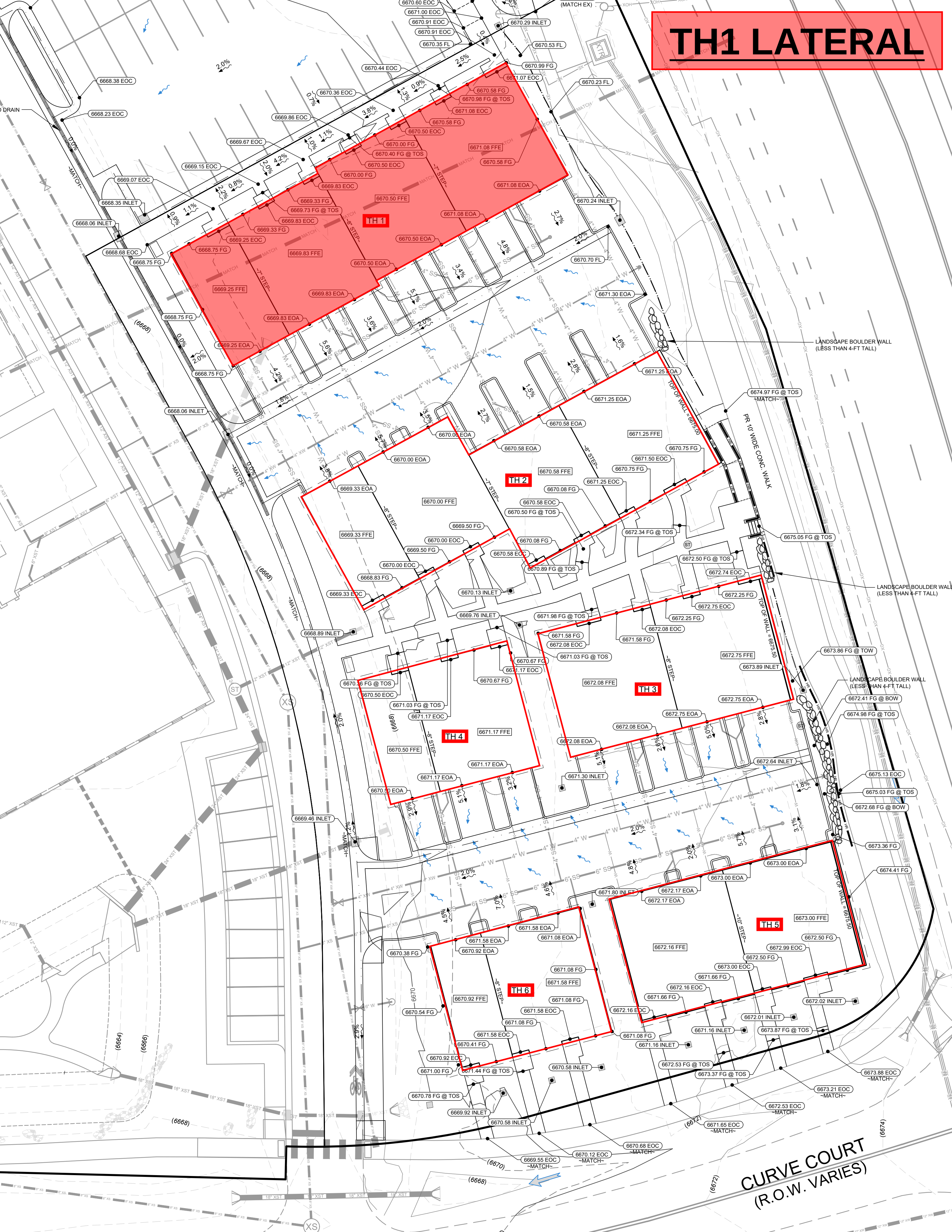
Model No.	Fasteners (in.)	L (in.)	Load Direction	DF/SP Allowable Loads				SPF/HF Allowable Loads				Code Ref.
				Floor (100)	Snow (115)	Roof (125)	Wind/ Seismic (160)	Floor (100)	Snow (115)	Roof (125)	Wind/ Seismic (160)	
GA1	(4) 0.148 x 1½	2¾	F_1, F_2	235	270	290	350	200	230	250	300	IBC, FL, LA
	(4) #9 x 1½" SD		F_1	340	375	375	375	225	260	280	325	IBC, LA
	(4) #9 x 1½" SD		F_2	340	395	430	435	225	260	280	360	
GA2	(6) 0.148 x 1½	3¼	F_1, F_2	355	405	435	550	305	350	375	475	IBC, FL, LA
	(6) #9 x 1½" SD		F_1	515	590	640	695	335	385	420	540	IBC, LA
	(6) #9 x 1½" SD		F_2	515	590	640	820	335	385	420	540	
L30	(4) 0.148 x 1½	3	F_1	245	250	250	250	210	215	215	215	IBC, FL, LA
			F_2	245	275	295	370	210	235	255	320	
L50	(6) 0.148 x 1½	5	F_1	365	415	445	525	315	355	385	450	
			F_2	365	415	445	555	315	355	385	475	
L70	(8) 0.148 x 1½	7	F_1, F_2	485	550	595	740	415	475	510	635	
L90	(10) 0.148 x 1½	9	F_1, F_2	610	690	740	925	525	595	635	795	
LS30	(6) 0.148 x 1½	3⅜	F_1	320	320	320	320	275	275	275	275	
	(6) 0.148 x 3		F_1	355	395	395	395	305	340	340	340	
SS LS50	(8) 0.148 x 1½	4⅞	F_1	475	540	560	560	410	465	480	480	
	(8) 0.148 x 3		F_1	475	540	580	730	410	465	500	630	
LS70	(10) 0.148 x 1½	6⅜	F_1	590	645	645	645	510	555	555	555	
	(10) 0.148 x 3		F_1	590	675	725	915	510	580	625	785	
LS90	(12) 0.148 x 1½	7⅞	F_1	710	805	870	890	610	690	750	765	
	(12) 0.148 x 3		F_1	710	805	870	1,040	610	690	750	895	

1. GA angles may be installed with 0.148" x 3" nails.
2. GA1 uplift is 425 lb. for DF and 300 lb. for SPF when installed with Strong-Tie® SD Connector screws.
3. GA2 uplift is 370 lb. for DF and 260 lb. for SPF when installed with Strong-Tie® SD Connector screws.
4. Connectors are required on both sides to achieve F_2 loads in both directions.
5. **Fasteners:** Nail dimensions are listed diameter by length. SD screws are Simpson Strong-Tie® Strong-Drive SD Connector screws. See pp. 21–22 for fastener information.

UPDATED 07/01/22

Straps and Ties

TH1 LATERAL


TH 1
TH 2
TH 3
TH 4
TH 5
TH 6
6669.25 FFE
6669.83 FFE
6670.50 FFE
6671.08 FFE
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6670.58 FFE
6669.33 FFE
6671.25 FFE
6672.08 FFE
6672.75 FFE
6671.17 FFE
6670.50 FFE
6673.00 FFE
6672.16 FFE
6670.92 FFE
6671.58 FFE
LANDSCAPE BOULDER WALL
(LESS THAN 4-FT TALL)
PR 10' WIDE CONC. WALK
TOP OF WALL = 6675.00
TOP OF WALL = 6675.50
CURVE COURT
(R.O.W. VARIES)

LATERAL DESIGN

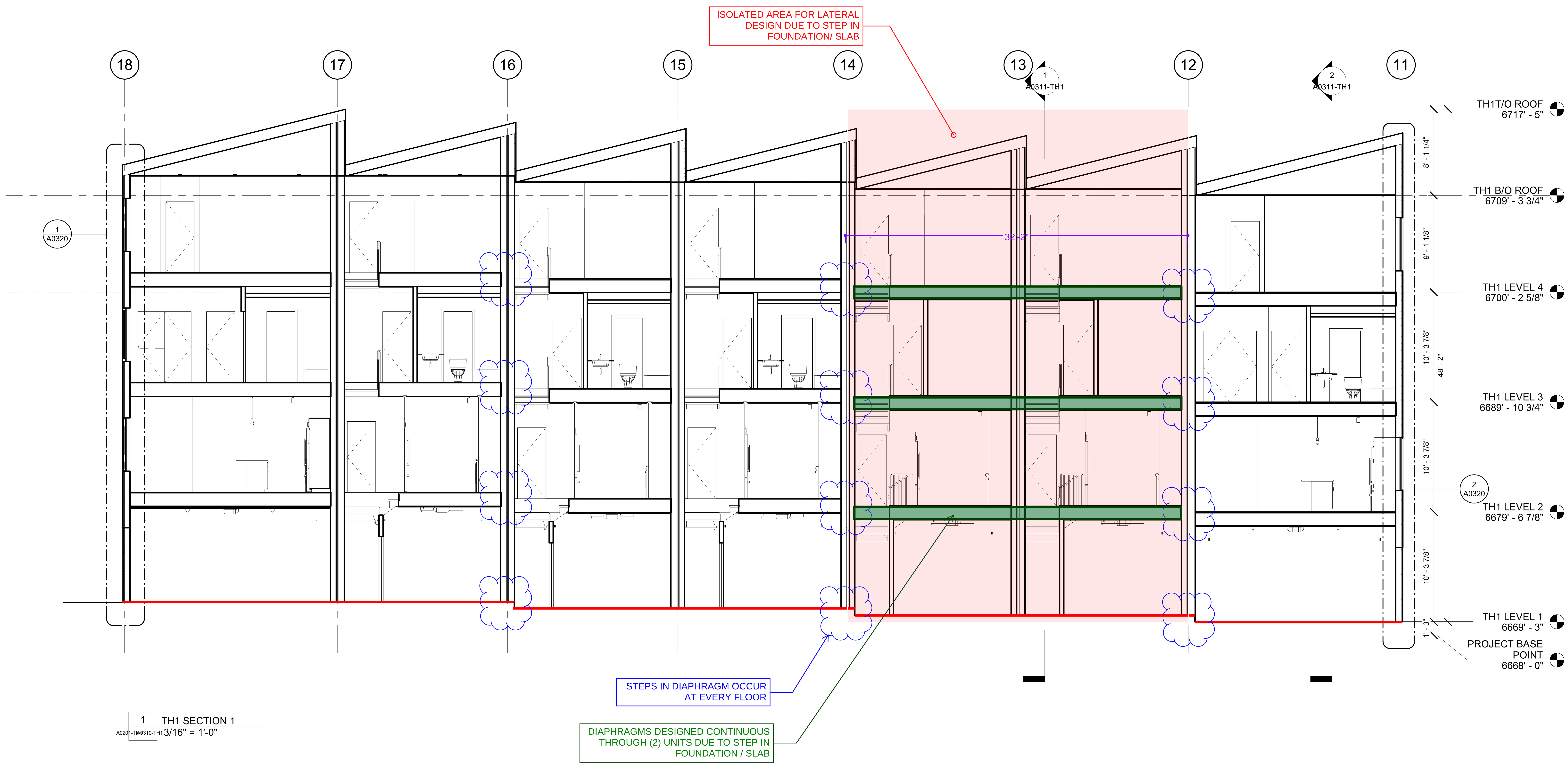
ASD LOAD SUMMARY:

SEISMIC:
$V = 18.6$ K
$V_{ISOLATED\ UNIT} = 4.5$ K

WIND:
$V_{N\text{-}S} = 61.0$ K WIND GOVERNS N-S DIRECTION
$V_{E\text{-}W} = 17.3$ K
$V_{ISOLATED\ GROUP} = 17.3$ K WIND GOVERNS E-W DIRECTION

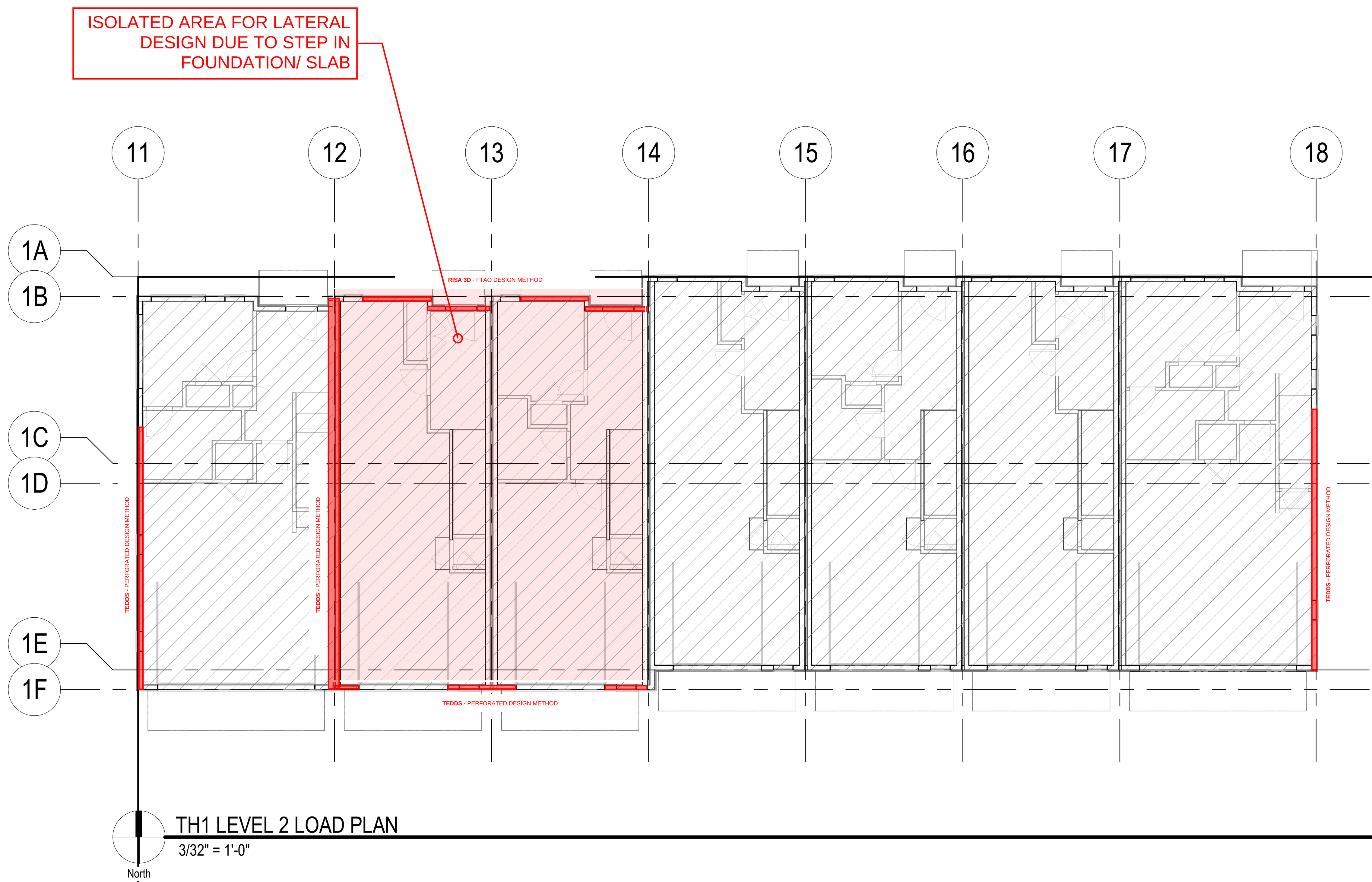
STEPS IN DIAPHRAGM/ FOUNDATION:

TH1

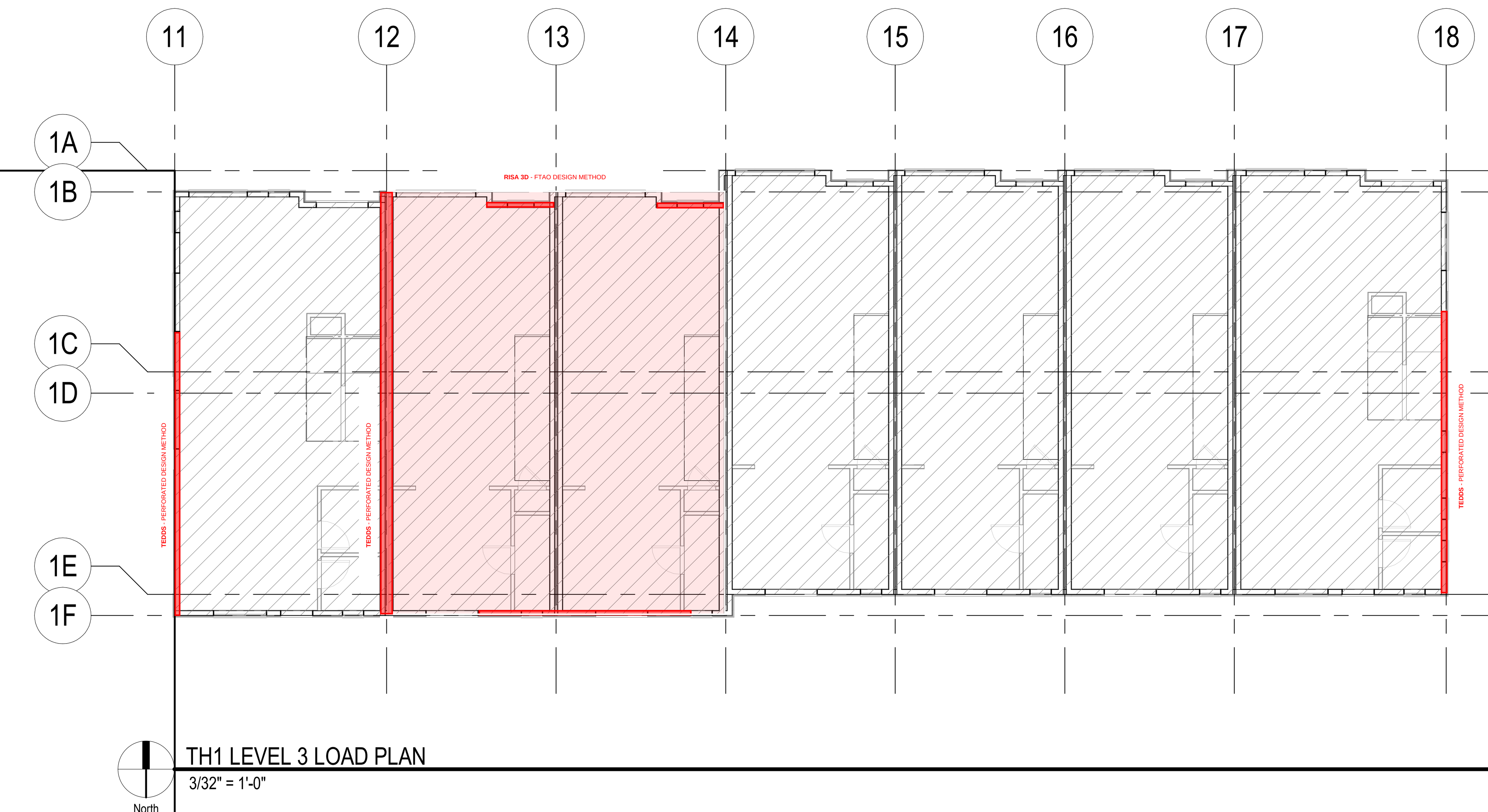


SHEAR WALL KEY PLAN:

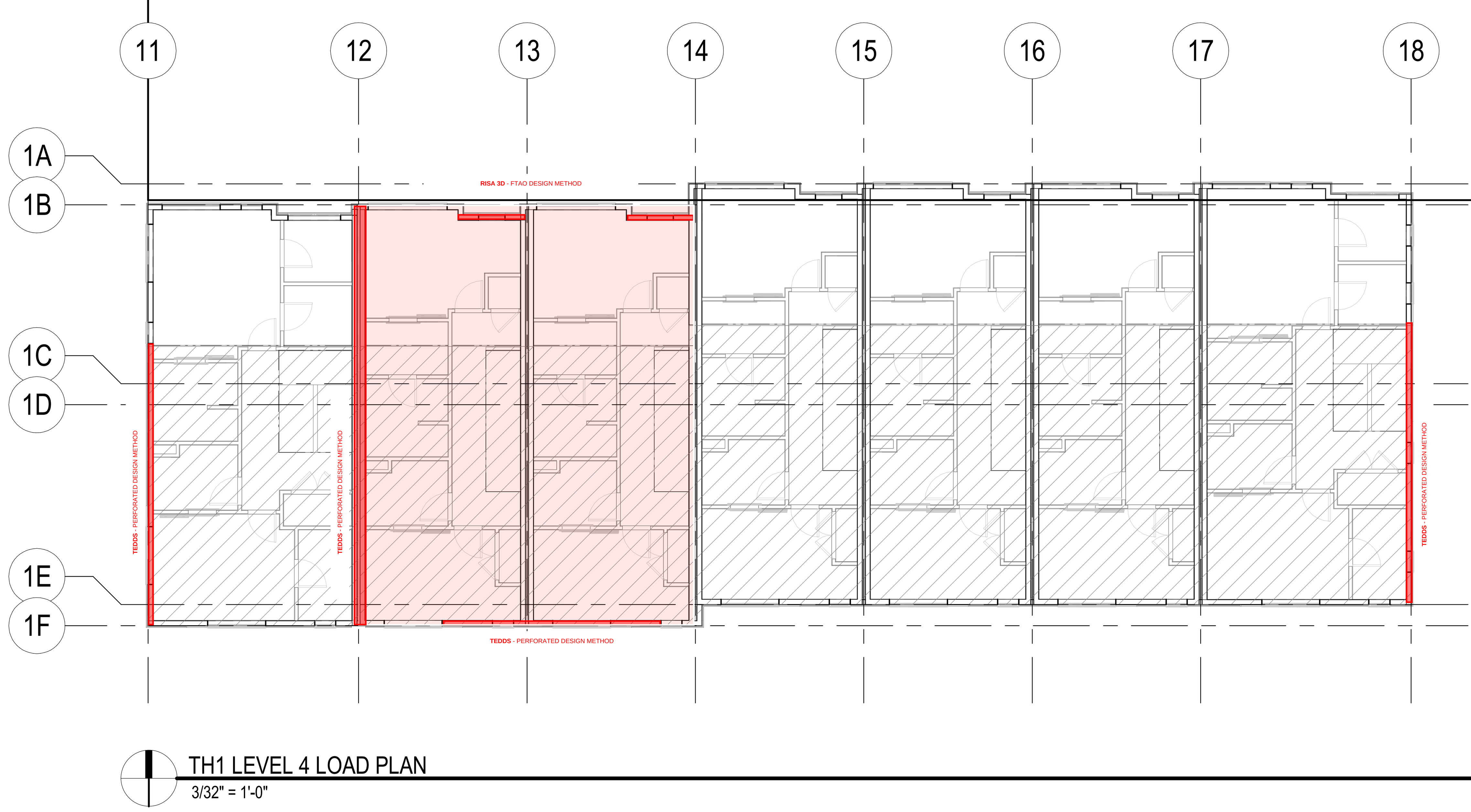
TH1



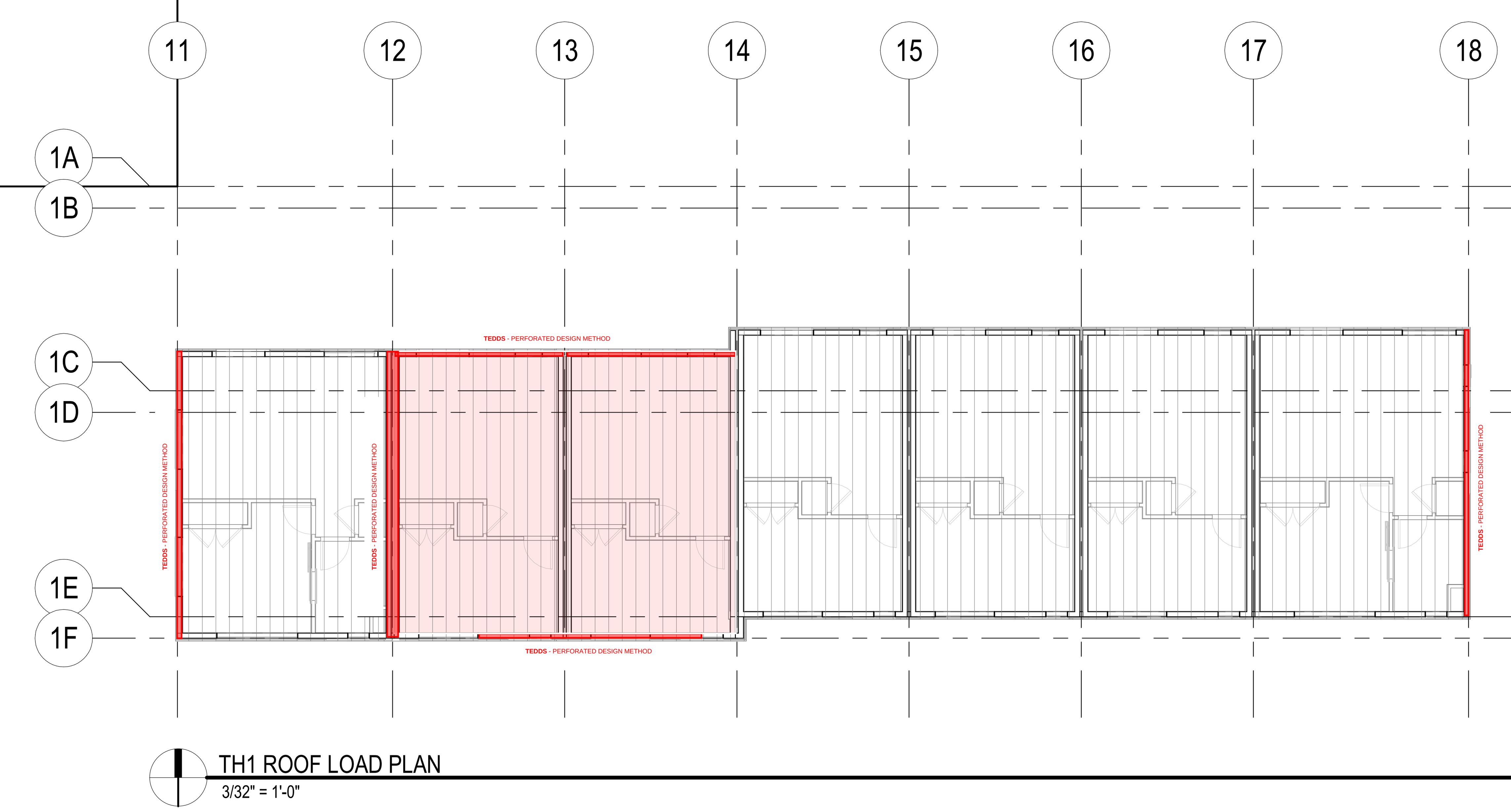
TH1 LEVEL 2 LOAD PLAN
3/32" = 1'-0"



TH1 LEVEL 3 LOAD PLAN
3/32" = 1'-0"



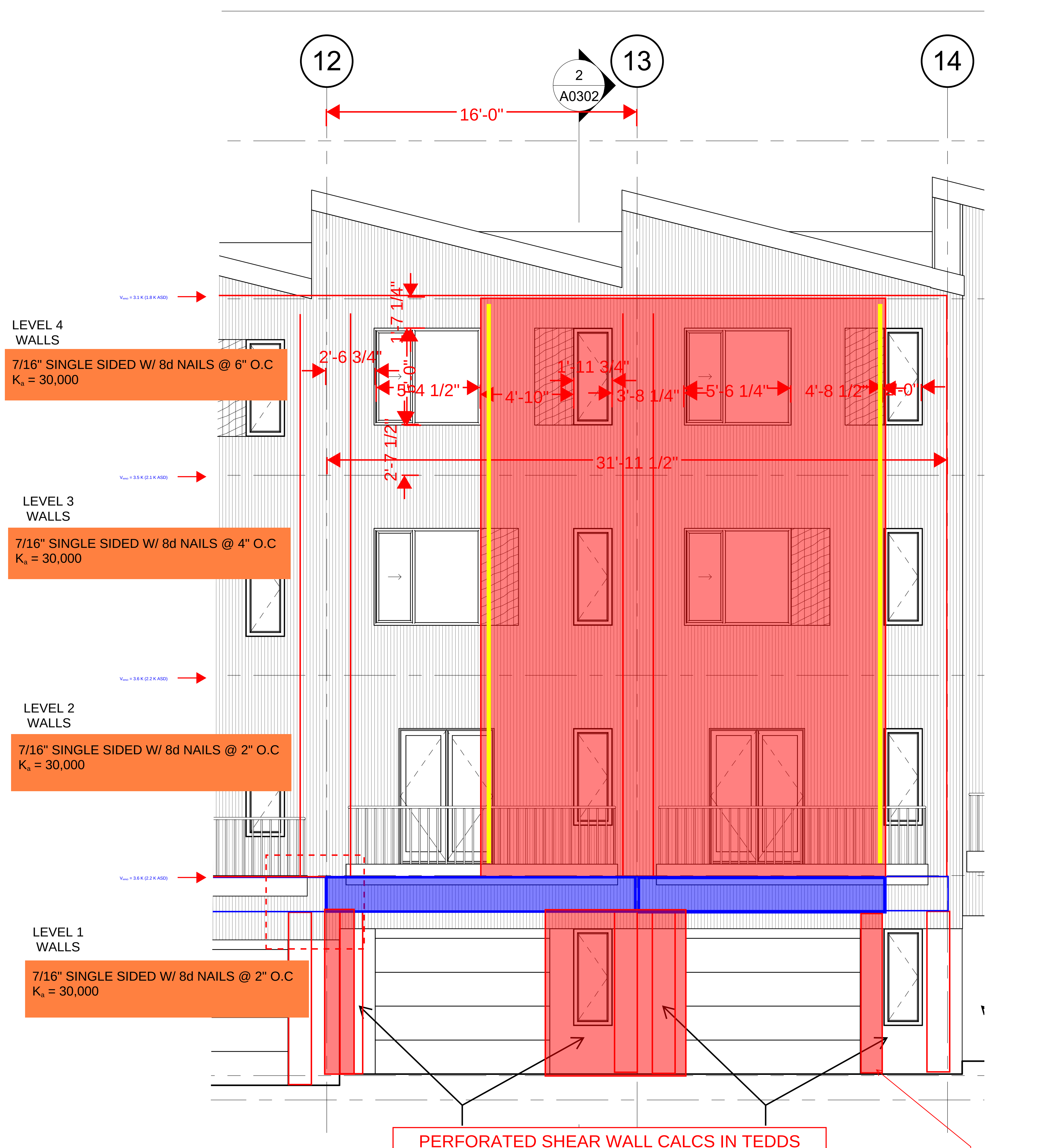
TH1 LEVEL 4 LOAD PLAN
3/32" = 1'-0"



TH1 ROOF LOAD PLAN
3/32" = 1'-0"

SHEAR WALL DESIGN SUMMARY:

NORTH AND SOUTH ELEVATIONS



SOUTH ELEVATION

NORTH ELEVATION

SEE "SHEAR WALL DESIGNS" SECTION FOR CALCULATIONS

WIND
LOADS

WIND LOADING FROM CODE SEARCH:

Wind Pressure Coefficients

	CASE A			CASE B		
		θ = 14 deg				
Surface	GCpf	w/-GCpi	w/+GCpi	GCpf	w/-GCpi	w/+GCpi
1	0.48	0.66	0.30	-0.45	-0.27	-0.63
2	-0.69	-0.51	-0.87	-0.69	-0.51	-0.87
3	-0.44	-0.26	-0.62	-0.37	-0.19	-0.55
4	-0.37	-0.19	-0.55	-0.45	-0.27	-0.63
5				0.40	0.58	0.22
6				-0.29	-0.11	-0.47
1E	0.72	0.90	0.54	-0.48	-0.30	-0.66
2E	-1.07	-0.89	-1.25	-1.07	-0.89	-1.25
3E	-0.63	-0.45	-0.81	-0.53	-0.35	-0.71
4E	-0.56	-0.38	-0.74	-0.48	-0.30	-0.66
5E				0.61	0.79	0.43
6E				-0.43	-0.25	-0.61

Ultimate Wind Surface Pressures (psf)

	Case A w/-GCpi	Case A w/+GCpi	Case B w/-GCpi	Case B w/+GCpi
1	16.2	7.3	-6.6	-15.5
2	-12.5	-21.4	-12.5	-21.4
3	-6.3	-15.1	-4.7	-13.5
4	-4.8	-13.6	-6.6	-15.5
5			14.2	5.4
6			-2.7	-11.5
1E	22.2	13.4	-7.4	-16.2
2E	-21.9	-30.7	-21.9	-30.7
3E	-11.0	-19.8	-8.6	-17.4
4E	-9.2	-18.1	-7.4	-16.2
5E			19.4	10.6
6E			-6.1	-15.0

Parapet

Windward parapet = 34.4 psf (GCpn = +1.5)
Leeward parapet = -22.9 psf (GCpn = -1.0)

Windward roof overhangs = 17.2 psf (upward) add to windward roof pressure

Horizontal MWFRS Simple Diaphragm Pressures (psf)

Transverse direction (normal to L)

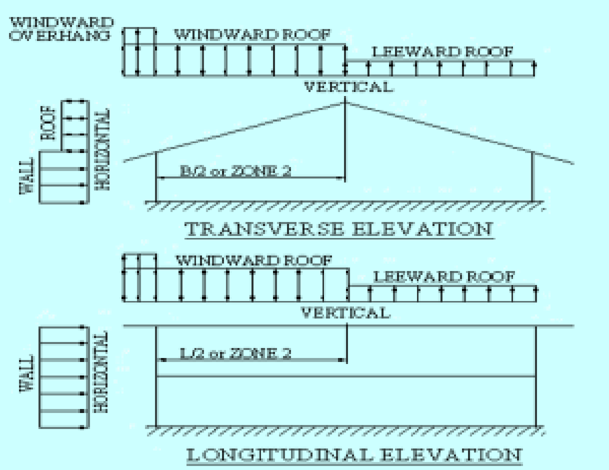
Interior Zone: Wall 20.9 psf
Roof -6.2 psf **
End Zone: Wall 31.5 psf
Roof -10.9 psf **

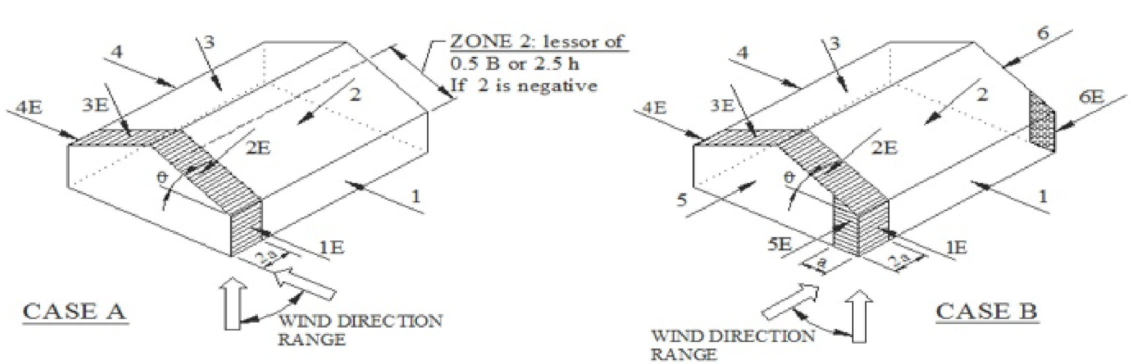
Longitudinal direction (parallel to L)

Interior Zone: Wall 17.0 psf
End Zone: Wall 25.6 psf

** NOTE: Total horiz force shall not be less than that determined by neglecting roof forces (except for MWFRS moment frames).

The code requires the MWFRS be designed for a min ultimate force of 16 psf multiplied by the wall area plus an 8 psf force applied to the vertical projection of the roof.

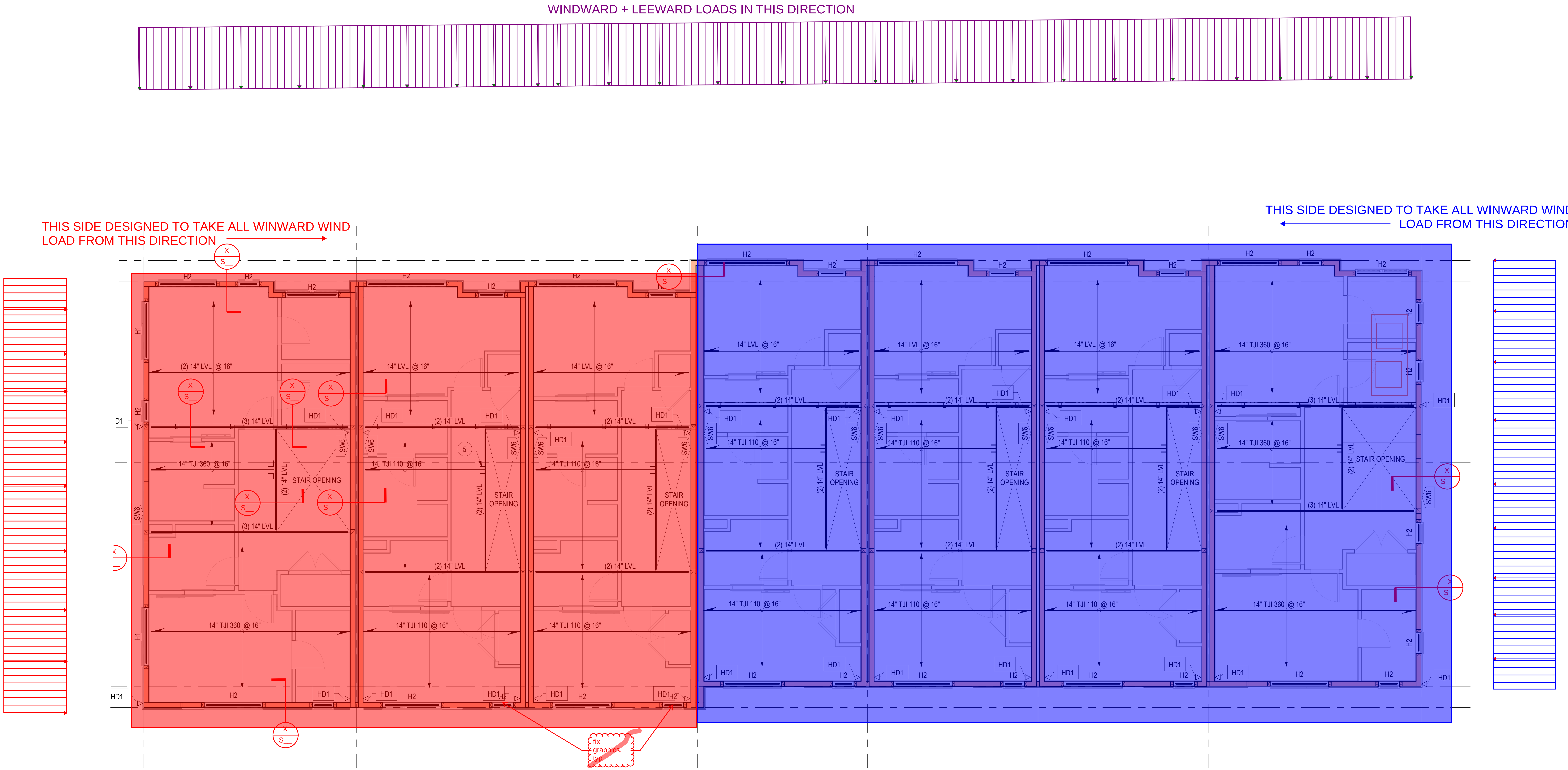




NOTE: Torsional loads are 25% of zones 1 - 6. See code for loading diagram.
Exception: One story buildings h<30' and 1 to 2 storybuildings framed with light-frame construction or with flexible diaphragms need not be designed for the torsional load case.

ASCE 7-98 & ASCE 7-10 (& later) - MWFRS wind pressure zones

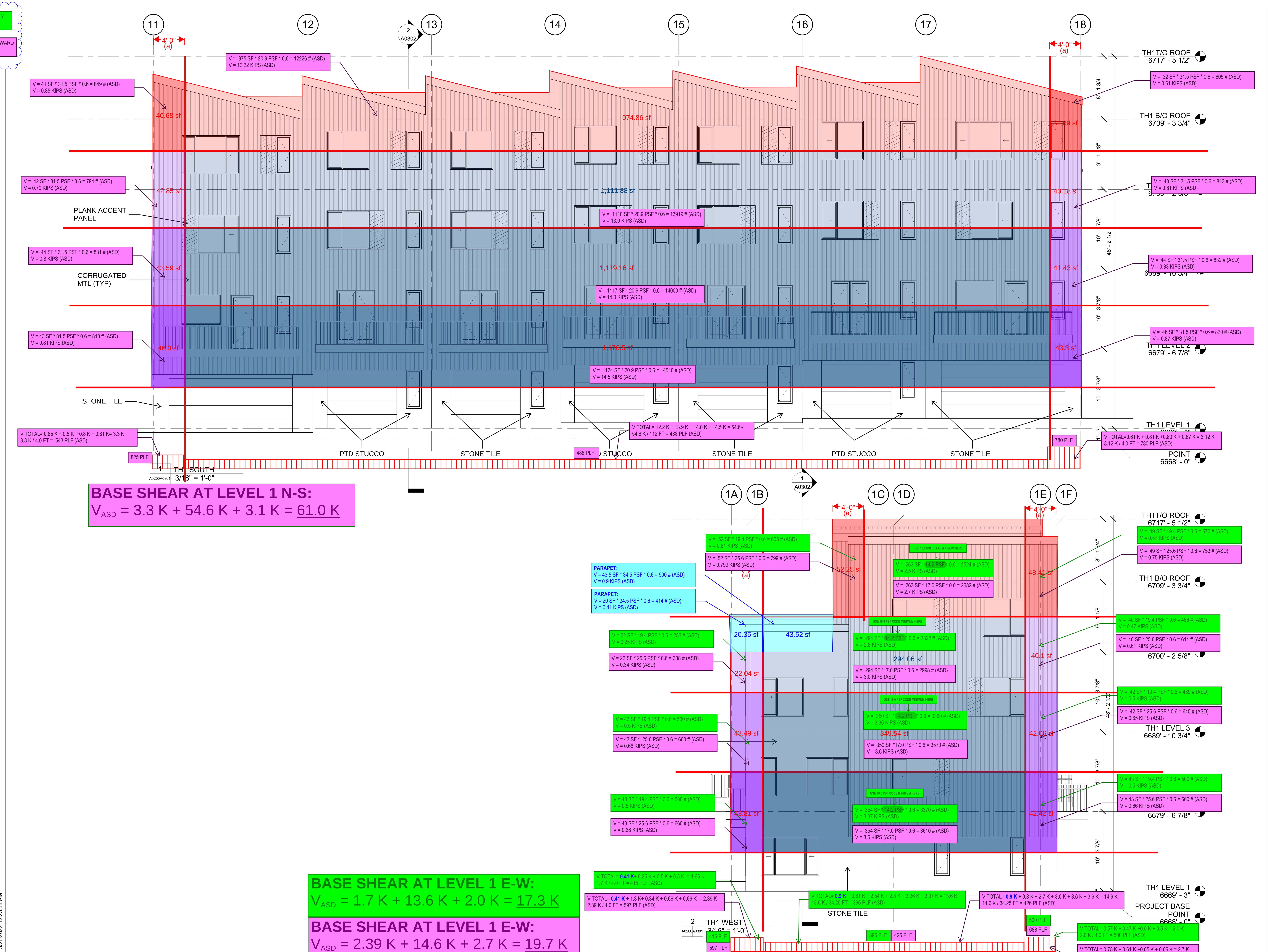
WIND LOAD JUSTIFICATION:



WIND:

WINDWARD ONLY
K_z=0.7855

WINDWARD + LEEWARD
K_z=0.7855



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APPROVAL STAMPS:

No.	Date	Description

SUBMISSIONS & REVISIONS

OWNER

MAY RIEGLER PROPERTIES
2201 WISCONSIN AVE NW
SUITE 200
WASHINGTON, DC 20007

ARCHITECT

K A S A

KEVIN & ASAKO SPERRY ARCHITECTURE
3318 N. Columbus Street
Arlington, VA 22207
T.312.636.3248 / 312.636.4252
www.kasa-arch.com

GENERAL CONTRACTOR

CIVIL ENGINEER

LANDMARK CONSULTANTS, INC.
141 9TH STREET
PO BOX 774943
STEAMBOAT SPRINGS, CO 80477

LANDSCAPE ARCHITECT

STRUCTURAL ENGINEER

ANTHEM STRUCTURAL ENGINEERS
430 YAMPA ST
STEAMBOAT SPRINGS, CO 80487

M.E.P. & F.P. ENGINEERS

BOULDER ENGINEERING
1717 15TH STREET
BOULDER, CO 80302

INTERIOR DESIGNER:

PROJECT LOCATION

BASECAMP TOWNHOME
1950 CURVE COURT
STEAMBOAT SPRINGS, CO 80487

DRAWING TITLE

TH1 SOUTH & WEST ELEVATIONS

SEAL

DATE: 05/27/22

DRAWN BY:

CHECKED BY:

PROJECT NO:

DRAWING NO:

A0301

WIND LOAD DISTRIBUTION
TH1

SEISMIC LOADS

REPORT SUMMARY

Seismic

S_S	0.597
S_1	0.103
F_a	1.261
F_v	1.5
S_{MS}	0.753
S_{M1}	0.155
S_{DS}	0.502
S_{D1}	0.103
T_L	4
PGA	0.419
PGA_M	0.502
F_{PGA}	1.2
I_e	1
C_v	0.998
Seismic Design Category	D

PER ROUTT COUNTY POLICY: (SEE NEXT PAGE)

$S_{DS\ MAX} = 0.333$ (NEED TO UPDATE F_a IN CODE SEARCH)

$S_{D1\ MAX} = 0.133$ (S_{D1} OK)

3. Agricultural storage structures intended only for incidental human occupancy.
4. Structures that require special consideration of their response characteristics and environment that are not addressed by this code or ASCE 7 and for which other regulations provide seismic criteria, such as vehicular bridges, electrical transmission towers, hydraulic structures, buried utility lines and their appurtenances and nuclear reactors.

Routt County Building Department Local Policy Amendment to Section 1613 Earth quake Loads: All properties within Routt County Incorporated and Unincorporated Jurisdictions have been adopted and approved to be a Seismic Design Category C designation through our Building Code Adoption Approval Processes. Structures shall be designed in accordance with our local amendment policy using a Seismic Design Category C designation as the base level design standard. When approved by the Structural Engineer of Record through review of the Geotechnical Soils Report and Soils Site Class, the Seismic Category may be reduced by the Engineer of Record based on the known Soils Site Class and in accordance with ASCE-7 and Chapter 16 of the IBC.

Structural Engineers Acceptable Design Parameters Local Routt County Building Department Policy: The Routt County Building Department has developed these design parameters to align with our Local Code Adoptions that were approved designating all of Routt County a Seismic Design Category C. This Policy has been created to provide maximum values for SDS and SD1 respectively to be used in the mapped areas throughout Routt County that have been designated Seismic Category D in accordance ASCE 7-16 USGS Seismic Design Data Map found at https://seismicmaps.org/. The parameters below may be used by Structural Engineers based on the Risk Factor of the Building to perform calculations to determine structural designs. The below parameters may be used with Site Class D- Default (See Section 11.4.3) being set on the ASCE 7-16 USGS Seismic Design Data Map found at https://seismicmaps.org/. Lower values may be used if justified by soil Site Class and resulting site-specific ground motion parameters set forth in ASCE 7-16 and USGS Seismic Design Data Map and approved by the Code Official.

- **Risk Category I, II, and II Building: SDS = 0.333 and SD1 = 0.133**
- **Risk Category IV Building: SDS = 0.499 and SD1 = 0.199**

The intent of setting these parameters and values is to help support Structural Engineers in designing buildings within the spirt of our Locally Approved Code Adoptions designating a standard Seismic Design Category C throughout all of Routt County, to avoid conflicts in what data would otherwise be provided through ASCE 7-16 USGS Seismic Design Data Map found at https://seismicmaps.org/.

Routt County Regional Building Department 2018 IRC Code Adoption

Table R301.2(1) CLIMATIC AND GEOGRAPHIC DESIGN CRITERIA, is completed as follows:

- Ground Snow Load – Case Study Area contact the Building Department for Ground Snow Load Valuations per site.
- Climate Zone 7
- Wind Speed – 115 MPH (ultimate design wind speed)
- Topographic Effects – No
- Seismic Design Category – C Note: When approved by the Structural Engineer of Record through review of the Geotechnical Soils Report and Soils Site Class, the Seismic Category may be reduced

SEISMIC:

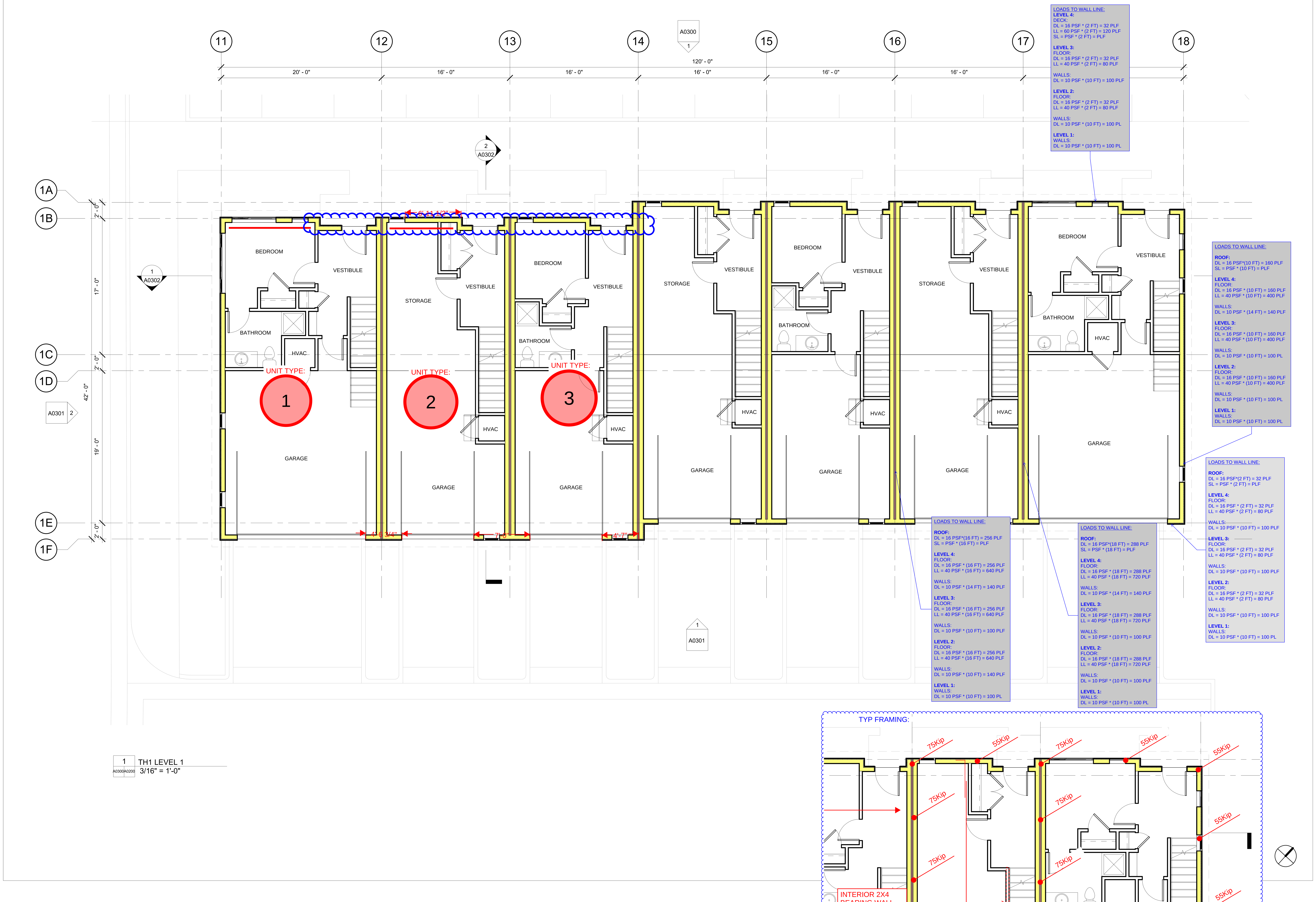
GRAVITY LOADS

WALLS:
DL = 10 PSF

BASE SHEAR AT LEVEL 1:

W= 52400 # +76864 # +76880 # +79064 # = 285.2 K
$V = C_S * W = 0.093 * W = 0.093 * 285.2\ K = \underline{26.5\ K}$

$V_{ASD} = 26.5\ K * 0.7 = \underline{18.6\ K\ (ASD)}$



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APPROVAL STAMPS:

No.	Date	Description

SUBMISSIONS & REVISIONS

OWNER

MAY RIEGLER PROPERTIES
2201 WISCONSIN AVE NW
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1717 15TH STREET
BOULDER, CO 80302

INTERIOR DESIGNER:

PROJECT LOCATION

BASECAMP TOWNHOME
1950 CURVE COURT
STEAMBOAT SPRINGS, CO 80487

DRAWING TITLE

TH1 LEVEL 1 PLAN

SEAL

DATE:
05/27/22

DRAWN BY:

CHECKED BY:

PROJECT NO:

DRAWING NO:
A0200

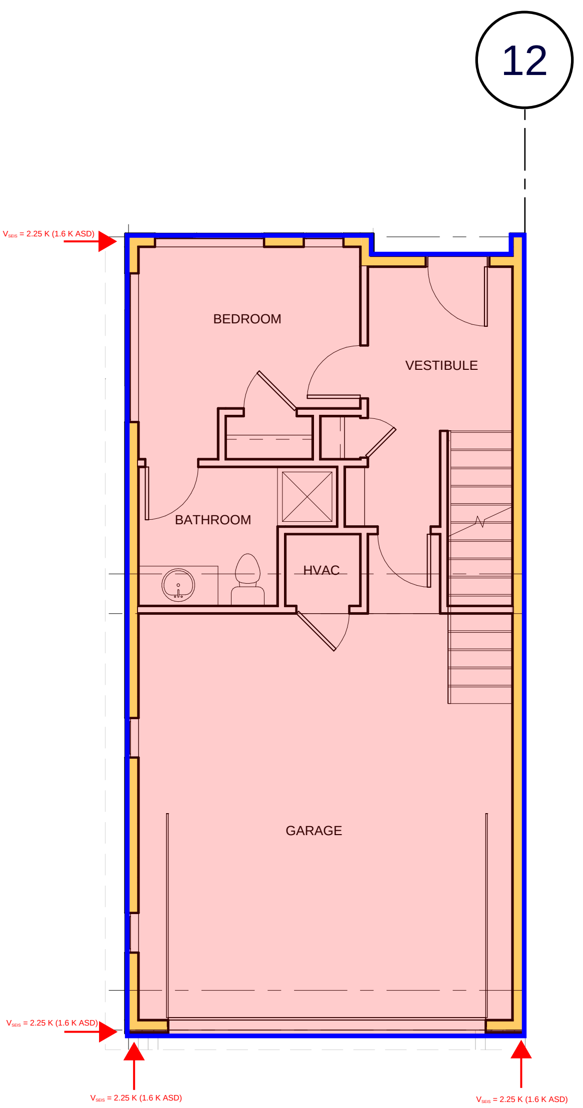
UNIT TYPE:

1

GRAVITY LOADS
BALCONY:
DL = 16 PSF
LL = 60PSF
SL =

FLOOR:
DL = 16 PSF
LL = 40 PSF

WALLS:
DL = 10 PSF

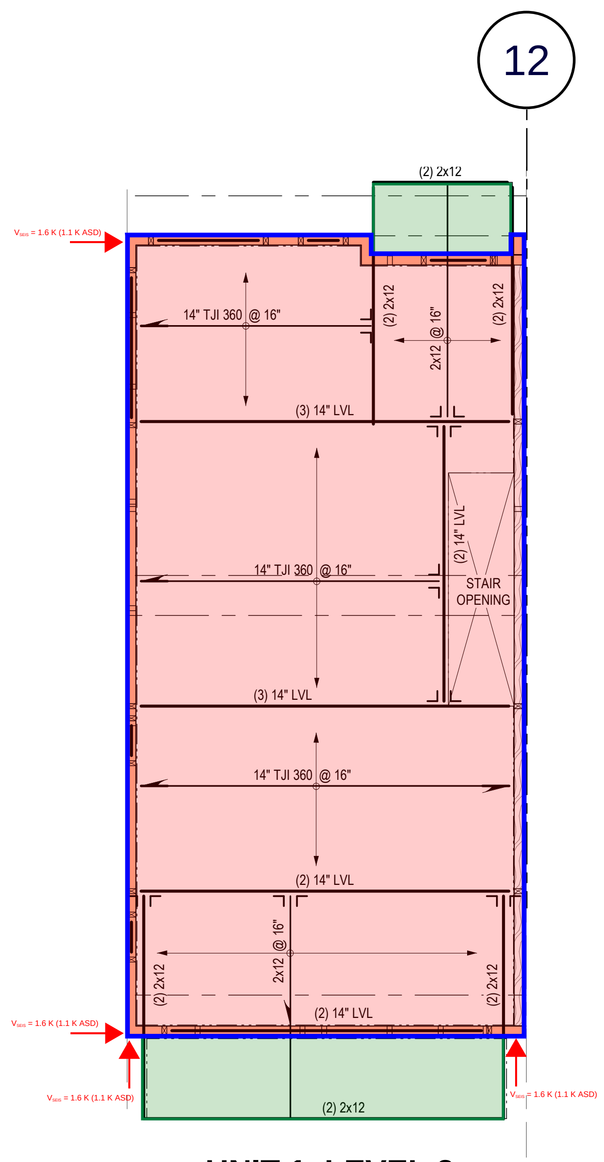


UNIT 1, LEVEL 1

$V_{1,1}$ = 4.5 K (3.2 K ASD)

BASE SHEAR AT LEVEL 1:
W= 14.4 K + 12.9 K + 12.9 K + 8.8 K = 49 K
V= C_S*W = 0.093*W = 0.093 * 49 K = 4.5 K

V_{ASD} = 4.5 K * 0.7 = 3.2 K (ASD)



UNIT 1, LEVEL 2

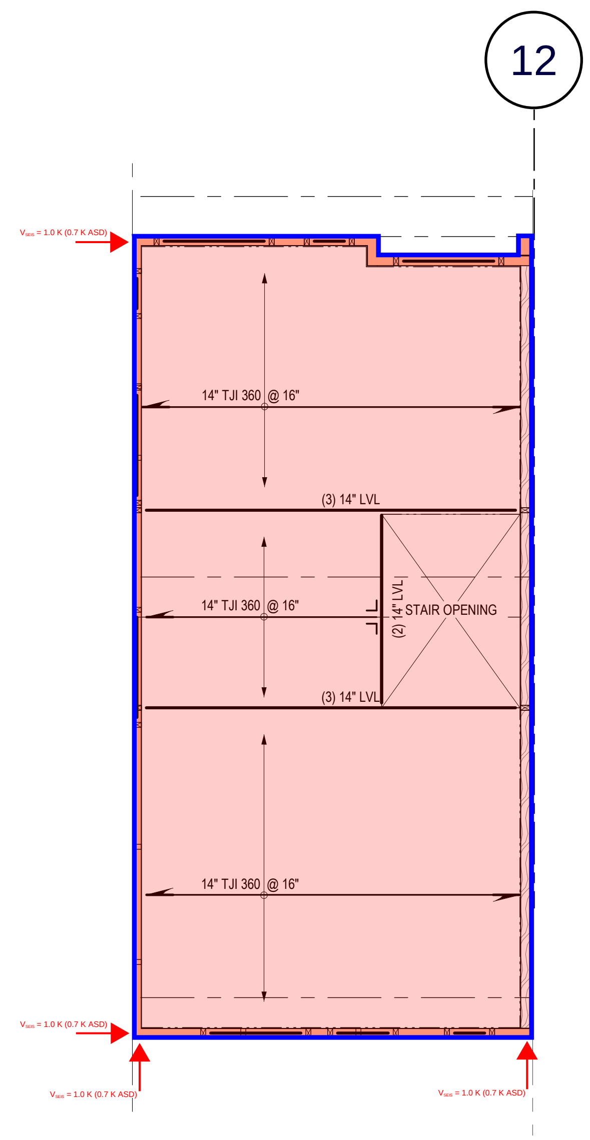
$V_{1,2}$ = 3.2 K (2.24 K ASD)

UNIT 1, LEVEL 2 SEISMIC WEIGHT:
FLOOR AREA = 805 SF
BALCONY AREA = 95 SF

W= (805 SF + 95 SF)* 16 PSF = 14.4 K

V = C_SW = 0.093W = 0.093*14.4 K = 1.34 K

SHEAR AT LEVEL 1 WALLS
V_{TOTAL}= 1.3 K + 1.2 K + 1.2 K + 0.82 K = 4.5 K



UNIT 1, LEVEL 3

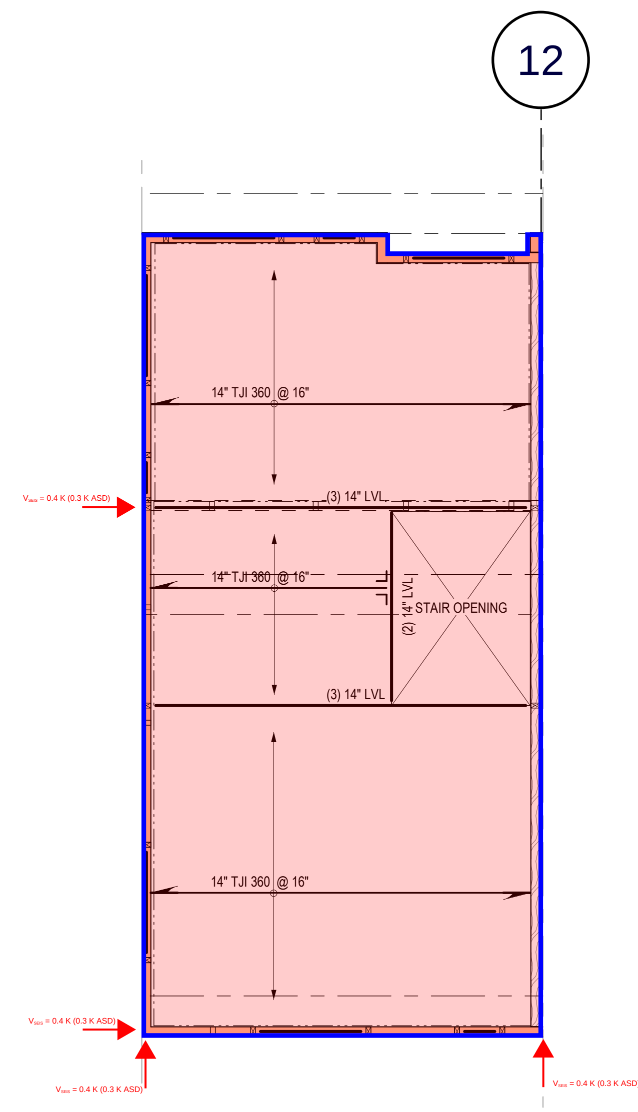
$V_{1,3}$ = 2.0 K (1.4 K ASD)

UNIT 1, LEVEL 3 SEISMIC WEIGHT:
FLOOR AREA = 805 SF

W= 805 SF* 16 PSF = 12.9 K

V = C_SW = 0.093W = 0.093*12.9 K = 1.2 K

SHEAR AT LEVEL 2 WALLS
V_{TOTAL}= 1.2 K + 1.2 K + 0.82 K = 3.2 K



UNIT 1, LEVEL 4

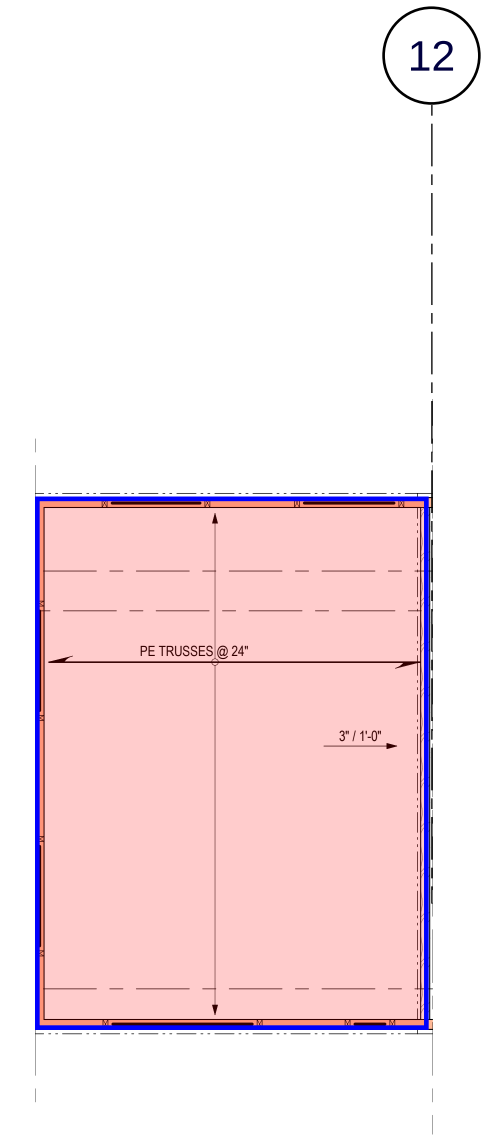
$V_{1,4}$ = 0.8 K (0.6 K ASD)

UNIT 1, LEVEL 4 SEISMIC WEIGHT:
FLOOR AREA = 540 SF
ROOF DECK = 265 SF

W= (540 SF + 265 SF)* 16 PSF = 12.9 K

V = C_SW = 0.093W = 0.093*12.9 K = 1.2 K

SHEAR AT LEVEL 3 WALLS
V_{TOTAL}= 1.2 K + 0.82 K = 2.0 K



UNIT 1, ROOF

UNIT 1, ROOF SEISMIC WEIGHT:
ROOF AREA = 550 SF

W= 550 SF* 16 PSF = 8.8 K

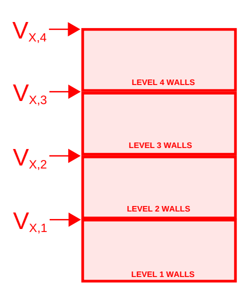
V = C_SW = 0.093W = 0.093*8.8 K = 0.82 K

SHEAR AT LEVEL 4 WALLS
V_{TOTAL}= 0.82 K

SEISMIC

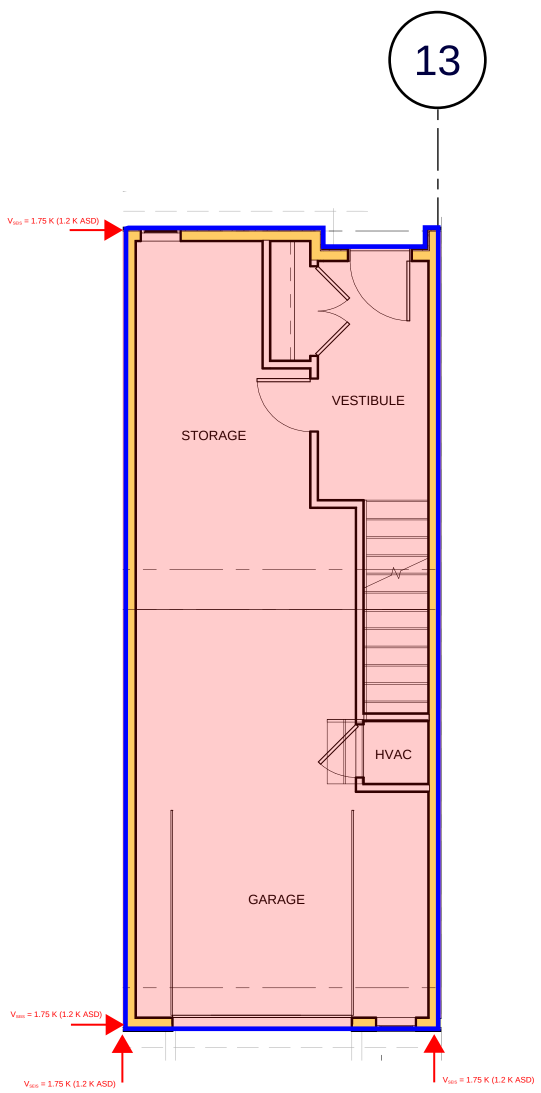
SEISMIC LOADS FOR UNIT 1:

$V = C_S W = 0.093W$



UNIT TYPE:

2

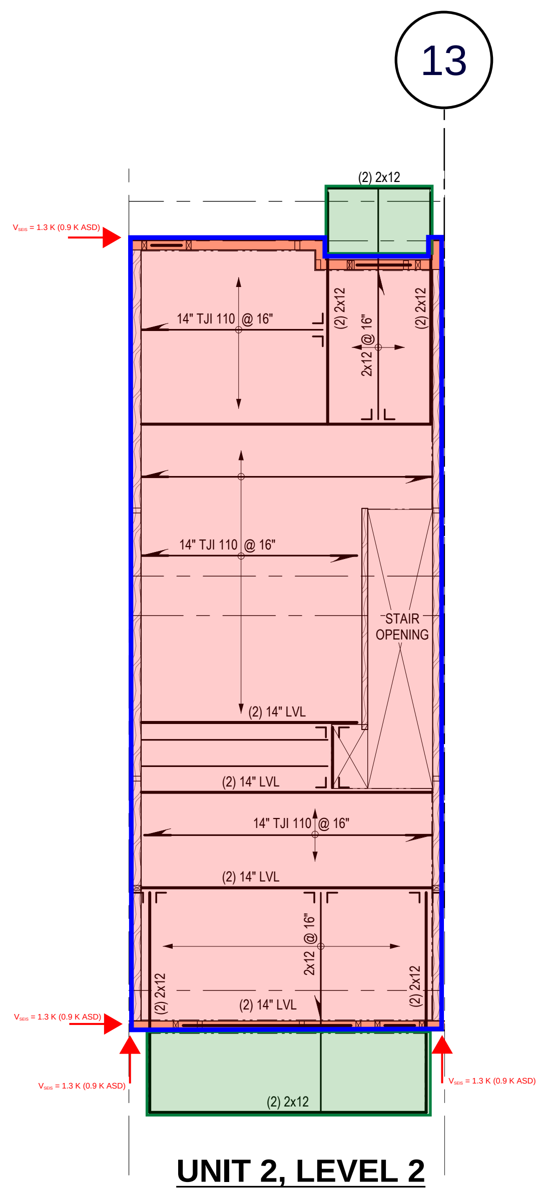


UNIT 2, LEVEL 1

$V_{2,1}$ = 3.5 K (2.45 K ASD)

BASE SHEAR AT LEVEL 1:
W=10.6 K +10.3 K + 10.2 K + 7.0 K = 38.1 K
V= C_S*W = 0.093*W = 0.093 * 38.1 K = <u>3.54 K</u>

V_{ASD} = 3.54 K * 0.7 = <u>2.5 K (ASD)</u>



UNIT 2, LEVEL 2

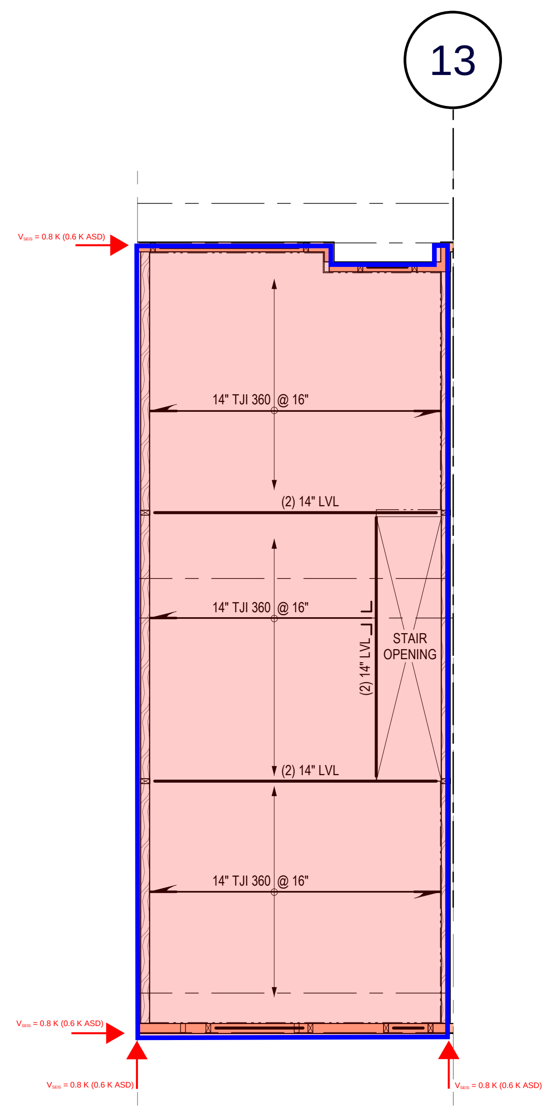
$V_{2,2}$ = 2.6 K (1.8 K ASD)

UNIT 2, LEVEL 2 SEISMIC WEIGHT:
FLOOR AREA = 645 SF
BALCONY AREA = 75 SF

W= (645 SF + 75 SF)* 16 PSF = 10.6 K

V = C_SW = 0.093W = 0.093*10.6 K = <u>0.98 K</u>

SHEAR AT LEVEL 1 WALLS
V_{TOTAL}= 0.98 K + 0.96 K + 0.95 K +0.65 K = <u>3.54 K</u>



UNIT 2, LEVEL 3

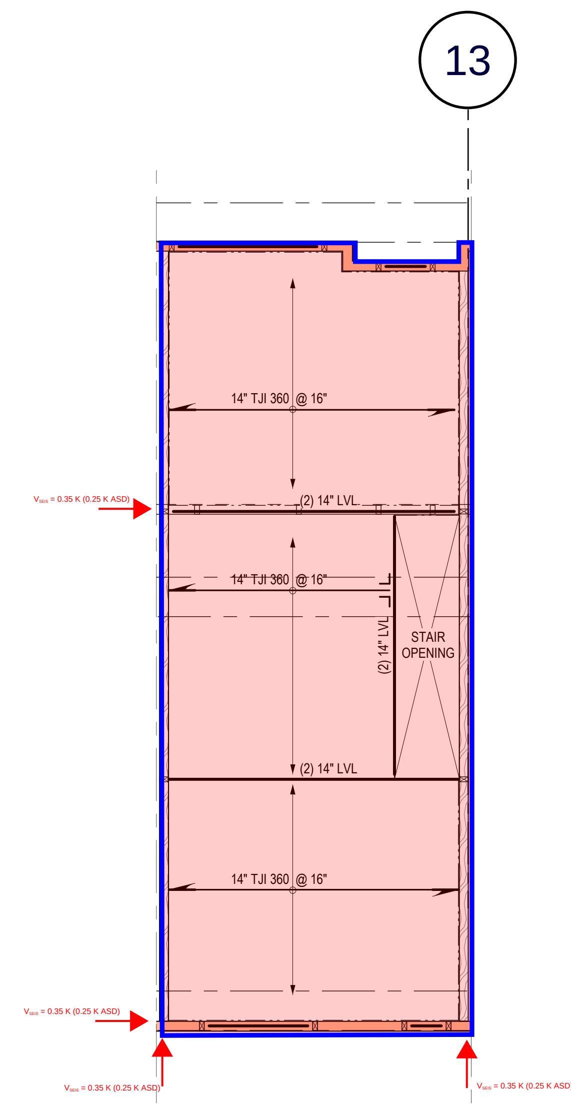
$V_{2,3}$ = 1.6 K (1.12 K ASD)

UNIT 2, LEVEL 3 SEISMIC WEIGHT:
FLOOR AREA = 645 SF

W= 645 SF* 16 PSF = 10.3 K

V = C_SW = 0.093W = 0.093*10.3 K = <u>0.96 K</u>

SHEAR AT LEVEL 2 WALLS
V_{TOTAL}= 0.96 K + 0.95 K +0.65 K = <u>2.56 K</u>



UNIT 2, LEVEL 4

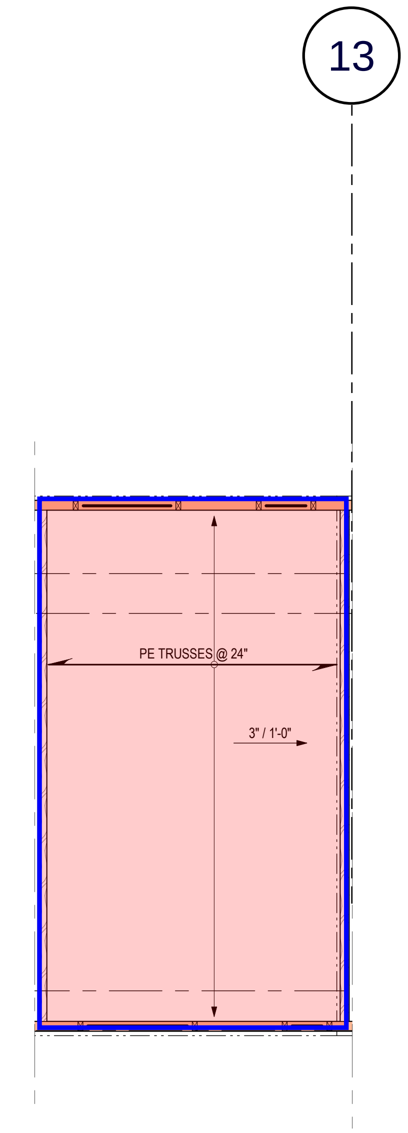
$V_{2,4}$ = 0.7 K (0.5 K ASD)

UNIT 2, LEVEL 4 SEISMIC WEIGHT:
FLOOR AREA = 430 SF
ROOF DECK = 210 SF

W= (430 SF + 210 SF)* 16 PSF = 10.2 K

V = C_SW = 0.093W = 0.093*10.2 K = <u>0.95 K</u>

SHEAR AT LEVEL 3 WALLS
V_{TOTAL}= 0.95 K +0.65 K = <u>1.6 K</u>



UNIT 2, ROOF

UNIT 2, ROOF SEISMIC WEIGHT:
ROOF AREA = 435 SF

W= 435 SF* 16 PSF = 7.0 K

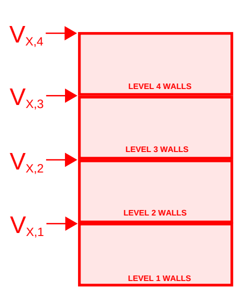
V = C_SW = 0.093W = 0.093*7.0 K = <u>0.65 K</u>

SHEAR AT LEVEL 4 WALLS
V_{TOTAL}= <u>0.65 K</u>

SEISMIC

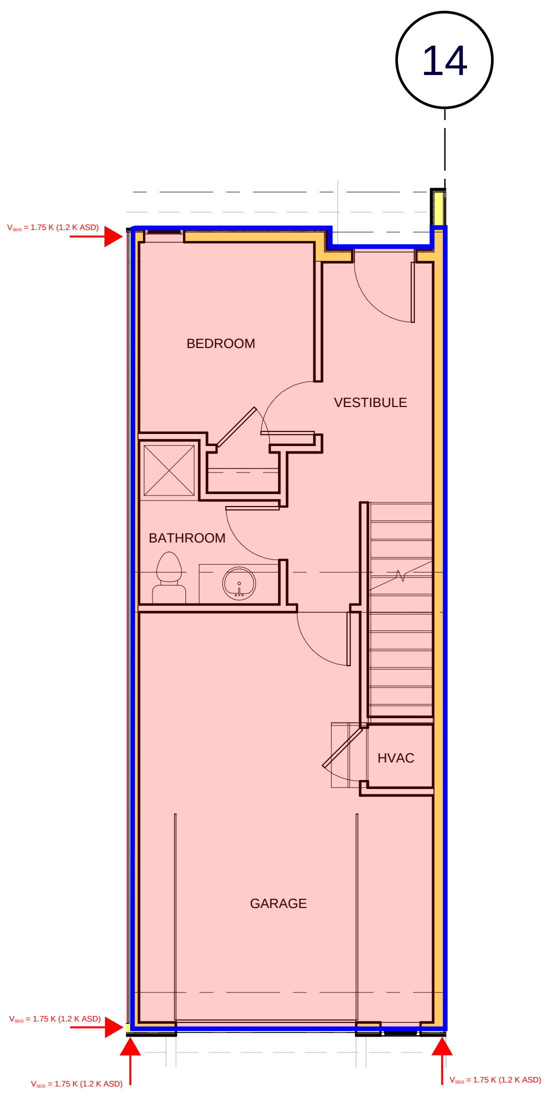
SEISMIC LOADS FOR UNIT 2:

$V = C_SW = 0.093W$



UNIT TYPE:

3

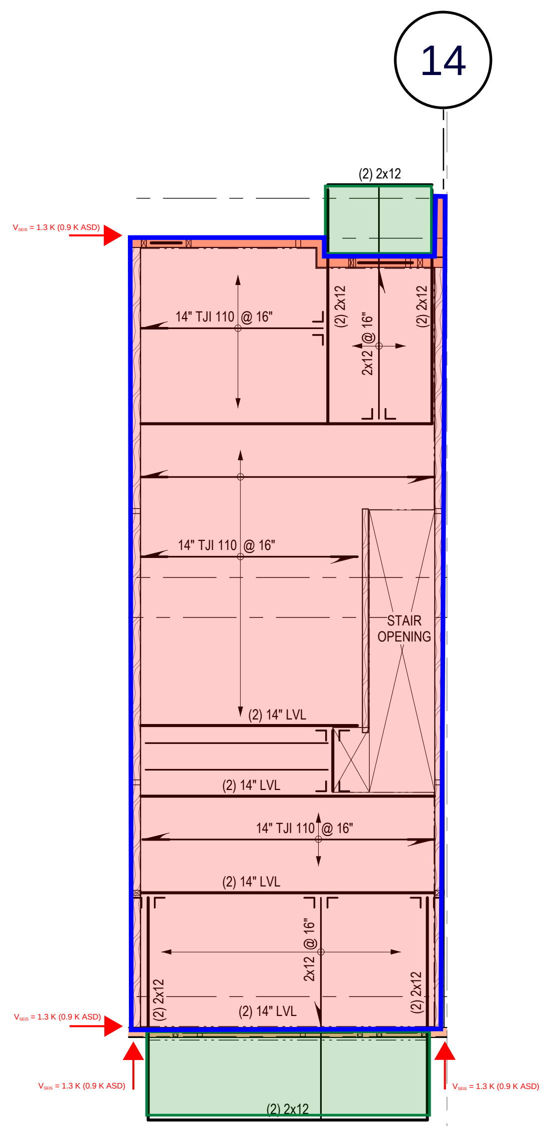


UNIT 3, LEVEL 1

$V_{3,1}$ = 3.5 K (2.45 K ASD)

BASE SHEAR AT LEVEL 1:
W=10.6 K +10.3 K + 10.2 K + 7.0 K = 38.1 K
V= C_S*W = 0.093*W = 0.093 * 38.1 K = 3.54 K

V_{ASD} = 3.54 K * 0.7 = 2.5 K (ASD)



UNIT 3, LEVEL 2

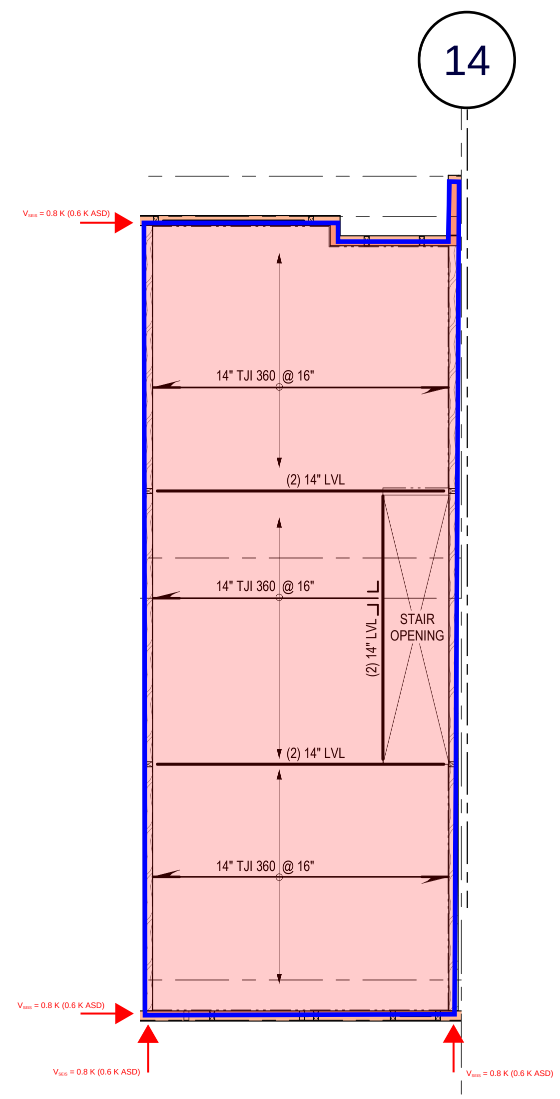
$V_{3,2}$ = 2.6 K (1.8 K ASD)

UNIT 3, LEVEL 2 SEISMIC WEIGHT:
FLOOR AREA = 645 SF
BALCONY AREA = 75 SF

W = (645 SF + 75 SF)* 16 PSF = 10.6 K

V = C_SW = 0.093W = 0.093*10.6 K = 0.98 K

SHEAR AT LEVEL 1 WALLS
V_{TOTAL}= 0.98 K + 0.96 K + 0.95 K +0.65 K = 3.54 K



UNIT 3, LEVEL 3

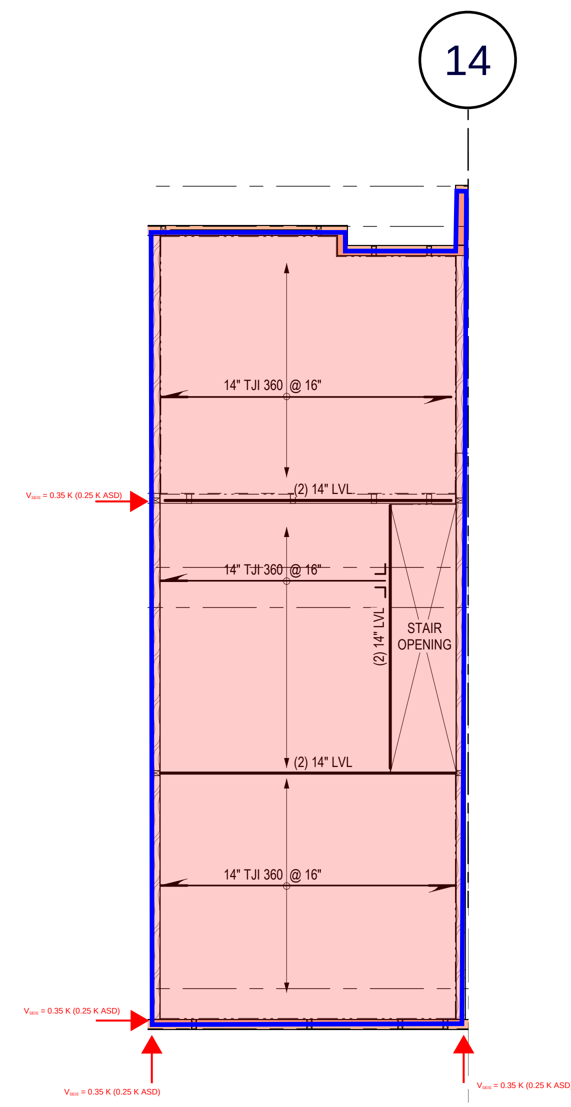
$V_{3,3}$ = 1.6 K (1.12 K ASD)

UNIT 3, LEVEL 3 SEISMIC WEIGHT:
FLOOR AREA = 645 SF

W= 645 SF* 16 PSF = 10.3 K

V = C_SW = 0.093W = 0.093*10.3 K = 0.96 K

SHEAR AT LEVEL 2 WALLS
V_{TOTAL}= 0.96 K + 0.95 K +0.65 K = 2.56 K



UNIT 3, LEVEL 4

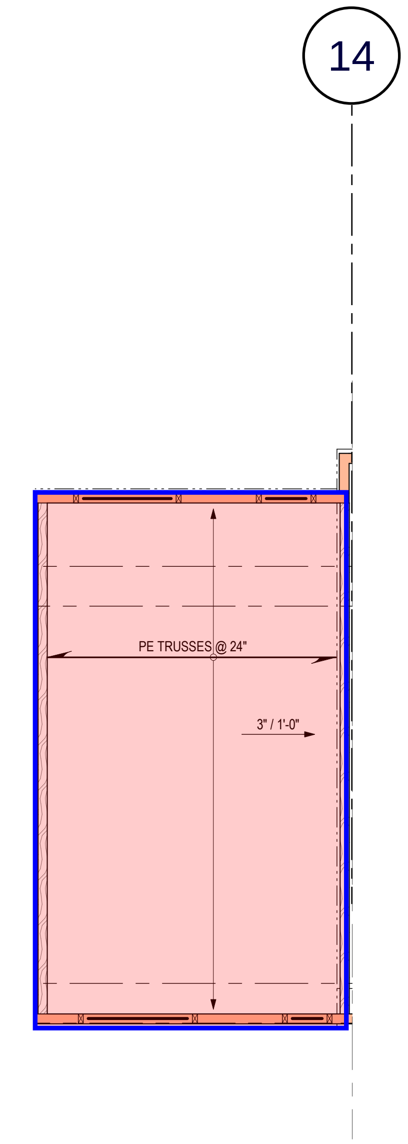
$V_{3,4}$ = 0.7 K (0.5 K ASD)

UNIT 3, LEVEL 4 SEISMIC WEIGHT:
FLOOR AREA = 430 SF
ROOF DECK = 210 SF

W= (430 SF + 210 SF)* 16 PSF = 10.2 K

V = C_SW = 0.093W = 0.093*10.2 K = 0.95 K

SHEAR AT LEVEL 3 WALLS
V_{TOTAL}= 0.95 K +0.65 K = 1.6 K



UNIT 3, ROOF

UNIT 3, ROOF SEISMIC WEIGHT:
ROOF AREA = 435 SF

W= 435 SF* 16 PSF = 7.0 K

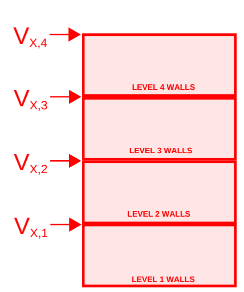
V = C_SW = 0.093W = 0.093*7.0 K = 0.65 K

SHEAR AT LEVEL 4 WALLS
V_{TOTAL}= 0.65 K

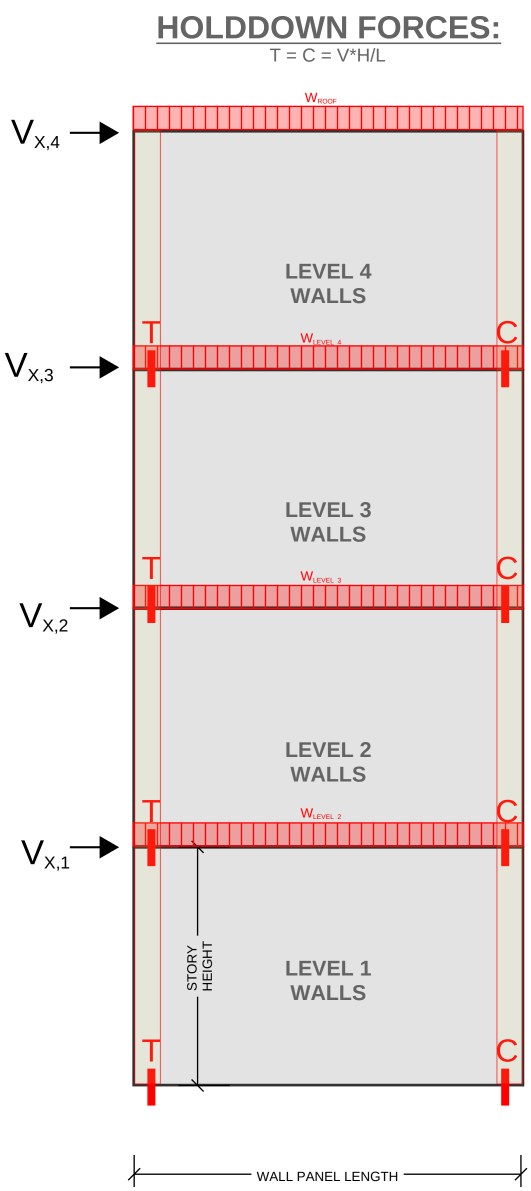
SEISMIC

SEISMIC LOADS FOR UNIT 3:

$V = C_SW = 0.093W$



LATERAL DESIGNS

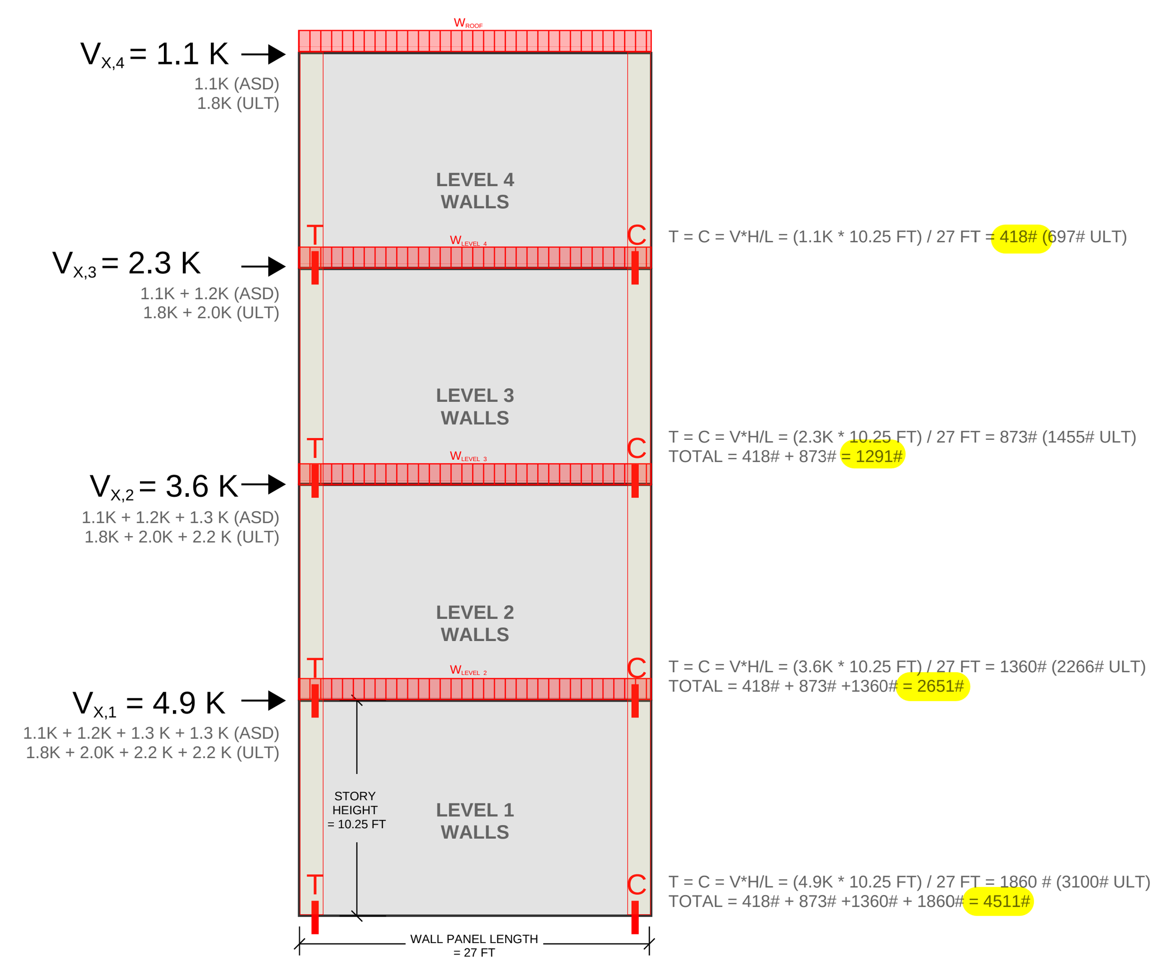


OVERTURNING FORCES:

TH1

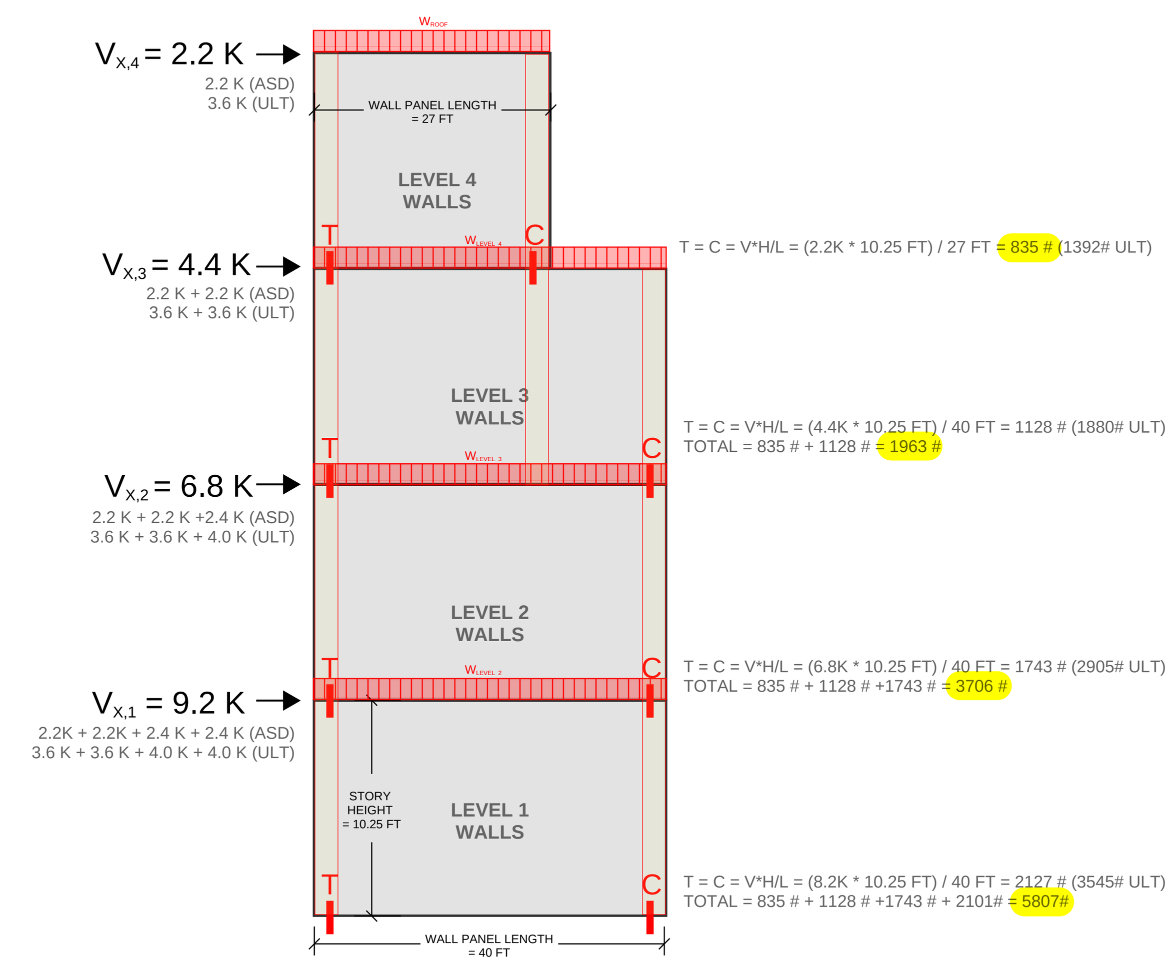
N-S WALLS

GRIDS 11, 18
HOLDDOWN FORCES:



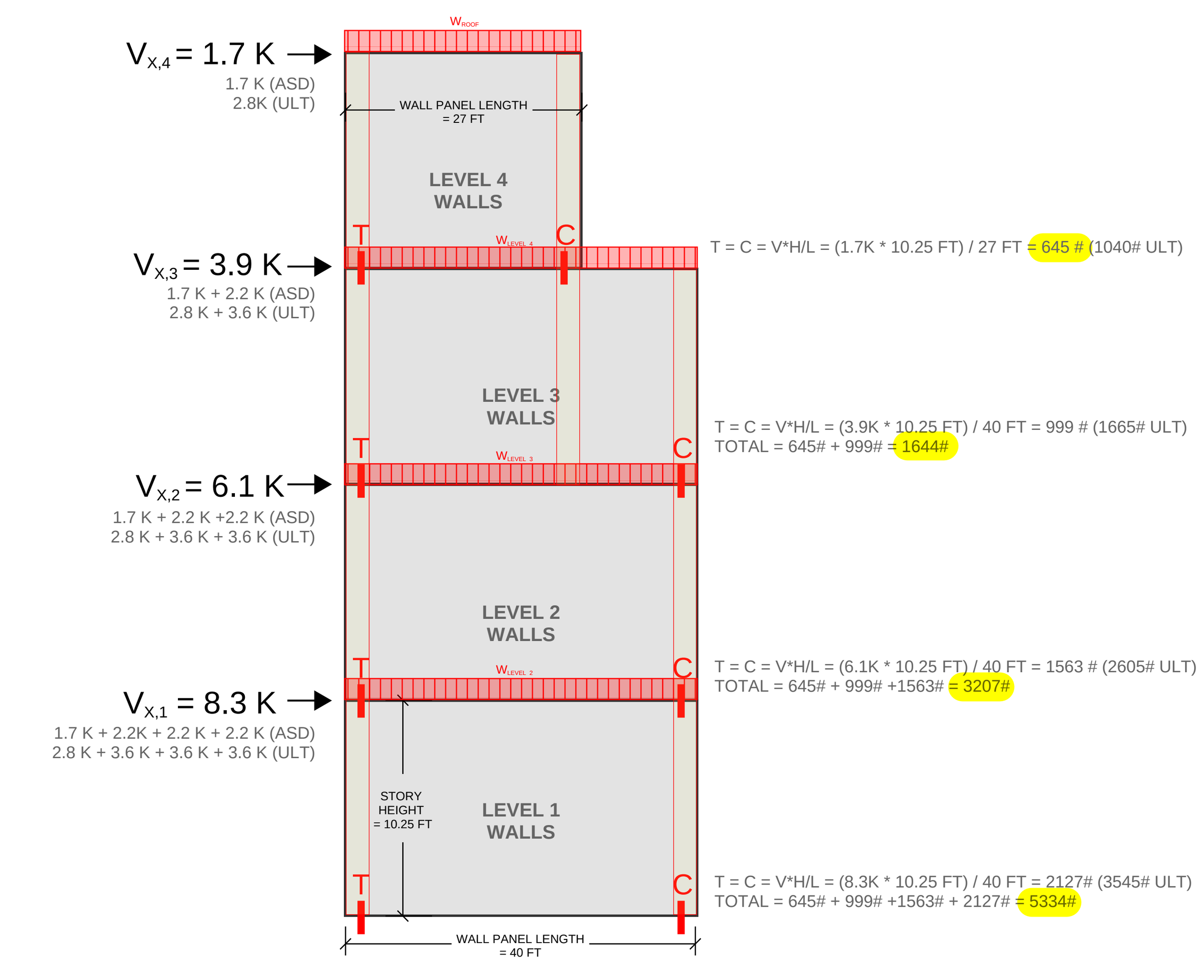
(PERFORATED - TEDDS)

GRIDS 12, 17
HOLDDOWN FORCES:



(PERFORATED - TEDDS)

GRIDS 13 - 16
HOLDDOWN FORCES:

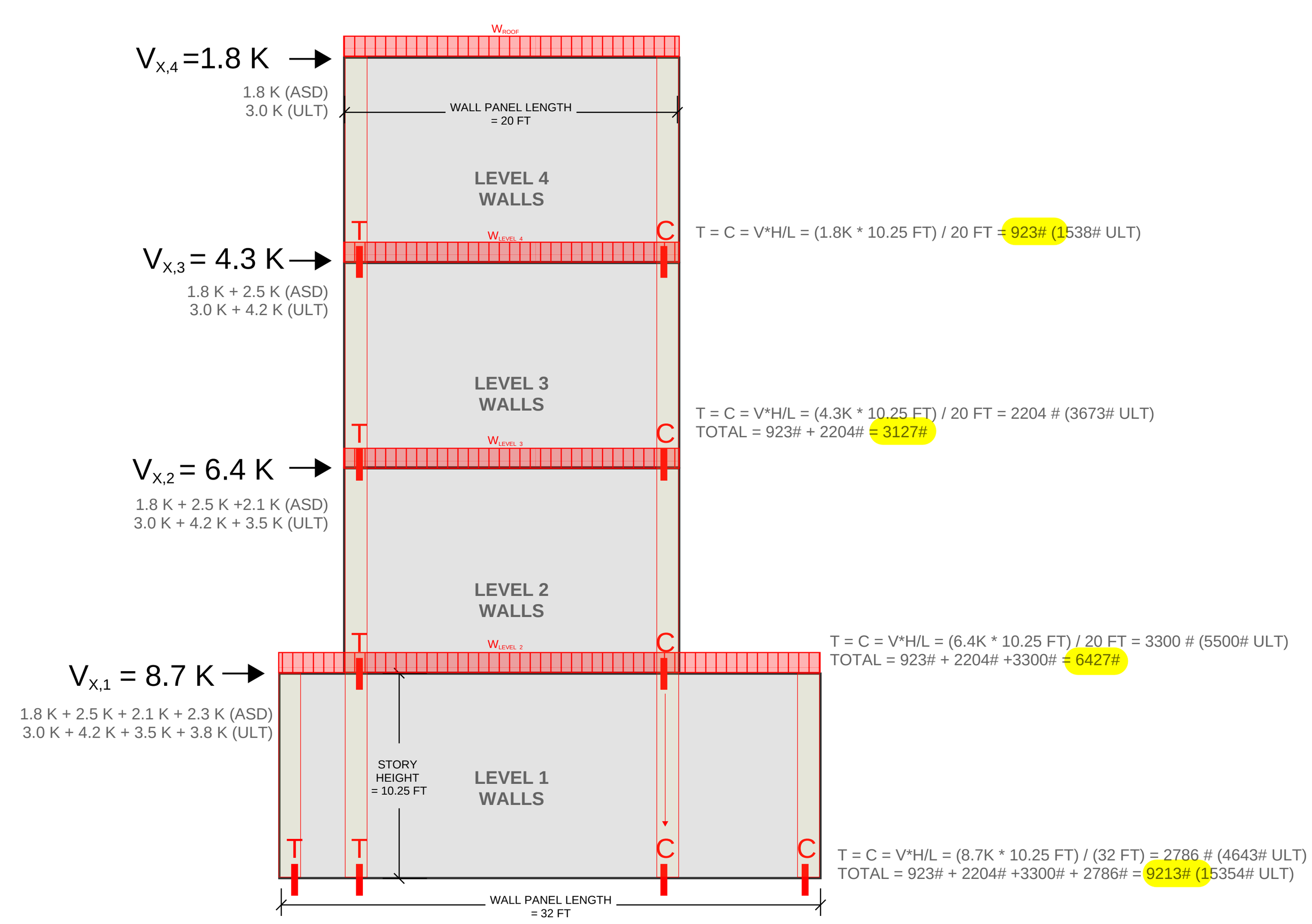


(PERFORATED - TEDDS)

E-W WALLS

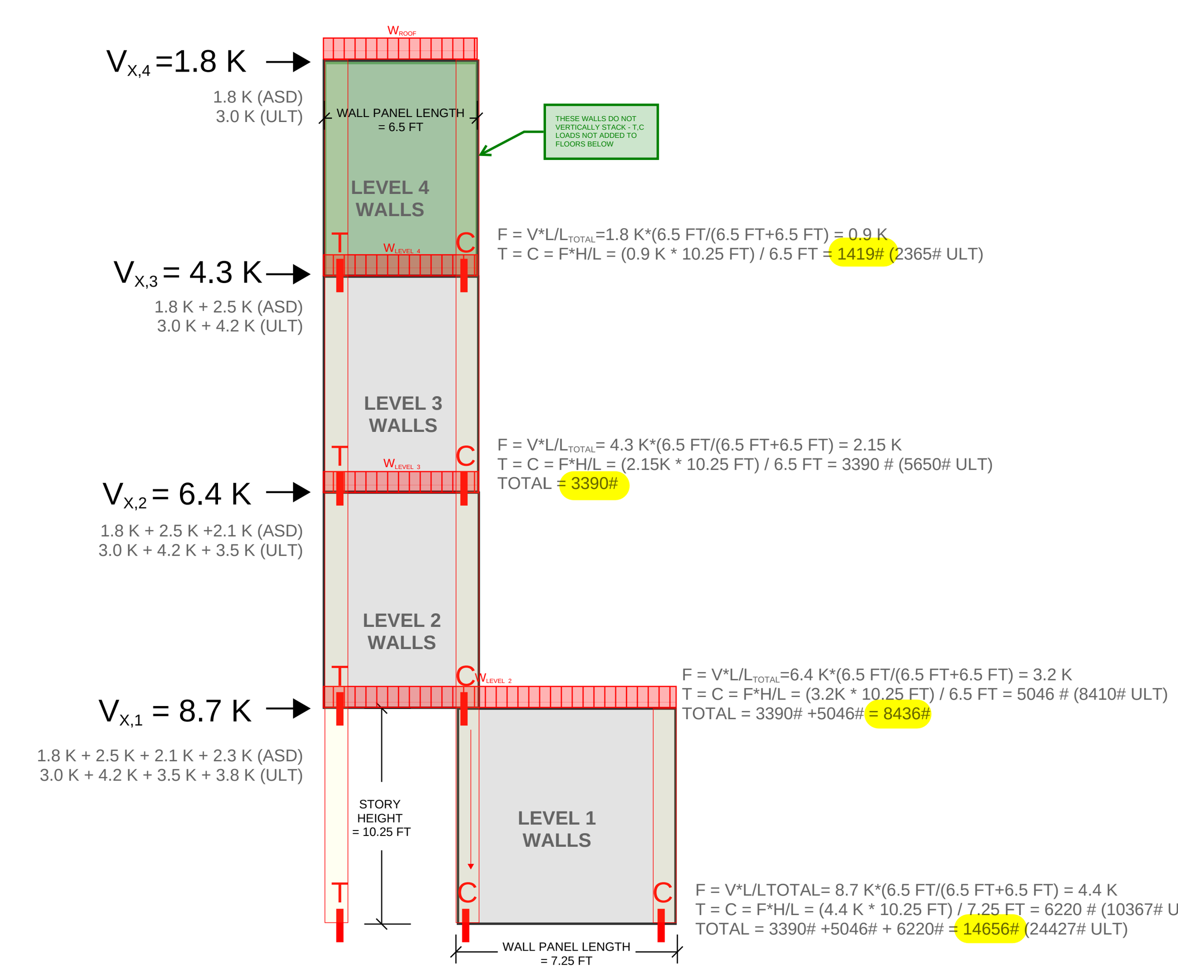
SEE "WIND LOADS" FOR LOAD DEVELOPMENT

GRIDS 1E, 1F
HOLDDOWN FORCES:



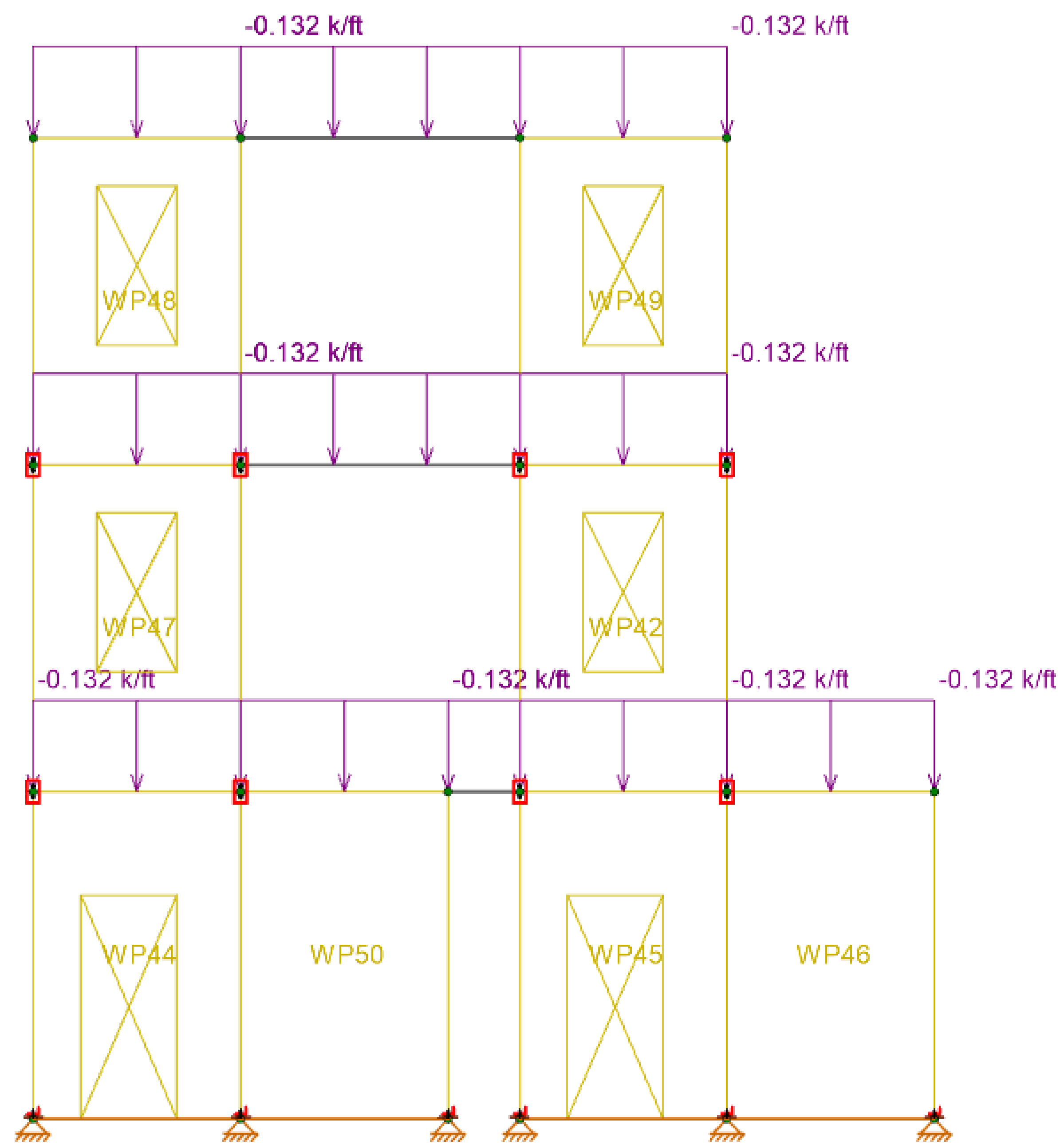
(PERFORATED - TEDDS)

GRIDS 1A, 1B
HOLDDOWN FORCES:

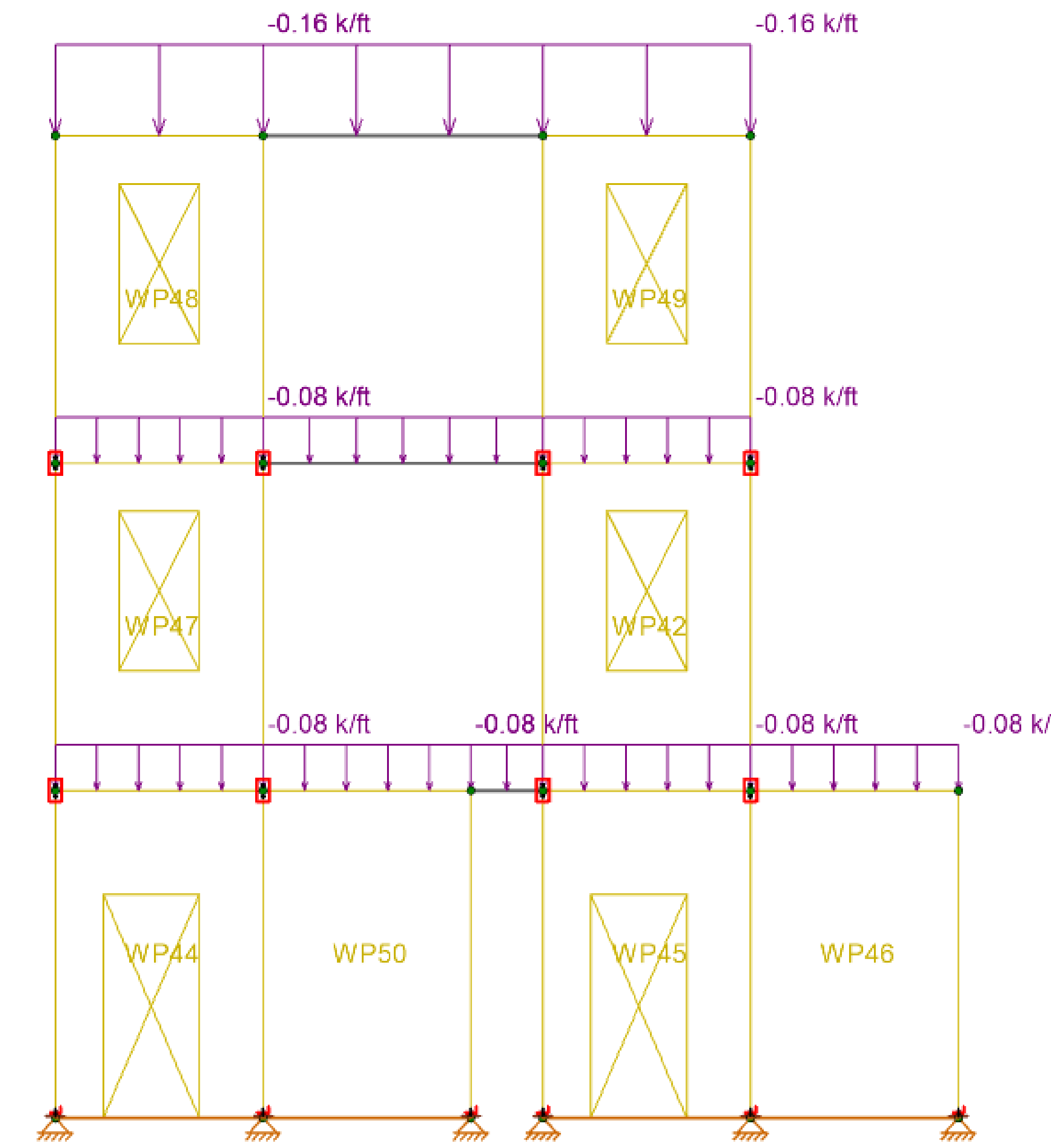


(FTAO - RISA 3D)

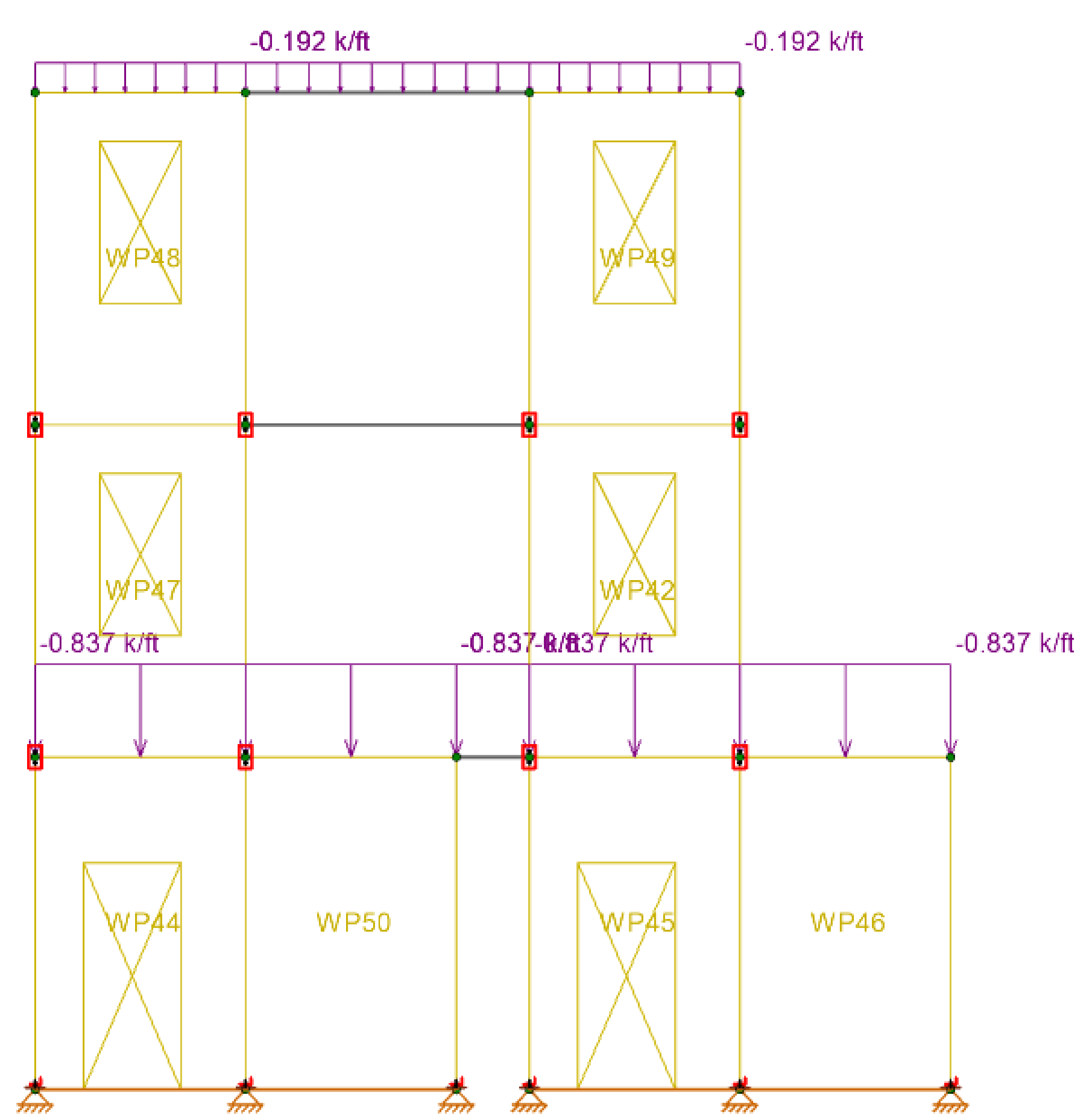
E-W WALLS
GRIDS 1A, 1B
(FTAO - RISA 3D)



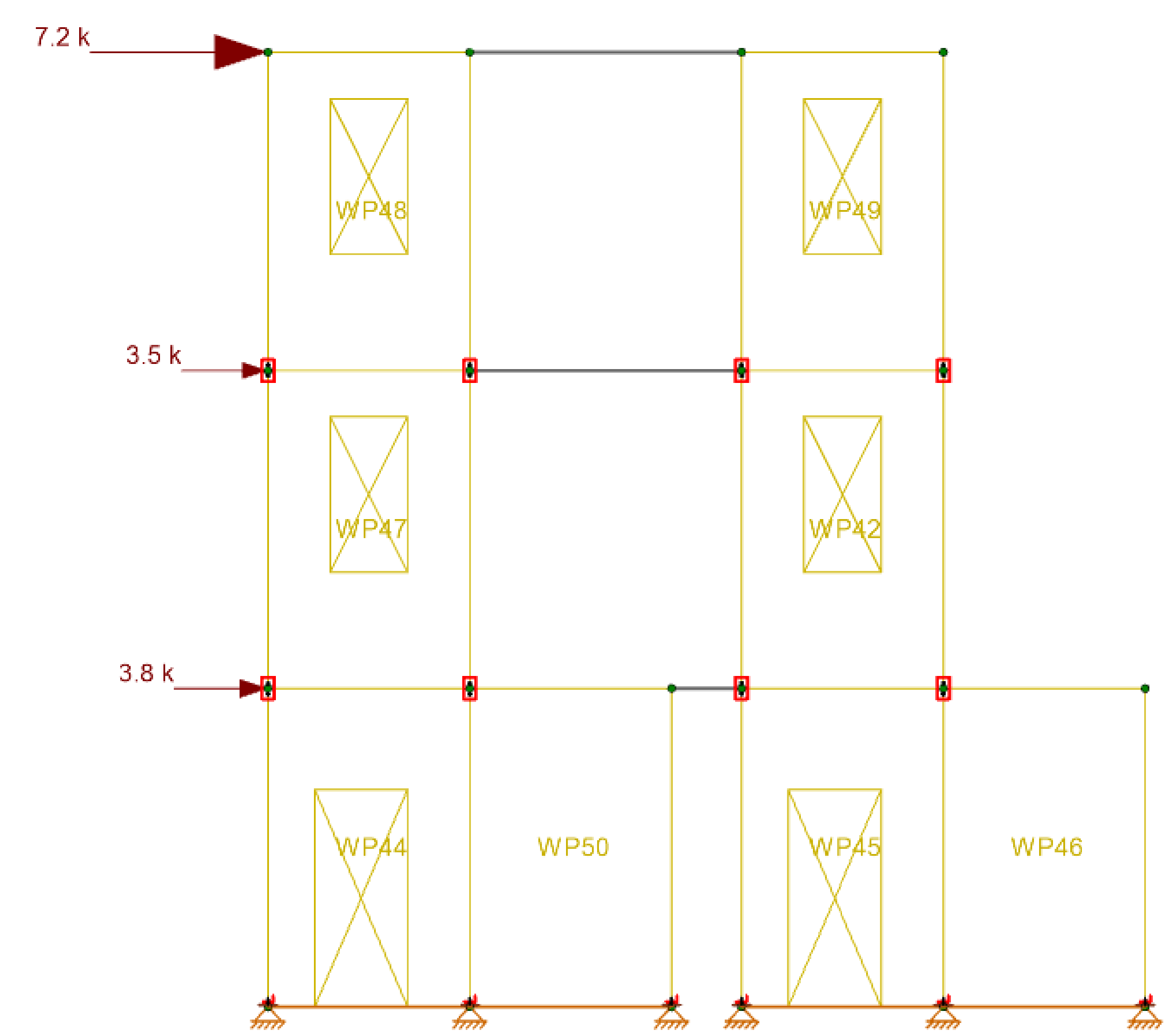
DEAD LOADS
(FROM GRAVITY LOAD TRACE)



LIVE LOADS
(FROM GRAVITY LOAD TRACE)

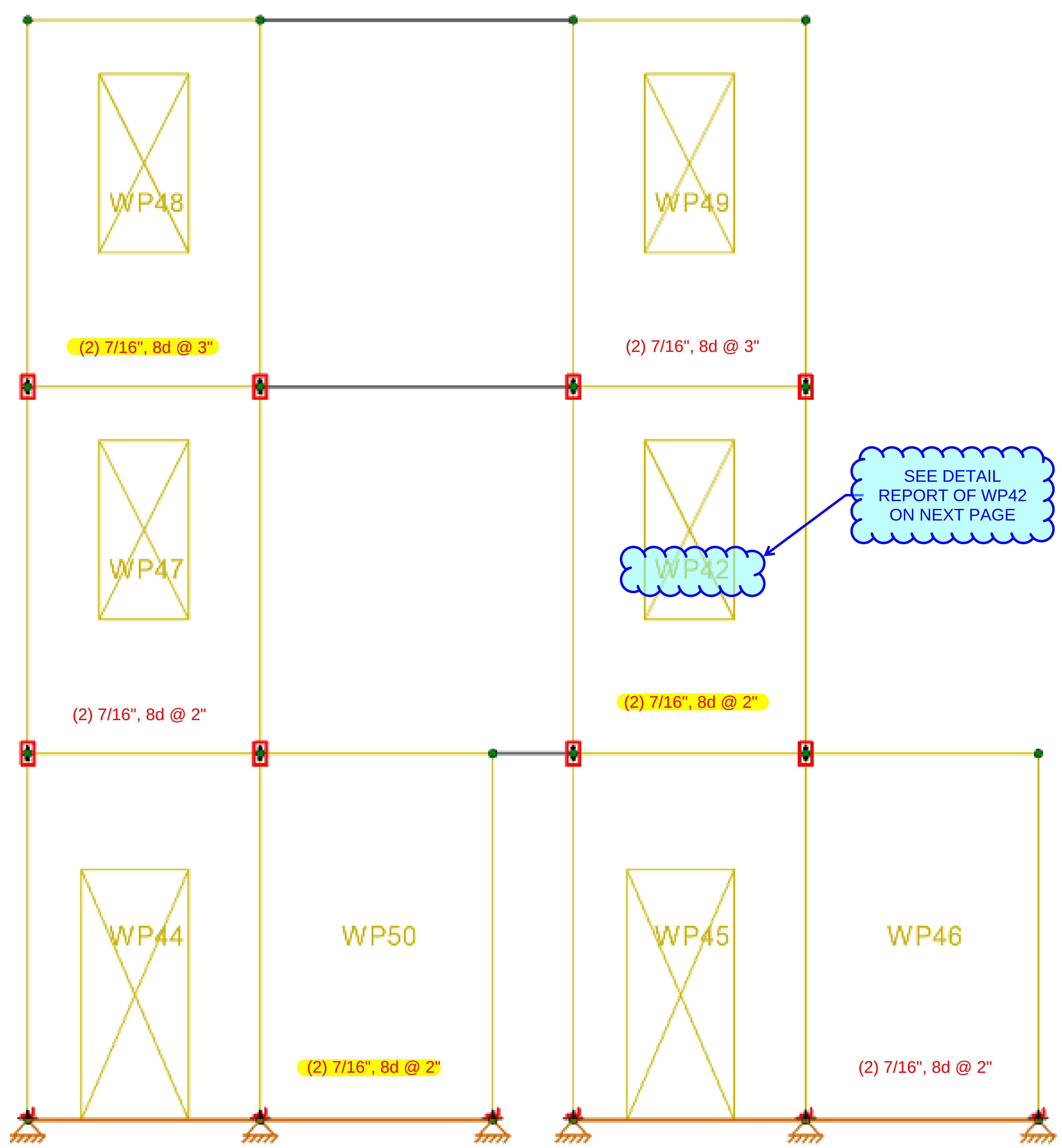


SNOW LOADS
(FROM GRAVITY LOAD TRACE)



WIND LOADS

E-W WALLS
GRIDS 1A, 1B
(FTAO - RISA 3D)



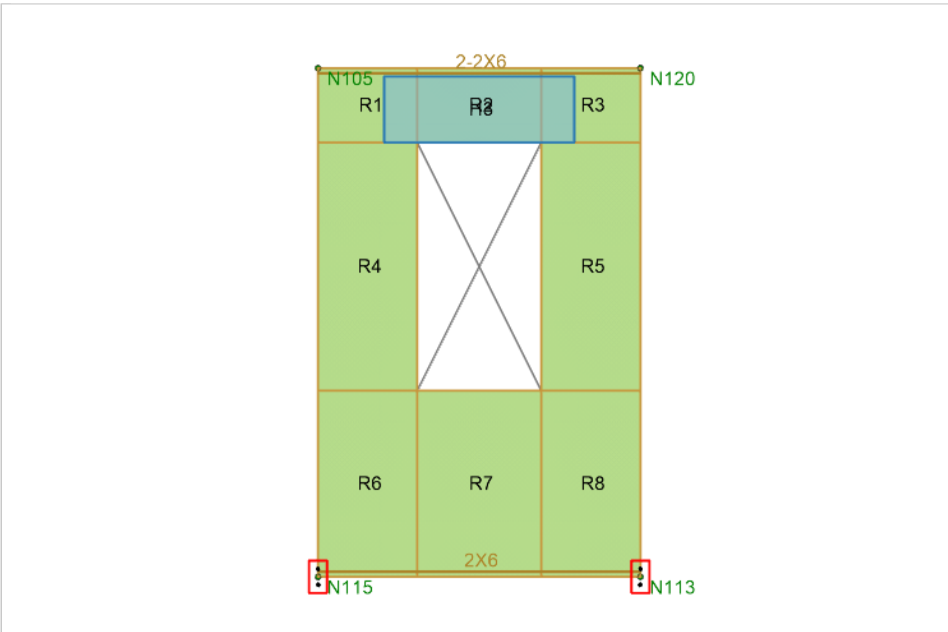
Wood Wall Panel In Plane Code Checks (AWC NDS-18: ASD)

Concrete In | Concrete Out | Concrete Seismic | Masonry In | Masonry Out | Masonry Lintel | Masonry Seismic | Wood Wall Axial | Wood Wall In-Plane | Wood Header | CFS Wall Axial | CFS Wall In-Plane

	Wall Pa...	Shear Panel Label	Region	Shear Ch...	Shear Force[k...	Gov LC	Hold-Down Label	Chord Strap Label	Tension Ch...	Tie-Down Forc...	Gov LC
1	WP42	S1_(2)7/16_8d@2	N/A	0.811	1.132	1	NC	HST5_SPF/HF_Bolt	0.795	7.844	8
2	WP46	S1_7/16_8d@3	R1	0.863	0.567	2	HDU8-SDS2.5_3_SPF-HF	NC	0.846	4.927	8
3	WP47	S1_(2)7/16_8d@3	N/A	0.831	0.874	2	NC	HST3_DF/SP_Bolt	0.976	7.465	7
4	WP48	S1_(2)7/16_8d@2	N/A	0.838	1.17	2	NC	MST48_SPF/HF_Bolt	0.987	3.349	8
5	WP49	S1_(2)7/16_8d@2	N/A	0.774	1.081	1	NC	MSTC40_18_DF/SP_...	0.99	2.664	7
6	WP50	S1_7/16_8d@2	R1	0.901	0.786	2	HDU11-SDS2.5_5.5_SPF...	NC	0.947	7.601	8

Detail Report: WP42

Enveloped Results



Input Data:

Code:	AWC NDS-18: ASD
Design Method:	FTAO
Height (ft):	10.25
Length (ft):	6.5
Wall Material:	Hem-Fir No.2
Panel Schedule:	AWC 2015 OSB
Sel. Shear Panel:	S1_(2)7/16_8d@2 | 16
Optimize HD:	Yes
HD Manufacturer:	SIMPSON

Wall Properties:

Top Plate:	2-2X6
Sill:	2X6
Wall Stud:	2X6
Chord:	6X8
Max H/W Ratio:	1.577
K:	1
Max Opening Ht:	5
Wall Capacity Adj. Factor (2w/h):	0.8
Aspect Ratio Factor:	0.938
Gov. H/W Ratio Factor:	1.12
Shear Cap. Adj Factor (Co):	0.849
Sheathing Area Ratio (r):	0.766

Material Properties:

Top Plate:	Hem-Fir No.2	Fb (ksi):	0.85	Ft (ksi):	0.525
Sill:	Hem-Fir No.2	Fv (ksi):	0.15	* All values per 2015 NDS Supplement Table 4A (Reference Design Values for Visually Graded Dimension Lumber)	
Wall Stud:	Hem-Fir No.2	Fc (ksi):	1.3		
Chord:	Hem-Fir No.2	Specific Gravity:	0.43		
E:	1300	Density (k/ft^3):	0.035		

Design Summary: Enveloped Results

Limit State	Gov. LC	Required	Available	Unity Check	Result
Wood Wall Summary					PASS
Whole Wall (Shear)					
Total Shear	1 (W)	3.653 k			
Max Unit Shear	1 (W)	1.132 k/ft			
ShearPanel					
S1_(2)7/16_8d@2	1 (W)	1.132 k/ft	1.396 k/ft	0.811	PASS
Chord Straps / Hold Downs					
Strap / Hold Down Manufacturer : Simpson Chord Straps					
HST5_SPF/HF_Bolt	8 (W)	7.844 k	9.87 k	0.795	PASS
Chords					

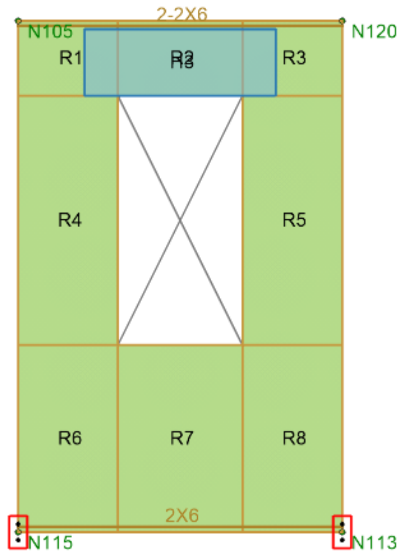
6X8 (Tension)	8 (W)	-8.504 k	24.75 k	0.344	PASS
6X8 (Tension)	7 (W)	-8.418 k	24.75 k	0.340	PASS
Studs					
2X6	0 (W)	0 k	0 k	0.000	PASS

Deflection Results

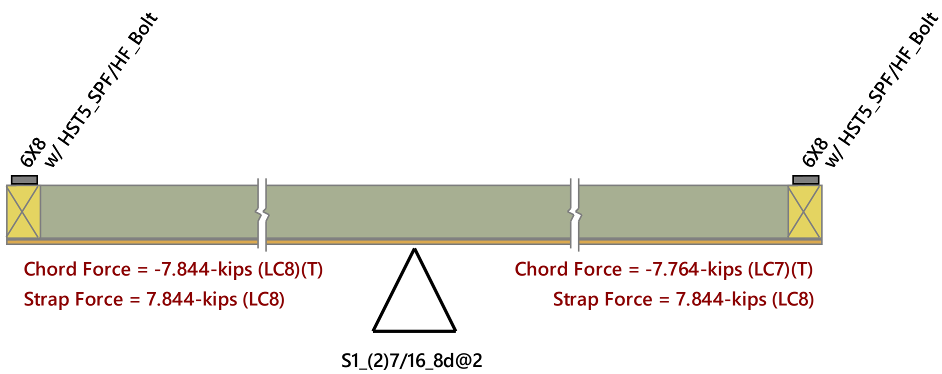
Total (Max)	Gov. LC	Elastic	HD	Shear	Shear Stiffness Adjustment Factor (SSAF)
0.178	1	0.033 in	0 in	0.145 in	1

Code Check:

Limit State	Required	Available	Unity Check	Result
Shear Panel Design	**1.132 k/ft**	**1.396 k/ft**	**0.811**	**PASS**
Shear Panel: S1_(2)7/16_8d@2				
Panel Grade				St-I
Panel Thickness				0.438in
Number Sides				Two
Over Gyp. Board				No
Nail Size				8.000000d
NOTE: AWC NDS-18 defines a 8.000000d nail as being 2.5" x 0.131" common, or 2.5" x 0.113" galvanized box				
Required Penetration				1.375 in
Required Spacing				2 in
Specific Gravity Adjustment Factor = [1-(0.5-G)]				0.93
Shear Capacity				1.34 k/ft
Adjusted Capacity				1.396 k/ft
Chord Design	**7.844 k**	**24.75 k**	**0.317**	**PASS**
Gov Compression LC = 2				
Compression Analysis	**9.205 k**	**30.348 k**	0.303	PASS
Gov Tension LC = 8				
Tension Analysis	**7.844 k**	**24.75 k**	0.317	PASS
Stud Design **NOTE: Stud design performed only for load combinations which do not contain seismic or wind load**				
Chord Strap Design	**7.844 k**	**9.87 k**	**0.795**	**PASS**
Selected Chord Strap / Hold Down:	HST5_SPF/HF_Bolt			
Governing Load Combination = 8				
Clear Span				0 in
Fasteners				(12) 0.625 inch in
End Length				0 in
Req'd Chord Mat'l				SPF/HF
Base Cap (C_D=1)				6.169 k
C_D Factor				1.6
Adjusted Capacity				9.87 k



Cross Section Detailing



Opening Design H3

Criteria:

Code:	AWC NDS-18:ASD
Design Method:	FTAO

Geometry:

Opening Ht:	5 ft
Opening Width:	2.5 ft
h/w Ratio:	2

Material Properties:

Header Material:	HF	Sill Material:	HF
Header Size:	2-2X8	Sill Size:	2X6

WARNING: No header design for load combinations containing seismic or wind categories.

Code Check:

Limit State	Location	Required	Available	Unity Check	Result
Header Bending Design	Note: No header design for load combinations containing seismic or wind categories.				
Header Shear Design	Note: No header design for load combinations containing seismic or wind categories.				

E-W WALLS
GRIDS 1E, 1F
(PERFORATED METHOD - TEDDS)

1 TH1 SOUTH
A0201-TH1 A0301-TH1 3/16" = 1'-0"

LEVEL 4

WOOD SHEAR WALL DESIGN (NDS)

In accordance with NDS2018 allowable stress design and the perforated shear wall method

Tedds calculation version 1.2.08

Design summary

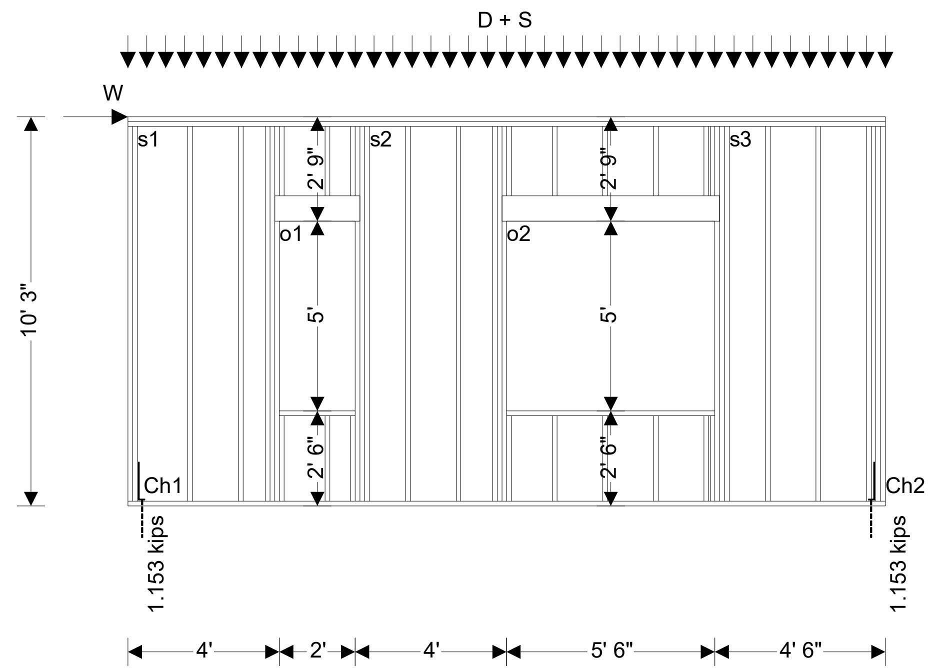
Description	Unit	Provided	Required	Utilization	Result
Shear capacity	lbs	3436	1800	0.524	PASS
Chord capacity	lb/in^2	818	124	0.152	PASS
Collector capacity	lb/in^2	1508	32	0.022	PASS
Deflection	in	0.246	0.302	1.227	FAIL

Panel details

Structural I wood panel sheathing on one side

Panel height h = **10.25** ft

Panel length b = **20** ft



Panel opening details

Width of opening w_{o1} = **2** ft

Height of opening h_{o1} = **5** ft

Height to underside of lintel over opening l_{o1} = **7.5** ft

Position of opening P_{o1} = **4** ft

Width of opening w_{o2} = **5.5** ft

Height of opening h_{o2} = **5** ft

Height to underside of lintel over opening l_{o2} = **7.5** ft

Position of opening P_{o2} = **10** ft

Total area of wall $A = h \times b - w_{o1} \times h_{o1} - w_{o2} \times h_{o2}$ = **167.5** ft^2

LEVEL 4

Panel construction

Nominal stud size	2" x 6"
Dressed stud size	1.5" x 5.5"
Cross-sectional area of studs	A_s = **8.25** in^2
Stud spacing	s = **16** in
Nominal end post size	2 x 2" x 6"
Dressed end post size	2 x 1.5" x 5.5"
Cross-sectional area of end posts	A_e = **16.5** in^2
Hole diameter	Dia = **1** in
Net cross-sectional area of end posts	A_{en} = **13.5** in^2
Nominal collector size	2 x 2" x 6"
Dressed collector size	2 x 1.5" x 5.5"
Service condition	Dry
Temperature	100 degF or less
Vertical anchor stiffness	k_a = **30000** lb/in

From NDS Supplement Table 4A - Reference design values for visually graded dimension lumber (2" - 4" thick)

Species, grade and size classification	Hem-Fir, no.1 & btr grade, 2" & wider
Specific gravity	G = **0.43**
Tension parallel to grain	F_t = **725** lb/in^2
Compression parallel to grain	F_c = **1350** lb/in^2
Modulus of elasticity	E = **1500000** lb/in^2
Minimum modulus of elasticity	E_{min} = **550000** lb/in^2

Sheathing details

Sheathing material	7/16" wood panel structural I oriented strandboard sheathing
Fastener type	8d common nails at 6"centers

From SDPWS Table 4.3A Nominal Unit Shear Capacities for Wood-Frame Shear Walls - Wood-based Panels

Nominal unit shear capacity for seismic design	v_s = min(560 plf × min[1 - (0.5 - G), 1], 1740 plf) = **520.8** lb/ft
Nominal unit shear capacity for wind design	v_w = min(785 plf × min[1 - (0.5 - G), 1], 2435 plf) = **730.1** lb/ft
Apparent shear wall shear stiffness	G_a = **16** kips/in

Loading details

Dead load acting on top of panel	D = **32** lb/ft
Snow load acting on top of panel	S = **96** lb/ft
Self weight of panel	S_{wt} = **10** lb/ft^2
In plane wind load acting at head of panel	W = **3000** lbs
Wind load serviceability factor	f_{Wserv} = **1.00**

From ASCE 7-16 - cl.2.4.1 and cl. 2.4.5 Basic combinations

Load combination no.1	D + 0.6W
Load combination no.2	D + 0.7E
Load combination no.3	D + 0.75L_f + 0.45W + 0.75(L_r or S or R)
Load combination no.4	D + 0.75L_f + 0.525E + 0.75S
Load combination no.5	0.6D + 0.6W
Load combination no.6	0.6D + 0.7E

LEVEL 4

Adjustment factors

Load duration factor – Table 2.3.2 C_D = **1.60**

Size factor for tension – Table 4A C_{Ft} = **1.30**

Size factor for compression – Table 4A C_{Fc} = **1.10**

Wet service factor for tension – Table 4A C_{Mt} = **1.00**

Wet service factor for compression – Table 4A C_{Mc} = **1.00**

Wet service factor for modulus of elasticity – Table 4A

C_{ME} = **1.00**

Temperature factor for tension – Table 2.3.3 C_{tt} = **1.00**

Temperature factor for compression – Table 2.3.3

C_{tc} = **1.00**

Temperature factor for modulus of elasticity – Table 2.3.3

C_{tE} = **1.00**

Incising factor – cl.4.3.8 C_i = **1.00**

Buckling stiffness factor – cl.4.4.2 C_T = **1.00**

Adjusted modulus of elasticity $E_{min}' = E_{min} \times C_{ME} \times C_{tE} \times C_i \times C_T$ = **550000** psi

Critical buckling design value $F_{cE} = 0.822 \times E_{min}' / (h / d)^2$ = **904** psi

Reference compression design value $F_c^* = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i$ = **2376** psi

For sawn lumber c = **0.8**

Column stability factor – eqn.3.7-1 $C_P = (1 + (F_{cE} / F_c^*)) / (2 \times c) - \sqrt{([(1 + (F_{cE} / F_c^*)) / (2 \times c)]^2 - (F_{cE} / F_c^*) / c)}$ = **0.34**

From SDPWS Table 4.3.4 Maximum Shear Wall Aspect Ratios

Maximum shear wall aspect ratio 3.5

Perforated wall length b_1 = **4** ft

Shear wall aspect ratio h / b_1 = **2.563**

Perforated wall length b_2 = **4** ft

Shear wall aspect ratio h / b_2 = **2.563**

Perforated wall length b_3 = **4.5** ft

Shear wall aspect ratio h / b_3 = **2.278**

Shear capacity adjustment factor – cl.4.3.3.5

Sum of perforated shear wall lengths $\Sigma L_i = b_1 \times 2 \times b_s / h + b_2 \times 2 \times b_s / h + b_3 \times 2 \times b_s / h$ = **9.756** ft

Total length of perforated shear wall $L_{tot} = b_1 + w_{o1} + b_2 + w_{o2} + b_3$ = **20** ft

Total area of openings $A_o = w_{o1} \times h_{o1} + w_{o2} \times h_{o2}$ = **37.5** ft^2

Sheathing area ratio (eqn. 4.3-6) $r = 1 / (1 + A_o / (h \times \Sigma L_i))$ = **0.727**

Shear capacity adjustment factor (eqn. 4.3-5) C_o = **0.965**

Perforated shear wall capacity

Maximum shear force under wind loading $V_{w\_max} = 0.6 \times W$ = **1.8** kips

Shear capacity for wind loading $V_w = v_w \times C_o \times \Sigma L_i / 2$ = **3.436** kips

V_{w_max} / V_w = **0.524**

PASS - Shear capacity for wind load exceeds maximum shear force

Chord capacity for chords 1 and 2

Load combination 5

Shear force for maximum tension $V = 0.6 \times W$ = **1.8** kips

LEVEL 4

Axial force for maximum tension | $P = (0.6 \times (D + S_{wt} \times h)) \times b / 2 =$ **0.807** kips

Maximum tensile force in chord | $T = V \times h / ((C_o \times \Sigma L_i)) - P =$ **1.153** kips ← HDU2-SDS2.5

Maximum applied tensile stress | $f_t = T / A_{en} =$ **85** lb/in^2

Design tensile stress | $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i =$ **1508** lb/in^2

$f_t / F_t' =$ **0.057**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 1

Shear force for maximum compression | $V = 0.6 \times W =$ **1.8** kips

Axial force for maximum compression | $P = ((D + S_{wt} \times h)) \times s / 2 =$ **0.09** kips

Maximum compressive force in chord | $C = V \times h / ((C_o \times \Sigma L_i)) + P =$ **2.050** kips

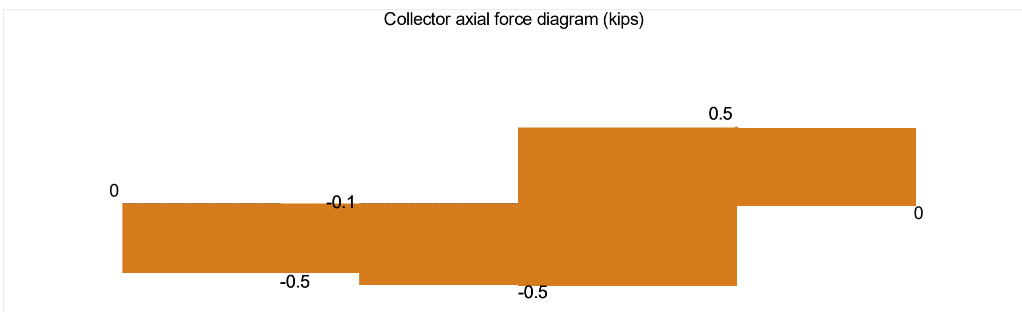
Maximum applied compressive stress | $f_c = C / A_e =$ **124** lb/in^2

Design compressive stress | $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P =$ **818** lb/in^2

$f_c / F_c' =$ **0.152**

PASS - Design compressive stress exceeds maximum applied compressive stress

Collector capacity



Maximum shear force on wall | $V_{max} = V_{w\_max} =$ **1.8** kips

Uniform shear applied to wall | $v_a = V_{max} / ((C_o \times \Sigma L_i)) =$ **191.3** plf

Shear resisted by wall segments | $v_b = v_a \times b / (b_1 + b_2 + b_3) =$ **306** plf

Maximum force in collector | $P_{coll} =$ **0.535** kips

Maximum applied tensile stress | $f_t = P_{coll} / (2 \times A_s) =$ **32** lb/in^2

Design tensile stress | $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i =$ **1508** lb/in^2

$f_t / F_t' =$ **0.022**

PASS - Design tensile stress exceeds maximum applied tensile stress

Maximum applied compressive stress | $f_c = P_{coll} / (2 \times A_s) =$ **32** lb/in^2

Column stability factor | $C_P =$ **1.00**

Design compressive stress | $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P =$ **2376** lb/in^2

$f_c / F_c' =$ **0.014**

PASS - Design compressive stress exceeds maximum applied compressive stress

Hold down force

Chord 1 | $T_1 =$ **1.153** kips

Chord 2 T_2 = **1.153** kips **LEVEL 4**

Wind load deflection

Design shear force $V_{\delta w} = f_{Wserv} \times W$ = **3** kips

Deflection limit $\Delta_{w\_allow} = h / 500$ = **0.246** in

Induced unit shear $v_{\delta w\_max} = V_{\delta w} / (C_o \times \Sigma L_i)$ = **318.75** lb/ft

Anchor tension force $T_\delta = \max(0 \text{ kips}, v_{\delta w\_max} \times h - 0.6 \times (D + S_{wt} \times h) \times b / 2)$ = **2.460** kips

Shear wall deflection – Eqn. 4.3-1 $\delta_{sww} = 2 \times v_{\delta w\_max} \times h^3 / (3 \times E \times A_e \times \Sigma L_i) + v_{\delta w\_max} \times h / (G_a) + h \times T_\delta / (k_a \times \Sigma L_i)$ = **0.302** in

$\delta_{sww} / \Delta_{w\_allow}$ = **1.227**

FAIL - Shear wall deflection exceeds deflection limit

LEVEL 3

WOOD SHEAR WALL DESIGN (NDS)

In accordance with NDS2018 allowable stress design and the perforated shear wall method

Tedds calculation version 1.2.08

Design summary

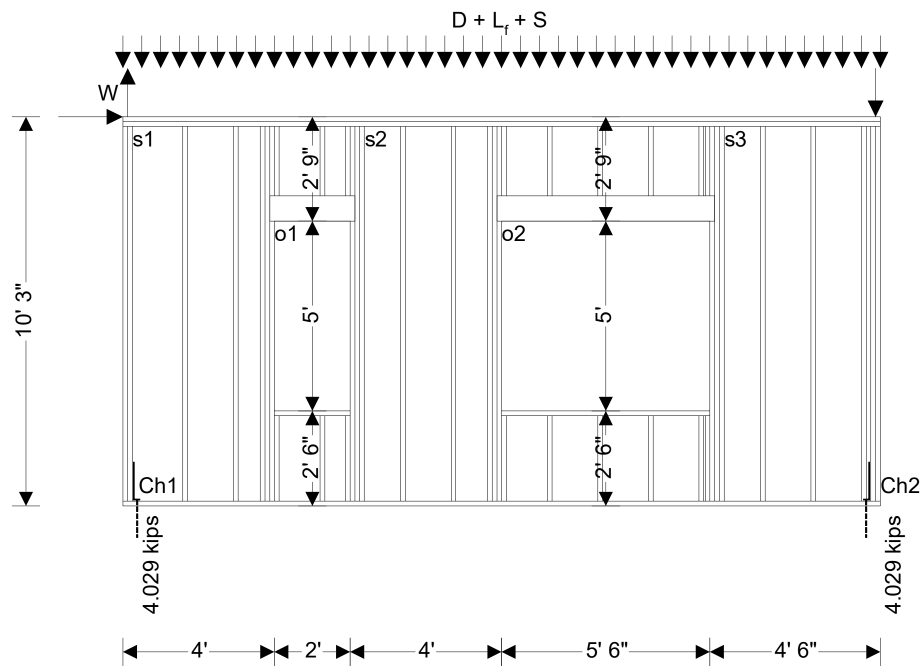
Description	Unit	Provided	Required	Utilization	Result
Shear capacity	lbs	5274	4320	0.819	PASS
Chord capacity	lb/in^2	818	352	0.430	PASS
Collector capacity	lb/in^2	1508	78	0.052	PASS
Deflection	in	0.246	0.673	2.736	FAIL

Panel details

Structural I wood panel sheathing on one side

Panel height $h = \mathbf{10.25}$ ft

Panel length $b = \mathbf{20}$ ft



Panel opening details

Width of opening $w_{o1} = \mathbf{2}$ ft

Height of opening $h_{o1} = \mathbf{5}$ ft

Height to underside of lintel over opening $l_{o1} = \mathbf{7.5}$ ft

Position of opening $P_{o1} = \mathbf{4}$ ft

Width of opening $w_{o2} = \mathbf{5.5}$ ft

Height of opening $h_{o2} = \mathbf{5}$ ft

Height to underside of lintel over opening $l_{o2} = \mathbf{7.5}$ ft

Position of opening $P_{o2} = \mathbf{10}$ ft

Total area of wall $A = h \times b - w_{o1} \times h_{o1} - w_{o2} \times h_{o2} = \mathbf{167.5}$ ft^2

LEVEL 3

Panel construction

Nominal stud size	2" x 6"
Dressed stud size	1.5" x 5.5"
Cross-sectional area of studs	A_s = **8.25** in^2
Stud spacing	s = **16** in
Nominal end post size	2 x 2" x 6"
Dressed end post size	2 x 1.5" x 5.5"
Cross-sectional area of end posts	A_e = **16.5** in^2
Hole diameter	Dia = **1** in
Net cross-sectional area of end posts	A_{en} = **13.5** in^2
Nominal collector size	2 x 2" x 6"
Dressed collector size	2 x 1.5" x 5.5"
Service condition	Dry
Temperature	100 degF or less
Vertical anchor stiffness	k_a = **30000** lb/in

From NDS Supplement Table 4A - Reference design values for visually graded dimension lumber (2" - 4" thick)

Species, grade and size classification	Hem-Fir, no.1 & btr grade, 2" & wider
Specific gravity	G = **0.43**
Tension parallel to grain	F_t = **725** lb/in^2
Compression parallel to grain	F_c = **1350** lb/in^2
Modulus of elasticity	E = **1500000** lb/in^2
Minimum modulus of elasticity	E_{min} = **550000** lb/in^2

Sheathing details

Sheathing material	7/16" wood panel structural I oriented strandboard sheathing
Fastener type	8d common nails at 4"centers

From SDPWS Table 4.3A Nominal Unit Shear Capacities for Wood-Frame Shear Walls - Wood-based Panels

Nominal unit shear capacity for seismic design	v_s = min(860 plf × min[1 - (0.5 - G), 1], 1740 plf) = **799.8** lb/ft
Nominal unit shear capacity for wind design	v_w = min(1205 plf × min[1 - (0.5 - G), 1], 2435 plf) = **1120.6** lb/ft
Apparent shear wall shear stiffness	G_a = **21** kips/in

Loading details

Dead load acting on top of panel	D = **164** lb/ft
Floor live load acting on top of panel	L_f = **80** lb/ft
Snow load acting on top of panel	S = **96** lb/ft
Self weight of panel	S_{wt} = **10** lb/ft^2
In plane wind load acting at head of panel	W = **7200** lbs
Wind load serviceability factor	f_{Wserv} = **1.00**

Chord forces from shear walls above

Chord	$W_{ch[i]}$ (lbs)	$E_{q\_ch[i]}$ (lbs)	$D_{C\_ch[i]}$ (lbs)	$D_{T\_ch[i]}$ (lbs)	$L_{f\_ch[i]}$ (lbs)	$L_{r\_ch[i]}$ (lbs)	$S_{ch[i]}$ (lbs)	$R_{ch[i]}$ (lbs)
Ch1	-1538;	0;	0;	0;	0;	0;	0;	0;
Ch2	1538;	0;	0;	0;	0;	0;	0;	0;

From ASCE 7-16 - cl.2.4.1 and cl. 2.4.5 Basic combinations

Load combination no.1	D + 0.6W
Load combination no.2	D + 0.7E

LEVEL 3

Load combination no.3 $D + 0.75L_f + 0.45W + 0.75(L_r \text{ or } S \text{ or } R)$
Load combination no.4 $D + 0.75L_f + 0.525E + 0.75S$
Load combination no.5 $0.6D + 0.6W$
Load combination no.6 $0.6D + 0.7E$

Adjustment factors

Load duration factor – Table 2.3.2 C_D = **1.60**
Size factor for tension – Table 4A C_{Ft} = **1.30**
Size factor for compression – Table 4A C_{Fc} = **1.10**
Wet service factor for tension – Table 4A C_{Mt} = **1.00**
Wet service factor for compression – Table 4A C_{Mc} = **1.00**
Wet service factor for modulus of elasticity – Table 4A
C_{ME} = **1.00**
Temperature factor for tension – Table 2.3.3 C_{tt} = **1.00**
Temperature factor for compression – Table 2.3.3
C_{tc} = **1.00**
Temperature factor for modulus of elasticity – Table 2.3.3
C_{tE} = **1.00**
Incising factor – cl.4.3.8 C_i = **1.00**
Buckling stiffness factor – cl.4.4.2 C_T = **1.00**
Adjusted modulus of elasticity $E_{min}' = E_{min} \times C_{ME} \times C_{tE} \times C_i \times C_T$ = **550000** psi
Critical buckling design value $F_{cE} = 0.822 \times E_{min}' / (h / d)^2$ = **904** psi
Reference compression design value $F_c^* = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i$ = **2376** psi
For sawn lumber c = **0.8**
Column stability factor – eqn.3.7-1 $C_P = (1 + (F_{cE} / F_c^*)) / (2 \times c) - \sqrt{[(1 + (F_{cE} / F_c^*)) / (2 \times c)]^2 - (F_{cE} / F_c^*) / c}$ = **0.34**

From SDPWS Table 4.3.4 Maximum Shear Wall Aspect Ratios

Maximum shear wall aspect ratio 3.5
Perforated wall length b_1 = **4** ft
Shear wall aspect ratio h / b_1 = **2.563**
Perforated wall length b_2 = **4** ft
Shear wall aspect ratio h / b_2 = **2.563**
Perforated wall length b_3 = **4.5** ft
Shear wall aspect ratio h / b_3 = **2.278**

Shear capacity adjustment factor – cl.4.3.3.5

Sum of perforated shear wall lengths $\Sigma L_i = b_1 \times 2 \times b_s / h + b_2 \times 2 \times b_s / h + b_3 \times 2 \times b_s / h$ = **9.756** ft
Total length of perforated shear wall $L_{tot} = b_1 + w_{o1} + b_2 + w_{o2} + b_3$ = **20** ft
Total area of openings $A_o = w_{o1} \times h_{o1} + w_{o2} \times h_{o2}$ = **37.5** ft^2
Sheathing area ratio (eqn. 4.3-6) $r = 1 / (1 + A_o / (h \times \Sigma L_i))$ = **0.727**
Shear capacity adjustment factor (eqn. 4.3-5) C_o = **0.965**

Perforated shear wall capacity

Maximum shear force under wind loading $V_{w\_max} = 0.6 \times W$ = **4.32** kips
Shear capacity for wind loading $V_w = v_w \times C_o \times \Sigma L_i / 2$ = **5.274** kips
V_{w_max} / V_w = **0.819**

LEVEL 3

PASS - Shear capacity for wind load exceeds maximum shear force

Chord capacity for chord 1

Load combination 5

Shear force for maximum tension $V = 0.6 \times W =$ **4.32** kips

Axial force for maximum tension $P = (0.6 \times (D + S_{wt} \times h)) \times b / 2 + 0.6 \times W_{ch1} =$ **0.676** kips

Maximum tensile force in chord $T = V \times h / ((C_o \times \Sigma L_i)) - P =$ $\boxed{\textbf{4.029 kips}}$ ← HDU5-SDS2.5

Maximum applied tensile stress $f_t = T / A_{en} =$ **298** lb/in^2

Design tensile stress $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i =$ **1508** lb/in^2

$f_t / F_t' =$ **0.198**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 1

Shear force for maximum compression $V = 0.6 \times W =$ **4.32** kips

Axial force for maximum compression $P = ((D + S_{wt} \times h)) \times s / 2 + -1 \times 0.6 \times W_{ch1} =$ **1.1** kips

Maximum compressive force in chord $C = V \times h / ((C_o \times \Sigma L_i)) + P =$ $\boxed{\textbf{5.805 kips}}$

Maximum applied compressive stress $f_c = C / A_e =$ **352** lb/in^2

Design compressive stress $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P =$ **818** lb/in^2

$f_c / F_c' =$ **0.430**

PASS - Design compressive stress exceeds maximum applied compressive stress

Chord capacity for chord 2

Load combination 5

Shear force for maximum tension $V = 0.6 \times W =$ **4.32** kips

Axial force for maximum tension $P = (0.6 \times (D + S_{wt} \times h)) \times b / 2 + -1 \times 0.6 \times W_{ch2} =$ **0.676** kips

Maximum tensile force in chord $T = V \times h / ((C_o \times \Sigma L_i)) - P =$ $\boxed{\textbf{4.029 kips}}$ ← HDU5-SDS2.5

Maximum applied tensile stress $f_t = T / A_{en} =$ **298** lb/in^2

Design tensile stress $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i =$ **1508** lb/in^2

$f_t / F_t' =$ **0.198**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 1

Shear force for maximum compression $V = 0.6 \times W =$ **4.32** kips

Axial force for maximum compression $P = ((D + S_{wt} \times h)) \times s / 2 + 0.6 \times W_{ch2} =$ **1.1** kips

Maximum compressive force in chord $C = V \times h / ((C_o \times \Sigma L_i)) + P =$ $\boxed{\textbf{5.805 kips}}$

Maximum applied compressive stress $f_c = C / A_e =$ **352** lb/in^2

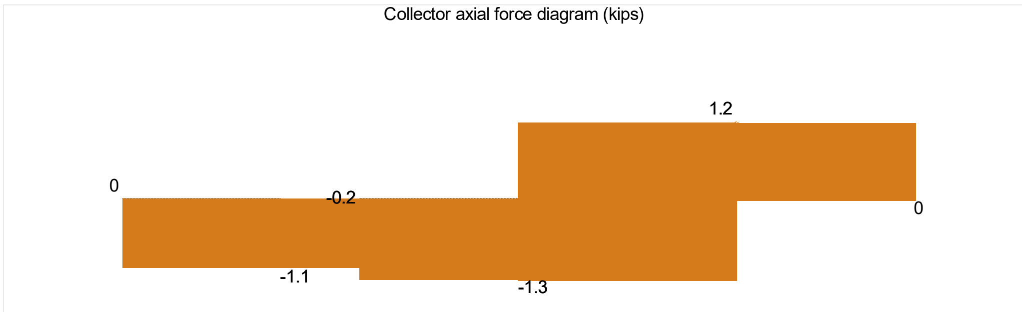
Design compressive stress $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P =$ **818** lb/in^2

$f_c / F_c' =$ **0.430**

PASS - Design compressive stress exceeds maximum applied compressive stress

LEVEL 3

Collector capacity



Maximum shear force on wall	$V_{max} = V_{w\_max} =$ **4.32** kips
Uniform shear applied to wall	$v_a = V_{max} / ((C_o \times \Sigma L_i)) =$ **459** plf
Shear resisted by wall segments	$v_b = v_a \times b / (b_1 + b_2 + b_3) =$ **734.4** plf
Maximum force in collector	$P_{coll} =$ **1.285** kips
Maximum applied tensile stress	$f_t = P_{coll} / (2 \times A_s) =$ **78** lb/in^2
Design tensile stress	$F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i =$ **1508** lb/in^2
	$f_t / F_t' =$ **0.052**
	PASS - Design tensile stress exceeds maximum applied tensile stress
Maximum applied compressive stress	$f_c = P_{coll} / (2 \times A_s) =$ **78** lb/in^2
Column stability factor	$C_P =$ **1.00**
Design compressive stress	$F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P =$ **2376** lb/in^2
	$f_c / F_c' =$ **0.033**
	PASS - Design compressive stress exceeds maximum applied compressive stress

Hold down force

Chord 1	$T_1 =$ **4.029** kips
Chord 2	$T_2 =$ **4.029** kips

Wind load deflection

Design shear force	$V_{\delta w} = f_{Wserv} \times W =$ **7.2** kips
Deflection limit	$\Delta_{w\_allow} = h / 500 =$ **0.246** in
Induced unit shear	$v_{\delta w\_max} = V_{\delta w} / (C_o \times \Sigma L_i) =$ **765** lb/ft
Anchor tension force	$T_\delta = \max(0 \text{ kips}, v_{\delta w\_max} \times h - 0.6 \times (D + S_{wt} \times h) \times b / 2 + \max(\text{abs}(W_{ch1}), \text{abs}(W_{ch2}))) =$ **7.780** kips
Shear wall deflection – Eqn. 4.3-1	$\delta_{sww} = 2 \times v_{\delta w\_max} \times h^3 / (3 \times E \times A_e \times \Sigma L_i) + v_{\delta w\_max} \times h / (G_a) + h \times T_\delta / (k_a \times \Sigma L_i) =$ **0.673** in
	$\delta_{sww} / \Delta_{w\_allow} =$ **2.736**
	FAIL - Shear wall deflection exceeds deflection limit

LEVEL 2

WOOD SHEAR WALL DESIGN (NDS)

In accordance with NDS2018 allowable stress design and the perforated shear wall method

Tedds calculation version 1.2.08

Design summary

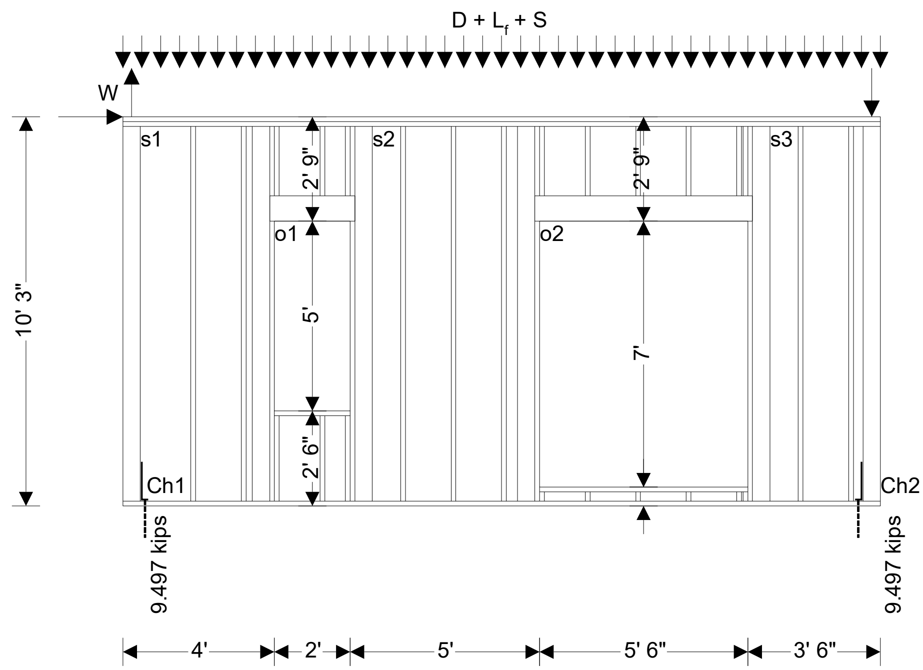
Description	Unit	Provided	Required	Utilization	Result
Shear capacity	lbs	7142	6420	0.899	PASS
Chord capacity	lb/in^2	818	402	0.491	PASS
Collector capacity	lb/in^2	1508	176	0.117	PASS
Deflection	in	0.246	1.094	4.447	FAIL

Panel details

Structural I wood panel sheathing on one side

Panel height h = **10.25** ft

Panel length b = **20** ft



Panel opening details

Width of opening w_{o1} = **2** ft

Height of opening h_{o1} = **5** ft

Height to underside of lintel over opening l_{o1} = **7.5** ft

Position of opening P_{o1} = **4** ft

Width of opening w_{o2} = **5.5** ft

Height of opening h_{o2} = **7** ft

Height to underside of lintel over opening l_{o2} = **7.5** ft

Position of opening P_{o2} = **11** ft

Total area of wall $A = h \times b - w_{o1} \times h_{o1} - w_{o2} \times h_{o2}$ = **156.5** ft^2

LEVEL 2

Panel construction

Nominal stud size 2" x 6"

Dressed stud size 1.5" x 5.5"

Cross-sectional area of studs A_s = **8.25** in^2

Stud spacing s = **16** in

Nominal end post size 6" x 6"

Dressed end post size 5.5" x 5.5"

Cross-sectional area of end posts A_e = **30.25** in^2

Hole diameter Dia = **1** in

Net cross-sectional area of end posts A_{en} = **24.75** in^2

Nominal collector size 2 x 2" x 6"

Dressed collector size 2 x 1.5" x 5.5"

Service condition Dry

Temperature 100 degF or less

Vertical anchor stiffness k_a = **30000** lb/in

From NDS Supplement Table 4A - Reference design values for visually graded dimension lumber (2" - 4" thick)

Species, grade and size classification Hem-Fir, no.1 & btr grade, 2" & wider

Specific gravity G = **0.43**

Tension parallel to grain F_t = **725** lb/in^2

Compression parallel to grain F_c = **1350** lb/in^2

Modulus of elasticity E = **1500000** lb/in^2

Minimum modulus of elasticity E_{min} = **550000** lb/in^2

Sheathing details

Sheathing material 7/16" wood panel structural I oriented strandboard sheathing

Fastener type 8d common nails at 2"centers

From SDPWS Table 4.3A Nominal Unit Shear Capacities for Wood-Frame Shear Walls - Wood-based Panels

Nominal unit shear capacity for seismic design v_s = min(1460 plf × min[1 - (0.5 - G), 1], 1740 plf) = **1357.8** lb/ft

Nominal unit shear capacity for wind design v_w = min(2045 plf × min[1 - (0.5 - G), 1], 2435 plf) = **1901.9** lb/ft

Apparent shear wall shear stiffness G_a = **40** kips/in

Loading details

Dead load acting on top of panel D = **296** lb/ft

Floor live load acting on top of panel L_f = **160** lb/ft

Snow load acting on top of panel S = **196** lb/ft

Self weight of panel S_{wt} = **10** lb/ft^2

In plane wind load acting at head of panel W = **10700** lbs

Wind load serviceability factor f_{Wserv} = **1.00**

Chord forces from shear walls above

Chord	$W_{ch[i]}$ (lbs)	$E_{q\_ch[i]}$ (lbs)	$D_{C\_ch[i]}$ (lbs)	$D_{T\_ch[i]}$ (lbs)	$L_{f\_ch[i]}$ (lbs)	$L_{r\_ch[i]}$ (lbs)	$S_{ch[i]}$ (lbs)	$R_{ch[i]}$ (lbs)
Ch1	-5211;	0;	0;	0;	0;	0;	0;	0;
Ch2	5211;	0;	0;	0;	0;	0;	0;	0;

From ASCE 7-16 - cl.2.4.1 and cl. 2.4.5 Basic combinations

Load combination no.1 D + 0.6W

Load combination no.2 D + 0.7E

LEVEL 2

Load combination no.3 $D + 0.75L_f + 0.45W + 0.75(L_r \text{ or } S \text{ or } R)$
Load combination no.4 $D + 0.75L_f + 0.525E + 0.75S$
Load combination no.5 $0.6D + 0.6W$
Load combination no.6 $0.6D + 0.7E$

Adjustment factors

Load duration factor – Table 2.3.2 C_D = **1.60**
Size factor for tension – Table 4A C_{Ft} = **1.30**
Size factor for compression – Table 4A C_{Fc} = **1.10**
Wet service factor for tension – Table 4A C_{Mt} = **1.00**
Wet service factor for compression – Table 4A C_{Mc} = **1.00**
Wet service factor for modulus of elasticity – Table 4A
C_{ME} = **1.00**
Temperature factor for tension – Table 2.3.3 C_{tt} = **1.00**
Temperature factor for compression – Table 2.3.3
C_{tc} = **1.00**
Temperature factor for modulus of elasticity – Table 2.3.3
C_{tE} = **1.00**
Incising factor – cl.4.3.8 C_i = **1.00**
Buckling stiffness factor – cl.4.4.2 C_T = **1.00**
Adjusted modulus of elasticity $E_{min}' = E_{min} \times C_{ME} \times C_{tE} \times C_i \times C_T$ = **550000** psi
Critical buckling design value $F_{cE} = 0.822 \times E_{min}' / (h / d)^2$ = **904** psi
Reference compression design value $F_c^* = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i$ = **2376** psi
For sawn lumber c = **0.8**
Column stability factor – eqn.3.7-1 $C_P = (1 + (F_{cE} / F_c^*)) / (2 \times c) - \sqrt{[(1 + (F_{cE} / F_c^*)) / (2 \times c)]^2 - (F_{cE} / F_c^*) / c}$ = **0.34**

From SDPWS Table 4.3.4 Maximum Shear Wall Aspect Ratios

Maximum shear wall aspect ratio 3.5
Perforated wall length b_1 = **4** ft
Shear wall aspect ratio h / b_1 = **2.563**
Perforated wall length b_2 = **5** ft
Shear wall aspect ratio h / b_2 = **2.05**
Perforated wall length b_3 = **3.5** ft
Shear wall aspect ratio h / b_3 = **2.929**

Shear capacity adjustment factor – cl.4.3.3.5

Sum of perforated shear wall lengths $\Sigma L_i = b_1 \times 2 \times b_s / h + b_2 \times 2 \times b_s / h + b_3 \times 2 \times b_s / h$ = **8.537** ft
Total length of perforated shear wall $L_{tot} = b_1 + w_{o1} + b_2 + w_{o2} + b_3$ = **20** ft
Total area of openings $A_o = w_{o1} \times h_{o1} + w_{o2} \times h_{o2}$ = **48.5** ft^2
Sheathing area ratio (eqn. 4.3-6) $r = 1 / (1 + A_o /(h \times \Sigma L_i))$ = **0.643**
Shear capacity adjustment factor (eqn. 4.3-5) C_o = **0.88**

Perforated shear wall capacity

Maximum shear force under wind loading $V_{w\_max} = 0.6 \times W$ = **6.42** kips
Shear capacity for wind loading $V_w = v_w \times C_o \times \Sigma L_i / 2$ = **7.142** kips
V_{w_max} / V_w = **0.899**

LEVEL 2

PASS - Shear capacity for wind load exceeds maximum shear force

Chord capacity for chord 1

Load combination 5

Shear force for maximum tension $V = 0.6 \times W =$ **6.42** kips

Axial force for maximum tension $P = (0.6 \times (D + S_{wt} \times h)) \times b / 2 + 0.6 \times W_{ch1} =$ **-0.736** kips

Maximum tensile force in chord $T = V \times h / ((C_o \times \Sigma L_i)) - P =$ $\boxed{\textbf{9.497 kips}}$ HDU11-SDS2.5

Maximum applied tensile stress $f_t = T / A_{en} =$ **384** lb/in^2

Design tensile stress $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i =$ **1508** lb/in^2

$f_t / F_t' =$ **0.254**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 1

Shear force for maximum compression $V = 0.6 \times W =$ **6.42** kips

Axial force for maximum compression $P = ((D + S_{wt} \times h)) \times s / 2 + -1 \times 0.6 \times W_{ch1} =$ **3.392** kips

Maximum compressive force in chord $C = V \times h / ((C_o \times \Sigma L_i)) + P =$ $\boxed{\textbf{12.154 kips}}$

Maximum applied compressive stress $f_c = C / A_e =$ **402** lb/in^2

Design compressive stress $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P =$ **818** lb/in^2

$f_c / F_c' =$ **0.491**

PASS - Design compressive stress exceeds maximum applied compressive stress

Chord capacity for chord 2

Load combination 5

Shear force for maximum tension $V = 0.6 \times W =$ **6.42** kips

Axial force for maximum tension $P = (0.6 \times (D + S_{wt} \times h)) \times b / 2 + -1 \times 0.6 \times W_{ch2} =$ **-0.736** kips

Maximum tensile force in chord $T = V \times h / ((C_o \times \Sigma L_i)) - P =$ $\boxed{\textbf{9.497 kips}}$

Maximum applied tensile stress $f_t = T / A_{en} =$ **384** lb/in^2

Design tensile stress $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i =$ **1508** lb/in^2

$f_t / F_t' =$ **0.254**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 1

Shear force for maximum compression $V = 0.6 \times W =$ **6.42** kips

Axial force for maximum compression $P = ((D + S_{wt} \times h)) \times s / 2 + 0.6 \times W_{ch2} =$ **3.392** kips

Maximum compressive force in chord $C = V \times h / ((C_o \times \Sigma L_i)) + P =$ $\boxed{\textbf{12.154 kips}}$ HDU14-SDS2.5

Maximum applied compressive stress $f_c = C / A_e =$ **402** lb/in^2

Design compressive stress $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P =$ **818** lb/in^2

$f_c / F_c' =$ **0.491**

PASS - Design compressive stress exceeds maximum applied compressive stress

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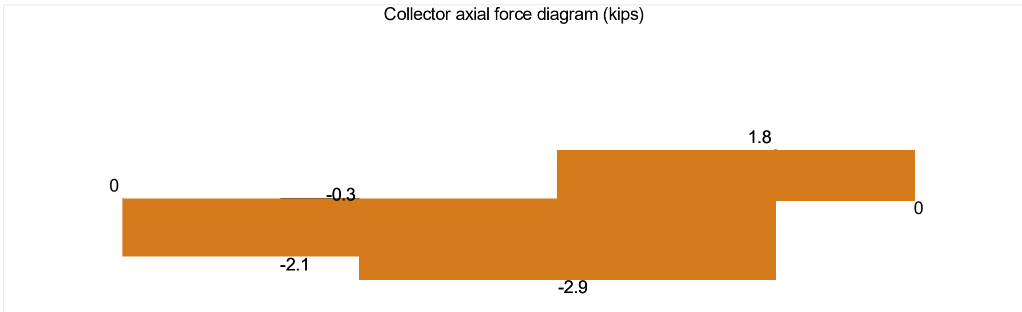
LEVEL 2

Collector capacity



Maximum shear force on wall $V_{max} = V_{w\_max} =$ **6.42** kips

Uniform shear applied to wall $v_a = V_{max} / ((C_o \times \Sigma L_i)) =$ **854.8** plf

Shear resisted by wall segments $v_b = v_a \times b / (b_1 + b_2 + b_3) =$ **1367.6** plf

Maximum force in collector $P_{coll} =$ **2.906** kips

Maximum applied tensile stress $f_t = P_{coll} / (2 \times A_s) =$ **176** lb/in^2

Design tensile stress $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i =$ **1508** lb/in^2

$f_t / F_t' =$ **0.117**

PASS - Design tensile stress exceeds maximum applied tensile stress

Maximum applied compressive stress $f_c = P_{coll} / (2 \times A_s) =$ **176** lb/in^2

Column stability factor $C_P =$ **1.00**

Design compressive stress $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P =$ **2376** lb/in^2

$f_c / F_c' =$ **0.074**

PASS - Design compressive stress exceeds maximum applied compressive stress

Hold down force

Chord 1 $T_1 =$ **9.497** kips

Chord 2 $T_2 =$ **9.497** kips

Wind load deflection

Design shear force $V_{\delta w} = f_{Wserv} \times W =$ **10.7** kips

Deflection limit $\Delta_{w\_allow} = h / 500 =$ **0.246** in

Induced unit shear $v_{\delta w\_max} = V_{\delta w} / (C_o \times \Sigma L_i) =$ **1424.63** lb/ft

Anchor tension force $T_\delta = \max(0 \text{ kips}, v_{\delta w\_max} \times h - 0.6 \times (D + S_{wt} \times h) \times b / 2 + \max(\text{abs}(W_{ch1}), \text{abs}(W_{ch2}))) =$ **17.422** kips

Shear wall deflection – Eqn. 4.3-1 $\delta_{sww} = 2 \times v_{\delta w\_max} \times h^3 / (3 \times E \times A_e \times \Sigma L_i) + v_{\delta w\_max} \times h / (G_a) + h \times T_\delta / (k_a \times \Sigma L_i) =$ **1.094** in

$\delta_{sww} / \Delta_{w\_allow} =$ **4.447**

FAIL - Shear wall deflection exceeds deflection limit

LEVEL 1

WOOD SHEAR WALL DESIGN (NDS)

In accordance with NDS2018 allowable stress design and the segmented shear wall method

Tedds calculation version 1.2.08

Design summary

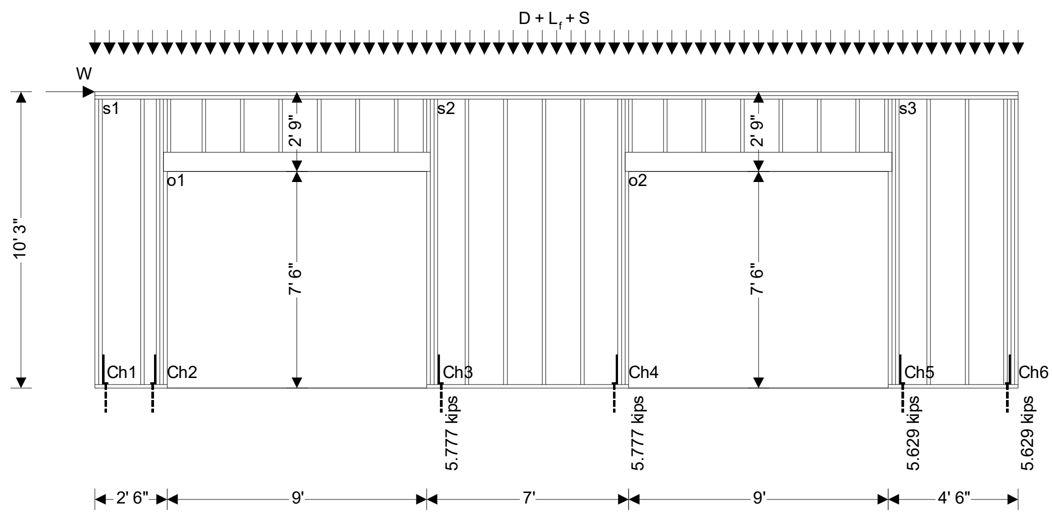
Description	Unit	Provided	Required	Utilization	Result
Shear capacity	lbs	10936	8700	0.796	PASS
Chord capacity	lb/in^2	818	479	0.585	PASS
Collector capacity	lb/in^2	1508	189	0.126	PASS

Panel details

Structural I wood panel sheathing on one side

Panel height h = **10.25** ft

Panel length b = **32** ft



Panel opening details

Width of opening w_{o1} = **9** ft

Height of opening h_{o1} = **7.5** ft

Height to underside of lintel over opening l_{o1} = **7.5** ft

Position of opening P_{o1} = **2.5** ft

Width of opening w_{o2} = **9** ft

Height of opening h_{o2} = **7.5** ft

Height to underside of lintel over opening l_{o2} = **7.5** ft

Position of opening P_{o2} = **18.5** ft

Total area of wall $A = h \times b - w_{o1} \times h_{o1} - w_{o2} \times h_{o2}$ = **193** ft^2

Panel construction

Nominal stud size 2" x 6"

Dressed stud size 1.5" x 5.5"

Cross-sectional area of studs A_s = **8.25** in^2

Stud spacing s = **16** in

Nominal end post size 2 x 2" x 6"

Dressed end post size 2 x 1.5" x 5.5"

LEVEL 1

Cross-sectional area of end posts	A_e = **16.5** in^2
Hole diameter	Dia = **1** in
Net cross-sectional area of end posts	A_{en} = **13.5** in^2
Nominal collector size	2 x 2" x 6"
Dressed collector size	2 x 1.5" x 5.5"
Service condition	Dry
Temperature	100 degF or less
Vertical anchor stiffness	k_a = **30000** lb/in

From NDS Supplement Table 4A - Reference design values for visually graded dimension lumber (2" - 4" thick)

Species, grade and size classification	Hem-Fir, no.1 & btr grade, 2" & wider
Specific gravity	G = **0.43**
Tension parallel to grain	F_t = **725** lb/in^2
Compression parallel to grain	F_c = **1350** lb/in^2
Modulus of elasticity	E = **1500000** lb/in^2
Minimum modulus of elasticity	E_{min} = **550000** lb/in^2

Sheathing details

Sheathing material	7/16" wood panel structural I oriented strandboard sheathing
Fastener type	8d common nails at 2"centers

From SDPWS Table 4.3A Nominal Unit Shear Capacities for Wood-Frame Shear Walls - Wood-based Panels

Nominal unit shear capacity for seismic design	v_s = 1460 plf × min[1 - (0.5 - G), 1] = **1357.8** lb/ft
Nominal unit shear capacity for wind design	v_w = 2045 plf × min[1 - (0.5 - G), 1] = **1901.9** lb/ft
Apparent shear wall shear stiffness	G_a = **40** kips/in

Loading details

Dead load acting on top of panel	D = **1024** lb/ft
Floor live load acting on top of panel	L_f = **1920** lb/ft
Snow load acting on top of panel	S = **192** lb/ft
Self weight of panel	S_{wt} = **10** lb/ft^2
In plane wind load acting at head of panel	W = **14500** lbs
Wind load serviceability factor	f_{Wserv} = **1.00**

From ASCE 7-16 - cl.2.4.1 and cl. 2.4.5 Basic combinations

Load combination no.1	D + 0.6W
Load combination no.2	D + 0.7E
Load combination no.3	D + 0.75L_f + 0.45W + 0.75(L_r or S or R)
Load combination no.4	D + 0.75L_f + 0.525E + 0.75S
Load combination no.5	0.6D + 0.6W
Load combination no.6	0.6D + 0.7E

Adjustment factors

Load duration factor – Table 2.3.2	C_D = **1.60**
Size factor for tension – Table 4A	C_{Ft} = **1.30**
Size factor for compression – Table 4A	C_{Fc} = **1.10**
Wet service factor for tension – Table 4A	C_{Mt} = **1.00**
Wet service factor for compression – Table 4A	C_{Mc} = **1.00**
Wet service factor for modulus of elasticity – Table 4A	

LEVEL 1

C_{ME} = **1.00**

Temperature factor for tension – Table 2.3.3 C_{tt} = **1.00**

Temperature factor for compression – Table 2.3.3

C_{tc} = **1.00**

Temperature factor for modulus of elasticity – Table 2.3.3

C_{tE} = **1.00**

Incising factor – cl.4.3.8 C_i = **1.00**

Buckling stiffness factor – cl.4.4.2 C_T = **1.00**

Adjusted modulus of elasticity $E_{min}' = E_{min} \times C_{ME} \times C_{tE} \times C_i \times C_T$ = **550000** psi

Critical buckling design value $F_{cE} = 0.822 \times E_{min}' / (h / d)^2$ = **904** psi

Reference compression design value $F_c^* = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i$ = **2376** psi

For sawn lumber c = **0.8**

Column stability factor – eqn.3.7-1 $C_P = (1 + (F_{cE} / F_c^*)) / (2 \times c) - \sqrt{([(1 + (F_{cE} / F_c^*)) / (2 \times c)]^2 - (F_{cE} / F_c^*) / c)}$ = **0.34**

From SDPWS Table 4.3.4 Maximum Shear Wall Aspect Ratios

Maximum shear wall aspect ratio 3.5

Segment 1 wall length b_1 = **2.5** ft

Shear wall aspect ratio h / b_1 = **4.1**

Segment 2 wall length b_2 = **7** ft

Shear wall aspect ratio h / b_2 = **1.464**

Segment 3 wall length b_3 = **4.5** ft

Shear wall aspect ratio h / b_3 = **2.278**

Segmented shear wall capacity - Strength distribution method

Maximum shear force under wind loading $V_{w\_max} = 0.6 \times W$ = **8.7** kips

Shear capacity for wind loading $V_w = v_w \times (b_2 + b_3) / 2$ = **10.936** kips

V_{w_max} / V_w = **0.796**

PASS - Shear capacity for wind load exceeds maximum shear force

Chord capacity for chords 1 and 2

Shear wall aspect ratio h / b_1 = **4.1**

Segment not considered, shear wall aspect ratio exceeds maximum allowable.

Chord capacity for chords 3 and 4

Shear wall aspect ratio h / b_2 = **1.464**

Load combination 5

Shear force for maximum tension $V = 0.6 \times W$ = **8.7** kips

Axial force for maximum tension $P = (0.6 \times (D + S_{wt} \times h)) \times b_2 / 2$ = **2.366** kips

Maximum tensile force in chord $T = V \times (b_2 / (b_2 + 2 \times b_3^2 / h)) \times (h / b_2) - P$ = **5.777** kips HDU8-SDS2.5

Maximum applied tensile stress $f_t = T / A_{en}$ = **428** lb/in^2

Design tensile stress $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i$ = **1508** lb/in^2

f_t / F_t' = **0.284**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 1

Shear force for maximum compression $V = 0.6 \times W$ = **8.7** kips

Axial force for maximum compression $P = ((D + S_{wt} \times h)) \times s / 2$ = **0.751** kips

LEVEL 1

Maximum compressive force in chord — $C = V \times (b_2 / (b_2 + 2 \times b_3^2 / h)) \times (h / b_2) + P =$ $\boxed{\textbf{8.894}\text{ kips}}$

Maximum applied compressive stress — $f_c = C / A_e =$ **539** lb/in^2

Design compressive stress — $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P =$ **818** lb/in^2

$f_c / F_c' =$ **0.659**

PASS - Design compressive stress exceeds maximum applied compressive stress

Chord capacity for chords 5 and 6

Shear wall aspect ratio — $h / b_3 =$ **2.278**

Load combination 5

Shear force for maximum tension — $V = 0.6 \times W =$ **8.7** kips

Axial force for maximum tension — $P = (0.6 \times (D + S_{wt} \times h)) \times b_3 / 2 =$ **1.521** kips

Maximum tensile force in chord — $T = V \times (2 \times b_3^2 / h / (b_2 + 2 \times b_3^2 / h)) \times (h / b_3) - P =$ $\boxed{\textbf{5.629}\text{ kips}}$ HDU8-SDS2.5

Maximum applied tensile stress — $f_t = T / A_{en} =$ **417** lb/in^2

Design tensile stress — $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i =$ **1508** lb/in^2

$f_t / F_t' =$ **0.277**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 1

Shear force for maximum compression — $V = 0.6 \times W =$ **8.7** kips

Axial force for maximum compression — $P = ((D + S_{wt} \times h)) \times s / 2 =$ **0.751** kips

Maximum compressive force in chord — $C = V \times (2 \times b_3^2 / h / (b_2 + 2 \times b_3^2 / h)) \times (h / b_3) + P =$ $\boxed{\textbf{7.901}\text{ kips}}$

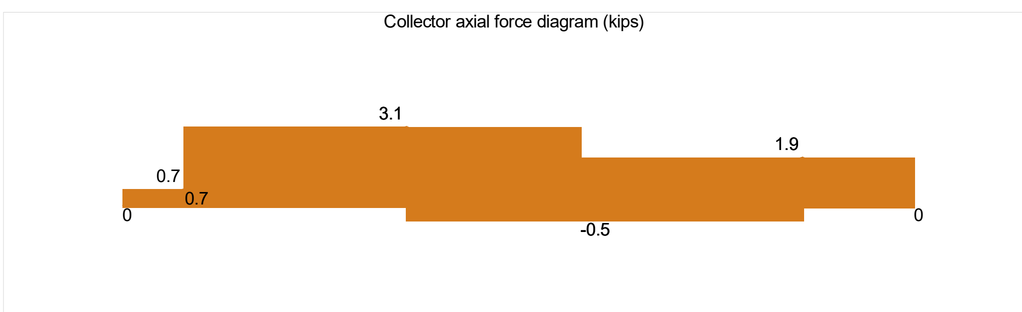
Maximum applied compressive stress — $f_c = C / A_e =$ **479** lb/in^2

Design compressive stress — $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P =$ **818** lb/in^2

$f_c / F_c' =$ **0.585**

PASS - Design compressive stress exceeds maximum applied compressive stress

Collector capacity



Maximum shear force on wall — $V_{max} = V_{w\_max} =$ **8.7** kips

Maximum force in collector — $P_{coll} =$ **3.127** kips

Maximum applied tensile stress — $f_t = P_{coll} / (2 \times A_s) =$ **189** lb/in^2

Design tensile stress — $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i =$ **1508** lb/in^2

$f_t / F_t' =$ **0.126**

PASS - Design tensile stress exceeds maximum applied tensile stress

LEVEL 1

Maximum applied compressive stress $f_c = P_{coll} / (2 \times A_s) =$ **189** lb/in^2

Column stability factor $C_P =$ **1.00**

Design compressive stress $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P =$ **2376** lb/in^2

$f_c / F_c' =$ **0.080**

PASS - Design compressive stress exceeds maximum applied compressive stress

Hold down force

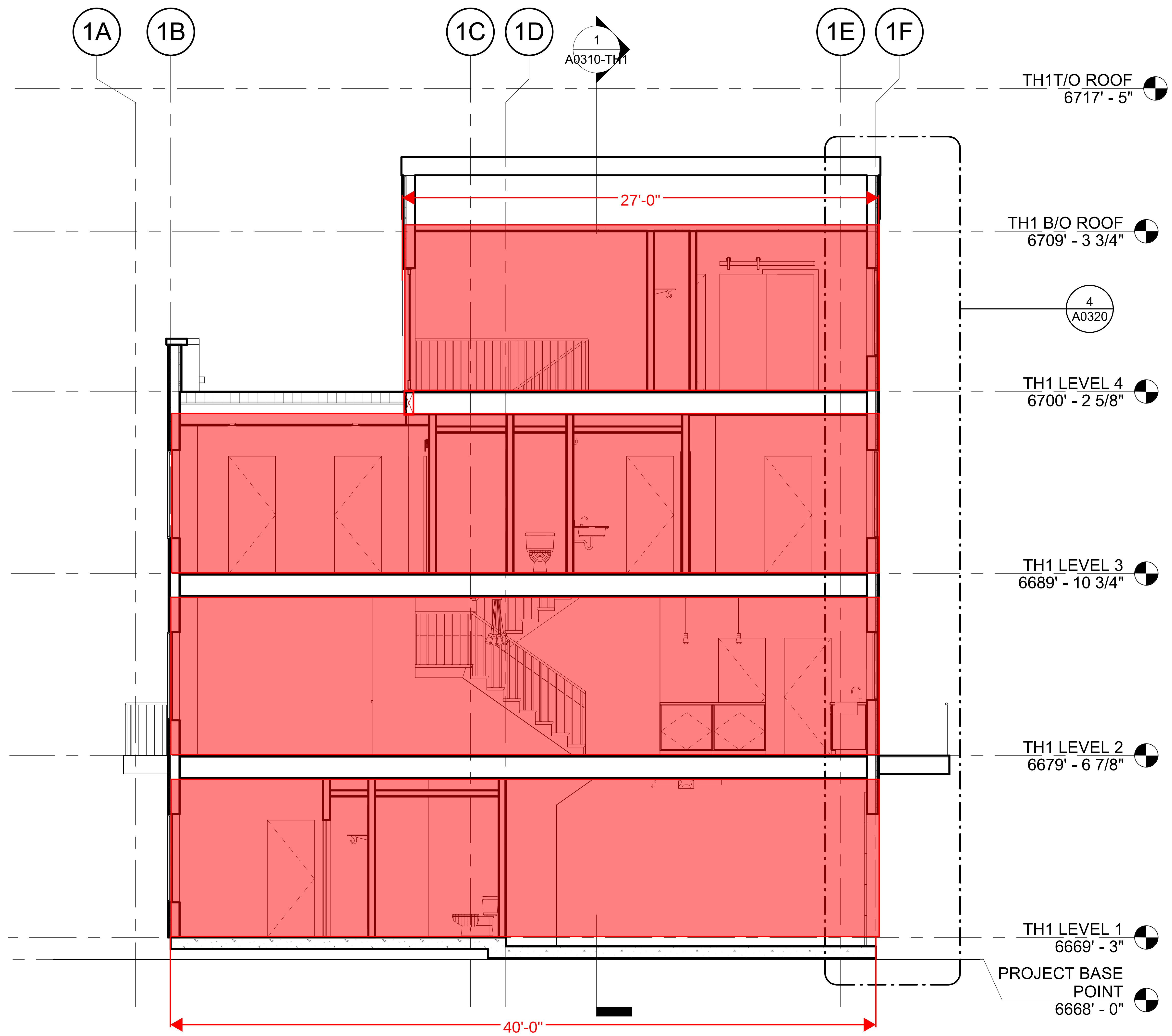
Chord 3 $T_3 =$ **5.777** kips

Chord 4 $T_4 =$ **5.777** kips

Chord 5 $T_5 =$ **5.629** kips

Chord 6 $T_6 =$ **5.629** kips

N-S WALLS
TYPICAL INTERIOR WALL
(SEGMENTED METHOD - TEDDS)



LEVEL 4

WOOD SHEAR WALL DESIGN (NDS)

In accordance with NDS2018 allowable stress design and the segmented shear wall method

Tedds calculation version 1.2.08

Design summary

Description	Unit	Provided	Required	Utilization	Result
Shear capacity	lbs	9856	2160	0.219	PASS
Chord capacity	lb/in^2	354	202	0.569	PASS
Deflection	in	0.205	0.091	0.443	PASS

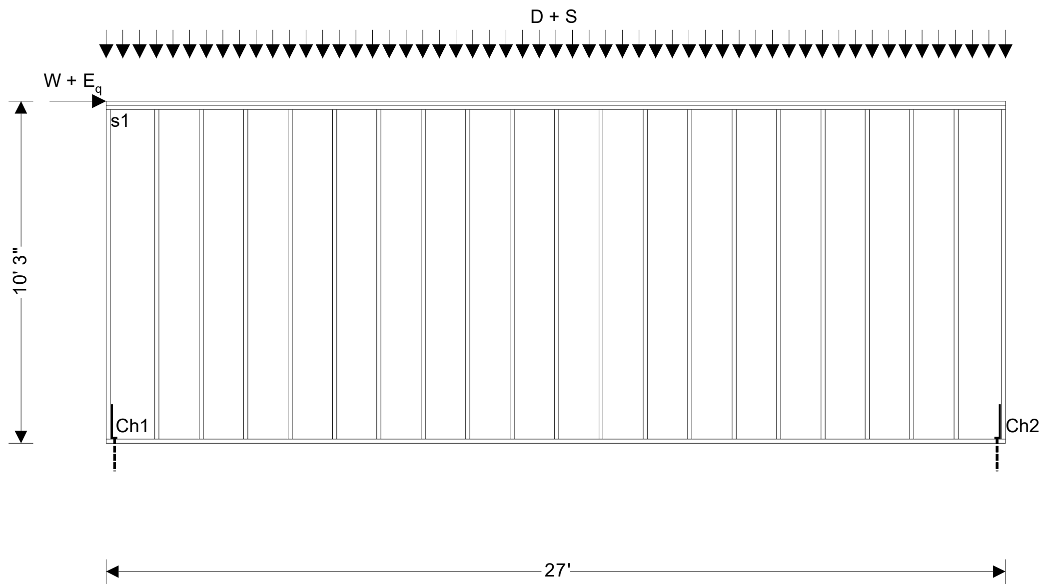
Panel details

Structural I wood panel sheathing on one side

Panel height h = **10.25** ft

Panel length b = **27** ft

Total area of wall A = h × b = **276.75** ft^2



Panel construction

Nominal stud size 2" x 4"

Dressed stud size 1.5" x 3.5"

Cross-sectional area of studs A_s = **5.25** in^2

Stud spacing s = **16** in

Nominal end post size 2" x 4"

Dressed end post size 1.5" x 3.5"

Cross-sectional area of end posts A_e = **5.25** in^2

Hole diameter Dia = **1** in

Net cross-sectional area of end posts A_{en} = **3.75** in^2

Nominal collector size 2 x 2" x 4"

Dressed collector size 2 x 1.5" x 3.5"

Service condition Dry

Temperature 100 degF or less

Vertical anchor stiffness k_a = **30000** lb/in **LEVEL 4**

From NDS Supplement Table 4A - Reference design values for visually graded dimension lumber (2" - 4" thick)

Species, grade and size classification Hem-Fir, no.1 & btr grade, 2" & wider

Specific gravity G = **0.43**

Tension parallel to grain F_t = **725** lb/in^2

Compression parallel to grain F_c = **1350** lb/in^2

Modulus of elasticity E = **1500000** lb/in^2

Minimum modulus of elasticity E_{min} = **550000** lb/in^2

Sheathing details

Sheathing material 7/16" wood panel structural I oriented strandboard sheathing

Fastener type 8d common nails at 6"centers

From SDPWS Table 4.3A Nominal Unit Shear Capacities for Wood-Frame Shear Walls - Wood-based Panels

Nominal unit shear capacity for seismic design v_s = 560 plf × min[1 - (0.5 - G), 1] = **520.8** lb/ft

Nominal unit shear capacity for wind design v_w = 785 plf × min[1 - (0.5 - G), 1] = **730.1** lb/ft

Apparent shear wall shear stiffness G_a = **16** kips/in

Loading details

Dead load acting on top of panel D = **256** lb/ft

Snow load acting on top of panel S = **1280** lb/ft

Self weight of panel S_{wt} = **10** lb/ft^2

In plane wind load acting at head of panel W = **3600** lbs

Wind load serviceability factor f_{Wserv} = **1.00**

In plane seismic load acting at head of panel E_q = **400** lbs

Design spectral response accel. par., short periods S_{DS} = **0.333**

Chord forces from shear walls above

Chord	$W_{ch[i]}$ (lbs)	$E_{q\_ch[i]}$ (lbs)	$D_{C\_ch[i]}$ (lbs)	$D_{T\_ch[i]}$ (lbs)	$L_{f\_ch[i]}$ (lbs)	$L_{r\_ch[i]}$ (lbs)	$S_{ch[i]}$ (lbs)	$R_{ch[i]}$ (lbs)
Ch1	0;	0;	0;	0;	0;	0;	0;	0;
Ch2	0;	0;	0;	0;	0;	0;	0;	0;

From IBC 2021 cl.1605.1 Basic load combinations from ASCE 7, section 2.4

Load combination no.1 D + 0.6W

Load combination no.2 D + 0.7E

Load combination no.3 D + 0.75L_f + 0.45W + 0.75(L_r or S or R)

Load combination no.4 D + 0.75L_f + 0.525E + 0.75S

Load combination no.5 0.6D + 0.6W

Load combination no.6 0.6D + 0.7E

Adjustment factors

Load duration factor – Table 2.3.2 C_D = **1.60**

Size factor for tension – Table 4A C_{Ft} = **1.50**

Size factor for compression – Table 4A C_{Fc} = **1.15**

Wet service factor for tension – Table 4A C_{Mt} = **1.00**

Wet service factor for compression – Table 4A C_{Mc} = **1.00**

Wet service factor for modulus of elasticity – Table 4A

C_{ME} = **1.00**

Temperature factor for tension – Table 2.3.3 C_{tt} = **1.00**

LEVEL 4

Temperature factor for compression – Table 2.3.3

C_{tc} = **1.00**

Temperature factor for modulus of elasticity – Table 2.3.3

C_{tE} = **1.00**

Incising factor – cl.4.3.8 — C_i = **1.00**

Buckling stiffness factor – cl.4.4.2 — C_T = **1.00**

Adjusted modulus of elasticity — $E_{min}' = E_{min} \times C_{ME} \times C_{tE} \times C_i \times C_T$ = **550000** psi

Critical buckling design value — $F_{cE} = 0.822 \times E_{min}' / (h / d)^2$ = **366** psi

Reference compression design value — $F_c^* = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i$ = **2484** psi

For sawn lumber — c = **0.8**

Column stability factor – eqn.3.7-1 — $C_P = (1 + (F_{cE} / F_c^*)) / (2 \times c) - \sqrt{[(1 + (F_{cE} / F_c^*)) / (2 \times c)]^2 - (F_{cE} / F_c^*) / c}$ = **0.14**

From SDPWS Table 4.3.4 Maximum Shear Wall Aspect Ratios

Maximum shear wall aspect ratio — 3.5

Shear wall length — b = **27** ft

Shear wall aspect ratio — h / b = **0.38**

Segmented shear wall capacity

Maximum shear force under wind loading — $V_{w\_max} = 0.6 \times W$ = **2.16** kips

Shear capacity for wind loading — $V_w = v_w \times b / 2$ = **9.856** kips

V_{w_max} / V_w = **0.219**

PASS - Shear capacity for wind load exceeds maximum shear force

Maximum shear force under seismic loading — $V_{s\_max} = 0.7 \times E_q$ = **0.28** kips

Shear capacity for seismic loading — $V_s = v_s \times b / 2$ = **7.031** kips

V_{s_max} / V_s = **0.04**

PASS - Shear capacity for seismic load exceeds maximum shear force

Chord capacity for chord 1

Shear wall aspect ratio — h / b = **0.38**

Load combination 5

Shear force for maximum tension — $V = 0.6 \times W$ = **2.16** kips

Axial force for maximum tension — $P = (0.6 \times (D + S_{wt} \times h)) \times b_1 / 2$ = **2.904** kips

Maximum tensile force in chord — $T = V \times h / (b) - P$ = $\boxed{\textbf{-2.084}\text{ kips}}$ NEGATIVE. NO OVERTURNING

Maximum applied tensile stress — $f_t = T / A_{en}$ = **-556** lb/in^2

Design tensile stress — $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i$ = **1740** lb/in^2

f_t / F_t' = **-0.319**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 1

Shear force for maximum compression — $V = 0.6 \times W$ = **2.16** kips

Axial force for maximum compression — $P = ((D + S_{wt} \times h)) \times s / 2$ = **0.239** kips

Maximum compressive force in chord — $C = V \times h / (b) + P$ = $\boxed{\textbf{1.059}\text{ kips}}$

Maximum applied compressive stress — $f_c = C / A_e$ = **202** lb/in^2

Design compressive stress — $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P$ = **354** lb/in^2

f_c / F_c' = **0.569**

LEVEL 4

PASS - Design compressive stress exceeds maximum applied compressive stress

Chord capacity for chord 2

Load combination 5

Shear force for maximum tension $V = 0.6 \times W =$ **2.16** kips

Axial force for maximum tension $P = (0.6 \times (D + S_{wt} \times h)) \times b_1 / 2 =$ **2.904** kips

Maximum tensile force in chord $T = V \times h / (b) - P = \boxed{\textbf{-2.084}\text{ kips}}$ NEGATIVE. NO OVERTURNING

Maximum applied tensile stress $f_t = T / A_{en} =$ **-556** lb/in^2

Design tensile stress $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i =$ **1740** lb/in^2

$f_t / F_t' =$ **-0.319**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 1

Shear force for maximum compression $V = 0.6 \times W =$ **2.16** kips

Axial force for maximum compression $P = ((D + S_{wt} \times h)) \times s / 2 =$ **0.239** kips

Maximum compressive force in chord $C = V \times h / (b) + P = \boxed{\textbf{1.059}\text{ kips}}$

Maximum applied compressive stress $f_c = C / A_e =$ **202** lb/in^2

Design compressive stress $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P =$ **354** lb/in^2

$f_c / F_c' =$ **0.569**

PASS - Design compressive stress exceeds maximum applied compressive stress

Wind load deflection

Design shear force $V_{\delta w} = f_{Wserv} \times W =$ **3.6** kips

Deflection limit $\Delta_{w\_allow} = h / 600 =$ **0.205** in

Induced unit shear $v_{\delta w} = V_{\delta w} / b =$ **133.33** lb/ft

Anchor tension force $T_\delta = \max(0\text{ kips}, v_{\delta w} \times h - 0.6 \times (D + S_{wt} \times h) \times b / 2) =$ **0.000** kips

Shear wall deflection – Eqn. 4.3-1 $\delta_{sww} = 2 \times v_{\delta w} \times h^3 / (3 \times E \times A_e \times b) + v_{\delta w} \times h / (G_a) + h \times T_\delta / (k_a \times b) =$ **0.091** in

$\delta_{sww} / \Delta_{w\_allow} =$ **0.443**

PASS - Shear wall deflection is less than deflection limit

Seismic deflection

Design shear force $V_{\delta s} = E_q =$ **0.4** kips

Deflection limit $\Delta_{s\_allow} = 0.020 \times h =$ **2.46** in

Induced unit shear $v_{\delta s} = V_{\delta s} / b =$ **14.81** lb/ft

Anchor tension force $T_\delta = \max(0\text{ kips}, v_{\delta s} \times h - (0.6 - 0.2 \times S_{DS}) \times (D + S_{wt} \times h) \times b / 2) =$ **0.000** kips

Shear wall elastic deflection – Eqn. 4.3-1 $\delta_{swse} = 2 \times v_{\delta s} \times h^3 / (3 \times E \times A_e \times b) + v_{\delta s} \times h / (G_a) + h \times T_\delta / (k_a \times b) =$ **0.01** in

Deflection ampification factor $C_{d\delta} =$ **4**

Seismic importance factor $I_e =$ **1.25**

Amp. seis. deflection – ASCE7 Eqn. 12.8-15 $\delta_{sws} = C_{d\delta} \times \delta_{swse} / I_e =$ **0.032** in

$\delta_{sws} / \Delta_{s\_allow} =$ **0.013**

PASS - Shear wall deflection is less than deflection limit

LEVEL 3

WOOD SHEAR WALL DESIGN (NDS)

In accordance with NDS2018 allowable stress design and the segmented shear wall method

Tedds calculation version 1.2.08

Design summary

Description	Unit	Provided	Required	Utilization	Result
Shear capacity	lbs	14601	4320	0.296	PASS
Chord capacity	lb/in^2	354	297	0.839	PASS
Deflection	in	0.205	0.120	0.587	PASS

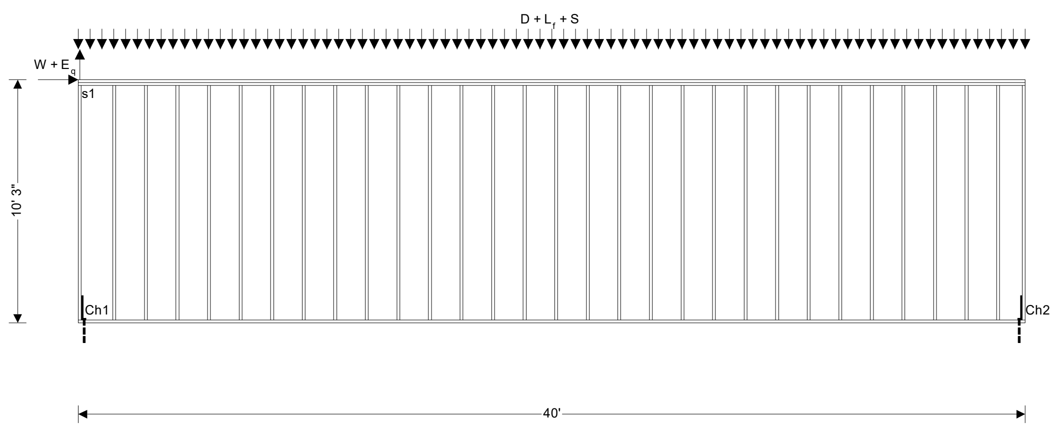
Panel details

Structural I wood panel sheathing on one side

Panel height h = **10.25** ft

Panel length b = **40** ft

Total area of wall A = h × b = **410** ft^2



Panel construction

Nominal stud size 2" x 4"

Dressed stud size 1.5" x 3.5"

Cross-sectional area of studs A_s = **5.25** in^2

Stud spacing s = **16** in

Nominal end post size 2" x 4"

Dressed end post size 1.5" x 3.5"

Cross-sectional area of end posts A_e = **5.25** in^2

Hole diameter Dia = **1** in

Net cross-sectional area of end posts A_{en} = **3.75** in^2

Nominal collector size 2 x 2" x 4"

Dressed collector size 2 x 1.5" x 3.5"

Service condition Dry

Temperature 100 degF or less

Vertical anchor stiffness k_a = **30000** lb/in

From NDS Supplement Table 4A - Reference design values for visually graded dimension lumber (2" - 4" thick)

Species, grade and size classification Hem-Fir, no.1 & btr grade, 2" & wider

Specific gravity G = **0.43**

LEVEL 3

Tension parallel to grain F_t = **725** lb/in^2
Compression parallel to grain F_c = **1350** lb/in^2
Modulus of elasticity E = **1500000** lb/in^2
Minimum modulus of elasticity E_{min} = **550000** lb/in^2

Sheathing details

Sheathing material 7/16" wood panel structural I oriented strandboard sheathing
Fastener type 8d common nails at 6"centers

From SDPWS Table 4.3A Nominal Unit Shear Capacities for Wood-Frame Shear Walls - Wood-based Panels

Nominal unit shear capacity for seismic design v_s = 560 plf × min[1 - (0.5 - G), 1] = **520.8** lb/ft
Nominal unit shear capacity for wind design v_w = 785 plf × min[1 - (0.5 - G), 1] = **730.1** lb/ft
Apparent shear wall shear stiffness G_a = **16** kips/in

Loading details

Dead load acting on top of panel D = **512** lb/ft
Floor live load acting on top of panel L_f = **640** lb/ft
Snow load acting on top of panel S = **1280** lb/ft
Self weight of panel S_{wt} = **10** lb/ft^2
In plane wind load acting at head of panel W = **7200** lbs
Wind load serviceability factor f_{Wserv} = **1.00**
In plane seismic load acting at head of panel E_q = **1000** lbs
Design spectral response accel. par., short periods S_{DS} = **0.333**

Chord forces from shear walls above

Chord	$W_{ch[i]}$ (lbs)	$E_{q\_ch[i]}$ (lbs)	$D_{C\_ch[i]}$ (lbs)	$D_{T\_ch[i]}$ (lbs)	$L_{f\_ch[i]}$ (lbs)	$L_{r\_ch[i]}$ (lbs)	$S_{ch[i]}$ (lbs)	$R_{ch[i]}$ (lbs)
Ch1	-1392;	0;	0;	0;	0;	0;	0;	0;
Ch2	0;	0;	0;	0;	0;	0;	0;	0;

From IBC 2021 cl.1605.1 Basic load combinations from ASCE 7, section 2.4

Load combination no.1 D + 0.6W
Load combination no.2 D + 0.7E
Load combination no.3 D + 0.75L_f + 0.45W + 0.75(L_r or S or R)
Load combination no.4 D + 0.75L_f + 0.525E + 0.75S
Load combination no.5 0.6D + 0.6W
Load combination no.6 0.6D + 0.7E

Adjustment factors

Load duration factor – Table 2.3.2 C_D = **1.60**
Size factor for tension – Table 4A C_{Ft} = **1.50**
Size factor for compression – Table 4A C_{Fc} = **1.15**
Wet service factor for tension – Table 4A C_{Mt} = **1.00**
Wet service factor for compression – Table 4A C_{Mc} = **1.00**
Wet service factor for modulus of elasticity – Table 4A
C_{ME} = **1.00**
Temperature factor for tension – Table 2.3.3 C_{tt} = **1.00**
Temperature factor for compression – Table 2.3.3
C_{tc} = **1.00**
Temperature factor for modulus of elasticity – Table 2.3.3

LEVEL 3

	C_{tE} = **1.00**
Incising factor – cl.4.3.8	C_i = **1.00**
Buckling stiffness factor – cl.4.4.2	C_T = **1.00**
Adjusted modulus of elasticity	$E_{min}' = E_{min} \times C_{ME} \times C_{tE} \times C_i \times C_T$ = **550000** psi
Critical buckling design value	$F_{cE} = 0.822 \times E_{min}' / (h / d)^2$ = **366** psi
Reference compression design value	$F_c^* = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i$ = **2484** psi
For sawn lumber	c = **0.8**
Column stability factor – eqn.3.7-1	$C_P = (1 + (F_{cE} / F_c^*)) / (2 \times c) - \sqrt{([(1 + (F_{cE} / F_c^*)) / (2 \times c)]^2 - (F_{cE} / F_c^*) / c)}$ = **0.14**

From SDPWS Table 4.3.4 Maximum Shear Wall Aspect Ratios

Maximum shear wall aspect ratio	3.5
Shear wall length	b = **40** ft
Shear wall aspect ratio	h / b = **0.256**

Segmented shear wall capacity

Maximum shear force under wind loading	$V_{w\_max} = 0.6 \times W$ = **4.32** kips
Shear capacity for wind loading	$V_w = v_w \times b / 2$ = **14.601** kips
	V_{w_max} / V_w = **0.296**
	PASS - Shear capacity for wind load exceeds maximum shear force
Maximum shear force under seismic loading	$V_{s\_max} = 0.7 \times E_q$ = **0.7** kips
Shear capacity for seismic loading	$V_s = v_s \times b / 2$ = **10.416** kips
	V_{s_max} / V_s = **0.067**
	PASS - Shear capacity for seismic load exceeds maximum shear force

Chord capacity for chord 1

Shear wall aspect ratio	h / b = **0.256**
Load combination 5	
Shear force for maximum tension	$V = 0.6 \times W$ = **4.32** kips
Axial force for maximum tension	$P = (0.6 \times (D + S_{wt} \times h)) \times b_1 / 2 + 0.6 \times W_{ch1}$ = **6.539** kips
Maximum tensile force in chord	$T = V \times h / (b) - P$ = $\boxed{\textbf{-5.432 kips}}$ NEGATIVE. NO OVERTURNING
Maximum applied tensile stress	$f_t = T / A_{en}$ = **-1448** lb/in^2
Design tensile stress	$F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i$ = **1740** lb/in^2
	f_t / F_t' = **-0.832**
	PASS - Design tensile stress exceeds maximum applied tensile stress
Load combination 1	
Shear force for maximum compression	$V = 0.6 \times W$ = **4.32** kips
Axial force for maximum compression	$P = ((D + S_{wt} \times h)) \times s / 2 + -1 \times 0.6 \times W_{ch1}$ = **1.245** kips
Maximum compressive force in chord	$C = V \times h / (b) + P$ = $\boxed{\textbf{2.352 kips}}$
Maximum applied compressive stress	$f_c = C / A_e$ = **448** lb/in^2
Design compressive stress	$F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P$ = **354** lb/in^2
	f_c / F_c' = **1.264**
	FAIL - Design compressive stress is less than maximum applied compressive stress

Chord capacity for chord 2

Load combination 5

LEVEL 3

Shear force for maximum tension	$V = 0.6 \times W =$ **4.32** kips
Axial force for maximum tension	$P = (0.6 \times (D + S_{wt} \times h)) \times b_1 / 2 =$ **7.374** kips
Maximum tensile force in chord	$T = V \times h / (b) - P =$ $\boxed{\textbf{-6.267}\text{ kips}}$ ← NEGATIVE. NO OVERTURNING
Maximum applied tensile stress	$f_t = T / A_{en} =$ **-1671** lb/in^2
Design tensile stress	$F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i =$ **1740** lb/in^2
	$f_t / F_t' =$ **-0.960**
	PASS - Design tensile stress exceeds maximum applied tensile stress
Load combination 3	
Shear force for maximum compression	$V = 0.45 \times W =$ **3.24** kips
Axial force for maximum compression	$P = ((D + S_{wt} \times h) + 0.75 \times L_f) \times s / 2 =$ **0.73** kips
Maximum compressive force in chord	$C = V \times h / (b) + P =$ $\boxed{\textbf{1.560}\text{ kips}}$
Maximum applied compressive stress	$f_c = C / A_e =$ **297** lb/in^2
Design compressive stress	$F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P =$ **354** lb/in^2
	$f_c / F_c' =$ **0.839**

PASS - Design compressive stress exceeds maximum applied compressive stress

Wind load deflection

Design shear force	$V_{\delta w} = f_{Wserv} \times W =$ **7.2** kips
Deflection limit	$\Delta_{w\_allow} = h / 600 =$ **0.205** in
Induced unit shear	$v_{\delta w} = V_{\delta w} / b =$ **180** lb/ft
Anchor tension force	$T_\delta = \max(0\text{ kips}, v_{\delta w} \times h - 0.6 \times (D + S_{wt} \times h) \times b / 2 + \max(\text{abs}(W_{ch1}), \text{abs}(W_{ch2}))) =$ **0.000** kips
Shear wall deflection – Eqn. 4.3-1	$\delta_{sww} = 2 \times v_{\delta w} \times h^3 / (3 \times E \times A_e \times b) + v_{\delta w} \times h / (G_a) + h \times T_\delta / (k_a \times b) =$ **0.12** in
	$\delta_{sww} / \Delta_{w\_allow} =$ **0.587**

PASS - Shear wall deflection is less than deflection limit

Seismic deflection

Design shear force	$V_{\delta s} = E_q =$ **1** kips
Deflection limit	$\Delta_{s\_allow} = 0.020 \times h =$ **2.46** in
Induced unit shear	$v_{\delta s} = V_{\delta s} / b =$ **25** lb/ft
Anchor tension force	$T_\delta = \max(0\text{ kips}, v_{\delta s} \times h - (0.6 - 0.2 \times S_{DS}) \times (D + S_{wt} \times h) \times b / 2) =$ **0.000** kips
Shear wall elastic deflection – Eqn. 4.3-1	$\delta_{swse} = 2 \times v_{\delta s} \times h^3 / (3 \times E \times A_e \times b) + v_{\delta s} \times h / (G_a) + h \times T_\delta / (k_a \times b) =$ **0.017** in
Deflection ampification factor	$C_{d\delta} =$ **4**
Seismic importance factor	$I_e =$ **1.25**
Amp. seis. deflection – ASCE7 Eqn. 12.8-15	$\delta_{sws} = C_{d\delta} \times \delta_{swse} / I_e =$ **0.053** in
	$\delta_{sws} / \Delta_{s\_allow} =$ **0.022**

PASS - Shear wall deflection is less than deflection limit

LEVEL 2

WOOD SHEAR WALL DESIGN (NDS)

In accordance with NDS2018 allowable stress design and the segmented shear wall method

Tedds calculation version 1.2.08

Design summary

Description	Unit	Provided	Required	Utilization	Result
Shear capacity	lbs	14601	6720	0.460	PASS
Chord capacity	lb/in^2	354	271	0.765	PASS
Deflection	in	0.205	0.182	0.887	PASS

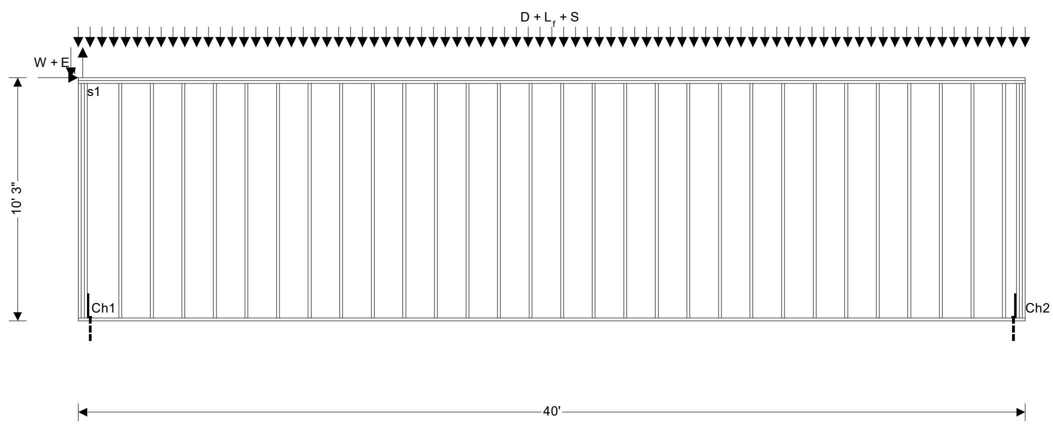
Panel details

Structural I wood panel sheathing on one side

Panel height h = **10.25** ft

Panel length b = **40** ft

Total area of wall A = h × b = **410** ft^2



Panel construction

Nominal stud size 2" x 4"

Dressed stud size 1.5" x 3.5"

Cross-sectional area of studs A_s = **5.25** in^2

Stud spacing s = **16** in

Nominal end post size 3 x 2" x 4"

Dressed end post size 3 x 1.5" x 3.5"

Cross-sectional area of end posts A_e = **15.75** in^2

Hole diameter Dia = **1** in

Net cross-sectional area of end posts A_{en} = **11.25** in^2

Nominal collector size 2 x 2" x 4"

Dressed collector size 2 x 1.5" x 3.5"

Service condition Dry

Temperature 100 degF or less

Vertical anchor stiffness k_a = **30000** lb/in

From NDS Supplement Table 4A - Reference design values for visually graded dimension lumber (2" - 4" thick)

Species, grade and size classification Hem-Fir, no.1 & btr grade, 2" & wider

Specific gravity G = **0.43**

LEVEL 2

Tension parallel to grain F_t = **725** lb/in^2

Compression parallel to grain F_c = **1350** lb/in^2

Modulus of elasticity E = **1500000** lb/in^2

Minimum modulus of elasticity E_{min} = **550000** lb/in^2

Sheathing details

Sheathing material 7/16" wood panel structural I oriented strandboard sheathing

Fastener type 8d common nails at 6"centers

From SDPWS Table 4.3A Nominal Unit Shear Capacities for Wood-Frame Shear Walls - Wood-based Panels

Nominal unit shear capacity for seismic design v_s = 560 plf × min[1 - (0.5 - G), 1] = **520.8** lb/ft

Nominal unit shear capacity for wind design v_w = 785 plf × min[1 - (0.5 - G), 1] = **730.1** lb/ft

Apparent shear wall shear stiffness G_a = **16** kips/in

Loading details

Dead load acting on top of panel D = **774** lb/ft

Floor live load acting on top of panel L_f = **512** lb/ft

Snow load acting on top of panel S = **1280** lb/ft

Self weight of panel S_{wt} = **10** lb/ft^2

In plane wind load acting at head of panel W = **11200** lbs

Wind load serviceability factor f_{Wserv} = **1.00**

In plane seismic load acting at head of panel E_q = **1600** lbs

Design spectral response accel. par., short periods S_{DS} = **0.333**

Chord forces from shear walls above

Chord	$W_{ch[i]}$ (lbs)	$E_{q\_ch[i]}$ (lbs)	$D_{C\_ch[i]}$ (lbs)	$D_{T\_ch[i]}$ (lbs)	$L_{f\_ch[i]}$ (lbs)	$L_{r\_ch[i]}$ (lbs)	$S_{ch[i]}$ (lbs)	$R_{ch[i]}$ (lbs)
Ch1	-3272;	0;	0;	0;	0;	0;	0;	0;
Ch2	3272;	0;	0;	0;	0;	0;	0;	0;

From IBC 2021 cl.1605.1 Basic load combinations from ASCE 7, section 2.4

Load combination no.1 D + 0.6W

Load combination no.2 D + 0.7E

Load combination no.3 D + 0.75L_f + 0.45W + 0.75(L_r or S or R)

Load combination no.4 D + 0.75L_f + 0.525E + 0.75S

Load combination no.5 0.6D + 0.6W

Load combination no.6 0.6D + 0.7E

Adjustment factors

Load duration factor – Table 2.3.2 C_D = **1.60**

Size factor for tension – Table 4A C_{Ft} = **1.50**

Size factor for compression – Table 4A C_{Fc} = **1.15**

Wet service factor for tension – Table 4A C_{Mt} = **1.00**

Wet service factor for compression – Table 4A C_{Mc} = **1.00**

Wet service factor for modulus of elasticity – Table 4A

C_{ME} = **1.00**

Temperature factor for tension – Table 2.3.3 C_{tt} = **1.00**

Temperature factor for compression – Table 2.3.3

C_{tc} = **1.00**

Temperature factor for modulus of elasticity – Table 2.3.3

LEVEL 2

	C_{tE} = **1.00**
Incising factor – cl.4.3.8	C_i = **1.00**
Buckling stiffness factor – cl.4.4.2	C_T = **1.00**
Adjusted modulus of elasticity	$E_{min}' = E_{min} \times C_{ME} \times C_{tE} \times C_i \times C_T$ = **550000** psi
Critical buckling design value	$F_{cE} = 0.822 \times E_{min}' / (h / d)^2$ = **366** psi
Reference compression design value	$F_c^* = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i$ = **2484** psi
For sawn lumber	c = **0.8**
Column stability factor – eqn.3.7-1	$C_P = (1 + (F_{cE} / F_c^*)) / (2 \times c) - \sqrt{[(1 + (F_{cE} / F_c^*)) / (2 \times c)]^2 - (F_{cE} / F_c^*) / c}$ = **0.14**

From SDPWS Table 4.3.4 Maximum Shear Wall Aspect Ratios

Maximum shear wall aspect ratio	3.5
Shear wall length	b = **40** ft
Shear wall aspect ratio	h / b = **0.256**

Segmented shear wall capacity

Maximum shear force under wind loading	$V_{w\_max} = 0.6 \times W$ = **6.72** kips
Shear capacity for wind loading	$V_w = v_w \times b / 2$ = **14.601** kips
	V_{w_max} / V_w = **0.46**

PASS - Shear capacity for wind load exceeds maximum shear force

Maximum shear force under seismic loading	$V_{s\_max} = 0.7 \times E_q$ = **1.12** kips
Shear capacity for seismic loading	$V_s = v_s \times b / 2$ = **10.416** kips
	V_{s_max} / V_s = **0.108**

PASS - Shear capacity for seismic load exceeds maximum shear force

Chord capacity for chord 1

Shear wall aspect ratio	h / b = **0.256**
Load combination 5	
Shear force for maximum tension	$V = 0.6 \times W$ = **6.72** kips
Axial force for maximum tension	$P = (0.6 \times (D + S_{wt} \times h)) \times b_1 / 2 + 0.6 \times W_{ch1}$ = **8.555** kips
Maximum tensile force in chord	$T = V \times h / (b) - P$ = $\boxed{\textbf{-6.833 kips}}$ NEGATIVE. NO OVERTURNING
Maximum applied tensile stress	$f_t = T / A_{en}$ = **-607** lb/in^2
Design tensile stress	$F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i$ = **1740** lb/in^2
	f_t / F_t' = **-0.349**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 1	
Shear force for maximum compression	$V = 0.6 \times W$ = **6.72** kips
Axial force for maximum compression	$P = ((D + S_{wt} \times h)) \times s / 2 + -1 \times 0.6 \times W_{ch1}$ = **2.548** kips
Maximum compressive force in chord	$C = V \times h / (b) + P$ = $\boxed{\textbf{4.270 kips}}$
Maximum applied compressive stress	$f_c = C / A_e$ = **271** lb/in^2
Design compressive stress	$F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P$ = **354** lb/in^2
	f_c / F_c' = **0.765**

PASS - Design compressive stress exceeds maximum applied compressive stress

Chord capacity for chord 2

Load combination 5

LEVEL 2

Shear force for maximum tension $V = 0.6 \times W = \mathbf{6.72}$ kips

Axial force for maximum tension $P = (0.6 \times (D + S_{wt} \times h)) \times b_1 / 2 + -1 \times 0.6 \times W_{ch2} = \mathbf{8.555}$ kips

Maximum tensile force in chord $T = V \times h / (b) - P = \boxed{\mathbf{-6.833}\text{ kips}}$ NEGATIVE. NO OVERTURNING

Maximum applied tensile stress $f_t = T / A_{en} = \mathbf{-607}$ lb/in^2

Design tensile stress $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i = \mathbf{1740}$ lb/in^2

$f_t / F_t' = \mathbf{-0.349}$

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 1

Shear force for maximum compression $V = 0.6 \times W = \mathbf{6.72}$ kips

Axial force for maximum compression $P = ((D + S_{wt} \times h)) \times s / 2 + 0.6 \times W_{ch2} = \mathbf{2.548}$ kips

Maximum compressive force in chord $C = V \times h / (b) + P = \boxed{\mathbf{4.270}\text{ kips}}$

Maximum applied compressive stress $f_c = C / A_e = \mathbf{271}$ lb/in^2

Design compressive stress $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P = \mathbf{354}$ lb/in^2

$f_c / F_c' = \mathbf{0.765}$

PASS - Design compressive stress exceeds maximum applied compressive stress

Wind load deflection

Design shear force $V_{\delta w} = f_{Wserv} \times W = \mathbf{11.2}$ kips

Deflection limit $\Delta_{w\_allow} = h / 600 = \mathbf{0.205}$ in

Induced unit shear $v_{\delta w} = V_{\delta w} / b = \mathbf{280}$ lb/ft

Anchor tension force $T_\delta = \max(0\text{ kips}, v_{\delta w} \times h - 0.6 \times (D + S_{wt} \times h) \times b / 2 + \max(\text{abs}(W_{ch1}), \text{abs}(W_{ch2}))) = \mathbf{0.000}$ kips

Shear wall deflection – Eqn. 4.3-1 $\delta_{sww} = 2 \times v_{\delta w} \times h^3 / (3 \times E \times A_e \times b) + v_{\delta w} \times h / (G_a) + h \times T_\delta / (k_a \times b) = \mathbf{0.182}$ in

$\delta_{sww} / \Delta_{w\_allow} = \mathbf{0.887}$

PASS - Shear wall deflection is less than deflection limit

Seismic deflection

Design shear force $V_{\delta s} = E_q = \mathbf{1.6}$ kips

Deflection limit $\Delta_{s\_allow} = 0.020 \times h = \mathbf{2.46}$ in

Induced unit shear $v_{\delta s} = V_{\delta s} / b = \mathbf{40}$ lb/ft

Anchor tension force $T_\delta = \max(0\text{ kips}, v_{\delta s} \times h - (0.6 - 0.2 \times S_{DS}) \times (D + S_{wt} \times h) \times b / 2) = \mathbf{0.000}$ kips

Shear wall elastic deflection – Eqn. 4.3-1 $\delta_{swse} = 2 \times v_{\delta s} \times h^3 / (3 \times E \times A_e \times b) + v_{\delta s} \times h / (G_a) + h \times T_\delta / (k_a \times b) = \mathbf{0.026}$ in

Deflection ampification factor $C_{d\delta} = \mathbf{4}$

Seismic importance factor $I_e = \mathbf{1.25}$

Amp. seis. deflection – ASCE7 Eqn. 12.8-15 $\delta_{sws} = C_{d\delta} \times \delta_{swse} / I_e = \mathbf{0.083}$ in

$\delta_{sws} / \Delta_{s\_allow} = \mathbf{0.034}$

PASS - Shear wall deflection is less than deflection limit

LEVEL 1

WOOD SHEAR WALL DESIGN (NDS)

In accordance with NDS2018 allowable stress design and the segmented shear wall method

Tedds calculation version 1.2.08

Design summary

Description	Unit	Provided	Required	Utilization	Result
Shear capacity	lbs	22413	9120	0.407	PASS
Chord capacity	lb/in^2	354	353	0.996	PASS
Deflection	in	0.205	0.188	0.919	PASS

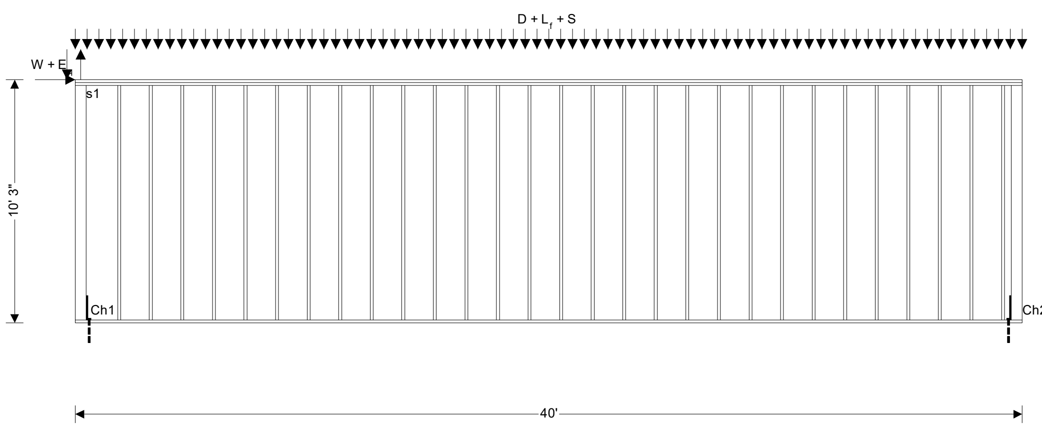
Panel details

Structural I wood panel sheathing on one side

Panel height h = **10.25** ft

Panel length b = **40** ft

Total area of wall A = h × b = **410** ft^2



Panel construction

Nominal stud size 2" x 4"

Dressed stud size 1.5" x 3.5"

Cross-sectional area of studs A_s = **5.25** in^2

Stud spacing s = **16** in

Nominal end post size 6" x 4"

Dressed end post size 5.5" x 3.5"

Cross-sectional area of end posts A_e = **19.25** in^2

Hole diameter Dia = **1** in

Net cross-sectional area of end posts A_{en} = **13.75** in^2

Nominal collector size 2 x 2" x 4"

Dressed collector size 2 x 1.5" x 3.5"

Service condition Dry

Temperature 100 degF or less

Vertical anchor stiffness k_a = **30000** lb/in

From NDS Supplement Table 4A - Reference design values for visually graded dimension lumber (2" - 4" thick)

Species, grade and size classification Hem-Fir, no.1 & btr grade, 2" & wider

Specific gravity G = **0.43**

LEVEL 1

Tension parallel to grain F_t = **725** lb/in^2
Compression parallel to grain F_c = **1350** lb/in^2
Modulus of elasticity E = **1500000** lb/in^2
Minimum modulus of elasticity E_{min} = **550000** lb/in^2

Sheathing details

Sheathing material 7/16" wood panel structural I oriented strandboard sheathing
Fastener type 8d common nails at 4"centers

From SDPWS Table 4.3A Nominal Unit Shear Capacities for Wood-Frame Shear Walls - Wood-based Panels

Nominal unit shear capacity for seismic design v_s = 860 plf × min[1 - (0.5 - G), 1] = **799.8** lb/ft
Nominal unit shear capacity for wind design v_w = 1205 plf × min[1 - (0.5 - G), 1] = **1120.6** lb/ft
Apparent shear wall shear stiffness G_a = **21** kips/in

Loading details

Dead load acting on top of panel D = **1024** lb/ft
Floor live load acting on top of panel L_f = **1920** lb/ft
Snow load acting on top of panel S = **1280** lb/ft
Self weight of panel S_{wt} = **10** lb/ft^2
In plane wind load acting at head of panel W = **15200** lbs
Wind load serviceability factor f_{Wserv} = **1.00**
In plane seismic load acting at head of panel E_q = **2250** lbs
Design spectral response accel. par., short periods S_{DS} = **0.333**

Chord forces from shear walls above

Chord	$W_{ch[i]}$ (lbs)	$E_{q\_ch[i]}$ (lbs)	$D_{C\_ch[i]}$ (lbs)	$D_{T\_ch[i]}$ (lbs)	$L_{f\_ch[i]}$ (lbs)	$L_{r\_ch[i]}$ (lbs)	$S_{ch[i]}$ (lbs)	$R_{ch[i]}$ (lbs)
Ch1	-6177;	0;	0;	0;	0;	0;	0;	0;
Ch2	6177;	0;	0;	0;	0;	0;	0;	0;

From IBC 2021 cl.1605.1 Basic load combinations from ASCE 7, section 2.4

Load combination no.1 D + 0.6W
Load combination no.2 D + 0.7E
Load combination no.3 D + 0.75L_f + 0.45W + 0.75(L_r or S or R)
Load combination no.4 D + 0.75L_f + 0.525E + 0.75S
Load combination no.5 0.6D + 0.6W
Load combination no.6 0.6D + 0.7E

Adjustment factors

Load duration factor – Table 2.3.2 C_D = **1.60**
Size factor for tension – Table 4A C_{Ft} = **1.50**
Size factor for compression – Table 4A C_{Fc} = **1.15**
Wet service factor for tension – Table 4A C_{Mt} = **1.00**
Wet service factor for compression – Table 4A C_{Mc} = **1.00**
Wet service factor for modulus of elasticity – Table 4A
C_{ME} = **1.00**
Temperature factor for tension – Table 2.3.3 C_{tt} = **1.00**
Temperature factor for compression – Table 2.3.3
C_{tc} = **1.00**
Temperature factor for modulus of elasticity – Table 2.3.3

LEVEL 1

	C_{tE} = **1.00**
Incising factor – cl.4.3.8	C_i = **1.00**
Buckling stiffness factor – cl.4.4.2	C_T = **1.00**
Adjusted modulus of elasticity	$E_{min}' = E_{min} \times C_{ME} \times C_{tE} \times C_i \times C_T$ = **550000** psi
Critical buckling design value	$F_{cE} = 0.822 \times E_{min}' / (h / d)^2$ = **366** psi
Reference compression design value	$F_c^* = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i$ = **2484** psi
For sawn lumber	c = **0.8**
Column stability factor – eqn.3.7-1	$C_P = (1 + (F_{cE} / F_c^*)) / (2 \times c) - \sqrt{([(1 + (F_{cE} / F_c^*)) / (2 \times c)]^2 - (F_{cE} / F_c^*) / c)}$ = **0.14**

From SDPWS Table 4.3.4 Maximum Shear Wall Aspect Ratios

Maximum shear wall aspect ratio	3.5
Shear wall length	b = **40** ft
Shear wall aspect ratio	h / b = **0.256**

Segmented shear wall capacity

Maximum shear force under wind loading	$V_{w\_max} = 0.6 \times W$ = **9.12** kips
Shear capacity for wind loading	$V_w = v_w \times b / 2$ = **22.413** kips
	V_{w_max} / V_w = **0.407**

PASS - Shear capacity for wind load exceeds maximum shear force

Maximum shear force under seismic loading	$V_{s\_max} = 0.7 \times E_q$ = **1.575** kips
Shear capacity for seismic loading	$V_s = v_s \times b / 2$ = **15.996** kips
	V_{s_max} / V_s = **0.098**

PASS - Shear capacity for seismic load exceeds maximum shear force

Chord capacity for chord 1

Shear wall aspect ratio	h / b = **0.256**
Load combination 5	
Shear force for maximum tension	$V = 0.6 \times W$ = **9.12** kips
Axial force for maximum tension	$P = (0.6 \times (D + S_{wt} \times h)) \times b_1 / 2 + 0.6 \times W_{ch1}$ = **9.812** kips
Maximum tensile force in chord	$T = V \times h / (b) - P$ = $\boxed{\textbf{-7.475 kips}}$ NEGATIVE. NO OVERTURNING
Maximum applied tensile stress	$f_t = T / A_{en}$ = **-544** lb/in^2
Design tensile stress	$F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i$ = **1740** lb/in^2
	f_t / F_t' = **-0.312**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 1	
Shear force for maximum compression	$V = 0.6 \times W$ = **9.12** kips
Axial force for maximum compression	$P = ((D + S_{wt} \times h)) \times s / 2 + -1 \times 0.6 \times W_{ch1}$ = **4.457** kips
Maximum compressive force in chord	$C = V \times h / (b) + P$ = $\boxed{\textbf{6.794 kips}}$
Maximum applied compressive stress	$f_c = C / A_e$ = **353** lb/in^2
Design compressive stress	$F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P$ = **354** lb/in^2
	f_c / F_c' = **0.996**

PASS - Design compressive stress exceeds maximum applied compressive stress

Chord capacity for chord 2

Load combination 5

LEVEL 1

Shear force for maximum tension — $V = 0.6 \times W =$ **9.12** kips

Axial force for maximum tension — $P = (0.6 \times (D + S_{wt} \times h)) \times b_1 / 2 + -1 \times 0.6 \times W_{ch2} =$ **9.812** kips

Maximum tensile force in chord — $T = V \times h / (b) - P =$ $\boxed{\textbf{-7.475} \text{ kips}}$ NEGATIVE. NO OVERTURNING

Maximum applied tensile stress — $f_t = T / A_{en} =$ **-544** lb/in^2

Design tensile stress — $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i =$ **1740** lb/in^2

$f_t / F_t' =$ **-0.312**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 1

Shear force for maximum compression — $V = 0.6 \times W =$ **9.12** kips

Axial force for maximum compression — $P = ((D + S_{wt} \times h)) \times s / 2 + 0.6 \times W_{ch2} =$ **4.457** kips

Maximum compressive force in chord — $C = V \times h / (b) + P =$ $\boxed{\textbf{6.794} \text{ kips}}$

Maximum applied compressive stress — $f_c = C / A_e =$ **353** lb/in^2

Design compressive stress — $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P =$ **354** lb/in^2

$f_c / F_c' =$ **0.996**

PASS - Design compressive stress exceeds maximum applied compressive stress

Wind load deflection

Design shear force — $V_{\delta w} = f_{Wserv} \times W =$ **15.2** kips

Deflection limit — $\Delta_{w\_allow} = h / 600 =$ **0.205** in

Induced unit shear — $v_{\delta w} = V_{\delta w} / b =$ **380** lb/ft

Anchor tension force — $T_\delta = \max(0 \text{ kips}, v_{\delta w} \times h - 0.6 \times (D + S_{wt} \times h) \times b / 2 + \max(\text{abs}(W_{ch1}), \text{abs}(W_{ch2}))) =$ **0.000** kips

Shear wall deflection – Eqn. 4.3-1 — $\delta_{sww} = 2 \times v_{\delta w} \times h^3 / (3 \times E \times A_e \times b) + v_{\delta w} \times h / (G_a) + h \times T_\delta / (k_a \times b) =$ **0.188** in

$\delta_{sww} / \Delta_{w\_allow} =$ **0.919**

PASS - Shear wall deflection is less than deflection limit

Seismic deflection

Design shear force — $V_{\delta s} = E_q =$ **2.25** kips

Deflection limit — $\Delta_{s\_allow} = 0.020 \times h =$ **2.46** in

Induced unit shear — $v_{\delta s} = V_{\delta s} / b =$ **56.25** lb/ft

Anchor tension force — $T_\delta = \max(0 \text{ kips}, v_{\delta s} \times h - (0.6 - 0.2 \times S_{DS}) \times (D + S_{wt} \times h) \times b / 2) =$ **0.000** kips

Shear wall elastic deflection – Eqn. 4.3-1 — $\delta_{swse} = 2 \times v_{\delta s} \times h^3 / (3 \times E \times A_e \times b) + v_{\delta s} \times h / (G_a) + h \times T_\delta / (k_a \times b) =$ **0.028** in

Deflection ampification factor — $C_{d\delta} =$ **4**

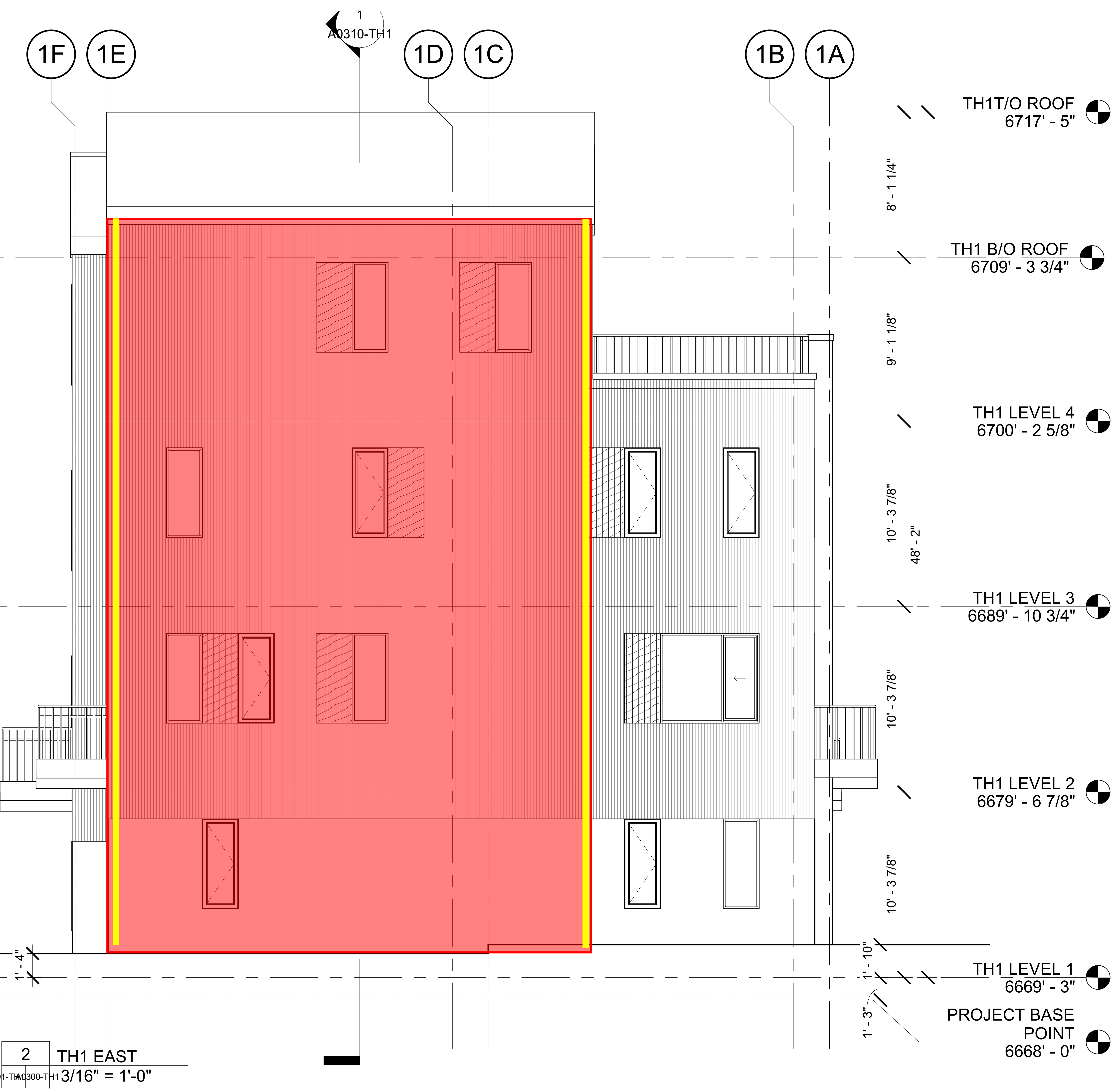
Seismic importance factor — $I_e =$ **1.25**

Amp. seis. deflection – ASCE7 Eqn. 12.8-15 — $\delta_{sws} = C_{d\delta} \times \delta_{swse} / I_e =$ **0.089** in

$\delta_{sws} / \Delta_{s\_allow} =$ **0.036**

PASS - Shear wall deflection is less than deflection limit

N-S WALLS
EAST, WEST ELEVATIONS
(PERFORATED METHOD - TEDDS)



2 TH1 EAST
3/16" = 1'-0"

2 TH1 WEST
A0201-TH1 A0301-TH1 3/16" = 1'-0"

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WEST, LEVEL 4

WOOD SHEAR WALL DESIGN (NDS)

In accordance with NDS2018 allowable stress design and the perforated shear wall method

Tedds calculation version 1.2.08

Design summary

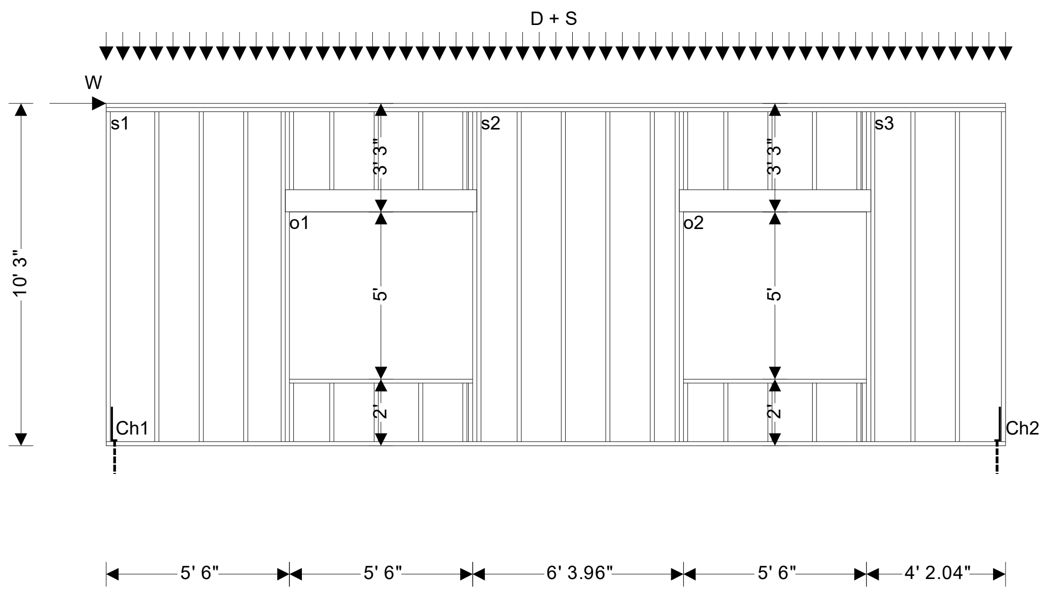
Description	Unit	Provided	Required	Utilization	Result
Shear capacity	lbs	4790	1080	0.225	PASS
Chord capacity	lb/in^2	709	123	0.174	PASS
Collector capacity	lb/in^2	1092	19	0.017	PASS
Deflection	in	0.246	0.095	0.387	PASS

Panel details

Structural I wood panel sheathing on one side

Panel height h = **10.25** ft

Panel length b = **27** ft



Panel opening details

Width of opening w_{o1} = **5.5** ft

Height of opening h_{o1} = **5** ft

Height to underside of lintel over opening l_{o1} = **7** ft

Position of opening P_{o1} = **5.5** ft

Width of opening w_{o2} = **5.5** ft

Height of opening h_{o2} = **5** ft

Height to underside of lintel over opening l_{o2} = **7** ft

Position of opening P_{o2} = **17.33** ft

Total area of wall $A = h \cdot b - w_{o1} \cdot h_{o1} - w_{o2} \cdot h_{o2}$ = **221.75** ft^2

Panel construction

Nominal stud size 2" x 6"

Dressed stud size 1.5" x 5.5"

WEST, LEVEL 4

Cross-sectional area of studs	A_s = **8.25** in^2
Stud spacing	s = **16** in
Nominal end post size	2" x 6"
Dressed end post size	1.5" x 5.5"
Cross-sectional area of end posts	A_e = **8.25** in^2
Hole diameter	Dia = **1** in
Net cross-sectional area of end posts	A_{en} = **6.75** in^2
Nominal collector size	2 x 2" x 6"
Dressed collector size	2 x 1.5" x 5.5"
Service condition	Dry
Temperature	100 degF or less
Vertical anchor stiffness	k_a = **30000** lb/in

From NDS Supplement Table 4A - Reference design values for visually graded dimension lumber (2" - 4" thick)

Species, grade and size classification	Hem-Fir, no.2 grade, 2" & wider
Specific gravity	G = **0.43**
Tension parallel to grain	F_t = **525** lb/in^2
Compression parallel to grain	F_c = **1300** lb/in^2
Modulus of elasticity	E = **1300000** lb/in^2
Minimum modulus of elasticity	E_{min} = **470000** lb/in^2

Sheathing details

Sheathing material	7/16" wood panel structural I oriented strandboard sheathing
Fastener type	8d common nails at 6"centers

From SDPWS Table 4.3A Nominal Unit Shear Capacities for Wood-Frame Shear Walls - Wood-based Panels

Nominal unit shear capacity for seismic design	v_s = min(560 plf · min[1 - (0.5 - G), 1], 1740 plf) = **520.8** lb/ft
Nominal unit shear capacity for wind design	v_w = min(785 plf · min[1 - (0.5 - G), 1], 2435 plf) = **730.1** lb/ft
Apparent shear wall shear stiffness	G_a = **16** kips/in

Loading details

Dead load acting on top of panel	D = **160** lb/ft
Snow load acting on top of panel	S = **800** lb/ft
Self weight of panel	S_{wt} = **10** lb/ft^2
In plane wind load acting at head of panel	W = **1800** lbs
Wind load serviceability factor	f_{Wserv} = **1.00**

From IBC 2021 cl.1605.1 Basic load combinations from ASCE 7, section 2.4

Load combination no.1	D + 0.6W
Load combination no.2	D + 0.7E
Load combination no.3	D + 0.75L_f + 0.45W + 0.75(L_r or S or R)
Load combination no.4	D + 0.75L_f + 0.525E + 0.75S
Load combination no.5	0.6D + 0.6W
Load combination no.6	0.6D + 0.7E

Adjustment factors

Load duration factor – Table 2.3.2	C_D = **1.60**
Size factor for tension – Table 4A	C_{Ft} = **1.30**
Size factor for compression – Table 4A	C_{Fc} = **1.10**

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Wet service factor for tension – Table 4A C_{Mt} = **1.00**

Wet service factor for compression – Table 4A C_{Mc} = **1.00**

Wet service factor for modulus of elasticity – Table 4A

C_{ME} = **1.00**

Temperature factor for tension – Table 2.3.3 C_{tt} = **1.00**

Temperature factor for compression – Table 2.3.3

C_{tc} = **1.00**

Temperature factor for modulus of elasticity – Table 2.3.3

C_{tE} = **1.00**

Incising factor – cl.4.3.8 C_i = **1.00**

Buckling stiffness factor – cl.4.4.2 C_T = **1.00**

Adjusted modulus of elasticity $E_{min}' = E_{min} \cdot C_{ME} \cdot C_{tE} \cdot C_i \cdot C_T$ = **470000** psi

Critical buckling design value $F_{cE} = 0.822 \cdot E_{min}' / (h / d)^2$ = **772** psi

Reference compression design value $F_c^* = F_c \cdot C_D \cdot C_{Mc} \cdot C_{tc} \cdot C_{Fc} \cdot C_i$ = **2288** psi

For sawn lumber c = **0.8**

Column stability factor – eqn.3.7-1 $C_P = (1 + (F_{cE} / F_c^*)) / (2 \cdot c) - \sqrt{[(1 + (F_{cE} / F_c^*)) / (2 \cdot c)]^2 - (F_{cE} / F_c^*) / c}$ = **0.31**

From SDPWS Table 4.3.4 Maximum Shear Wall Aspect Ratios

Maximum shear wall aspect ratio 3.5

Perforated wall length b_1 = **5.5** ft

Shear wall aspect ratio h / b_1 = **1.864**

Perforated wall length b_2 = **6.33** ft

Shear wall aspect ratio h / b_2 = **1.619**

Perforated wall length b_3 = **4.17** ft

Shear wall aspect ratio h / b_3 = **2.458**

Shear capacity adjustment factor – cl.4.3.3.5

Sum of perforated shear wall lengths $SL_i = b_1 + b_2 + b_3 \cdot 2 \cdot b_s / h$ = **15.223** ft

Total length of perforated shear wall $L_{tot} = b_1 + w_{o1} + b_2 + w_{o2} + b_3$ = **27** ft

Total area of openings $A_o = w_{o1} \cdot h_{o1} + w_{o2} \cdot h_{o2}$ = **55** ft^2

Sheathing area ratio (eqn. 4.3-6) $r = 1 / (1 + A_o /(h \cdot SL_i))$ = **0.739**

Shear capacity adjustment factor (eqn. 4.3-5) C_o = **0.862**

Perforated shear wall capacity

Maximum shear force under wind loading $V_{w\_max} = 0.6 \cdot W$ = **1.08** kips

Shear capacity for wind loading $V_w = v_w \cdot C_o \cdot SL_i / 2$ = **4.79** kips

V_{w_max} / V_w = **0.225**

PASS - Shear capacity for wind load exceeds maximum shear force

Chord capacity for chords 1 and 2

Load combination 5

Shear force for maximum tension $V = 0.6 \cdot W$ = **1.08** kips

Axial force for maximum tension $P = (0.6 \cdot (D + S_{wt} \cdot h)) \cdot b / 2$ = **2.126** kips

Maximum tensile force in chord $T = V \cdot h / ((C_o \cdot SL_i)) - P$ = **-1.283** kips

NEGATIVE. NO OVERTURNING

Maximum applied tensile stress $f_t = T / A_{en}$ = **-190** lb/in^2

Design tensile stress $F_t' = F_t \cdot C_D \cdot C_{Mt} \cdot C_{tt} \cdot C_{Ft} \cdot C_i$ = **1092** lb/in^2

f_t / F_t' = **-0.174**

WEST, LEVEL 4

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 1

Shear force for maximum compression — V = 0.6 · W = **1.08** kips

Axial force for maximum compression — P = ((D + S_{wt} · h)) · s / 2 = **0.175** kips

Maximum compressive force in chord — C = V · h / ((C_o · SL_i)) + P = **1.019** kips

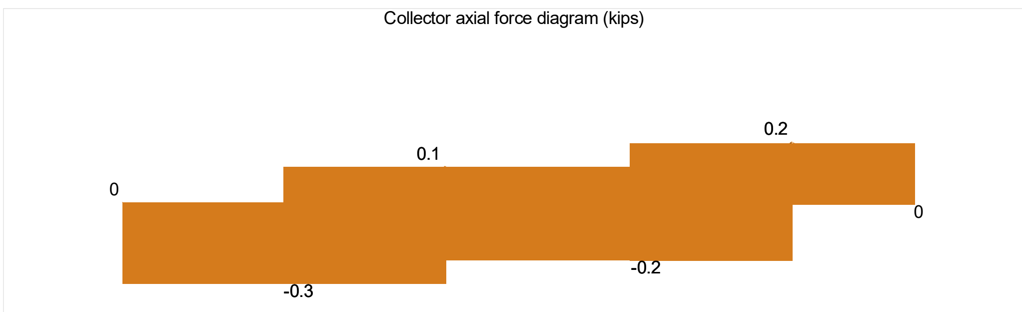
Maximum applied compressive stress — f_c = C / A_e = **123** lb/in^2

Design compressive stress — F_c' = F_c · C_D · C_{Mc} · C_{tc} · C_{Fc} · C_i · C_P = **709** lb/in^2

f_c / F_c' = **0.174**

PASS - Design compressive stress exceeds maximum applied compressive stress

Collector capacity



Maximum shear force on wall — V_{max} = V_{w_max} = **1.08** kips

Uniform shear applied to wall — v_a = V_{max} / ((C_o · SL_i)) = **82.3** plf

Shear resisted by wall segments — v_b = v_a · b / (b_1 + b_2 + b_3) = **138.9** plf

Maximum force in collector — P_{coll} = **0.311** kips

Maximum applied tensile stress — f_t = P_{coll} / (2 · A_s) = **19** lb/in^2

Design tensile stress — F_t' = F_t · C_D · C_{Mt} · C_{tt} · C_{Ft} · C_i = **1092** lb/in^2

f_t / F_t' = **0.017**

PASS - Design tensile stress exceeds maximum applied tensile stress

Maximum applied compressive stress — f_c = P_{coll} / (2 · A_s) = **19** lb/in^2

Column stability factor — C_P = **1.00**

Design compressive stress — F_c' = F_c · C_D · C_{Mc} · C_{tc} · C_{Fc} · C_i · C_P = **2288** lb/in^2

f_c / F_c' = **0.008**

PASS - Design compressive stress exceeds maximum applied compressive stress

Wind load deflection

Design shear force — V_{dw} = f_{Wserv} · W = **1.8** kips

Deflection limit — D_{w_allow}= h / 500 = **0.246** in

Induced unit shear — v_{dw_max} = V_{dw} / (C_o · SL_i) = **137.16** lb/ft

Anchor tension force — T_d = max(0 kips,v_{dw_max} · h - 0.6 · (D + S_{wt} · h) · b / 2) = **0.000** kips

WEST, LEVEL 4

Shear wall deflection – Eqn. 4.3-1

$d_{sww} = 2 \cdot v_{dw\_max} \cdot h^3 / (3 \cdot E \cdot A_e \cdot SL_i) + v_{dw\_max} \cdot h / (G_a) + h \cdot T_d / (k_a \cdot SL_i)$ = **0.095** in

d_{sww} / D_{w_allow} = **0.387**

PASS - Shear wall deflection is less than deflection limit

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WEST, LEVEL 3

WOOD SHEAR WALL DESIGN (NDS)

In accordance with NDS2018 allowable stress design and the perforated shear wall method

Tedds calculation version 1.2.08

Design summary

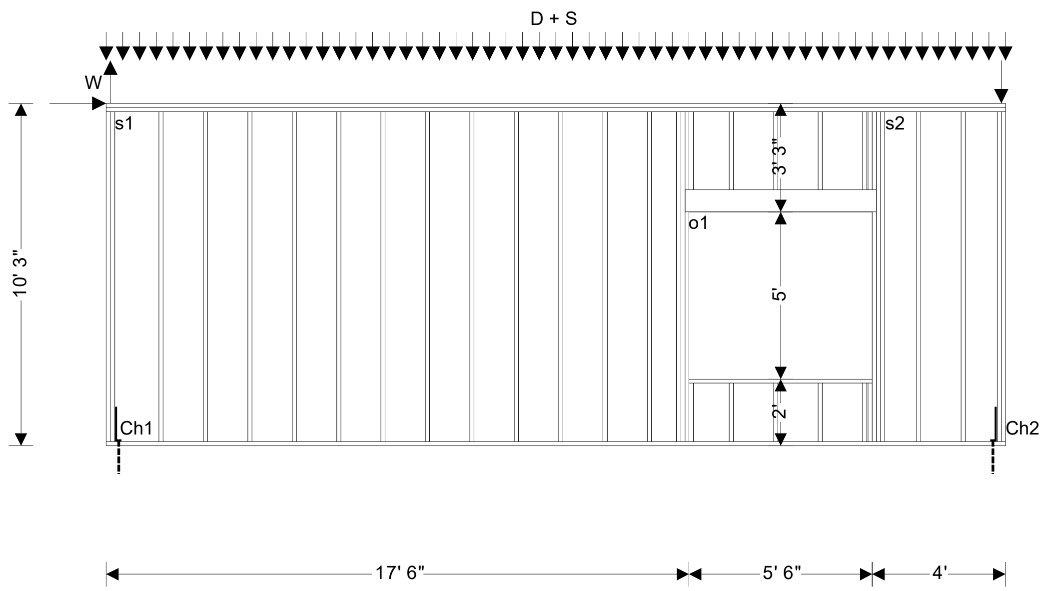
Description	Unit	Provided	Required	Utilization	Result
Shear capacity	lbs	7089	2280	0.322	PASS
Chord capacity	lb/in^2	818	121	0.148	PASS
Collector capacity	lb/in^2	1508	32	0.021	PASS
Deflection	in	0.246	0.129	0.523	PASS

Panel details

Structural I wood panel sheathing on one side

Panel height h = **10.25** ft

Panel length b = **27** ft



Panel opening details

Width of opening w_{o1} = **5.5** ft

Height of opening h_{o1} = **5** ft

Height to underside of lintel over opening l_{o1} = **7** ft

Position of opening P_{o1} = **17.5** ft

Total area of wall $A = h \times b - w_{o1} \times h_{o1}$ = **249.25** ft^2

Panel construction

Nominal stud size 2" x 6"

Dressed stud size 1.5" x 5.5"

Cross-sectional area of studs A_s = **8.25** in^2

Stud spacing s = **16** in

Nominal end post size 2 x 2" x 6"

Dressed end post size 2 x 1.5" x 5.5"

WEST, LEVEL 3

Cross-sectional area of end posts A_e = **16.5** in^2
Hole diameter Dia = **1** in
Net cross-sectional area of end posts A_{en} = **13.5** in^2
Nominal collector size 2 x 2" x 6"
Dressed collector size 2 x 1.5" x 5.5"
Service condition Dry
Temperature 100 degF or less
Vertical anchor stiffness k_a = **30000** lb/in

From NDS Supplement Table 4A - Reference design values for visually graded dimension lumber (2" - 4" thick)

Species, grade and size classification Hem-Fir, no.1 & btr grade, 2" & wider
Specific gravity G = **0.43**
Tension parallel to grain F_t = **725** lb/in^2
Compression parallel to grain F_c = **1350** lb/in^2
Modulus of elasticity E = **1500000** lb/in^2
Minimum modulus of elasticity E_{min} = **550000** lb/in^2

Sheathing details

Sheathing material 7/16" wood panel structural I oriented strandboard sheathing
Fastener type 8d common nails at 6"centers

From SDPWS Table 4.3A Nominal Unit Shear Capacities for Wood-Frame Shear Walls - Wood-based Panels

Nominal unit shear capacity for seismic design v_s = min(560 plf × min[1 - (0.5 - G), 1], 1740 plf) = **520.8** lb/ft
Nominal unit shear capacity for wind design v_w = min(785 plf × min[1 - (0.5 - G), 1], 2435 plf) = **730.1** lb/ft
Apparent shear wall shear stiffness G_a = **16** kips/in

Loading details

Dead load acting on top of panel D = **460** lb/ft
Snow load acting on top of panel S = **800** lb/ft
Self weight of panel S_{wt} = **10** lb/ft^2
In plane wind load acting at head of panel W = **3800** lbs
Wind load serviceability factor f_{Wserv} = **1.00**

Chord forces from shear walls above

Chord	$W_{ch[i]}$ (lbs)	$E_{q\_ch[i]}$ (lbs)	$D_{C\_ch[i]}$ (lbs)	$D_{T\_ch[i]}$ (lbs)	$L_{f\_ch[i]}$ (lbs)	$L_{r\_ch[i]}$ (lbs)	$S_{ch[i]}$ (lbs)	$R_{ch[i]}$ (lbs)
Ch1	-697;	0;	0;	0;	0;	0;	0;	0;
Ch2	697;	0;	0;	0;	0;	0;	0;	0;

From IBC 2021 cl.1605.1 Basic load combinations from ASCE 7, section 2.4

Load combination no.1 D + 0.6W
Load combination no.2 D + 0.7E
Load combination no.3 D + 0.75L_f + 0.45W + 0.75(L_r or S or R)
Load combination no.4 D + 0.75L_f + 0.525E + 0.75S
Load combination no.5 0.6D + 0.6W
Load combination no.6 0.6D + 0.7E

Adjustment factors

Load duration factor – Table 2.3.2 C_D = **1.60**
Size factor for tension – Table 4A C_{Ft} = **1.30**
Size factor for compression – Table 4A C_{Fc} = **1.10**

WEST, LEVEL 3

Wet service factor for tension – Table 4A C_{Mt} = **1.00**

Wet service factor for compression – Table 4A C_{Mc} = **1.00**

Wet service factor for modulus of elasticity – Table 4A

C_{ME} = **1.00**

Temperature factor for tension – Table 2.3.3 C_{tt} = **1.00**

Temperature factor for compression – Table 2.3.3

C_{tc} = **1.00**

Temperature factor for modulus of elasticity – Table 2.3.3

C_{tE} = **1.00**

Incising factor – cl.4.3.8 C_i = **1.00**

Buckling stiffness factor – cl.4.4.2 C_T = **1.00**

Adjusted modulus of elasticity $E_{min}' = E_{min} \times C_{ME} \times C_{tE} \times C_i \times C_T$ = **550000** psi

Critical buckling design value $F_{cE} = 0.822 \times E_{min}' / (h / d)^2$ = **904** psi

Reference compression design value $F_c^* = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i$ = **2376** psi

For sawn lumber c = **0.8**

Column stability factor – eqn.3.7-1 $C_P = (1 + (F_{cE} / F_c^*)) / (2 \times c) – \sqrt{([(1 + (F_{cE} / F_c^*)) / (2 \times c)]^2 - (F_{cE} / F_c^*) / c)}$ = **0.34**

From SDPWS Table 4.3.4 Maximum Shear Wall Aspect Ratios

Maximum shear wall aspect ratio 3.5

Perforated wall length b_1 = **17.5** ft

Shear wall aspect ratio h / b_1 = **0.586**

Perforated wall length b_2 = **4** ft

Shear wall aspect ratio h / b_2 = **2.563**

Shear capacity adjustment factor – cl.4.3.3.5

Sum of perforated shear wall lengths $\Sigma L_i = b_1 + b_2 \times 2 \times b_s / h$ = **20.622** ft

Total length of perforated shear wall $L_{tot} = b_1 + w_{o1} + b_2$ = **27** ft

Total area of openings $A_o = w_{o1} \times h_{o1}$ = **27.5** ft^2

Sheathing area ratio (eqn. 4.3-6) $r = 1 / (1 + A_o /(h \times \Sigma L_i))$ = **0.885**

Shear capacity adjustment factor (eqn. 4.3-5) C_o = **0.942**

Perforated shear wall capacity

Maximum shear force under wind loading $V_{w\_max} = 0.6 \times W$ = **2.28** kips

Shear capacity for wind loading $V_w = v_w \times C_o \times \Sigma L_i / 2$ = **7.089** kips

V_{w_max} / V_w = **0.322**

PASS - Shear capacity for wind load exceeds maximum shear force

Chord capacity for chord 1

Load combination 5

Shear force for maximum tension $V = 0.6 \times W$ = **2.28** kips

Axial force for maximum tension $P = (0.6 \times (D + S_{wt} \times h)) \times b / 2 + 0.6 \times W_{ch1}$ = **4.138** kips

Maximum tensile force in chord $T = V \times h / ((C_o \times \Sigma L_i)) - P$ = $\boxed{\textbf{-2.935}\text{ kips}}$ NEGATIVE. NO OVERTURNING

Maximum applied tensile stress $f_t = T / A_{en}$ = **-217** lb/in^2

Design tensile stress $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i$ = **1508** lb/in^2

f_t / F_t' = **-0.144**

PASS - Design tensile stress exceeds maximum applied tensile stress

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WEST, LEVEL 3

Load combination 1

Shear force for maximum compression: $V = 0.6 \times W =$ **2.28** kips

Axial force for maximum compression: $P = ((D + S_{wt} \times h)) \times s / 2 + -1 \times 0.6 \times W_{ch1} =$ **0.793** kips

Maximum compressive force in chord: $C = V \times h / ((C_o \times \Sigma L_i)) + P =$ **1.997** kips

Maximum applied compressive stress: $f_c = C / A_e =$ **121** lb/in^2

Design compressive stress: $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P =$ **818** lb/in^2

$f_c / F_c' =$ **0.148**

PASS - Design compressive stress exceeds maximum applied compressive stress

Chord capacity for chord 2

Load combination 5

Shear force for maximum tension: $V = 0.6 \times W =$ **2.28** kips

Axial force for maximum tension: $P = (0.6 \times (D + S_{wt} \times h)) \times b / 2 + -1 \times 0.6 \times W_{ch2} =$ **4.138** kips

Maximum tensile force in chord: $T = V \times h / ((C_o \times \Sigma L_i)) - P =$ **-2.935** kips — NEGATIVE. NO OVERTURNING

Maximum applied tensile stress: $f_t = T / A_{en} =$ **-217** lb/in^2

Design tensile stress: $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i =$ **1508** lb/in^2

$f_t / F_t' =$ **-0.144**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 1

Shear force for maximum compression: $V = 0.6 \times W =$ **2.28** kips

Axial force for maximum compression: $P = ((D + S_{wt} \times h)) \times s / 2 + 0.6 \times W_{ch2} =$ **0.793** kips

Maximum compressive force in chord: $C = V \times h / ((C_o \times \Sigma L_i)) + P =$ **1.997** kips

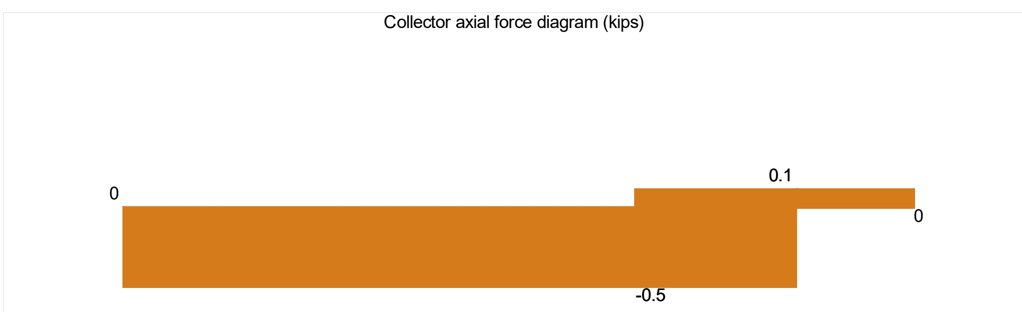
Maximum applied compressive stress: $f_c = C / A_e =$ **121** lb/in^2

Design compressive stress: $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P =$ **818** lb/in^2

$f_c / F_c' =$ **0.148**

PASS - Design compressive stress exceeds maximum applied compressive stress

Collector capacity



Maximum shear force on wall: $V_{max} = V_{w\_max} =$ **2.28** kips

Uniform shear applied to wall: $v_a = V_{max} / ((C_o \times \Sigma L_i)) =$ **117.4** plf

Shear resisted by wall segments: $v_b = v_a \times b / (b_1 + b_2) =$ **147.4** plf

Maximum force in collector: $P_{coll} =$ **0.526** kips

WEST, LEVEL 3

Maximum applied tensile stress $f_t = P_{coll} / (2 \times A_s) =$ **32** lb/in^2

Design tensile stress $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i =$ **1508** lb/in^2

$f_t / F_t' =$ **0.021**

PASS - Design tensile stress exceeds maximum applied tensile stress

Maximum applied compressive stress $f_c = P_{coll} / (2 \times A_s) =$ **32** lb/in^2

Column stability factor $C_P =$ **1.00**

Design compressive stress $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P =$ **2376** lb/in^2

$f_c / F_c' =$ **0.013**

PASS - Design compressive stress exceeds maximum applied compressive stress

Wind load deflection

Design shear force $V_{\delta w} = f_{Wserv} \times W =$ **3.8** kips

Deflection limit $\Delta_{w\_allow} = h / 500 =$ **0.246** in

Induced unit shear $v_{\delta w\_max} = V_{\delta w} / (C_o \times \Sigma L_i) =$ **195.67** lb/ft

Anchor tension force $T_\delta = \max(0 \text{ kips}, v_{\delta w\_max} \times h - 0.6 \times (D + S_{wt} \times h) \times b / 2 + \max(\text{abs}(W_{ch1}), \text{abs}(W_{ch2}))) =$ **0.000** kips

Shear wall deflection – Eqn. 4.3-1 $\delta_{sww} = 2 \times v_{\delta w\_max} \times h^3 / (3 \times E \times A_e \times \Sigma L_i) + v_{\delta w\_max} \times h / (G_a) + h \times T_\delta / (k_a \times \Sigma L_i) =$ **0.129** in

$\delta_{sww} / \Delta_{w\_allow} =$ **0.523**

PASS - Shear wall deflection is less than deflection limit

WEST, LEVEL 2

WOOD SHEAR WALL DESIGN (NDS)

In accordance with NDS2018 allowable stress design and the perforated shear wall method

Tedds calculation version 1.2.08

Design summary

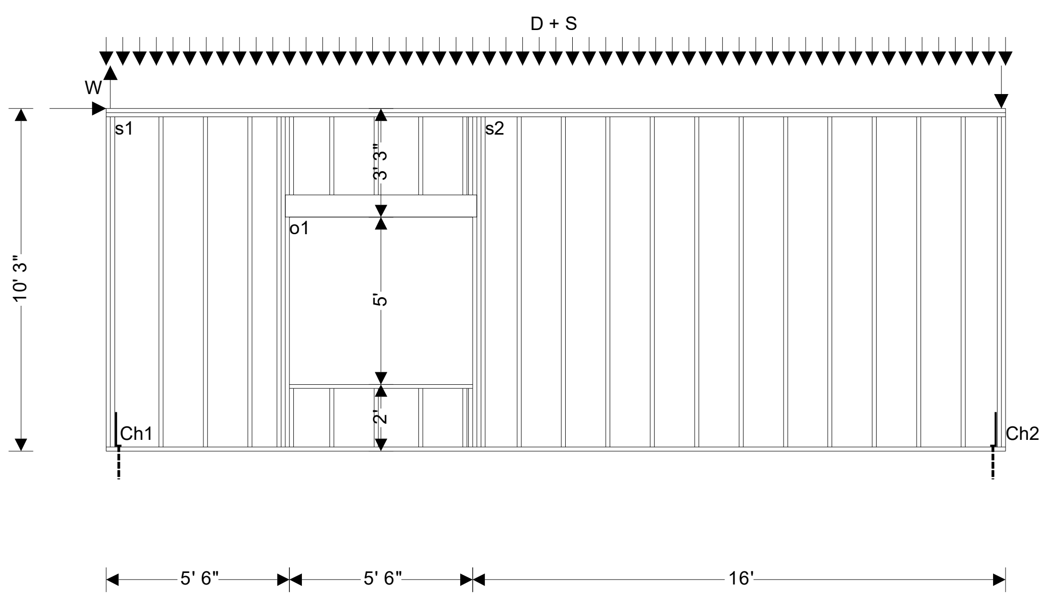
Description	Unit	Provided	Required	Utilization	Result
Shear capacity	lbs	7171	3600	0.502	PASS
Chord capacity	lb/in^2	818	204	0.250	PASS
Collector capacity	lb/in^2	1508	45	0.030	PASS
Deflection	in	0.246	0.201	0.815	PASS

Panel details

Structural I wood panel sheathing on one side

Panel height h = **10.25** ft

Panel length b = **27** ft



Panel opening details

Width of opening w_{o1} = **5.5** ft

Height of opening h_{o1} = **5** ft

Height to underside of lintel over opening l_{o1} = **7** ft

Position of opening P_{o1} = **5.5** ft

Total area of wall $A = h \times b - w_{o1} \times h_{o1}$ = **249.25** ft^2

Panel construction

Nominal stud size 2" x 6"

Dressed stud size 1.5" x 5.5"

Cross-sectional area of studs A_s = **8.25** in^2

Stud spacing s = **16** in

Nominal end post size 2 x 2" x 6"

Dressed end post size 2 x 1.5" x 5.5"

WEST, LEVEL 2

Cross-sectional area of end posts A_e = **16.5** in^2
Hole diameter Dia = **1** in
Net cross-sectional area of end posts A_{en} = **13.5** in^2
Nominal collector size 2 x 2" x 6"
Dressed collector size 2 x 1.5" x 5.5"
Service condition Dry
Temperature 100 degF or less
Vertical anchor stiffness k_a = **30000** lb/in

From NDS Supplement Table 4A - Reference design values for visually graded dimension lumber (2" - 4" thick)

Species, grade and size classification Hem-Fir, no.1 & btr grade, 2" & wider
Specific gravity G = **0.43**
Tension parallel to grain F_t = **725** lb/in^2
Compression parallel to grain F_c = **1350** lb/in^2
Modulus of elasticity E = **1500000** lb/in^2
Minimum modulus of elasticity E_{min} = **550000** lb/in^2

Sheathing details

Sheathing material 7/16" wood panel structural I oriented strandboard sheathing
Fastener type 8d common nails at 6"centers

From SDPWS Table 4.3A Nominal Unit Shear Capacities for Wood-Frame Shear Walls - Wood-based Panels

Nominal unit shear capacity for seismic design v_s = min(560 plf × min[1 - (0.5 - G), 1], 1740 plf) = **520.8** lb/ft
Nominal unit shear capacity for wind design v_w = min(785 plf × min[1 - (0.5 - G), 1], 2435 plf) = **730.1** lb/ft
Apparent shear wall shear stiffness G_a = **16** kips/in

Loading details

Dead load acting on top of panel D = **720** lb/ft
Snow load acting on top of panel S = **800** lb/ft
Self weight of panel S_{wt} = **10** lb/ft^2
In plane wind load acting at head of panel W = **6000** lbs
Wind load serviceability factor f_{Wserv} = **1.00**

Chord forces from shear walls above

Chord	$W_{ch[i]}$ (lbs)	$E_{q\_ch[i]}$ (lbs)	$D_{C\_ch[i]}$ (lbs)	$D_{T\_ch[i]}$ (lbs)	$L_{f\_ch[i]}$ (lbs)	$L_{r\_ch[i]}$ (lbs)	$S_{ch[i]}$ (lbs)	$R_{ch[i]}$ (lbs)
Ch1	-1570;	0;	0;	0;	0;	0;	0;	0;
Ch2	1570;	0;	0;	0;	0;	0;	0;	0;

From IBC 2021 cl.1605.1 Basic load combinations from ASCE 7, section 2.4

Load combination no.1 D + 0.6W
Load combination no.2 D + 0.7E
Load combination no.3 D + 0.75L_f + 0.45W + 0.75(L_r or S or R)
Load combination no.4 D + 0.75L_f + 0.525E + 0.75S
Load combination no.5 0.6D + 0.6W
Load combination no.6 0.6D + 0.7E

Adjustment factors

Load duration factor – Table 2.3.2 C_D = **1.60**
Size factor for tension – Table 4A C_{Ft} = **1.30**
Size factor for compression – Table 4A C_{Fc} = **1.10**

WEST, LEVEL 2

Wet service factor for tension – Table 4A C_{Mt} = **1.00**

Wet service factor for compression – Table 4A C_{Mc} = **1.00**

Wet service factor for modulus of elasticity – Table 4A C_{ME} = **1.00**

Temperature factor for tension – Table 2.3.3 C_{tt} = **1.00**

Temperature factor for compression – Table 2.3.3 C_{tc} = **1.00**

Temperature factor for modulus of elasticity – Table 2.3.3 C_{tE} = **1.00**

Incising factor – cl.4.3.8 C_i = **1.00**

Buckling stiffness factor – cl.4.4.2 C_T = **1.00**

Adjusted modulus of elasticity $E_{min}' = E_{min} \times C_{ME} \times C_{tE} \times C_i \times C_T$ = **550000** psi

Critical buckling design value $F_{cE} = 0.822 \times E_{min}' / (h / d)^2$ = **904** psi

Reference compression design value $F_c^* = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i$ = **2376** psi

For sawn lumber c = **0.8**

Column stability factor – eqn.3.7-1 $C_P = (1 + (F_{cE} / F_c^*)) / (2 \times c) - \sqrt{([(1 + (F_{cE} / F_c^*)) / (2 \times c)]^2 - (F_{cE} / F_c^*) / c)}$ = **0.34**

From SDPWS Table 4.3.4 Maximum Shear Wall Aspect Ratios

Maximum shear wall aspect ratio 3.5

Perforated wall length b_1 = **5.5** ft

Shear wall aspect ratio h / b_1 = **1.864**

Perforated wall length b_2 = **16** ft

Shear wall aspect ratio h / b_2 = **0.641**

Shear capacity adjustment factor – cl.4.3.3.5

Sum of perforated shear wall lengths $\Sigma L_i = b_1 + b_2$ = **21.5** ft

Total length of perforated shear wall $L_{tot} = b_1 + w_{o1} + b_2$ = **27** ft

Total area of openings $A_o = w_{o1} \times h_{o1}$ = **27.5** ft^2

Sheathing area ratio (eqn. 4.3-6) $r = 1 / (1 + A_o /(h \times \Sigma L_i))$ = **0.889**

Shear capacity adjustment factor (eqn. 4.3-5) C_o = **0.914**

Perforated shear wall capacity

Maximum shear force under wind loading $V_{w\_max} = 0.6 \times W$ = **3.6** kips

Shear capacity for wind loading $V_w = v_w \times C_o \times \Sigma L_i / 2$ = **7.171** kips

V_{w_max} / V_w = **0.502**

PASS - Shear capacity for wind load exceeds maximum shear force

Chord capacity for chord 1

Load combination 5

Shear force for maximum tension $V = 0.6 \times W$ = **3.6** kips

Axial force for maximum tension $P = (0.6 \times (D + S_{wt} \times h)) \times b / 2 + 0.6 \times W_{ch1}$ = **5.72** kips

Maximum tensile force in chord $T = V \times h / ((C_o \times \Sigma L_i)) - P$ = $\boxed{\textbf{-3.842} \text{ kips}}$ NEGATIVE. NO OVERTURNING

Maximum applied tensile stress $f_t = T / A_{en}$ = **-285** lb/in^2

Design tensile stress $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i$ = **1508** lb/in^2

f_t / F_t' = **-0.189**

PASS - Design tensile stress exceeds maximum applied tensile stress

WEST, LEVEL 2

Load combination 1

Shear force for maximum compression $V = 0.6 \times W =$ **3.6** kips

Axial force for maximum compression $P = ((D + S_{wt} \times h)) \times s / 2 + -1 \times 0.6 \times W_{ch1} =$ **1.49** kips

Maximum compressive force in chord $C = V \times h / ((C_o \times \Sigma L_i)) + P =$ **3.369** kips

Maximum applied compressive stress $f_c = C / A_e =$ **204** lb/in^2

Design compressive stress $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P =$ **818** lb/in^2

$f_c / F_c' =$ **0.250**

PASS - Design compressive stress exceeds maximum applied compressive stress

Chord capacity for chord 2

Load combination 5

Shear force for maximum tension $V = 0.6 \times W =$ **3.6** kips

Axial force for maximum tension $P = (0.6 \times (D + S_{wt} \times h)) \times b / 2 + -1 \times 0.6 \times W_{ch2} =$ **5.72** kips

Maximum tensile force in chord $T = V \times h / ((C_o \times \Sigma L_i)) - P =$ **-3.842** kips ← NEGATIVE. NO OVERTURNING

Maximum applied tensile stress $f_t = T / A_{en} =$ **-285** lb/in^2

Design tensile stress $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i =$ **1508** lb/in^2

$f_t / F_t' =$ **-0.189**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 1

Shear force for maximum compression $V = 0.6 \times W =$ **3.6** kips

Axial force for maximum compression $P = ((D + S_{wt} \times h)) \times s / 2 + 0.6 \times W_{ch2} =$ **1.49** kips

Maximum compressive force in chord $C = V \times h / ((C_o \times \Sigma L_i)) + P =$ **3.369** kips

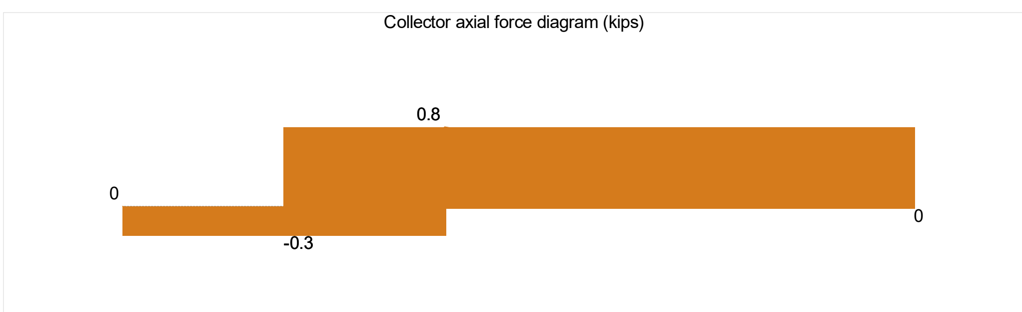
Maximum applied compressive stress $f_c = C / A_e =$ **204** lb/in^2

Design compressive stress $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P =$ **818** lb/in^2

$f_c / F_c' =$ **0.250**

PASS - Design compressive stress exceeds maximum applied compressive stress

Collector capacity



Maximum shear force on wall $V_{max} = V_{w\_max} =$ **3.6** kips

Uniform shear applied to wall $v_a = V_{max} / ((C_o \times \Sigma L_i)) =$ **183.2** plf

Shear resisted by wall segments $v_b = v_a \times b / (b_1 + b_2) =$ **230.1** plf

Maximum force in collector $P_{coll} =$ **0.75** kips

WEST, LEVEL 2

Maximum applied tensile stress $f_t = P_{coll} / (2 \times A_s) =$ **45** lb/in^2

Design tensile stress $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i =$ **1508** lb/in^2

$f_t / F_t' =$ **0.030**

PASS - Design tensile stress exceeds maximum applied tensile stress

Maximum applied compressive stress $f_c = P_{coll} / (2 \times A_s) =$ **45** lb/in^2

Column stability factor $C_P =$ **1.00**

Design compressive stress $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P =$ **2376** lb/in^2

$f_c / F_c' =$ **0.019**

PASS - Design compressive stress exceeds maximum applied compressive stress

Wind load deflection

Design shear force $V_{\delta w} = f_{Wserv} \times W =$ **6** kips

Deflection limit $\Delta_{w\_allow} = h / 500 =$ **0.246** in

Induced unit shear $v_{\delta w\_max} = V_{\delta w} / (C_o \times \Sigma L_i) =$ **305.41** lb/ft

Anchor tension force $T_\delta = \max(0 \text{ kips}, v_{\delta w\_max} \times h - 0.6 \times (D + S_{wt} \times h) \times b / 2 + \max(\text{abs}(W_{ch1}), \text{abs}(W_{ch2}))) =$ **0.000** kips

Shear wall deflection – Eqn. 4.3-1 $\delta_{sww} = 2 \times v_{\delta w\_max} \times h^3 / (3 \times E \times A_e \times \Sigma L_i) + v_{\delta w\_max} \times h / (G_a) + h \times T_\delta / (k_a \times \Sigma L_i) =$ **0.201** in

$\delta_{sww} / \Delta_{w\_allow} =$ **0.815**

PASS - Shear wall deflection is less than deflection limit

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WEST, LEVEL 1

WOOD SHEAR WALL DESIGN (NDS)

In accordance with NDS2018 allowable stress design and the perforated shear wall method

Tedds calculation version 1.2.08

Design summary

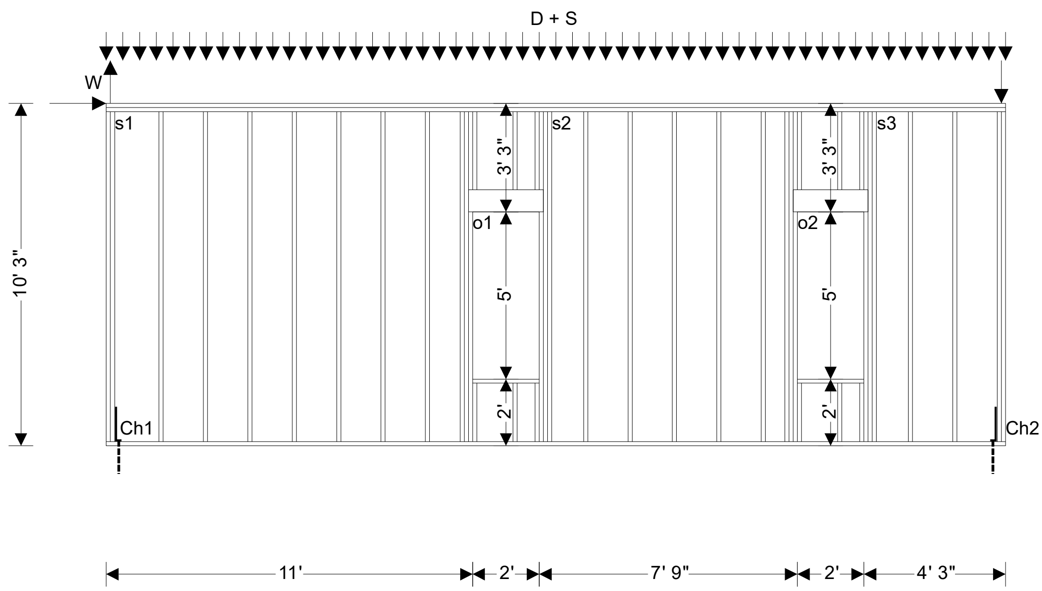
Description	Unit	Provided	Required	Utilization	Result
Shear capacity	lbs	11980	4920	0.411	PASS
Chord capacity	lb/in^2	818	347	0.425	PASS
Collector capacity	lb/in^2	1508	27	0.018	PASS
Deflection	in	0.246	0.193	0.785	PASS

Panel details

Structural I wood panel sheathing on one side

Panel height h = **10.25** ft

Panel length b = **27** ft



Panel opening details

Width of opening w_{o1} = **2** ft

Height of opening h_{o1} = **5** ft

Height to underside of lintel over opening l_{o1} = **7** ft

Position of opening P_{o1} = **11** ft

Width of opening w_{o2} = **2** ft

Height of opening h_{o2} = **5** ft

Height to underside of lintel over opening l_{o2} = **7** ft

Position of opening P_{o2} = **20.75** ft

Total area of wall $A = h \times b - w_{o1} \times h_{o1} - w_{o2} \times h_{o2}$ = **256.75** ft^2

Panel construction

Nominal stud size 2" x 6"

Dressed stud size 1.5" x 5.5"

WEST, LEVEL 1

Cross-sectional area of studs A_s = **8.25** in^2
Stud spacing s = **16** in
Nominal end post size 2 x 2" x 6"
Dressed end post size 2 x 1.5" x 5.5"
Cross-sectional area of end posts A_e = **16.5** in^2
Hole diameter Dia = **1** in
Net cross-sectional area of end posts A_{en} = **13.5** in^2
Nominal collector size 2 x 2" x 6"
Dressed collector size 2 x 1.5" x 5.5"
Service condition Dry
Temperature 100 degF or less
Vertical anchor stiffness k_a = **30000** lb/in

From NDS Supplement Table 4A - Reference design values for visually graded dimension lumber (2" - 4" thick)

Species, grade and size classification Hem-Fir, no.1 & btr grade, 2" & wider
Specific gravity G = **0.43**
Tension parallel to grain F_t = **725** lb/in^2
Compression parallel to grain F_c = **1350** lb/in^2
Modulus of elasticity E = **1500000** lb/in^2
Minimum modulus of elasticity E_{min} = **550000** lb/in^2

Sheathing details

Sheathing material 7/16" wood panel structural I oriented strandboard sheathing
Fastener type 8d common nails at 4"centers

From SDPWS Table 4.3A Nominal Unit Shear Capacities for Wood-Frame Shear Walls - Wood-based Panels

Nominal unit shear capacity for seismic design v_s = min(860 plf × min[1 - (0.5 - G), 1], 1740 plf) = **799.8** lb/ft
Nominal unit shear capacity for wind design v_w = min(1205 plf × min[1 - (0.5 - G), 1], 2435 plf) = **1120.6** lb/ft
Apparent shear wall shear stiffness G_a = **21** kips/in

Loading details

Dead load acting on top of panel D = **980** lb/ft
Snow load acting on top of panel S = **800** lb/ft
Self weight of panel S_{wt} = **10** lb/ft^2
In plane wind load acting at head of panel W = **8200** lbs
Wind load serviceability factor f_{Wserv} = **1.00**

Chord forces from shear walls above

Chord	$W_{ch[i]}$ (lbs)	$E_{q\_ch[i]}$ (lbs)	$D_{C\_ch[i]}$ (lbs)	$D_{T\_ch[i]}$ (lbs)	$L_{f\_ch[i]}$ (lbs)	$L_{r\_ch[i]}$ (lbs)	$S_{ch[i]}$ (lbs)	$R_{ch[i]}$ (lbs)
Ch1	-4418;	0;	0;	0;	0;	0;	0;	0;
Ch2	4418;	0;	0;	0;	0;	0;	0;	0;

From IBC 2021 cl.1605.1 Basic load combinations from ASCE 7, section 2.4

Load combination no.1 D + 0.6W
Load combination no.2 D + 0.7E
Load combination no.3 D + 0.75L_f + 0.45W + 0.75(L_r or S or R)
Load combination no.4 D + 0.75L_f + 0.525E + 0.75S
Load combination no.5 0.6D + 0.6W
Load combination no.6 0.6D + 0.7E

WEST, LEVEL 1

Adjustment factors

Load duration factor – Table 2.3.2 C_D = **1.60**

Size factor for tension – Table 4A C_{Ft} = **1.30**

Size factor for compression – Table 4A C_{Fc} = **1.10**

Wet service factor for tension – Table 4A C_{Mt} = **1.00**

Wet service factor for compression – Table 4A C_{Mc} = **1.00**

Wet service factor for modulus of elasticity – Table 4A

C_{ME} = **1.00**

Temperature factor for tension – Table 2.3.3 C_{tt} = **1.00**

Temperature factor for compression – Table 2.3.3

C_{tc} = **1.00**

Temperature factor for modulus of elasticity – Table 2.3.3

C_{tE} = **1.00**

Incising factor – cl.4.3.8 C_i = **1.00**

Buckling stiffness factor – cl.4.4.2 C_T = **1.00**

Adjusted modulus of elasticity $E_{min}' = E_{min} \times C_{ME} \times C_{tE} \times C_i \times C_T$ = **550000** psi

Critical buckling design value $F_{cE} = 0.822 \times E_{min}' / (h / d)^2$ = **904** psi

Reference compression design value $F_c^* = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i$ = **2376** psi

For sawn lumber c = **0.8**

Column stability factor – eqn.3.7-1 $C_P = (1 + (F_{cE} / F_c^*)) / (2 \times c) - \sqrt{([(1 + (F_{cE} / F_c^*)) / (2 \times c)]^2 - (F_{cE} / F_c^*) / c)}$ = **0.34**

From SDPWS Table 4.3.4 Maximum Shear Wall Aspect Ratios

Maximum shear wall aspect ratio 3.5

Perforated wall length b_1 = **11** ft

Shear wall aspect ratio h / b_1 = **0.932**

Perforated wall length b_2 = **7.75** ft

Shear wall aspect ratio h / b_2 = **1.323**

Perforated wall length b_3 = **4.25** ft

Shear wall aspect ratio h / b_3 = **2.412**

Shear capacity adjustment factor – cl.4.3.3.5

Sum of perforated shear wall lengths $\Sigma L_i = b_1 + b_2 + b_3 \times 2 \times b_s / h$ = **22.274** ft

Total length of perforated shear wall $L_{tot} = b_1 + w_{o1} + b_2 + w_{o2} + b_3$ = **27** ft

Total area of openings $A_o = w_{o1} \times h_{o1} + w_{o2} \times h_{o2}$ = **20** ft^2

Sheathing area ratio (eqn. 4.3-6) $r = 1 / (1 + A_o /(h \times \Sigma L_i))$ = **0.919**

Shear capacity adjustment factor (eqn. 4.3-5) C_o = **0.96**

Perforated shear wall capacity

Maximum shear force under wind loading $V_{w\_max} = 0.6 \times W$ = **4.92** kips

Shear capacity for wind loading $V_w = v_w \times C_o \times \Sigma L_i / 2$ = **11.98** kips

V_{w_max} / V_w = **0.411**

PASS - Shear capacity for wind load exceeds maximum shear force

Chord capacity for chord 1

Load combination 5

Shear force for maximum tension $V = 0.6 \times W$ = **4.92** kips

WEST, LEVEL 1

Axial force for maximum tension $P = (0.6 \times (D + S_{wt} \times h)) \times b / 2 + 0.6 \times W_{ch1}$ = **6.117** kips

Maximum tensile force in chord $T = V \times h / ((C_o \times \Sigma L_i)) - P$ = $\boxed{\textbf{-3.759}\text{ kips}}$ NEGATIVE. NO OVERTURNING

Maximum applied tensile stress $f_t = T / A_{en}$ = **-278** lb/in^2

Design tensile stress $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i$ = **1508** lb/in^2

f_t / F_t' = **-0.185**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 1

Shear force for maximum compression $V = 0.6 \times W$ = **4.92** kips

Axial force for maximum compression $P = ((D + S_{wt} \times h)) \times s / 2 + -1 \times 0.6 \times W_{ch1}$ = **3.372** kips

Maximum compressive force in chord $C = V \times h / ((C_o \times \Sigma L_i)) + P$ = $\boxed{\textbf{5.731}\text{ kips}}$

Maximum applied compressive stress $f_c = C / A_e$ = **347** lb/in^2

Design compressive stress $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P$ = **818** lb/in^2

f_c / F_c' = **0.425**

PASS - Design compressive stress exceeds maximum applied compressive stress

Chord capacity for chord 2

Load combination 5

Shear force for maximum tension $V = 0.6 \times W$ = **4.92** kips

Axial force for maximum tension $P = (0.6 \times (D + S_{wt} \times h)) \times b / 2 + -1 \times 0.6 \times W_{ch2}$ = **6.117** kips

Maximum tensile force in chord $T = V \times h / ((C_o \times \Sigma L_i)) - P$ = $\boxed{\textbf{-3.759}\text{ kips}}$ NEGATIVE. NO OVERTURNING

Maximum applied tensile stress $f_t = T / A_{en}$ = **-278** lb/in^2

Design tensile stress $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i$ = **1508** lb/in^2

f_t / F_t' = **-0.185**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 1

Shear force for maximum compression $V = 0.6 \times W$ = **4.92** kips

Axial force for maximum compression $P = ((D + S_{wt} \times h)) \times s / 2 + 0.6 \times W_{ch2}$ = **3.372** kips

Maximum compressive force in chord $C = V \times h / ((C_o \times \Sigma L_i)) + P$ = $\boxed{\textbf{5.731}\text{ kips}}$

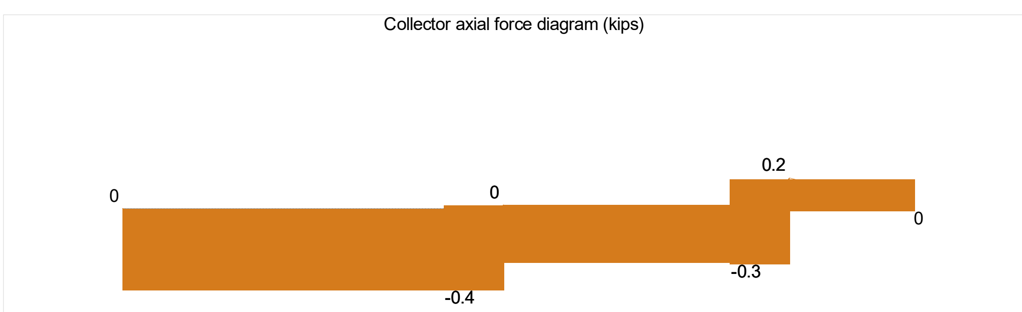
Maximum applied compressive stress $f_c = C / A_e$ = **347** lb/in^2

Design compressive stress $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P$ = **818** lb/in^2

f_c / F_c' = **0.425**

PASS - Design compressive stress exceeds maximum applied compressive stress

Collector capacity



WEST, LEVEL 1

Maximum shear force on wall $V_{max} = V_{w\_max} =$ **4.92** kips

Uniform shear applied to wall $v_a = V_{max} / ((C_o \times \Sigma L_i)) =$ **230.1** plf

Shear resisted by wall segments $v_b = v_a \times b / (b_1 + b_2 + b_3) =$ **270.1** plf

Maximum force in collector $P_{coll} =$ **0.44** kips

Maximum applied tensile stress $f_t = P_{coll} / (2 \times A_s) =$ **27** lb/in^2

Design tensile stress $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i =$ **1508** lb/in^2

$f_t / F_t' =$ **0.018**

PASS - Design tensile stress exceeds maximum applied tensile stress

Maximum applied compressive stress $f_c = P_{coll} / (2 \times A_s) =$ **27** lb/in^2

Column stability factor $C_P =$ **1.00**

Design compressive stress $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P =$ **2376** lb/in^2

$f_c / F_c' =$ **0.011**

PASS - Design compressive stress exceeds maximum applied compressive stress

Wind load deflection

Design shear force $V_{\delta w} = f_{Wserv} \times W =$ **8.2** kips

Deflection limit $\Delta_{w\_allow} = h / 500 =$ **0.246** in

Induced unit shear $v_{\delta w\_max} = V_{\delta w} / (C_o \times \Sigma L_i) =$ **383.52** lb/ft

Anchor tension force $T_\delta = \max(0 \text{ kips}, v_{\delta w\_max} \times h - 0.6 \times (D + S_{wt} \times h) \times b / 2 + \max(\text{abs}(W_{ch1}), \text{abs}(W_{ch2}))) =$ **0.000** kips

Shear wall deflection – Eqn. 4.3-1 $\delta_{sww} = 2 \times v_{\delta w\_max} \times h^3 / (3 \times E \times A_e \times \Sigma L_i) + v_{\delta w\_max} \times h / (G_a) + h \times T_\delta / (k_a \times \Sigma L_i) =$ **0.193** in

$\delta_{sww} / \Delta_{w\_allow} =$ **0.785**

PASS - Shear wall deflection is less than deflection limit

EAST, LEVEL 4

WOOD SHEAR WALL DESIGN (NDS)

In accordance with NDS2018 allowable stress design and the perforated shear wall method

Tedds calculation version 1.2.08

Design summary

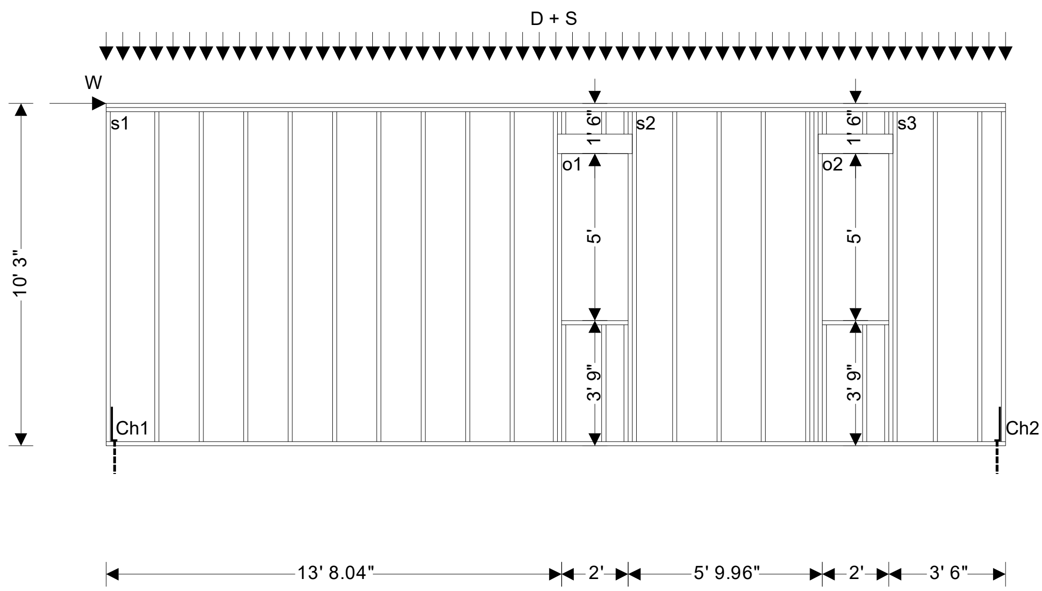
Description	Unit	Provided	Required	Utilization	Result
Shear capacity	lbs	7776	1080	0.139	PASS
Chord capacity	lb/in^2	818	84	0.103	PASS
Collector capacity	lb/in^2	1508	7	0.005	PASS
Deflection	in	0.246	0.057	0.231	PASS

Panel details

Structural I wood panel sheathing on one side

Panel height h = **10.25** ft

Panel length b = **27** ft



Panel opening details

Width of opening w_{o1} = **2** ft

Height of opening h_{o1} = **5** ft

Height to underside of lintel over opening l_{o1} = **8.75** ft

Position of opening P_{o1} = **13.67** ft

Width of opening w_{o2} = **2** ft

Height of opening h_{o2} = **5** ft

Height to underside of lintel over opening l_{o2} = **8.75** ft

Position of opening P_{o2} = **21.5** ft

Total area of wall $A = h \times b - w_{o1} \times h_{o1} - w_{o2} \times h_{o2}$ = **256.75** ft^2

Panel construction

Nominal stud size 2" x 6"

Dressed stud size 1.5" x 5.5"

EAST, LEVEL 4

Cross-sectional area of studs	A_s = **8.25** in^2
Stud spacing	s = **16** in
Nominal end post size	2" x 6"
Dressed end post size	1.5" x 5.5"
Cross-sectional area of end posts	A_e = **8.25** in^2
Hole diameter	Dia = **1** in
Net cross-sectional area of end posts	A_{en} = **6.75** in^2
Nominal collector size	2 x 2" x 6"
Dressed collector size	2 x 1.5" x 5.5"
Service condition	Dry
Temperature	100 degF or less
Vertical anchor stiffness	k_a = **30000** lb/in

From NDS Supplement Table 4A - Reference design values for visually graded dimension lumber (2" - 4" thick)

Species, grade and size classification	Hem-Fir, no.1 & btr grade, 2" & wider
Specific gravity	G = **0.43**
Tension parallel to grain	F_t = **725** lb/in^2
Compression parallel to grain	F_c = **1350** lb/in^2
Modulus of elasticity	E = **1500000** lb/in^2
Minimum modulus of elasticity	E_{min} = **550000** lb/in^2

Sheathing details

Sheathing material	7/16" wood panel structural I oriented strandboard sheathing
Fastener type	8d common nails at 6"centers

From SDPWS Table 4.3A Nominal Unit Shear Capacities for Wood-Frame Shear Walls - Wood-based Panels

Nominal unit shear capacity for seismic design	v_s = min(560 plf $\times$ min[1 - (0.5 - G), 1], 1740 plf) = **520.8** lb/ft
Nominal unit shear capacity for wind design	v_w = min(785 plf $\times$ min[1 - (0.5 - G), 1], 2435 plf) = **730.1** lb/ft
Apparent shear wall shear stiffness	G_a = **16** kips/in

Loading details

Dead load acting on top of panel	D = **160** lb/ft
Snow load acting on top of panel	S = **800** lb/ft
Self weight of panel	S_{wt} = **10** lb/ft^2
In plane wind load acting at head of panel	W = **1800** lbs
Wind load serviceability factor	f_{Wserv} = **1.00**

From IBC 2021 cl.1605.1 Basic load combinations from ASCE 7, section 2.4

Load combination no.1	D + 0.6W
Load combination no.2	D + 0.7E
Load combination no.3	D + 0.75L_f + 0.45W + 0.75(L_r or S or R)
Load combination no.4	D + 0.75L_f + 0.525E + 0.75S
Load combination no.5	0.6D + 0.6W
Load combination no.6	0.6D + 0.7E

Adjustment factors

Load duration factor – Table 2.3.2	C_D = **1.60**
Size factor for tension – Table 4A	C_{Ft} = **1.30**
Size factor for compression – Table 4A	C_{Fc} = **1.10**

EAST, LEVEL 4

Wet service factor for tension – Table 4A C_{Mt} = **1.00**

Wet service factor for compression – Table 4A C_{Mc} = **1.00**

Wet service factor for modulus of elasticity – Table 4A C_{ME} = **1.00**

Temperature factor for tension – Table 2.3.3 C_{tt} = **1.00**

Temperature factor for compression – Table 2.3.3 C_{tc} = **1.00**

Temperature factor for modulus of elasticity – Table 2.3.3 C_{tE} = **1.00**

Incising factor – cl.4.3.8 C_i = **1.00**

Buckling stiffness factor – cl.4.4.2 C_T = **1.00**

Adjusted modulus of elasticity $E_{min}' = E_{min} \times C_{ME} \times C_{tE} \times C_i \times C_T$ = **550000** psi

Critical buckling design value $F_{cE} = 0.822 \times E_{min}' / (h / d)^2$ = **904** psi

Reference compression design value $F_c^* = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i$ = **2376** psi

For sawn lumber c = **0.8**

Column stability factor – eqn.3.7-1 $C_P = (1 + (F_{cE} / F_c^*)) / (2 \times c) - \sqrt{[(1 + (F_{cE} / F_c^*)) / (2 \times c)]^2 - (F_{cE} / F_c^*) / c}$ = **0.34**

From SDPWS Table 4.3.4 Maximum Shear Wall Aspect Ratios

Maximum shear wall aspect ratio 3.5

Perforated wall length b_1 = **13.67** ft

Shear wall aspect ratio h / b_1 = **0.75**

Perforated wall length b_2 = **5.83** ft

Shear wall aspect ratio h / b_2 = **1.758**

Perforated wall length b_3 = **3.5** ft

Shear wall aspect ratio h / b_3 = **2.929**

Shear capacity adjustment factor – cl.4.3.3.5

Sum of perforated shear wall lengths $\Sigma L_i = b_1 + b_2 + b_3 \times 2 \times b_s / h$ = **21.89** ft

Total length of perforated shear wall $L_{tot} = b_1 + w_{o1} + b_2 + w_{o2} + b_3$ = **27** ft

Total area of openings $A_o = w_{o1} \times h_{o1} + w_{o2} \times h_{o2}$ = **20** ft^2

Sheathing area ratio (eqn. 4.3-6) $r = 1 / (1 + A_o /(h \times \Sigma L_i))$ = **0.918**

Shear capacity adjustment factor (eqn. 4.3-5) C_o = **0.973**

Perforated shear wall capacity

Maximum shear force under wind loading $V_{w\_max} = 0.6 \times W$ = **1.08** kips

Shear capacity for wind loading $V_w = v_w \times C_o \times \Sigma L_i / 2$ = **7.776** kips

V_{w_max} / V_w = **0.139**

PASS - Shear capacity for wind load exceeds maximum shear force

Chord capacity for chords 1 and 2

Load combination 5

Shear force for maximum tension $V = 0.6 \times W$ = **1.08** kips

Axial force for maximum tension $P = (0.6 \times (D + S_{wt} \times h)) \times b / 2$ = **2.126** kips

Maximum tensile force in chord $T = V \times h / ((C_o \times \Sigma L_i)) - P$ = $\boxed{\textbf{-1.607}\text{ kips}}$ ← NEGATIVE. NO OVERTURNING

Maximum applied tensile stress $f_t = T / A_{en}$ = **-238** lb/in^2

Design tensile stress $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i$ = **1508** lb/in^2

f_t / F_t' = **-0.158** **EAST, LEVEL 4**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 1

Shear force for maximum compression $V = 0.6 \times W =$ **1.08** kips

Axial force for maximum compression $P = ((D + S_{wt} \times h)) \times s / 2 =$ **0.175** kips

Maximum compressive force in chord $C = V \times h / ((C_o \times \Sigma L_i)) + P =$ $\boxed{\textbf{0.695}\text{ kips}}$

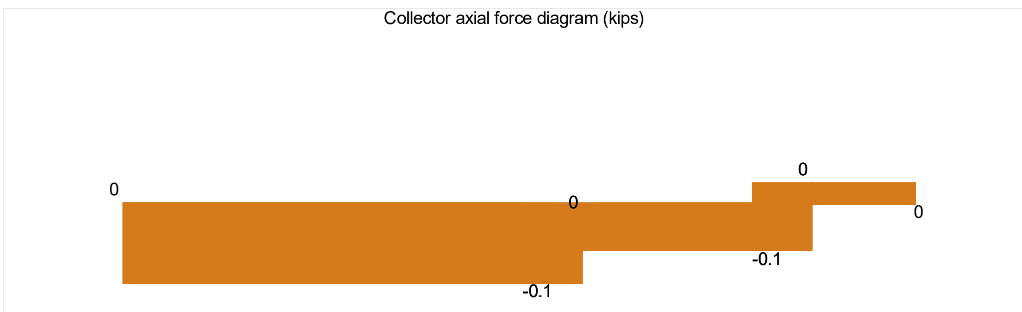
Maximum applied compressive stress $f_c = C / A_e =$ **84** lb/in^2

Design compressive stress $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P =$ **818** lb/in^2

$f_c / F_c' =$ **0.103**

PASS - Design compressive stress exceeds maximum applied compressive stress

Collector capacity



Maximum shear force on wall $V_{max} = V_{w\_max} =$ **1.08** kips

Uniform shear applied to wall $v_a = V_{max} / ((C_o \times \Sigma L_i)) =$ **50.7** plf

Shear resisted by wall segments $v_b = v_a \times b / (b_1 + b_2 + b_3) =$ **59.5** plf

Maximum force in collector $P_{coll} =$ **0.121** kips

Maximum applied tensile stress $f_t = P_{coll} / (2 \times A_s) =$ **7** lb/in^2

Design tensile stress $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i =$ **1508** lb/in^2

$f_t / F_t' =$ **0.005**

PASS - Design tensile stress exceeds maximum applied tensile stress

Maximum applied compressive stress $f_c = P_{coll} / (2 \times A_s) =$ **7** lb/in^2

Column stability factor $C_P =$ **1.00**

Design compressive stress $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P =$ **2376** lb/in^2

$f_c / F_c' =$ **0.003**

PASS - Design compressive stress exceeds maximum applied compressive stress

Wind load deflection

Design shear force $V_{\delta w} = f_{Wserv} \times W =$ **1.8** kips

Deflection limit $\Delta_{w\_allow} = h / 500 =$ **0.246** in

Induced unit shear $v_{\delta w\_max} = V_{\delta w} / (C_o \times \Sigma L_i) =$ **84.49** lb/ft

Anchor tension force $T_\delta = \max(0 \text{ kips}, v_{\delta w\_max} \times h - 0.6 \times (D + S_{wt} \times h) \times b / 2) =$ **0.000** kips

EAST, LEVEL 4

Shear wall deflection – Eqn. 4.3-1

$\delta_{sww} = 2 \times v_{\delta w\_max} \times h^3 / (3 \times E \times A_e \times \Sigma L_i) + v_{\delta w\_max} \times h / (G_a) + h \times \Gamma_\delta / (K_a \times \Sigma L_i)$ = **0.057** in

$\delta_{sww} / \Delta_{w\_allow}$ = **0.231**

PASS - Shear wall deflection is less than deflection limit

EAST, LEVEL 3

WOOD SHEAR WALL DESIGN (NDS)

In accordance with NDS2018 allowable stress design and the perforated shear wall method

Tedds calculation version 1.2.08

Design summary

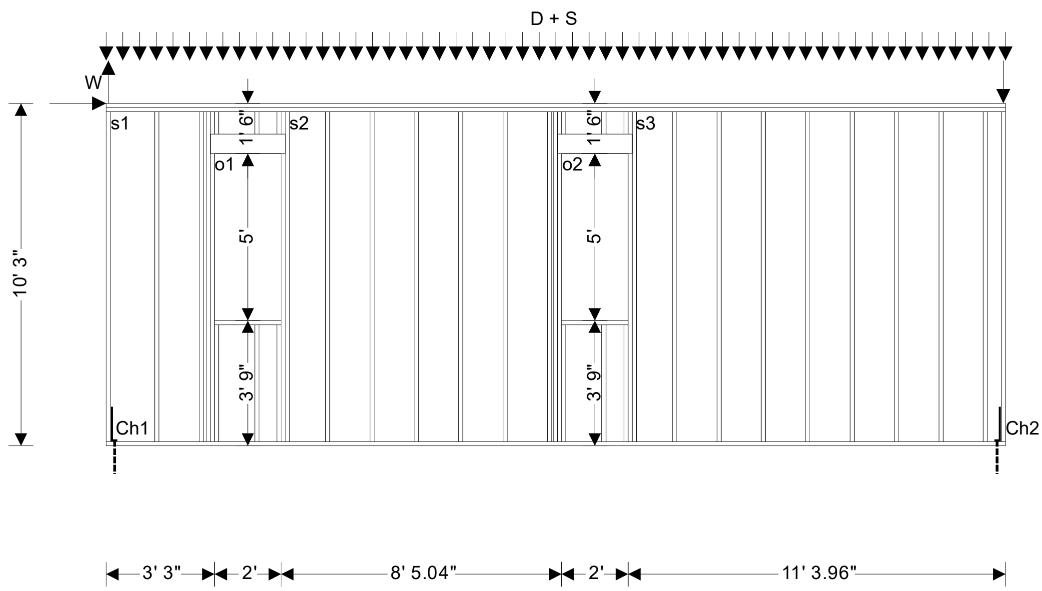
Description	Unit	Provided	Required	Utilization	Result
Shear capacity	lbs	7770	2280	0.293	PASS
Chord capacity	lb/in^2	709	229	0.323	PASS
Collector capacity	lb/in^2	1092	13	0.012	PASS
Deflection	in	0.246	0.121	0.492	PASS

Panel details

Structural I wood panel sheathing on one side

Panel height h = **10.25** ft

Panel length b = **27** ft



Panel opening details

Width of opening w_{o1} = **2** ft

Height of opening h_{o1} = **5** ft

Height to underside of lintel over opening l_{o1} = **8.75** ft

Position of opening P_{o1} = **3.25** ft

Width of opening w_{o2} = **2** ft

Height of opening h_{o2} = **5** ft

Height to underside of lintel over opening l_{o2} = **8.75** ft

Position of opening P_{o2} = **13.67** ft

Total area of wall $A = h \times b - w_{o1} \times h_{o1} - w_{o2} \times h_{o2}$ = **256.75** ft^2

Panel construction

Nominal stud size 2" x 6"

Dressed stud size 1.5" x 5.5"

EAST, LEVEL 3

Cross-sectional area of studs	A_s = **8.25** in^2
Stud spacing	s = **16** in
Nominal end post size	2" x 6"
Dressed end post size	1.5" x 5.5"
Cross-sectional area of end posts	A_e = **8.25** in^2
Hole diameter	Dia = **1** in
Net cross-sectional area of end posts	A_{en} = **6.75** in^2
Nominal collector size	2 x 2" x 6"
Dressed collector size	2 x 1.5" x 5.5"
Service condition	Dry
Temperature	100 degF or less
Vertical anchor stiffness	k_a = **30000** lb/in

From NDS Supplement Table 4A - Reference design values for visually graded dimension lumber (2" - 4" thick)

Species, grade and size classification	Hem-Fir, no.2 grade, 2" & wider
Specific gravity	G = **0.43**
Tension parallel to grain	F_t = **525** lb/in^2
Compression parallel to grain	F_c = **1300** lb/in^2
Modulus of elasticity	E = **1300000** lb/in^2
Minimum modulus of elasticity	E_{min} = **470000** lb/in^2

Sheathing details

Sheathing material	7/16" wood panel structural I oriented strandboard sheathing
Fastener type	8d common nails at 6"centers

From SDPWS Table 4.3A Nominal Unit Shear Capacities for Wood-Frame Shear Walls - Wood-based Panels

Nominal unit shear capacity for seismic design	v_s = min(560 plf $\times$ min[1 - (0.5 - G), 1], 1740 plf) = **520.8** lb/ft
Nominal unit shear capacity for wind design	v_w = min(785 plf $\times$ min[1 - (0.5 - G), 1], 2435 plf) = **730.1** lb/ft
Apparent shear wall shear stiffness	G_a = **16** kips/in

Loading details

Dead load acting on top of panel	D = **460** lb/ft
Snow load acting on top of panel	S = **800** lb/ft
Self weight of panel	S_{wt} = **10** lb/ft^2
In plane wind load acting at head of panel	W = **3800** lbs
Wind load serviceability factor	f_{Wserv} = **1.00**

Chord forces from shear walls above

Chord	$W_{ch[i]}$ (lbs)	$E_{q\_ch[i]}$ (lbs)	$D_{C\_ch[i]}$ (lbs)	$D_{T\_ch[i]}$ (lbs)	$L_{f\_ch[i]}$ (lbs)	$L_{r\_ch[i]}$ (lbs)	$S_{ch[i]}$ (lbs)	$R_{ch[i]}$ (lbs)
Ch1	-697;	0;	0;	0;	0;	0;	0;	0;
Ch2	697;	0;	0;	0;	0;	0;	0;	0;

From IBC 2021 cl.1605.1 Basic load combinations from ASCE 7, section 2.4

Load combination no.1	D + 0.6W
Load combination no.2	D + 0.7E
Load combination no.3	D + 0.75L_f + 0.45W + 0.75(L_r or S or R)
Load combination no.4	D + 0.75L_f + 0.525E + 0.75S
Load combination no.5	0.6D + 0.6W
Load combination no.6	0.6D + 0.7E

EAST, LEVEL 3

Adjustment factors

Load duration factor – Table 2.3.2 C_D = **1.60**

Size factor for tension – Table 4A C_{Ft} = **1.30**

Size factor for compression – Table 4A C_{Fc} = **1.10**

Wet service factor for tension – Table 4A C_{Mt} = **1.00**

Wet service factor for compression – Table 4A C_{Mc} = **1.00**

Wet service factor for modulus of elasticity – Table 4A C_{ME} = **1.00**

Temperature factor for tension – Table 2.3.3 C_{tt} = **1.00**

Temperature factor for compression – Table 2.3.3 C_{tc} = **1.00**

Temperature factor for modulus of elasticity – Table 2.3.3 C_{tE} = **1.00**

Incising factor – cl.4.3.8 C_i = **1.00**

Buckling stiffness factor – cl.4.4.2 C_T = **1.00**

Adjusted modulus of elasticity $E_{min}' = E_{min} \times C_{ME} \times C_{tE} \times C_i \times C_T$ = **470000** psi

Critical buckling design value $F_{cE} = 0.822 \times E_{min}' / (h / d)^2$ = **772** psi

Reference compression design value $F_c^* = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i$ = **2288** psi

For sawn lumber c = **0.8**

Column stability factor – eqn.3.7-1 $C_P = (1 + (F_{cE} / F_c^*)) / (2 \times c) - \sqrt{([(1 + (F_{cE} / F_c^*)) / (2 \times c)]^2 - (F_{cE} / F_c^*) / c)}$ = **0.31**

From SDPWS Table 4.3.4 Maximum Shear Wall Aspect Ratios

Maximum shear wall aspect ratio 3.5

Perforated wall length b_1 = **3.25** ft

Shear wall aspect ratio h / b_1 = **3.154**

Perforated wall length b_2 = **8.42** ft

Shear wall aspect ratio h / b_2 = **1.217**

Perforated wall length b_3 = **11.33** ft

Shear wall aspect ratio h / b_3 = **0.905**

Shear capacity adjustment factor – cl.4.3.3.5

Sum of perforated shear wall lengths $\Sigma L_i = b_1 \times 2 \times b_s / h + b_2 + b_3$ = **21.811** ft

Total length of perforated shear wall $L_{tot} = b_1 + w_{o1} + b_2 + w_{o2} + b_3$ = **27** ft

Total area of openings $A_o = w_{o1} \times h_{o1} + w_{o2} \times h_{o2}$ = **20** ft^2

Sheathing area ratio (eqn. 4.3-6) $r = 1 / (1 + A_o /(h \times \Sigma L_i))$ = **0.918**

Shear capacity adjustment factor (eqn. 4.3-5) C_o = **0.976**

Perforated shear wall capacity

Maximum shear force under wind loading $V_{w\_max} = 0.6 \times W$ = **2.28** kips

Shear capacity for wind loading $V_w = v_w \times C_o \times \Sigma L_i / 2$ = **7.77** kips

V_{w_max} / V_w = **0.293**

PASS - Shear capacity for wind load exceeds maximum shear force

Chord capacity for chord 1

Load combination 5

Shear force for maximum tension $V = 0.6 \times W$ = **2.28** kips

EAST, LEVEL 3

Axial force for maximum tension — $P = (0.6 \times (D + S_{wt} \times h)) \times b / 2 + 0.6 \times W_{ch1}$ = **4.138** kips

Maximum tensile force in chord — $T = V \times h / ((C_o \times \Sigma L_i)) - P$ = $\boxed{\textbf{-3.040}\text{ kips}}$ ← NEGATIVE. NO OVERTURNING

Maximum applied tensile stress — $f_t = T / A_{en}$ = **-450** lb/in²

Design tensile stress — $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i$ = **1092** lb/in²

f_t / F_t' = **-0.412**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 1

Shear force for maximum compression — $V = 0.6 \times W$ = **2.28** kips

Axial force for maximum compression — $P = ((D + S_{wt} \times h)) \times s / 2 + -1 \times 0.6 \times W_{ch1}$ = **0.793** kips

Maximum compressive force in chord — $C = V \times h / ((C_o \times \Sigma L_i)) + P$ = $\boxed{\textbf{1.891}\text{ kips}}$

Maximum applied compressive stress — $f_c = C / A_e$ = **229** lb/in²

Design compressive stress — $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P$ = **709** lb/in²

f_c / F_c' = **0.323**

PASS - Design compressive stress exceeds maximum applied compressive stress

Chord capacity for chord 2

Load combination 5

Shear force for maximum tension — $V = 0.6 \times W$ = **2.28** kips

Axial force for maximum tension — $P = (0.6 \times (D + S_{wt} \times h)) \times b / 2 + -1 \times 0.6 \times W_{ch2}$ = **4.138** kips

Maximum tensile force in chord — $T = V \times h / ((C_o \times \Sigma L_i)) - P$ = $\boxed{\textbf{-3.040}\text{ kips}}$ ← NEGATIVE. NO OVERTURNING

Maximum applied tensile stress — $f_t = T / A_{en}$ = **-450** lb/in²

Design tensile stress — $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i$ = **1092** lb/in²

f_t / F_t' = **-0.412**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 1

Shear force for maximum compression — $V = 0.6 \times W$ = **2.28** kips

Axial force for maximum compression — $P = ((D + S_{wt} \times h)) \times s / 2 + 0.6 \times W_{ch2}$ = **0.793** kips

Maximum compressive force in chord — $C = V \times h / ((C_o \times \Sigma L_i)) + P$ = $\boxed{\textbf{1.891}\text{ kips}}$

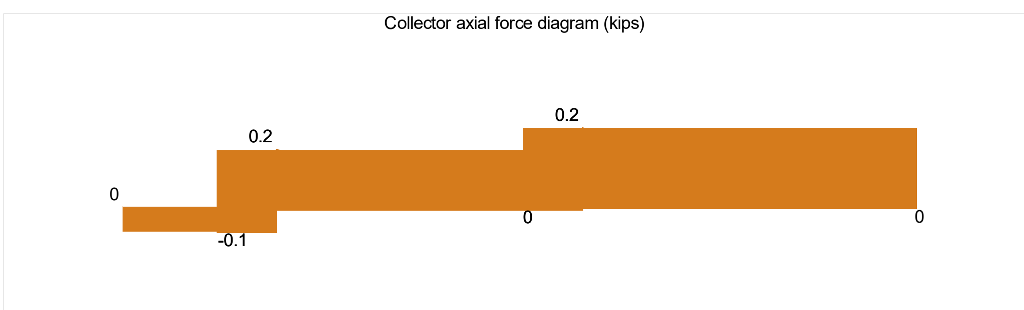
Maximum applied compressive stress — $f_c = C / A_e$ = **229** lb/in²

Design compressive stress — $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P$ = **709** lb/in²

f_c / F_c' = **0.323**

PASS - Design compressive stress exceeds maximum applied compressive stress

Collector capacity



EAST, LEVEL 3

Maximum shear force on wall $V_{max} = V_{w\_max} =$ **2.28** kips

Uniform shear applied to wall $v_a = V_{max} / ((C_o \times \Sigma L_i)) =$ **107.1** plf

Shear resisted by wall segments $v_b = v_a \times b / (b_1 + b_2 + b_3) =$ **125.7** plf

Maximum force in collector $P_{coll} =$ **0.211** kips

Maximum applied tensile stress $f_t = P_{coll} / (2 \times A_s) =$ **13** lb/in^2

Design tensile stress $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i =$ **1092** lb/in^2

$f_t / F_t' =$ **0.012**

PASS - Design tensile stress exceeds maximum applied tensile stress

Maximum applied compressive stress $f_c = P_{coll} / (2 \times A_s) =$ **13** lb/in^2

Column stability factor $C_P =$ **1.00**

Design compressive stress $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P =$ **2288** lb/in^2

$f_c / F_c' =$ **0.006**

PASS - Design compressive stress exceeds maximum applied compressive stress

Wind load deflection

Design shear force $V_{\delta w} = f_{Wserv} \times W =$ **3.8** kips

Deflection limit $\Delta_{w\_allow} = h / 500 =$ **0.246** in

Induced unit shear $v_{\delta w\_max} = V_{\delta w} / (C_o \times \Sigma L_i) =$ **178.51** lb/ft

Anchor tension force $T_\delta = \max(0 \text{ kips}, v_{\delta w\_max} \times h - 0.6 \times (D + S_{wt} \times h) \times b / 2 + \max(\text{abs}(W_{ch1}), \text{abs}(W_{ch2}))) =$ **0.000** kips

Shear wall deflection – Eqn. 4.3-1 $\delta_{sww} = 2 \times v_{\delta w\_max} \times h^3 / (3 \times E \times A_e \times \Sigma L_i) + v_{\delta w\_max} \times h / (G_a) + h \times T_\delta / (k_a \times \Sigma L_i) =$ **0.121** in

$\delta_{sww} / \Delta_{w\_allow} =$ **0.492**

PASS - Shear wall deflection is less than deflection limit

EAST, LEVEL 2

WOOD SHEAR WALL DESIGN (NDS)

In accordance with NDS2018 allowable stress design and the perforated shear wall method

Tedds calculation version 1.2.08

Design summary

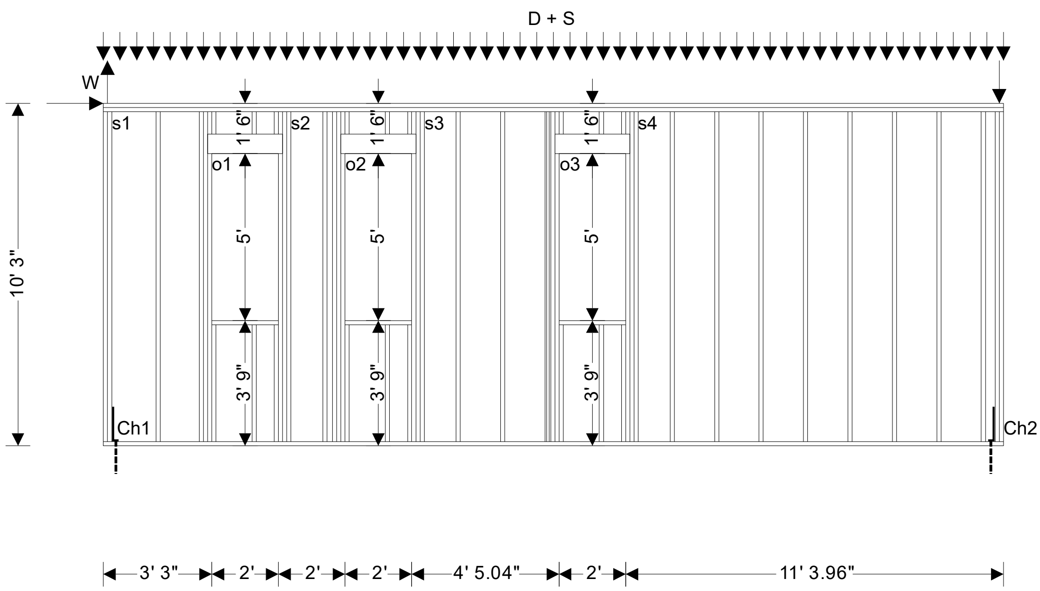
Description	Unit	Provided	Required	Utilization	Result
Shear capacity	lbs	5911	3600	0.609	PASS
Chord capacity	lb/in^2	818	250	0.305	PASS
Collector capacity	lb/in^2	1508	64	0.043	PASS
Deflection	in	0.246	0.245	0.997	PASS

Panel details

Structural I wood panel sheathing on one side

Panel height h = **10.25** ft

Panel length b = **27** ft



Panel opening details

Width of opening w_{o1} = **2** ft

Height of opening h_{o1} = **5** ft

Height to underside of lintel over opening l_{o1} = **8.75** ft

Position of opening P_{o1} = **3.25** ft

Width of opening w_{o2} = **2** ft

Height of opening h_{o2} = **5** ft

Height to underside of lintel over opening l_{o2} = **8.75** ft

Position of opening P_{o2} = **7.25** ft

Width of opening w_{o3} = **2** ft

Height of opening h_{o3} = **5** ft

Height to underside of lintel over opening l_{o3} = **8.75** ft

Position of opening P_{o3} = **13.67** ft

Total area of wall $A = h \times b - w_{o1} \times h_{o1} - w_{o2} \times h_{o2} - w_{o3} \times h_{o3}$ = **246.75** ft^2

EAST, LEVEL 2

Panel construction

Nominal stud size 2" x 6"

Dressed stud size 1.5" x 5.5"

Cross-sectional area of studs A_s = **8.25** in^2

Stud spacing s = **16** in

Nominal end post size 2 x 2" x 6"

Dressed end post size 2 x 1.5" x 5.5"

Cross-sectional area of end posts A_e = **16.5** in^2

Hole diameter Dia = **1** in

Net cross-sectional area of end posts A_{en} = **13.5** in^2

Nominal collector size 2 x 2" x 6"

Dressed collector size 2 x 1.5" x 5.5"

Service condition Dry

Temperature 100 degF or less

Vertical anchor stiffness k_a = **69646** lb/in

From NDS Supplement Table 4A - Reference design values for visually graded dimension lumber (2" - 4" thick)

Species, grade and size classification Hem-Fir, no.1 & btr grade, 2" & wider

Specific gravity G = **0.43**

Tension parallel to grain F_t = **725** lb/in^2

Compression parallel to grain F_c = **1350** lb/in^2

Modulus of elasticity E = **1500000** lb/in^2

Minimum modulus of elasticity E_{min} = **550000** lb/in^2

Sheathing details

Sheathing material 7/16" wood panel structural I oriented strandboard sheathing

Fastener type 8d common nails at 6"centers

From SDPWS Table 4.3A Nominal Unit Shear Capacities for Wood-Frame Shear Walls - Wood-based Panels

Nominal unit shear capacity for seismic design v_s = min(560 plf × min[1 - (0.5 - G), 1], 1740 plf) = **520.8** lb/ft

Nominal unit shear capacity for wind design v_w = min(785 plf × min[1 - (0.5 - G), 1], 2435 plf) = **730.1** lb/ft

Apparent shear wall shear stiffness G_a = **16** kips/in

Loading details

Dead load acting on top of panel D = **720** lb/ft

Snow load acting on top of panel S = **800** lb/ft

Self weight of panel S_{wt} = **10** lb/ft^2

In plane wind load acting at head of panel W = **6000** lbs

Wind load serviceability factor f_{Wserv} = **1.00**

Chord forces from shear walls above

Chord	$W_{ch[i]}$ (lbs)	$E_{q\_ch[i]}$ (lbs)	$D_{C\_ch[i]}$ (lbs)	$D_{T\_ch[i]}$ (lbs)	$L_{f\_ch[i]}$ (lbs)	$L_{r\_ch[i]}$ (lbs)	$S_{ch[i]}$ (lbs)	$R_{ch[i]}$ (lbs)
Ch1	-2152;	0;	0;	0;	0;	0;	0;	0;
Ch2	2152;	0;	0;	0;	0;	0;	0;	0;

From IBC 2021 cl.1605.1 Basic load combinations from ASCE 7, section 2.4

Load combination no.1 D + 0.6W

Load combination no.2 D + 0.7E

Load combination no.3 D + 0.75L_f + 0.45W + 0.75(L_r or S or R)

EAST, LEVEL 2

Load combination no.4 — D + 0.75L$_f$ + 0.525E + 0.75S

Load combination no.5 — 0.6D + 0.6W

Load combination no.6 — 0.6D + 0.7E

Adjustment factors

Load duration factor – Table 2.3.2 — C_D = **1.60**

Size factor for tension – Table 4A — C_{Ft} = **1.30**

Size factor for compression – Table 4A — C_{Fc} = **1.10**

Wet service factor for tension – Table 4A — C_{Mt} = **1.00**

Wet service factor for compression – Table 4A — C_{Mc} = **1.00**

Wet service factor for modulus of elasticity – Table 4A — C_{ME} = **1.00**

Temperature factor for tension – Table 2.3.3 — C_{tt} = **1.00**

Temperature factor for compression – Table 2.3.3 — C_{tc} = **1.00**

Temperature factor for modulus of elasticity – Table 2.3.3 — C_{tE} = **1.00**

Incising factor – cl.4.3.8 — C_i = **1.00**

Buckling stiffness factor – cl.4.4.2 — C_T = **1.00**

Adjusted modulus of elasticity — $E_{min}' = E_{min} \times C_{ME} \times C_{tE} \times C_i \times C_T$ = **550000** psi

Critical buckling design value — $F_{cE} = 0.822 \times E_{min}' / (h / d)^2$ = **904** psi

Reference compression design value — $F_c^* = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i$ = **2376** psi

For sawn lumber — c = **0.8**

Column stability factor – eqn.3.7-1 — $C_P = (1 + (F_{cE} / F_c^*)) / (2 \times c) - \sqrt{([(1 + (F_{cE} / F_c^*)) / (2 \times c)]^2 - (F_{cE} / F_c^*) / c)}$ = **0.34**

From SDPWS Table 4.3.4 Maximum Shear Wall Aspect Ratios

Maximum shear wall aspect ratio — 3.5

Perforated wall length — b_1 = **3.25** ft

Shear wall aspect ratio — h / b_1 = **3.154**

Perforated wall length — b_2 = **2** ft

Shear wall aspect ratio — h / b_2 = **5.125**

Perforated wall length — b_3 = **4.42** ft

Shear wall aspect ratio — h / b_3 = **2.319**

Perforated wall length — b_4 = **11.33** ft

Shear wall aspect ratio — h / b_4 = **0.905**

Shear capacity adjustment factor – cl.4.3.3.5

Sum of perforated shear wall lengths — $\Sigma L_i = b_1 \times 2 \times b_s / h + b_3 \times 2 \times b_s / h + b_4$ = **16.194** ft

Total length of perforated shear wall — $L_{tot} = b_1 + w_{o1} + w_{o2} + b_3 + w_{o3} + b_4$ = **25** ft

Total area of openings — $A_o = w_{o1} \times h_{o1} + w_{o2} \times h_{o2} + w_{o3} \times h_{o3}$ = **30** ft^2

Sheathing area ratio (eqn. 4.3-6) — $r = 1 / (1 + A_o /(h \times \Sigma L_i))$ = **0.847**

Shear capacity adjustment factor (eqn. 4.3-5) — C_o = **1**

Perforated shear wall capacity

Maximum shear force under wind loading — $V_{w\_max} = 0.6 \times W$ = **3.6** kips

Shear capacity for wind loading — $V_w = v_w \times C_o \times \Sigma L_i / 2$ = **5.911** kips

EAST, LEVEL 2

V_{w_max} / V_w = **0.609**

PASS - Shear capacity for wind load exceeds maximum shear force

Chord capacity for chord 1

Load combination 5

Shear force for maximum tension — $V = 0.6 \times W$ = **3.6** kips

Axial force for maximum tension — $P = (0.6 \times (D + S_{wt} \times h)) \times b / 2 + 0.6 \times W_{ch1}$ = **5.371** kips

Maximum tensile force in chord — $T = V \times h / ((C_o \times \Sigma L_i)) - P$ = $\boxed{\textbf{-3.092}\text{ kips}}$ NEGATIVE. NO OVERTURNING

Maximum applied tensile stress — $f_t = T / A_{en}$ = **-229** lb/in^2

Design tensile stress — $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i$ = **1508** lb/in^2

f_t / F_t' = **-0.152**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 1

Shear force for maximum compression — $V = 0.6 \times W$ = **3.6** kips

Axial force for maximum compression — $P = ((D + S_{wt} \times h)) \times s / 2 + -1 \times 0.6 \times W_{ch1}$ = **1.84** kips

Maximum compressive force in chord — $C = V \times h / ((C_o \times \Sigma L_i)) + P$ = $\boxed{\textbf{4.118}\text{ kips}}$

Maximum applied compressive stress — $f_c = C / A_e$ = **250** lb/in^2

Design compressive stress — $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P$ = **818** lb/in^2

f_c / F_c' = **0.305**

PASS - Design compressive stress exceeds maximum applied compressive stress

Chord capacity for chord 2

Load combination 5

Shear force for maximum tension — $V = 0.6 \times W$ = **3.6** kips

Axial force for maximum tension — $P = (0.6 \times (D + S_{wt} \times h)) \times b / 2 + -1 \times 0.6 \times W_{ch2}$ = **5.371** kips

Maximum tensile force in chord — $T = V \times h / ((C_o \times \Sigma L_i)) - P$ = $\boxed{\textbf{-3.092}\text{ kips}}$ NEGATIVE. NO OVERTURNING

Maximum applied tensile stress — $f_t = T / A_{en}$ = **-229** lb/in^2

Design tensile stress — $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i$ = **1508** lb/in^2

f_t / F_t' = **-0.152**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 1

Shear force for maximum compression — $V = 0.6 \times W$ = **3.6** kips

Axial force for maximum compression — $P = ((D + S_{wt} \times h)) \times s / 2 + 0.6 \times W_{ch2}$ = **1.84** kips

Maximum compressive force in chord — $C = V \times h / ((C_o \times \Sigma L_i)) + P$ = $\boxed{\textbf{4.118}\text{ kips}}$

Maximum applied compressive stress — $f_c = C / A_e$ = **250** lb/in^2

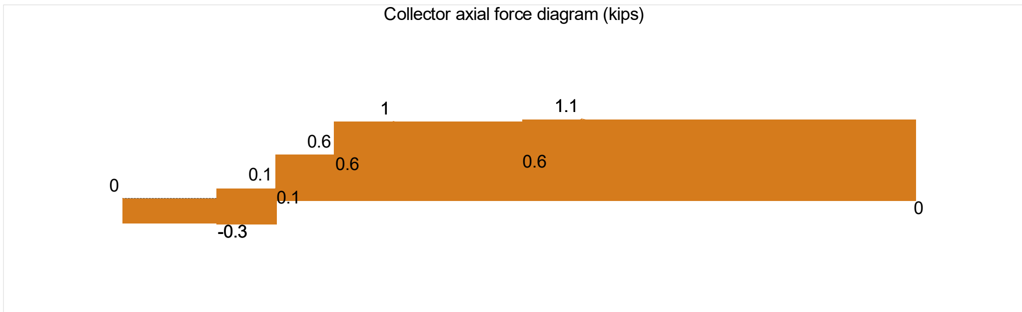
Design compressive stress — $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P$ = **818** lb/in^2

f_c / F_c' = **0.305**

PASS - Design compressive stress exceeds maximum applied compressive stress

EAST, LEVEL 2

Collector capacity



Maximum shear force on wall $V_{max} = V_{w\_max} =$ **3.6** kips

Uniform shear applied to wall $v_a = V_{max} / ((C_o \times \Sigma L_i)) =$ **222.3** plf

Shear resisted by wall segments $v_b = v_a \times b / (b_1 + b_3 + b_4) =$ **315.9** plf

Maximum force in collector $P_{coll} =$ **1.061** kips

Maximum applied tensile stress $f_t = P_{coll} / (2 \times A_s) =$ **64** lb/in^2

Design tensile stress $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i =$ **1508** lb/in^2

$f_t / F_t' =$ **0.043**

PASS - Design tensile stress exceeds maximum applied tensile stress

Maximum applied compressive stress $f_c = P_{coll} / (2 \times A_s) =$ **64** lb/in^2

Column stability factor $C_P =$ **1.00**

Design compressive stress $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P =$ **2376** lb/in^2

$f_c / F_c' =$ **0.027**

PASS - Design compressive stress exceeds maximum applied compressive stress

Wind load deflection

Design shear force $V_{\delta w} = f_{Wserv} \times W =$ **6** kips

Deflection limit $\Delta_{w\_allow} = h / 500 =$ **0.246** in

Induced unit shear $v_{\delta w\_max} = V_{\delta w} / (C_o \times \Sigma L_i) =$ **370.51** lb/ft

Anchor tension force $T_\delta = \max(0 \text{ kips}, v_{\delta w\_max} \times h - 0.6 \times (D + S_{wt} \times h) \times b / 2 + \max(\text{abs}(W_{ch1}), \text{abs}(W_{ch2}))) =$ **0.000** kips

Shear wall deflection – Eqn. 4.3-1 $\delta_{sww} = 2 \times v_{\delta w\_max} \times h^3 / (3 \times E \times A_e \times \Sigma L_i) + v_{\delta w\_max} \times h / (G_a) + h \times T_\delta / (k_a \times \Sigma L_i) =$ **0.245** in

$\delta_{sww} / \Delta_{w\_allow} =$ **0.997**

PASS - Shear wall deflection is less than deflection limit

EAST, LEVEL 1

WOOD SHEAR WALL DESIGN (NDS)

In accordance with NDS2018 allowable stress design and the perforated shear wall method

Tedds calculation version 1.2.08

Design summary

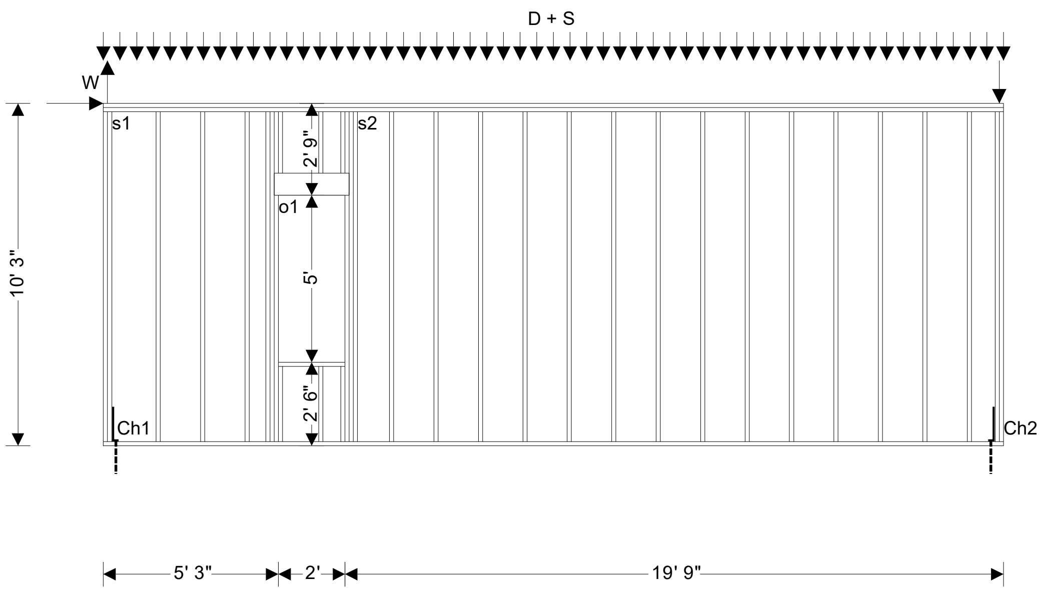
Description	Unit	Provided	Required	Utilization	Result
Shear capacity	lbs	8823	4920	0.558	PASS
Chord capacity	lb/in^2	818	331	0.404	PASS
Collector capacity	lb/in^2	1508	19	0.013	PASS
Deflection	in	0.246	0.222	0.903	PASS

Panel details

Structural I wood panel sheathing on one side

Panel height h = **10.25** ft

Panel length b = **27** ft



Panel opening details

Width of opening w_{o1} = **2** ft

Height of opening h_{o1} = **5** ft

Height to underside of lintel over opening l_{o1} = **7.5** ft

Position of opening P_{o1} = **5.25** ft

Total area of wall $A = h \times b - w_{o1} \times h_{o1}$ = **266.75** ft^2

Panel construction

Nominal stud size 2" x 6"

Dressed stud size 1.5" x 5.5"

Cross-sectional area of studs A_s = **8.25** in^2

Stud spacing s = **16** in

Nominal end post size 2 x 2" x 6"

Dressed end post size 2 x 1.5" x 5.5"

EAST, LEVEL 1

Cross-sectional area of end posts A_e = **16.5** in^2
Hole diameter Dia = **1** in
Net cross-sectional area of end posts A_{en} = **13.5** in^2
Nominal collector size 2 x 2" x 6"
Dressed collector size 2 x 1.5" x 5.5"
Service condition Dry
Temperature 100 degF or less
Vertical anchor stiffness k_a = **30000** lb/in

From NDS Supplement Table 4A - Reference design values for visually graded dimension lumber (2" - 4" thick)

Species, grade and size classification Hem-Fir, no.1 & btr grade, 2" & wider
Specific gravity G = **0.43**
Tension parallel to grain F_t = **725** lb/in^2
Compression parallel to grain F_c = **1350** lb/in^2
Modulus of elasticity E = **1500000** lb/in^2
Minimum modulus of elasticity E_{min} = **550000** lb/in^2

Sheathing details

Sheathing material 7/16" wood panel structural I oriented strandboard sheathing
Fastener type 8d common nails at 6"centers

From SDPWS Table 4.3A Nominal Unit Shear Capacities for Wood-Frame Shear Walls - Wood-based Panels

Nominal unit shear capacity for seismic design v_s = min(560 plf × min[1 - (0.5 - G), 1], 1740 plf) = **520.8** lb/ft
Nominal unit shear capacity for wind design v_w = min(785 plf × min[1 - (0.5 - G), 1], 2435 plf) = **730.1** lb/ft
Apparent shear wall shear stiffness G_a = **16** kips/in

Loading details

Dead load acting on top of panel D = **980** lb/ft
Snow load acting on top of panel S = **800** lb/ft
Self weight of panel S_{wt} = **10** lb/ft^2
In plane wind load acting at head of panel W = **8200** lbs
Wind load serviceability factor f_{Wserv} = **1.00**

Chord forces from shear walls above

Chord	$W_{ch[i]}$ (lbs)	$E_{q\_ch[i]}$ (lbs)	$D_{C\_ch[i]}$ (lbs)	$D_{T\_ch[i]}$ (lbs)	$L_{f\_ch[i]}$ (lbs)	$L_{r\_ch[i]}$ (lbs)	$S_{ch[i]}$ (lbs)	$R_{ch[i]}$ (lbs)
Ch1	-4418;	0;	0;	0;	0;	0;	0;	0;
Ch2	4418;	0;	0;	0;	0;	0;	0;	0;

From IBC 2021 cl.1605.1 Basic load combinations from ASCE 7, section 2.4

Load combination no.1 D + 0.6W
Load combination no.2 D + 0.7E
Load combination no.3 D + 0.75L_f + 0.45W + 0.75(L_r or S or R)
Load combination no.4 D + 0.75L_f + 0.525E + 0.75S
Load combination no.5 0.6D + 0.6W
Load combination no.6 0.6D + 0.7E

Adjustment factors

Load duration factor – Table 2.3.2 C_D = **1.60**
Size factor for tension – Table 4A C_{Ft} = **1.30**
Size factor for compression – Table 4A C_{Fc} = **1.10**

EAST, LEVEL 1

Wet service factor for tension – Table 4A C_{Mt} = **1.00**

Wet service factor for compression – Table 4A C_{Mc} = **1.00**

Wet service factor for modulus of elasticity – Table 4A

C_{ME} = **1.00**

Temperature factor for tension – Table 2.3.3 C_{tt} = **1.00**

Temperature factor for compression – Table 2.3.3

C_{tc} = **1.00**

Temperature factor for modulus of elasticity – Table 2.3.3

C_{tE} = **1.00**

Incising factor – cl.4.3.8 C_i = **1.00**

Buckling stiffness factor – cl.4.4.2 C_T = **1.00**

Adjusted modulus of elasticity $E_{min}' = E_{min} \times C_{ME} \times C_{tE} \times C_i \times C_T$ = **550000** psi

Critical buckling design value $F_{cE} = 0.822 \times E_{min}' / (h / d)^2$ = **904** psi

Reference compression design value $F_c^* = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i$ = **2376** psi

For sawn lumber c = **0.8**

Column stability factor – eqn.3.7-1 $C_P = (1 + (F_{cE} / F_c^*)) / (2 \times c) - \sqrt{([(1 + (F_{cE} / F_c^*)) / (2 \times c)]^2 - (F_{cE} / F_c^*) / c)}$ = **0.34**

From SDPWS Table 4.3.4 Maximum Shear Wall Aspect Ratios

Maximum shear wall aspect ratio 3.5

Perforated wall length b_1 = **5.25** ft

Shear wall aspect ratio h / b_1 = **1.952**

Perforated wall length b_2 = **19.75** ft

Shear wall aspect ratio h / b_2 = **0.519**

Shear capacity adjustment factor – cl.4.3.3.5

Sum of perforated shear wall lengths $\Sigma L_i = b_1 + b_2$ = **25** ft

Total length of perforated shear wall $L_{tot} = b_1 + w_{o1} + b_2$ = **27** ft

Total area of openings $A_o = w_{o1} \times h_{o1}$ = **10** ft^2

Sheathing area ratio (eqn. 4.3-6) $r = 1 / (1 + A_o /(h \times \Sigma L_i))$ = **0.962**

Shear capacity adjustment factor (eqn. 4.3-5) C_o = **0.967**

Perforated shear wall capacity

Maximum shear force under wind loading $V_{w\_max} = 0.6 \times W$ = **4.92** kips

Shear capacity for wind loading $V_w = v_w \times C_o \times \Sigma L_i / 2$ = **8.823** kips

V_{w_max} / V_w = **0.558**

PASS - Shear capacity for wind load exceeds maximum shear force

Chord capacity for chord 1

Load combination 5

Shear force for maximum tension $V = 0.6 \times W$ = **4.92** kips

Axial force for maximum tension $P = (0.6 \times (D + S_{wt} \times h)) \times b / 2 + 0.6 \times W_{ch1}$ = **6.117** kips

Maximum tensile force in chord $T = V \times h / ((C_o \times \Sigma L_i)) - P$ = **-4.031** kips NEGATIVE. NO OVERTURNING

Maximum applied tensile stress $f_t = T / A_{en}$ = **-299** lb/in^2

Design tensile stress $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i$ = **1508** lb/in^2

f_t / F_t' = **-0.198**

PASS - Design tensile stress exceeds maximum applied tensile stress

EAST, LEVEL 1

Load combination 1

Shear force for maximum compression — $V = 0.6 \times W =$ **4.92** kips

Axial force for maximum compression — $P = ((D + S_{wt} \times h)) \times s / 2 + -1 \times 0.6 \times W_{ch1} =$ **3.372** kips

Maximum compressive force in chord — $C = V \times h / ((C_o \times \Sigma L_i)) + P =$ $\boxed{\textbf{5.459 kips}}$

Maximum applied compressive stress — $f_c = C / A_e =$ **331** lb/in^2

Design compressive stress — $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P =$ **818** lb/in^2

$f_c / F_c' =$ **0.404**

PASS - Design compressive stress exceeds maximum applied compressive stress

Chord capacity for chord 2

Load combination 5

Shear force for maximum tension — $V = 0.6 \times W =$ **4.92** kips

Axial force for maximum tension — $P = (0.6 \times (D + S_{wt} \times h)) \times b / 2 + -1 \times 0.6 \times W_{ch2} =$ **6.117** kips

Maximum tensile force in chord — $T = V \times h / ((C_o \times \Sigma L_i)) - P =$ $\boxed{\textbf{-4.031 kips}}$ ← NEGATIVE. NO OVERTURNING

Maximum applied tensile stress — $f_t = T / A_{en} =$ **-299** lb/in^2

Design tensile stress — $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i =$ **1508** lb/in^2

$f_t / F_t' =$ **-0.198**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 1

Shear force for maximum compression — $V = 0.6 \times W =$ **4.92** kips

Axial force for maximum compression — $P = ((D + S_{wt} \times h)) \times s / 2 + 0.6 \times W_{ch2} =$ **3.372** kips

Maximum compressive force in chord — $C = V \times h / ((C_o \times \Sigma L_i)) + P =$ $\boxed{\textbf{5.459 kips}}$

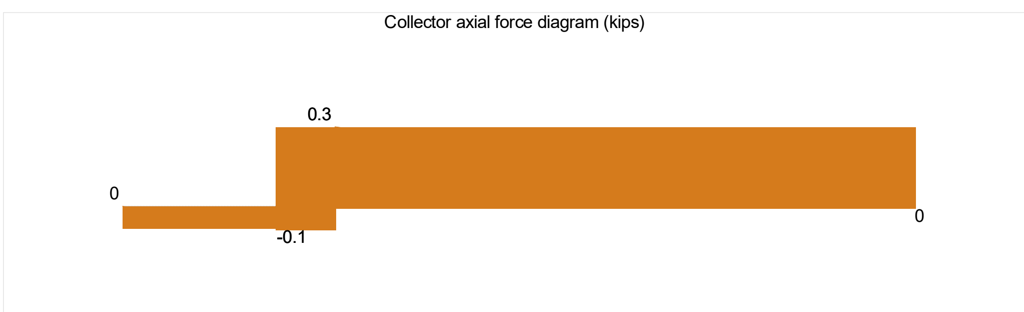
Maximum applied compressive stress — $f_c = C / A_e =$ **331** lb/in^2

Design compressive stress — $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P =$ **818** lb/in^2

$f_c / F_c' =$ **0.404**

PASS - Design compressive stress exceeds maximum applied compressive stress

Collector capacity



Maximum shear force on wall — $V_{max} = V_{w\_max} =$ **4.92** kips

Uniform shear applied to wall — $v_a = V_{max} / ((C_o \times \Sigma L_i)) =$ **203.6** plf

Shear resisted by wall segments — $v_b = v_a \times b / (b_1 + b_2) =$ **219.8** plf

Maximum force in collector — $P_{coll} =$ **0.322** kips

EAST, LEVEL 1

Maximum applied tensile stress $f_t = P_{coll} / (2 \times A_s) =$ **19** lb/in^2

Design tensile stress $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i =$ **1508** lb/in^2

$f_t / F_t' =$ **0.013**

PASS - Design tensile stress exceeds maximum applied tensile stress

Maximum applied compressive stress $f_c = P_{coll} / (2 \times A_s) =$ **19** lb/in^2

Column stability factor $C_P =$ **1.00**

Design compressive stress $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P =$ **2376** lb/in^2

$f_c / F_c' =$ **0.008**

PASS - Design compressive stress exceeds maximum applied compressive stress

Wind load deflection

Design shear force $V_{\delta w} = f_{Wserv} \times W =$ **8.2** kips

Deflection limit $\Delta_{w\_allow} = h / 500 =$ **0.246** in

Induced unit shear $v_{\delta w\_max} = V_{\delta w} / (C_o \times \Sigma L_i) =$ **339.26** lb/ft

Anchor tension force $T_\delta = \max(0 \text{ kips}, v_{\delta w\_max} \times h - 0.6 \times (D + S_{wt} \times h) \times b / 2 + \max(\text{abs}(W_{ch1}), \text{abs}(W_{ch2}))) =$ **0.000** kips

Shear wall deflection – Eqn. 4.3-1 $\delta_{sww} = 2 \times v_{\delta w\_max} \times h^3 / (3 \times E \times A_e \times \Sigma L_i) + v_{\delta w\_max} \times h / (G_a) + h \times T_\delta / (k_a \times \Sigma L_i) =$ **0.222** in

$\delta_{sww} / \Delta_{w\_allow} =$ **0.903**

PASS - Shear wall deflection is less than deflection limit

REFERENCES

Holdowns (cont.)

These products are available with additional corrosion protection. For more information, see p. 14.

SS For stainless-steel fasteners, see p. 21.

SD Many of these products are approved for installation with Strong-Drive® SD Connector screws. See pp. 348–352 for more information.

Model No.	Ga.	Dimensions (in.)					Fasteners (in.)		Minimum Wood Member Size (in.)	Allowable Tension Loads (160)			Code Ref.
		W	H	B	CL	SO	Anchor Bolt Dia. (in.)	Wood Fasteners		DF/SP	SPF/HF	Deflection at Allowable Load (in.)	
DTT1Z	14	1½	7⅛	1 7/16	¾	3/16	⅜	(6) #9 x 1½" SD	1½ x 5½	840	840	0.17	IBC, FL, LA
								(6) 0.148 x 1½		910	640	0.167	
								(8) 0.148 x 1½		910	850	0.167	
DTT2Z	14	3¼	6 15/16	1⅝	13/16	3/16	½	(8) ¼ x 1½ SDS	1½ x 3½	1,825	1,800	0.105	
								(8) ¼ x 1½ SDS	3 x 3½	2,145	1,835	0.128	
DTT2Z-SDS2.5								(8) ¼ x 2½ SDS	3 x 3½	2,145	2,105	0.128	
HDU2-SDS2.5	14	3	8 11/16	3¼	1 5/16	1⅜	⅝	(6) ¼ x 2½ SDS	3 x 3½	3,075	2,215	0.088	
HDU4-SDS2.5	14	3	10 15/16	3¼	1 5/16	1⅜	⅝	(10) ¼ x 2½ SDS	3 x 3½	4,565	3,285	0.114	
HDU5-SDS2.5	14	3	13 3/16	3¼	1 5/16	1⅜	⅝	(14) ¼ x 2½ SDS	3 x 3½	5,645	4,340	0.115	
HDU8-SDS2.5	10	3	16⅝	3½	1⅜	1½	⅞	(20) ¼ x 2½ SDS	3 x 3½	6,765	5,820	0.11	
									3½ x 3½	6,970	5,995	0.116	
									3½ x 4½	7,870	6,580	0.113	
HDU11-SDS2.5	10	3	22¼	3½	1⅜	1½	1	(30) ¼ x 2½ SDS	3½ x 5½	9,535	8,030	0.137	
									3½ x 7¼	11,175	9,610	0.137	
HDU14-SDS2.5	7	3	25 11/16	3½	1 9/16	1 9/16	1	(36) ¼ x 2½ SDS	3½ x 5½	10,770	9,260	0.122	—
									3½ x 7¼	14,390	12,375	0.177	IBC, FL, LA
									5½ x 5½	14,445	12,425	0.172	

1. HDU14 requires heavy-hex anchor nut to achieve tabulated loads (supplied with holdown).
2. HDU14 loads on 4x6 post are applicable to installation on either the narrow or the wide face of the post.
3. **Fasteners:** Nail dimensions are listed diameter by length. SD and SDS screws are Simpson Strong-Tie® Strong-Drive SD Connector and SDS Heavy-Duty Connector screws. See pp. 21–22 for fastener information.

HRS/ST/HTP/LSTA/LSTI/MST/MSTA/MSTC/MSTI

Strap Ties (cont.)

Codes: See p. 11 for Code Reference Key Chart

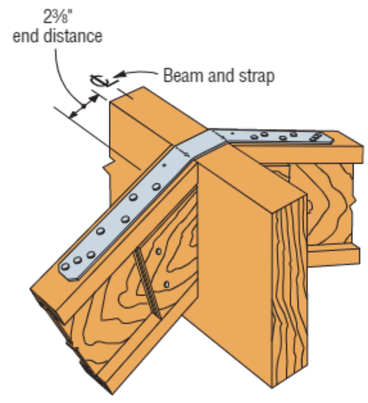
These products are available with additional corrosion protection. For more information, see p. 14.

SS For stainless-steel fasteners, see p. 21.

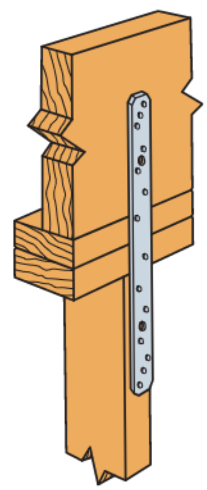
SD Many of these products are approved for installation with Strong-Drive® SD Connector screws. See pp. 348–352 for more information.

	Model No.	Ga.	Dimensions (in.) W	Dimensions (in.) L	Fasteners (Total) (in.)	Allowable Tension Loads (DF/SP) (160)	Allowable Tension Loads (SPF/HF) (160)	Code Ref.
	ST2115	20	¾	16 5/16	(10) 0.162 x 2½	660	660	IBC, FL, LA
	LSTA9		1¼	9	(8) 0.148 x 2½	740	635	
	LSTA12		1¼	12	(10) 0.148 x 2½	925	795	
	LSTA15		1¼	15	(12) 0.148 x 2½	1,110	955	
	LSTA18		1¼	18	(14) 0.148 x 2½	1,235	1,115	
	LSTA21		1¼	21	(16) 0.148 x 2½	1,235	1,235	
	LSTA24		1¼	24	(18) 0.148 x 2½	1,235	1,235	
	LSTA30	18	1¼	30	(22) 0.148 x 2½	1,640	1,640	
	LSTA36		1¼	36	(24) 0.148 x 2½	1,640	1,640	
	MSTA9		1¼	9	(8) 0.148 x 2½	750	650	
SS	MSTA12		1¼	12	(10) 0.148 x 2½	940	810	
	MSTA15		1¼	15	(12) 0.148 x 2½	1,130	970	
SS	MSTA18		1¼	18	(14) 0.148 x 2½	1,315	1,135	
	MSTA21		1¼	21	(16) 0.148 x 2½	1,505	1,295	
SS	MSTA24		1¼	24	(18) 0.148 x 2½	1,640	1,460	
	MSTA30	16	1¼	30	(22) 0.148 x 2½	2,050	1,825	
SS	MSTA36		1¼	36	(26) 0.148 x 2½	2,050	2,050	
	MSTA49		1¼	49	(26) 0.148 x 2½	2,020	2,020	
	ST9		1¼	9	(8) 0.162 x 2½	885	765	
	ST12		1¼	11 5/8	(10) 0.162 x 2½	1,105	955	
	ST18		1¼	17 3/4	(14) 0.162 x 2½	1,420	1,335	
	ST22		1¼	21 5/8	(18) 0.162 x 2½	1,420	1,420	
	HRS6	12	1 3/8	6	(6) 0.148 x 2½	605	530	
	HRS8		1 3/8	8	(10) 0.148 x 2½	1,010	880	
	HRS12		1 3/8	12	(14) 0.148 x 2½	1,415	1,230	
	ST292	20	2 1/16	9 5/16	(12) 0.162 x 2½	1,260	1,120	
	ST2122		2 1/16	12 13/16	(16) 0.162 x 2½	1,530	1,510	
	ST2215		2 1/16	16 5/16	(20) 0.162 x 2½	1,875	1,875	
	ST6215	16	2 1/16	16 5/16	(20) 0.162 x 2½	2,090	1,910	
	ST6224		2 1/16	23 5/16	(28) 0.162 x 2½	2,535	2,535	
	ST6236	14	2 1/16	33 13/16	(40) 0.162 x 2½	3,845	3,845	
	MSTI26	12	2 1/16	26	(26) 0.148 x 1½	2,745	2,380	
	MSTI36		2 1/16	36	(36) 0.148 x 1½	3,800	3,295	
	MSTI48		2 1/16	48	(48) 0.148 x 1½	5,070	4,390	
	MSTI60		2 1/16	60	(60) 0.148 x 1½	5,070	5,070	
	MSTI72		2 1/16	72	(72) 0.148 x 1½	5,070	5,070	
	HTP37Z	16	3	7	(20) 0.148 x 1½	900	690	
	MSTC28		3	28 1/4	(36) 0.148 x 3¼	3,460	2,990	
	MSTC40		3	40 1/4	(52) 0.148 x 3¼	4,735	4,315	
	MSTC52		3	52 1/4	(62) 0.148 x 3¼	4,735	4,735	
	MSTC66	14	3	65 3/4	(76) 0.148 x 3¼	5,850	5,850	
	MSTC78		3	77 3/4	(76) 0.148 x 3¼	5,850	5,850	
	HRS416Z	12	3¼	16	(16) ¼ x 1½ SDS	2,835	2,305	—
	LSTI49	18	3¾	49	(32) 0.148 x 1½	2,970	2,560	IBC, FL, LA
	LSTI73		3¾	73	(48) 0.148 x 1½	4,205	3,840	

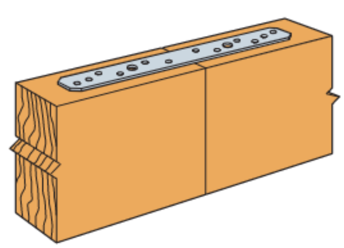
1. See pp. 266–267 for Straps and Ties General Notes.
2. **Fasteners:** Nail dimensions are listed diameter by length. SDS screws are Simpson Strong-Tie® Strong-Drive SDS Heavy-Duty Connector screws. See pp. 21–22 for fastener information.



Typical LSTA Installation
(hanger not shown)
Bend strap one time only, max. 12/12 joist pitch.



Typical LSTA18 Installation



Typical MSTA15 Installation

TABLE 6—ALLOWABLE SHEAR FOR WIND OR SEISMIC LOADING FOR WOOD STRUCTURAL PANEL HORIZONTAL DIAPHRAGMS WITH FRAMING OF DOUGLAS FIR-LARCH OR SOUTHERN PINE AND RATED SHEATHING (plf)[1,2,3,4,5,6,7,8,9]

NOMINAL NAIL DIAMETER (inch) or STAPLE GAGE (Nails must be smooth or deformed and must be carbon steel (bright or galvanized).)	MINIMUM REQUIRED FASTENER LENGTH (inches)	MINIMUM WIDTH OF FRAMING MEMBER (inches)	BLOCKED DIAPHRAGMS: FASTENER SPACING (inch) AT DIAPHRAGM BOUNDARIES (ALL CASES), AT CONTINUOUS PANEL EDGES PARALLEL TO LOAD (CASES 3, 4), AND AT ALL PANEL EDGES (CASES 5 & 6)								UNBLOCKED DIAPHRAGMS: FASTENERS SPACED 6" MAX. AT SUPPORTED EDGES			
			6		4		$2^1/_2$		2		Case 1 (No unblocked edges or continuous joints parallel to load)		All other configurations (Cases 2, 3, 4, 5 & 6)	
			Nail spacing at other panel edges (Cases 1, 2, 3 & 4): 6		6		4		3					
			Seismic	Wind	Seismic	Wind	Seismic	Wind	Seismic	Wind	Seismic	Wind	Seismic	Wind
$^3/_8$-inch Nominal Panel Thickness														
0.131	$1^3/_4$	2 3	240 270	335 375	320 360	445 505	480 540	670 755	545 610	760 855	215 240	300 335	160 180	225 250
0.120	$1^3/_4$	2 3	205 230	285 315	270 305	375 425	405 455	565 640	460 515	640 720	180 205	255 285	135 150	190 210
0.113	$1^3/_4$	2 3	180 205	255 285	240 270	335 380	360 405	505 570	410 460	575 645	160 180	225 255	120 135	170 190
14, 15,16 Gage	$1^1/_2$ Leg Length	2 3	160 180	225 250	210 235	295 330	315 355	440 495	360 400	505 560	140 160	195 225	105 120	145 170
$^7/_{16}$-inch Nominal Panel Thickness														
0.131	2	2 3	255 285	360 400	340 380	475 530	505 570	705 800	575 645	805 900	230 255	320 355	170 190	235 265
0.120	2	2 3	215 240	305 340	290 325	405 450	430 485	600 680	490 550	685 765	190 215	270 300	145 160	200 225
0.113	2	2 3	195 215	275 305	260 290	360 405	385 435	540 610	440 490	615 685	175 195	245 270	130 145	180 200
14, 15, 16 Gage	$1^1/_2$ Leg Length	2 3	165 190	230 265	225 250	315 350	335 375	470 525	380 425	530 595	150 165	210 230	110 125	155 175
$^{15}/_{32}$-inch Nominal Panel Thickness														
0.148	2	2 3	290 325	405 455	385 430	540 605	575 650	805 910	655 735	920 1030	255 290	360 405	190 215	265 300
0.135	2	2 3	255 285	355 400	340 380	475 530	505 575	710 800	580 650	810 910	225 255	315 355	170 190	235 265
0.131	2	2 3	270 300	380 420	360 400	505 560	530 600	740 840	600 675	840 945	240 265	335 370	180 200	255 280
0.120	2	2 3	230 255	325 360	305 340	430 480	450510	630 715	510 575	715 805	205 225	285 315	155 170	220 240
0.113 8d	2	2 3	205 230	290 320	275 305	385 430	405 460	570 645	460 520	645 725	185 205	255 285	140 155	195 215
14, 15, 16 Gage	$1^1/_2$ Leg Length	2 3	160 180	225 250	210 235	295 330	315 355	440 495	360 405	505 565	140 160	195 225	105 120	145 170
$^{19}/_{32}$-inch Nominal Panel Thickness[10]														
0.148	$2^1/_4$	2 3	320 360	445 505	425 480	595 675	640 720	895 1010	730 820	1025 1150	285 320	400 445	215 240	300 335
0.135	$2^1/_4$	2 3	285 320	395 450	375 425	525 595	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	355 395	190 215	265 295
0.131	$2^1/_4$	2 3	270 305	375 425	360 405	500 565	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	335 375	180 200	255 285
0.120	$2^1/_4$	2 3	235 260	325 365	310 350	435 490	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	290 325	155 175	220 245
0.113	$2^1/_4$	2 3	210 240	295 335	280 315	395 445	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	265 295	140 160	200 220
14, 15, 16 Gage	$1^1/_2$ Leg Length	2 3	175 200	245 280	235 265	330 370	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	215 245	115 130	160 180

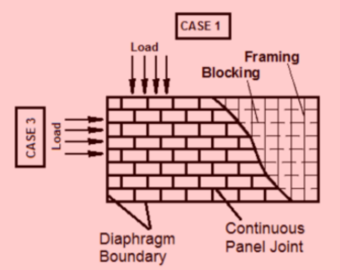
See page 11 for footnote explanations and case diagrams.

STANDARD ALLOWABLE DIAPHRAGM VALUE (8d GUN NAILS)
REDUCE BY SPECIFIC GRAVITY FACTOR FOR HEM FIR
(PER TABLE 6, FOOTNOTE 4)
HEM FIR, G = 0.43
(1-(0.5-0.43)) = 0.93

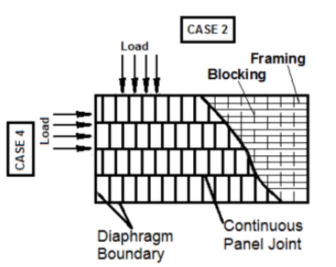
255 PLF (0.93) = **237.2 PLF**

STANDARD ALLOWABLE DIAPHRAGM VALUE (8d GUN NAILS)
REDUCE BY SPECIFIC GRAVITY FACTOR FOR HEM FIR
(PER TABLE 6, FOOTNOTE 4)
HEM FIR, G = 0.43
(1-(0.5-0.43)) = 0.93

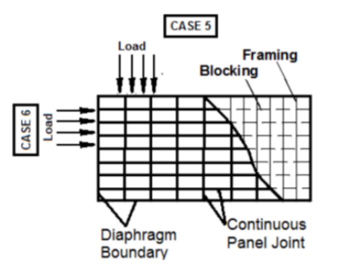
195 PLF (0.93) = **181.4 PLF**



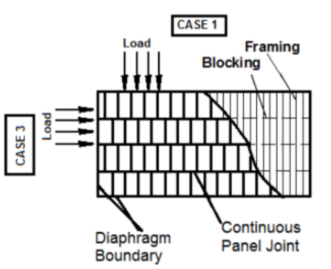
Continuous Panel Joints Perpendicular to Framing Long Panel Direction Perpendicular to Support



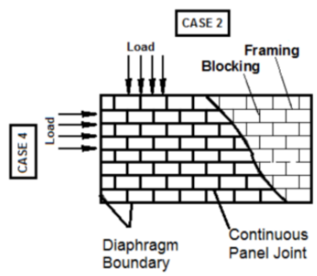
Continuous Panel Joints Parallel to Framing Long Panel Direction Perpendicular to Supports



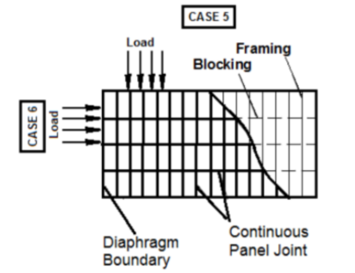
Continuous Panel Joints Perpendicular and Parallel To Framing Long Panel Direction Perpendicular to Supports



Continuous Panel Joints Perpendicular to Framing Long Panel Direction Parallel to Supports



Continuous Panel Joints Parallel to Framing Long Panel Direction Parallel to Supports

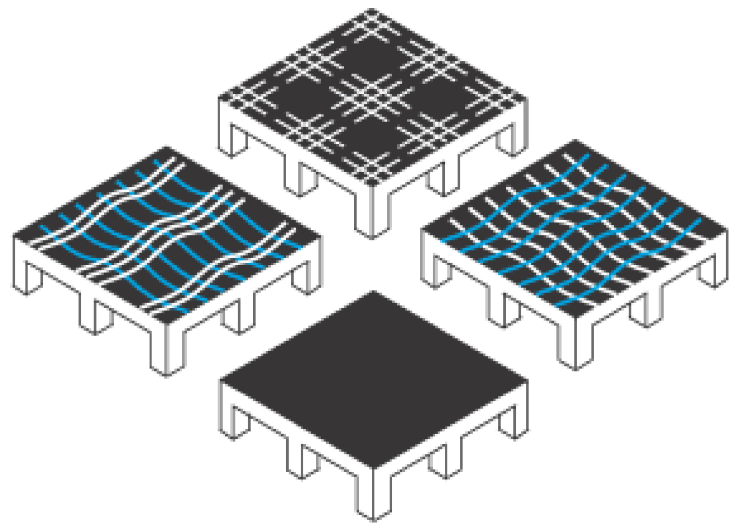


Continuous Panel Joints Perpendicular and Parallel To Framing Long Panel Direction Parallel to Supports

CASE DIAGRAMS FOR HORIZONTAL DIAPHRAGM TABLES 5 AND 6

FOUNDATION DESIGN

STRUCTURAL SLAB



Basecamp TH1 Slab and Foundation.cpt
9/6/2022

8.3.1

Table of Contents

Materials

Concrete Mix

Mix Name	Density (pcf)	Density For Loads (pcf)	f'ci (psi)	f'c (psi)	fcui (psi)	fcu (psi)	Poissons Ratio	Thermal Exp Coeff	Ec Calc	User Eci (psi)	User Ec (psi)
3000 psi	150	150	3000	3000	3725	3725	0.2	5.556e-6	Code	2500000	3000000
4000 psi	150	150	3000	4000	3725	4975	0.2	5.556e-6	Code	2500000	3000000
5000 psi	150	150	3000	5000	3725	6399	0.2	5.556e-6	Code	2500000	3000000
6000 psi	150	150	3000	6000	3725	7450	0.2	5.556e-6	Code	2500000	3000000

PT Systems

System Name	Type	Aps (in²)	Eps (ksi)	fse (ksi)	fpy (ksi)	fpu (ksi)	Duct Width (inches)	Strands Per Duct	Min Radius (feet)
½" Unbonded	unbonded	0.153	28000	175	243	270	0.5	1	6
½" Bonded	bonded	0.153	28000	160	243	270	3	4	6
0.6" Unbonded	unbonded	0.217	28000	175	243	270	0.6	1	8
0.6" Bonded	bonded	0.217	28000	160	243	270	4	4	8

PT Stressing Parameters

System Name	Jacking Stress (ksi)	Seating Loss (inches)	Anchor Friction	Wobble Friction (1/feet)	Angular Friction (1/radians)	Long-Term Losses (ksi)
½" Unbonded	216	0.25	0	0.0014	0.07	22
½" Bonded	216	0.25	0.02	0.001	0.2	22
0.6" Unbonded	216	0.25	0	0.0014	0.07	22
0.6" Bonded	216	0.25	0.02	0.001	0.2	22

Materials (2)

Reinforcing Bars

Bar Name	*As (in²)*	*Es (ksi)*	*Fy (ksi)*	*Coating*	*Straight Ld/Db*	*90 Hook Ld/Db*	*180 Hook Ld/Db*
#3	0.11	29000	60	None	Code	Code	Code
#4	0.2	29000	60	None	Code	Code	Code
#5	0.31	29000	60	None	Code	Code	Code
#6	0.44	29000	60	None	Code	Code	Code
#7	0.6	29000	60	None	Code	Code	Code
#8	0.79	29000	60	None	Code	Code	Code
#9	1	29000	60	None	Code	Code	Code
#10	1.27	29000	60	None	Code	Code	Code
#11	1.56	29000	60	None	Code	Code	Code

SSR Systems

SSR System Name	*Stud Area (in²)*	*Head Area (in²)*	*Min Clear Head Spacing (inches)*	*Specified Stud Spacing (inches)*	*Fy (ksi)*	*Stud Spacing Rounding Increment (inches)*	*Min Studs Per Rail*	*System Type*
3/8" SSR	0.11	1.11	0.5	None	50	0.25	2	Rail
1/2" SSR	0.196	1.96	0.5	None	50	0.25	2	Rail
5/8" SSR	0.307	3.07	0.5	None	50	0.25	2	Rail
3/4" SSR	0.442	4.42	0.5	None	50	0.25	2	Rail
Ancon Shearfix Auto-Size	0.1217	1.096	0.5906	None	72.52	0.03937	2	Rail
Ancon Shearfix 10 mm	0.1217	1.096	0.5906	None	72.52	0.03937	2	Rail
Ancon Shearfix 12 mm	0.1753	1.578	0.5906	None	72.52	0.03937	2	Rail
Ancon Shearfix 14 mm	0.2386	2.147	0.5906	None	72.52	0.03937	2	Rail
Ancon Shearfix 16 mm	0.3116	2.805	0.5906	None	72.52	0.03937	2	Rail
Ancon Shearfix 20 mm	0.4869	4.383	0.5906	None	72.52	0.03937	2	Rail
Ancon Shearfix 24 mm	0.7012	6.311	0.5906	None	72.52	0.03937	2	Rail

Loadings

Loading Name	*Type*	*Analysis*	*On-Pattern Factor*	*Off-Pattern Factor*
Self-Dead Loading	Self-Weight	Normal	1	1
Balance Loading	Balance	Normal	1	1
Hyperstatic Loading	Hyperstatic	Hyperstatic	1	1
Temporary Construction (At Stressing) Loading	Stressing Dead	Normal	1	1
Other Dead Loading	Dead	Normal	1	1
Live (Reducible) Loading	Live (Reducible)	Normal	1	0
Live (Unreducible) Loading	Live (Unreducible)	Normal	1	0
Live (Storage) Loading	Live (Storage)	Normal	1	0
Live (Parking) Loading	Live (Parking)	Normal	1	0
Live (Roof) Loading	Live (Roof)	Normal	1	0
Snow Loading	Snow	Normal	1	1

Load Combinations

All Dead LC

Active Design Criteria: <none>
Analysis: Linear

Loading	*Standard Factor*	*Alt. Envelope Factor*
Self-Dead Loading	1	1
Other Dead Loading	1	1

Dead + Balance LC

Active Design Criteria: <none>
Analysis: Linear

Loading	*Standard Factor*	*Alt. Envelope Factor*
Self-Dead Loading	1	1
Balance Loading	1	1
Other Dead Loading	1	1

Initial Service LC

Active Design Criteria: Initial Service Desig
Analysis: Linear

Loading	*Standard Factor*	*Alt. Envelope Factor*
Self-Dead Loading	1	1
Balance Loading	1.13	1.13
Temporary Construction (At Stressing) Loading	1	1

Load Combinations (2)

Service LC: D + L

Active Design Criteria: User Minimum Design, Code Minimum Design, Service Desi

Analysis: Linear

Loading	*Standard Factor*	*Alt. Envelope Factor*
Self-Dead Loading	1	1
Balance Loading	1	1
Other Dead Loading	1	1
Live (Reducible) Loading	1	0
Live (Unreducible) Loading	1	0
Live (Storage) Loading	1	0
Live (Parking) Loading	1	0

Service LC: D + Lr

Active Design Criteria: User Minimum Design, Code Minimum Design, Service Desi

Analysis: Linear

Loading	*Standard Factor*	*Alt. Envelope Factor*
Self-Dead Loading	1	1
Balance Loading	1	1
Other Dead Loading	1	1
Live (Roof) Loading	1	0

Service LC: D + S

Active Design Criteria: User Minimum Design, Code Minimum Design, Service Desi

Analysis: Linear

Loading	*Standard Factor*	*Alt. Envelope Factor*
Self-Dead Loading	1	1
Balance Loading	1	1
Other Dead Loading	1	1
Snow Loading	1	0

Load Combinations (3)

Service LC: D + 0.75L + 0.75Lr

Active Design Criteria: User Minimum Design, Code Minimum Design, Service Desi

Analysis: Linear

Loading	*Standard Factor*	*Alt. Envelope Factor*
Self-Dead Loading	1	1
Balance Loading	1	1
Other Dead Loading	1	1
Live (Reducible) Loading	0.75	0
Live (Unreducible) Loading	0.75	0
Live (Storage) Loading	0.75	0
Live (Parking) Loading	0.75	0
Live (Roof) Loading	0.75	0

Service LC: D + 0.75L + 0.75S

Active Design Criteria: User Minimum Design, Code Minimum Design, Service Desi

Analysis: Linear

Loading	*Standard Factor*	*Alt. Envelope Factor*
Self-Dead Loading	1	1
Balance Loading	1	1
Other Dead Loading	1	1
Live (Reducible) Loading	0.75	0
Live (Unreducible) Loading	0.75	0
Live (Storage) Loading	0.75	0
Live (Parking) Loading	0.75	0
Snow Loading	0.75	0

Load Combinations (4)

Sustained Service LC

Active Design Criteria: Sustained Service Desig

Analysis: Linear

Loading	*Standard Factor*	*Alt. Envelope Factor*
Self-Dead Loading	1	1
Balance Loading	1	1
Other Dead Loading	1	1
Live (Reducible) Loading	0.5	0.5
Live (Unreducible) Loading	0.5	0.5
Live (Storage) Loading	1	1
Live (Parking) Loading	0.5	0.5
Live (Roof) Loading	0.5	0.5

Factored LC: 1.4D

Active Design Criteria: User Minimum Design, Code Minimum Design, Strength Design, Ductility Des

Analysis: Linear

Loading	*Standard Factor*	*Alt. Envelope Factor*
Self-Dead Loading	1.4	0.9
Hyperstatic Loading	1	1
Other Dead Loading	1.4	0.9

Factored LC: 1.2D + 1.6L + 0.5Lr

Active Design Criteria: User Minimum Design, Code Minimum Design, Strength Design, Ductility Des

Analysis: Linear

Loading	*Standard Factor*	*Alt. Envelope Factor*
Self-Dead Loading	1.2	0.9
Hyperstatic Loading	1	1
Other Dead Loading	1.2	0.9
Live (Reducible) Loading	1.6	0
Live (Unreducible) Loading	1.6	0
Live (Storage) Loading	1.6	0
Live (Parking) Loading	1.6	0
Live (Roof) Loading	0.5	0

Load Combinations (5)

Factored LC: 1.2D + f1L + 1.6Lr

Active Design Criteria: User Minimum Design, Code Minimum Design, Strength Design, Ductility Des
Analysis: Linear

Loading	*Standard Factor*	*Alt. Envelope Factor*
Self-Dead Loading	1.2	0.9
Hyperstatic Loading	1	1
Other Dead Loading	1.2	0.9
Live (Reducible) Loading	0.5	0
Live (Unreducible) Loading	1	0
Live (Storage) Loading	1	0
Live (Parking) Loading	1	0
Live (Roof) Loading	1.6	0

Factored LC: 1.2D + 1.6L + 0.5S

Active Design Criteria: User Minimum Design, Code Minimum Design, Strength Design, Ductility Des
Analysis: Linear

Loading	*Standard Factor*	*Alt. Envelope Factor*
Self-Dead Loading	1.2	0.9
Hyperstatic Loading	1	1
Other Dead Loading	1.2	0.9
Live (Reducible) Loading	1.6	0
Live (Unreducible) Loading	1.6	0
Live (Storage) Loading	1.6	0
Live (Parking) Loading	1.6	0
Snow Loading	0.5	0

Load Combinations (6)

Factored LC: 1.2D + f1L + 1.6S

Active Design Criteria: User Minimum Design, Code Minimum Design, Strength Design, Ductility Des
Analysis: Linear

Loading	*Standard Factor*	*Alt. Envelope Factor*
Self-Dead Loading	1.2	0.9
Hyperstatic Loading	1	1
Other Dead Loading	1.2	0.9
Live (Reducible) Loading	0.5	0
Live (Unreducible) Loading	1	0
Live (Storage) Loading	1	0
Live (Parking) Loading	1	0
Snow Loading	1.6	0

Mesh Input: Standard Plan

Mesh Input: Beams; Slab Area Hatching; Walls Below; User Lines; User Dimensions;
Drawing Import: A-DETL; A-FLOR; A-ANNO-NOTE; S-FNDN; S-GRID-IDEN; S-GRID;
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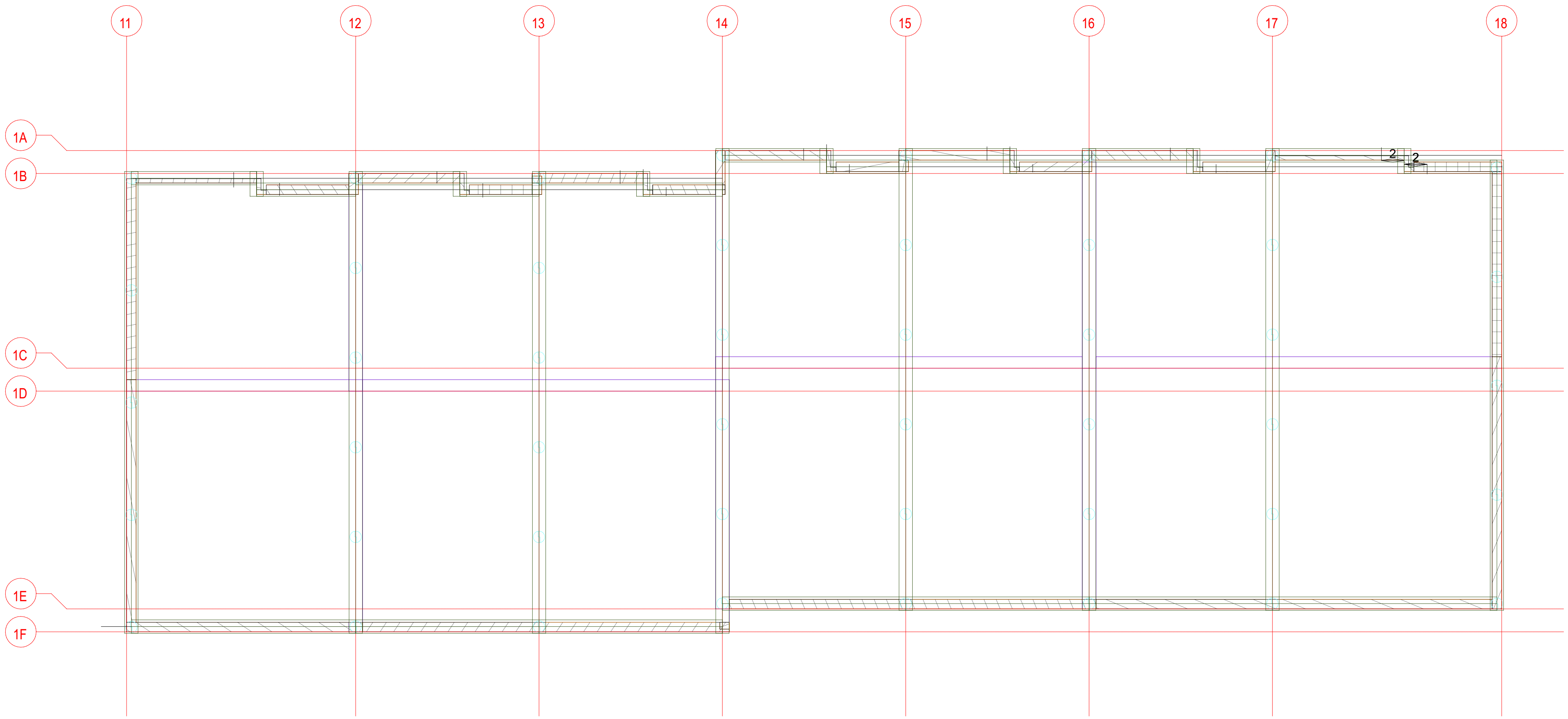

Element: Standard Plan

Element: User Lines; User Notes; User Dimensions; Wall Elements Below; Wall Elements Above; Wall Element Groups Below; Wall Element Groups Above; Wall Element Group Axes; Column Elements Below; Column Elements Above; Slab Elements; Point Springs; Point Spring Icons; Line Springs; Line Spring Icons; Area Spring
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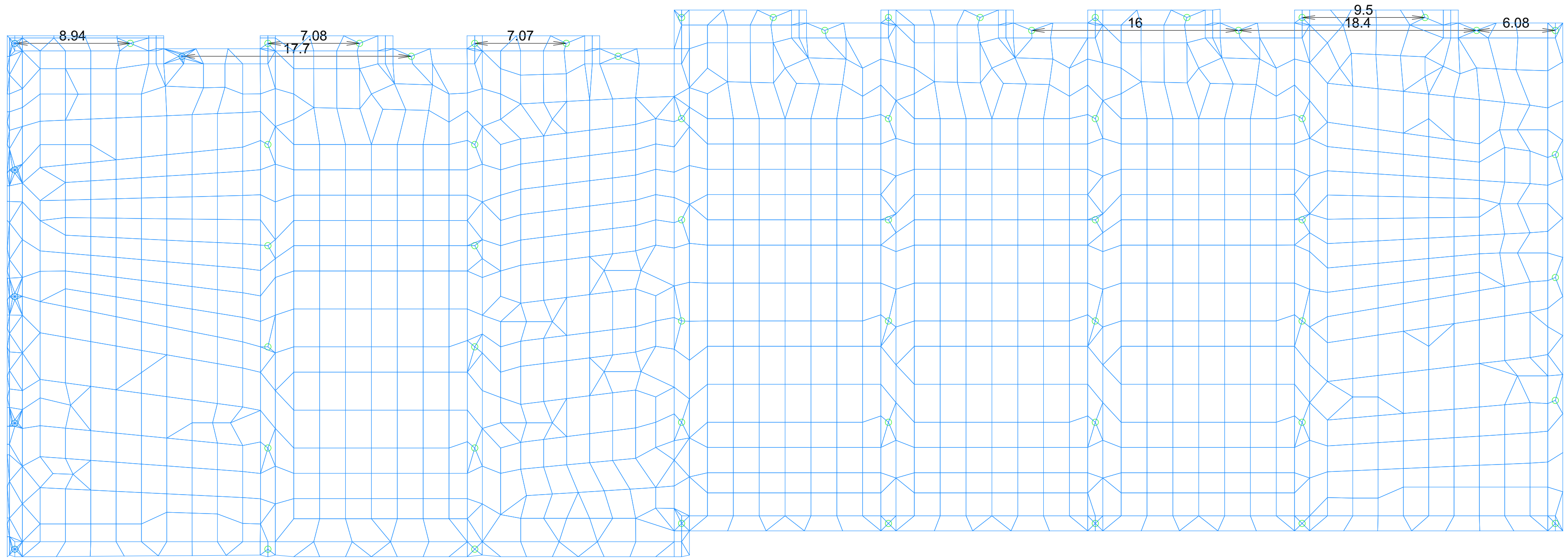

Element: Slab Summary Plan

Element: User Lines; User Notes; User Dimensions; Wall Elements Below; Wall Elements Above; Column Elements Below; Column Elements Above; Point Springs; Point Spring Icons; Line Springs; Line Spring Icons; Slab Elements; Slab Element Outline Only; Slab Element Thicknesses; Slab Element Elevations; Slab Element C
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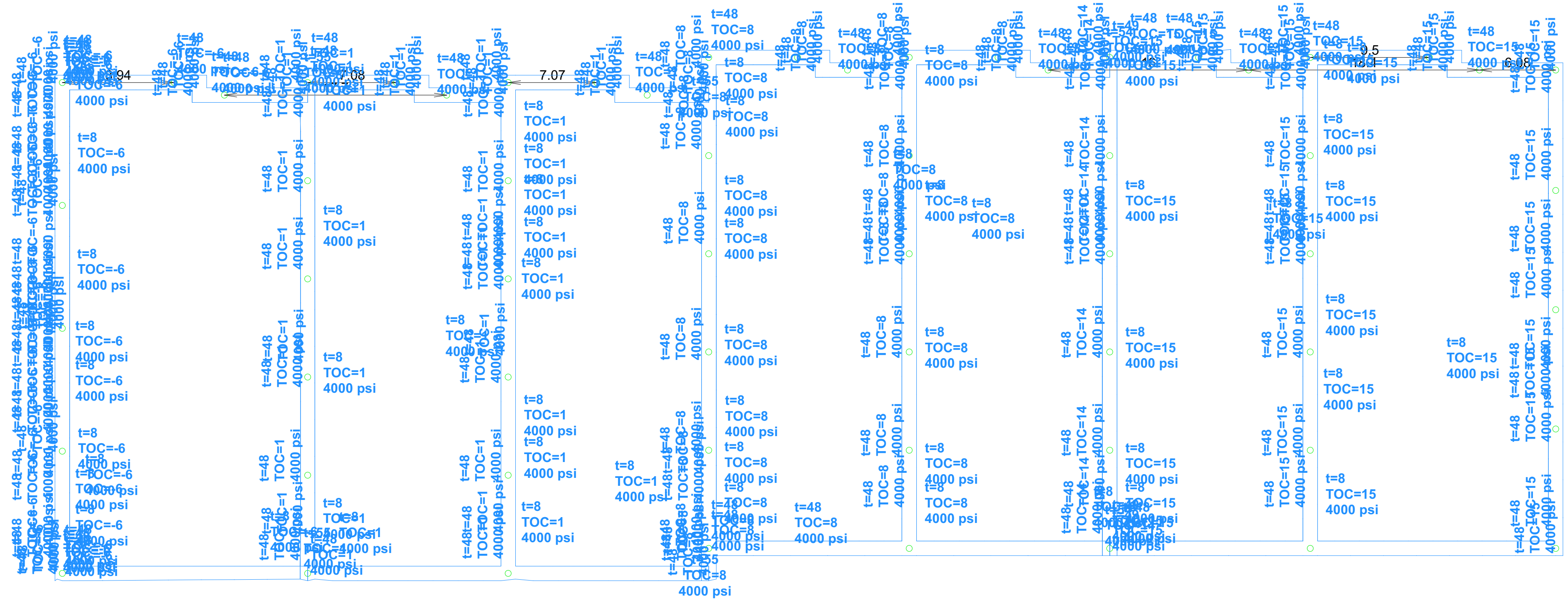

Other Dead Loading: All Loads Plan

Other Dead Loading: User Lines; User Notes; User Dimensions; Point Loads; Point Load Icons; Point Load Values; Line Loads; Line Load Icons; Line Load Values; Area Loads; Area Load Icons; Area Load Values;
Element: Wall Elements Below; Wall Elements Above; Wall Element Outline Only; Column Elements Below; Column Elements Above; Slab Elements; Slab Element Outline Only;
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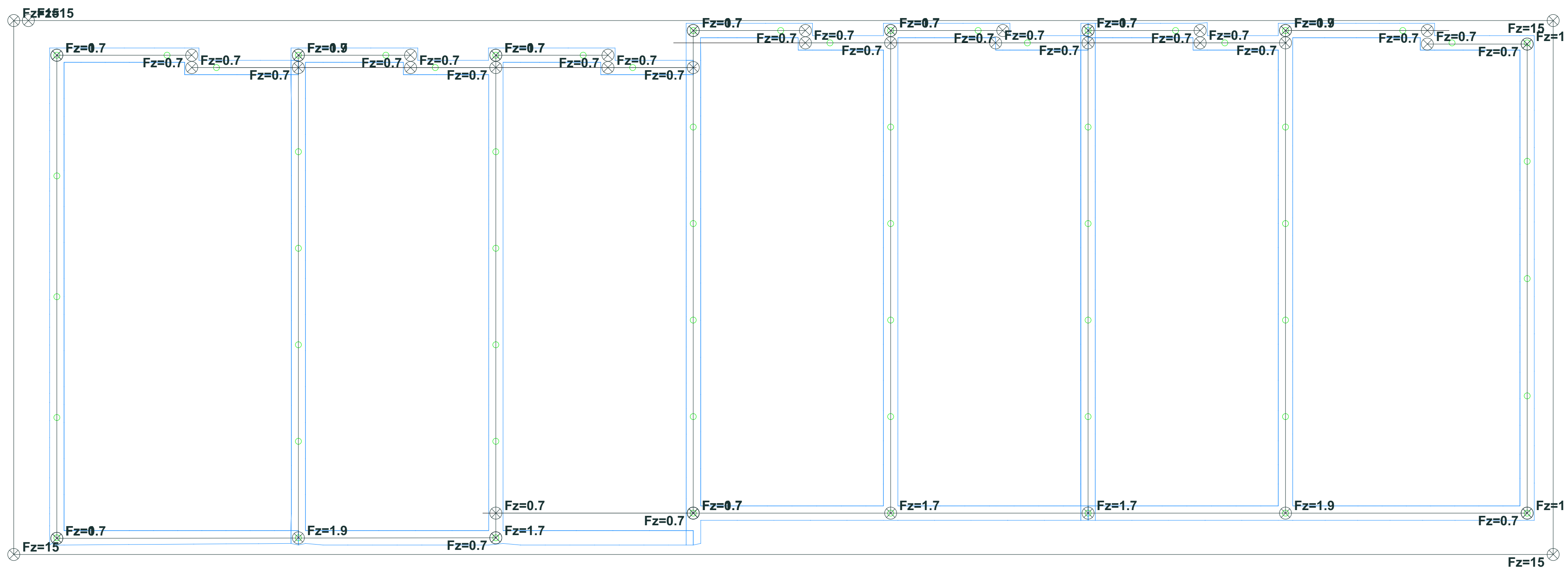

Live (Unreducible) Loading: All Loads Plan

Live (Unreducible) Loading: User Lines; User Notes; User Dimensions; Point Loads; Point Load Icons; Point Load Values; Line Loads; Line Load Icons; Line Load Values; Area Loads; Area Load Icons; Area Load Values;
Element: Wall Elements Below; Wall Elements Above; Wall Element Outline Only; Column Elements Below; Column Elements Above; Slab Elements; Slab Element Outline Only;
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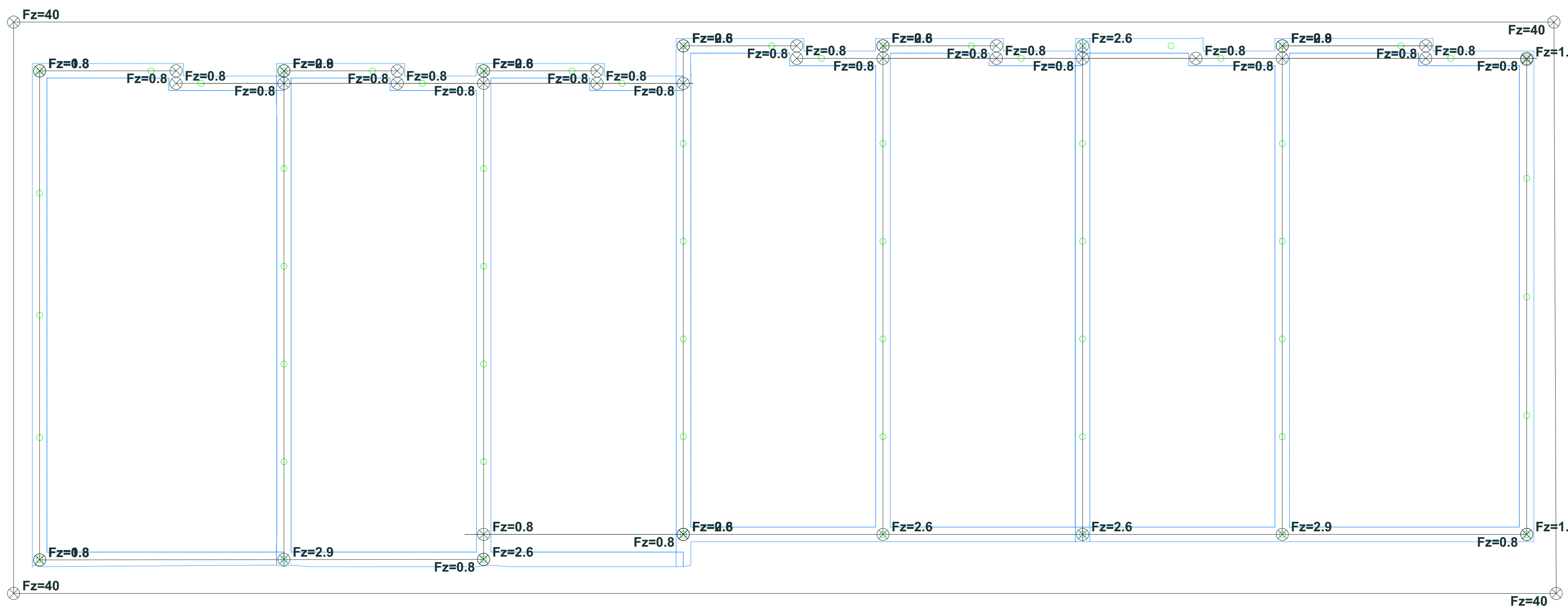

Snow Loading: All Loads Plan

Snow Loading: User Lines; User Notes; User Dimensions; Point Loads; Point Load Icons; Point Load Values; Line Loads; Line Load Icons; Line Load Values; Area Loads; Area Load Icons; Area Load Values;
Element: Wall Elements Below; Wall Elements Above; Wall Element Outline Only; Column Elements Below; Column Elements Above; Slab Elements; Slab Element Outline Only;
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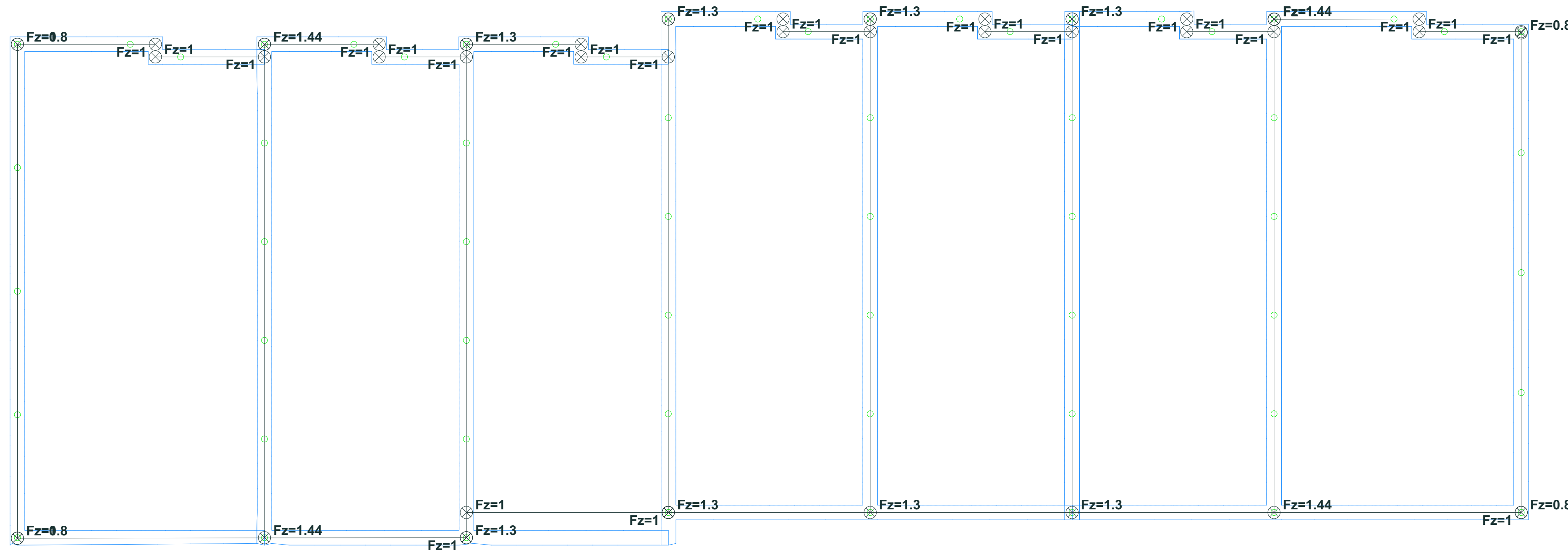
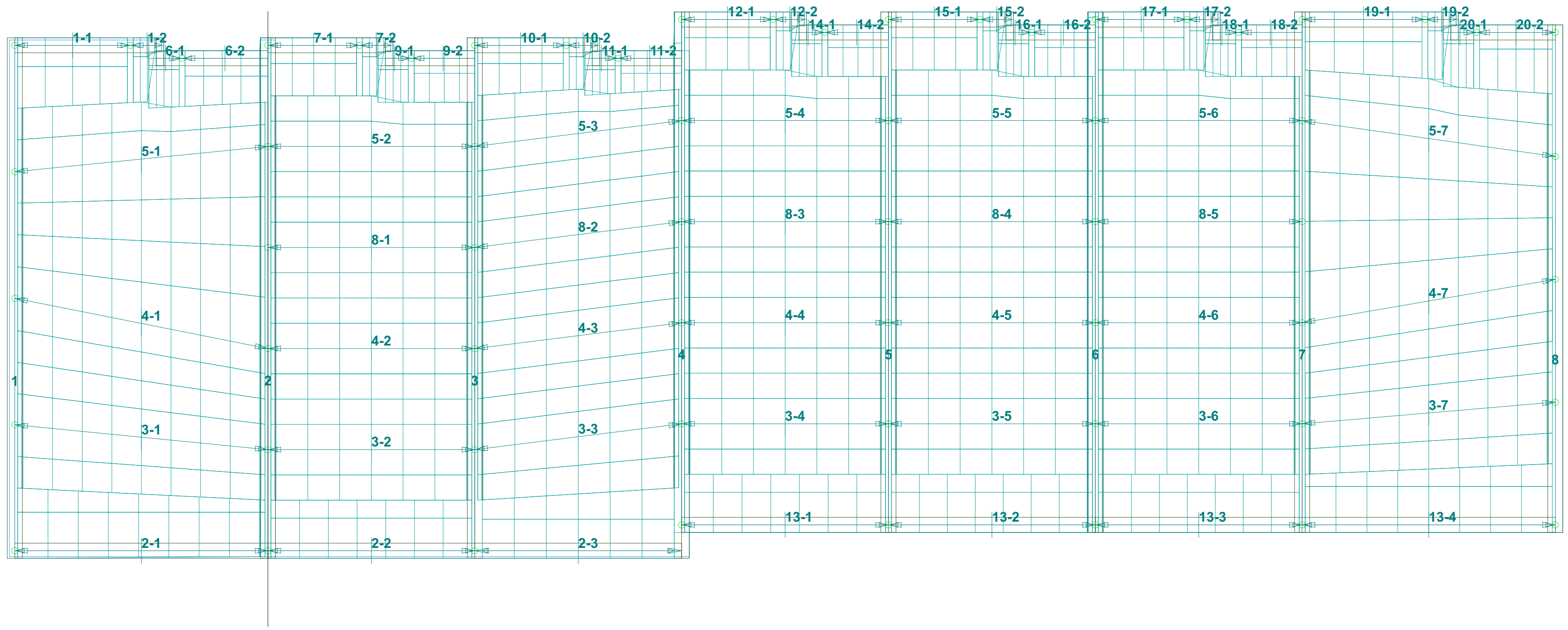


Design Strip: Latitude Design Spans Plan

Design Strip: Latitude Span Boundaries; Latitude SSs; SS Numbers; Latitude DSs; DS Numbers; Latitude Strip Boundaries; Latitude SSSs; SSS Internal Sections; User Notes; User Lines; User Dimensions;
Mesh Input: Beams; Slab Areas; Columns Below;
Scale = 1:153



Design Strip: Longitude Design Spans Plan

Design Strip: Longitude Span Boundaries; Longitude SSs; Longitude DSs; Longitude Strip Boundaries; Longitude SSSs; SSS Hatching; User Notes; User Lines; User Dimensions;
Mesh Input: Beams;
Element: Wall Elements Above; Wall Elements Below; Wall Element Outline Only; Column Elements Above; Column Elements Below; Slab Elements; Slab Element Outline Only;
Reinforcement: Longitude User Concentrated Reinf.; Longitude User Distributed Reinf.; Longitude User Individual Bars; Longitude User Transverse Reinf.; Longitude User Individual Transverse Bars;
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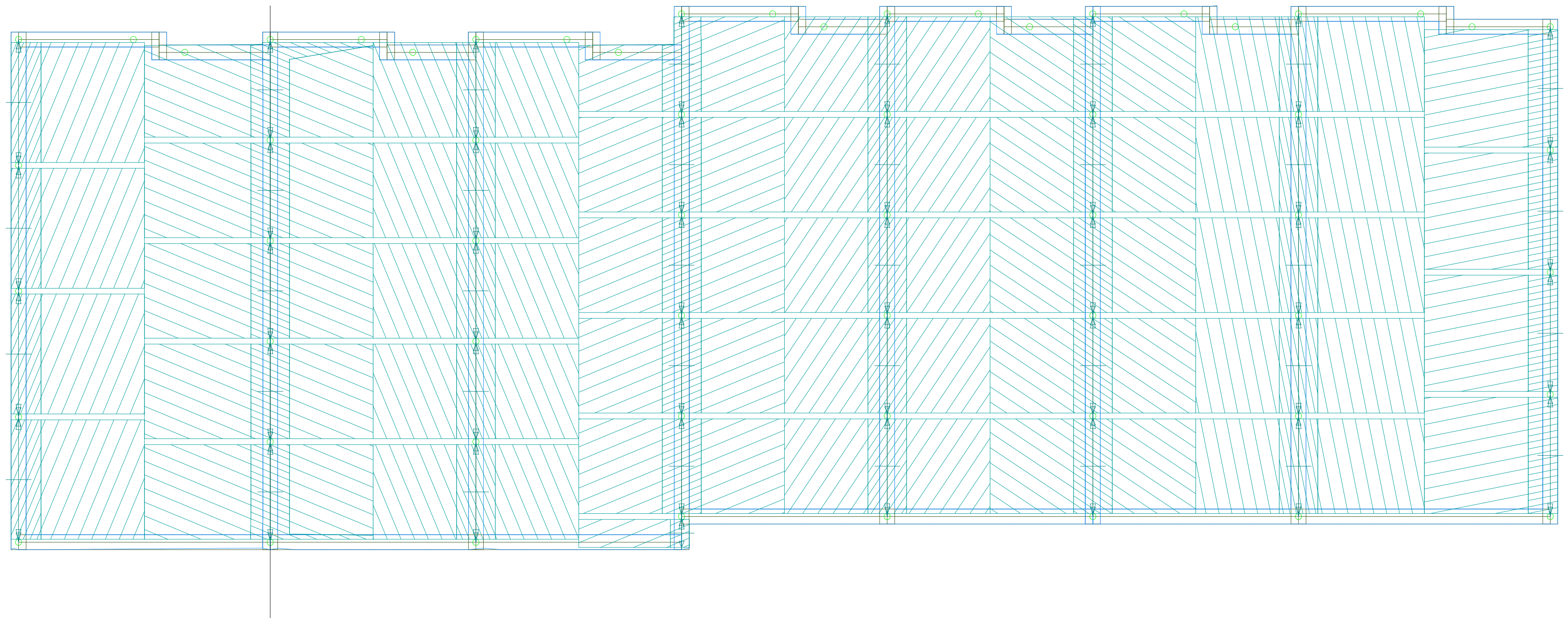

Reinforcement: Bottom Bars Plan

Reinforcement: Latitude User Concentrated Reinf.; Longitude User Concentrated Reinf.; Latitude Program Concentrated Reinf.; Longitude Program Concentrated Reinf.; Bottom Face Concentrated Reinf.; Concentrated Reinf. Descriptions; Concentrated Reinf. Extent; Latitude User Distributed Reinf.; Longitude User Distributed Re
Element: Wall Elements Above; Wall Elements Below; Wall Element Outline Only; Column Elements Above; Column Elements Below; Slab Elements; Slab Element Outline Only;
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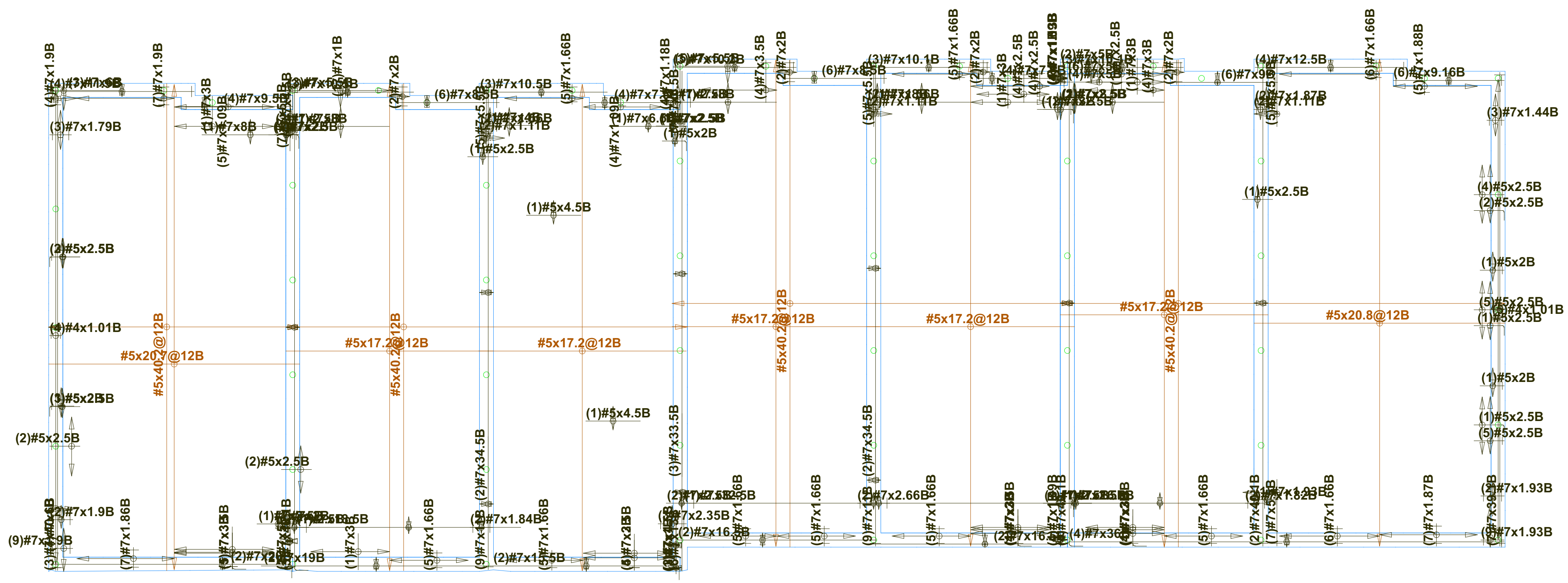

Reinforcement: Top Bars Plan

Reinforcement: Latitude User Concentrated Reinf.; Longitude User Concentrated Reinf.; Latitude Program Concentrated Reinf.; Longitude Program Concentrated Reinf.; Top Face Concentrated Reinf.; Concentrated Reinf. Descriptions; Concentrated Reinf. Extent; Latitude User Distributed Reinf.; Longitude User Distributed Reinf.
Element: Wall Elements Above; Wall Elements Below; Wall Element Outline Only; Column Elements Above; Column Elements Below; Slab Elements; Slab Element Outline Only;
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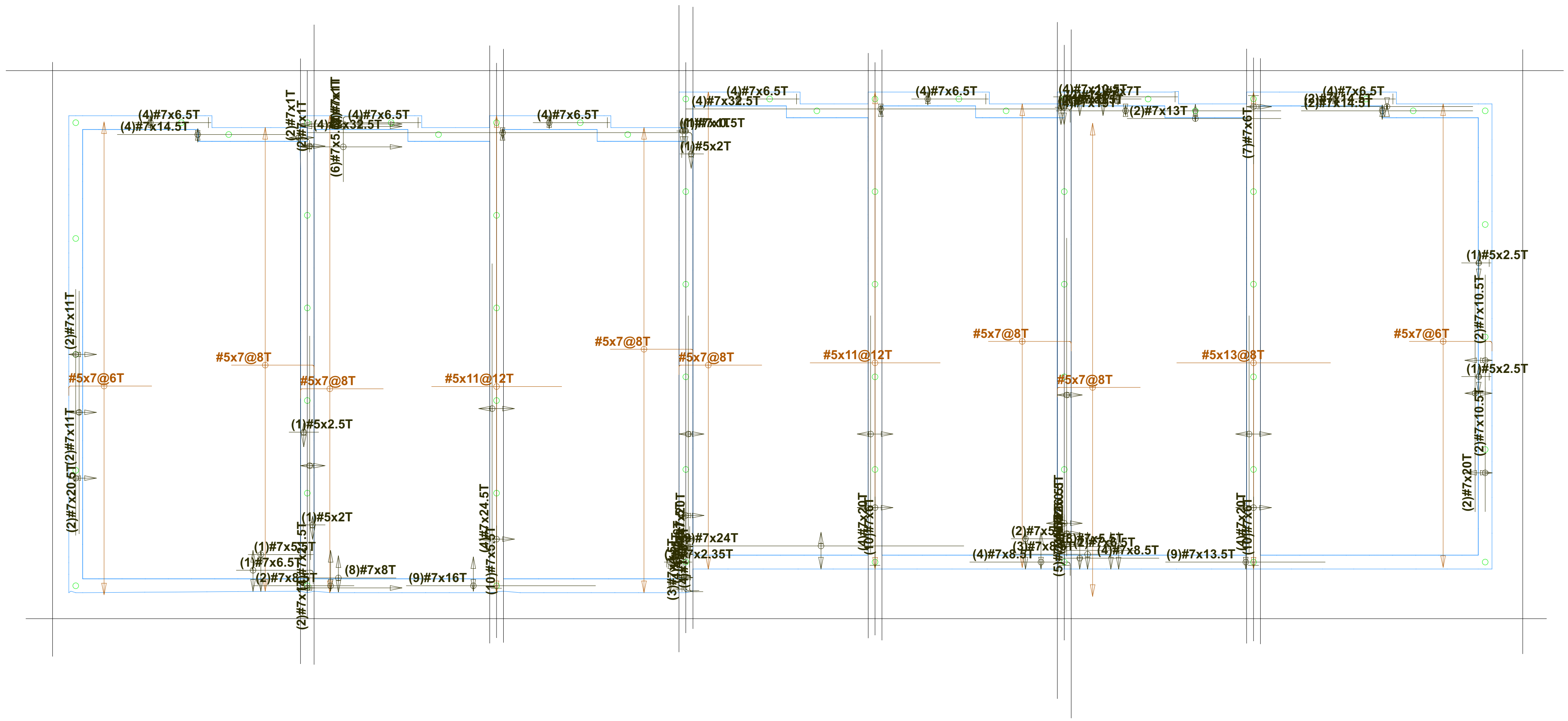
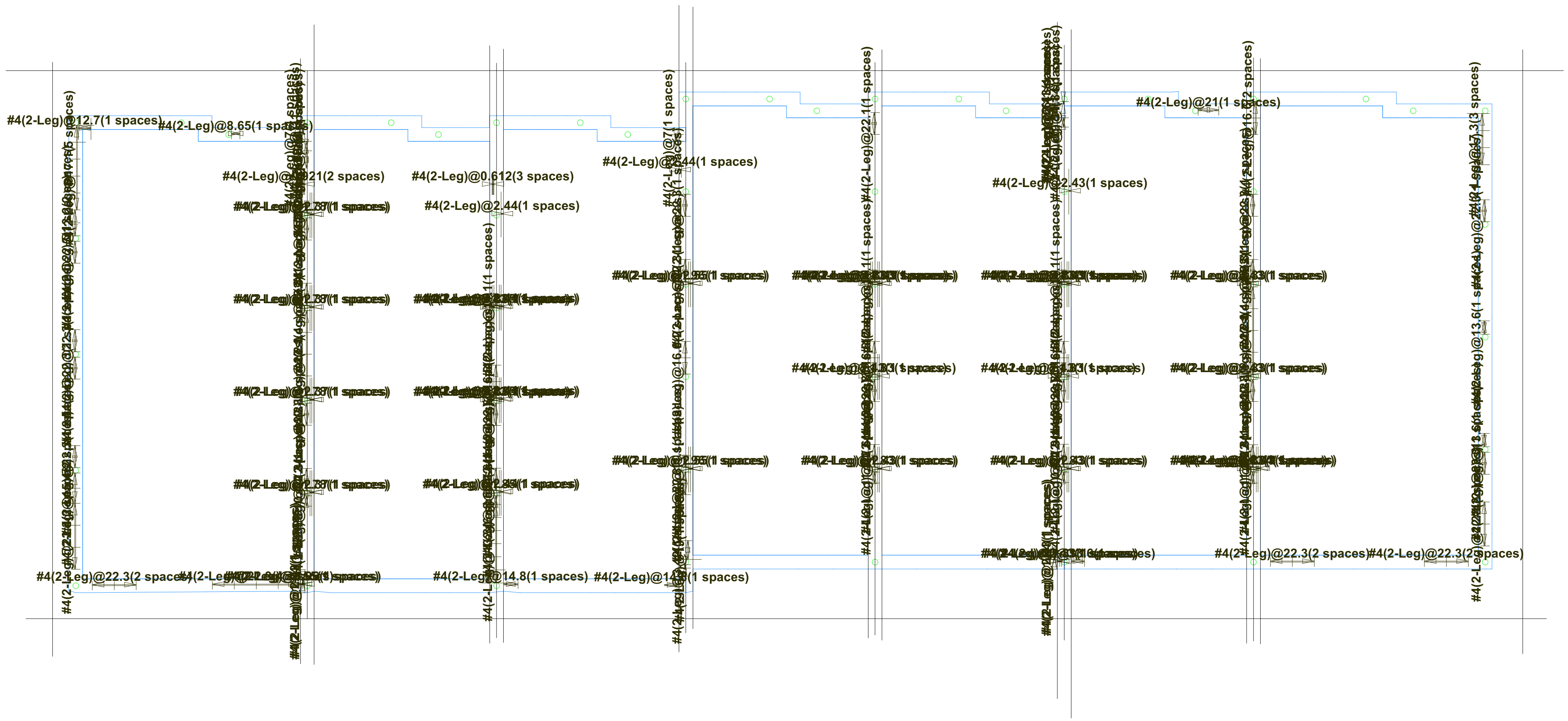


Reinforcement: Shear Bars Plan

Reinforcement: User Lines; User Notes; User Dimensions; Latitude User Transverse Reinf.; Latitude Program Transverse Reinf.; Latitude User Individual Transverse Bars; Latitude Program Individual Transverse Bars; Longitude User Transverse Reinf.; Longitude Program Transverse Reinf.; Longitude User Individual Transverse E
Element: Wall Elements Below; Wall Elements Above; Wall Element Outline Only; Column Elements Below; Column Elements Above; Slab Elements; Slab Element Outline Only;
Scale = 1:153



Design Status: Status Plan

Design Status: Latitude Span Designs; Longitude Span Designs; Span Design Numbers; Span Design Status; Latitude DS Designs; Longitude DS Designs; DS Design Numbers; DS Design Status; PC Designs; PC Design Numbers; PC Design Status; User Notes; User Lines; User Dimensions;
Element: Wall Elements Above; Wall Elements Below; Wall Element Outline Only; Column Elements Above; Column Elements Below; Slab Elements; Slab Element Outline Only;
Design Strip: Latitude SSSs; SSS Internal Sections;
Scale = 1:153