

Structural Calculations

Basecamp Steamboat Townhomes – TH1

Anthem Job # 22-048

1950 Curce Court Steamboat Springs, CO 80487

Sept. 6th, 2022



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DESIGN CRITERIA

JOB TITLE BASECAMP STEAMBOAT TOWNHOMES

Boulder | Golden | Steamboat (303) 848-8497 | (970) 300-3338

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

CS2018 Ver 2019.05.15

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STRUCTURAL CALCULATIONS

FOR

BASECAMP STEAMBOAT TOWNHOMES

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| Boulder Golden Steamboat | JOB NO. | SHEET NO. |
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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:

| U | 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:

| <u>Roof</u> | 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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Wind Loads - MWFRS h≤60' (Low-rise Buildings) except for open buildings

| Kz = Kh (case 1) = | 1.09 | Edge Strip (a) = |
|----------------------|----------|------------------|
| Base pressure (qh) = | 24.6 psf | End Zone (2a) = |
| GCpi = | +/-0.18 | Zone 2 length = |

Wind Pressure Coefficients

| | C | ASE 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)

| 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 |
| 2E 3E 4E 5E | | 19.4 | 10.6 |
| 6E | | -6.1 | -15.0 |

Parapet

Windward parapet = Leeward parapet =

0.0 psf (GCpn = +1.5) 0.0 psf (GCpn = -1.0)

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).

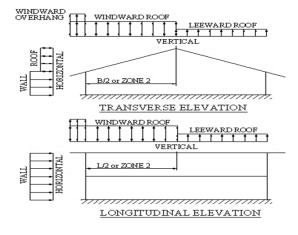
The code requires the MWFRS be designed for a min ultimate force of 16 psf multiplied by the wall area plus an 8 psf force applied to the vertical projection of the roof.

Windward roof

overhangs =

17.2 psf (upward) add to windward roof pressure

4.0 ft 8.0 ft 20.0 ft



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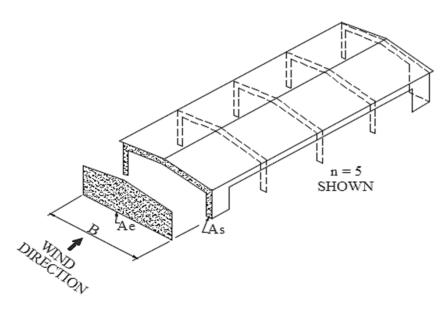
Wind Loads - h≤60' Longitudinal Direction MWFRS On Open or Partially

Enclosed Buildings with Transverse Frames and Pitched Roofs

Base pressure (qh) = $\begin{array}{c} 24.6 \text{ psf} \\ \text{GCpi} = +/-0.18 \text{ Enclosed bldg, procdure doesn't apply} \\ \text{Roof Angle } (\theta) = 14.0 \text{ deg} \end{array}$

ASCE 7-16 procedure

100 0 0



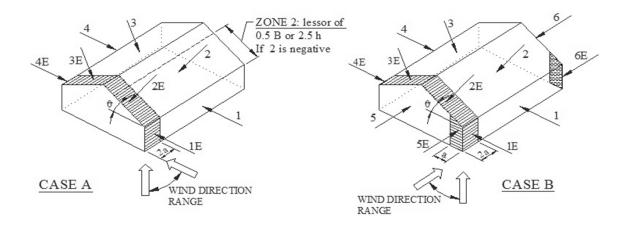
| | B= | 120.0 ft |
|---|-----------------------------------|--------------------|
| | # of frames (n) = | 5 |
| Solid are of end wall inclu | uding fascia (Às) = | 1,500.0 sf |
| F | Roof ridge height = | 56.0 ft |
| | Roof eave height = | 41.0 ft |
| | area if soild (Ăe) = | 5,820.0 sf |
| | · · · · | |
| | | |
| Longidinal Directional Force (| F) = pAe | |
| p= qh [(GCpf)windward -(GCpf)leewa | rd] 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 appli | ed at the centroid |
| , () | • • • | end wall area Ae |
| | | |

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

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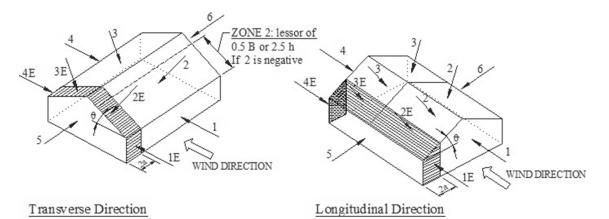
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NOTE: Torsional loads are 25% of zones 1 - 6. See code for loading diagram. Exception: One story buildings h<30' and 1 to 2 storybuildings framed with light-frame construction or with flexible diaphragms need not be designed for the torsional load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

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| Wind Loads : | ASCE 7- 16 | | |
|---|--|---------------|--|
| Ultimate Wind Speed Nominal Wind Speed Risk Category Exposure Category Enclosure Classif. Internal pressure Directionality (Kd) Kh case 1 Kh case 2 Type of roof | 115 mph 89.1 mph II C Enclosed Building +/-0.18 0.85 1.087 1.087 Sawtooth | | |
| Topographic Factor (I | ≺zt) | | ∠≰ |
| Topography | Flat | | Speed-up |
| Hill Height (H) | 0.0 ft | H< 15ft;exp C | V(z) |
| Half Hill Length (Lh) | 0.0 ft | ∴ Kzt=1.0 | \vee (ζ) x(upwind) x(downwind) |
| Actual H/Lh = | 0.00 | | H/2 |
| Use H/Lh = | 0.00 | | |
| Modified Lh = | 0.0 ft | | |
| From top of crest: x = | 0.0 ft | | and a start of the |
| Bldg up/down wind? | downwind | | ESCARPMENT |
| H/Lh= 0.00 | $K_1 = 0.000$ | | |
| x/Lh = 0.00 | $K_2 = 0.000$ | | V(z) |
| z/Lh = 0.00 | K ₃ = 1.000 | | Z A Speed-up |
| At Mean Roof Ht: | | |)/(7) 月 |
| Kzt = | $(1+K_1K_2K_3)^2 = 1.00$ | | H/2 + H/2 |

| | |
|--------------------|------|
| AXISYMMETRICAL | |
| AAISTIVIIVIETRIGAL | HILL |

| <u>Gust</u> | Effect | Factor |
|-------------|--------|----------|
| h | 1 = | 48.5 ft |
| В | = | 120.0 ft |
| /z (0.6h) |) = | 29.1 ft |

| Flexible structure if natural frequen | icy < 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

| Rigi | d Structure | Flexible or Dyn | namically Se | nsitive St | tructure | | |
|--------------------|-------------------|----------------------------|--------------|------------|----------|-----|---------|
| ē = | 0.20 | 34 rcy (η ₁) = | 0.0 Hz | | | | |
| ł = | 500 ft | Damping ratio (β) = | 0 | | | | |
| z _{min} = | 15 ft | /b = | 0.65 | | | | |
| с = | 0.20 | /α = | 0.15 | | | | |
| $g_Q, g_v =$ | 3.4 | Vz = | 107.5 | | | | |
| $L_z =$ | 487.6 ft | N ₁ = | 0.00 | | | | |
| Q = | 0.87 | R _n = | 0.000 | | | | |
| $I_z =$ | 0.20 | R _h = | 28.282 | η = | 0.000 | h = | 48.5 ft |
| G = | 0.86 use G = 0.85 | 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:

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| Test for Open Building: | All walls are at least 80% open. Ao ≥ 0.8Ag | |
|--|--|--|
| Test for Partially Enclosed Buil | ing: Predominately open on one side only | |
| Ao ≥ 1.1Aoi Ao > smaller of Aoi / Agi ≤ 0.20 Where: Ao = the total area of o Ag = the gross area of Aoi = the sum of the are | Input Test Ao 500.0 sf Ao ≥ 1.1Aoi Ag 600.0 sf Ao > 4' or 0.01Ag YES Aoi 1000.0 sf Aoi / Agi ≤ 0.20 YES Partially Enclosed Building. Must satisfy all of the following: Y' or 0.01 Ag enings in a wall that receives positive external pressure. hat wall in which Ao is identified. as of openings in the building envelope (walls and roof) not including Ao. ss 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 | | |

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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 | | |

| Root | (| GCp +/- Gcp | pi | | Surface Pr | essure (psf) | | |
|------------------------|-------|-------------|--------|--------|------------|--------------|--------|--------|
| Area | 10 sf | 50 sf | 100 sf | 500 sf | 10 sf | 50 sf | 100 sf | 500 sf |
| Negative Zone 1 | -2.38 | -1.93 | -1.73 | -1.28 | -58.5 | -47.4 | -42.6 | -31.4 |
| Negative Zone 2 | -3.38 | -2.72 | -2.44 | -1.78 | -83.0 | -66.9 | -59.9 | -43.7 |
| Span A Negative Zone 3 | -4.28 | -4 | -3.88 | -2.28 | -105.2 | -98.3 | -95.3 | -56.0 |
| Span B,C&D Neg. Zone 3 | -2.78 | -2.78 | -2.78 | -2.08 | -68.3 | -68.3 | -68.3 | -51.1 |
| Positive Zone 1 | 0.88 | 0.76 | 0.7 | 0.58 | 21.6 | 18.6 | 17.3 | 16.0 |
| Positive Zone 2 | 1.28 | 1.07 | 0.98 | 0.98 | 31.4 | 26.3 | 24.1 | 24.1 |
| 'Positive Zone 3 | 0.98 | 0.91 | 0.88 | 0.88 | 24.1 | 22.4 | 21.6 | 21.6 |
| | | | | | | | | |
| | | | | | | | | |

| Use | r input |
|-------|---------|
| 75 sf | 300 sf |
| -44.6 | -35.0 |
| -62.8 | -48.9 |
| -96.6 | -68.5 |
| -68.3 | -56.6 |
| 17.8 | 16.0 |
| 25.0 | 24.1 |
| 21.9 | 21.6 |
| | |
| | |

per de la composition de la co

| .0 psf | | Surta | ce Pressure | e (pst) | | |
|--------------------------|-------|-------|-------------|---------|--------|--------|
| Solid Parapet Pressure | 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 |

V

| <u>Walls</u> | GCp +/- GCpi | | | Surface Pressure at h | | | | |
|---------------------|--------------|--------|--------|-----------------------|-------|--------|--------|--------|
| Area | 10 sf | 100 sf | 200 sf | 500 sf | 10 sf | 100 sf | 200 sf | 500 sf |
| Negative Zone 4 | -1.28 | -1.10 | -1.05 | -0.98 | -31.4 | -27.1 | -25.8 | -24.1 |
| Negative Zone 5 | -1.58 | -1.23 | -1.12 | -0.98 | -38.8 | -30.1 | -27.5 | -24.1 |
| Positive Zone 4 & 5 | 1.18 | 1.00 | 0.95 | 0.88 | 29.0 | 24.7 | 23.3 | 21.6 |

| | 0.0 0.0 |
|----------------|----------------|
| | |
| | |
| | input |
| 46 sf | input 83 sf |
| 46 sf -28.6 | 83 sf -27.5 |
| 46 sf | 83 sf |

User input

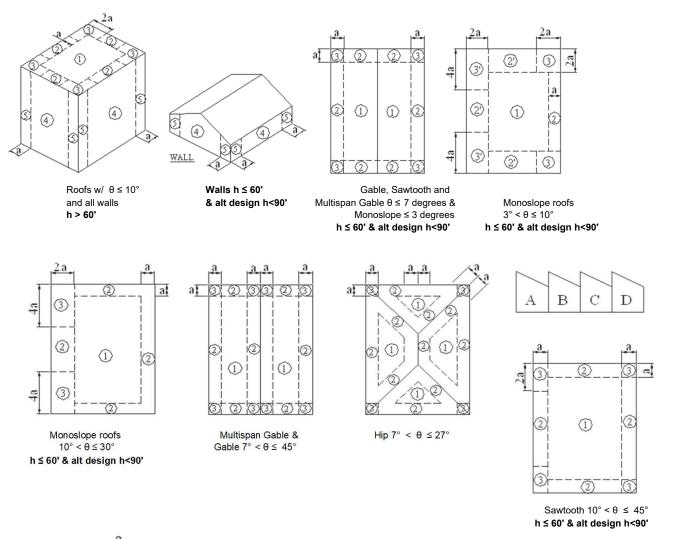
40 sf

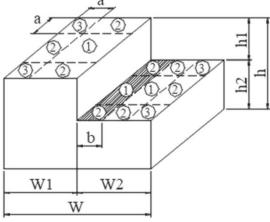
0.0 0.0 0.0

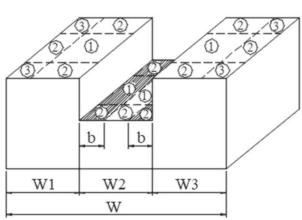
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Location of C&C Wind Pressure Zones - ASCE 7-10 & earlier



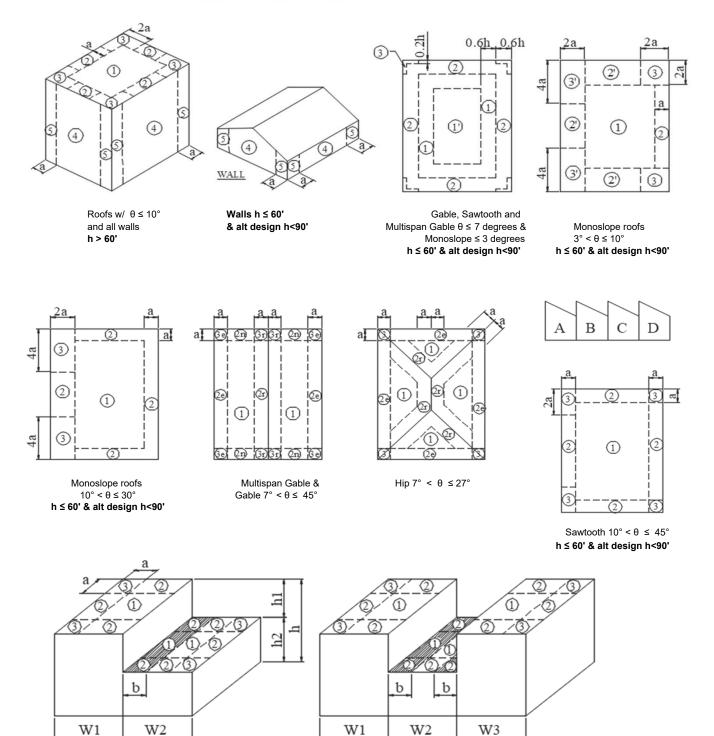




Stepped roofs $\theta \le 3^{\circ}$ h $\le 60'$ & alt design h<90'

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



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

W

W

| | | | | | | T TOWNHOMES |
|---|--|---|--------------------|---|---|---|
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| | | | CHECKE | D BY | | DATE |
| Vind Loads - Other S | Structure | es: ASCE 7- 16 | | | Ultimat | e Wind Pressure |
| Wir Gust Effect Fa | nd Factor = actor (G) = Kzt = | 1.00 0.85 Ultimate Wind S 1.00 Expo | Speed = osure = | 115 mph C | | |
| A. Solid Freestanding W | alls & Sol | lid Signs (& open sig | ans with le | ess than 30% or | <u>pen)</u> | |
| | | 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 | | Open reduction | | | <u>CaseC</u> | |
| to gross area | | factor = | 1.00 | Horiz dist from | | |
| | | | | windward edge | | =qzGCfAs (psf) |
| | ! | Case C reduction factors | 0.00 | 0 to s | 3.29 | 53.7 As |
| | | Factor if s/h>0.8 = | 0.80 | s to 2s | 2.07 | 33.7 As |
| | , | Wall return factor | 4.00 | 2s to 3s | 1.59 | 25.9 As |
| | | for Cf at 0 to s = | 1.00 | 3s to 4s | 1.31 1.23 | 21.3 As 20.1 As |
| | | | | 4s to 5s 5s to 10s | 0.78 | 20.1 AS 16.0 As |
| | | | | >10s | 0.78 | 16.0 As |
| . Open Signs & Single-Pl | ano Onon i | Frames (openings 30° | / or more | | 0.44 | 10.0 43 |
| Height to centroid of Af (z) | | rames (openings 50) | | or gross arear | Kz = | 0.849 |
| | | | | Base pressu | | 19.2 psf |
| Width (zero if round) | 0.0 ft | | | _ | | |
| Diameter (zero if rect | | D(qz)^.5 = | 8.76 | | $GC_fA_f =$ | 17.9 Af |
| Percent of open area | | = | 0.65 | Solid A | rea: A _f = | 10.0 sf |
| to gross area Directionality (Kd) | 35.0% 0.85 | C _f = | 1.1 | | F = | 179 lbs |
| . Chimneys, Tanks, & Sin | nilar Struc | <u>tures</u> | | | | |
| Height to centroid of Af (z) | 15.0 ft | | | | Kz = | 0.849 |
| Cross-Section | Square | | | Base pressu | ıre (qz) = | 20.3 psf |
| Directionality (Kd) | 0.90 | | | | , | h/D = 15.00 |
| Height (h) | 15.0 ft | | | | | |
| Width (D) | 1.0 ft | | | | | |
| | N/A | | | | | |
| Type of Surface | 1 1/7 1 | | | | | <u>d normal to face)</u> |
| Type of Surface | | ind along diagonal) | | <u>S</u> | <u>Square (win</u> | |
| Type of Surface | | ind along diagonal) Cf = 1.28 | | <u>c</u> | C _f = | 1.67 |
| Type of Surface | <u>Square (wi</u> | | | | | |
| Type of Surface | <u>Square (wi</u> | Cf = 1.28 | | | C _f = | 1.67 |
| Type of Surface | <u>Square (wi</u> | Cf = 1.28 G Cf Af = 22.1 Af | | | $C_f =$ G C _f A _f = | 1.67 28.8 Af |
| | <u>Square (wi</u> | Cf = 1.28 G Cf Af = 22.1 Af Af = sf | | | $C_{f} =$ G C _f A _f = A _f = | 1.67 28.8 Af 10.0 sf |
|). Trussed Towers | <u>Square (wi</u> F = qz C | Cf = 1.28 G Cf Af = 22.1 Af Af = sf | | | $C_{f} =$ G C _f A _f = A _f = | 1.67 28.8 Af 10.0 sf |
|). Trussed Towers Height to centroid of Af (z) | <u>Square (win</u> F = qz G 15.0 ft | Cf = 1.28 G Cf Af = 22.1 Af Af = sf | | F = q _z (| $C_{f} =$ $G C_{f} A_{f} =$ $A_{f} =$ $F =$ $Kz =$ | 1.67 28.8 Af 10.0 sf 288 lbs 0.849 |
| D. Trussed Towers Height to centroid of Af (z) \in = | <u>Square (win</u> F = qz G 15.0 ft 0.27 | Cf = 1.28 G Cf Af = 22.1 Af Af = sf | | | $C_{f} =$ $G C_{f} A_{f} =$ $A_{f} =$ $F =$ $Kz =$ | 1.67 28.8 Af 10.0 sf 288 lbs |
| D. Trussed Towers Height to centroid of Af (z) \in = Tower Cross Section | <u>Square (win</u> F = qz C 15.0 ft 0.27 triangle | Cf = 1.28 G Cf Af = 22.1 Af Af = sf | | F = q _z (Base pressu | $C_{f} =$ $G C_{f} A_{f} =$ $A_{f} =$ $F =$ $Kz =$ $Kz =$ $Kz =$ $Kz =$ $Kz =$ | 1.67 28.8 Af 10.0 sf 288 lbs 0.849 21.4 psf |
| 9. Trussed Towers Height to centroid of Af (z) \in = | <u>Square (win</u> F = qz G 15.0 ft 0.27 | Cf = 1.28 G Cf Af = 22.1 Af Af = sf | | F = q _z (| $C_{f} =$ $G C_{f} A_{f} =$ $A_{f} =$ $F =$ $Kz =$ $Kz =$ $Ire (qz) =$ $d factor =$ | 1.67 28.8 Af 10.0 sf 288 lbs 0.849 |
| D. Trussed Towers Height to centroid of Af (z) ∈ = Tower Cross Section Member Shape | Square (win F = qz C 15.0 ft 0.27 triangle flat | Cf = 1.28 G Cf Af = 22.1 Af Af = sf | | F = q _z (Base pressu Diagonal wind Round membe | $C_{f} =$ $G C_{f} A_{f} =$ $A_{f} =$ $F =$ $Kz =$ $Ire (qz) =$ $I factor =$ $r factor =$ $r factor =$ $Triangular C$ | 1.67 28.8 Af 10.0 sf 288 lbs 0.849 21.4 psf 1 1.000 cross Section |
| D. Trussed Towers Height to centroid of Af (z) ∈ = Tower Cross Section Member Shape | Square (win F = qz C 15.0 ft 0.27 triangle flat | Cf = 1.28 G Cf Af = 22.1 Af Af = sf | | F = q _z (Base pressu Diagonal wind Round membe | $C_{f} =$ $G C_{f} A_{f} =$ $A_{f} =$ $F =$ $Kz =$ $Ire (qz) =$ $I factor =$ $r factor =$ $\frac{riangular C}{C_{f}} =$ | 1.67 28.8 Af 10.0 sf 288 lbs 0.849 21.4 psf 1 1.000 cross Section 2.38 |
|). Trussed Towers Height to centroid of Af (z) ∈ = Tower Cross Section Member Shape | Square (win F = qz C 15.0 ft 0.27 triangle flat | Cf = 1.28 G Cf Af = 22.1 Af Af = sf F = 0 lbs | | F = q _z (Base pressu Diagonal wind Round membe <u>1</u> F = q _z (| $C_{f} =$ $G C_{f} A_{f} =$ $A_{f} =$ $F =$ $Kz =$ $Ire (qz) =$ $I factor =$ $r factor =$ $\frac{riangular C}{C_{f}} =$ $G C_{f} A_{f} =$ | 1.67 28.8 Af 10.0 sf 288 lbs 0.849 21.4 psf 1 1.000 <u>cross Section</u> 2.38 43.4 Af |
| D. Trussed Towers Height to centroid of Af (z) ∈ = Tower Cross Section Member Shape | Square (win F = qz C 15.0 ft 0.27 triangle flat | Cf = 1.28 G Cf Af = 22.1 Af Af = sf | | F = q _z (Base pressu Diagonal wind Round membe <u>1</u> F = q _z (| $C_{f} =$ $G C_{f} A_{f} =$ $A_{f} =$ $F =$ $Kz =$ $Ire (qz) =$ $I factor =$ $r factor =$ $\frac{riangular C}{C_{f}} =$ | 1.67 28.8 Af 10.0 sf 288 lbs 0.849 21.4 psf 1 1.000 cross Section 2.38 |

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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** = Ш Importance Factor | = 1.0 Thermal Factor Ct = 1.00 Ce = **Exposure Factor** 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 Near ground level surface balanced snow load = 104.0 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 = NOTE: Alternate spans of continuous beams Minimum Snow Load Pm = 20.0 psf shall be loaded with half the design roof snow Uniform Roof Design Snow Load = 80.1 psf 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

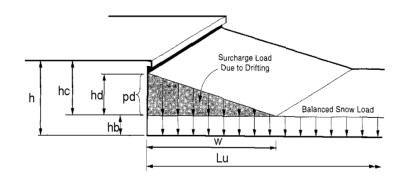
Unbalanced snow loads must be applied

| Ridge or Crown snow load = | 40 |
|----------------------------|-----|
| Vallev snow load = | 145 |

0.0 psf = 0.5*Pf 5.6 psf = 2*Pf/Ce 1.45 ft of snow 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, d | esign 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 |
| Windward Snow Drifts 2 - Aga | ainst walls, pai | rapets, 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, d | esign 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 |
| | | |



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

Nominal Snow Forces

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ASCE 7-16

Lu

hd

pd

hb

hc

h

Snow Loads - from adjacent building or roof:

| Roof slope Horiz. eave to ridge dist (W) Roof length parallel to ridge (L) Projection height (roof step) h Building separation s | = = = = | igher Roof 14.0 deg 60.0 ft 40.0 ft | Lower Roof 0.25 / 12 = 1 24.0 ft 4.5 ft 30.0 ft 0.0 ft | I.2 deg |
|--|--------------------|--|---|---------|
| Type of Roof | Sa | wtooth, etc. | Monoslope | |
| Ground Snow Load Pg | = | 104.0 psf | 104.0 psf | |
| Risk Category | = | II | 11 | |
| Importance Factor | = | 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 Unobstructed Slippery Surface | = | 80.1 psf no | 96.1 psf no | |
| Sloped-roof Factor Cs | = | 1.00 | 1.00 | |
| Balanced Snow Load Ps | = | 80.1 psf | 96.1 psf | |
| Rain on Snow Surcharge Angle Code Maximum Rain Surcharge Rain on Snow Surcharge Ps plus rain surcharge Minimum Snow Load Pm | = = = | 1.20 deg 5.0 psf 0.0 psf 80.1 psf 20.0 psf | 0.48 deg 5.0 psf 0.0 psf 96.1 psf 20.0 psf | |
| Uniform Roof Design Snow Load | = | 80.1 psf | 96.1 psf | |

=

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.

> Surcharge Load Due to Drifting

> > Balanced Snow Load

Leeward Snow Drifts - from adjacent higher roof

Building Official Minimum

| Upper roof length | lu = | 38.7 ft |
|----------------------|--------------|------------------|
| Snow density | γ = | 27.5 pcf |
| Balanced snow height | hb = | 3.49 ft |
| | hc = | 26.51 ft |
| hc/hb >0.2 = 7.6 | Therefore, d | lesign 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 |
| | | 405 0 () |

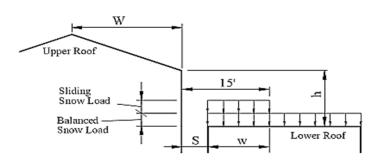
185.6 psf Leeward drift controls

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

| 1921.9 plf |
|-----------------|
| 128.1 psf |
| 8.15 ft |
| 128.1 psf |
| <u>96.1 psf</u> |
| 224.2 psf |
| 15.00 ft |
| |



| Anthem Struct | ural | | | IOB TITLE | BASECAMP S | STEAMBO | DAT TOWNHOM | ES |
|--|---|---|---|--|----------------|---------------------|--------------------------------|--------------|
| Boulder Golden Ste (303) 848-8497 (970) 3 | | | | JOB NO LATED BY ECKED BY | | | SHEET NO. DATE DATE | |
| Seismic Loads: | BC 2018 | | | | | | Strength Leve | I Forces |
| Risk Category : Importance Factor (I) : Site Class : | ІІ 1.00 С | | | | | | | |
| Ss (0.2 sec) = S1 (1.0 sec) = | 59.70 %g 10.30 %g | | | | | | | |
| Fa = 1.261 Fv = 1.500 | use 0.84 | Sms = Sm1 = | 0.500 0.155 | S _{DS} = S _{D1} = | 0.333 0.103 | - | ın Category = ın Category = | C B |
| Seismic Design Category = | с | | | | | | | |
| Redundancy Coefficient ρ = Number of Stories | 1.30 4 | | | | | | | |
| Structure Type: / Horizontal Struct Irregularities:N Vertical Structural Irregularities:1 | lo plan Irregul | arity | omo Soft Story | | | | | |
| DESIGN COEFFICIENTS AN Response Modification Coe Over-Strength F Deflection Amplification F | efficient (R) = Factor (Ωo) = Factor (Cd) = S _{DS} = | 6.5 2.5 4 0.333 | | | | | | |
| Seismic Load | S _{D1} = I Effect (E) = | 0.103 Eh +/-Ev = ρ | o C _E +/- 0.2S _{DS} D | = | = 1.3Qe +/- 0 | | Q _E = horizontal | seismic fora |
| Special Seismic Load I PERMITTED ANALYTICAL I | | | o G _E +/- 0.2S _{DS} D | = | = 2.5Qe +/- 0 | .067D | D = dead loac | |
| | | | | | | | | |
| Simplified Analysis | - Use Equivale | nt Lateral Forc | e Analysis | | | | | |
| Equivalent Lateral-Forc Building period | | Permittec 0.020 | | | | Cu = | 1.69 | |
| Approx fundamental User calculated fundamenta Long Period Transition I | l period (T) Period (TL) : | $C_T h_n^x =$ | sec 4 | x= 0.75 | Tmax | = CuTa = Use T = | | |
| | e coef. (Cs) = exceed Cs = ss than Cs = USE Cs = | S _{DS} I/R = Sd1 I /RT = 0.044SdsI = | 0.051 0.043 0.015 0.043 | | 043144 | | | |
| Madalogo | | | Design Base S | | | | | |
| Model & Seismic Respo | onse Analysis | - | Permitted (see | code for p | rocedure | | | |
| ALLOWABLE STORY DRIF | I | | | | | | | |

Structure Type: All other structures

Allowable story drift $\Delta a = 0.020$ hsx where hsx is the story height below level

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| Strength Level Forces | Seismic Desig | n Category (SDC)= C le = 1.00 | |
| <u>CONNECTIONS</u> Force to connect smaller portions of structure to re | mainder of structure | Sds = 0.333 | |

or $Fp = 0.05w_{p} =$ 0.05 W_n 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

0.044 W_p

Anchorage of Structural Walls to elements providing lateral support

Fp = not less than 0.2KaleWp Flexible diaphragm span Lf = Enter Lf to calculate Fp for flexible diaphragm Fp =0.4SdskaleWp = 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)

| Fp = 0.4SdsleWw = | 0.133 w _w | | |
|-------------------|----------------------|----------|---------------------|
| but not less than | 0.10 w _w | Use Fp = | 0.13 w _w |

Diaphragms

 $Fp = 0.133 Sdsw_{p} =$

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 ($\dot{R_p}$) = | 2.5 | z= | 50.0 feet | z/h = | 1.00 |
| Over-Strength Factor (Ωo) = | 2 | | | | |
| Fp = 0.4a _p SdsIpWp(1+2z/h)/Rp = | 0.400 Wp | | | | |
| not greater than Fp = 1.6SdslpWp = | 0.533 Wp | | - | 0.400.144 | |
| but not less than Fp = 0.3SdslpWp = | 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. C-atan (Im)

| Importance Factor (Ip) : 1.0 | | | | | |
|---|----------|-----|-----------|----------|------|
| Component Amplification Factor $(a_p) =$ | 1 | h= | 48.5 feet | | |
| Comp Response Modification Factor $(\dot{R_p}) =$ | 2.5 | z= | 50.0 feet | z/h = | 1.00 |
| Over-Strength Factor (Ωo) = | 2 | | | | |
| Fp = 0.4a _p SdsIpWp(1+2z/h)/Rp = | 0.160 Wp | | | | |
| not greater than Fp = 1.6SdslpWp = | 0.533 Wp | | | | |
| but not less than Fp = 0.3SdslpWp = | 0.100 Wp | use | e Fp = | 0.160 Wp | |

JOB TITLE BASECAMP STEAMBOAT TOWNHOMES

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| CHECKED BY | DATE |

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 |
| | | | 1 |
| | Actual Dead Load | 14.4 O | 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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| ltems | 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 |
| | | | | ļ |
| | | Actual Dead Load | 15.4 | 9.7 |
| | | Use this DL instead | <u>16.0</u> | 6 5.0 |
| | | Partitions | 15.0 | 0.0 |
| | | Live Load | 40.0 | 0.0 |
| | | Total Live Load | 55.0 | 0.0 |
| | | Total Load | 71.0 | 65.0 |

IBC alternate procedure

Floor Design Loads

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%.

| | | Smallest of: | |
|--------------------------------------|--|------------------------|-------------|
| | L=Lo(0.25+15/√K _{LL} A _T) | R= .08%(SF - 150) | |
| Unreduced design live load: Lo = | = 40 psf | R= 23.1(1+D/L) = | 32.3% |
| | | R= 40% member support | s 1 floor |
| Floor member & 1 floor cols K_{LL} | = 2 | R= 60% member support | s ≥2 floors |
| Tributary Area A _T | = 300 sf | R = | 12.0% |
| Reduced live load: L = | = 34.5 psf | Reduced live load: L = | 35.2 psf |
| Columns (2 or more floors) K_{LL} | = 4 | | |
| Tributary Area A _T | = 500 sf | R = | 28.0% |
| Reduced live load: L = | = 23.4 psf | Reduced live load: L = | 28.8 psf |

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| 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 |
| | | | | |
| | | | | |
| | Acti | ual Dead Load O | 9.2 O | 6.2 |
| | Use t | his DL instead 🔘 | 10.0 🖲 | 40.0 |

Wall Design Loads

Wall Design Loads

| Items | Description Mu | ultiple | 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 O | 18.8 | O 15.2 |
| Use this DL instead 20.0 40.0 | | | | 40.0 |

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CODE SUMMARY

| Code: | International Building Code 2018 |
|--|---|
| Live Loads: | |
| Roof 0 to 200 sf: 200 to 600 sf: over 600 sf: | 24 - 0.02Area, but not less than 12 psf |
| Typical Floor Partitions Lobbies & first floor corridors Corridors above first floor Balconies (1.5 times live load) | 40 psf 15 psf 100 psf 80 psf 60 psf |
| <u>Dead Loads:</u> Floor Roof | 16.0 psf 16.0 psf |
| Wind Design Data: Ultimate Design Wind Speed Nominal Design Wind Speed Risk Category Mean Roof Ht (h) Exposure Category Enclosure Classif. Internal pressure Coef. Directionality (Kd) | 115 mph 89.08 mph II 48.5 ft C Enclosed Building +/-0.18 0.85 |
| Roof Snow Loads: Design Uniform Roof Snow load Flat Roof Snow Load Balanced Snow Load Ground Snow Load Importance Factor Snow Exposure Factor Thermal Factor Sloped-roof Factor Drift Surcharge load Width of Snow Drift | $\begin{array}{rrrrrrrrrrrrrrrrrrrrrrrrrrrrrrrrrrrr$ |
| <u>Earthquake Design Data:</u> Risk Category Importance Factor Mapped spectral response acceleratio | $ \begin{array}{cccc} = & II \\ I = & 1.00 \\ S S = & 59.70 \\ S 1 = & 10.30 \end{array} $ |
| Site Class Spectral Response Coef. | = C Sds = 0.333 Sd1 = 0.103 |
| Seismic Design Category Basic Structural System Seismic Resisting System Design Base Shear Seismic Response Coef. | = C = Bearing Wall Systems = Light frame (wood) walls with structural wood shear panels V = 0.043W Cs = 0.043 |
| Response Modification Factor Analysis Procedure | R = 6.5 = Equivalent Lateral-Force Analysis |

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Anthem Structural

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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: Z | one 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 | | |

JOB TITLE BASECAMP STEAMBOAT TOWNHOMES

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

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

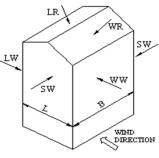
| Kh (case 2) = | 1.09 | h = | 48.5 ft | - GCpi = | +/-0.18 |
|-----------------------------------|----------|------------|----------|-------------|---------|
| Base pressure (q _h) = | 24.6 psf | ridge ht = | 48.5 ft | G = | 0.85 |
| Roof Angle (θ) = | 14.0 deg | L = | 40.0 ft | qi = qh | |
| Roof tributary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

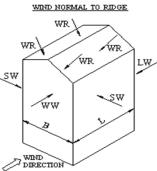
Ultimate Wind Surface Pressures (psf)

| Wind Normal to Ridge | | | | Wind | nd Parallel to Ridge | | | |
|----------------------|---|--|--|--|--|--|--|--|
| B/L = 3.00 | | h/L = 0.40 | | | L/B = 0.33 | | h/L = 1.21 | |
| Ср | $q_h GC_p$ | w/+q _i GC _{pi} | w/-q _h GCpi | Dist.* | Ср | $q_h GC_p$ | w/ +q _i GC _{pi} | w/ -q _h GC _{pi} |
| 0.80 | 16.7 | see tab | e below | | 0.80 | 16.7 | see tabl | e below |
| -0.25 | -5.2 | -9.6 | -0.8 | | -0.50 | -10.4 | -14.9 | -6.0 |
| -0.70 | -14.6 | -19.0 | -10.2 | | -0.70 | -14.6 | -19.0 | -10.2 |
| -0.49 | -10.1 | -14.6 | -5.7 | | Ind | cluded in w | indward roof | |
| -0.66 | -13.8 | -18.2 | -9.4 | 0 to h/2* | -1.04 | -21.7 | -26.1 | -17.3 |
| -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 |
| | B/L = Cp 0.80 -0.25 -0.70 -0.49 -0.66 | $\begin{array}{c c c c c c c c c c c c c c c c c c c $ | $\begin{array}{c c c c c c c c c c c c c c c c c c c $ | $\begin{array}{c c c c c c c c c c c c c c c c c c c $ | $\begin{array}{c c c c c c c c c c c c c c c c c c c $ | $\begin{array}{c c c c c c c c c c c c c c c c c c c $ | $\begin{array}{c c c c c c c c c c c c c c c c c c c $ | $B/L = 3.00$ $h/L = 0.40$ $L/B = 0.33$ $h/L =$ Cp q_hGC_p $w/+q_iGC_{pi}$ $w/-q_hGCpi$ Dist.* Cp q_hGC_p $w/+q_iGC_{pi}$ 0.80 16.7 see table below 0.80 16.7 see table -0.25 -5.2 -9.6 -0.8 -0.50 -10.4 -14.9 -0.70 -14.6 -19.0 -10.2 -0.70 -14.6 -19.0 -0.49 -10.1 -14.6 -5.7 Included in windward roof -0.66 -13.8 -18.2 -9.4 0 to $h/2^*$ -1.04 -21.7 -0.12 -2.6 -7.0 1.8 $> h/2^*$ -0.70 -14.6 -19.0 |

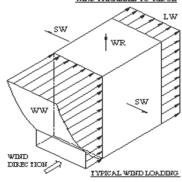
*Horizontal distance from windward edge

| | Windward | Combined WW + LW | | | | | | |
|----|----------|------------------|------|------------|------------------------------------|------------------------------------|----------|----------|
| _ | | | _ | V | Vindward Wa | all | Normal | Parallel |
| | Z | Kz | Kzt | $q_z GC_p$ | w/+q _i GC _{pi} | w/-q _h GC _{pi} | to Ridge | 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 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 | |
| Windwa | ard parapet: | 0.0 psf | (GCpn = · | |
| Leewa | ard parapet: | 0.0 psf | (GCpn = - | ·1.0) |

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



Proudly Serving Rural Routt County * City of Steamboat Springs * Town of Hayden * Town of Oak Creek * Town of Yampa * Routt County School Districts

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 <u>https://seismicmaps.org/</u> 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.

ROUTT County Regional Building Department

136 6th Street, Ste 201, Steamboat Springs, CO 80487 PH: 970-870-5566 Fax 970-870-5489 Email: Building@co.routt.co.us

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

ROUTT County Regional Building Department

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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 -15°F (-26°C)
 - o 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: $10^{\circ} \text{ F}(-12^{\circ} \text{ C})$
- Elevation: Varies Elevation by address can be found at: <u>https://elevation.maplogs.com/poi/routt_county_co_usa.12879.html</u>
- Altitude Correction: Varies
 - o 7,000' 0.77
 - o 8,000' 0.75
 - o 9,000' 0.72
 - o 10,000' 0.69
 - o 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 40-45°F (4.5-7.2°C)
- 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.

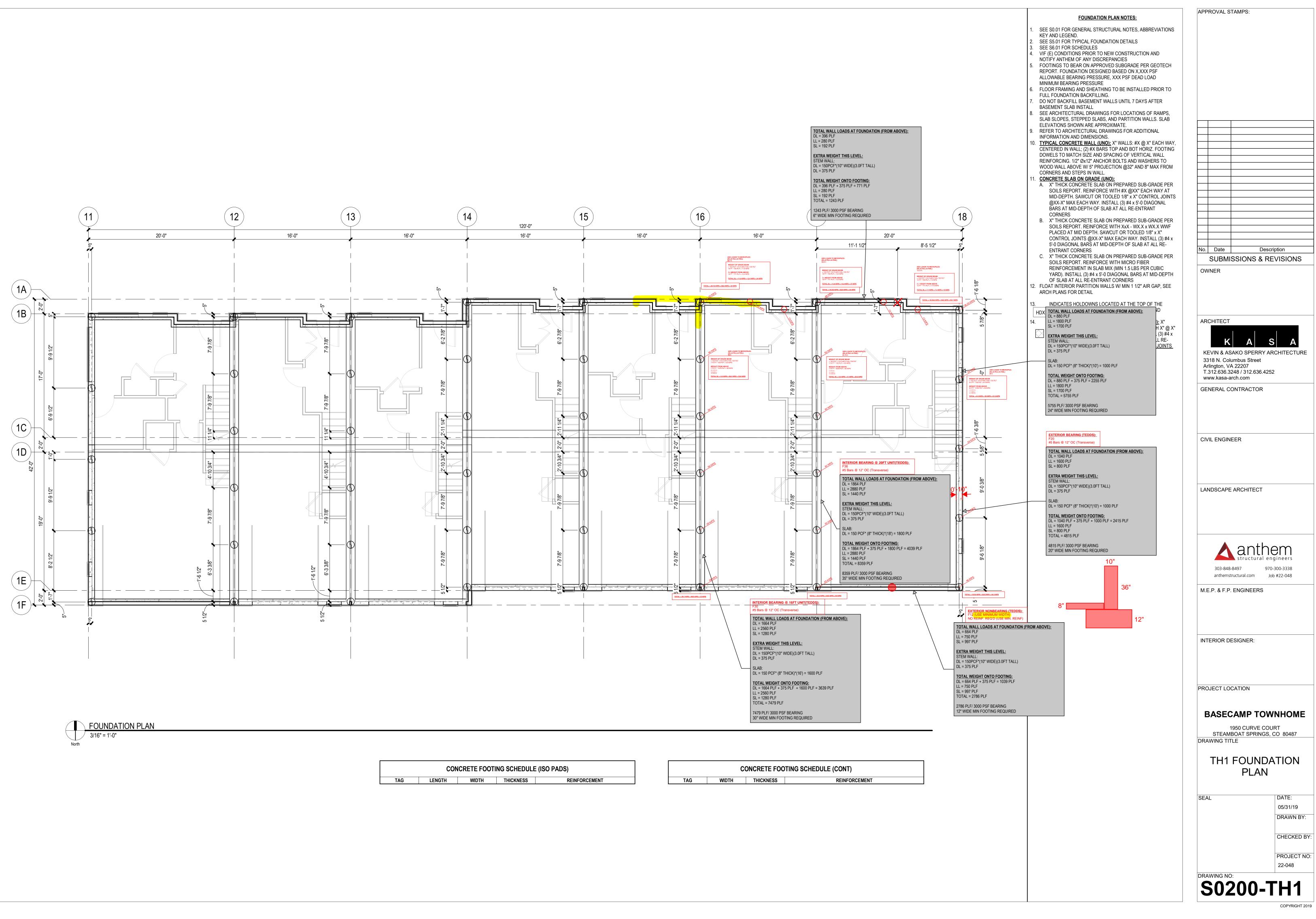
Sincerely,

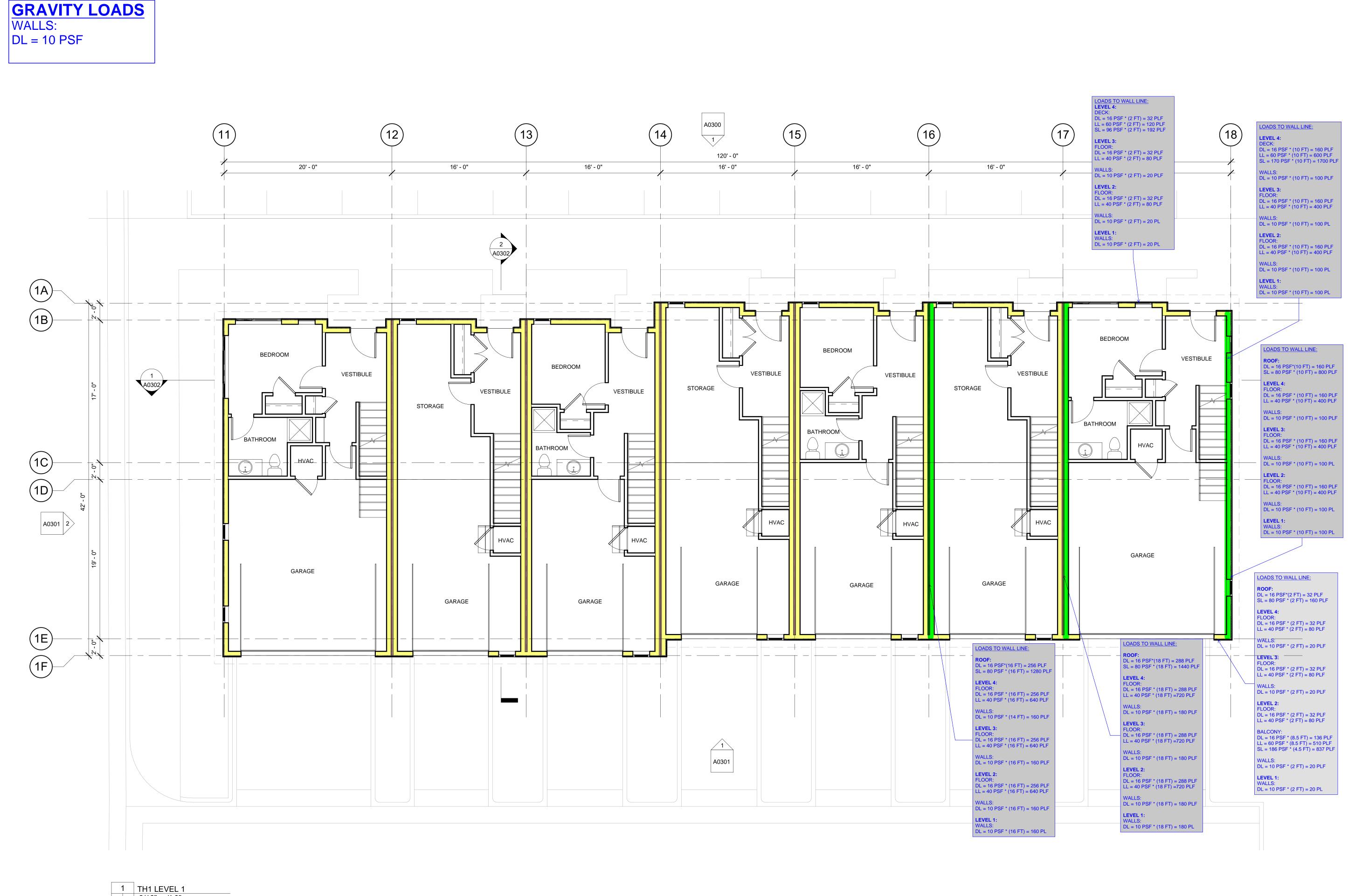
Tode lan

Todd Carr, Building Official Routt County Building Department

GRAVITY DESIGN

GRAVITY LOAD TRACE



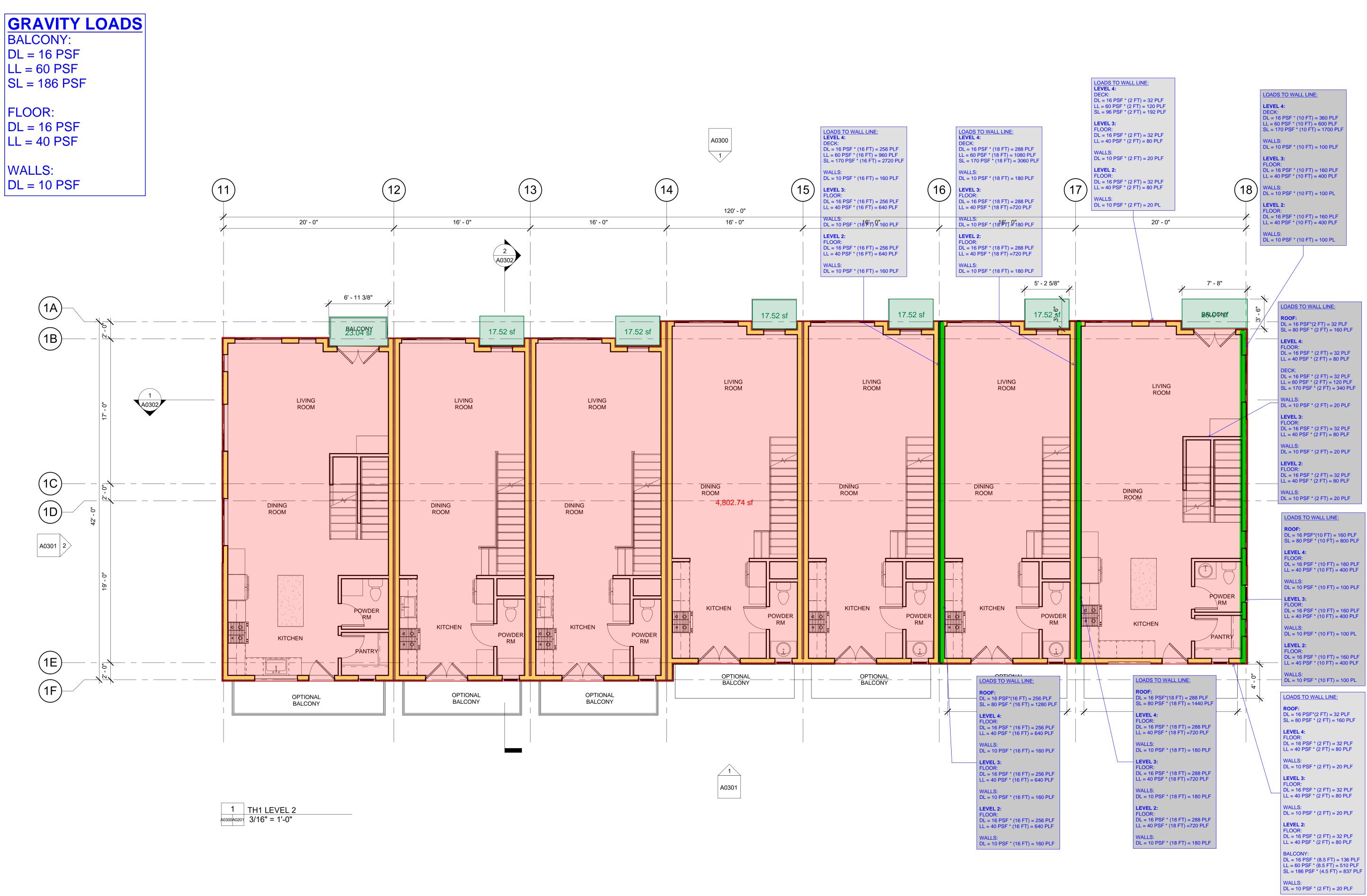


1 TH1 LEVEL 1 10300 10200 3/16" = 1'-0"



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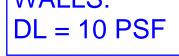
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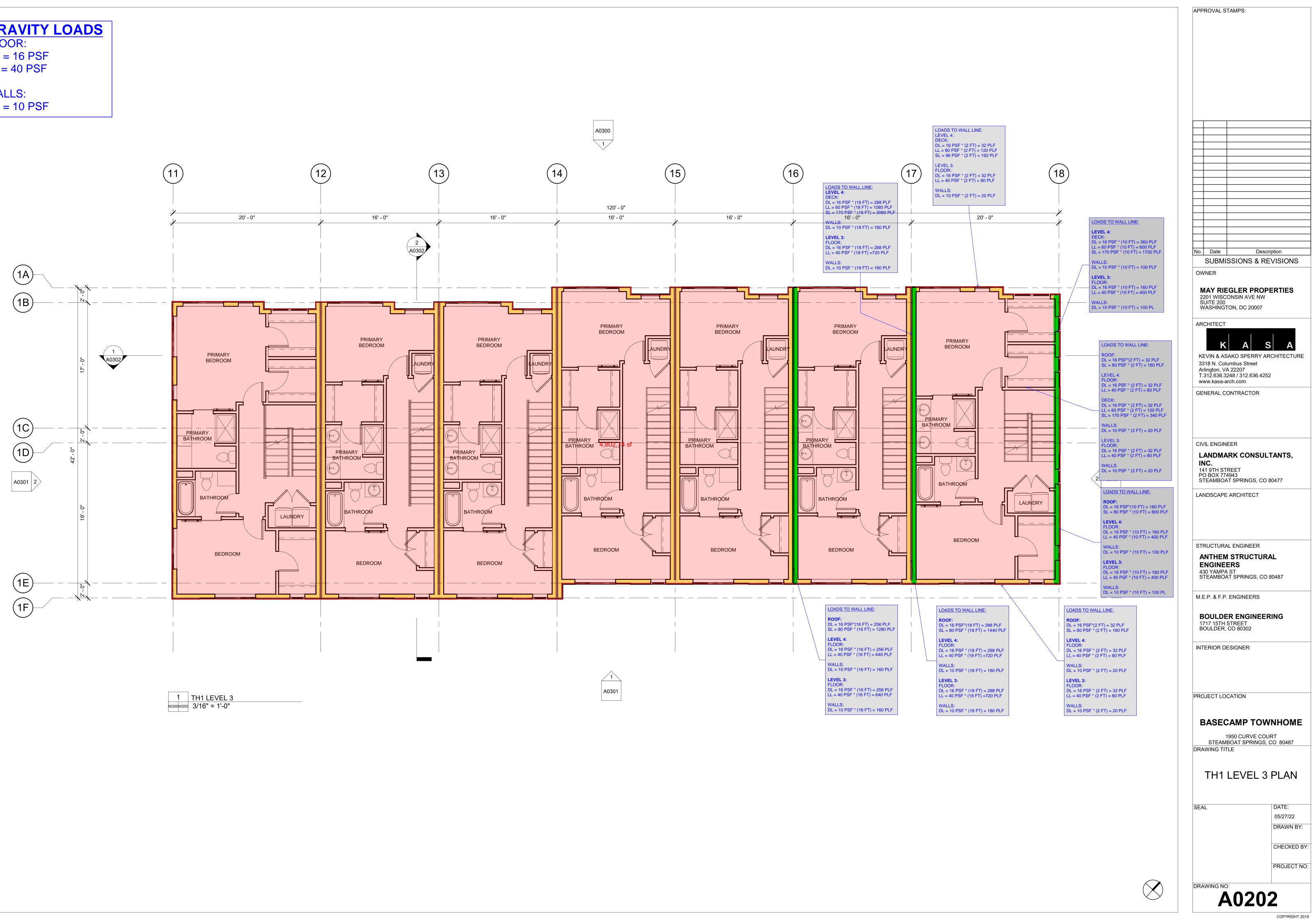


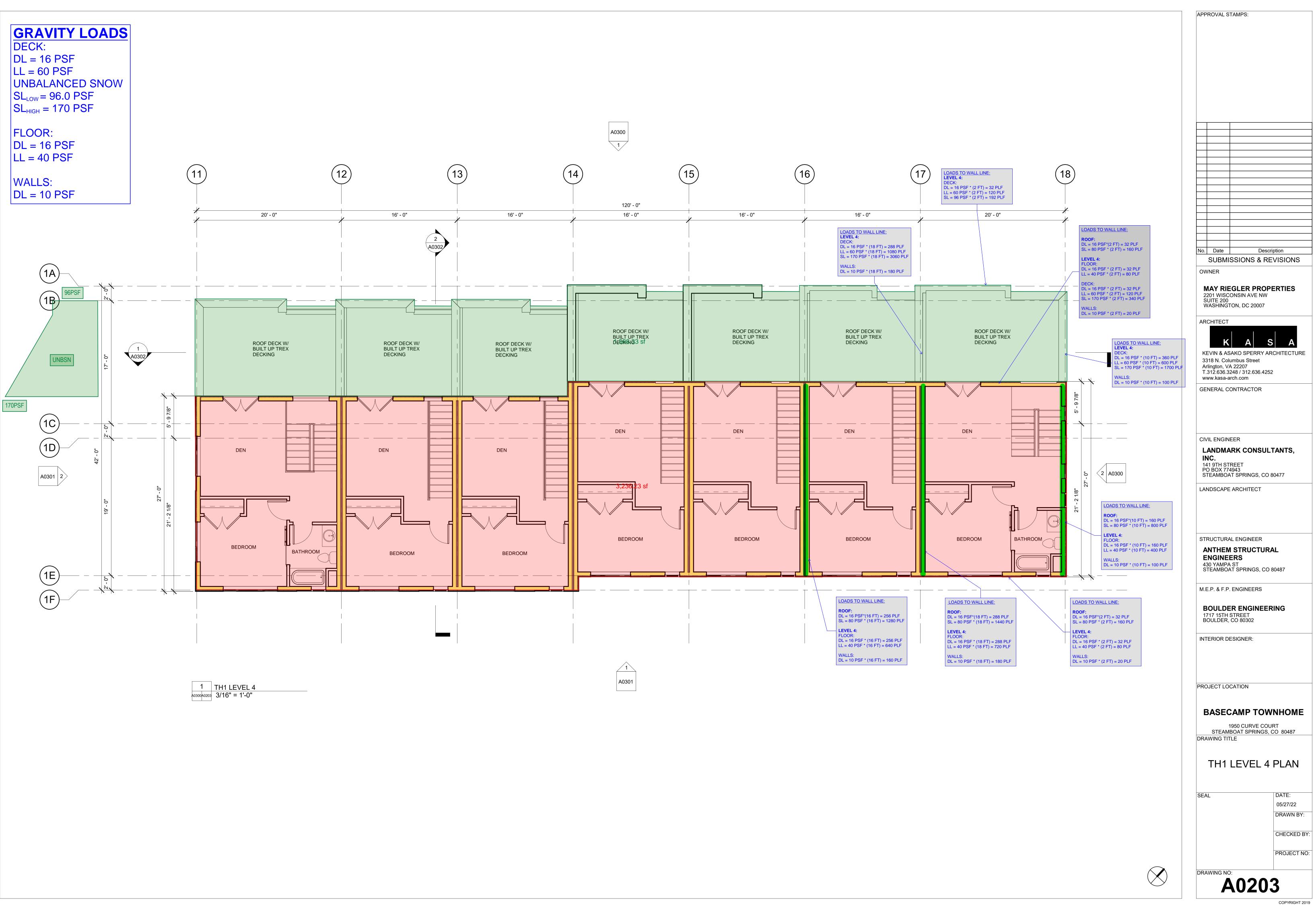
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GRAVITY LOADS FLOOR: DL = 16 PSF LL = 40 PSFWALLS:

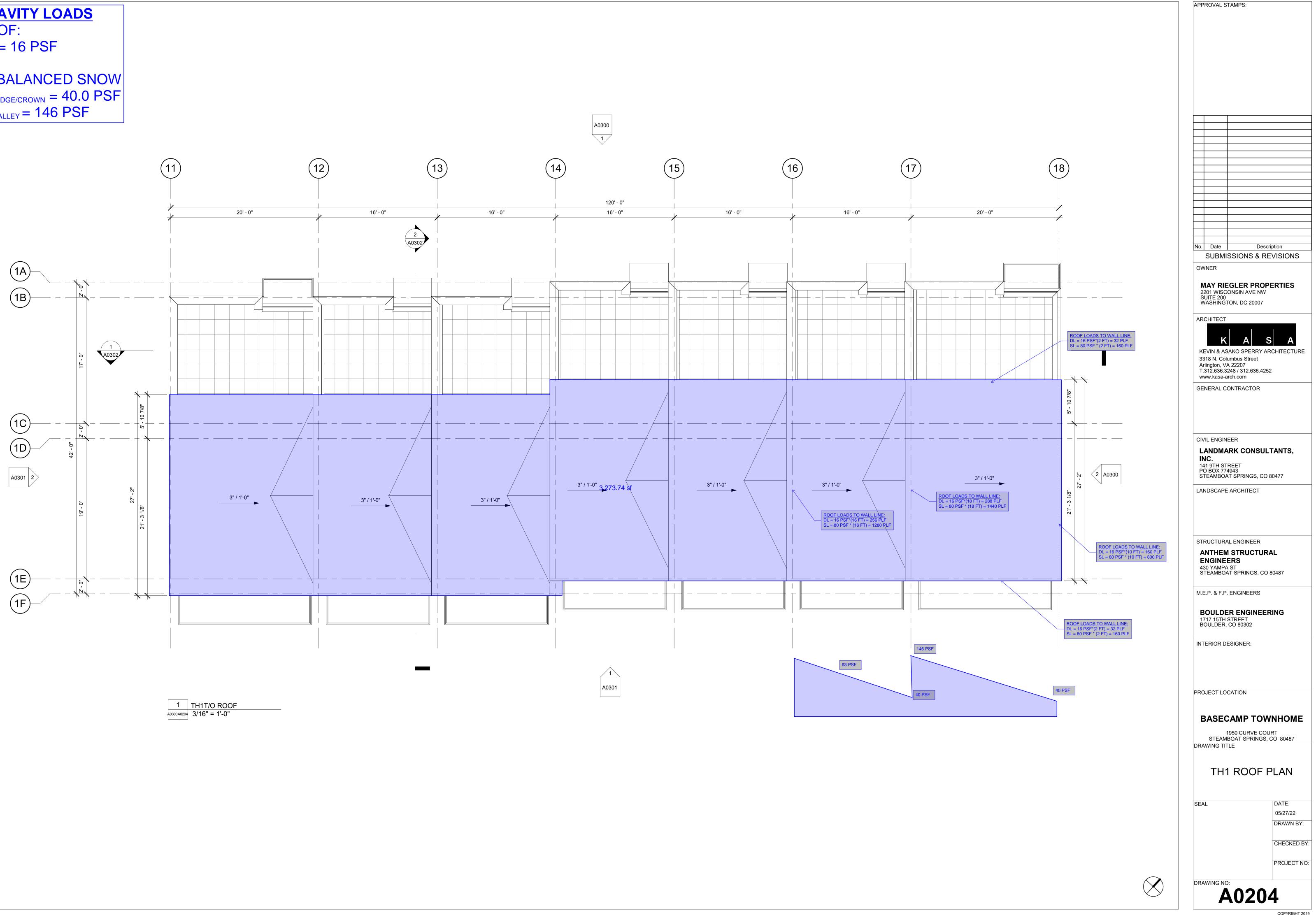


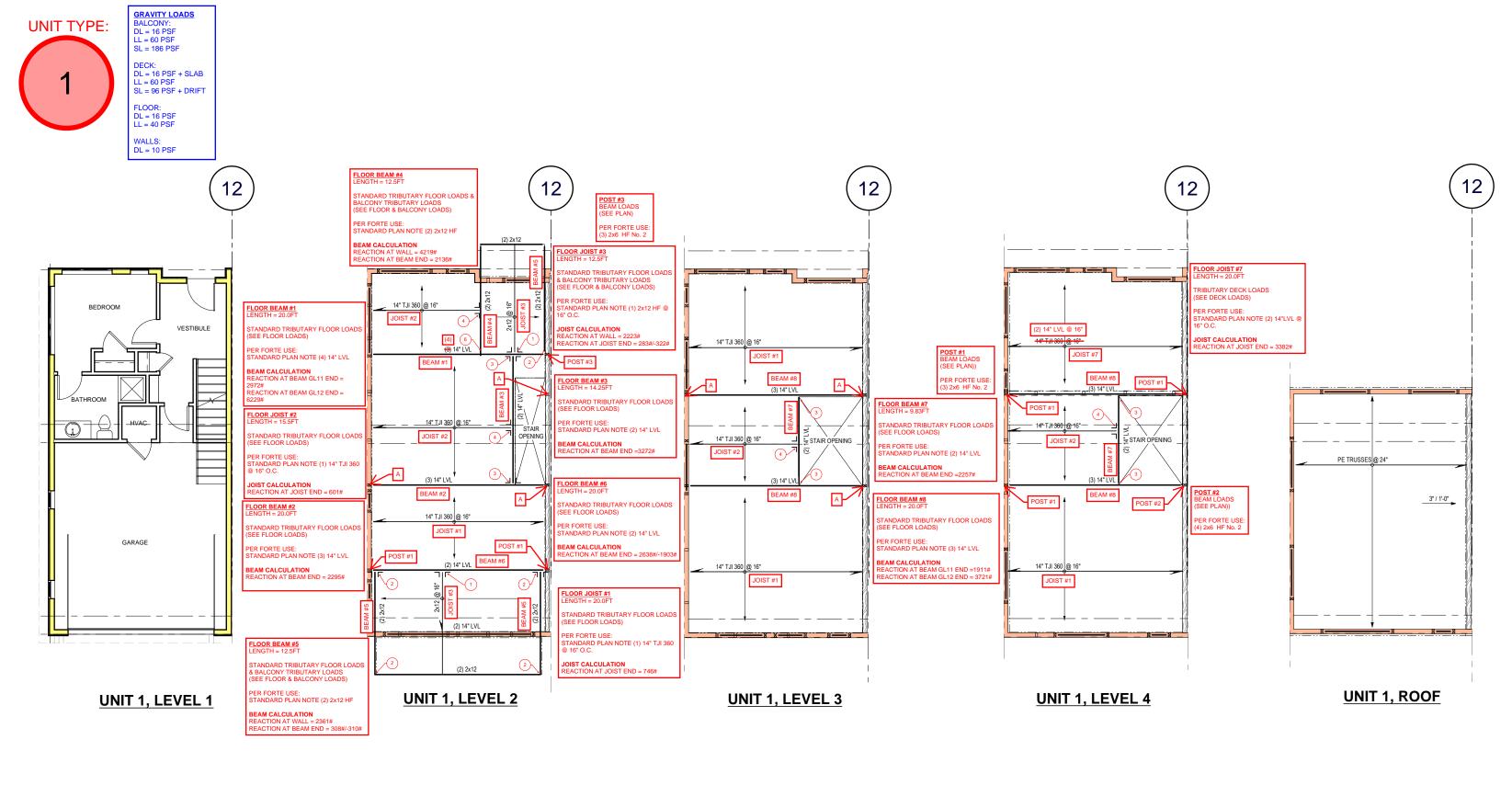


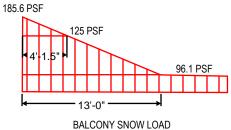


GRAVITY LOADS ROOF: DL = 16 PSF

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

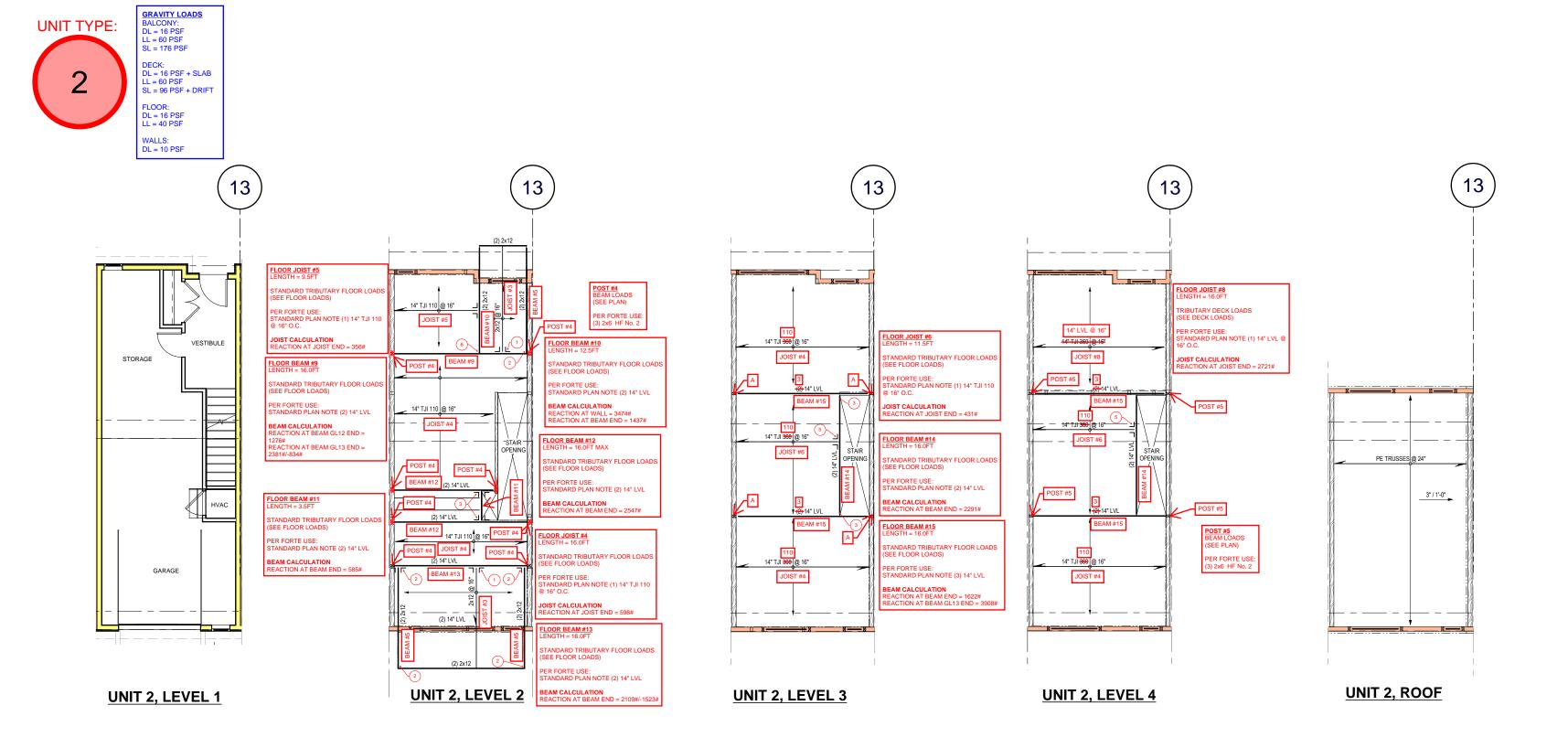


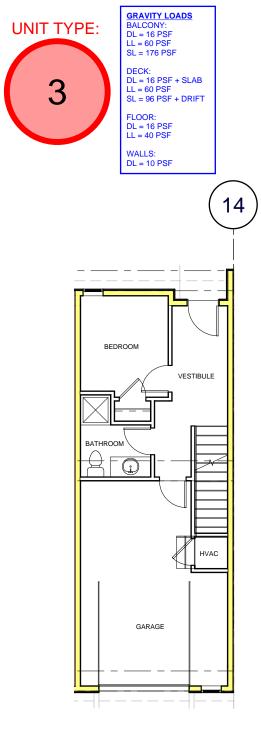




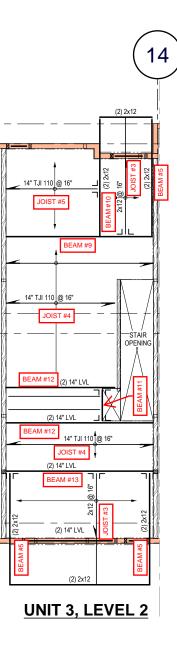
HANGER SCHEDULE 1. ALL HANGERS NOTED TO BE INSTALLED WITH NUMBER AND SIZE FASTENERS SPECIFIED BY MNFR. ANY SUBSTITUTIONS SHALL BE REVIEWED AND APPROVED BY ANTHEM 2. INSTALL HANGERS NOTED OD APPROVED FOUNDALENT.

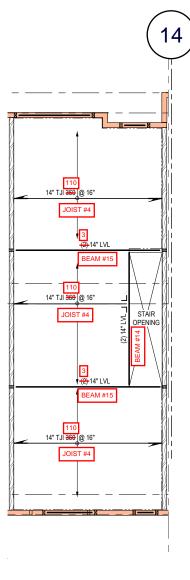
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| 1 | LUS28 | (6) 10d X 3" NAILS | (4) 10d X 1.5" NAILS | | | | | |
| 2 | LUS28-2 | (6) 10d X 3" NAILS | (4) 10d X 3" NAILS | | | | | |
| 3 | HHUS410 | (30) 10d X 3" NAILS | (10) 10d X 3" NAILS | | | | | |
| 4 | ISU2.37/14 | (12) 10d X 3" NAILS | -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



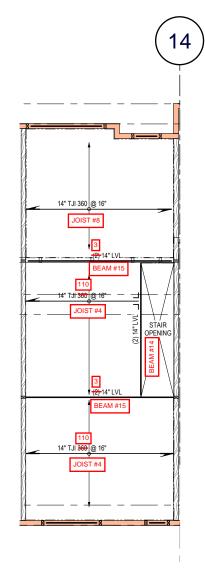


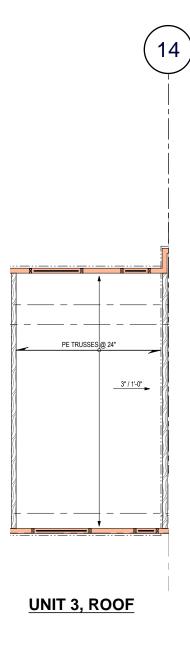




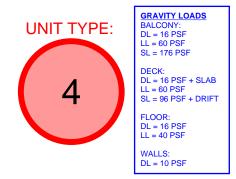


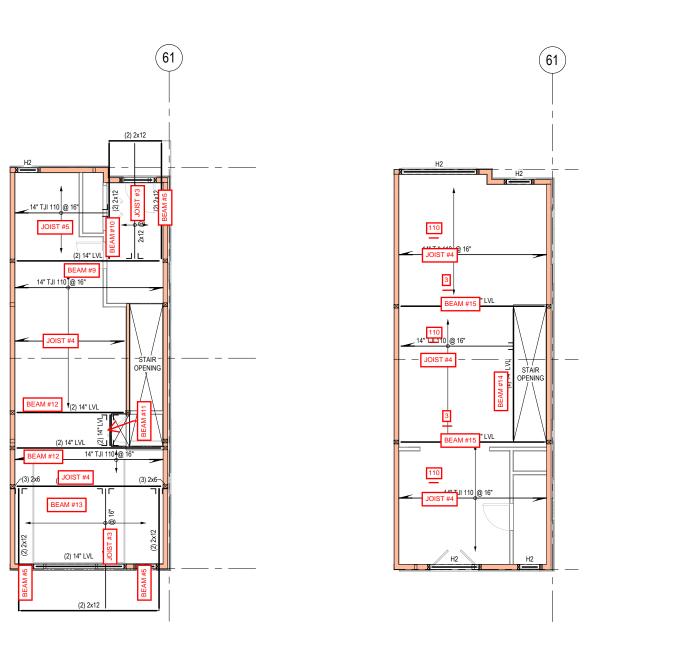
UNIT 3, LEVEL 3





UNIT 3, LEVEL 4





<u>UNIT 4, LEVEL 1</u>

<u>UNIT 4, LEVEL 2</u>

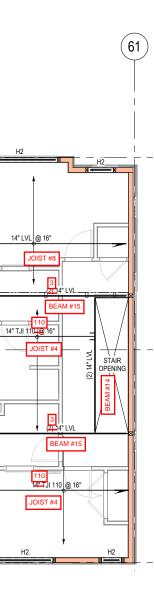
UNIT 4, LEVEL 3

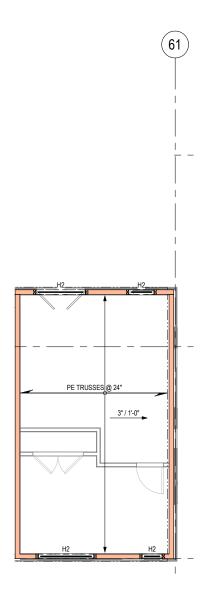
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H2

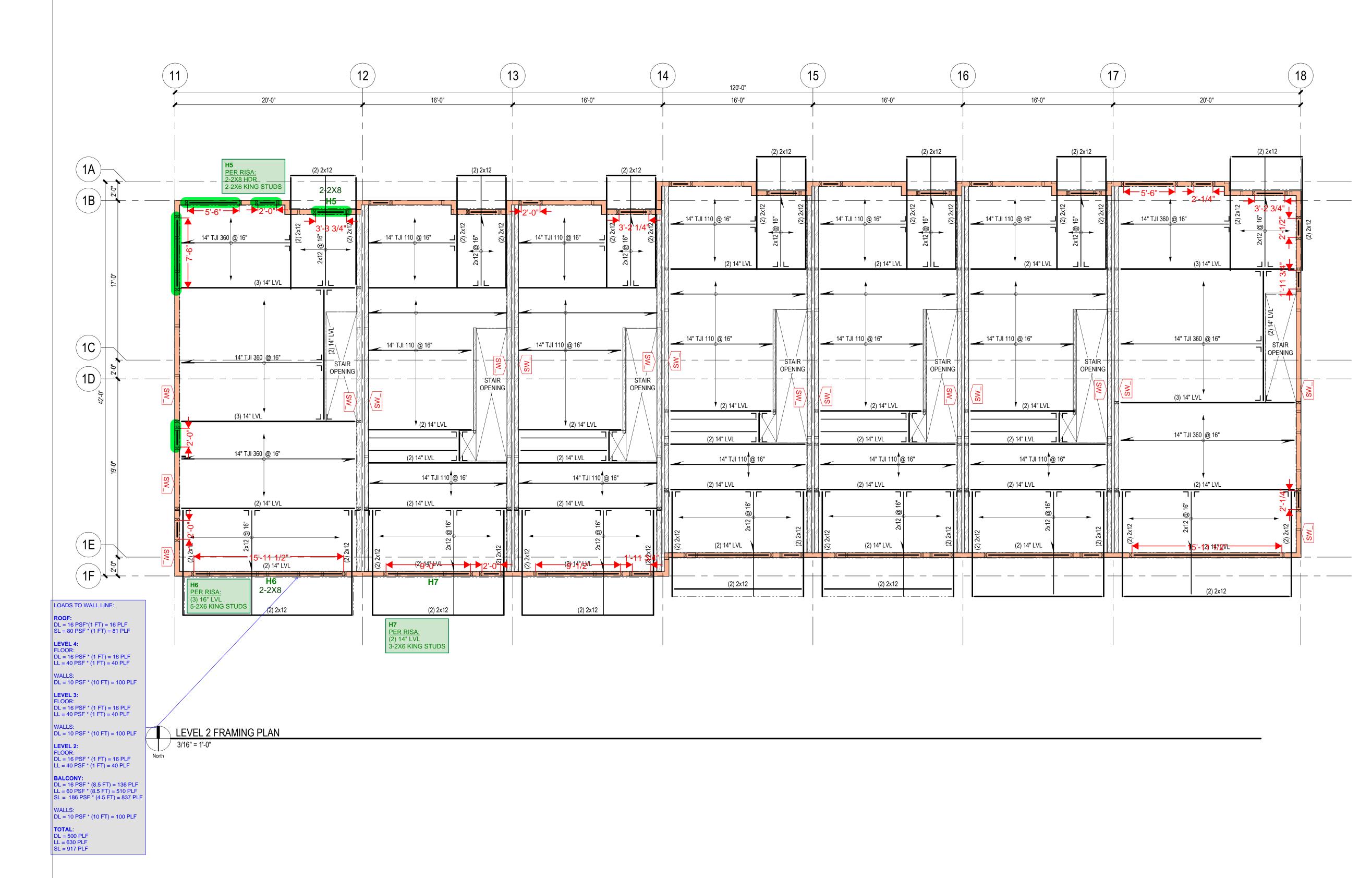
H2

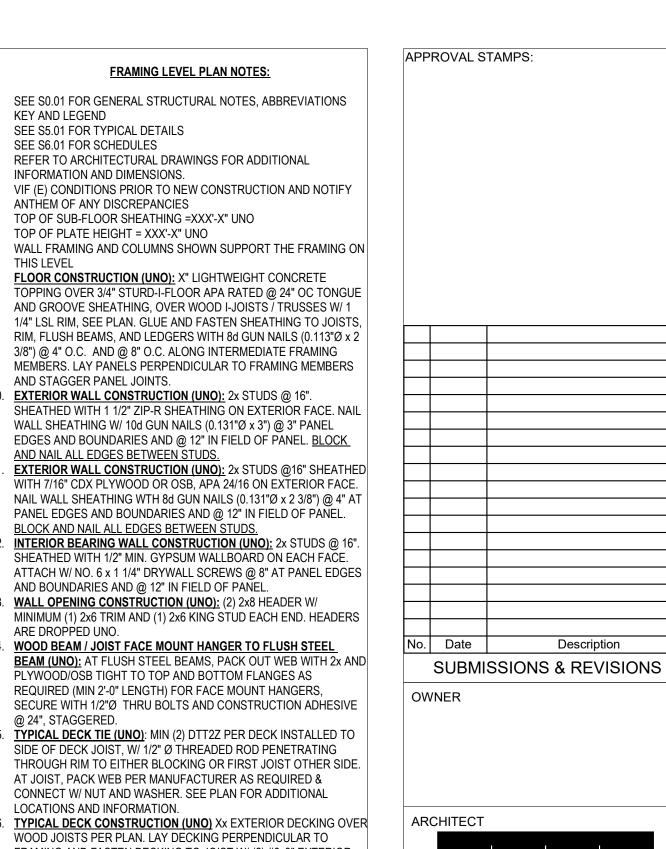
14" LVL @ 16





<u>UNIT 4, ROOF</u>





LOCATIONS AND INFORMATION. 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 /

INDICATES HOLDOWN THROUGH LEVEL SHOWN, SEE SX.XX. CONTRACTOR TO VERIFY LOCATIONS AND LAYOUT WITH

INDICATES SHEAR WALL TO BE SHEATHED ON SIDE INDICATED SWX BY ARROW (UNO) WITH SHEATHING PER SHEAR WALL SCHEDULE. SEE S6.01

LEVEL 2 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

(X) DESCRIPTION

| | LANDSCAPE ARCHITECT | | |
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| | | | |
| | structural engineers | | |

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

Description

KEVIN & ASAKO SPERRY ARCHITECTURE

3318 N. Columbus Street

GENERAL CONTRACTOR

T.312.636.3248 / 312.636.4252

Arlington, VA 22207

www.kasa-arch.com

CIVIL ENGINEER

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**

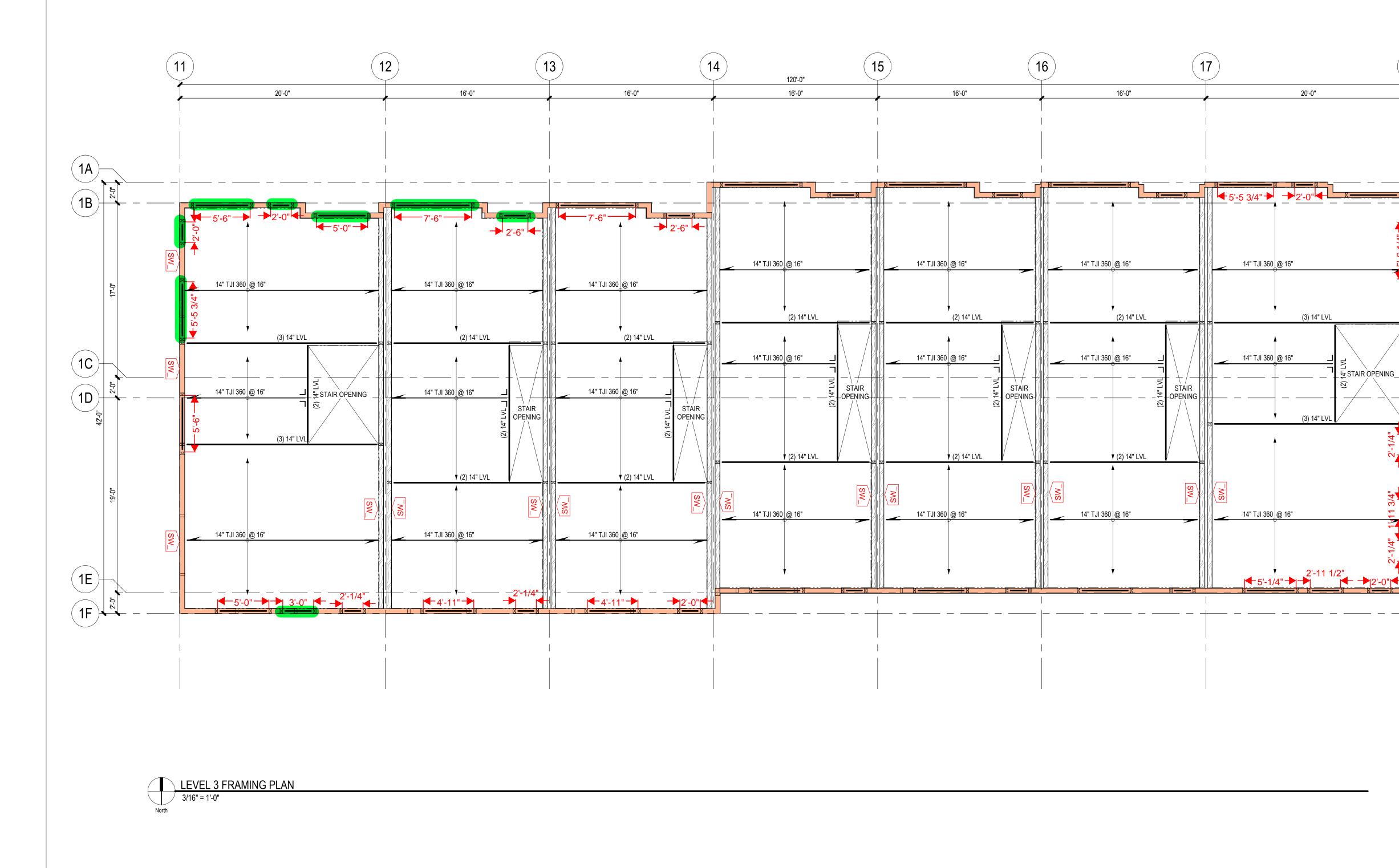
KEY AND LEGEND

- REFER TO ARCHITECTURAL DRAWINGS FOR ADDITIONAL
- INFORMATION AND DIMENSIONS.

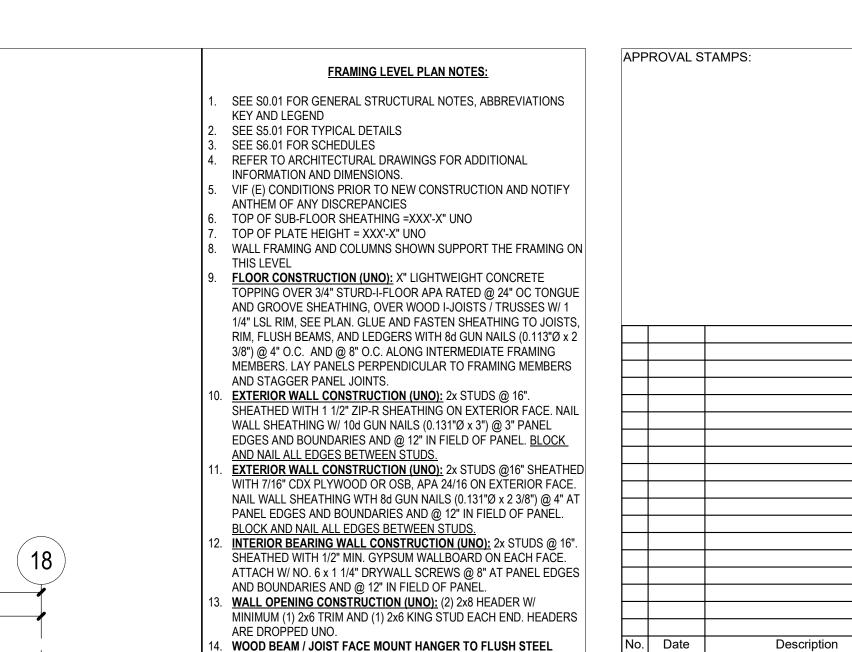
- TOP OF PLATE HEIGHT = XXX'-X" UNO
- WALL FRAMING AND COLUMNS SHOWN SUPPORT THE FRAMING ON
- THIS LEVEL FLOOR CONSTRUCTION (UNO): X" LIGHTWEIGHT CONCRETE TOPPING OVER 3/4" STURD-I-FLOOR APA RATED @ 24" OC TONGUE AND GROOVE SHEATHING, OVER WOOD I-JOISTS / TRUSSES W/ 1 1/4" LSL RIM, SEE PLAN. GLUE AND 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
- . 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.
- 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.
- 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.
- WALL OPENING CONSTRUCTION (UNO): (2) 2x8 HEADER W/ MINIMUM (1) 2x6 TRIM AND (1) 2x6 KING STUD EACH END. HEADERS
- 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
- 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
- TYPICAL DECK CONSTRUCTION (UNO) XX EXTERIOR DECKING OVER BEAMS.

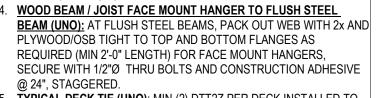
HDX FRAMING ABOVE

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5. <u>TYPICAL DECK TIE (UNO)</u>: 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. <u>TYPICAL DECK CONSTRUCTION (UNO)</u> 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.

7. INDICATES HOLDOWN THROUGH LEVEL SHOWN, SEE SX.XX. CONTRACTOR TO VERIFY LOCATIONS AND LAYOUT WITH HDX FRAMING ABOVE

18. 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

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

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| 303-848-8497 | 970-300-3338 |
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| anthemstructural.com | Job #22-048 |
| M.E.P. & F.P. ENGINEER | S |
| INTERIOR DESIGNER: | |
| PROJECT LOCATION | |
| BASECAMP T | OWNHOME |
| 1950 CURVE STEAMBOAT SPRII DRAWING TITLE | |
| LEVEL 3 F PLA | |

A anthem structural engineers

SUBMISSIONS & REVISIONS

KEVIN & ASAKO SPERRY ARCHITECTURE

3318 N. Columbus Street

GENERAL CONTRACTOR

T.312.636.3248 / 312.636.4252

Arlington, VA 22207

www.kasa-arch.com

CIVIL ENGINEER

LANDSCAPE ARCHITECT

OWNER

ARCHITECT

SEAL

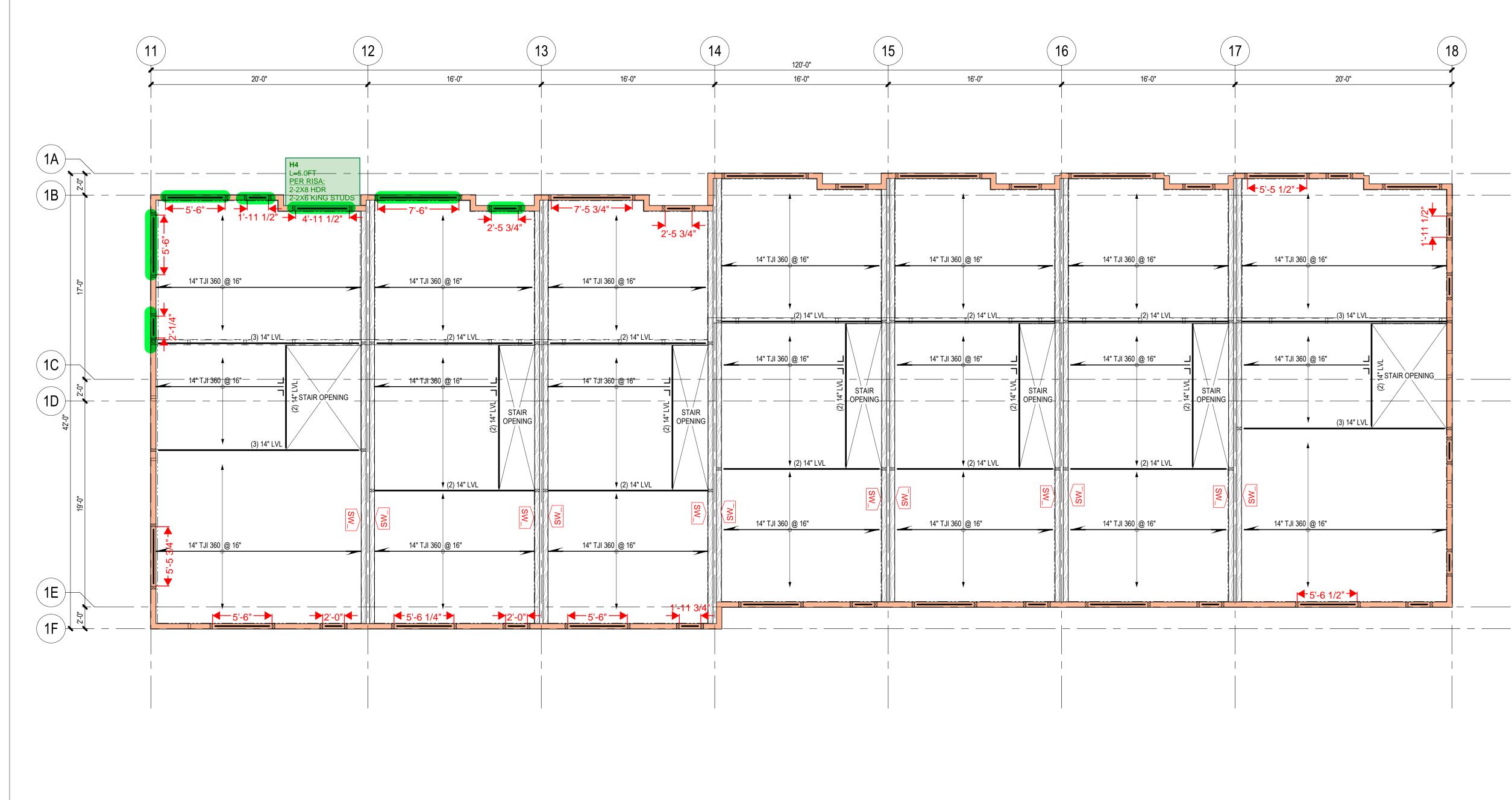
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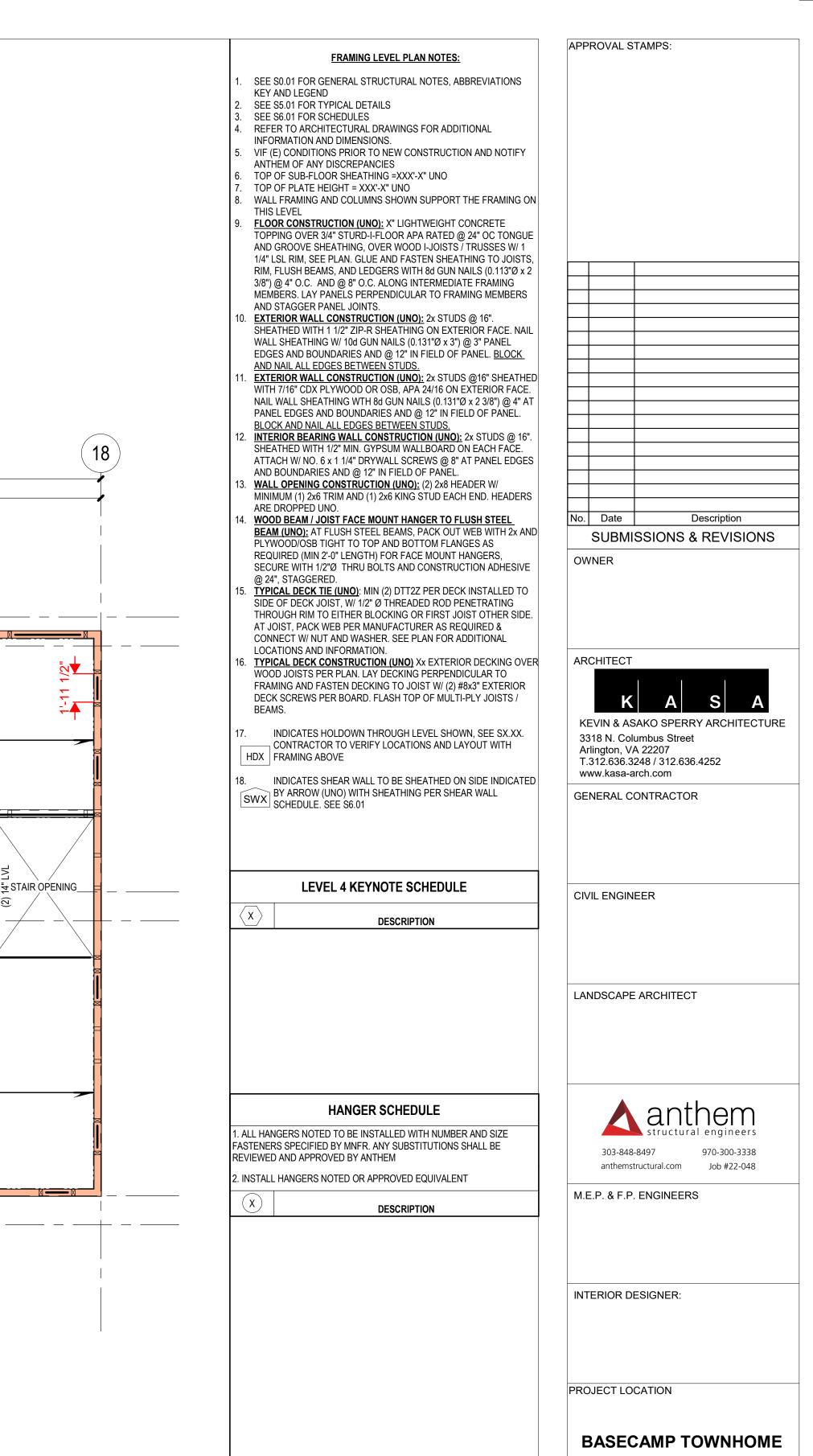
DRAWING NO: **S1.04**

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LEVEL 4 FRAMING PLAN 3/16" = 1'-0"

3/2022 2-05-29 P



| STEAMBOAT SPRINGS, CO 80487 |
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| LEVEL 4 FRAMING |
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1950 CURVE COURT

PLAN

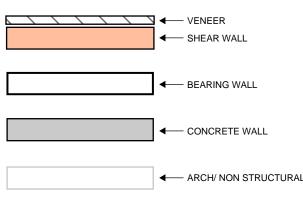
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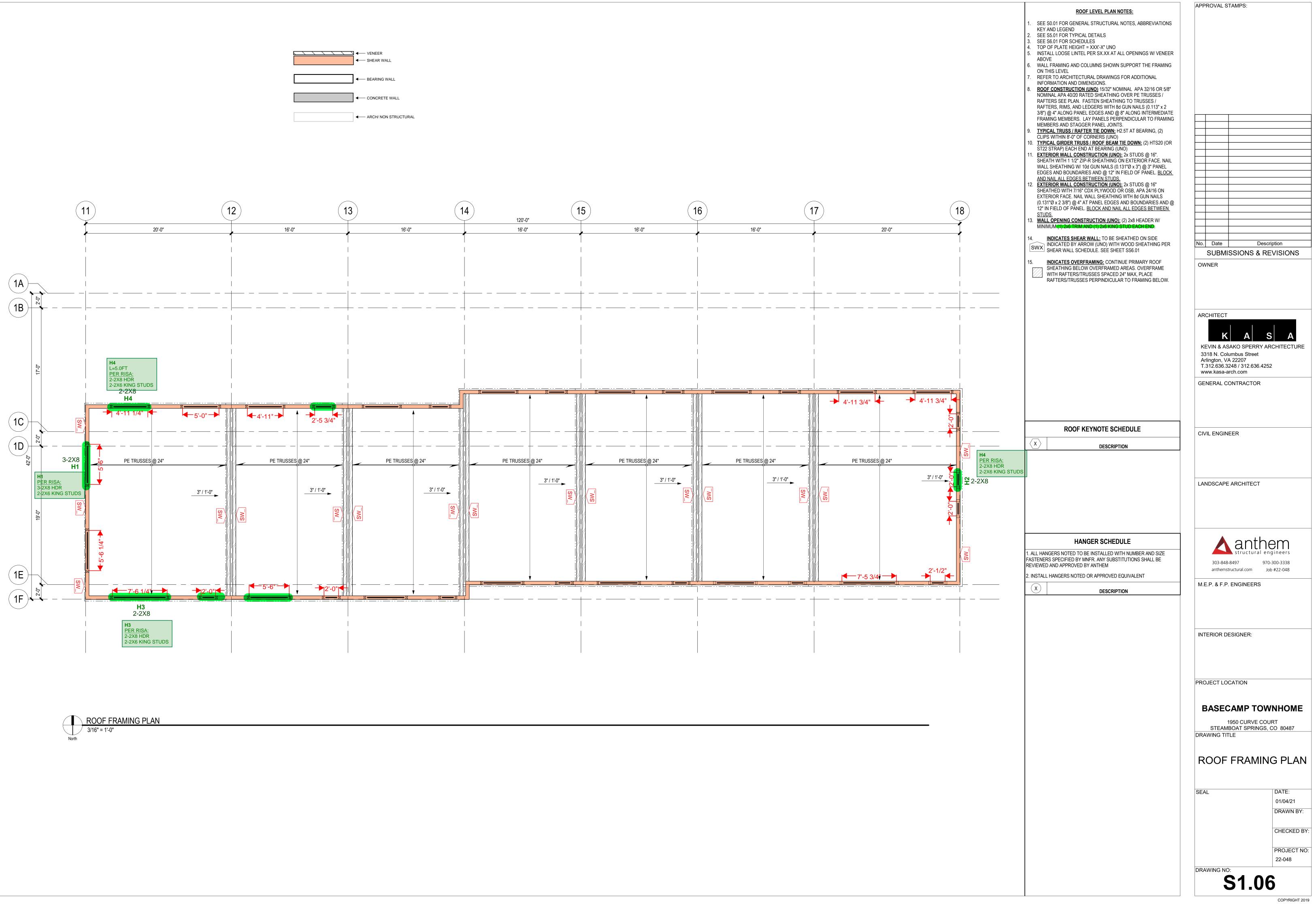
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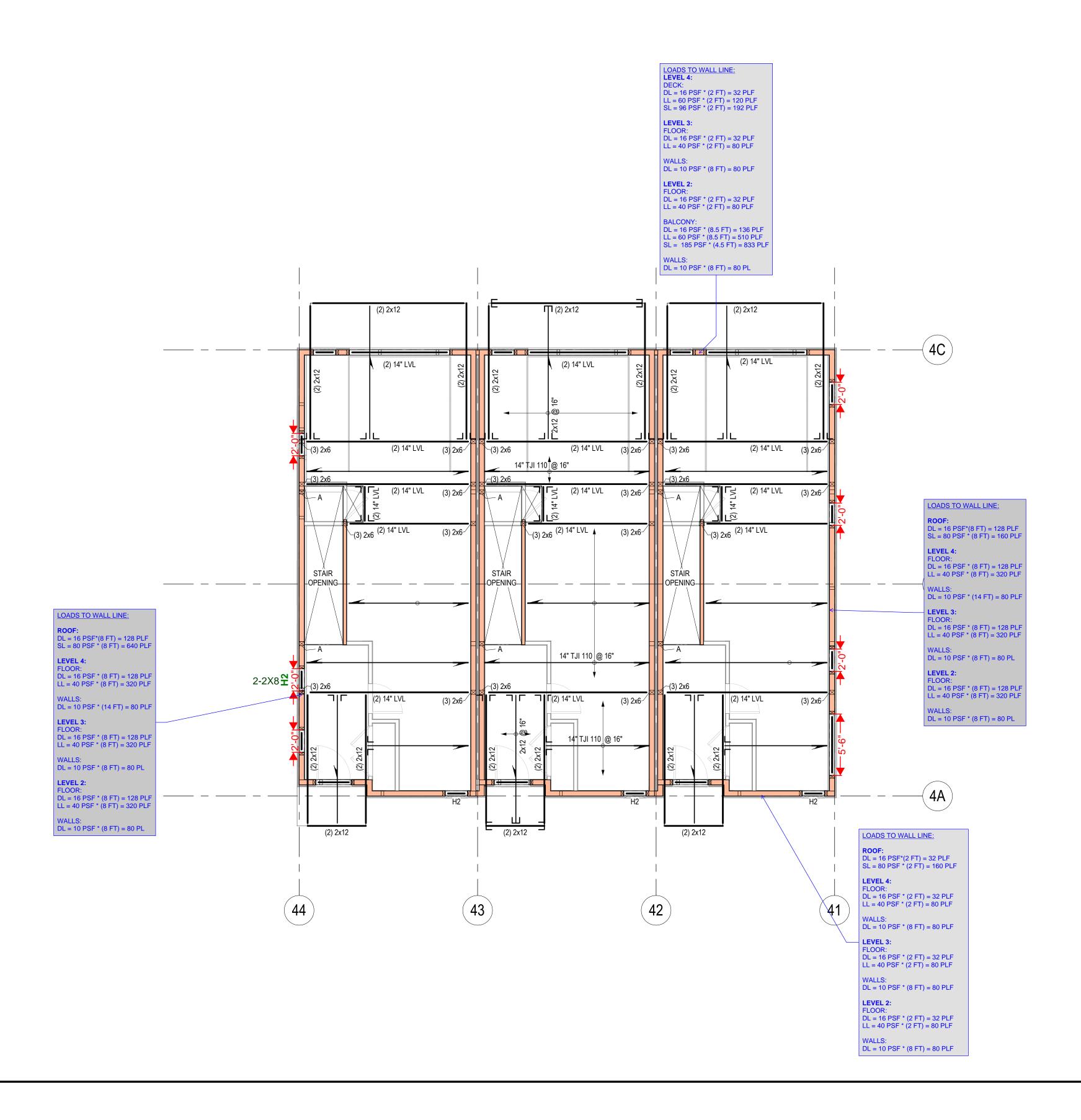
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FRAMING LEVEL PLAN NOTES:

| SEE S0.01 FOR | GENERAL S | TRUCTURAL | NOTES, | ABBREVIATIONS |
|---------------|-----------|-----------|--------|---------------|

KEY AND LEGEND

- 2. SEE S5.01 FOR TYPICAL DETAILS 3. SEE S6.01 FOR SCHEDULES
- 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
- TOP OF PLATE HEIGHT = XXX'-X" UNO
 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. <u>BLOCK</u> <u>AND NAIL ALL EDGES BETWEEN STUDS.</u>
- 11. EXTERIOR WALL CONSTRUCTION (UNO): 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. <u>WALL OPENING CONSTRUCTION (UNO):</u> (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.
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- 16. <u>TYPICAL DECK CONSTRUCTION (UNO)</u> 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.
- 7. INDICATES HOLDOWN THROUGH LEVEL SHOWN, SEE SX.XX. CONTRACTOR TO VERIFY LOCATIONS AND LAYOUT WITH HDX FRAMING ABOVE

18. 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

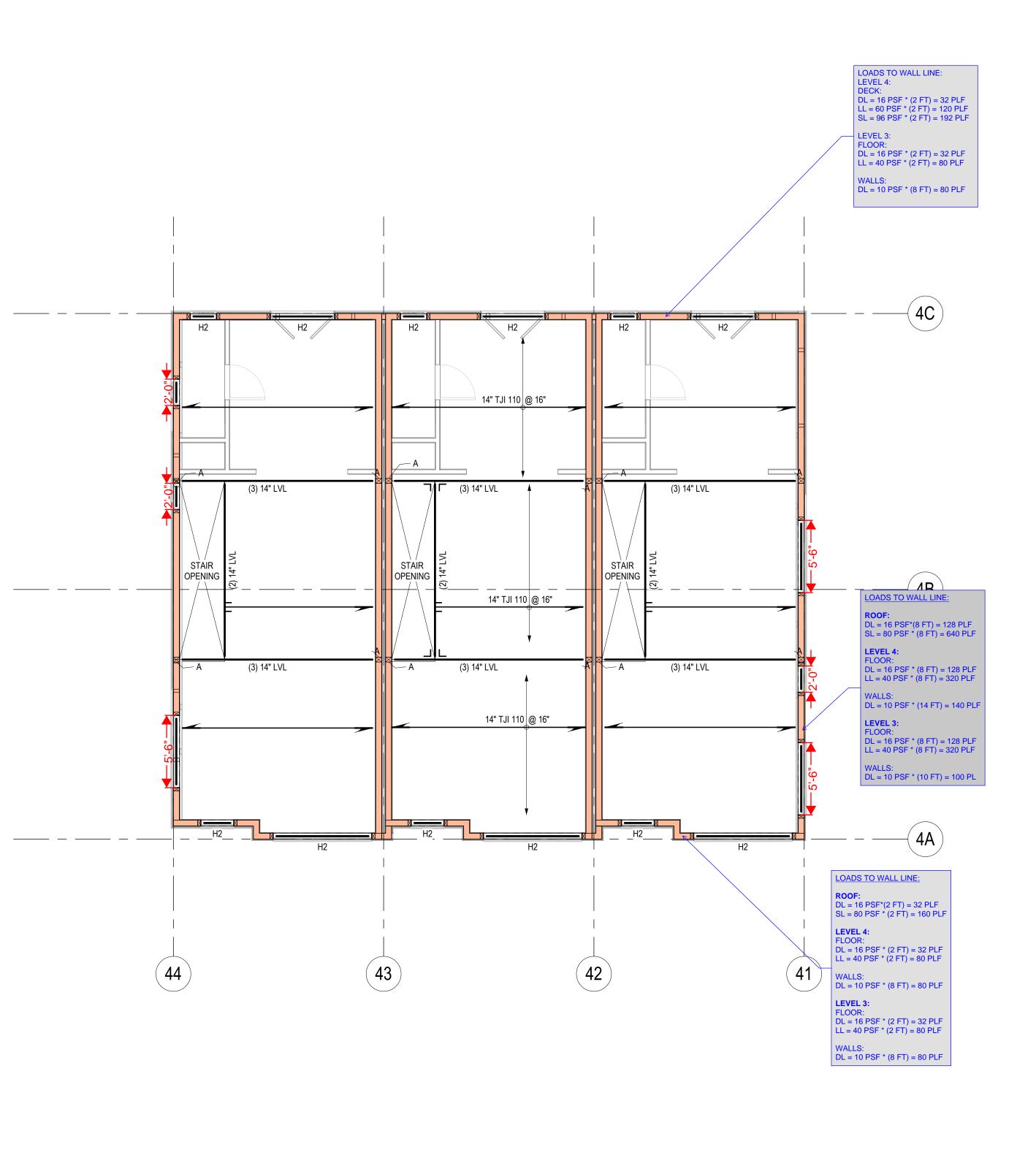
| | DESCRIPTION | Face Fasteners | Joist Fasteners |
|---|-------------|-------------------------|---------------------|
| 1 | LUS28 | (6) 10d x 3" NAILS | (4) 10d x 3" NAILS |
| 2 | LUS28-2 | (6) 10d x 3" NAILS | (4) 10d x 3" NAILS |
| 3 | HHUS410 | (30) 10d x 3" NAILS | (10) 10d x 3" NAILS |
| 4 | 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 |
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DRAWING NO:

S1.21

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

FRAMING LEVEL PLAN NOTES:

- SEE S0.01 FOR GENERAL STRUCTURAL NOTES, ABBREVIATIONS
- KEY AND LEGEND
- SEE S5.01 FOR TYPICAL DETAILS
 SEE S6.01 FOR SCHEDULES
- . REFER TO ARCHITECTURAL DRAWINGS FOR ADDITIONAL INFORMATION AND DIMENSIONS.
- VIF (E) CONDITIONS PRIOR TO NEW CONSTRUCTION AND NOTIFY
- ANTHEM OF ANY DISCREPANCIES
- 6. TOP OF SUB-FLOOR SHEATHING =XXX'-X" UNO
- TOP OF PLATE HEIGHT = XXX'-X" UNO
 WALL FRAMING AND COLUMNS SHOWN SUPPORT THE FRAMING ON
- WALL FRAMING AND COLOMING STROWN SOFFORT THE FRAMING ON THIS LEVEL
 <u>FLOOR CONSTRUCTION (UNO):</u> 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.
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 EXTERIOR WALL CONSTRUCTION (UNO): 2x STUDS @16" SHEATHED
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- 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. <u>TYPICAL DECK TIE (UNO)</u>: 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. <u>TYPICAL DECK CONSTRUCTION (UNO)</u> 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.
- 7. INDICATES HOLDOWN THROUGH LEVEL SHOWN, SEE SX.XX. CONTRACTOR TO VERIFY LOCATIONS AND LAYOUT WITH FRAMING ABOVE

18. 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

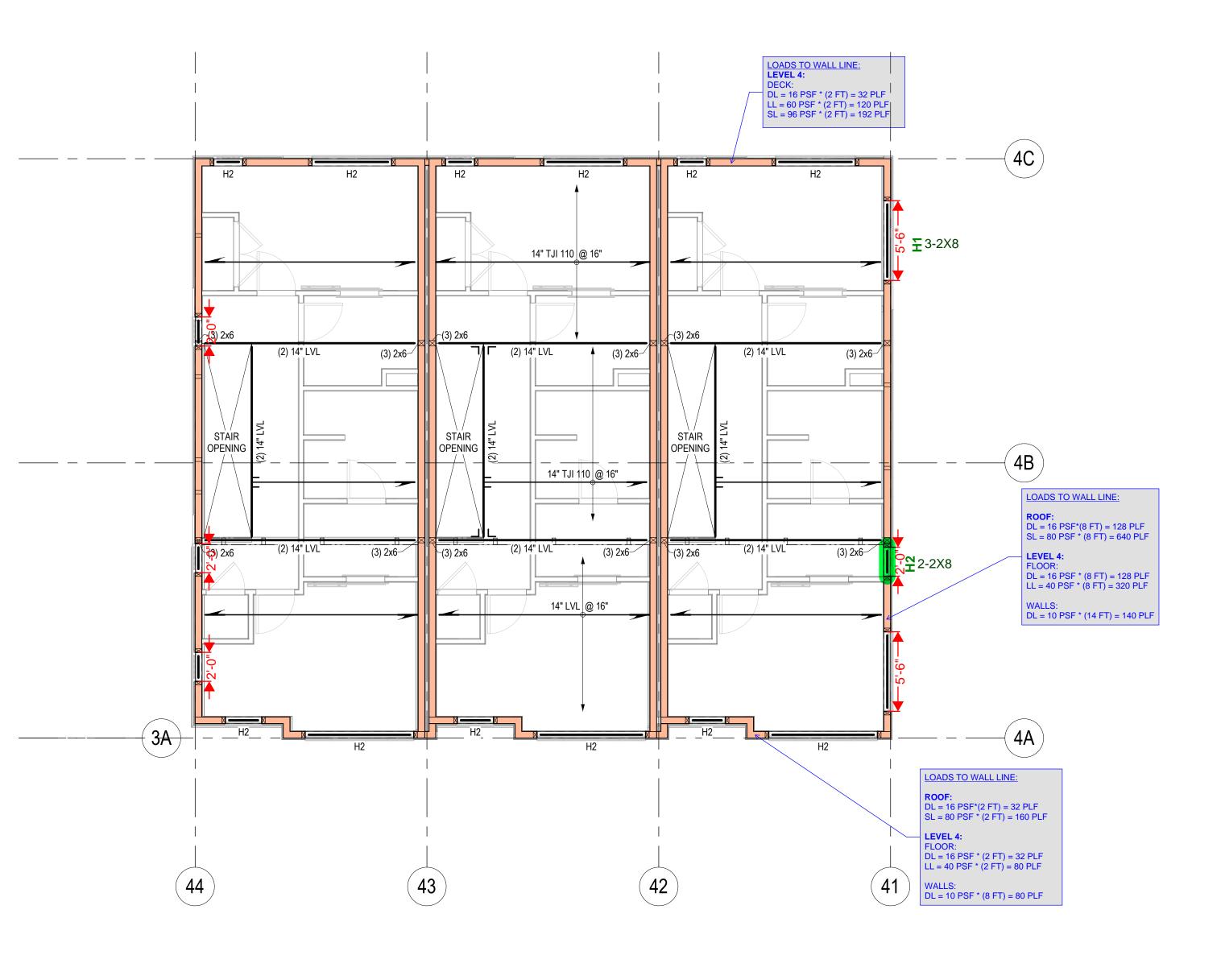
2. INSTALL HANGERS NOTED OR APPROVED EQUIVALENT

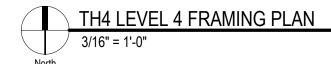
| | DESCRIPTION | Face Fasteners | Joist Fasteners |
|---|-------------|-------------------------|---------------------|
| 1 | LUS28 | (6) 10d x 3" NAILS | (4) 10d x 3" NAILS |
| 2 | LUS28-2 | (6) 10d x 3" NAILS | (4) 10d x 3" NAILS |
| 3 | HHUS410 | (30) 10d x 3" NAILS | (10) 10d x 3" NAILS |
| 4 | 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 |
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| | 22-048 |

DRAWING NO:

S1.22





FRAMING LEVEL PLAN NOTES:

| SEE S0.01 FOR | GENERAL STRU | CTURAL NOTES, | ABBREVIATIONS |
|---------------|---------------------|---------------|---------------|

KEY AND LEGEND

- SEE S5.01 FOR TYPICAL DETAILS
 SEE S6.01 FOR SCHEDULES
- 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
- TOP OF PLATE HEIGHT = XXX'-X" UNO
 WALL FRAMING AND COLUMNS SHOWN SUPPORT THE FRAMING ON
- THIS LEVEL
- 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.
- 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. <u>BLOCK</u> <u>AND NAIL ALL EDGES BETWEEN STUDS.</u>
 EXTERIOR WALL CONSTRUCTION (UNO): 2x STUDS @16" SHEATHED
- 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.
- <u>WALL OPENING CONSTRUCTION (UNO)</u>: (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. <u>TYPICAL DECK TIE (UNO)</u>: 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.
- <u>TYPICAL DECK CONSTRUCTION (UNO)</u> 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.
- 7. INDICATES HOLDOWN THROUGH LEVEL SHOWN, SEE SX.XX. CONTRACTOR TO VERIFY LOCATIONS AND LAYOUT WITH HDX FRAMING ABOVE

18. 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 |
| | | | |

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PROJECT LOCATION

BASECAMP TOWNHOME

1950 CURVE COURT STEAMBOAT SPRINGS, CO 80487 DRAWING TITLE

TH4 LEVEL 4 FRAMING PLAN

SEAL

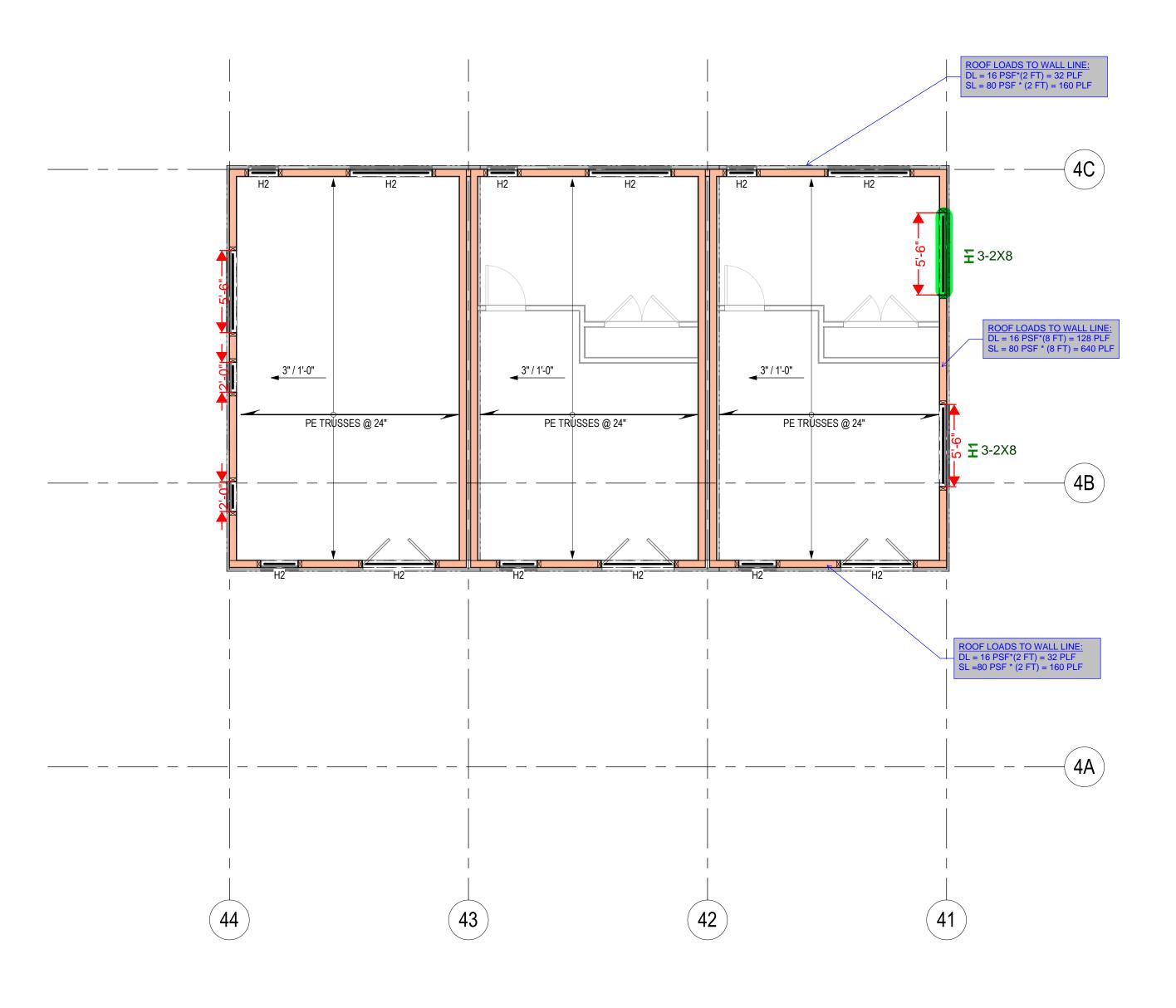
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DRAWING NO: **S1.23**

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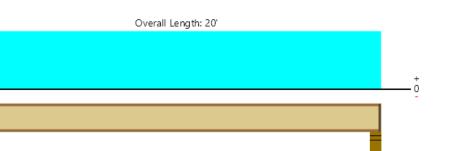


TH4 ROOF FRAMING PLAN 3/16" = 1'-0" North

| | ROOF LEVEL PLAN NOTES: | APPROVAL STAMPS: |
|--|---|---|
| 2. 3. 4. 5. 6. 7. 8. 9. 10 11 12 12 12 12 12 | ROOF LEVEL PLAN NOTES: SEE S0.01 FOR GENERAL STRUCTURAL NOTES, ABBREVIATIONS KEY AND LEGEND SEE S0.1 FOR SCHEDULES SEE S0.1 FOR SCHEDULES SEE S0.1 FOR SCHEDULES TOP OF PLATE HEIGHT = XXX:X T ALL OPENINGS W/ VENEER ABOVE WALL FRAMING AND COLUMNS SHOWN SUPPORT THE FRAMING ON THIS LEVEL REFER TO ARCHITECTURAL DRAWINGS FOR ADDITIONAL INFORMATION AND DIMENSIONS. ROOF CONSTRUCTION (UNO) 15/32" NOMINAL APA 32/16 OR 5/8" NOMINAL APA 40/20 RATED SHEATHING OVER PE TRUSSES / RAFTERS, RIMS, AND LEDGERS WITH 8d GUN NALLS (0.113" x 2 39") @ 4" ALONG PANEL DOES AND @ 7LONG INTEMEDIATE FRAMING MEMBERS, LAY PANELS PERFENDICULAR TO FRAMING MEMBERS AND STAGGER PANEL JOINTS. TYPICAL TRUSS / RAFTER TIE DOWN: (2) HTS20 (OR ST22 STRAP) EACH END AT BEARING (UNO): SHEATHING ONE STERIOR FACE: NAIL WALL SHEATHING WI 100 GUN NAILS (0.113" 9x 3") @ 3" PANEL EDEGS AND @ 12" IN FIELD OF PANEL BLOCK AND NAIL ALL EDGES BETWEEN STUDS. EXTERIOR WALL CONSTRUCTION (UNO): 2X STUDS @ 16" SHEATHWITH 11/2" ZIP-R SHEATHING ON CSTERIOR FACE: NAIL WALL SHEATHING WI 100 GUN NAILS (0.131" 9x 3") @ 3" PANEL EDEGS AND BOUNDARIES AND @ 12" IN FIELD OF PANEL BLOCK AND NAIL ALL EDGES BETWEEN STUDS. SHEATHING WILL CONSTRUCTION (UNO): 2X STUDS @ 16" SHEATHWINH 11/2" CDX PLYWOOD OR OS R.APA 24/16 ON EXTERIOR FACE: NAIL WALL SHEATHING WTH B6 GUN NAILS (0.131" 9x 2 3/8") @ 4" AT PANEL EDGES AND B0UNDARIES AND @ 10" INFLOAD FACE NAIL WALL CONSTRUCTION (UNO): 2X STUDS @ 16" SHEATHWE A | APPROVAL STAMPS: |
| FA RE | HANGER SCHEDULE ALL HANGERS NOTED TO BE INSTALLED WITH NUMBER AND SIZE STEENERS SPECIFIED BY MNIR. ANY SUBSTITUTIONS SHALL BE SUEWED AND APPROVED BY ANTHEM INSTALL HANGERS NOTED OR APPROVED EQUIVALENT DESCRIPTION Tace Fasteners 1 LUS28 (6) 10d x 3" NAILS 2 2 2 2 2 2 2 2 2 2 2 2 3 1 1 2 2 2 2 3 1 1 1 2 2 2 141 | LANDSCAPE ARCHITECT LANDSCAPE ARCHITECT Image: Construction of the constr |



20'



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.

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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.

| | Bearing Length | | | Loads | to Supports | | |
|--------------------|----------------|-----------|----------|-------|-------------|----------|------------------|
| Supports | Total | Available | Required | Dead | Floor Live | Factored | Accessories |
| 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 | | | | | |
| TTI jejete are only analyzed using Mavimum Allewable bracing celutions | | | | | | |

•TJI joists are only analyzed using Maximum Allowable bracing solutions.

•Maximum allowable bracing intervals based on applied load.

| | | | Dead | Floor Live | |
|-------------------|----------|---------|--------|------------|--------------|
| Vertical Load | Location | Spacing | (0.90) | (1.00) | Comments |
| 1 - Uniform (PSF) | 0 to 20' | 16" | 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 No Samantha Taylor Anthem Structural Engineers (303) 848-8497 staylor@anthemstructural.com

Job Notes



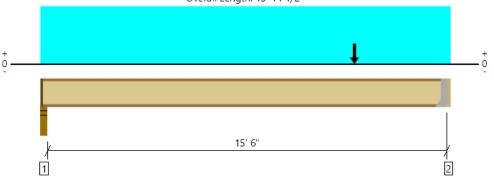
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4 bed unit typ joist, Floor Joist #2 - 15'-6" span 1 piece(s) 14" TJI ® 360 @ 16" OC

PASSED

Overall Length: 15' 11 1/2"



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) | 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.

| | Bearing Length | | | Loads | to Supports | | |
|---------------------------|----------------|---------------------|-------------|-------|-------------|----------|------------------|
| Supports | Total | Available | Required | Dead | Floor Live | Factored | Accessories |
| 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

| Bracing Intervals | Comments |
|-------------------|-----------|
| 6' 5" o/c | |
| 15' 8" o/c | |
| | 6' 5" o/c |

•TJI joists are only analyzed using Maximum Allowable bracing solutions.

•Maximum allowable bracing intervals based on applied load.

| Connector: Simpson Strong-T | īe | | | | | |
|-----------------------------|------------|-------------|---------------|----------------|------------------|-------------|
| 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.

| | | | Dead | Floor Live | |
|-------------------|------------------|---------|--------|------------|-----------------|
| Vertical Loads | Location | Spacing | (0.90) | (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 |

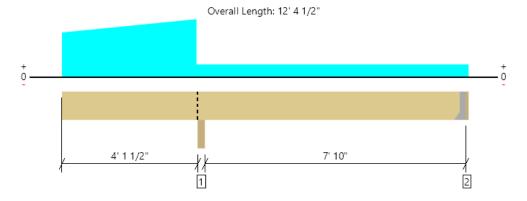
ForteWEB Software Operator Job Notes Samantha Taylor Anthem Structural Engineers (303) 848-8497 staylor@anthemstructural.com





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

PASSED

• 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.

| | Bearing Length | | | Loads to Supports (Ibs) | | | | |
|--|----------------|---------------------|----------|-------------------------|------------|------|----------|-------------|
| Supports | Total | Available | Required | Dead | Floor Live | Snow | Factored | Accessories |
| 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. | | | | | | | | |

• blocking Panels are assumed to carry no loads applied directly above them and the full load is applied to the member being t

• 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 | | | | | | |

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

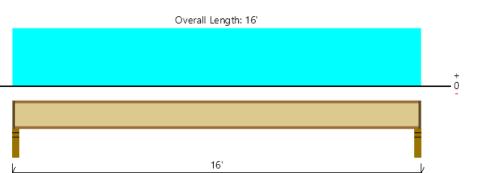
• Refer to manufacturer notes and instructions for proper installation and use of all connectors.

| | | | Dead | Floor Live | Snow | Wind | |
|-------------------|-----------------|---------|--------|------------|----------------|--------|-------------------|
| Vertical Loads | Location (Side) | Spacing | (0.90) | (1.00) | (1.15) | (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 |
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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.

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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.

| | Bearing Length | | | Loads to Supports (lbs) | | | |
|--------------------|----------------|-----------|----------|-------------------------|------------|----------|------------------|
| Supports | Total | Available | Required | Dead | Floor Live | Factored | Accessories |
| 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 | | | | |
| TTI jaista ava antu anaturad using Mavimum Allaurahla hypeing calutions | | | | | |

•TJI joists are only analyzed using Maximum Allowable bracing solutions.

•Maximum allowable bracing intervals based on applied load.

| | | | Dead | Floor Live | |
|-------------------|----------|---------|--------|------------|------------|
| Vertical Load | Location | Spacing | (0.90) | (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

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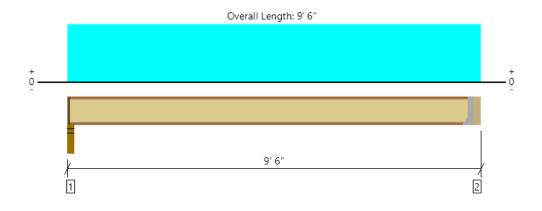
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4 bed unit typ joist, Floor Joist #5 - 9.5' span 1 piece(s) 14" TJI ® 110 @ 16" OC



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-Ibs) | 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 EdgeTM Panel (24" Span Rating) that is glued and nailed down.

• Additional considerations for the TJ-Pro[™] Rating include: None.

| | Bearing Length | | | Loads to Supports (lbs) | | | |
|---------------------------|----------------|---------------------|-------------|-------------------------|------------|----------|------------------|
| Supports | Total | Available | Required | Dead | Floor Live | Factored | Accessories |
| 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" | Hanger ¹ | 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

| Bracing Intervals | Comments |
|-------------------|-----------|
| 7' 3" o/c | |
| 9' 1" o/c | |
| | 7' 3" o/c |

•TJI joists are only analyzed using Maximum Allowable bracing solutions.

•Maximum allowable bracing intervals based on applied load.

| Connector: Simpson Strong-T | īe | | | | | |
|-----------------------------|------------|-------------|---------------|----------------|------------------|-------------|
| 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.

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

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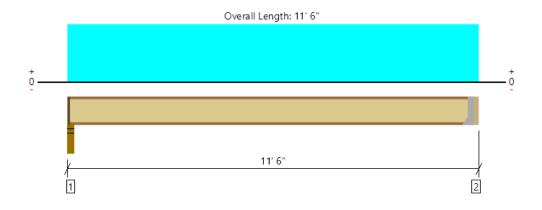
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|--|-----------|
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4 bed unit typ joist, Floor Joist #6 - 11'-6" span 1 piece(s) 14" TJI ® 110 @ 16" OC



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.

| | Bearing Length | | Loads | to Supports | | | |
|---------------------------|----------------|---------------------|-------------|-------------|------------|----------|------------------|
| Supports | Total | Available | Required | Dead | Floor Live | Factored | Accessories |
| 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

| Bracing Intervals | Comments |
|-------------------|------------|
| 5' 10" o/c | |
| 11' 2" o/c | |
| | 5' 10" 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.

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

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



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

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-Ibs) | 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 | | |

• 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.

| | Bearing Length | | Loads to Supports (lbs) | | | | | |
|--------------------|----------------|-----------|-------------------------|------|------------|------|----------|------------------|
| Supports | Total | Available | Required | Dead | Floor Live | Snow | Factored | Accessories |
| 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.

| | | | Dead | Floor Live | Snow | |
|-------------------|------------------|---------|--------|------------|--------|-------------------|
| Vertical Loads | Location (Side) | Spacing | (0.90) | (1.00) | (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 |

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



tion (Pattern) System : FI Member Ty

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-Ibs) | 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.

| | Bearing Length | | | Loads to Supports (lbs) | | | | |
|--------------------|----------------|-----------|----------|-------------------------|------------|------|----------|------------------|
| Supports | Total | Available | Required | Dead | Floor Live | Snow | Factored | Accessories |
| 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.

| | | | Dead | Floor Live | Snow | |
|-------------------|-----------------|---------|--------|------------|--------|-------------------|
| Vertical Loads | Location (Side) | Spacing | (0.90) | (1.00) | (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

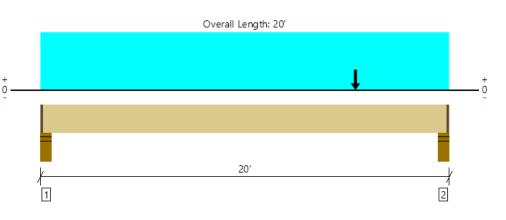
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All locations are measured from the outside face of left support (or left cantilever end). All dimensions are horizontal.

| | | | | | - |
|-----------------------|----------------------|--------------|----------------|------|-----------------------------|
| 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) |

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.

| | Bearing Length | | | Loads to Supports (lbs) | | | |
|--------------------|----------------|-----------|----------|-------------------------|------------|----------|------------------|
| Supports | Total | Available | Required | Dead | Floor Live | Factored | Accessories |
| 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.

| | | | Dead | Floor Live | |
|-----------------------|-----------------------|-----------------|--------|------------|---|
| Vertical Loads | Location (Side) | Tributary Width | (0.90) | (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

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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

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 staylor@anthemstructural.com

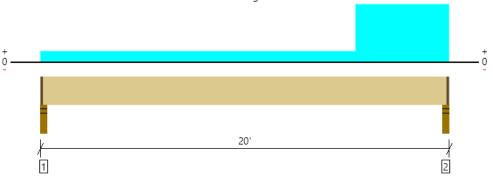
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4 bed unit typ joist, Floor Beam #2 3 piece(s) 1 3/4" x 14" 2.0E Microllam® LVL

Overall Length: 20'



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) |

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.

| | Bearing Length | | | Loads to Supports (lbs) | | | |
|--------------------|----------------|-----------|----------|-------------------------|------------|----------|------------------|
| Supports | Total | Available | Required | Dead | Floor Live | Factored | Accessories |
| 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

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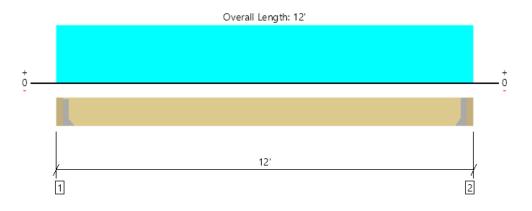
The product application, input design loads, dimensions and support information have been provided by ForteWEB Software Operator

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



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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) |

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.

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

Maximum allowable bracing intervals based on applied load.

| Connector: Simpson Strong-Tie | | | | | | | | | |
|-------------------------------|-----------------------------|---|---|--|---|--|--|--|--|
| Model | Seat Length | Top Fasteners | Face Fasteners | Member Fasteners | Accessories | | | | |
| HHUS410 | 3.00" | N/A | 30-10d | 10-10d | | | | | |
| HHUS410 | 3.00" | N/A | 30-10d | 10-10d | | | | | |
| | Model HHUS410 HHUS410 | Model Seat Length HHUS410 3.00" HHUS410 3.00" | Model Seat Length Top Fasteners HHUS410 3.00" N/A HHUS410 3.00" N/A | Model Seat Length Top Fasteners Face Fasteners HHUS410 3.00" N/A 30-10d HHUS410 3.00" N/A 30-10d | Model Seat Length Top Fasteners Face Fasteners Member Fasteners HHUS410 3.00" N/A 30-10d 10-10d HHUS410 3.00" N/A 30-10d 10-10d | | | | |

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

| | | | Dead | Floor Live | |
|-----------------------|----------------------|-----------------|--------|------------|--|
| Vertical Loads | Location (Side) | Tributary Width | (0.90) | (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

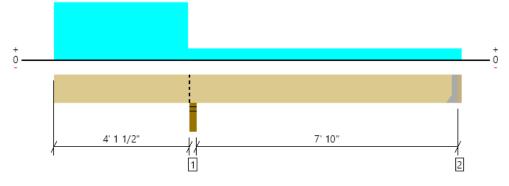
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4 bed unit typ joist, Floor Beam #5 2 piece(s) 2 x 12 HF No.2

Overall Length: 12' 5"



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.

| | Bearing Length | | | Loads to Supports (lbs) | | | | |
|-------------------------------|----------------|---------------------|----------|-------------------------|------------|------|----------|-------------|
| Supports | Total | Available | Required | Dead | Floor Live | Snow | Factored | Accessories |
| 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 | |
| Marrianum allaurable bus size intere | | |

•Maximum allowable bracing intervals based on applied load.

| 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 | Connector: Simpson Strong-T | Гie | | | | | |
|---|-----------------------------|---------|-------------|---------------|----------------|------------------|-------------|
| 2 - Face Mount Hanger LUS28-2 2.00" N/A 6-10dx1.5 3-10d | 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.

| | | | Dead | Floor Live | Snow | Wind | |
|-----------------------|----------------------|-----------------|--------|------------|--------|--------|---------------------|
| Vertical Loads | Location (Side) | Tributary Width | (0.90) | (1.00) | (1.15) | (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

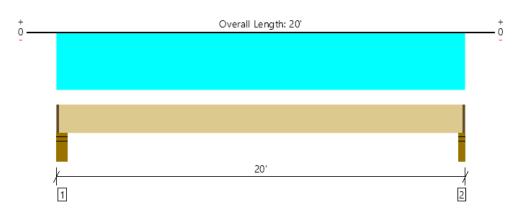
Stair Framing Beam

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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.

| | Bearing Length | | | | Loads to Su | | | |
|--------------------|----------------|-----------|----------|------|-------------|-------|-----------|------------------|
| Supports | Total | Available | Required | Dead | Floor Live | Snow | Factored | Accessories |
| 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. | | | | | | |

app

| | | | Dead | Floor Live | Snow | |
|-----------------------|-----------------------|-----------------|--------|-------------|--------|--|
| Vertical Loads | Location (Side) | Tributary Width | (0.90) | (1.00) | (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 |

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

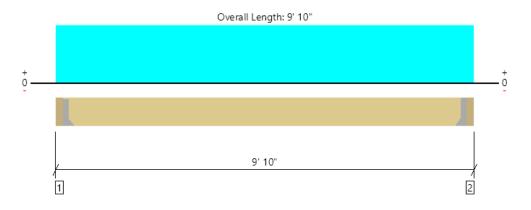
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Job Notes



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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) |

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.

| | Bearing Length | | | Loads | to Supports | | |
|---------------------------|----------------|---------------------|----------|-------|-------------|----------|-------------|
| Supports | Total | Available | Required | Dead | Floor Live | Factored | Accessories |
| 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 inten | Maximum allowable bracing integrale based on applied load | | | | | | |

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 | | | | | | |
| | с <u>с нин</u> 1 | C 11 1 | | | | | | | | | |

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

| | | | Dead | Floor Live | |
|-----------------------|---------------------|-----------------|--------|------------|--|
| Vertical Loads | Location (Side) | Tributary Width | (0.90) | (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 |

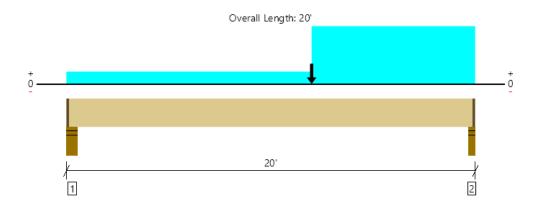
Member Notes

Stair Framing Beam

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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.

| | Bearing Length | | | Loads | to Supports | | |
|--------------------|----------------|-----------|----------|-------|-------------|----------|------------------|
| Supports | Total | Available | Required | Dead | Floor Live | Factored | Accessories |
| 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.

| | | | Dead | Floor Live | |
|-----------------------|-----------------------|-----------------|--------|------------|---|
| Vertical Loads | Location (Side) | Tributary Width | (0.90) | (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

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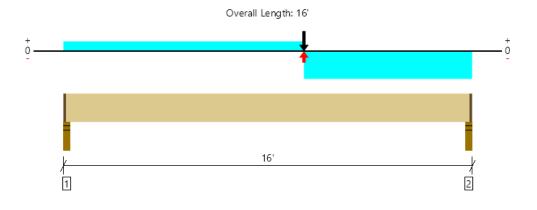
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4 bed unit typ joist, Floor Beam #9 2 piece(s) 1 3/4" x 14" 2.0E Microllam® LVL



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.

| | Bearing Length | | | Loads to Supports (lbs) | | | | |
|--------------------|----------------|-----------|----------|-------------------------|------------|------|-----------|------------------|
| Supports | Total | Available | Required | Dead | Floor Live | Snow | Factored | Accessories |
| 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.

| | | | Dead | Floor Live | Snow | |
|-----------------------|-----------------------|-----------------|--------|-------------|--------|--|
| Vertical Loads | Location (Side) | Tributary Width | (0.90) | (1.00) | (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

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

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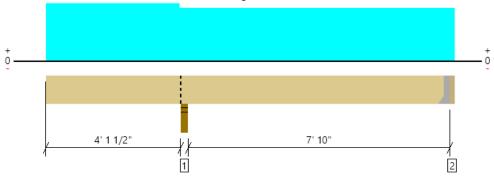


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4 bed unit typ joist, Floor Beam #10 2 piece(s) 2 x 12 HF No.2

Overall Length: 12' 5 1/2"



 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)
 System

| 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.

| | Bearing Length | | | Loads to Supports (lbs) | | | | |
|-------------------------------|----------------|---------------------|----------|-------------------------|------------|------|----------|-------------|
| Supports | Total | Available | Required | Dead | Floor Live | Snow | Factored | Accessories |
| 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 ellevenda harafa interneta harafan analiad haraf | | | | | | | |

•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 instructi | ons for proper installation and use | of all connectors | | | | |

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

| | | | Dead | Floor Live | Snow | Wind | |
|-----------------------|------------------------------------|-----------------|--------|------------|--------|--------|--|
| Vertical Loads | Location (Side) | Tributary Width | (0.90) | (1.00) | (1.15) | (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

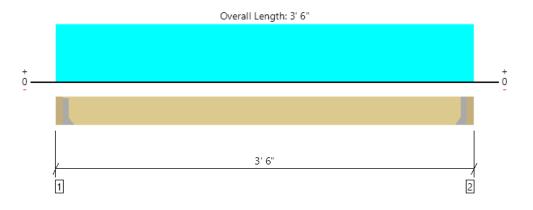
Stair Framing Beam

| ForteWEB Software Operator | Job Notes |
|--|-----------|
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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) |

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.

| | Bearing Length | | | Loads | to Supports | | |
|---------------------------|----------------|---------------------|----------|-------|-------------|----------|-------------|
| Supports | Total | Available | Required | Dead | Floor Live | Factored | Accessories |
| 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 allaurable brasing intervals based on applied load | | | | | | | |

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.

| | | | Dead | Floor Live | |
|-----------------------|---------------------|-----------------|--------|------------|----------------------|
| Vertical Loads | Location (Side) | Tributary Width | (0.90) | (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 |

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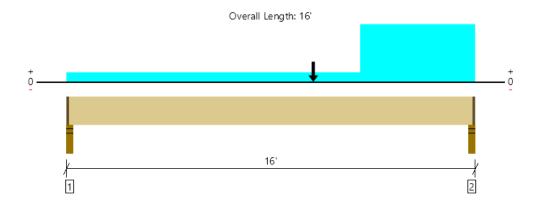
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4 bed unit typ joist, Floor Beam #12 2 piece(s) 1 3/4" x 14" 2.0E Microllam® LVL



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) |

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.

| | Bearing Length | | | Loads | to Supports | | |
|--------------------|----------------|-----------|----------|-------|-------------|----------|------------------|
| Supports | Total | Available | Required | Dead | Floor Live | Factored | Accessories |
| 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.

| | | | Dead | Floor Live | |
|-----------------------|-----------------------|-----------------|--------|------------|--|
| Vertical Loads | Location (Side) | Tributary Width | (0.90) | (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 |

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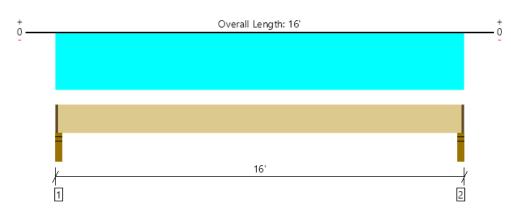
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ForteWEB Software Operator Jo Samantha Taylor Anthem Structural Engineers (303) 848-8497 staylor@anthemstructural.com

Job Notes







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

PASSED

• 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.

| | Bearing Length | | | Loads to Supports (lbs) | | | | |
|--------------------|----------------|-----------|----------|-------------------------|------------|-------|-----------|------------------|
| Supports | Total | Available | Required | Dead | Floor Live | Snow | Factored | Accessories |
| 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. | | | | | | |

lowable 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 |

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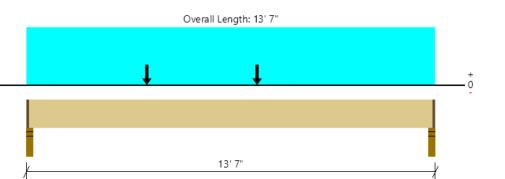
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Job Notes







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) |

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

2

• Deflection criteria: LL (L/360) and TL (L/240).

• Allowed moment does not reflect the adjustment for the beam stability factor.

0

1

| | Bearing Length | | Loads to Supports (lbs) | | | | |
|--------------------|----------------|-----------|-------------------------|------|------------|----------|------------------|
| Supports | Total | Available | Required | Dead | Floor Live | Factored | Accessories |
| 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.

| | | | Dead | Floor Live | |
|-----------------------|----------------------|-----------------|--------|------------|--|
| Vertical Loads | Location (Side) | Tributary Width | (0.90) | (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 |

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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

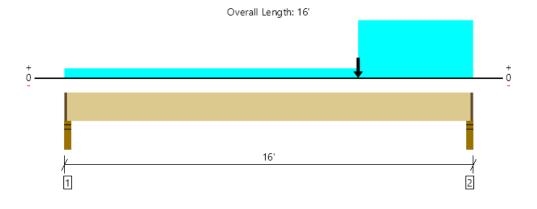
 Samantha Taylor
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4 bed unit typ joist, Floor Beam #15 3 piece(s) 1 3/4" x 14" 2.0E Microllam® LVL



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.

| | Bearing Length | | | Loads to Supports (lbs) | | | |
|--------------------|----------------|-----------|----------|-------------------------|------------|----------|------------------|
| Supports | Total | Available | Required | Dead | Floor Live | Factored | Accessories |
| 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.

| | | | Dead | Floor Live | |
|-----------------------|-----------------------|-----------------|--------|------------|--|
| Vertical Loads | Location (Side) | Tributary Width | (0.90) | (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 |

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Job Notes





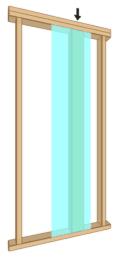
PASSED

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"



| 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 | Туре | Material | System : Wall |
|----------|--------|----------|--|
| Тор | Dbl 2X | Hem Fir | Member Type : Column Building Code : IBC 2018 Design Methodology : ASD |
| Base | 2X | Hem Fir | |

Drawing is Conceptual

| Max Unbraced Length | Comments |
|---------------------|----------|
| 1' | |

| Lateral Connections | | | | | | | |
|---------------------|-----------|----------------------------|----------|-------------------|--|--|--|
| Supports | Connector | Type/Model | Quantity | Connector Nailing | | | |
| Тор | 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 (Ib) | 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 |

| | | | Wind | |
|-------------------|-------------|-----------------|--------|----------|
| Lateral Load | Location | Tributary Width | (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.

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| ForteWEB Software Opera | tor | Job Notes |
|--|-----|-----------|
| Samantha Taylor Anthem Structural Engineers (303) 848-8497 staylor@anthemstructural.com | 1 | |



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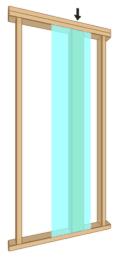
PASSED

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"



| 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 | Туре | Material | System : Wall |
|----------|--------|----------|--|
| Тор | Dbl 2X | Hem Fir | Member Type : Column Building Code : IBC 2018 |
| Base | 2X | Hem Fir | Design Methodology : ASD |

Drawing is Conceptual

| Max Unbraced Length | Comments |
|---------------------|----------|
| 1' | |

| Lateral Connections | | | | | | | |
|---------------------|-----------|----------------------------|----------|-------------------|--|--|--|
| Supports | Connector | Type/Model | Quantity | Connector Nailing | | | |
| Тор | 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.

| | | Dead | Floor Live | |
|----------------|-----------------|--------|------------|--|
| Vertical Loads | Tributary Width | (0.90) | (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 (Ib) | N/A | 1478 | 2909 | Linked from: Floor Beam #8, Support 2 |
| 4 - Point (Ib) | N/A | 1478 | 2909 | Linked from: Floor Beam #8, Support 2 |

| | | | Wind | |
|-------------------|-------------|-----------------|--------|----------|
| Lateral Load | Location | Tributary Width | (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

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|--|-----------|
| Samantha Taylor Anthem Structural Engineers (303) 848-8497 staylor@anthemstructural.com | |



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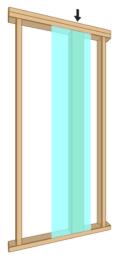
PASSED

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"



| 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.

Comments

| Supports | Туре | Material | System : Wall |
|----------|--------|----------|--|
| Тор | Dbl 2X | Hem Fir | Member Type : Column Building Code : IBC 2018 |
| Base | 2X | Hem Fir | Design Methodology : ASD |

Drawing is Conceptual

| Lateral Connections | | | | | | | | |
|---------------------|-----------|----------------------------|----------|-------------------|--|--|--|--|
| Supports | Connector | Type/Model | Quantity | Connector Nailing | | | | |
| Тор | 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.

| | | Dead | Floor Live | |
|----------------|-----------------|--------|------------|--|
| Vertical Loads | Tributary Width | (0.90) | (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 |

Max Unbraced Length

1'

| | | | Wind | |
|-------------------|-------------|-----------------|--------|----------|
| Lateral Load | Location | Tributary Width | (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

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|--|-----------|
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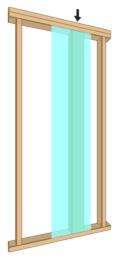


4 bed unit typ joist, Post #4 3 piece(s) 2 x 4 HF No.2

Wall Height: 10' 3 7/8"

Member Height: 9' 11 3/8"

Tributary Width: 1' 4"



| 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.

Comments

| Top Dbl 2X H | | Member Type : Column |
|--------------|---------|--------------------------|
| DDIZX | lem Fir | Building Code : IBC 2018 |
| Base 2X H | lem Fir | Design Methodology : ASD |

Drawing is Conceptual

| Lateral Connections | | | | | | |
|---------------------|-----------|----------------------------|----------|-------------------|--|--|
| Supports | Connector | Type/Model | Quantity | Connector Nailing | | |
| Тор | 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.

| | | Dead | Floor Live | |
|----------------|-----------------|--------|------------|---|
| Vertical Loads | Tributary Width | (0.90) | (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 |

Max Unbraced Length

1'

| | | | Wind | |
|-------------------|-------------|-----------------|--------|----------|
| Lateral Load | Location | Tributary Width | (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

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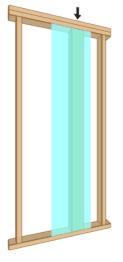
4 bed unit typ joist, Post #5 4 piece(s) 2 x 6 HF No.2

Wall Height: 10' 3 7/8"

Member Height: 9' 11 3/8"

Tributary Width: 1' 4"

PASSED



| 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 | Туре | Material | System : Wall |
|----------|--------|----------|--|
| Тор | Dbl 2X | Hem Fir | Member Type : Column Building Code : IBC 2018 |
| Base | 2X | Hem Fir | Design Methodology : ASD |

Drawing is Conceptual

Max Unbraced Length Comments 1'

| Lateral Connectio | ns | | | |
|-------------------|-----------|----------------------------|----------|-------------------|
| Supports | Connector | Type/Model | Quantity | Connector Nailing |
| Тор | 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.

| | | Dead | Floor Live | |
|----------------|-----------------|--------|------------|---|
| Vertical Loads | Tributary Width | (0.90) | (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 |

| | | | Wind | |
|-------------------|-------------|-----------------|--------|----------|
| Lateral Load | Location | Tributary Width | (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

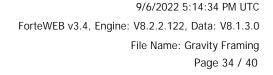
Stud pack at Beam #1

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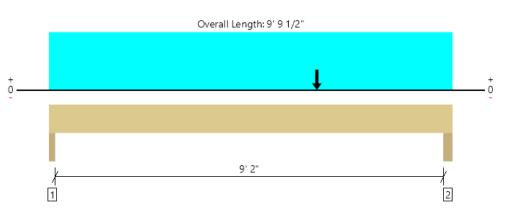
Weyerhaeuser

| ForteWEB Software Operator | Job Notes | |
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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-Ibs) | 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 (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) |

ystem : Wall Member Type : Header uilding Use : Residential uilding 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.

| | Bearing Length | | | Loads | | | | | |
|-------------------|----------------|-----------|----------|-------|------------|------|------|----------|-------------|
| Supports | Total | Available | Required | Dead | Floor Live | Snow | Wind | Factored | Accessories |
| 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 alloughle brasing intervale based on applied lead | | | | | | | |

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 (Ib) | 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 |

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

| ForteWEB Software Operator | Job Notes |
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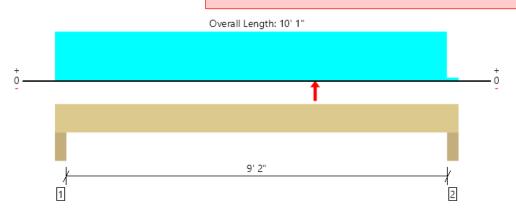
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MEMBER REPORT

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.

| | Bearing Length | | | | Loads | | | | |
|-------------------|----------------|-----------|----------|------|------------|------|-------|----------------|-------------|
| Supports | Total | Available | Required | Dead | Floor Live | Snow | Wind | Factored | Accessories |
| 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.

| | | | Dead | Floor Live | Snow | Wind | |
|-----------------------|----------------|-----------------|--------|------------|--------|--------|--|
| Vertical Loads | Location | Tributary Width | (0.90) | (1.00) | (1.15) | (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 |

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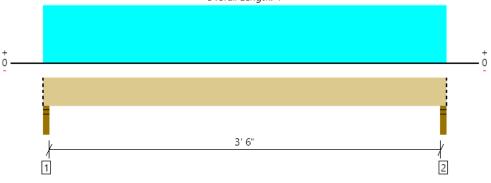
The product application, input design loads, dimensions and support information have been provided by ForteWEB Software Operator

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



4 bed unit typ joist, STAIR LEDGER 1 piece(s) 2 x 8 HF No.2

Overall Length: 4'



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.

| | Bearing Length | | | Loads | to Supports | | |
|--|----------------|-----------|----------|-------|-------------|----------|-------------|
| Supports | Total | Available | Required | Dead | Floor Live | Factored | Accessories |
| 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.

| | | | Dead | Floor Live | |
|-----------------------|-----------------|-----------------|--------|------------|--------------|
| Vertical Loads | Location (Side) | Tributary Width | (0.90) | (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 |

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| ForteWEB Software Operator | Job Notes |
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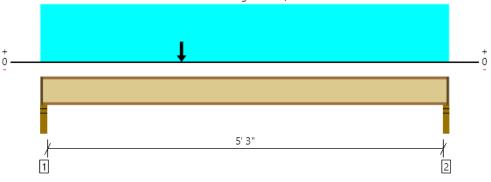
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4 bed unit typ joist, TRANSFER BM @ STAIR POST 1 piece(s) 14" TJI ® 360

PASSED

Overall Length: 5' 9 1/2"



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) | 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.

| | Bearing Length | | | Loads | to Supports | | |
|--------------------|----------------|-----------|----------|-------|-------------|----------|------------------|
| Supports | Total | Available | Required | Dead | Floor Live | Factored | Accessories |
| 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.

| | | | Dead | Floor Live | |
|-----------------------|---------------------|-----------------|--------|------------|---|
| Vertical Loads | Location | Tributary Width | (0.90) | (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 |

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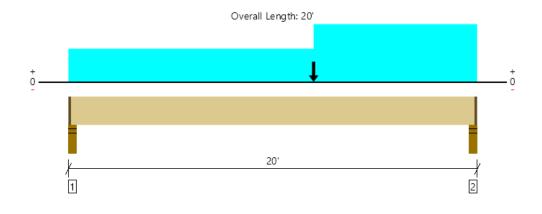
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| 1 | ForteWEB Software Operator | Job Notes |
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4 bed unit typ joist, Copy of Floor Beam #8 - L4 ROOF TRANSFER 4 piece(s) 1 3/4" x 14" 2.0E Microllam® LVL



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.

| | Bearing Length | | | Loads to Sup | | | | |
|---|----------------|-----------|----------|--------------|------------|------|----------|------------------|
| Supports | Total | Available | Required | Dead | Floor Live | Snow | Factored | Accessories |
| 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. | | | | | | | | |

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| 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

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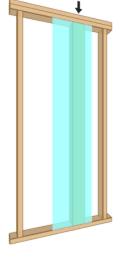
4 bed unit typ joist, Copy of Post #1 - CENTER POST 4 piece(s) 2 x 6 HF No.2

Wall Height: 9'

Member Height: 8' 7 1/2"

Tributary Width: 1' 4"

PASSED



| 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.

Max Unbraced Length

• 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

Comments

the U.S. Wall Guide for multiple-member connection requirements.

| Supports | Туре | Material | System : |
|----------|--------|-------------------|----------------------|
| Тор | Dbl 2X | Douglas Fir-Larch | Member |
| Base | 2X | Douglas Fir-Larch | Building Design M |

ystem : Wall lember Type : Column uilding Code : IBC 2018 lesign Methodology : ASD

Drawing is Conceptual

Latoral Connoc

| | | 1' | | |
|------|----|----|--|--|
| | | | | |
| ctio | ns | | | |
| | | | | |

| Supports | Connector | Type/Model | Quantity | Connector Nailing | | | |
|----------|-----------|----------------------------|----------|-------------------|--|--|--|
| Тор | 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.

| | | Dead | Floor Live | Snow | |
|----------------|-----------------|--------|------------|--------|--|
| Vertical Loads | Tributary Width | (0.90) | (1.00) | (1.15) | Comments |
| 1 - Point (lb) | N/A | 10 | - | - | Wall Dead Load |
| 2 - Point (Ib) | N/A | 1478 | 2909 | - | Linked from: Floor Beam #8, Support 2 |
| 3 - Point (Ib) | 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 |

| | | | Wind | |
|-------------------|-------------|-----------------|--------|----------|
| Lateral Load | Location | Tributary Width | (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.

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

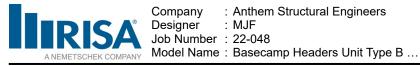
 ForteWEB Software Operator
 Job Notes

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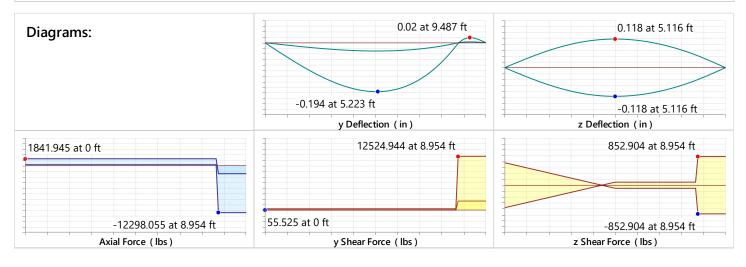


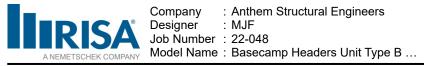
9/6/2022 5:14:34 PM UTC ForteWEB v3.4, Engine: V8.2.2.122, Data: V8.1.3.0 File Name: Gravity Framing Page 40 / 40

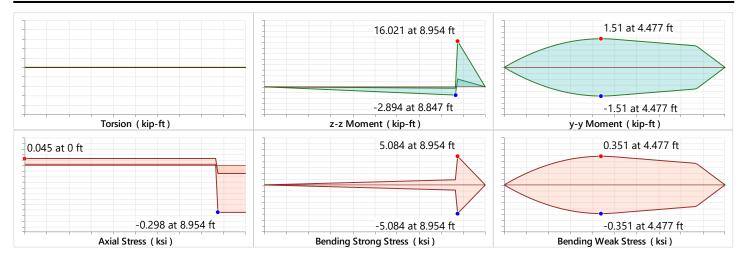


| Detail Report: M24 | | Unity Check: 4.381 | (LC 7) | L | oad Combination: Envelop |
|-----------------------|-----------------|------------------------------|-----------------|---|--------------------------|
| Ň | № | nput Data: | | | |
| | x | Shape: | 5-2X6 (nominal) | I Node: | N45 |
| | | Member Type: | Column | J Node: | N48 |
| z | z | 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 | E mod: | 1 | d (actual) (in): | 5.5 |
| F _v (ksi): | 0.15 | COV _F (Table F1): | 0.25 | # of Plies: | 5 |
| F _c (ksi): | 1.3 | E _{min} (ksi): | 474.901 | К _f : | 0.6 |
| Design Properties: | | | | | |
| le2 (ft): | 0.5 | С _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 | | |
| К _{у-у} : | 1 | C _{fu} : | 1 | | |
| K _{z-z} : | 1 | C _P : | 0.417 | | |
| y sway: | No | -μ. K _f : | 0.6 | | |
| z sway: | No | - T | 0.0 | | |





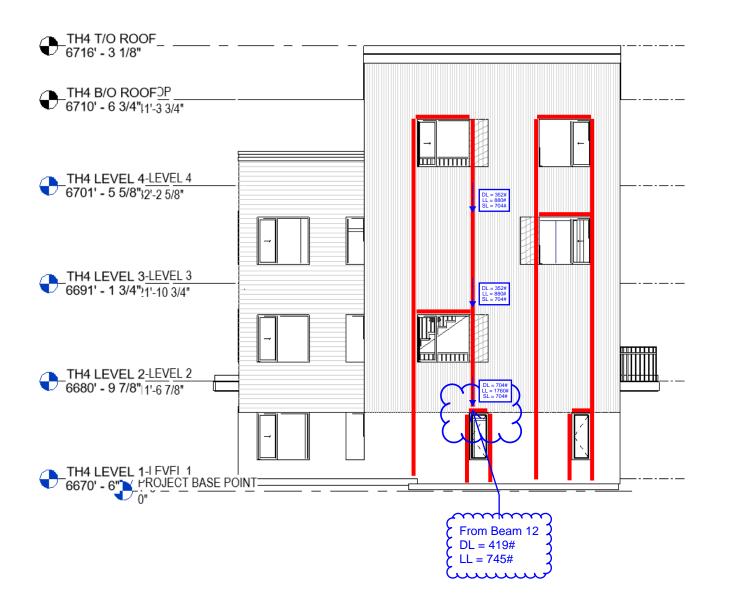


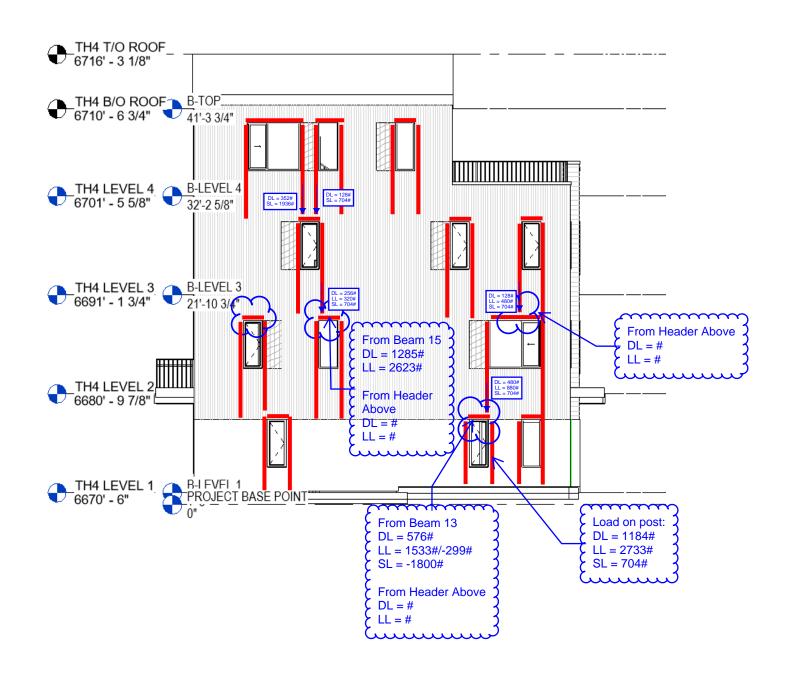


AWC NDS-18: ASD Code Check

| 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 (FLIP) WEST ELEVATION



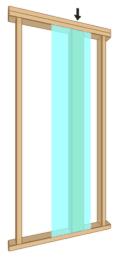
PASSED

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"



| 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 requirements.

| the U.S. Wall Guide for | r multiple-member | connection I |
|-------------------------|-------------------|--------------|
|-------------------------|-------------------|--------------|

| Supports | Туре | Material | System : Wall |
|----------|--------|----------|--|
| Тор | Dbl 2X | Hem Fir | Member Type : Column Building Code : IBC 2018 |
| Base | 2X | Hem Fir | Design Methodology : ASD |

Drawing is Conceptual

| Max Unbraced Length | Comments |
|---------------------|----------|
| 1' | |

| Lateral Connections | | | | | | | |
|---------------------|-----------|-------------------------|----------|-------------------|--|--|--|
| Supports | Connector | Type/Model | Quantity | Connector Nailing | | | |
| Тор | 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.

| | | Dead | Floor Live | Snow | |
|----------------|-----------------|--------|------------|--------|--------------|
| Vertical Load | Tributary Width | (0.90) | (1.00) | (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.

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|--|-----------|
| Samantha Taylor Anthem Structural Engineers (303) 848-8497 staylor@anthemstructural.com | |





Level, 20' Garage Header Column

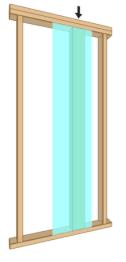
4 piece(s) 2 x 6 HF No.2

Wall Height: 9'

Member Height: 8' 7 1/2"

Tributary Width: 4' 6"

PASSED



| 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)

Max Unbraced Length

• 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.

Comments

| Supports | Туре | Material | System : Wall |
|----------|--------|-------------------|--|
| Тор | Dbl 2X | Douglas Fir-Larch | Member Type : Column Building Code : IBC 2018 |
| Base | 2X | Douglas Fir-Larch | Design Methodology : ASD |

Drawing is Conceptual

| Lateral Connections | | | | | | | | | |
|---------------------|-----------|----------------------------|-------------------|-----|--|--|--|--|--|
| Supports | Connector | Quantity | Connector Nailing | | | | | | |
| Тор | 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.

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

| | | | Wind | |
|-------------------|-------------|-----------------|--------|----------|
| Lateral Load | Location | Tributary Width | (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.

1'

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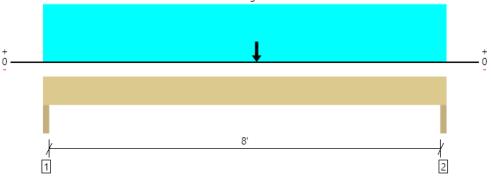
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|--|-----------|
| Samantha Taylor Anthem Structural Engineers (303) 848-8497 staylor@anthemstructural.com | |





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.

| | Bearing Length | | | Loads to Supports (lbs) | | | | |
|------------------|----------------|-----------|----------|-------------------------|------------|------|----------|-------------|
| Supports | Total | Available | Required | Dead | Floor Live | Snow | Factored | Accessories |
| 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 |

•Maximum allowable bracing intervals based on applied load.

| | | | Dead | Floor Live | Snow | |
|-----------------------|------------|-----------------|--------|------------|--------|--------------|
| Vertical Loads | Location | Tributary Width | (0.90) | (1.00) | (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 | |

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STUD WALLS



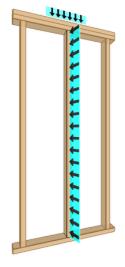
PASSED

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"



| 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 | Туре | | Material | System : Wall |
|---------------------|--------|---|----------|--|
| Тор | Dbl 2X | (| Hem Fir | Member Type : Stud Building Code : IBC 2018 |
| Base | 2X | | Hem Fir | Design Methodology : ASD |
| Max Unbraced Length | 1 | | Comments | |

Drawing is Conceptual

| Lateral Connections | | | | | | | | |
|---------------------|-----------|----------------------------|----------|-------------------|--|--|--|--|
| Supports | Connector | Type/Model | Quantity | Connector Nailing | | | | |
| Тор | 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.

| | | Dead | Snow | |
|-----------------|---------|--------|--------|----------|
| Vertical Load | Spacing | (0.90) | (1.15) | Comments |
| 1 - Point (PLF) | 16.00" | 160.0 | 800.0 | ROOF |

| | | | Wind | |
|-------------------|-------------|---------|--------|------------------|
| Lateral Load | Location | Spacing | (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.

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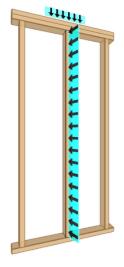


EXTERIOR BEARING WALLS, GRID 11, LEVEL 3 1 piece(s) 2 x 6 HF No.2 @ 16" OC

Wall Height: 9' 7 1/8"

Member Height: 9' 2 5/8"

O. C. Spacing: 16.00"



| 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 | Туре | | Material | System : Wall |
|---------------------|--------|--|----------|--|
| Тор | Dbl 2X | | Hem Fir | Member Type : Stud Building Code : IBC 2018 |
| Base | 2X | | Hem Fir | Design Methodology : ASD |
| | | | | |
| Max Unbraced Length | | | Comments | |

Drawing is Conceptual

| Lateral Connections | | | | | | | | |
|---------------------|-----------|----------------------------|----------|-------------------|--|--|--|--|
| Supports | Connector | Type/Model | Quantity | Connector Nailing | | | | |
| Тор | 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.

| | | Dead | Floor Live | Snow | |
|-----------------|---------|--------|------------|--------|----------|
| Vertical Loads | Spacing | (0.90) | (1.00) | (1.15) | Comments |
| 1 - Point (PLF) | 16.00" | 160.0 | - | 800.0 | ROOF |
| 2 - Point (PLF) | 16.00" | 260.0 | 400.0 | - | LEVEL 4 |

| | | | Wind | |
|-------------------|-------------|---------|--------|------------------|
| Lateral Load | Location | Spacing | (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.

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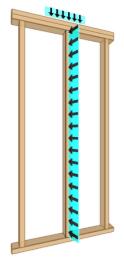


EXTERIOR BEARING WALLS, GRID 11, LEVEL 2 1 piece(s) 2 x 6 HF No.2 @ 16" OC

Wall Height: 9' 7 1/8"

Member Height: 9' 2 5/8"

O. C. Spacing: 16.00"



| 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 | Туре | | Material | System : Wall |
|---------------------|--------|--|----------|--|
| Тор | Dbl 2X | | Hem Fir | Member Type : Stud Building Code : IBC 2018 |
| Base | 2X | | Hem Fir | Design Methodology : AS |
| Max Unbraced Length | | | Comments | |

Drawing is Conceptual

| Lateral Connections | | | | | | | |
|---------------------|-----------|----------------------------|----------|-------------------|--|--|--|
| Supports | Connector | Type/Model | Quantity | Connector Nailing | | | |
| Тор | 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.

| | | Dead | Floor Live | Snow | |
|-----------------|---------|--------|------------|--------|----------|
| Vertical Loads | Spacing | (0.90) | (1.00) | (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 |

| | | | Wind | | | | | |
|-----------------------------------|--|---------|--------|------------------|--|--|--|--|
| Lateral Load | Location | Spacing | (1.60) | Comments | | | | |
| 1 - Uniform (PLF) | Full Length | N/A | 50.6 | 37.9PSF * 1.33FT | | | | |
| - IBC Table 1604.2 feetnate fr Da | - TPC Table 1604.2 features fu Deflection checks are performed using 40% of this lateral wind lead | | | | | | | |

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

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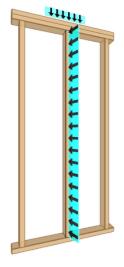


EXTERIOR BEARING WALLS, GRID 11, LEVEL 1 1 piece(s) 2 x 6 HF No.2 @ 16" OC

Wall Height: 9' 7 1/8"

Member Height: 9' 2 5/8"

O. C. Spacing: 16.00"



| 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 | Туре | | Material | System : Wa |
|---------------------|--------|--|-------------------|----------------------------|
| Тор | Dbl 2X | | Douglas Fir-Larch | Member Typ Building Cod |
| Base | 2X | | Douglas Fir-Larch | |
| Max Unbraced Length | 1 | | Comments | Design Meth |

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

Drawing is Conceptual

| Lateral Connections | | | | | | | |
|---------------------|-----------|----------------------------|----------|-------------------|--|--|--|
| Supports | Connector | Type/Model | Quantity | Connector Nailing | | | |
| Тор | 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.

| | | Dead | Floor Live | Snow | |
|-----------------|---------|--------|------------|--------|----------|
| Vertical Loads | Spacing | (0.90) | (1.00) | (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 |

| | | | Wind | |
|-------------------|-------------|---------|--------|------------------|
| Lateral Load | Location | Spacing | (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.

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EXTERIOR BEARING WALLS, GRID 11, LEVEL 1 (AT UPPER DECK)

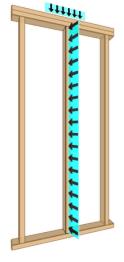
2 piece(s) 2 x 6 HF No.2 @ 16" OC

Wall Height: 10' 3"

Member Height: 9' 10 1/2"

O. C. Spacing: 16.00"

PASSED



| 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.

Max Unbraced Length

1'

• 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.

Comments

· A 15% increase in the moment capacity has been added to account for repetitive member usage

| Supports | Туре | Material | System : Wall |
|----------|--------|-------------------|--|
| Тор | Dbl 2X | Douglas Fir-Larch | Member Type : Stud Building Code : IBC 2018 |
| Base | 2X | Douglas Fir-Larch | Design Methodology : ASD |

Drawing is Conceptual

| Lateral Connections | | | | | | | | |
|---------------------|-----------|----------------------------|----------|-------------------|--|--|--|--|
| Supports | Connector | Type/Model | Quantity | Connector Nailing | | | | |
| Тор | 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 | | | | |

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| | | Dead | Floor Live | Snow | |
|-----------------|---------|--------|------------|--------|----------------|
| Vertical Loads | Spacing | (0.90) | (1.00) | (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 |

| | | | Wind | | | | | | |
|---------------------------------|---|---------|--------|------------------|--|--|--|--|--|
| Lateral Load | Location | Spacing | (1.60) | Comments | | | | | |
| 1 - Uniform (PLF) | Full Length | N/A | 50.6 | 37.9PSF * 1.33FT | | | | | |
| IDC Table 1604.2 feetnate fr De | - IDC Table 1604.2. feetnate fr. Deflection checks are notformed using 420% of this lateral wind lead | | | | | | | | |

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

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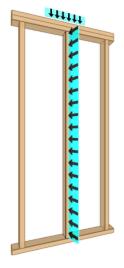


EXTERIOR NON-BEARING, GRID 1A, LEVEL 3 TYP 1 piece(s) 2 x 6 HF No.2 @ 16" OC

Wall Height: 9' 7 1/8"

Member Height: 9' 2 5/8"

O. C. Spacing: 16.00"



| 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 | Туре | | Material | System : Wall |
|---------------------|--------|--|----------|--|
| Тор | Dbl 2X | | Hem Fir | Member Type : Stud Building Code : IBC 2018 |
| Base | 2X | | Hem Fir | Design Methodology : ASD |
| | | | | - |
| Max Unbraced Length | | | Comments | |

Drawing is Conceptual

| Lateral Connections | | | | | | | | |
|---------------------|-----------|----------------------------|----------|-------------------|--|--|--|--|
| Supports | Connector | Type/Model | Quantity | Connector Nailing | | | | |
| Тор | 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.

| | | Dead | Floor Live | Snow | |
|-----------------|---------|--------|------------|--------|----------------|
| Vertical Load | Spacing | (0.90) | (1.00) | (1.15) | Comments |
| 1 - Point (PLF) | 16.00" | 32.0 | 120.0 | 192.0 | LEVEL 4 (DECK) |

| | | | Wind | |
|-------------------|-------------|---------|--------|------------------|
| Lateral Load | Location | Spacing | (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.

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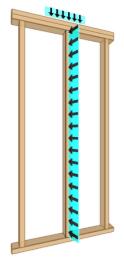


EXTERIOR NON-BEARING, GRID 1A, LEVEL 2 TYP 1 piece(s) 2 x 6 HF No.2 @ 16" OC

Wall Height: 9' 7 1/8"

Member Height: 9' 2 5/8"

O. C. Spacing: 16.00"



| 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 | Туре | | Material | | System : Wall |
|---------------------|--------|----------|----------|--|--|
| Тор | Dbl 2X | | Hem Fir | | Member Type : Stud Building Code : IBC 2018 Design Methodology : ASD |
| Base | 2X | | Hem Fir | | |
| | | | - | | |
| Max Unbraced Length | L | Comments | | | |

Drawing is Conceptual

| Lateral Connections | | | | | | | | |
|---------------------|-----------|----------------------------|----------|-------------------|--|--|--|--|
| Supports | Connector | Type/Model | Quantity | Connector Nailing | | | | |
| Тор | 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 |

| | | | Wind | |
|-------------------|-------------|---------|--------|------------------|
| Lateral Load | Location | Spacing | (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.

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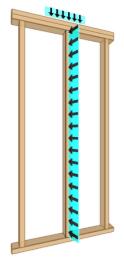


EXTERIOR NON-BEARING, GRID 1A, LEVEL 1 TYP 1 piece(s) 2 x 6 HF No.2 @ 16" OC

Wall Height: 9' 7 1/8"

Member Height: 9' 2 5/8"

O. C. Spacing: 16.00"



| 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 | Туре | | Material | System : Wall |
|-------------------|--------|--|----------|--|
| Тор | Dbl 2X | | Hem Fir | Member Type : Stud Building Code : IBC 2018 |
| Base | 2X | | Hem Fir | Design Methodology : ASD |
| | | | | |
| Max Unbraced Leng | th | | Comments | |

Drawing is Conceptual

| Lateral Connections | | | | | | | |
|---------------------|-----------|----------------------------|----------|-------------------|--|--|--|
| Supports | Connector | Type/Model | Quantity | Connector Nailing | | | |
| Тор | 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.

| | | Dead | Floor Live | Snow | |
|-----------------|---------|--------|------------|--------|----------|
| Vertical Loads | Spacing | (0.90) | (1.00) | (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 |

| | | | Wind | | | | | |
|-----------------------------------|---|---------|--------|------------------|--|--|--|--|
| Lateral Load | Location | Spacing | (1.60) | Comments | | | | |
| 1 - Uniform (PLF) | Full Length | N/A | 50.6 | 37.9PSF * 1.33FT | | | | |
| • IPC Table 1604.2 feetnate fr De | IBC Table 1604.3 footnote fr. Deflection checks are performed using 42% of this lateral wind load | | | | | | | |

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

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| ForteWEB Software Operator | Job Notes |
|--|-----------|
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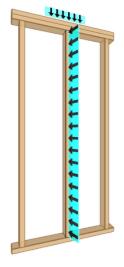


EXTERIOR NON-BEARING, GRID 1E, LEVEL 4 TYP 1 piece(s) 2 x 6 HF No.2 @ 16" OC

Wall Height: 9' 7 1/8"

Member Height: 9' 2 5/8"

O. C. Spacing: 16.00"



| 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 | Туре | | Material | System : Wall |
|---------------------|--------|--|----------|--|
| Тор | Dbl 2X | | Hem Fir | Member Type : Stud Building Code : IBC 2018 |
| Base | 2X | | Hem Fir | Design Methodology : ASD |
| Max Unbraced Length | 1 | | Comments | |

Drawing is Conceptual

| Lateral Connections | | | | | | | |
|---------------------|-----------|----------------------------|----------|-------------------|--|--|--|
| Supports | Connector | Type/Model | Quantity | Connector Nailing | | | |
| Тор | 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.

| | | Dead | Snow | |
|-----------------|---------|--------|--------|----------|
| Vertical Load | Spacing | (0.90) | (1.15) | Comments |
| 1 - Point (PLF) | 16.00" | 32.0 | 160.0 | ROOF |

| | | | Wind | |
|-------------------|-------------|---------|--------|------------------|
| Lateral Load | Location | Spacing | (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.

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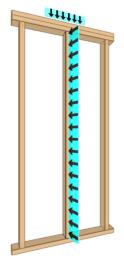


EXTERIOR NON-BEARING, GRID 1E, LEVEL 3 TYP 1 piece(s) 2 x 6 HF No.2 @ 16" OC

Wall Height: 9' 7 1/8"

Member Height: 9' 2 5/8"

O. C. Spacing: 16.00"



| 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 | Туре | | Material | System : Wall Member Type : Stud Building Code : IBC 2018 |
|---------------------|--------|----------|----------|---|
| Тор | Dbl 2X | | Hem Fir | |
| Base | 2X | | Hem Fir | Design Methodology : ASD |
| | | | | |
| Max Unbraced Length | | Comments | | |

Drawing is Conceptual

| Lateral Connections | | | | | | | | |
|---------------------|-----------|----------------------------|----------|-------------------|--|--|--|--|
| Supports | Connector | Type/Model | Quantity | Connector Nailing | | | | |
| Тор | 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 |

| | | | Wind | |
|-------------------|-------------|---------|--------|------------------|
| Lateral Load | Location | Spacing | (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.

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|--|-----------|
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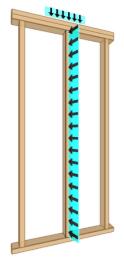


EXTERIOR NON-BEARING, GRID 1E, LEVEL 2 TYP 1 piece(s) 2 x 6 HF No.2 @ 16" OC

Wall Height: 9' 7 1/8"

Member Height: 9' 2 5/8"

O. C. Spacing: 16.00"



| 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 | Туре | | Material | System : Wall Member Type : Stud Building Code : IBC 2018 |
|-------------------|--------|--|----------|---|
| Тор | Dbl 2X | | Hem Fir | |
| Base | 2X | | Hem Fir | Design Methodology : ASD |
| | | | | □ |
| Max Unbraced Leng | ith | | Comments | |

Drawing is Conceptual

| Lateral Connections | | | | | | | |
|---------------------|-----------|----------------------------|----------|-------------------|--|--|--|
| Supports | Connector | Type/Model | Quantity | Connector Nailing | | | |
| Тор | 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.

| | | Dead | Floor Live | Snow | |
|-----------------|---------|--------|------------|--------|----------|
| Vertical Loads | Spacing | (0.90) | (1.00) | (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 |

| | | | Wind | | | | |
|---|-------------|---------|--------|------------------|--|--|--|
| Lateral Load | Location | Spacing | (1.60) | Comments | | | |
| 1 - Uniform (PLF) | Full Length | N/A | 64.0 | 37.9PSF * 1.33FT | | | |
| a IPC Table 1604.2 featnate fr. Deflection checks are performed using 420% of this lateral using load | | | | | | | |

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

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|--|-----------|
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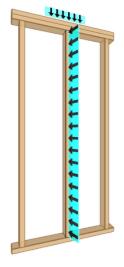


EXTERIOR NON-BEARING, GRID 1E, LEVEL 1 TYP 1 piece(s) 2 x 6 HF No.2 @ 16" OC

Wall Height: 9' 7 1/8"

Member Height: 9' 2 5/8"

O. C. Spacing: 16.00"



| 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 | Туре | | Material | System : Wall |
|---------------------|--------|--|----------|--|
| Тор | Dbl 2X | | Hem Fir | Member Type : Stud Building Code : IBC 2018 |
| Base | 2X | | Hem Fir | Design Methodology : ASD |
| | | | | |
| Max Unbraced Length | | | Comments | |

Drawing is Conceptual

| Lateral Connections | | | | | | | |
|---------------------|-----------|----------------------------|----------|-------------------|--|--|--|
| Supports | Connector | Type/Model | Quantity | Connector Nailing | | | |
| Тор | 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.

| | | Dead | Floor Live | Snow | |
|-----------------|---------|--------|------------|--------|----------|
| Vertical Loads | Spacing | (0.90) | (1.00) | (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 |

| | | | Wind | |
|-------------------|-------------|---------|--------|------------------|
| Lateral Load | Location | Spacing | (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 | |



9/6/2022 4:52:18 PM UTC ForteWEB v3.4, Engine: V8.2.2.122, Data: V8.1.3.0 File Name: Stud Walls Page 13 / 35



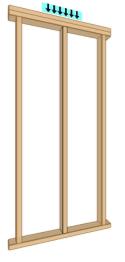
INTERIOR BEARING STAGGERED 2X4, GRID 12, LEVEL 4 (FOOTING STEP) 1 piece(s) 2 x 4 HF No.2 @ 16" OC

Wall Height: 9' 7 1/8"

Member Height: 9' 2 5/8"

O. C. Spacing: 16.00"

PASSED



| 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 | Туре | Material | System : Wall |
|----------|--------|-------------------|--|
| Тор | Dbl 2X | Douglas Fir-Larch | Member Type : Stud Building Code : IBC 2018 |
| Base | 2X | Douglas Fir-Larch | Design Methodology : ASD |
| | | | |

Drawing is Conceptual

| Max Unbraced Length | Comments |
|---------------------|----------|
| 1' | |
| | |

| | | Dead | Snow | |
|-----------------|---------|--------|--------|----------|
| Vertical Load | Spacing | (0.90) | (1.15) | Comments |
| 1 - Point (PLF) | 16.00" | 160.0 | 800.0 | ROOF |

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

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





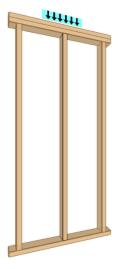
INTERIOR BEARING STAGGERED 2X4, GRID 12, LEVEL 3 (FOOTING STEP) 2 piece(s) 2 x 4 HF No.2 @ 16" OC

Wall Height: 9' 7 1/8"

Member Height: 9' 2 5/8"

O. C. Spacing: 16.00"

PASSED



| 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.

Max Unbraced Length

1'

• 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.

Comments

· A 15% increase in the moment capacity has been added to account for repetitive member usage

| Top Dbl 2X Douglas Fir-Larch Building Code : IBC 2 Base 2V Douglas Fir Larch Building Code : IBC 2 | Supports | Туре | Material | System : Wall |
|--|----------|--------|-------------------|--------------------------|
| Paso 2V Douglas Eir Larch | Тор | Dbl 2X | Douglas Fir-Larch | Member Type : Stud |
| | Base | 2X | Douglas Fir-Larch | Design Methodology : ASD |

Drawing is Conceptual

| | | | - | | |
|-----------------|---------|--------|------------|--------|----------|
| | | Dead | Floor Live | Snow | |
| Vertical Loads | Spacing | (0.90) | (1.00) | (1.15) | Comments |
| 1 - Point (PLF) | 16.00" | 160.0 | - | 800.0 | ROOF |
| 2 - Point (PLF) | 16.00" | 260.0 | 400.0 | - | LEVEL 4 |

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

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



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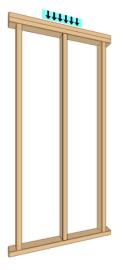
INTERIOR BEARING STAGGERED 2X4, GRID 12, LEVEL 3 (FOOTING STEP - ROOF DECK) 2 piece(s) 2 x 4 HF No.2 @ 16" OC

Wall Height: 9' 7 1/8"

Member Height: 9' 2 5/8"

O. C. Spacing: 16.00"

PASSED



| 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.

Comments

· A 15% increase in the moment capacity has been added to account for repetitive member usage

| Supports | Туре | Material | System : Wall |
|----------|--------|-------------------|----------------------------------|
| Тор | Dbl 2X | Douglas Fir-Larch | Member Type : Building Code : |
| Base | 2X | Douglas Fir-Larch | Design Methodo |

| Drawing | IS | Concept | tual | |
|---------|----|---------|------|--|
| | | | | |

Vortical

| | 1 | | | | |
|-------|---------|--------|------------|--------|----------|
| | | | | | |
| | | - | | | |
| | | Dead | Floor Live | Snow | |
| Loads | Spacing | (0.90) | (1.00) | (1.15) | Comments |
| | | | | | |

ember Type : Stud wilding Code : IBC 2018 esign Methodology : ASD

| Vertical Loads | | | | | |
|-----------------|--------|-------|-------|--------|----------------|
| 1 - Point (PLF) | 16.00" | - | - | - | ROOF |
| 2 - Point (PLF) | 16.00" | 260.0 | 600.0 | 1700.0 | LEVEL 4 (DECK) |
| | | | | | |

Max Unbraced Length

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| ForteWEB Software Operator |
|------------------------------|
| Samantha Taylor |
| Anthem Structural Engineers |
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| staylor@anthemstructural.com |
| |

Job Notes



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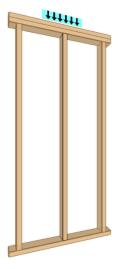
INTERIOR BEARING STAGGERED 2X4, GRID 12, LEVEL 2 (FOOTING STEP) 2 piece(s) 2 x 4 HF No.2 @ 16" OC

Wall Height: 9' 7 1/8"

Member Height: 9' 2 5/8"

O. C. Spacing: 16.00"

PASSED



| 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.

Max Linhang and Longeth

• 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 | Туре | Material | System : Wall Member Type : Stud Building Code : IBC 2018 |
|----------|--------|-------------------|---|
| Тор | Dbl 2X | Douglas Fir-Larch | |
| Base | 2X | Douglas Fir-Larch | Design Methodology : ASD |

Drawing is Conceptual

| Max Unbraced Length | Comments | |
|---------------------|----------|--|
| 1' | | |
| | | |
| | | |

| | | Dead | Floor Live | Snow | |
|-----------------|---------|--------|------------|--------|----------|
| Vertical Loads | Spacing | (0.90) | (1.00) | (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 |

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| orteWEB Software Operator | |
|--|--|
| Samantha Taylor Anthem Structural Engineers | |
| 303) 848-8497 | |
| taylor@anthemstructural.com | |





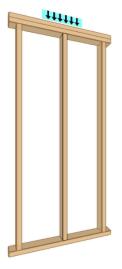
INTERIOR BEARING STAGGERED 2X4, GRID 12, LEVEL 1 (FOOTING STEP) 3 piece(s) 2 x 4 HF No.2 @ 16" OC

Wall Height: 9' 7 1/8"

Member Height: 9' 2 5/8"

O. C. Spacing: 16.00"

PASSED



| 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 | Туре | Material | System : Wall Member Type : Stud Building Code : IBC 2018 |
|----------|--------|-------------------|---|
| Тор | Dbl 2X | Douglas Fir-Larch | |
| Base | 2X | Douglas Fir-Larch | Design Methodology : ASD |

Drawing is Conceptual

| Max Unbraced Length | Comments |
|---------------------|----------|
| 1' | |

| | | Dead | Floor Live | Snow | |
|-----------------|---------|--------|------------|--------|----------|
| Vertical Loads | Spacing | (0.90) | (1.00) | (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 |

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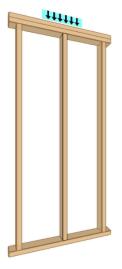
INTERIOR BEARING STAGGERED 2X4, GRID 12, LEVEL 1 (FOOTING STEP - ROOF DECK) 3 piece(s) 2 x 4 HF No.2 @ 16" OC

Wall Height: 9' 7 1/8"

Member Height: 9' 2 5/8"

O. C. Spacing: 16.00"

FAILED



| 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

| - | | | |
|------|--------|-------------------|--|
| Тор | Dbl 2X | | Member Type : Stud Building Code : IBC 2018 |
| Base | 2X | Douglas Fir-Larch | Design Methodology : ASD |

Drawing is Conceptual

| Max Unbraced Length | Comments |
|---------------------|----------|
| 1' | |
| | |

| | | Dead | Floor Live | Snow | |
|-----------------|---------|--------|------------|--------|----------------|
| Vertical Loads | Spacing | (0.90) | (1.00) | (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 |

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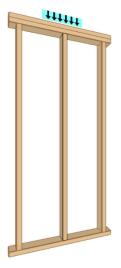
INTERIOR BEARING STAGGERED 2X4, GRID 17, LEVEL 4

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"



| 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 |

Comments

· 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 | Туре | Material | System : Wall | |
|----------|--------|-------------------|--|--|
| Тор | Dbl 2X | Douglas Fir-Larch | Member Type : Stud Building Code : IBC 2018 | |
| Base | 2X | Douglas Fir-Larch | Design Methodology : ASD | |

Drawing is Conceptual

| | | Dead | Snow | |
|-----------------|---------|--------|--------|----------|
| Vertical Load | Spacing | (0.90) | (1.15) | Comments |
| 1 - Point (PLF) | 8.00" | 288.0 | 1440.0 | ROOF |

Max Unbraced Length

1'

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|--|
| Samantha Taylor Anthem Structural Engineers (303) 848-8497 staylor@anthemstructural.com |
| |



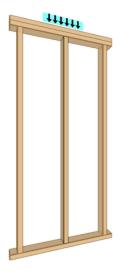


INTERIOR BEARING STAGGERED 2X4, GRID 17, LEVEL 3 2 piece(s) 2 x 4 HF No.2 @ 12" OC

Wall Height: 9' 7 1/8"

Member Height: 9' 2 5/8"

O. C. Spacing: 12.00"



| 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

| Top Dbl 2X Douglas Fir-Larch Building Code : IBC 2 Base 2V Douglas Fir Larch Building Code : IBC 2 | Supports | Туре | Material | System : Wall |
|--|----------|--------|-------------------|--------------------------|
| Paso 2V Douglas Eir Larch | Тор | Dbl 2X | Douglas Fir-Larch | Member Type : Stud |
| | Base | 2X | Douglas Fir-Larch | Design Methodology : ASD |

Drawing is Conceptual

| 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 |

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| ForteWEB Software Operator | |
|--|--|
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| staylor@anthemstructural.com | |





INTERIOR BEARING STAGGERED 2X4, GRID 17, LEVEL 2

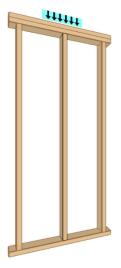
2 piece(s) 2 x 4 HF No.2 @ 8" OC

Wall Height: 9' 7 1/8"

Member Height: 9' 2 5/8"

O. C. Spacing: 8.00"

PASSED



| 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 | Туре | Material | System : Wall |
|----------|--------|-------------------|--|
| Тор | Dbl 2X | Douglas Fir-Larch | Member Type : Stud Building Code : IBC 2018 |
| Base | 2X | Douglas Fir-Larch | Design Methodology : ASD |
| | | | = 5 |

Drawing is Conceptual

| Max Unbraced Length | Comments |
|---------------------|----------|
| 1' | |
| | |

| | | Dead | Floor Live | Snow | |
|-----------------|---------|--------|------------|--------|----------|
| Vertical Loads | Spacing | (0.90) | (1.00) | (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 |

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

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





INTERIOR BEARING STAGGERED 2X4, GRID 17, LEVEL 1

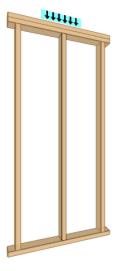
3 piece(s) 2 x 4 HF No.2 @ 8" OC

Wall Height: 9' 7 1/8"

Member Height: 9' 2 5/8"

O. C. Spacing: 8.00"

PASSED



| 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 | Туре | Material | System : Wall |
|----------|--------|-------------------|--|
| Тор | Dbl 2X | Douglas Fir-Larch | Member Type : Stud Building Code : IBC 2018 |
| Base | 2X | Douglas Fir-Larch | Design Methodology : ASD |

Drawing is Conceptual

| Max Unbraced Length | Comments |
|---------------------|----------|
| 1' | |

| | | Dead | Floor Live | Snow | |
|-----------------|---------|--------|------------|--------|----------|
| Vertical Loads | Spacing | (0.90) | (1.00) | (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 |

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





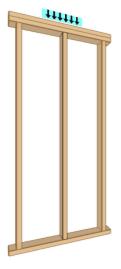
INTERIOR BEARING STAGGERED 2X4, GRID 17, LEVEL 1 (ROOF DECK) 3 piece(s) 2 x 4 HF No.2 @ 8" OC

Wall Height: 9' 7 1/8"

Member Height: 9' 2 5/8"

O. C. Spacing: 8.00"

PASSED



| 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

| | Туре | Material | System : Wall |
|------|--------|-------------------|--|
| Тор | Dbl 2X | Douglas Fir-Larch | Member Type : Stud Building Code : IBC 2018 |
| Base | 2X | Douglas Fir-Larch | Design Methodology : ASD |

Drawing is Conceptual

| Max Unbraced Length | Comments | | |
|---------------------|----------|--|--|
| 1' | | | |
| | | | |

| | | Dead | Floor Live | Snow | |
|-----------------|---------|--------|------------|--------|----------------|
| Vertical Loads | Spacing | (0.90) | (1.00) | (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 |

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| ForteWEB Software Operator | |
|--|--|
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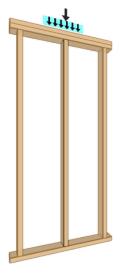
PASSED

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"



| 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 | Туре | Material | System : Wall |
|----------|--------|----------|--|
| Тор | Dbl 2X | Hem Fir | Member Type : Stud Building Code : IBC 2018 Design Methodology : ASD |
| Base | 2X | Hem Fir | |
| | | | |

Drawing is Conceptual

| Max Unbraced Length | Comments |
|---------------------|----------|
| 1' | |

| | | Dead | Floor Live | |
|-----------------|---------|--------|------------|----------|
| Vertical Loads | Spacing | (0.90) | (1.00) | Comments |
| 1 - Point (PLF) | 16.00" | 130.0 | 320.0 | LEVEL 2 |
| 2 - Point (lb) | N/A | 130 | 320 | LEVEL 3 |

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| ForteWEB Software Operator | |
|------------------------------|--|
| Samantha Taylor | |
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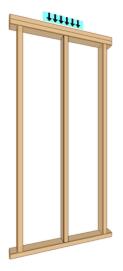


INTERIOR BEARING 2X6, GRID 17, LEVEL 4 1 piece(s) 2 x 6 HF No.2 @ 16" OC

Wall Height: 9' 7 1/8"

Member Height: 9' 2 5/8"

O. C. Spacing: 16.00"



| 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 |

Comments

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 | Туре | Material | System : Wall |
|----------|--------|-------------------|--|
| Тор | Dbl 2X | Douglas Fir-Larch | Member Type : Stud Building Code : IBC 2018 |
| Base | 2X | Douglas Fir-Larch | Design Methodology : ASD |
| | | | 9, 9, |

Drawing is Conceptual

| | | Dead | Snow | |
|-----------------|---------|--------|--------|----------|
| Vertical Load | Spacing | (0.90) | (1.15) | Comments |
| 1 - Point (PLF) | 16.00" | 288.0 | 1440.0 | ROOF |

Max Unbraced Length

1

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



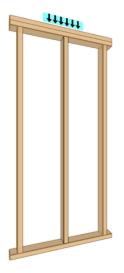


INTERIOR BEARING 2X6, GRID 17, LEVEL 3 1 piece(s) 2 x 6 HF No.2 @ 16" OC

Wall Height: 9' 7 1/8"

Member Height: 9' 2 5/8"

O. C. Spacing: 16.00"



| 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 |

Comments

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.

Max Unbraced Length

1'

• 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 | Туре | Material | System : W |
|----------|--------|-------------------|--------------------------|
| Тор | Dbl 2X | Douglas Fir-Larch | Member Ty Building Co |
| Base | 2X | Douglas Fir-Larch | Design Met |
| | | | |

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

Drawing is Conceptual

| 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 |

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| ForteWEB Software Operator |
|--|
| Samantha Taylor Anthem Structural Engineers |
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| staylor@anthemstructural.com |





INTERIOR BEARING 2X6, GRID 17, LEVEL 3 (ROOF DECK)

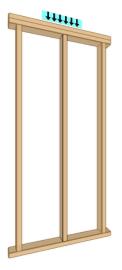
2 piece(s) 2 x 6 HF No.2 @ 16" OC

Wall Height: 9' 7 1/8"

Member Height: 9' 2 5/8"

O. C. Spacing: 16.00"

PASSED



| 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.

Max Unbraced Length

1'

• 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.

Comments

· A 15% increase in the moment capacity has been added to account for repetitive member usage

| Top Dbl 2X | Douglas Fir-Larch | Member Type : Stud Building Code : IBC 2018 |
|------------|-------------------|--|
| Base 2X | Douglas Fir-Larch | Design Methodology : ASD |

Drawing is Conceptual

| | | Dead | Floor Live | Snow | |
|-----------------|---------|--------|------------|--------|----------------|
| Vertical Loads | Spacing | (0.90) | (1.00) | (1.15) | Comments |
| 1 - Point (PLF) | 16.00" | - | - | - | ROOF |
| 2 - Point (PLF) | 16.00" | 468.0 | 1080.0 | 3060.0 | LEVEL 4 (DECK) |

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



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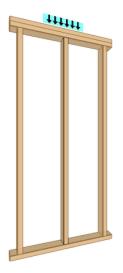
PASSED

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"



| 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 | Туре | Material | System : Wall |
|----------|--------|-------------------|--|
| Тор | Dbl 2X | Douglas Fir-Larch | Member Type : Stud Building Code : IBC 2018 |
| Base | 2X | Douglas Fir-Larch | Design Methodology : ASD |

Drawing is Conceptual

| Max Unbraced Length | Comments |
|---------------------|----------|
| 1' | |

| | | Dead | Floor Live | Snow | |
|-----------------|---------|--------|------------|--------|----------|
| Vertical Loads | Spacing | (0.90) | (1.00) | (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 |

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





INTERIOR BEARING 2X6, GRID 17, LEVEL 2 (ROOF DECK)

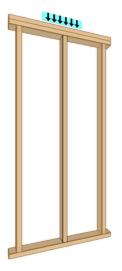
2 piece(s) 2 x 6 HF No.2 @ 16" OC

Wall Height: 9' 7 1/8"

Member Height: 9' 2 5/8"

O. C. Spacing: 16.00"

PASSED



| Actual | Allowed | Result | LDF | Load: Combination |
|-----------------|---|--|--|--|
| 20 | 50 | Passed (40%) | | |
| 7548 | 13236 | Passed (57%) | 1.15 | 1.0 D + 0.75 L + 0.75 S |
| 7548 | 11602 | Passed (65%) | | 1.0 D + 0.75 L + 0.75 S |
| 0 | | | | N/A |
| 0 | N/A | Passed (N/A) | | N/A |
| 0 @ mid-span | N/A | Passed (N/A) | | N/A |
| 0.10 @ mid-span | 0.92 | Passed (L/1101) | | 1.0 D + 0.75 L + 0.75 S |
| 0.99 | 1 | Passed (99%) | 1.15 | 1.0 D + 0.75 L + 0.75 S |
| | 20 7548 7548 0 0 0 @ mid-span 0.10 @ mid-span | 20 50 7548 13236 7548 11602 0 0 N/A 0@mid-span N/A 0.10@mid-span 0.92 | 20 50 Passed (40%) 7548 13236 Passed (57%) 7548 11602 Passed (65%) 0 0 N/A Passed (N/A) 0@mid-span N/A Passed (N/A) 0.10@mid-span 0.92 Passed (L/1101) | 20 50 Passed (40%) 7548 13236 Passed (57%) 1.15 7548 11602 Passed (65%) 0 0 N/A Passed (N/A) 0@mid-span N/A Passed (N/A) 0.10@mid-span 0.92 Passed (L/1101) |

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 | Туре | Material | System : Wall |
|----------|--------|-------------------|--|
| Тор | Dbl 2X | Douglas Fir-Larch | Member Type : Stud Building Code : IBC 2018 |
| Base | 2X | Douglas Fir-Larch | Design Methodology : ASD |

Drawing is Conceptual

| Max | Unbraced Lei | ngth | | Comments | | | |
|-----|--------------|--------------|---|----------|--|--|--|
| 1' | | | | | | | |
| | | | | | | | |
| | | F1 11 | 0 | | | | |

| | | Dead | Floor Live | Snow | |
|-----------------|---------|--------|------------|--------|---------------------|
| Vertical Loads | Spacing | (0.90) | (1.00) | (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 |

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| orteWEB Software Operator |
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| taylor@anthemstructural.com |





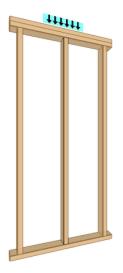
PASSED

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"



| 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 | Туре | Material | System : Wall |
|----------|--------|-------------------|--|
| Тор | Dbl 2X | Douglas Fir-Larch | Member Type : Stud Building Code : IBC 2018 Design Methodology : ASD |
| Base | 2X | Douglas Fir-Larch | |

Drawing is Conceptual

| Max Unbraced Length | Comments |
|---------------------|----------|
| 1' | |

| | | Dead | Floor Live | Snow | |
|-----------------|---------|--------|------------|--------|----------|
| Vertical Loads | Spacing | (0.90) | (1.00) | (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 |

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|--|---|
| Samantha Taylor Anthem Structural Engineers (303) 848-8497 staylor@anthemstructural.com | |





INTERIOR BEARING 2X6, GRID 17, LEVEL 1 (ROOF DECK)

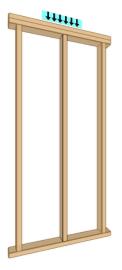
2 piece(s) 2 x 6 HF No.2 @ 16" OC

Wall Height: 9' 7 1/8"

Member Height: 9' 2 5/8"

O. C. Spacing: 16.00"

PASSED



| 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 | Туре | Material | System : Wall |
|----------|--------|-------------------|--|
| Тор | Dbl 2X | Douglas Fir-Larch | Member Type : Stud Building Code : IBC 2018 |
| Base | 2X | Douglas Fir-Larch | Design Methodology : ASD |

Drawing is Conceptual

| Max Unbraced Length | Comments |
|---------------------|----------|
| 1' | |

| | | Dead | Floor Live | Snow | |
|-----------------|---------|--------|------------|--------|----------------|
| Vertical Loads | Spacing | (0.90) | (1.00) | (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 |

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| staylor@anthemstructural.com | |

Job Notes



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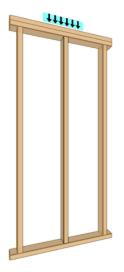


INTERIOR BEARING 2X6, STAIR WALL, LEVEL 1 1 piece(s) 2 x 4 HF No.2 @ 16" OC

Wall Height: 10' 3"

Member Height: 9' 10 1/2"

O. C. Spacing: 16.00"



| 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 | Туре | Material | System : Wall |
|----------|--------|----------|--|
| Тор | Dbl 2X | Hem Fir | Member Type : Stud Building Code : IBC 2018 |
| Base | 2X | Hem Fir | Design Methodology : ASD |
| | | | 5 |

Drawing is Conceptual

| Max Unbraced Length | Comments |
|---------------------|----------|
| 1' | |
| | |

| | | | Floor Live | |
|-----------------|---------|--------|------------|----------|
| Vertical Load | Spacing | (0.90) | (1.00) | Comments |
| 1 - Point (PLF) | 16.00" | 130.0 | 320.0 | LEVEL 2 |

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| |





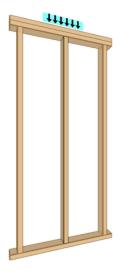
PASSED

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"



| Actual | Allowed | Result | LDF | Load: Combination |
|-----------------|---|---|---|---|
| 20 | 50 | Passed (40%) | | |
| 4235 | 13236 | Passed (32%) | 1.15 | 1.0 D + 0.75 L + 0.75 S |
| 4235 | 11602 | Passed (37%) | | 1.0 D + 0.75 L + 0.75 S |
| 0 | | | | N/A |
| 0 | N/A | Passed (N/A) | | N/A |
| 0 @ mid-span | N/A | Passed (N/A) | | N/A |
| 0.06 @ mid-span | 0.92 | Passed (L/1963) | | 1.0 D + 0.75 L + 0.75 S |
| 0.36 | 1 | Passed (36%) | 1.15 | 1.0 D + 0.75 L + 0.75 S |
| | 20 4235 4235 0 0 0 @ mid-span 0.06 @ mid-span | 20 50 4235 13236 4235 11602 0 0 N/A 0@mid-span N/A 0.06 @mid-span 0.92 | 20 50 Passed (40%) 4235 13236 Passed (32%) 4235 11602 Passed (37%) 0 0 N/A Passed (N/A) 0@mid-span N/A Passed (N/A) 0.06 @mid-span 0.92 Passed (L/1963) | 20 50 Passed (40%) 4235 13236 Passed (32%) 1.15 4235 11602 Passed (37%) 0 0 Passed (N/A) 0 N/A Passed (N/A) 0@mid-span N/A Passed (N/A) 0.06@mid-span 0.92 Passed (L/1963) |

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 | Туре | Material System : Wall | | | |
|----------|--------|---|-------------------------|--|--|
| Тор | Dbl 2X | Douglas Fir-Larch Member Type : Stud Building Code : IBC 2 | | | |
| Base | 2X | Douglas Fir-Larch | Design Methodology : AS | | |

Drawing is Conceptual

| 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) |

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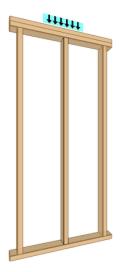
PASSED

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"



| 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 | Туре | Material System : Wall Douglas Fir-Larch Building Code : IRC | |
|----------|--------|--|--|
| Тор | Dbl 2X | | |
| Base | 2X | Douglas Fir-Larch | Building Code : IBC 2018 Design Methodology : ASD |

Drawing is Conceptual

| Max Unbraced Length | Comments |
|---------------------|----------|
| 1' | |

| | | Dead | Floor Live | Snow | |
|-----------------|---------|--------|------------|--------|----------|
| Vertical Loads | Spacing | (0.90) | (1.00) | (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 |

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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 |
|----|------------------------|---|----------|-------|-----------------|-------------|---------------|
| La | amimated 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

| SI | 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 |
|----|-----------|----------------|------------|-----------------|------------------|------------------------------|------------------|-------------------------------|----------------------|----------------------------|
| Ν | 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

| N | Member Type | Lumber Species | Width | Depth | Number of Plies | Member ID | Lumber Finish |
|--------|--------------------|---|----------|-------|-----------------|-------------------|---------------|
| Lamima | ated Veneer Lumber | DF/SP (Douglas fir or Southern Pine) | 1 3/4" x | 14" | 2 | Beam #6, Beam #13 | No |

Joist

| Me | lember Type | Lumber Species | Width | Depth | Number of Plies | Member ID | Lumber Finish | Download (ASD) | Uplift (ASD) |
|----|-------------|----------------|-------------|--------------|-----------------|-----------|---------------|----------------|--------------|
| S | Solid Sawn | HF (Hem Fir) | 2x (1 1/2") | 12 (11 1/4") | 2 | Beam #5 | No | 282 | 333 |

Hanger Options

| SI | 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 |
|----|-----------|----------------|------------|-----------------|------------------|------------------------------|------------------|-------------------------------|----------------------|----------------------------|
| Ν | 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

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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

| Me | ember Type | Lumber Species | Width | Depth | Number of Plies | Member ID | Lumber Finish |
|----------|------------------|---|----------|-------|-----------------|-------------------|---------------|
| Lamimate | ed 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) |
|-----------------|----------------|----------------|-------|--------|---------|----------|--------------|----------------|-----------------|----------------|--------------|
| | <u>HHUS410</u> | Lowest | 3.625 | 9 | 3 | - | - | 30-10d | 10-10d | 5330 | 2425 |
| | <u>HHUS410</u> | +2.00% | 3.625 | 9 | 3 | - | - | 30-16d | 10-16d | 6440 | 3550 |
| | <u>HU412</u> | +51.00% | 3.563 | 10.344 | 2.5 | - | - | 22-16d | 10-10d | 3695 | 1780 |
| A second second | <u>HUC412</u> | +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 |
| Second B. | <u>HUS412</u> | +63.00% | 3.563 | 10.563 | 2 | - | - | 10-SD10212 | 10-SD10212 | 3820 | 2830 |
| | <u>HU414</u> | +80.00% | 3.563 | 11.906 | 2.5 | - | - | 24-10d | 12-10d | 3420 | 1780 |
| | <u>HUC414</u> | +80.00% | 3.563 | 11.906 | 2.5 | - | - | 24-10d | 12-10d | 3420 | 1780 |

Table Notes

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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) |
|-----------------------|----------------|-------|--------|---------|----------|--------------|----------------|-----------------|----------------|--------------|
| <u>IUS2.37/14*</u> | Lowest | 2.438 | 13.969 | 2 | - | - | 12-10d | 2-Strong-Grip | 1615 | 70 |
| <u>IUS2.37/14*</u> | +2.00% | 2.438 | 13.969 | 2 | - | - | 14-10d | 2-Strong-Grip | 1805 | 70 |
| <u>IUS2.37/14*</u> | +2.00% | 2.438 | 13.969 | 2 | - | - | 12-10dx1.5 | 2-Strong-Grip | 1325 | 70 |
| <u>IUS2.37/14*</u> | +4.00% | 2.438 | 13.969 | 2 | - | - | 14-10dx1.5 | 2-Strong-Grip | 1450 | 70 |
| <u>ITS2.37/14</u> | +8.00% | 2.438 | 13.938 | 2 | 1.438 | - | 2-10d | 2-Strong-Grip | 1550 | 120 |
| <u>ITS2.37/14</u> | +8.00% | 2.438 | 13.938 | 2 | 1.438 | - | 2-16d | 2-Strong-Grip | 1785 | 120 |
| <u>ITS2.37/14</u> | +9.00% | 2.438 | 13.938 | 2 | 1.438 | - | 2-10dx1.5 | 2-Strong-Grip | 1395 | 120 |
| <u>IUS2.37/9.5*</u> | +9.00% | 2.438 | 9.469 | 2 | - | - | 8-10d | 2-10dx1.5 | 1075 | 345 |
| <u>IUS2.37/9.5*</u> | +10.00% | 2.438 | 9.469 | 2 | - | - | 8-10dx1.5 | 2-10dx1.5 | 885 | 345 |
| <u>IUS2.37/11.88*</u> | +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

| N | Member Type | Lumber Species | Width | Depth | Number of Plies | Member ID | Lumber Finish |
|--------|--------------------|---|----------|-------|-----------------|-----------|---------------|
| Lamima | ated Veneer Lumber | DF/SP (Douglas fir or Southern Pine) | 1 3/4" x | 14" | 2 | Beam #12 | No |

Joist

| Member T | pe Lumb | ber Species | Width | Depth | Number of Plies | Member ID | Lumber Finish | Download (ASD) | Uplift (ASD) |
|----------|---------|--------------|--------|-------|-----------------|-----------|---------------|----------------|--------------|
| I-Joist | DF (D | 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) |
|-----------------------|----------------|-------|--------|---------|----------|--------------|----------------|-----------------|----------------|--------------|
| <u>IUS1.81/14*</u> | Lowest | 1.875 | 13.969 | 2 | - | - | 12-10d | 2-Strong-Grip | 1615 | 70 |
| <u>ITS1.81/14</u> | +2.00% | 1.875 | 13.938 | 2 | 1.375 | - | 2-10d | 2-Strong-Grip | 1550 | 120 |
| <u>IUS1.81/14*</u> | +2.00% | 1.875 | 13.969 | 2 | - | - | 12-10dx1.5 | 2-Strong-Grip | 1325 | 70 |
| <u>IUS1.81/14*</u> | +2.00% | 1.875 | 13.969 | 2 | - | - | 14-10d | 2-Strong-Grip | 1805 | 70 |
| <u>IUS1.81/9.5*</u> | +2.00% | 1.875 | 9.469 | 2 | - | - | 8-10d | 2-10dx1.5 | 1075 | 345 |
| <u>ITS1.81/14</u> | +3.00% | 1.875 | 13.938 | 2 | 1.375 | - | 2-16d | 2-Strong-Grip | 1785 | 120 |
| <u>ITS1.81/14</u> | +3.00% | 1.875 | 13.938 | 2 | 1.375 | - | 2-10dx1.5 | 2-Strong-Grip | 1395 | 120 |
| <u>IUS1.81/9.5*</u> | +3.00% | 1.875 | 9.469 | 2 | - | - | 8-10dx1.5 | 2-10dx1.5 | 885 | 345 |
| <u>IUS1.81/14*</u> | +4.00% | 1.875 | 13.969 | 2 | - | - | 14-10dx1.5 | 2-Strong-Grip | 1450 | 70 |
| <u>IUS1.81/11.88*</u> | +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

| SI | 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 |
|----|-----------|----------------|------------|-----------------|------------------|------------------------------|------------------|-------------------------------|----------------------|----------------------------|
| Ν | 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 |
| <u>U210-2</u> | +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 |
| <u>HUS28-2</u> | +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 |
| <u>HUS212-2</u> | +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:

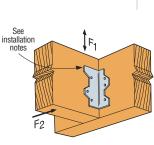
- Use all specified fasteners; see General Notes
- LS Field skewable; bend one time only
- Joist must be constrained against rotation (for example, with solid blocking) when using a single LS per connection
- Nail the L angle's wider leg into the joist to ensure table loads and allow correct nailing

These products are available with additional corrosion

protection. For more information, see p. 14.

Codes: See p. 11 for Code Reference Key Chart





Typical L50 Installation

SS For stainless-steel fasteners, see p.21.

L90

L70

. L50

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 | | | | |
|--------------|--------------------|------------|---------------------------------|-----------------------|---------------|---------------|---------------------------|------------------------|---------------|---------------|---------------------------|--------------|
| | | | | Floor (100) | Snow (115) | Roof (125) | Wind/ Seismic (160) | Floor (100) | Snow (115) | Roof (125) | Wind/ Seismic (160) | Code Ref. |
| GA1 | (4) 0.148 x 11/2 | 2¾ | F ₁ , F ₂ | 235 | 270 | 290 | 350 | 200 | 230 | 250 | 300 | IBC, FL, LA |
| | (4) #9 x 1 1⁄2" SD | | F1 | 340 | 375 | 375 | 375 | 225 | 260 | 280 | 325 | IBC, LA |
| | (4) #9 x 1 1⁄2" SD | | F ₂ | 340 | 395 | 430 | 435 | 225 | 260 | 280 | 360 | |
| | (6) 0.148 x 1 1/2 | 31⁄4 | F1, F2 | 355 | 405 | 435 | 550 | 305 | 350 | 375 | 475 | IBC, FL, LA |
| GA2 | (6) #9 x 1 1/2" SD | | F1 | 515 | 590 | 640 | 695 | 335 | 385 | 420 | 540 | IBC, LA |
| | (6) #9 x 11/2" SD | | F ₂ | 515 | 590 | 640 | 820 | 335 | 385 | 420 | 540 | |
| 1.00 | (4) 0.148 x 1 ½ | 3 | F1 | 245 | 250 | 250 | 250 | 210 | 215 | 215 | 215 | |
| L30 | | | F ₂ | 245 | 275 | 295 | 370 | 210 | 235 | 255 | 320 | |
| 150 | (6) 0.148 x 1½ | 5 | F1 | 365 | 415 | 445 | 525 | 315 | 355 | 385 | 450 | |
| L50 | | | F ₂ | 365 | 415 | 445 | 555 | 315 | 355 | 385 | 475 | |
| L70 | (8) 0.148 x 1 1/2 | 7 | F ₁ , F ₂ | 485 | 550 | 595 | 740 | 415 | 475 | 510 | 635 | |
| L90 | (10) 0.148 x 11/2 | 9 | F ₁ , F ₂ | 610 | 690 | 740 | 925 | 525 | 595 | 635 | 795 | 1 |
| 1000 | (6) 0.148 x 1 1/2 | 3% | F1 | 320 | 320 | 320 | 320 | 275 | 275 | 275 | 275 | IBC, |
| LS30 | (6) 0.148 x 3 | | F1 | 355 | 395 | 395 | 395 | 305 | 340 | 340 | 340 | FL, LA |
| LS50 | (8) 0.148 x 1 1/2 | 41⁄8 | F ₁ | 475 | 540 | 560 | 560 | 410 | 465 | 480 | 480 |] |
| L300 | (8) 0.148 x 3 | | F1 | 475 | 540 | 580 | 730 | 410 | 465 | 500 | 630 | |
| LS70 | (10) 0.148 x 11⁄2 | 6¾ | F1 | 590 | 645 | 645 | 645 | 510 | 555 | 555 | 555 | |
| L3/0 | (10) 0.148 x 3 | | F1 | 590 | 675 | 725 | 915 | 510 | 580 | 625 | 785 | |
| LS90 | (12) 0.148 x 11/2 | 71⁄8 | F ₁ | 710 | 805 | 870 | 890 | 610 | 690 | 750 | 765 |] |
| L290 | (12) 0.148 x 3 | | F1 | 710 | 805 | 870 | 1,040 | 610 | 690 | 750 | 895 |] |

1. GA angles may be installed with 0.148" x 3" nails.

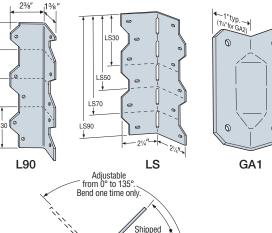
2. GA1 uplift is 425 lb. for DF and 300 lb. for SPF when installed with Strong-Tie® SD Connector screws.

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 F2 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



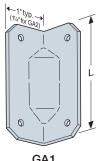
at 45°

LS Top View

Uplift

Typical GA Installation

SD



Typical LS70 Installation

Straps and Ties



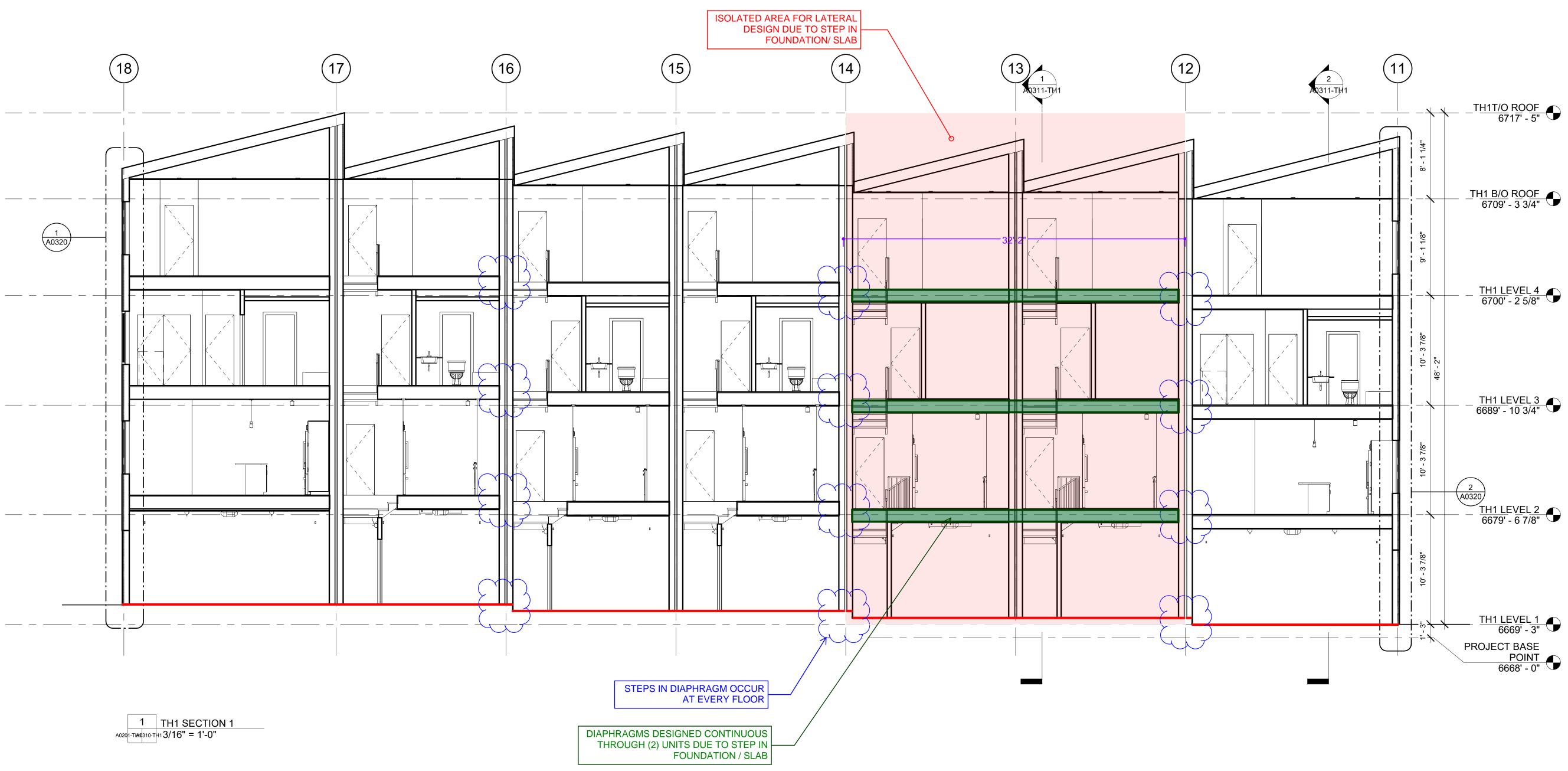
LATERAL DESIGN

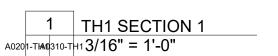
ASD LOAD SUMMARY:

SEISMIC: V = 18.6 K $V_{ISOLATED UNIT} = 4.5 K$

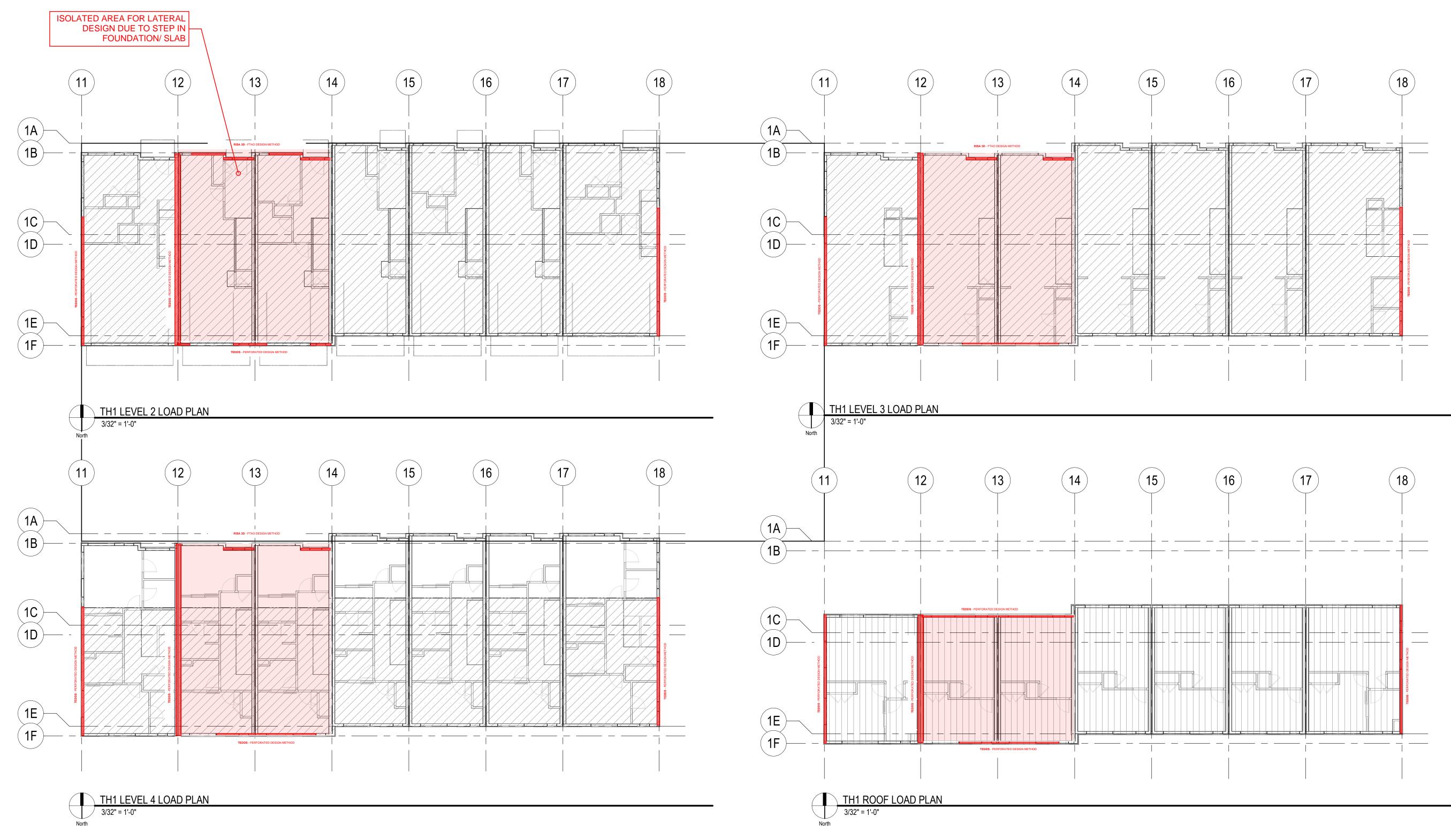
WIND: $V_{N-S} = 61.0 \text{ K}$ wind governs N-S direction $V_{E-W} = 17.3 \text{ K}$ Visolated group = 17.3 K wind governs E-W direction

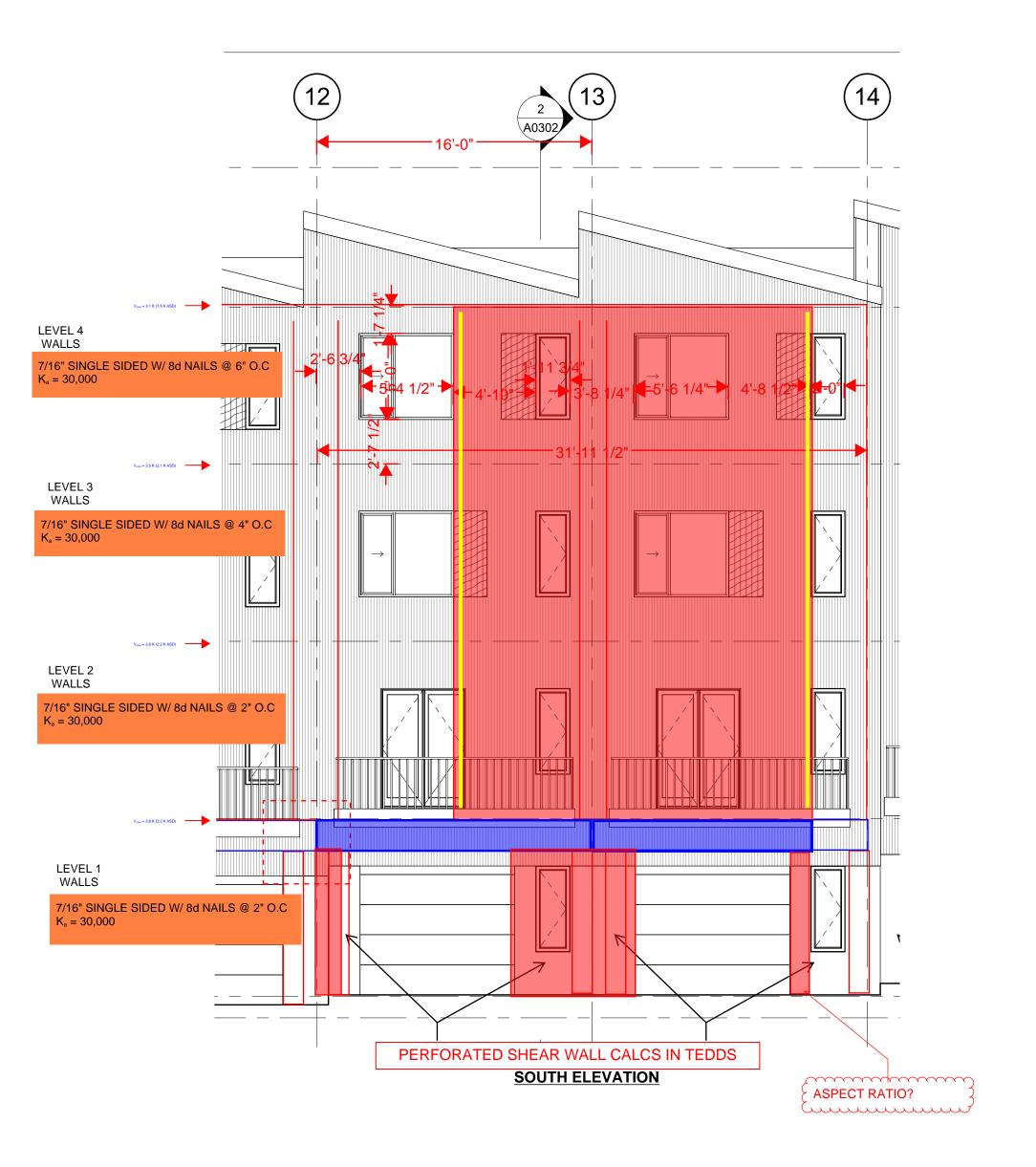
STEPS IN DIAPHRAGM/ FOUNDATION: TH1





SHEAR WALL KEY PLAN: TH1







WIND LOADS

WIND LOADING FROM CODE SEARCH:

Wind Pressure Coefficients

| | CASE A | | | CASE B | | | | |
|---------|--------|------------|---------|--------|-------|---------|---------|--|
| | | 8 = 14 deg | | | | | | |
| Surface | GCpf | wł-GCpi | wł+GCpi | | GCpf | wł-GCpi | wł+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 = Leeward parapet =

34.4 psf (GCpn = +1.5) -22.9 psf (GCpn = -1.0)

Horizontal MWFRS Simple Diaphragm Pressures (psf)

| ransverse 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).

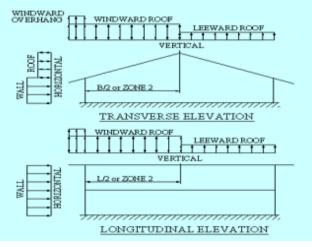
The code requires the MWFRS be designed for a min ultimate force of 16 psf multiplied by the wall area plus an 8 psf force applied to the vertical projection of the roof. Windward roof overhangs =

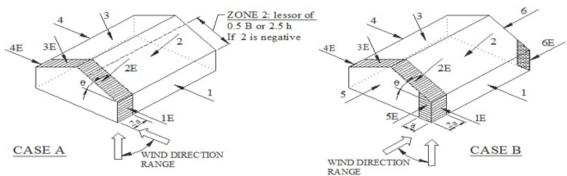
overnangs =

Zone Zhengin -

17.2 psf (upward) add to windward roof pressure

20.0 10

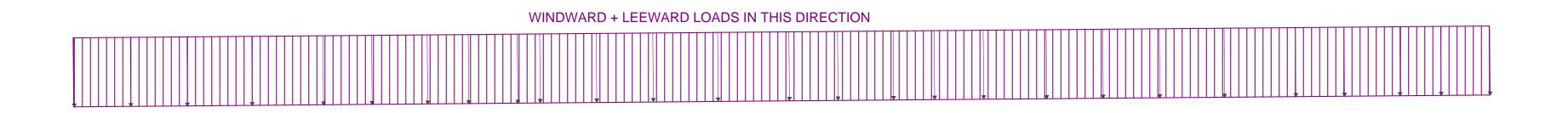


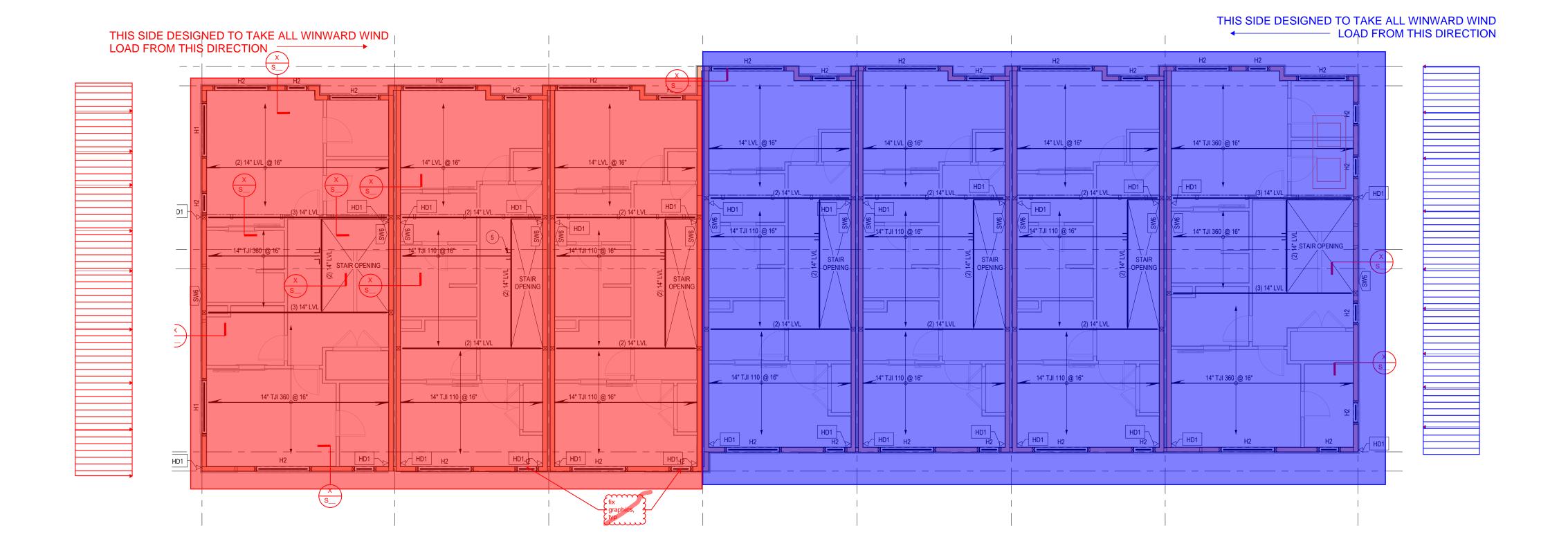


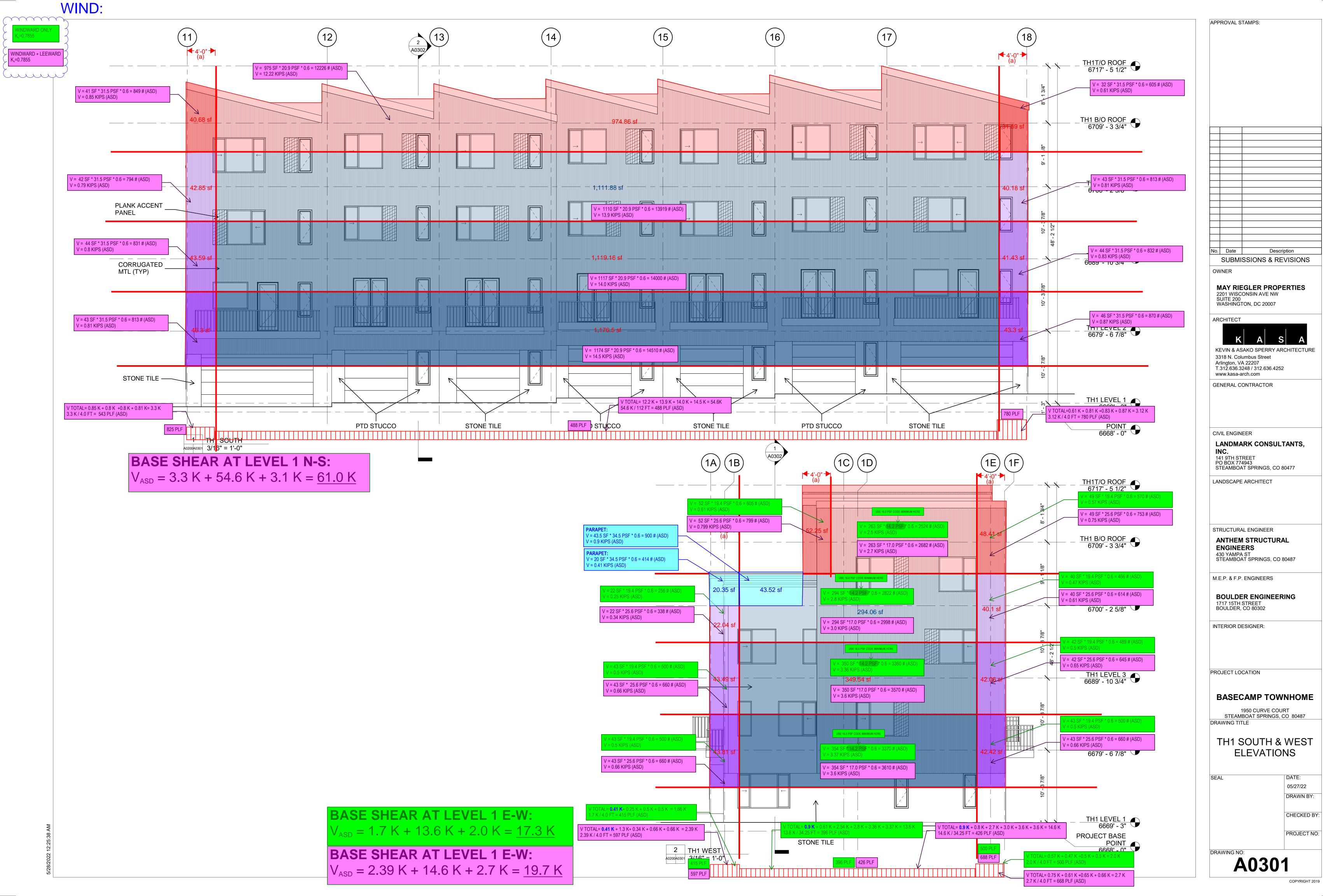
NOTE: Torsional loads are 25% of zones 1 - 6. See code for loading diagram. Exception: One story buildings h<30' and 1 to 2 storybuildings framed with light-frame construction or with flexible diaphragms need not be designed for the torsional load case.

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

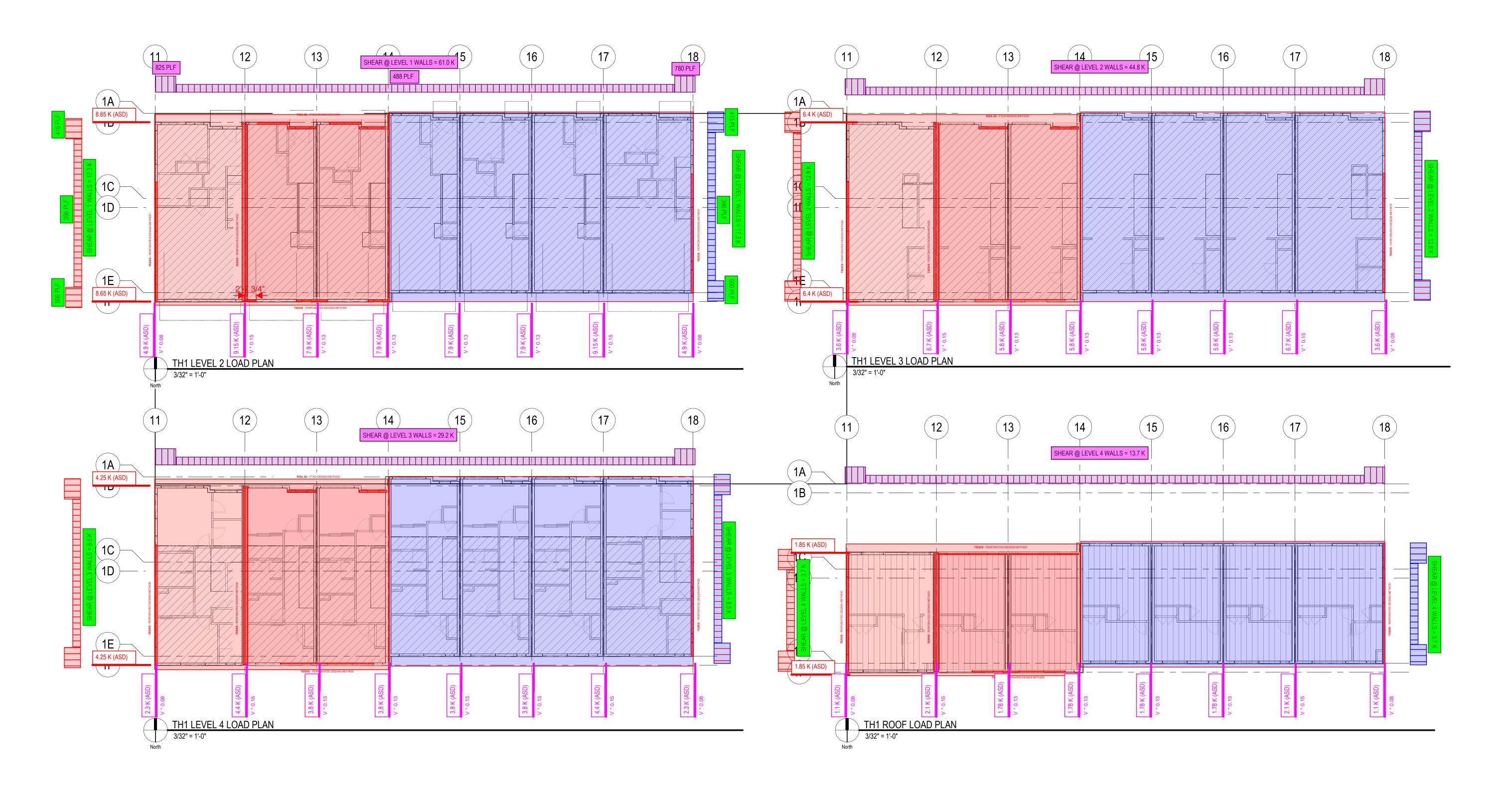
WIND LOAD JUSTIFICATION:







WIND LOAD DISTRIBUTION TH1



SEISMIC LOADS

| Seismic | |
|------------------|-------|
| | 0.597 |
| Ss | |
| S ₁ | 0.103 |
| Fa | 1.261 |
| Fv | 1.5 |
| S _{MS} | 0.753 |
| S _{M1} | 0.155 |
| S _{DS} | 0.502 |
| S _{D1} | 0.103 |
| TL | 4 |
| PGA | 0.419 |
| PGA _M | 0.502 |
| F _{PGA} | 1.2 |
| le | 1 |
| Cv | 0.998 |

 $\begin{array}{l} \label{eq:period} \text{PER ROUTT COUNTY POLICY: (see next page)} \\ S_{\text{DS MAX}} = 0.333 \text{ (need to update F, in code search)} \\ S_{\text{D1 MAX}} = 0.133 \text{ (see nk)} \end{array}$

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

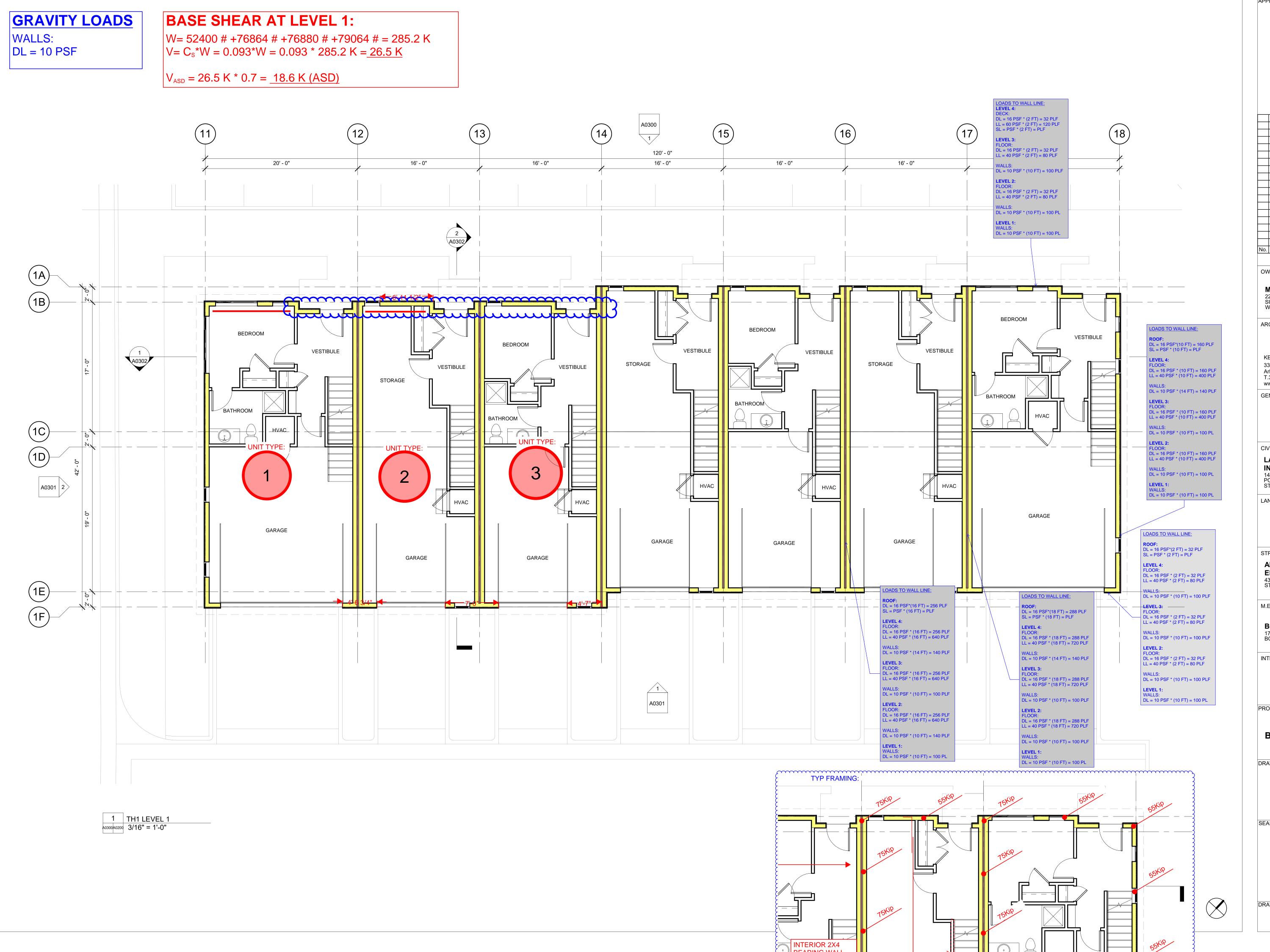
 Table R301.2(1) CLIMATIC AND GEOGRAPHIC DESIGN CRITERIA, is completed as follows:

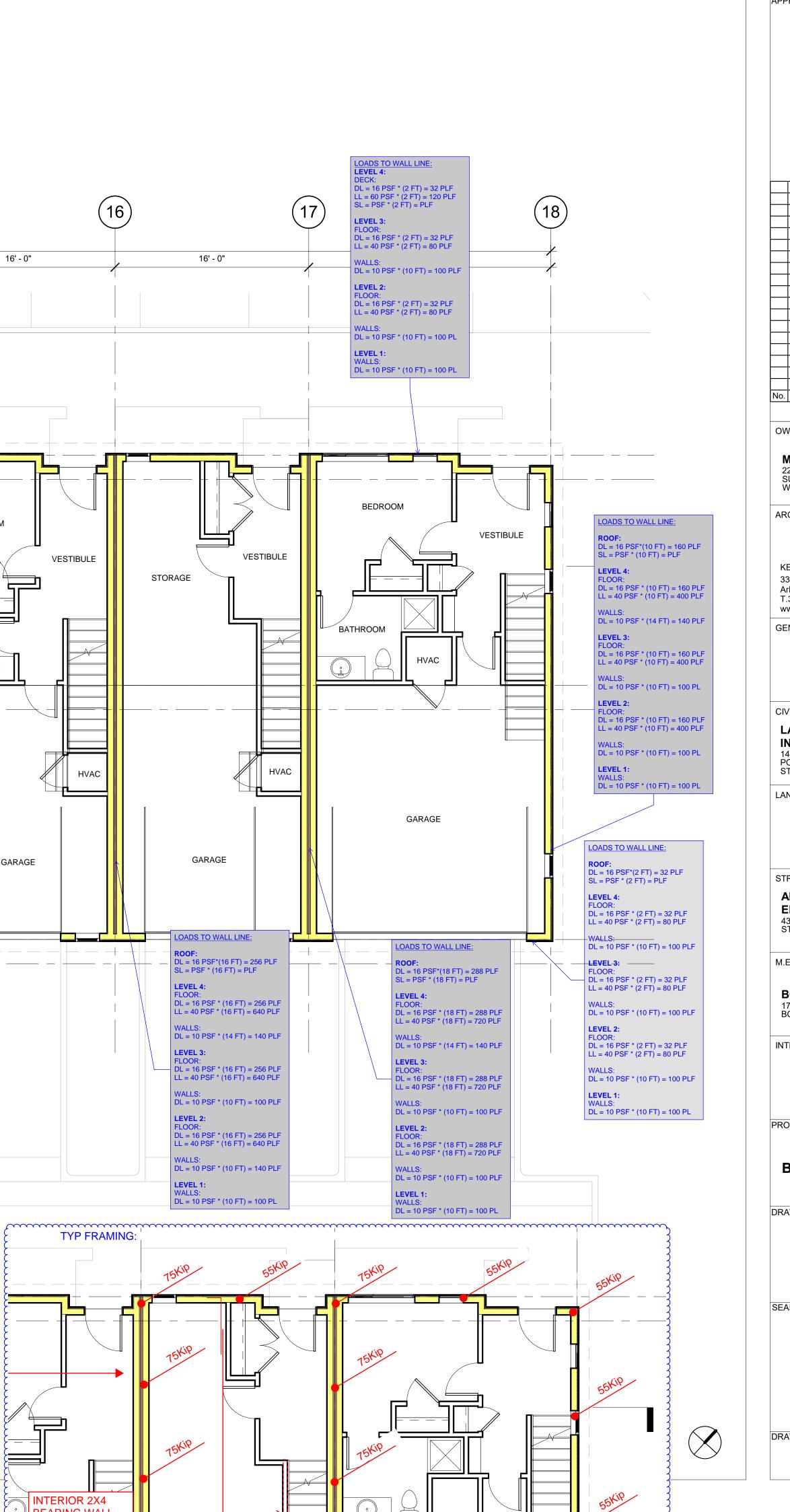
- Ground Snow Load Case Study Area contact the Building Department for Ground Snow Load Valuations per site.
- Climate Zone 7
- Wind Speed 115 MPH (ultimate design wind speed)
- Topographic Effects No
- Seismic Design Category C Note: When approved by the Structural Engineer of Record through review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

ROUTT County Regional Building Department

136 6th Street, Ste 201, Steamboat Springs, CO 80487 PH: 970-870-5566 Fax 970-870-5489 Email: Building@co.routt.co.us

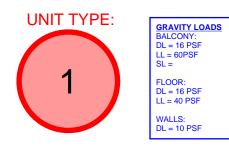
SEISMIC:

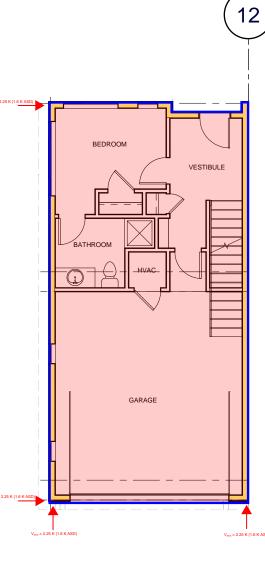




| No. Date Des SUBMISSIONS & R | cription EVISIONS |
|---|---|
| OWNER | |
| MAY RIEGLER PRO | PERTIES |
| 2201 WISCONSIN AVE NW SUITE 200 WASHINGTON, DC 20007 | |
| ARCHITECT | |
| | |
| KEVIN & ASAKO SPERRY A | |
| 3318 N. Columbus Street Arlington, VA 22207 T.312.636.3248 / 312.636.42 | 52 |
| GENERAL CONTRACTOR | |
| JENERAL CONTRACTOR | |
| | |
| 141 9TH STREET PO BOX 774943 STEAMBOAT SPRINGS, CO LANDSCAPE ARCHITECT | 0 80477 |
| STRUCTURAL ENGINEER ANTHEM STRUCTUR ENGINEERS 430 YAMPA ST | |
| STEAMBOAT SPRINGS, CO | |
| STEAMBOAT SPRINGS, CO M.E.P. & F.P. ENGINEERS | |
| M.E.P. & F.P. ENGINEERS | RING |
| | RING |
| M.E.P. & F.P. ENGINEERS BOULDER ENGINEE 1717 15TH STREET | RING |
| M.E.P. & F.P. ENGINEERS BOULDER ENGINEE 1717 15TH STREET BOULDER, CO 80302 | RING |
| M.E.P. & F.P. ENGINEERS BOULDER ENGINEE 1717 15TH STREET BOULDER, CO 80302 | RING |
| M.E.P. & F.P. ENGINEERS BOULDER ENGINEE 1717 15TH STREET BOULDER, CO 80302 INTERIOR DESIGNER: | RING |
| M.E.P. & F.P. ENGINEERS BOULDER ENGINEE 1717 15TH STREET BOULDER, CO 80302 INTERIOR DESIGNER: | RING |
| M.E.P. & F.P. ENGINEERS BOULDER ENGINEE 1717 15TH STREET BOULDER, CO 80302 INTERIOR DESIGNER: | |
| M.E.P. & F.P. ENGINEERS BOULDER ENGINEE 1717 15TH STREET BOULDER, CO 80302 INTERIOR DESIGNER: PROJECT LOCATION | VNHOME URT |
| M.E.P. & F.P. ENGINEERS BOULDER ENGINEE 1717 15TH STREET BOULDER, CO 80302 INTERIOR DESIGNER: PROJECT LOCATION BASECAMP TOV 1950 CURVE CO STEAMBOAT SPRINGS | VNHOME URT , CO 80487 |
| M.E.P. & F.P. ENGINEERS BOULDER ENGINEE 1717 15TH STREET BOULDER, CO 80302 INTERIOR DESIGNER: PROJECT LOCATION BASECAMP TOV 1950 CURVE CO STEAMBOAT SPRINGS DRAWING TITLE | VNHOME URT , CO 80487 |
| M.E.P. & F.P. ENGINEERS BOULDER ENGINEE 1717 15TH STREET BOULDER, CO 80302 INTERIOR DESIGNER: PROJECT LOCATION BASECAMP TOV 1950 CURVE CO STEAMBOAT SPRINGS DRAWING TITLE TH1 LEVEL 1 | VNHOME URT , CO 80487 PLAN DATE: 05/27/22 |
| M.E.P. & F.P. ENGINEERS BOULDER ENGINEE 1717 15TH STREET BOULDER, CO 80302 INTERIOR DESIGNER: PROJECT LOCATION BASECAMP TOV 1950 CURVE CO STEAMBOAT SPRINGS DRAWING TITLE TH1 LEVEL 1 | VNHOME URT CO 80487 PLAN DATE: 05/27/22 DRAWN BY: |
| M.E.P. & F.P. ENGINEERS BOULDER ENGINEE 1717 15TH STREET BOULDER, CO 80302 INTERIOR DESIGNER: PROJECT LOCATION BASECAMP TOV 1950 CURVE CO STEAMBOAT SPRINGS DRAWING TITLE TH1 LEVEL 1 | VNHOME URT , CO 80487 PLAN DATE: 05/27/22 |
| M.E.P. & F.P. ENGINEERS BOULDER ENGINEE 1717 15TH STREET BOULDER, CO 80302 INTERIOR DESIGNER: PROJECT LOCATION BASECAMP TOV 1950 CURVE CO STEAMBOAT SPRINGS DRAWING TITLE TH1 LEVEL 1 | VNHOME URT CO 80487 PLAN DATE: 05/27/22 DRAWN BY: |

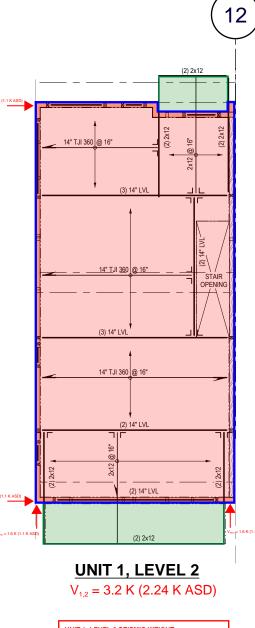
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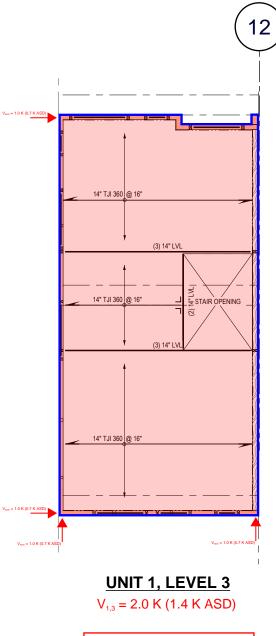




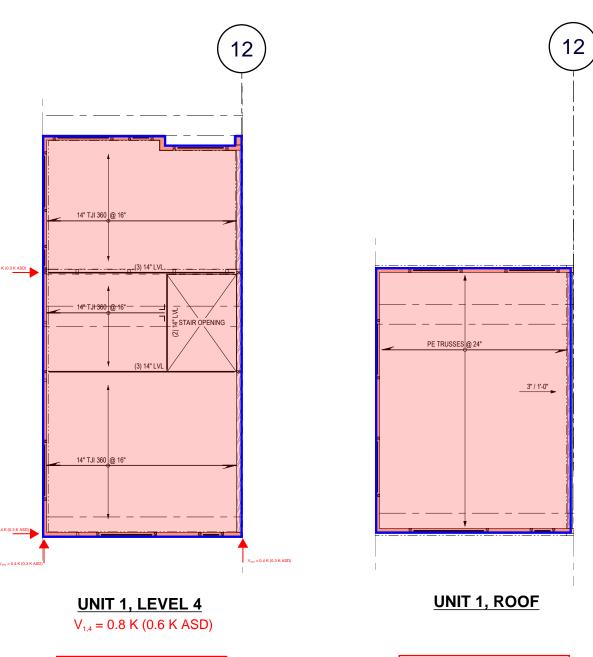
 $\begin{array}{l} \textbf{BASE SHEAR AT LEVEL 1:} \\ W= 14.4 \ K+12.9 \ K+3.8 \ K=49 \ K \\ V= C_s \ W= 0.093 \ W= 0.093 \ 49 \ K= \underline{4.5 \ K} \\ V_{\text{ASD}} = 4.5 \ K \ ^{\circ} 0.7 = \underline{3.2 \ K \ (ASD)} \end{array}$





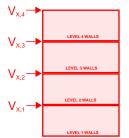






| UNIT 1, LEVEL 4 SEISMIC WEI FLOOR AREA = 540 SF ROOF DECK = 265 SF |
|---|
| W= (540 SF + 265 SF)* 16 PSF |
| V = C _s W = 0.093W = 0.093*12.9 |
| SHEAR AT LEVEL 3 WALLS V _{TOTAL} = 1.2 K + 0.82 K = <u>2.0 K</u> |

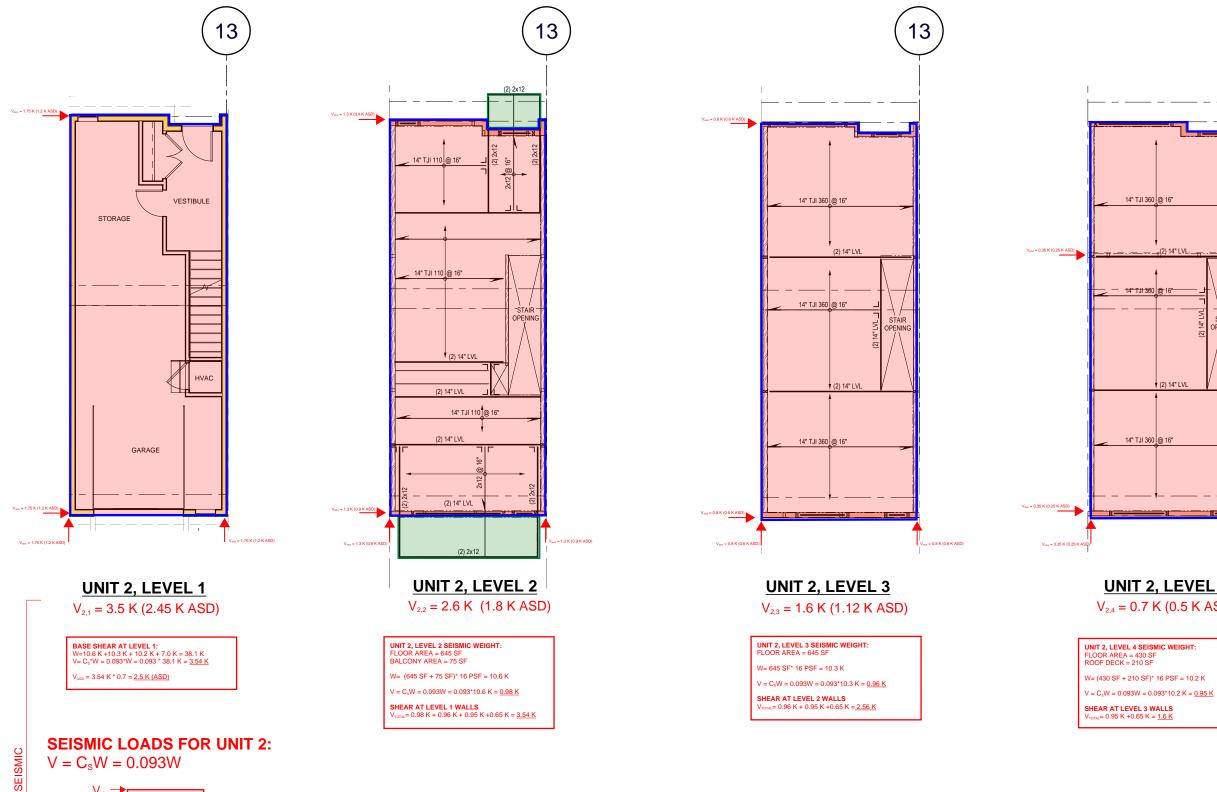


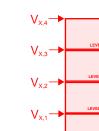


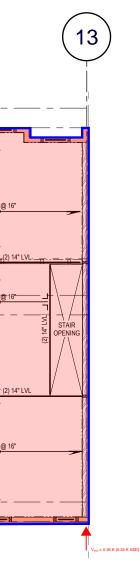


| UNIT 1, ROOF SEISMIC WEIGHT: ROOF AREA = 550 SF |
|---|
| W= 550 SF* 16 PSF = 8.8 K |
| $V = C_s W = 0.093W = 0.093^* 8.8 \text{ K} = 0.82 \text{ K}$ |
| SHEAR AT LEVEL 4 WALLS VTOTAL= 0.82 K |



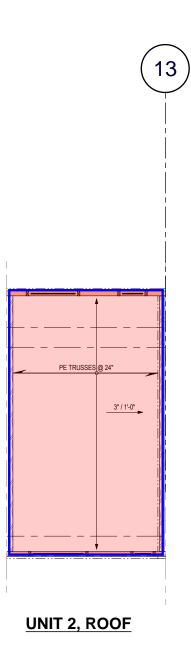






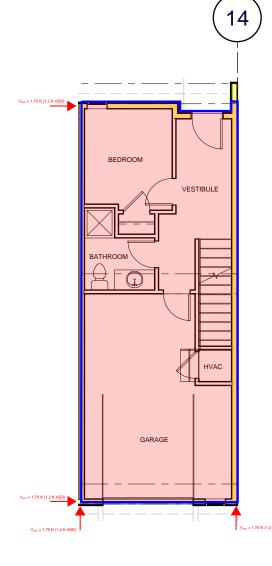


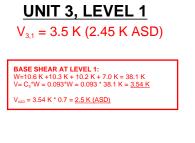






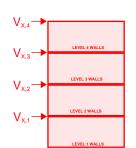


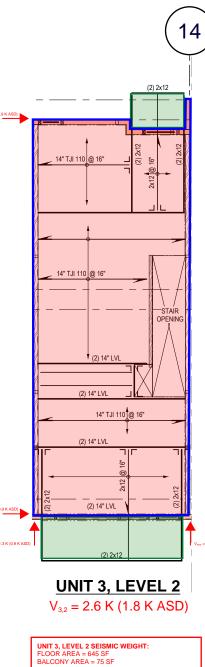


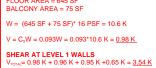


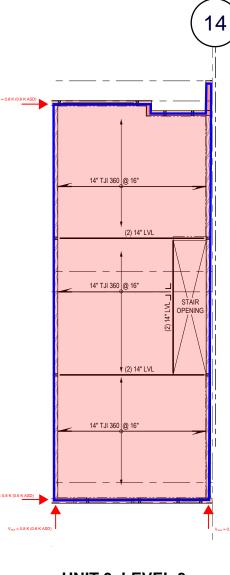
SEISMIC





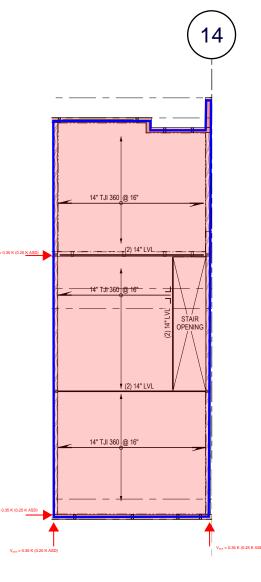






UNIT 3, LEVEL 3 V_{3,3} = 1.6 K (1.12 K ASD)

| UNIT 3, LEVEL 3 SEISMIC WEIGHT: FLOOR AREA = 645 SF |
|---|
| W= 645 SF* 16 PSF = 10.3 K |
| $V = C_s W = 0.093 W = 0.093^{*}10.3 K = 0.96 K$ |
| SHEAR AT LEVEL 2 WALLS V _{TOTAL} = 0.96 K + 0.95 K +0.65 K = <u>2.56 K</u> |

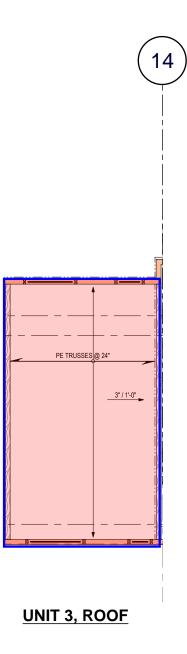




| UNIT 3, LEVEL 4 SEISMIC WE FLOOR AREA = 430 SF ROOF DECK = 210 SF |
|---|
| W= (430 SF + 210 SF)* 16 PS |
| V = C _s W = 0.093W = 0.093*10. |
| SHEAR AT LEVEL 3 WALLS V _{TOTAL} = 0.95 K +0.65 K = <u>1.6 K</u> |



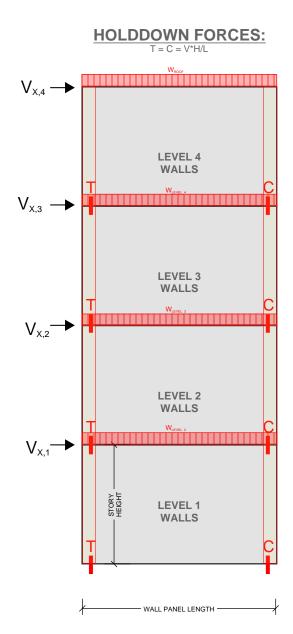




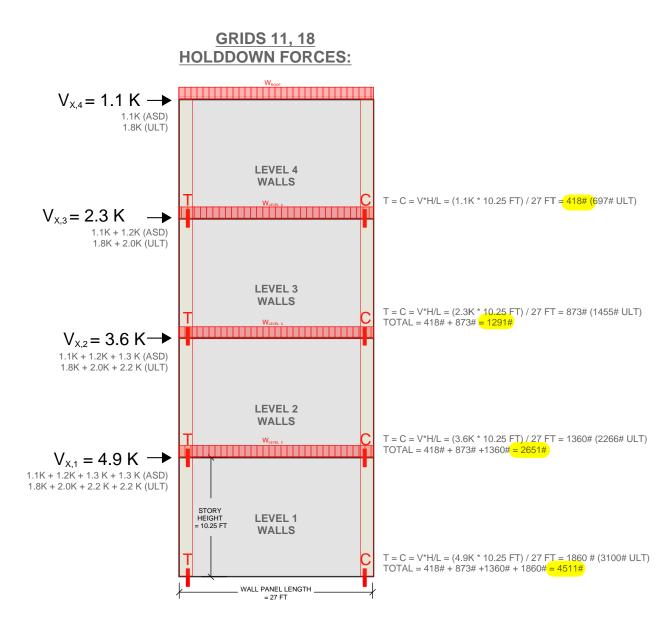
UNIT 3, ROOF SEISMIC WEIGHT: ROOF AREA = 435 SF W= 435 SF* 16 PSF = 7.0 K $V = C_{s}W = 0.093W = 0.093^{*}7.0 \text{ K} = \underline{0.65 \text{ K}}$

SHEAR AT LEVEL 4 WALLS

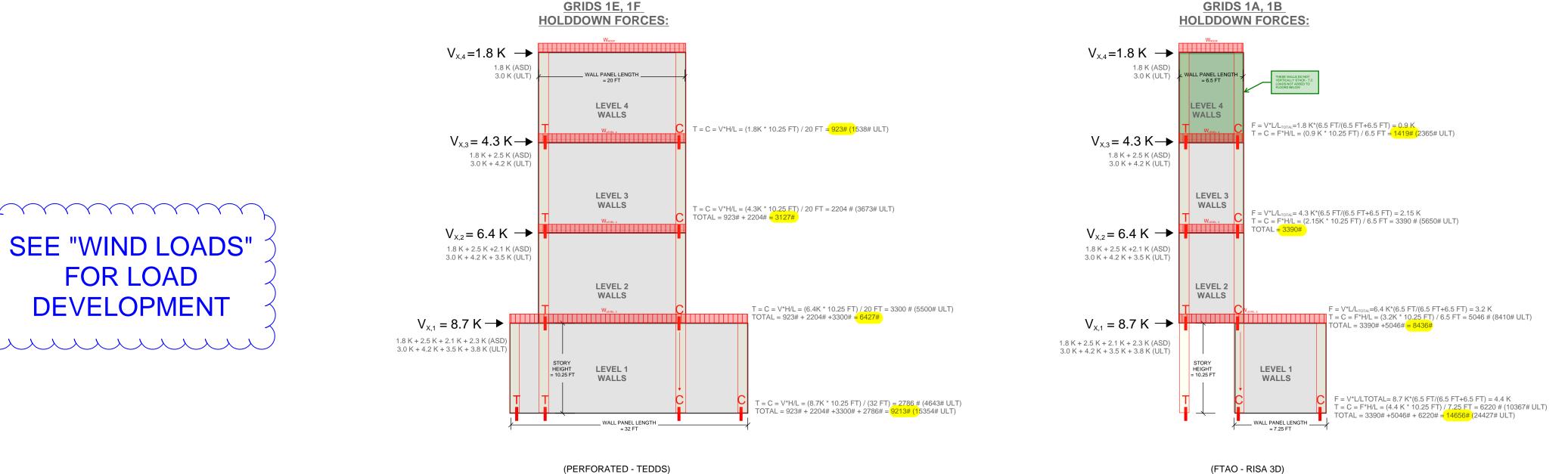
LATERAL DESIGNS



OVERTURNING FORCES: TH1

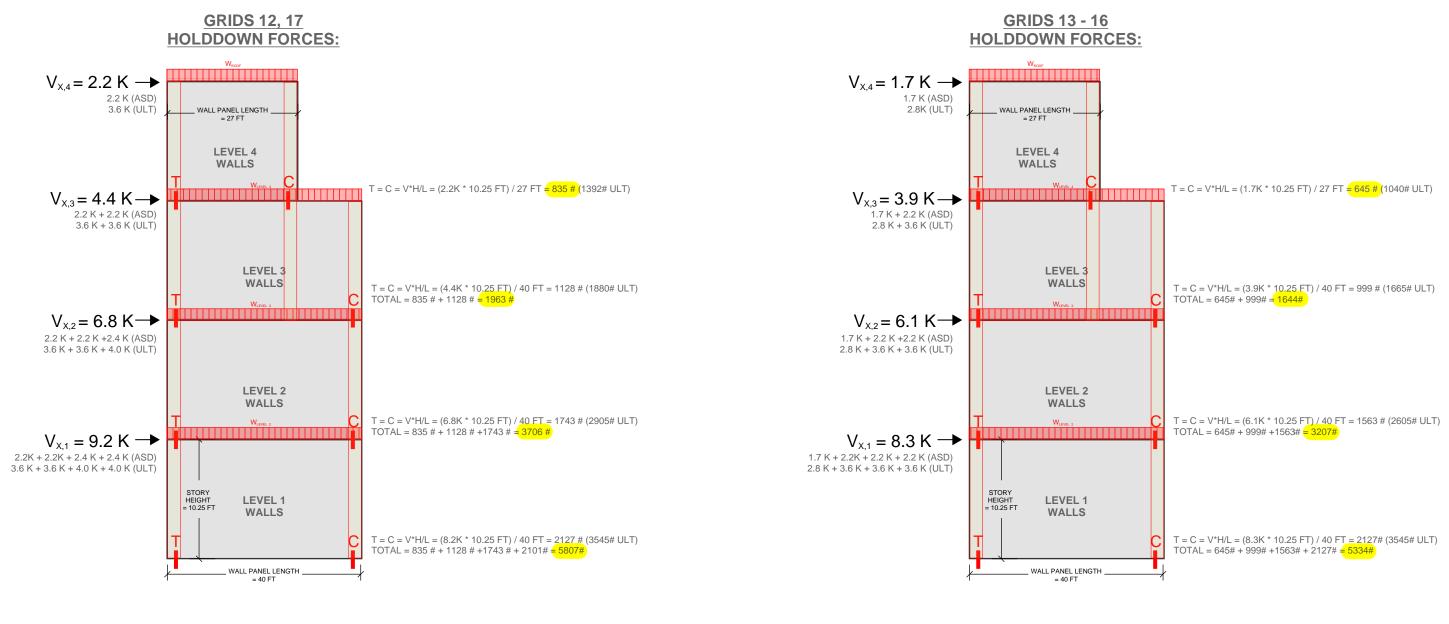


(PERFORATED - TEDDS)



(PERFORATED - TEDDS)

N-S WALLS



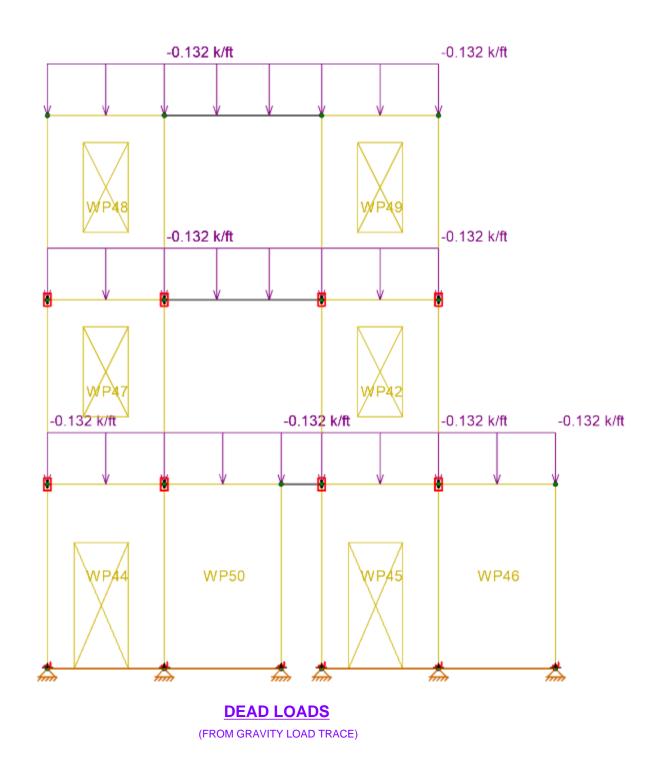
E-W WALLS

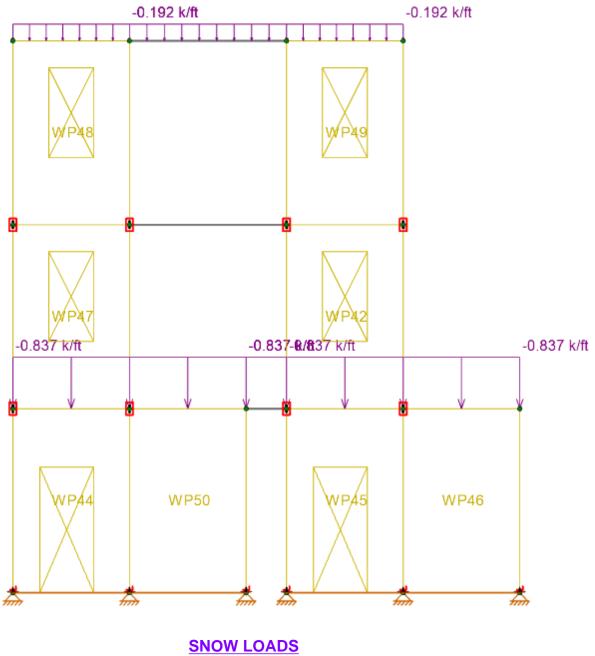
(PERFORATED - TEDDS)



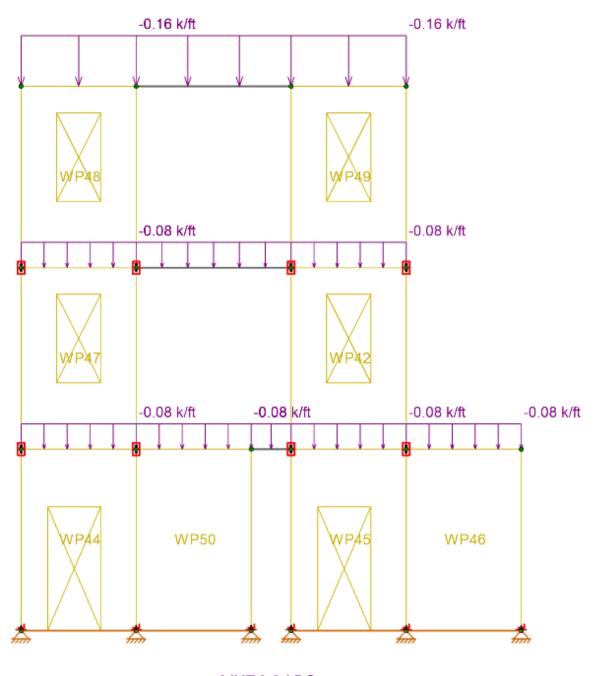
(PERFORATED - TEDDS)

E-W WALLS GRIDS 1A, 1B (FTAO - RISA 3D)

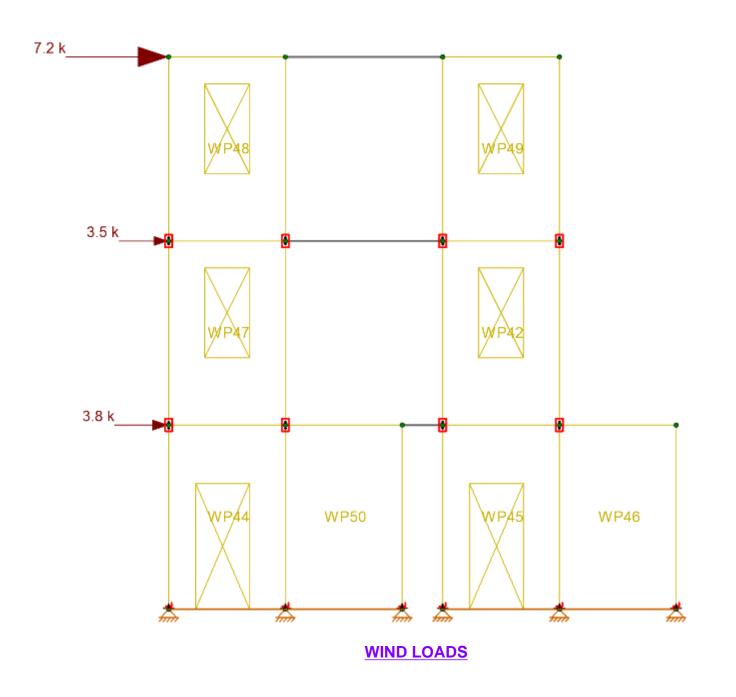


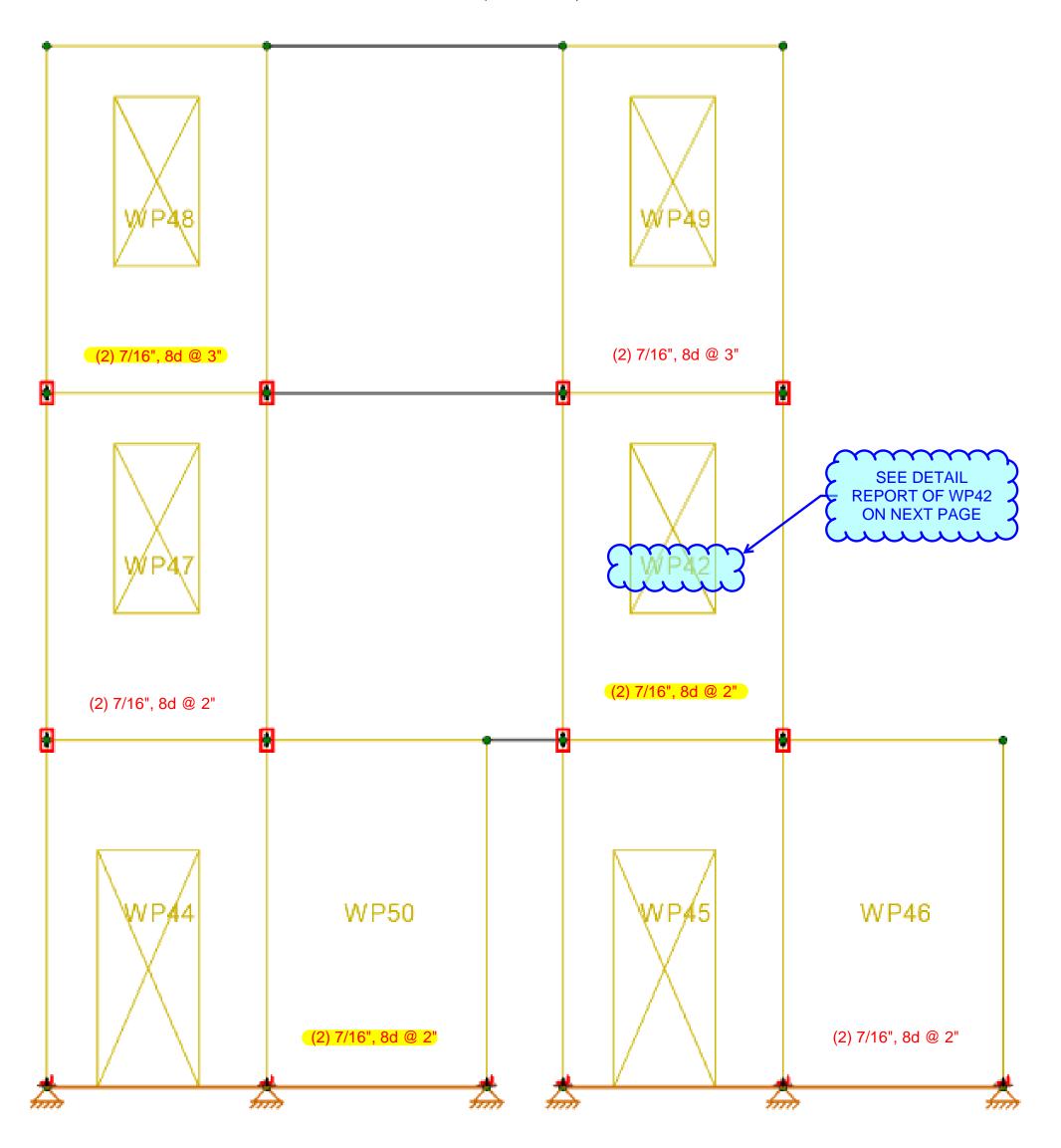


(FROM GRAVITY LOAD TRACE)



LIVE LOADS (FROM GRAVITY LOAD TRACE)





| Wood Wall Panel In Plane Code Checks (AWC NDS-18: ASD) | | | | | | | | | | | |
|--|-----------|--------------------------|-----------------|------------|------------------|---------------|-----------------------|--------------------|-------------|----------------|-----------------|
| Concrete | In Concre | ete Out Concrete Seismic | Masonry In Maso | onry Out M | lasonry Lintel N | lasonry Seisi | mic Wood Wall Axial W | Wood Wall In-Plane | Nood Header | CFS Wall Axial | FS Wall In-Plan |
| | 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 | t 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 |
| | | | - | | - | - | | - | | - | |



Enveloped Results

| ····· | | | | | Іпри | ıt Data: | |
|---------------------|--------------|--------------|--------------|-------|-------------------------------|---------------------------------|-------------------|
| | | | | | Code | AW | C NDS-18: ASD |
| | | | | | Desig | n Method: FTA | 0 |
| | | | | | Heigh | nt (ft): 10.2 | 25 |
| | N105 | 2-2X6 | | N120 | Lengt | h (ft): 6.5 | |
| | R1 | R8 | R3 | N120 | Wall | Material: Her | m-Fir No.2 |
| | | 110 | | | Panel | Schedule: AW | C 2015 OSB |
| | | \land | | | Sel. S | hear Panel: S1_ | (2)7/16_8d@2 16 |
| | | \backslash | | | Optin | nize HD: Yes | |
| | | | | | | lanufacturer: SIN | IPSON |
| | R4 | \land | R5 | | Wall | Properties: | |
| | | | | | Top P | late: | 2-2X6 |
| | | | | | Sill: | | 2X6 |
| | | | | | Wall S | Stud: | 2X6 |
| | | | | | Chorc | | 6X8 |
| | R6 | R7 | R8 | | Max I | H/W Ratio: | 1.577 |
| | | | | | К: | | 1 |
| | | 2X6 | | | Max | Opening Ht: | 5 |
| | •N115 | 270 | ÷ | N113 | Wall ((2w/h | Capacity Adj. Factor): | 0.8 |
| | | | | | Aspec | t Ratio Factor: | 0.938 |
| | | | | | Gov. I | H/W Ratio Factor: | 1.12 |
| | | | | | Shear | Cap. Adj Factor (Co |): 0.849 |
| | | | | | Sheat | hing Area Ratio (r): | 0.766 |
| Material Properties | 5: | | | | | | |
| Top Plate: | Hem-Fir No.2 | Fb (ks | i): | 0.85 | Ft (ksi): | 0.525 | |
| Sill: | Hem-Fir No.2 | Fv (ks | i): | 0.15 | | er 2015 NDS Supple | |
| Wall Stud: | Hem-Fir No.2 | Fc (ks | i): | 1.3 | (Reference De Dimension Lu | esign Values for Visu Imber) | ally Graded |
| Chord: | Hem-Fir No.2 | Specif | fic Gravity: | 0.43 | | | |
| E: | 1300 | Densi | ty (k/ft³): | 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 : Simp | son Chord Straps | | | | |
| HST5_SPF/HF_Bolt | 8 (W) | 7.844 k | 9.87 k | 0.795 | PASS |
| Chords | | | | | |



_

| 0.178 | 1 | 0.033 in | 0 in | 0.145 in | Adjustment Fa | |
|-----------------------------------|---------|----------------|----------------------|--------------------|-----------------|--------------|
| Deflection Results Total (Max) | Gov. LC | Elastic | HD | Shear | Shear Stiffness | |
| Studs 2X6 | | 0 (W) | 0 k | 0 k | 0.000 | PASS |
| 6X8 (Tension) 6X8 (Tension) | | 8 (W) 7 (W) | -8.504 k -8.418 k | 24.75 k 24.75 k | 0.344 0.340 | PASS PASS |

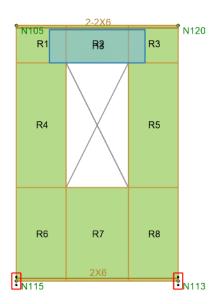
Code Check:

| Limit State | | Required | Available | Unity Check | Result |
|--|--------------------------------------|--------------------------------|------------------|--------------------------|---|
| Shear Panel Design | | 1.132 k/ft | 1.396 k/ft | 0.811 | PASS |
| Shear Panel: S1_(Panel Grade Panel Thickness Number Sides Over Gyp. Board Nail Size NOTE: AWC NDS Required Penetra | -18 defines a 8.000000d na tion | ail as being 2.5" x 0.131" con | | .113" galvanized box | St-I 0.438in Two No 8.000000d 1.375 in 2 in |
| Required Spacing | } Adjustment Factor = [1-(0.5 | -G)] | | | 2 in 0.93 |
| Shear Capacity | | 5)] | | | 1.34 k/ft |
| Adjusted Capacit | :y | | | | 1.396 k/ft |
| Chord Design | | 7.844 k | 24.75 k | 0.317 | PASS |
| Gov Compressio Compression An Gov Tension LC | alysis | 9.205 k | 30.348 k | 0.303 | PASS |
| Tension Analysis | | 7.844 k | 24.75 k | 0.317 | PASS |
| Stud Design | NOTE: Stud design per | formed only for load comb | inations which d | lo not contain seismic o | r wind load |
| Chord Strap Design | | 7.844 k | 9.87 k | 0.795 | PASS |
| | trap / Hold Down: Combination = 8 | HST5_SPF/HF_Bolt | | | |

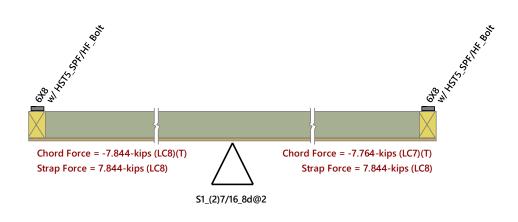
| Governing Load Combination = 8 | |
|--------------------------------|--------------------|
| Clear Span | 0 in |
| Fasteners | (12) 0.625 inch in |
| End Length | 0 in |
| Req'd Chord Mat'l | SPF/HF |
| Base Cap (C _D =1) | 6.169 k |
| C D Factor | 1.6 |
| Adjusted Capacity | 9.87 k |
| | |



Checked By : _____



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 f | or load combination | ations containin | g seismic or wind ca | tegories. |
| Header Shear Design | Note: No header design fo | or load combina | tions containing | seismic or wind cat | tegories. |





| Tekla Tedds Anthem Structural Engineers | Project | | | | | Job Ref. | |
|---|---------------|------------------|----------|------|----------------|---------------------|--|
| | Section | | | | Sheet no./rev. | Sheet no./rev. 1 | |
| | Calc. by S | Date 8/1/2022 | Chk'd by | Date | App'd by | Date | |

LEVEL 4

Tedds calculation version 1.2.08

WOOD SHEAR WALL DESIGN (NDS)

In accordance with NDS2018 allowable stress design and the perforated shear wall method

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 |

h = **10.25** ft

Panel details

Structural I wood panel sheathing on one side

Panel height Panel length

b = **20** ft D + S W s1 s2 s3 δ ā o2 <u>_1</u> 10' 3"īο īo • 0 Ъ 2 1.153 kips Ch2 V 1.153 kips 5' 6" 4' 6"

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 |
| | |

| Tekla Tedds | Project | | Job Ref. | | | | | |
|--|-----------------------|---|---|---------------|-------------------|----------------|--|--|
| Anthem Structural Engineers | Section | | | | Sheet no./rev. | Sheet no./rev. | | |
| | Calc. by | Date | | Date | | Date | | |
| | S | 8/1/2022 | Chk'd by | Date | App'd by | Date | | |
| Panel construction | | | | | | LEV | | |
| 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 po | osts | A _{en} = 13.5 in ² | 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 | k _a = 30000 lb/in | | | | | |
| From NDS Supplement Table 44 | - Reference | design values fo | or visually gra | aded dimensio | on lumber (2" - 4 | " thick) | | |
| Species, grade and size classifica | | - | 1 & btr grade, | | | | | |
| Specific gravity | | G = 0.43 | 5, | | | | | |
| 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 | nanel structu | ral Loriented | strandboard she | eathing | | |
| Fastener type | | | nails at 6"ce | | Strandboard Sh | cauning | | |
| | | | | | | | | |
| From SDPWS Table 4.3A Nomin Nominal unit shear capacity for | | - | | | | | | |
| Nominal unit shear capacity for | | | • • | · · · | • • | | | |
| | - | | v_w = min(785 plf × min[1 - (0.5 - G), 1], 2435 plf) = 730.1 lb/ft G _a = 16 kips/in | | | | | |
| Apparent shear wall shear stiff | 1000 | $G_a - IO KIPS$ | 9/111 | | | | | |
| 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 | | | $S_{wt} = 10 \text{ lb/ft}^2$ | | | | | |
| In plane wind load acting at head | of panel | | W = 3000 lbs | | | | | |
| Wind load serviceability factor | | f _{Wserv} = 1.00 | f _{Wserv} = 1.00 | | | | | |
| From ASCE 7-16 - cl.2.4.1 and c | . 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 + | D + 0.75L _f + 0.45W + 0.75(L _r or S or R) | | | | | |
| Load combination no.3 | Load combination no.4 | | $D + 0.75L_f + 0.525E + 0.75S$ | | | | | |
| | | | 0.525E + 0.75 | 0 | | | | |
| | | 0.6D + 0.6W | | 0 | | | | |

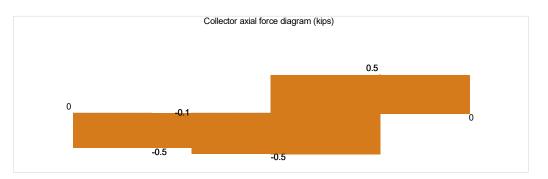
| Tekla Tedds | Project | | | | Job Ref. | | | | |
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| Anthem Structural Engineers | Section | | | | Sheet no./rev. | | | | |
| | | | | | 3 | | | | |
| | Calc. by | Date | Chk'd by | Date | App'd by | Date | | | |
| | S | 8/1/2022 | | | | | | | |
| Adjustment fasters | | | | | | LEVEL | | | |
| 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 – Tab | le 4A | C _{Fc} = 1.10 | | | | | | | |
| Wet service factor for tension – Ta | | C _{Mt} = 1.00 | | | | | | | |
| Wet service factor for compressio | | C _{Mc} = 1.00 | | | | | | | |
| Wet service factor for modulus of | | | | | | | | | |
| | | Сме = 1.00 | | | | | | | |
| Temperature factor for tension – | Table 2.3.3 | $C_{tt} = 1.00$ | | | | | | | |
| Temperature factor for compressi | | | | | | | | | |
| | | C _{tc} = 1.00 | | | | | | | |
| Temperature factor for modulus o | f elasticity – Tab | | | | | | | | |
| • | , | C _{tE} = 1.00 | | | | | | | |
| Incising factor – cl.4.3.8 | | C _i = 1.00 | | | | | | | |
| Buckling stiffness factor – cl.4.4.2 | | $C_{\rm T} = 1.00$ | | | | | | | |
| Adjusted modulus of elasticity | | $E_{min}' = E_{min} \times C_{ME} \times C_{tE} \times C_i \times C_T = 550000 \text{ psi}$ | | | | | | | |
| Critical buckling design value | | $F_{cE} = 0.822 \times E_{min}' / (h / d)^2 = 904 psi$ | | | | | | | |
| | | $F_{c}^{*} = F_{c} \times C_{D} \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_{i} = 2376 \text{ psi}$ | | | | | | | |
| Reference compression design va | alue | c = 0.8 | | | | | | | |
| For sawn lumber | | c = 0.8 $C_P = (1 + (F_{cE} / F_c^*)) / (2 \times c) - \sqrt{([(1 + (F_{cE} / F_c^*)) / (2 \times c)]^2 - (F_{cE} / C_c^*))}$ | | | | | | | |
| Column stability factor – eqn.3 | ./-1 | $C_P = (1 + (F_c + F_c^*) / c) = 0.3$ | <i>,,</i> , , |) – √([(1 + (⊢ _{cE} | / F _c *)) / (2× | C)] [∠] - (⊢ _{cE} / | | | |
| | | , , | 4 | | | | | | |
| | <u> </u> | | | | | | | | |
| From SDPWS Table 4.3.4 Maxim | num Shear Wal | - | | | | | | | |
| Maximum shear wall aspect ratio | num Shear Wal | 3.5 | | | | | | | |
| Maximum shear wall aspect ratio Perforated wall length | num Shear Wal | 3.5 b ₁ = 4 ft | | | | | | | |
| Maximum shear wall aspect ratio Perforated wall length Shear wall aspect ratio | num Shear Wal | 3.5 b ₁ = 4 ft h / b ₁ = 2.563 | | | | | | | |
| Maximum shear wall aspect ratio Perforated wall length Shear wall aspect ratio Perforated wall length | num Shear Wal | 3.5 b ₁ = 4 ft h / b ₁ = 2.563 b ₂ = 4 ft | | | | | | | |
| Maximum shear wall aspect ratio Perforated wall length Shear wall aspect ratio Perforated wall length Shear wall aspect ratio | num Shear Wal | 3.5 $b_1 = 4$ ft $h / b_1 = 2.563$ $b_2 = 4$ ft $h / b_2 = 2.563$ | | | | | | | |
| Maximum shear wall aspect ratio Perforated wall length Shear wall aspect ratio Perforated wall length Shear wall aspect ratio Perforated wall length | num Shear Wal | 3.5 $b_1 = 4$ ft $h / b_1 = 2.563$ $b_2 = 4$ ft $h / b_2 = 2.563$ $b_3 = 4.5$ ft | | | | | | | |
| Maximum shear wall aspect ratio Perforated wall length Shear wall aspect ratio Perforated wall length Shear wall aspect ratio Perforated wall length Shear wall aspect ratio | | 3.5 $b_1 = 4$ ft $h / b_1 = 2.563$ $b_2 = 4$ ft $h / b_2 = 2.563$ | | | | | | | |
| Maximum shear wall aspect ratio Perforated wall length Shear wall aspect ratio Perforated wall length Shear wall aspect ratio Perforated wall length Shear wall aspect ratio Shear capacity adjustment facto | or – cl.4.3.3.5 | 3.5 $b_1 = 4$ ft $h / b_1 = 2.563$ $b_2 = 4$ ft $h / b_2 = 2.563$ $b_3 = 4.5$ ft $h / b_3 = 2.278$ | | | | | | | |
| Maximum shear wall aspect ratio Perforated wall length Shear wall aspect ratio Perforated wall length Shear wall aspect ratio Perforated wall length Shear wall aspect ratio Shear capacity adjustment facto Sum of perforated shear wall length | or – cl.4.3.3.5 iths | 3.5 $b_1 = 4$ ft $h / b_1 = 2.563$ $b_2 = 4$ ft $h / b_2 = 2.563$ $b_3 = 4.5$ ft $h / b_3 = 2.278$ $\Sigma L_i = b_1 \times 2 \times 3$ | $b_s / h + b_2 \times 2 \times l$ | | b₅ / h = 9.756 | ft | | | |
| Maximum shear wall aspect ratio Perforated wall length Shear wall aspect ratio Perforated wall length Shear wall aspect ratio Perforated wall length Shear wall aspect ratio Shear capacity adjustment facto Sum of perforated shear wall lengt Total length of perforated shear wall | or – cl.4.3.3.5 iths | 3.5 $b_1 = 4$ ft $h / b_1 = 2.563$ $b_2 = 4$ ft $h / b_2 = 2.563$ $b_3 = 4.5$ ft $h / b_3 = 2.278$ $\Sigma L_i = b_1 \times 2 \times$ $L_{tot} = b_1 + w_{o1}$ | + b ₂ + w _{o2} + b ₃ = | 20 ft | b₅ / h = 9.756 | ft | | | |
| Maximum shear wall aspect ratio Perforated wall length Shear wall aspect ratio Perforated wall length Shear wall aspect ratio Perforated wall length Shear wall aspect ratio Shear capacity adjustment facto Sum of perforated shear wall length | or – cl.4.3.3.5 iths | 3.5 $b_1 = 4$ ft $h / b_1 = 2.563$ $b_2 = 4$ ft $h / b_2 = 2.563$ $b_3 = 4.5$ ft $h / b_3 = 2.278$ $\Sigma L_i = b_1 \times 2 \times$ $L_{tot} = b_1 + w_{o1}$ | | 20 ft | b₅ / h = 9.756 | ft | | | |
| Maximum shear wall aspect ratio Perforated wall length Shear wall aspect ratio Perforated wall length Shear wall aspect ratio Perforated wall length Shear wall aspect ratio Shear capacity adjustment facto Sum of perforated shear wall lengt Total length of perforated shear wall | or – cl.4.3.3.5 iths | 3.5 $b_1 = 4$ ft $h / b_1 = 2.563$ $b_2 = 4$ ft $h / b_2 = 2.563$ $b_3 = 4.5$ ft $h / b_3 = 2.278$ $\Sigma L_i = b_1 \times 2 \times$ $L_{tot} = b_1 + w_{o1}$ $A_o = w_{o1} \times h_{o1}$ | + b ₂ + w _{o2} + b ₃ = | : 20 ft . 5 ft ² | b₅ / h = 9.756 | ft | | | |
| Maximum shear wall aspect ratio Perforated wall length Shear wall aspect ratio Perforated wall length Shear wall aspect ratio Perforated wall length Shear wall aspect ratio Shear capacity adjustment fact Sum of perforated shear wall lengt Total length of perforated shear wall | or – cl.4.3.3.5 ths vall | 3.5 $b_1 = 4$ ft $h / b_1 = 2.563$ $b_2 = 4$ ft $h / b_2 = 2.563$ $b_3 = 4.5$ ft $h / b_3 = 2.278$ $\Sigma L_i = b_1 \times 2 \times$ $L_{tot} = b_1 + w_{o1}$ $A_o = w_{o1} \times h_{o1}$ | + b_2 + w_{o2} + b_3 = + $w_{o2} \times h_{o2}$ = 37 . | : 20 ft . 5 ft ² | b₅ / h = 9.756 | ft | | | |
| Maximum shear wall aspect ratio Perforated wall length Shear wall aspect ratio Perforated wall length Shear wall aspect ratio Perforated wall length Shear wall aspect ratio Shear capacity adjustment facto Sum of perforated shear wall lengt Total length of perforated shear wall Total area of openings Sheathing area ratio (eqn. 4.3-6) | or – cl.4.3.3.5 ths vall | 3.5 $b_1 = 4$ ft $h / b_1 = 2.563$ $b_2 = 4$ ft $h / b_2 = 2.563$ $b_3 = 4.5$ ft $h / b_3 = 2.278$ $\Sigma L_i = b_1 \times 2 \times$ $L_{tot} = b_1 + w_{o1}$ $A_o = w_{o1} \times h_{o1}$ $r = 1 / (1 + A_o)$ | + b_2 + w_{o2} + b_3 = + $w_{o2} \times h_{o2}$ = 37 . | : 20 ft . 5 ft ² | b _s / h = 9.756 | ft | | | |
| Maximum shear wall aspect ratio Perforated wall length Shear wall aspect ratio Perforated wall length Shear wall aspect ratio Perforated wall length Shear wall aspect ratio Shear capacity adjustment factor Sum of perforated shear wall length Total length of perforated shear wall Total area of openings Sheathing area ratio (eqn. 4.3-6) Shear capacity adjustment factor | or – cl.4.3.3.5 _J ths vall (eqn. 4.3-5) | 3.5 $b_1 = 4$ ft $h / b_1 = 2.563$ $b_2 = 4$ ft $h / b_2 = 2.563$ $b_3 = 4.5$ ft $h / b_3 = 2.278$ $\Sigma L_i = b_1 \times 2 \times$ $L_{tot} = b_1 + w_{o1}$ $A_o = w_{o1} \times h_{o1}$ $r = 1 / (1 + A_o)$ $C_o = 0.965$ | + b_2 + w_{o2} + b_3 = + $w_{o2} \times h_{o2}$ = 37 . | : 20 ft . 5 ft ² | b₅ / h = 9.756 | ft | | | |
| Maximum shear wall aspect ratio Perforated wall length Shear wall aspect ratio Perforated wall length Shear wall aspect ratio Perforated wall length Shear wall aspect ratio Shear capacity adjustment factor Sum of perforated shear wall length Total length of perforated shear wall Total area of openings Sheathing area ratio (eqn. 4.3-6) Shear capacity adjustment factor Perforated shear wall capacity Maximum shear force under wall | or – cl.4.3.3.5 iths [/] all (eqn. 4.3-5) ind loading | 3.5 $b_1 = 4$ ft $h / b_1 = 2.563$ $b_2 = 4$ ft $h / b_2 = 2.563$ $b_3 = 4.5$ ft $h / b_3 = 2.278$ $\Sigma L_i = b_1 \times 2 \times$ $L_{tot} = b_1 + w_{01}$ $A_0 = w_{01} \times h_{01}$ $r = 1 / (1 + A_0)$ $C_0 = 0.965$ $V_{w_max} = 0.6 \Sigma$ | + b_2 + w_{o2} + b_3 = + $w_{o2} \times h_{o2}$ = 37 . /($h \times \Sigma L_i$)) = 0.72 × W = 1.8 kips | : 20 ft 5 ft ² 27 | b₅ / h = 9.756 | ft | | | |
| Maximum shear wall aspect ratio Perforated wall length Shear wall aspect ratio Perforated wall length Shear wall aspect ratio Perforated wall length Shear wall aspect ratio Shear capacity adjustment factor Sum of perforated shear wall lengt Total length of perforated shear wall Sheathing area ratio (eqn. 4.3-6) Shear capacity adjustment factor Perforated shear wall capacity | or – cl.4.3.3.5 iths [/] all (eqn. 4.3-5) ind loading | 3.5 $b_1 = 4$ ft $h / b_1 = 2.563$ $b_2 = 4$ ft $h / b_2 = 2.563$ $b_3 = 4.5$ ft $h / b_3 = 2.278$ $\Sigma L_i = b_1 \times 2 \times L_{tot} = b_1 + w_{o1}$ $A_o = w_{o1} \times h_{o1}$ $r = 1 / (1 + A_o)$ $C_o = 0.965$ $V_{w_max} = 0.6 \times V_w = v_w \times C_o$ | + b_2 + w_{o2} + b_3 = + $w_{o2} \times h_{o2}$ = 37. /($h \times \Sigma L_i$)) = 0.72 × W = 1.8 kips × ΣL_i / 2 = 3.43 | : 20 ft 5 ft ² 27 | b _s / h = 9.756 | ft | | | |
| Maximum shear wall aspect ratio Perforated wall length Shear wall aspect ratio Perforated wall length Shear wall aspect ratio Perforated wall length Shear wall aspect ratio Shear capacity adjustment factor Sum of perforated shear wall length Total length of perforated shear wall Total area of openings Sheathing area ratio (eqn. 4.3-6) Shear capacity adjustment factor Perforated shear wall capacity Maximum shear force under wall | or – cl.4.3.3.5 iths [/] all (eqn. 4.3-5) ind loading | 3.5 $b_1 = 4$ ft $h / b_1 = 2.563$ $b_2 = 4$ ft $h / b_2 = 2.563$ $b_3 = 4.5$ ft $h / b_3 = 2.278$ $\Sigma L_i = b_1 \times 2 \times $ | + b_2 + w_{o2} + b_3 = + $w_{o2} \times h_{o2}$ = 37. /($h \times \Sigma L_i$)) = 0.72 × W = 1.8 kips × ΣL_i / 2 = 3.430 | 20 ft 5 ft ² 27 6 kips | | | | | |
| Maximum shear wall aspect ratio Perforated wall length Shear wall aspect ratio Perforated wall length Shear wall aspect ratio Perforated wall length Shear wall aspect ratio Shear capacity adjustment facto Sum of perforated shear wall lengt Total length of perforated shear wall Total area of openings Sheathing area ratio (eqn. 4.3-6) Shear capacity adjustment factor Perforated shear wall capacity Maximum shear force under wa Shear capacity for wind loading | or – cl.4.3.3.5 _j ths _{rall} (eqn. 4.3-5) ind loading | 3.5 $b_1 = 4$ ft $h / b_1 = 2.563$ $b_2 = 4$ ft $h / b_2 = 2.563$ $b_3 = 4.5$ ft $h / b_3 = 2.278$ $\Sigma L_i = b_1 \times 2 \times $ | + b_2 + w_{o2} + b_3 = + $w_{o2} \times h_{o2}$ = 37. /($h \times \Sigma L_i$)) = 0.72 × W = 1.8 kips × ΣL_i / 2 = 3.43 | 20 ft 5 ft ² 27 6 kips | | | | | |
| Maximum shear wall aspect ratio Perforated wall length Shear wall aspect ratio Perforated wall length Shear wall aspect ratio Perforated wall length Shear wall aspect ratio Shear capacity adjustment factor Sum of perforated shear wall length Total length of perforated shear wall Total area of openings Sheathing area ratio (eqn. 4.3-6) Shear capacity adjustment factor Perforated shear wall capacity Maximum shear force under wall | or – cl.4.3.3.5 _j ths _{rall} (eqn. 4.3-5) ind loading | 3.5 $b_1 = 4$ ft $h / b_1 = 2.563$ $b_2 = 4$ ft $h / b_2 = 2.563$ $b_3 = 4.5$ ft $h / b_3 = 2.278$ $\Sigma L_i = b_1 \times 2 \times $ | + b_2 + w_{o2} + b_3 = + $w_{o2} \times h_{o2}$ = 37. /($h \times \Sigma L_i$)) = 0.72 × W = 1.8 kips × ΣL_i / 2 = 3.430 | 20 ft 5 ft ² 27 6 kips | | | | | |

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| 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 × h / ((C _o × ΣL _i)) - P = 1.153 kips - HDU2-SDS2.5 | | | | | | |
| Maximum applied tensile stress | | $f_t = T / A_{en} = 85 \text{ lb/in}^2$ | | | | | | |
| Design tensile stress | | $F_{t}' = F_{t} \times C_{D} \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_{i} = 1508 \text{ lb/in}^{2}$ | | | | | | |
| Design tensile stress | | | | f _t / F _t ' = 0.057 | | | | |
| Design tensile stress | | f _t / F _t ' = 0.05 | 7 | | | | | |

| Load combination 1 | |
|-------------------------------------|--|
| Shear force for maximum compression | V = 0.6 × 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 | $C = V \times h / ((C_o \times \Sigma L_i)) + P = 2.050 \text{ kips}$ |
| Maximum applied compressive stress | f _c = C / A _e = 124 lb/in ² |
| Design compressive stress | $\textbf{F_c'} = \textbf{F_c} \times \textbf{C_D} \times \textbf{C_{Mc}} \times \textbf{C_{tc}} \times \textbf{C_{Fc}} \times \textbf{C_i} \times \textbf{C_P} = \textbf{818} \ \textbf{lb}/\textbf{in}^2$ |
| | fc / Fc' = 0.152 |



Collector capacity



Maximum shear force on wall Uniform shear applied to wall Shear resisted by wall segments Maximum force in collector Maximum applied tensile stress Design tensile stress

Maximum applied compressive stress Column stability factor Design compressive stress

$\begin{array}{l} V_{max} = V_{w_max} = \textbf{1.8 kips} \\ v_a = V_{max} / \left((C_o \times \Sigma L_i) \right) = \textbf{191.3 plf} \\ v_b = v_a \times b / (b_1 + b_2 + b_3) = \textbf{306 plf} \\ P_{coll} = \textbf{0.535 kips} \\ f_t = P_{coll} / (2 \times A_s) = \textbf{32 lb/in}^2 \\ F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i = \textbf{1508 lb/in}^2 \\ f_t / F_t' = \textbf{0.022} \\ \textbf{PASS - Design tensile stress exceeds maximum applied tensile stress} \\ f_c = P_{coll} / (2 \times A_s) = \textbf{32 lb/in}^2 \\ C_P = \textbf{1.00} \\ F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P = \textbf{2376 lb/in}^2 \\ f_c / F_c' = \textbf{0.014} \\ \\ \textbf{PASS - Design compressive stress exceeds maximum applied compressive stress} \\ \end{array}$

Hold down force Chord 1

T1 = 1.153 kips

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| Chord 2 | | T ₂ = 1.153 k | ips | | | 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 ir | ı | | |
| Induced unit shear | | $v_{\delta w_{max}} = V_{\delta w}$ | / (C _o ×ΣL _i) = 3 | 18.75 lb/ft | | |
| Anchor tension force | | $T_{\delta} = max(0)$ | $kips, v_{\delta w_{max}} \times h$ | • 0.6 × (D + S | _{wt} × h) × b / 2) = 2 | 460 kips |
| Shear wall deflection - Eqn. 4.3- | 1 | $\delta_{sww} = 2 \times v_{\delta}$ | $_{w_{max}} 	imes h^3$ / (3 $	imes$ | $E \times A_{e} \times \Sigma L_{i}$) | + $v_{\delta w_{max}} \times h$ / (Ga |) + h $	imes$ T $_{\delta}$ / (k _a $	imes$ |
| | | ΣL _i) = 0.302 | in | | | |
| | | $\delta_{sww} / \Delta_{w allow}$ | 4 007 | | | |

FAIL - Shear wall deflection exceeds deflection limit

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LEVEL 3

Tedds calculation version 1.2.08

WOOD SHEAR WALL DESIGN (NDS)

In accordance with NDS2018 allowable stress design and the perforated shear wall method

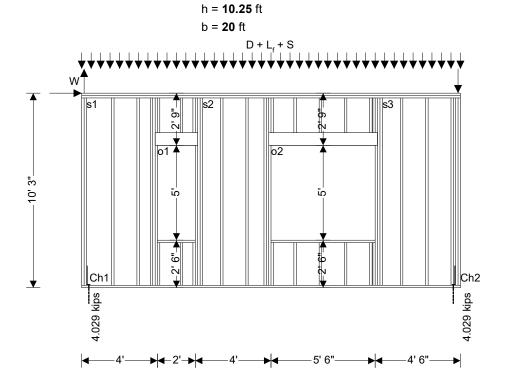
Design summary

| 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 Panel length



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 |
| | |

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| Pan | el constructio | 'n | | | | | | LEVE | | | |
| | ninal stud size | | | 2" x 6" | | | | | | | |
| | ssed stud size | | | 1.5" x 5.5" | | | | | | | |
| | ss-sectional are | ea of studs | | As = 8.25 i | 1 ² | | | | | | |
| Stuc | d spacing | | | s = 16 in | | | | | | | |
| | ninal end post s | size | | 2 x 2" x 6" | | | | | | | |
| | ssed end post | | | 2 x 1.5" x 5 | 5.5" | | | | | | |
| | ss-sectional are | | 3 | A _e = 16.5 i | 1 ² | | | | | | |
| Hole | e diameter | • | | Dia = 1 in | | | | | | | |
| Net | Net cross-sectional area of end posts | | | | in² | | | | | | |
| Nom | Nominal collector size | | | 2 x 2" x 6" | | | | | | | |
| Dres | Dressed collector size | | | | 5.5" | | | | | | |
| Serv | vice condition | | | Dry | | | | | | | |
| Tem | Temperature | | | 100 degF o | 100 degF or less | | | | | | |
| Vertical anchor stiffness | | | | k _a = 30000 | k _a = 30000 lb/in | | | | | | |
| From | m NDS Supple | ement Table 4 | A - Reference | design values | for visually g | raded dimens | ion lumber (2 | " - 4" thick) | | | |
| Spe | cies, grade and | d size classifica | ation | Hem-Fir, n | o.1 & btr grade | e, 2'' & wider | | | | | |
| Spe | cific gravity | | | G = 0.43 | | | | | | | |
| Ten | sion parallel to | grain | | Ft = 725 lb. | /in² | | | | | | |
| Corr | npression para | llel to grain | | F _c = 1350 | b/in² | | | | | | |
| Mod | lulus of elastici | ty | | E = 15000 | 00 lb/in ² | | | | | | |
| Mini | mum modulus | of elasticity | | E _{min} = 5500 | 000 lb/in ² | | | | | | |
| She | athing details | | | | | | | | | | |
| She | athing materi | ial | | 7/16" woo | d panel struc | tural I oriente | d strandboard | d sheathing | | | |
| Fas | tener type | | | 8d comm | on nails at 4" | centers | | _ | | | |
| From | m SDPWS Tab | ole 4.3A Nomiı | nal Unit Shear | Capacities for | r Wood-Frame | e Shear Walls | - Wood-based | l Panels | | | |
| Non | ninal unit she | ar capacity fo | r seismic des | ign v _s = min(8 | 60 plf \times min[| 1 - (0.5 - G), 1 | l], 1740 plf) = | 799.8 lb/ft | | | |
| Non | ninal unit she | ar capacity fo | r wind design | $v_w = min(2)$ | 205 plf $	imes$ mir | n[1 - (0.5 - G), | 1], 2435 plf) | = 1120.6 lb/ft | | | |
| | oarent shear v | | • | , | $G_a = 21$ kips/in | | | | | | |
| | | | | u | P - , | | | | | | |
| | ding details | | | | 161 | | | | | | |
| | d load acting o | | | D = 164 lb/ | | | | | | | |
| | or live load actin | | nei | L _f = 80 lb/f | | | | | | | |
| | w load acting c | | | S = 96 lb/fl | | | | | | | |
| | weight of pane | | of papel | S _{wt} = 10 lb/ W = 7200 | | | | | | | |
| | lane wind load d load servicea | - | | f _{Wserv} = 1.0 | | | | | | | |
| | | | | TWserv - 1.U | U | | | | | | |
| | ord forces from | | | | | | | <u> </u> | | | |
| hord | W _{ch[i]} (lbs) | Eq_ch[i] (lbs) | Dc_ch[i] (lbs) | DT_ch[i] (lbs) | Lf_ch[i] (lbs) | Lr_ch[i] (lbs) | S _{ch[i]} (Ibs) | R _{ch[i]} (lbs) | | | |
| | | | 0; | 0; | 0. | 0; | 0; | 1 11 | | | |
| Ch1 Ch2 | -1538; 1538; | 0; 0; | 0; | 0; | 0; 0; | 0; | 0; | 0; 0; | | | |

D + 0.6W

D + 0.7E

Load combination no.1 Load combination no.2

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| Load combination no.3 | | D + 0.75L _f + 0 | .45W + 0.75(I | L _r or S or R) | | LEVEL |
| Load combination no.4 | | D + 0.75L _f + 0 | .525E + 0.758 | S | | |
| 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 – Tab | le 4A | C _{Fc} = 1.10 | | | | |
| Wet service factor for tension - Ta | able 4A | C _{Mt} = 1.00 | | | | |
| Wet service factor for compressio | n – Table 4A | C _{Mc} = 1.00 | | | | |
| Wet service factor for modulus of | elasticity – Table | | | | | |
| | | C _{ME} = 1.00 | | | | |
| Temperature factor for tension – 1 | | C _{tt} = 1.00 | | | | |
| Temperature factor for compression | on – Table 2.3.3 | a | | | | |
| Tomporature factor for modulus of | folgoticity Tabl | $C_{tc} = 1.00$ | | | | |
| Temperature factor for modulus of | relasticity – radi | e 2.3.3 C _{tE} = 1.00 | | | | |
| Incising factor – cl.4.3.8 | | CtE = 1.00 Ci = 1.00 | | | | |
| Buckling stiffness factor – cl.4.4.2 | | C₁ = 1.00 C⊤ = 1.00 | | | | |
| Adjusted modulus of elasticity | | | | × C _T = 550000 | nsi | |
| Critical buckling design value | | $F_{cE} = 0.822 \times$ | | | poi | |
| Reference compression design value | | | . , | $F_{Fc} \times C_i = 2376$ | nai | |
| For sawn lumber | alue | $r_{c} = r_{c} \times C_{D}$ | CMc X Ctc X C | Fc × Ci - 2376 | psi | |
| | 7 1 | | / | $(a) = \sqrt{(1/1)}$ | | ν a)12 (Γ / |
| Column stability factor – eqn.3. | .7-1 | | | < 0) - \([(1 + (| (F _{cE} / F _c *)) / (2 | × C)] ⁻ - (F _{cE} / |
| | | F _c *) / c) = 0.3 | 4 | | | |
| From SDPWS Table 4.3.4 Maxim | num 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 Shear wall aspect ratio | | b ₃ = 4.5 ft | | | | |
| | | h / b ₃ = 2.278 | | | | |
| Shear capacity adjustment facto | | - | | | o i () -: | |
| Sum of perforated shear wall leng | | | | | 2 × b _s / h = 9.75 | 6 ft |
| Total length of perforated shear w | all | $L_{tot} = b_1 + w_{o1}$ | | | | |
| Total area of openings | | $A_o = w_{o1} \times h_{o1}$ | | | | |
| Sheathing area ratio (eqn. 4.3-6) | | $r = 1 / (1 + A_o)$ | $((\mathbf{n} \times \Sigma \mathbf{L}_i)) = 0$ | .727 | | |
| Shear capacity adjustment factor | (eqn. 4.3-5) | C _o = 0.965 | | | | |
| Perforated shear wall capacity | | | | | | |
| | ind loading | $V_{w max} = 0.6$ | × W = 4.32 ki | ips | | |
| Maximum shear force under w | ind loading | - | | | | |
| | • | $V_w = v_w \times C_o$ | | 274 kips | | |

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PASS - Shear capacity for wind load exceeds maximum shear force Chord capacity for chord 1 V = 0.6 × W = **4.32** kips Shear force for maximum tension 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 × h / ((C_o × ΣL_i)) - P = 4.029 kips + HDU5-SDS2.5 $f_t = T / A_{en} = 298 \text{ lb/in}^2$ Maximum applied tensile stress $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i = 1508 \text{ lb/in}^2$ f_t / F_t' = 0.198

Shear force for maximum compression Axial force for maximum compression Maximum compressive force in chord Maximum applied compressive stress Design compressive stress

Load combination 5

Design tensile stress

Load combination 1

Chord capacity for chord 2

Shear force for maximum tension

Maximum tensile force in chord

Maximum applied tensile stress

Axial force for maximum tension

Load combination 5

Design tensile stress

PASS - Design tensile stress exceeds maximum applied tensile stress V = 0.6 × W = 4.32 kips $P = ((D + S_{wt} \times h)) \times s / 2 + -1 \times 0.6 \times W_{ch1} = 1.1 kips$ $C = V \times h / ((C_o \times \Sigma L_i)) + P = 5.805 kips$ $f_c = C / A_e = 352 \text{ lb/in}^2$ $F_{c}{'} = F_{c} \times C_{D} \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_{i} \times C_{P} = \textbf{818} \text{ lb/in}^{2}$

 $f_c / F_c' = 0.430$

PASS - Design compressive stress exceeds maximum applied compressive stress

V = 0.6 × W = 4.32 kips $P = (0.6 \times (D + S_{wt} \times h)) \times b / 2 + -1 \times 0.6 \times W_{ch2} = 0.676 \text{ kips}$ $T = V \times h / ((C_0 \times \Sigma L_i)) - P = 4.029 kips + HDU5-SDS2.5$ f_t = T / A_{en} = **298** lb/in² F_t = $F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i$ = **1508** lb/in² ft / Ft' = **0.198** PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 1 Shear force for maximum compression Axial force for maximum compression Maximum compressive force in chord Maximum applied compressive stress Design compressive stress

V = 0.6 × W = 4.32 kips $P = ((D + S_{wt} \times h)) \times s / 2 + 0.6 \times W_{ch2} = 1.1 kips$ $C = V \times h / ((C_o \times \Sigma L_i)) + P = 5.805 kips$ f_c = C / A_e = **352** lb/in² $F_{c}' = F_{c} \times C_{D} \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_{i} \times C_{P} = 818 \text{ lb/in}^{2}$ fc / Fc' = 0.430

PASS - Design compressive stress exceeds maximum applied compressive stress

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| | | | | | | | |
| Collector capacity | | | | | | LEVE | |
| | (| Collector axial force dia | gram (kips) | | | | |
| | | | | _ | | | |
| | | | 1. | .2 | | | |
| 0 | -0.2 | | | | | | |
| | | | | | U | | |
| | -1.1 | -1.3 | | | | | |
| | | | | | | | |
| | | | | | | | |
| | | | | | | | |
| Maximum shear force on wall | | V _{max} = V _{w_ma} | _× = 4.32 kips | | | | |
| Uniform shear applied to wall | | $v_a = V_{max} / (($ | C _o × ΣL _i)) = 45 9 | 9 plf | | | |
| Shear resisted by wall segments | , | | $(b_1 + b_2 + b_3) =$ | 734.4 plf | | | |
| Maximum force in collector | | $P_{coll} = 1.285$ kips | | | | | |
| Maximum applied tensile stress | | $f_t = P_{coll} / (2 \times A_s) = 78 \text{ lb/in}^2$ | | | | | |
| Design tensile stress | | $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i = 1508 \text{ lb/in}^2$ $f_t / F_t' = 0.052$ | | | | | |
| | | | _ | ess exceeds i | maximum appli | ed tensile s | |
| Maximum applied compressive s | tress | | × A _s) = 78 lb/in ² | | | | |
| Column stability factor | | C _P = 1.00 | | | | | |
| Design compressive stress | | $F_c' = F_c \times C_D$ | $X 	imes C_{Mc} 	imes C_{tc} 	imes C$ | $C_{Fc} \times C_i \times C_P =$ | 2376 lb/in ² | | |
| | | f _c / F _c ' = 0.0 3 | | | | | |
| | PASS - E | Design compres | sive stress ex | ceeds maxim | um applied con | npressive s | |
| | | | | | | | |
| Hold down force | | T 1 029 k | ine | | | | |
| Chord 1 | | T₁ = 4.029 k T₂ = 4.029 k | - | | | | |
| Chord 1 Chord 2 | | T₁ = 4.029 k T₂ = 4.029 k | - | | | | |
| Chord 1 Chord 2 Wind load deflection | | T ₂ = 4.029 k | ips | S | | | |
| Chord 1 Chord 2 Wind load deflection Design shear force | | T ₂ = 4.029 k V _{δw} = f _{Wserv} | ips × W = 7.2 kips | | | | |
| Chord 1 Chord 2 Wind load deflection Design shear force Deflection limit | | T ₂ = 4.029 k V _{δw} = f _{Wserv} Δ _{w_allow} = h / | ips × W = 7.2 kips 500 = 0.246 ir | ı | | | |
| Chord 1 Chord 2 Wind load deflection Design shear force Deflection limit Induced unit shear | | $T_2 = 4.029 \text{ k}$ $V_{\delta w} = f_{Wserv}$ $\Delta_{w_allow} = h / v_{\delta w_max} = V_{\delta w}$ | · ips × W = 7.2 kips 500 = 0.246 ir / (C _o × ΣL _i) = 7 | ר 65 lb/ft | ∉ × h) × h / 2 + | | |
| Chord 1 Chord 2 Wind load deflection Design shear force Deflection limit | | $T_2 = 4.029 \text{ k}$ $V_{\delta w} = f_{W \text{serv}}$ $\Delta_{w_a \text{llow}} = h /$ $v_{\delta w_m \text{max}} = V_{\delta w}$ $T_{\delta} = \max(0 \text{ k})$ | · ips × W = 7.2 kips 500 = 0.246 ir / (C _o × ΣL _i) = 7 kips,v _{δw_max} × h · | n 65 lb/ft - 0.6 × (D + S _w | t × h) × b / 2 + | | |
| Chord 1 Chord 2 Wind load deflection Design shear force Deflection limit Induced unit shear | 1 | $T_2 = 4.029 \text{ k}$ $V_{\delta w} = f_{W \text{serv}}$ $\Delta_{w_allow} = h / V_{\delta w_max} = V_{\delta w}$ $T_{\delta} = \max(0 \text{ k})$ $\max(abs(W_a)$ | × W = 7.2 kips 500 = 0.246 ir / (C ₀ × ΣL _i) = 7 kips,v _{δw_max} × h - | n 65 lb/ft - 0.6 × (D + S _w = 7.780 kips | ıt × h) × b / 2 + + vδw_max × h / (Ga | | |
| Chord 1 Chord 2 Wind load deflection Design shear force Deflection limit Induced unit shear Anchor tension force | 1 | $T_2 = 4.029 \text{ k}$ $V_{\delta w} = f_{W \text{serv}}$ $\Delta_{w_allow} = h / V_{\delta w_max} = V_{\delta w}$ $T_{\delta} = \max(0 \text{ k})$ $\max(abs(W_a)$ | \times W = 7.2 kips 500 = 0.246 ir $/ (C_o \times \Sigma L_i) = 7$ $kips, v_{\delta w_max} \times h + k_{h1}, abs(W_{ch2})))$ $w_{max} \times h^3 / (3 \times 10^{-1})$ | n 65 lb/ft - 0.6 × (D + S _w = 7.780 kips | · | | |
| Chord 1 Chord 2 Wind load deflection Design shear force Deflection limit Induced unit shear Anchor tension force | 1 | $T_2 = 4.029 \text{ k}$ $V_{\delta w} = f_{W \text{serv}}$ $\Delta_{w_allow} = h /$ $v_{\delta w_max} = V_{\delta w}$ $T_{\delta} = max(0 \text{ k}$ $max(abs(W_d \delta_{sww} = 2 \times v_{\delta})$ | \times W = 7.2 kips 500 = 0.246 ir $/ (C_0 \times \Sigma L_i) = 7$ $(c_{h1}),abs(W_{ch2})))$ $w_{max} \times h^3 / (3 \times in)$ | n 65 lb/ft - 0.6 × (D + S _w = 7.780 kips | · | | |

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LEVEL 2

Tedds calculation version 1.2.08

WOOD SHEAR WALL DESIGN (NDS)

In accordance with NDS2018 allowable stress design and the perforated shear wall method

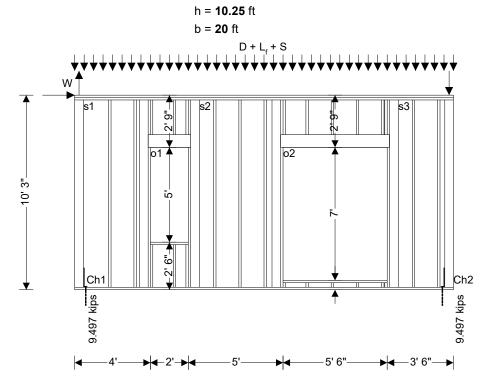
Design summary

| 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 Panel length



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 |
| | |

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| Pan | nel construction | n | | | | | | LEVE | | | |
| | ninal stud size | | | 2" x 6" | | | | | | | |
| | ssed stud size | | | 1.5" x 5.5" | | | | | | | |
| Cro | ss-sectional are | a of studs | | A _s = 8.25 ir | 1 ² | | | | | | |
| Stu | d spacing | | | s = 16 in | | | | | | | |
| | ninal end post s | ize | | 6" x 6" | | | | | | | |
| Dre | ssed end post s | ize | | 5.5" x 5.5" | | | | | | | |
| Cro | ss-sectional are | a of end posts | 5 | A _e = 30.25 | in ² | | | | | | |
| Hole | e diameter | | | Dia = 1 in | | | | | | | |
| Net | Net cross-sectional area of end posts | | | | in² | | | | | | |
| Non | ninal collector s | ize | | 2 x 2" x 6" | | | | | | | |
| | Dressed collector size | | | | 5.5" | | | | | | |
| Ser | Service condition Temperature | | | Dry | - | | | | | | |
| | | | | 100 degF o | | | | | | | |
| Ver | Vertical anchor stiffness | | | | k _a = 30000 lb/in | | | | | | |
| Fro | m NDS Supple | ment Table 4 | A - Reference | design values | for visually g | graded dimens | ion lumber (2 | " - 4" thick) | | | |
| Spe | cies, grade and | size classifica | ation | Hem-Fir, n | o.1 & btr grade | e, 2" & wider | | | | | |
| Spe | cific gravity | | | G = 0.43 | | | | | | | |
| Ten | sion parallel to | grain | | Ft = 725 lb/ | ′in² | | | | | | |
| Con | npression paral | lel to grain | | F _c = 1350 l | b/in² | | | | | | |
| | dulus of elasticit | - | | E = 15000 |)0 lb/in ² | | | | | | |
| Min | imum modulus o | of elasticity | | E _{min} = 5500 |)00 lb/in ² | | | | | | |
| She | eathing details | | | | | | | | | | |
| She | eathing materia | al | | 7/16" woo | d panel struc | ctural I oriente | d strandboard | d sheathing | | | |
| Fas | stener type | | | 8d commo | on nails at 2" | centers | | | | | |
| Fro | m SDPWS Tab | le 4.3A Nomir | nal Unit Shear | Capacities for | Wood-Frame | e Shear Walls | - Wood-based | l Panels | | | |
| Nor | minal unit shea | ar capacity fo | r seismic des | ign v₅ = min(1 | 460 plf $	imes$ mir | n[1 - (0.5 - G), | 1], 1740 plf) | = 1357.8 lb/ft | | | |
| Nor | minal unit shea | ar capacity fo | r wind design | $v_w = min(2)$ | 2045 plf $	imes$ mir | n[1 - (0.5 - G), | 1], 2435 plf) | = 1901.9 lb/ft | | | |
| App | parent shear w | all shear stiff | ness | G _a = 40 ki | ps/in | | | | | | |
| Loa | ding details | | | | | | | | | | |
| | ad load acting or | n top of panel | | D = 296 lb/ | ft | | | | | | |
| | or live load actin | | nel | L _f = 160 lb/ | | | | | | | |
| Floo | w load acting o | | | S = 196 lb/ | | | | | | | |
| | - | | | S _{wt} = 10 lb/ | | | | | | | |
| Sno | r weight of pane | | of panel | W = 10700 | | | | | | | |
| Sno Self | f weight of pane lane wind load a | | •. p ••. | | n | | | | | | |
| Sno Self In p | | acting at head | p= | f _{Wserv} = 1.0 | 0 | | | | | | |
| Sno Self In p Win | lane wind load and load servicea | acting at head bility factor | | 1 _{Wserv} = 1.0 | | | | | | | |
| Sno Self In p Win Cho | lane wind load and load servicea | acting at head bility factor I shear walls a | above | | | Lr chiii (Ibs) | S _{chlil} (lbs) | R _{cbrit} (lbs) | | | |
| Sno Self In p Win | lane wind load and load servicea | acting at head bility factor | | t _{Wserv} = 1.0 Dτ_ch[i] (Ibs) 0; | L _{f_ch[i]} (Ibs) | Lr_ch[i] (Ibs) | S ch[i] (Ibs) 0; | R ch[i] (Ibs) 0; | | | |

D + 0.6W

D + 0.7E

Load combination no.1

Load combination no.2

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| Load combination no.3 | | D + 0.75Lf | + 0.45W + 0.75 | (L _r or S or R) | · · · · · · · · · · · · · · · · · · · | LEVE | | | |
| Load combination no.4 | | | + 0.525E + 0.75 | . , | | | | | |
| Load combination no.5 | | 0.6D + 0.6V | V | | | | | | |
| Load combination no.6 | | 0.6D + 0.7E | Ē | | | | | | |
| Adjustment factors | | | | | | | | | |
| Load duration factor – Table 2.3.2 | | C _D = 1.60 | | | | | | | |
| Size factor for tension – Table 4A Size factor for compression – Table 4A Wet service factor for tension – Table 4A Wet service factor for compression – Table 4A | | C _{Ft} = 1.30 | | | | | | | |
| | | C _{Fc} = 1.10 | | | | | | | |
| | | C _{Mt} = 1.00 | | | | | | | |
| | | C _{Mc} = 1.00 | | | | | | | |
| Wet service factor for modulus of | elasticity – Tab | ole 4A | | | | | | | |
| | | C _{ME} = 1.00 | | | | | | | |
| Temperature factor for tension - | Table 2.3.3 | C _{tt} = 1.00 | | | | | | | |
| Temperature factor for compressi | on – Table 2.3 | .3 | | | | | | | |
| | | C _{tc} = 1.00 | | | | | | | |
| Temperature factor for modulus o | f elasticity – Ta | | | | | | | | |
| | | C _{tE} = 1.00 | | | | | | | |
| Incising factor – cl.4.3.8 | | C _i = 1.00 | | | | | | | |
| Buckling stiffness factor – cl.4.4.2 | | C⊤ = 1.00 | | | | | | | |
| Adjusted modulus of elasticity | | $E_{min}' = E_{min}$ | \times C _{ME} \times C _{tE} \times C _i | × C _T = 550000 | psi | | | | |
| Critical buckling design value | | F _{cE} = 0.822 | imes E _{min} ' / (h / d) ² | = 904 psi | | | | | |
| Reference compression design va | alue | $F_{c}^{*} = F_{c} \times C$ | $C_{\text{D}} 	imes C_{\text{Mc}} 	imes C_{\text{tc}} 	imes 0$ | C _{Fc} × 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 | × c) – √([(1 + | (F _{cE} / F _c *)) / (2 | \times c)] ² - (F _{cE} | | | |
| | | F _c *) / c) = | 0.34 | | | | | | |
| From SDPWS Table 4.3.4 Maxin | num Shear Wa | all Aspect Ratio | os | | | | | | |
| Maximum shear wall aspect ratio | | 3.5 | | | | | | | |
| Perforated wall length | | b₁ = 4 ft | | | | | | | |
| Shear wall aspect ratio | | h / b1 = 2.5 | 63 | | | | | | |
| Perforated wall length | | b ₂ = 5 ft | | | | | | | |
| Shear wall aspect ratio | | h / b ₂ = 2.0 | 5 | | | | | | |
| Perforated wall length | | b₃ = 3.5 ft | | | | | | | |
| Shear wall aspect ratio | | h / b ₃ = 2.9 | 29 | | | | | | |
| Shear capacity adjustment factor | or – cl.4.3.3.5 | | | | | | | | |
| Sum of perforated shear wall leng | ths | $\Sigma L_i = b_1 \times 2$ | imes b _s / h + b ₂ $	imes$ 2 | $2 \times b_s$ / h + b ₃ × | 2 × b _s / h = 8.5 | 37 ft | | | |
| Total length of perforated shear w | all | $L_{tot} = b_1 + w$ | / ₀₁ + b ₂ + w ₀₂ + k | o ₃ = 20 ft | | | | | |
| Total area of openings | | $A_o = w_{o1} \times I$ | $h_{o1} + w_{o2} \times h_{o2} =$ | 48.5 ft ² | | | | | |
| Sheathing area ratio (eqn. 4.3-6) | | r = 1 / (1 + . | $A_o /(h 	imes \Sigma L_i)) = 0$ | .643 | | | | | |
| Shear capacity adjustment factor | (eqn. 4.3-5) | C _o = 0.88 | | | | | | | |
| Perforated shear wall capacity | | | | | | | | | |
| Maximum shear force under w | ind loading | $V_{w_{max}} = 0.$ | 6 × W = 6.42 k | tips | | | | | |
| Shaar appacity for wind loading | a | $V_w = v_w \times 0$ | $C_o \times \Sigma L_i / 2 = 7$ | .142 kips | | | | | |
| Shear capacity for wind loading | | | | | | | | | |

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PASS - Shear capacity for wind load exceeds maximum shear force V = 0.6 × W = **6.42** kips Axial force for maximum tension $P = (0.6 \times (D + S_{wt} \times h)) \times b / 2 + 0.6 \times W_{ch1} = -0.736 \text{ kips}$ T = V × h / ((C_o × ΣL_i)) - P = 9.497 kips + HDU11-SDS2.5 ft = T / Aen = **384** lb/in²

> $F_t = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i = \textbf{1508} \ lb/in^2$ f_t / F_t' = 0.254

PASS - Design tensile stress exceeds maximum applied tensile stress

| Shear force for maximum compression | V = 0.6 × W = 6.42 kips |
|-------------------------------------|--|
| Axial force for maximum compression | $P = ((D + S_{wt} \times h)) \times s / 2 + -1 \times 0.6 \times W_{ch1} = 3.392 \text{ kips}$ |
| Maximum compressive force in chord | C = V × h / ((C _o × Σ L _i)) + P = 12.154 kips |
| Maximum applied compressive stress | f _c = C / A _e = 402 lb/in ² |
| Design compressive stress | $\textbf{F_c'} = \textbf{F_c} \times \textbf{C}_{D} \times \textbf{C}_{Mc} \times \textbf{C}_{tc} \times \textbf{C}_{Fc} \times \textbf{C}_{i} \times \textbf{C}_{P} = \textbf{818} ~ lb/in^2$ |
| | f _c / F _c ' = 0.491 |
| | |

PASS - Design compressive stress exceeds maximum applied compressive stress

| V = 0.6 × W = 6.42 kips |
|---|
| P = $(0.6 \times (D + S_{wt} \times h)) \times b / 2 + -1 \times 0.6 \times W_{ch2}$ = -0.736 kips |
| T = V × h / (($C_o × \Sigma L_i$)) - P = 9.497 kips |
| f _t = T / A _{en} = 384 lb/in ² |
| $F_t = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i = \textbf{1508} \text{ lb/in}^2$ |
| ft / Ft' = 0.254 |
| PASS - Design tensile stress exceeds maximum applied tensile stress |
| |

Shear force for maximum compression Axial force for maximum compression Maximum compressive force in chord Maximum applied compressive stress Design compressive stress

Chord capacity for chord 1

Shear force for maximum tension

Maximum tensile force in chord

Maximum applied tensile stress

Chord capacity for chord 2

Shear force for maximum tension

Maximum tensile force in chord

Maximum applied tensile stress

Axial force for maximum tension

Load combination 5

Design tensile stress

Load combination 1

Load combination 5

Design tensile stress

Load combination 1

V = 0.6 × W = 6.42 kips $P = ((D + S_{wt} \times h)) \times s / 2 + 0.6 \times W_{ch2} = 3.392$ kips $C = V \times h / ((C_o \times \Sigma L_i)) + P = 12.154 \text{ kips} + HDU14-SDS2.5$ f_c = C / A_e = 402 lb/in² $F_{c}' = F_{c} \times C_{D} \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_{i} \times C_{P} = 818 \text{ lb/in}^{2}$ fc / Fc' = 0.491

PASS - Design compressive stress exceeds maximum applied compressive stress

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| Collector capacity | | | | | | LEVE | | |
| | | Collector axial force dia | agram (kips) | | | | | |
| | | | | | | | | |
| | | | | 1.8 | | | | |
| 0 | -0.3 | | | | | | | |
| | | | | | 0 | | | |
| | -2.1 | | -2.9 | | | | | |
| | | | | | | | | |
| | | | | | | | | |
| | | | | | | | | |
| Maximum shear force on wall | | V _{max} = V _{w_m} | _{ax} = 6.42 kips | | | | | |
| Uniform shear applied to wall | | | (C _o × ΣL _i)) = 85 4 | 4.8 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 | 3 kips | | | | | |
| Maximum applied tensile stress | | $f_t = P_{coll} / (2$ | \times A _s) = 176 lb/ir | 1 ² | | | | |
| Design tensile stress | | F_t = $F_t \times C_D$ | $0 \times C_{Mt} \times C_{tt} \times C_{F}$ | $t \times C_i = 1508$ lb | o/in² | | | |
| | | f _t / F _t ' = 0.1 1 | | | | | | |
| | 4 | | - | | naximum applie | ed tensile s | | |
| Maximum applied compressive s Column stability factor | stress | $f_c = P_{coll} / (2 \times A_s) = 176 \text{ lb/in}^2$ $C_P = 1.00$ | | | | | | |
| Design compressive stress | | $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P = 2376 \text{ lb/in}^2$ | | | | | | |
| Design compressive succes | | $f_c / F_c' = 0.074$ | | | | | | |
| | PASS - | Design compres | | ceeds maxim | um applied con | npressive s | | |
| Hold down force | | | | | | | | |
| Chord 1 | | T ₁ = 9.497 | kips | | | | | |
| Chord 2 | | T ₂ = 9.497 | kips | | | | | |
| Wind load deflection | | | | | | | | |
| Design shear force | | $V_{\delta w} = f_{Wserv}$ | ,×W = 10.7 kij | ps | | | | |
| Deflection limit | | $\Delta_{w_{allow}} = h$ | / 500 = 0.246 ir | า | | | | |
| Induced unit shear | | $v_{\delta w_{max}} = V_{\delta w}$ | w / (C _o × ΣL _i) = 1 | 424.63 lb/ft | | | | |
| Anchor tension force | | | kips, $v_{\delta w_{max}} \times h$ | | :×h)×b/2+ | | | |
| | | | / _{ch1}), abs(W _{ch2}))) | | | | | |
| Shear wall deflection - Eqn. 4.3- | 1 | | | | $v_{\delta w_max} 	imes h / (G_a)$ |) + h \times T _{δ} / (| | |
| | | ΣLi) = 1.09 4 | 1 in | | | | | |
| | | , | | | | | | |
| | | $\delta_{ m sww}$ / $\Delta_{ m w}$ allow | | | | | | |

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| WOOD SHEAR WALL DESIGN | | | | | | | |
| In accordance with NDS2018 a | llowable stro | ess design and | the segment | ed shear wall | | edds calcul | ation version 1 |
| 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 sheathin | g on one side | • | | | | | |
| Panel height | | h = 10.2 | 5 ft | | | | |
| Panel length | | b = 32 ft | | | | | |
| | | | D + L _f + S + + + + + + + + + + + + + + + + + + | | | | |
| W | * * * * * * * * * * | **** | * * * * * * * * * * * | · • • • • • • • • • • • • • | * * * * * * * * * * * * | • • • | |
| | | s2 | | | s3 | | |
| | 5 i0 | | | 5, 6 | | | |
| p1 | * | | 02 | ¥ | | | |
| | | | | | | | |
| -0- -0- | | | | | | | |
| | 7' 6" | | | 7. 6" | | | |
| | | | | | | | |
| | | | | | | | |
| Ch1 Ch2 | • | Chβ සු | Ch4 | • | Ch5 ද | Ch6 | |
| 1 1 | | 7 kip | 2 | | 0 Ki | 9 kips | |
| | | 5.777 ki | 5.777 | | 5.629 ki | 5.629 ki | |
| | 9' | • | 7' | 9' | 4' 6"- | | |
| | | Ĭ | Ĩ | | ļ | I | |
| | | | | | | | |
| Panel opening details | | | | | | | |
| | | w _{o1} = 9 f | t | | | | |
| Width of opening | | | | | | | |
| Width of opening Height of opening | opening | h _{o1} = 7.5 | ft | | | | |
| Width of opening | opening | | ft ft | | | | |

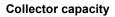
| Position of opening | P _{o1} = 2.5 ft |
|--|---|
| Width of opening | w _{o2} = 9 ft |
| Height of opening | h _{o2} = 7.5 ft |
| Height to underside of lintel over opening | l _{o2} = 7.5 ft |
| Position of opening | P _{o2} = 18.5 ft |
| Total area of wall | A = $h \times b$ - $w_{o1} \times h_{o1}$ - $w_{o2} \times h_{o2}$ = 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" |

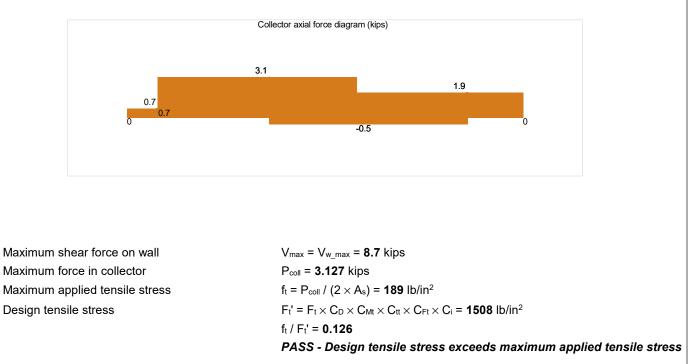
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| Cross-sectional area of end posts | 3 | A _e = 16.5 in ² | | | | LEVE | | |
| Hole diameter | | Dia = 1 in | | | | | | |
| Net cross-sectional area of end p | osts | A _{en} = 13.5 in ² | | | | | | |
| Nominal collector size | | 2 x 2" x 6" | | | | | | |
| Dressed collector size | | 2 x 1.5" x 5.5" | | | | | | |
| Service condition | | Dry | | | | | | |
| Temperature | | 100 degF or le | ess | | | | | |
| Vertical anchor stiffness | | k _a = 30000 lb/ | in | | | | | |
| From NDS Supplement Table 4 | A - Reference d | lesign values fo | r visually gra | ded dimensio | on lumber (2" - 4 | l" thick) | | |
| Species, grade and size classification | ation | Hem-Fir, no.1 | & btr grade, 2 | 2" & wider | | | | |
| Specific gravity | | G = 0.43 | | | | | | |
| Tension parallel to grain | | Ft = 725 lb/in ² | | | | | | |
| Compression parallel to grain | | F _c = 1350 lb/ir | 1 ² | | | | | |
| Modulus of elasticity | | E = 1500000 | b/in ² | | | | | |
| Minimum modulus of elasticity | | E _{min} = 550000 | lb/in ² | | | | | |
| Sheathing details | | | | | | | | |
| Sheathing material | | 7/16'' wood r | oanel structu | ral I oriented | strandboard sh | eathing | | |
| Fastener type | | 8d common nails at 2"centers | | | | | | |
| | | | | | | mala | | |
| From SDPWS Table 4.3A Nomin | | - | | | | lieis | | |
| Nominal unit shear capacity fo | | | | · - | | | | |
| Nominal unit shear capacity fo | - | | | 0.5 - G), 1] = 1 | 1901.9 lb/ft | | | |
| Apparent shear wall shear stift | ness | G _a = 40 kips/ | ïn | | | | | |
| Loading details | | | | | | | | |
| Dead load acting on top of panel | | D = 1024 lb/ft | | | | | | |
| Floor live load acting on top of pa | nel | 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 c | :I. 2.4.5 Basic c | ombinations | | | | | | |
| | :l. 2.4.5 Basic c | ombinations D + 0.6W | | | | | | |
| Load combination no.1 | :l. 2.4.5 Basic c | | | | | | | |
| Load combination no.1 Load combination no.2 | :l. 2.4.5 Basic c | D + 0.6W | 9.45W + 0.75(| [L _r or S or R) | | | | |
| Load combination no.1 Load combination no.2 Load combination no.3 | :I. 2.4.5 Basic c | D + 0.6W D + 0.7E | | | | | | |
| Load combination no.1 Load combination no.2 Load combination no.3 Load combination no.4 | :I. 2.4.5 Basic c | D + 0.6W D + 0.7E D + 0.75L _f + 0 | | | | | | |
| Load combination no.1 Load combination no.2 Load combination no.3 Load combination no.4 Load combination no.5 | :l. 2.4.5 Basic c | D + 0.6W D + 0.7E D + 0.75Lf + 0 D + 0.75Lf + 0 | | | | | | |
| Load combination no.1 Load combination no.2 Load combination no.3 Load combination no.4 Load combination no.5 Load combination no.6 | :I. 2.4.5 Basic c | D + 0.6W D + 0.7E D + 0.75Lf + 0 D + 0.75Lf + 0 0.6D + 0.6W | | | | | | |
| Load combination no.1 Load combination no.2 Load combination no.3 Load combination no.4 Load combination no.5 Load combination no.6 Adjustment factors | | D + 0.6W D + 0.7E D + 0.75Lf + 0 D + 0.75Lf + 0 0.6D + 0.6W | | | | | | |
| Load combination no.1 Load combination no.2 Load combination no.3 Load combination no.4 Load combination no.5 Load combination no.6 Adjustment factors Load duration factor – Table 2.3.2 | 2 | D + 0.6W D + 0.7E D + 0.75Lf + 0 D + 0.75Lf + 0 0.6D + 0.6W 0.6D + 0.7E $C_{D} = 1.60$ | | | | | | |
| Load combination no.1 Load combination no.2 Load combination no.3 Load combination no.4 Load combination no.5 Load combination no.6 Adjustment factors Load duration factor – Table 2.3.2 Size factor for tension – Table 4A | 2 | $D + 0.6W$ $D + 0.7E$ $D + 0.75L_{f} + 0$ $D + 0.75L_{f} + 0$ $0.6D + 0.6W$ $0.6D + 0.7E$ $C_{D} = 1.60$ $C_{Ft} = 1.30$ | | | | | | |
| From ASCE 7-16 - cl.2.4.1 and c Load combination no.1 Load combination no.2 Load combination no.3 Load combination no.4 Load combination no.5 Load combination no.6 Adjustment factors Load duration factor – Table 2.3.2 Size factor for tension – Table 4A Size factor for compression – Table 4A Size factor for compression – Table 4A | 2 le 4A | $D + 0.6W$ $D + 0.7E$ $D + 0.75L_{f} + 0$ $D + 0.75L_{f} + 0$ $0.6D + 0.6W$ $0.6D + 0.7E$ $C_{D} = 1.60$ $C_{Ft} = 1.30$ $C_{Fc} = 1.10$ | | | | | | |
| Load combination no.1 Load combination no.2 Load combination no.3 Load combination no.4 Load combination no.5 Load combination no.6 Adjustment factors Load duration factor – Table 2.3.2 Size factor for tension – Table 4A | 2 le 4A able 4A | $D + 0.6W$ $D + 0.7E$ $D + 0.75L_{f} + 0$ $D + 0.75L_{f} + 0$ $0.6D + 0.6W$ $0.6D + 0.7E$ $C_{D} = 1.60$ $C_{Ft} = 1.30$ | | | | | | |

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| | | Сме = 1.00 | | | | LEVEL 1 | | | | |
| Temperature factor for tension – Ta Temperature factor for compression | | C _{tt} = 1.00 | | | | | | | | |
| | – | C _{tc} = 1.00 | | | | | | | | |
| Temperature factor for modulus of | elasticity – I | able 2.3.3 Ct∈ = 1.00 | | | | | | | | |
| Incising factor – cl.4.3.8 | | CtE = 1.00 Ci = 1.00 | | | | | | | | |
| Buckling stiffness factor – cl.4.4.2 | | C _T = 1.00 | | | | | | | | |
| Adjusted modulus of elasticity | | | $C_{ME} \times C_{tE} \times C_i >$ | < C⊤ = 550000 r | osi | | | | | |
| Critical buckling design value | | | < E _{min} ' / (h / d) ² = | | | | | | | |
| Reference compression design value | IA | | $\times C_{Mc} \times C_{tc} \times C$ | - | si | | | | | |
| For sawn lumber | | c = 0.8 | | | | | | | | |
| Column stability factor – eqn.3.7 | '_1 | | | $(c) = \sqrt{((1 +))}$ | = / F_*)) / (2 \ | x c)1 ² - (E _{-F} / | | | | |
| | | F_{c}^{*} / c) = 0 | $C_{P} = (1 + (F_{cE} / F_{c}^{*})) / (2 \times c) - \sqrt{((1 + (F_{cE} / F_{c}^{*})) / (2 \times c))^{2} - (F_{cE} / F_{c}^{*}))}$ | | | | | | | |
| | | , , | | | | | | | | |
| From SDPWS Table 4.3.4 Maximu | ım Shear W | - | S | | | | | | | |
| Maximum shear wall aspect ratio | | 3.5 | | | | | | | | |
| Segment 1 wall length | | b₁ = 2.5 ft h / b₁ = 4.1 | | | | | | | | |
| Shear wall aspect ratio Segment 2 wall length | | h / b ₁ = 4. 1 b ₂ = 7 ft | | | | | | | | |
| Shear wall aspect ratio | | h / b ₂ = 1.46 | 4 | | | | | | | |
| Segment 3 wall length | | $b_3 = 4.5$ ft | • | | | | | | | |
| Shear wall aspect ratio | | h / b ₃ = 2.27 | 8 | | | | | | | |
| Segmented shear wall capacity - | Strenath di | stribution metho | bd | | | | | | | |
| Maximum shear force under wir | - | | | s | | | | | | |
| Shear capacity for wind loading | la localing | | $b_2 + b_3) / 2 = 10$ | | | | | | | |
| Shear capacity for wind loading | | V _w – V _w ∧ (L V _{w max} / V _w = | , | | | | | | | |
| | | _ | | for wind load e | vceeds maxin | num shear force | | | | |
| Chand consists for should 1 and | • | 1400 0 | neur oupuony i | | | | | | | |
| Chord capacity for chords 1 and Shear wall aspect ratio | 2 | h / b₁ = 4.1 | | | | | | | | |
| onear wan aspect failo | Sea | ment not consid | lered. shear wa | ll aspect ratio | exceeds maxi | mum allowable. | | | | |
| Chord capacity for chords 3 and | - | | , | | | | | | | |
| Shear wall aspect ratio | 4 | h / b ₂ = 1.46 | 4 | | | | | | | |
| Load combination 5 | | 117 D2 - 1.40 | + | | | | | | | |
| Shear force for maximum tension | | $V = 0.6 \times W$ | = 8.7 kips | | | | | | | |
| Axial force for maximum tensior | 1 | | $D + S_{wt} \times h) \times$ | $b_2/2 = 2.366$ | kins | | | | | |
| Maximum tensile force in chord | | | | | | HDU8-SI | | | | |
| Maximum applied tensile stress | | $f_t = T / A_{en} =$ | | // ^ (ii / 62) - i ² - | | | | | | |
| Design tensile stress | | | \times C _{Mt} \times C _{tt} \times C _{Ft} | × Ci = 1508 lb/i | n ² | | | | | |
| | | ft / Ft' = 0.28 | | | | | | | | |
| | | | | ss exceeds m | aximum applie | ed tensile stress | | | | |
| Load combination 1 | | | - 1 | | | | | | | |
| Shear force for maximum compress | sion | $V = 0.6 \times W$ | = 8.7 kips | | | | | | | |
| | ession | | | = 0.751 kips | | | | | | |

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| Maximum compressive force in c | hord | $C = V \times (b_2)$ | $(b_2 + 2 \times b_3^2 / 1)$ | h)) × (h / b2) + | P = 8.894 kips | LEVEL 1 | | | |
| Maximum applied compressive s | tress | $f_c = C / A_e =$ | 539 lb/in ² | | | | | | |
| Design compressive stress | | $F_c' = F_c \times C_c$ | $0 	imes C_{Mc} 	imes C_{tc} 	imes C_{tc}$ | $C_{Fc} \times C_i \times C_P =$ | 818 lb/in ² | | | | |
| | | fc / Fc' = 0.6 | 59 | | | | | | |
| | PASS - | Design compres | sive stress ex | ceeds maxim | um applied con | pressive stress | | | |
| Chord capacity for chords 5 an | d 6 | | | | | | | | |
| Shear wall aspect ratio Load combination 5 | | h / b ₃ = 2.27 | h / b ₃ = 2.278 | | | | | | |
| Shear force for maximum tensior | 1 | V = 0.6 × W = 8.7 kips | | | | | | | |
| Axial force for maximum tensi | on | $P = (0.6 \times (D + S_{wt} \times h)) \times b_3 / 2 = 1.521 \text{ kips}$ | | | | | | | |
| Maximum tensile force in chord | | $T = V \times (2 \times b_3^2 / h / (b_2 + 2 \times b_3^2 / h)) \times (h / b_3) - P = 5.629 \text{ kips} + HDU8 \text{ SE}$ | | | | | | | |
| Maximum applied tensile stress | | $f_t = T / A_{en} = 417 \text{ lb/in}^2$ | | | | | | | |
| Design tensile stress | | $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i = 1508 \text{ lb/in}^2$ | | | | | | | |
| | | ft / Ft' = 0.27 | 7 | | | | | | |
| | | PASS - Des | ign tensile str | ess exceeds l | maximum applie | ed tensile stress | | | |
| Load combination 1 | | | | | | | | | |
| Shear force for maximum compre | ession | $V = 0.6 \times W$ | = 8.7 kips | | | | | | |
| Axial force for maximum comp | pression | P = ((D + S | $S_{wt} \times h)) \times s / 2$ | = 0.751 kips | | | | | |
| Maximum compressive force in c | hord | $C = V \times (2 \times b_3^2 / h / (b_2 + 2 \times b_3^2 / h)) \times (h / b_3) + P = 7.901 \text{ kips}$ | | | | | | | |
| Maximum applied compressive s | tress | $f_c = C / A_e =$ | 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 | | | | | | | |
| | | | | | | | | | |

PASS - Design compressive stress exceeds maximum applied compressive stress



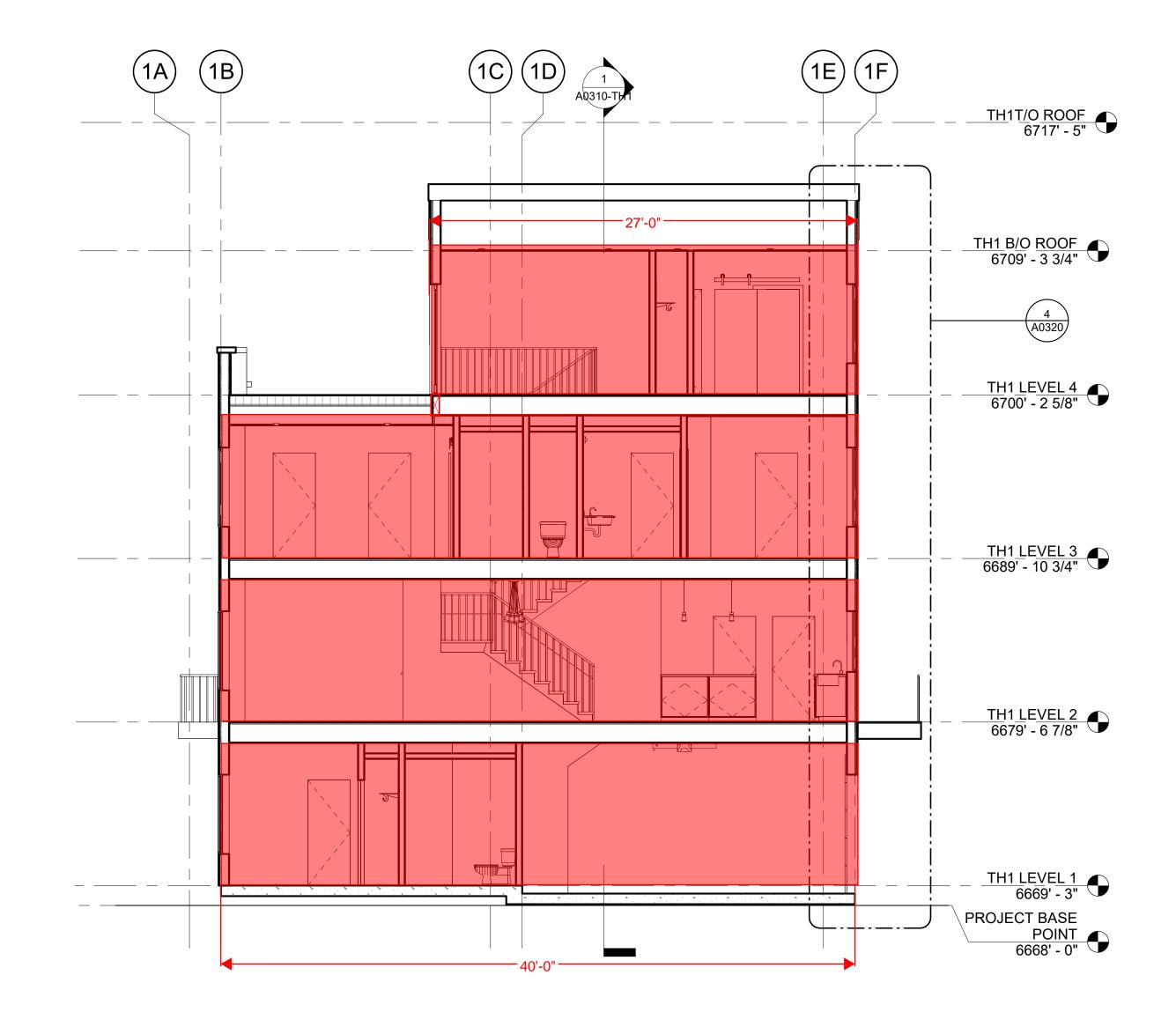


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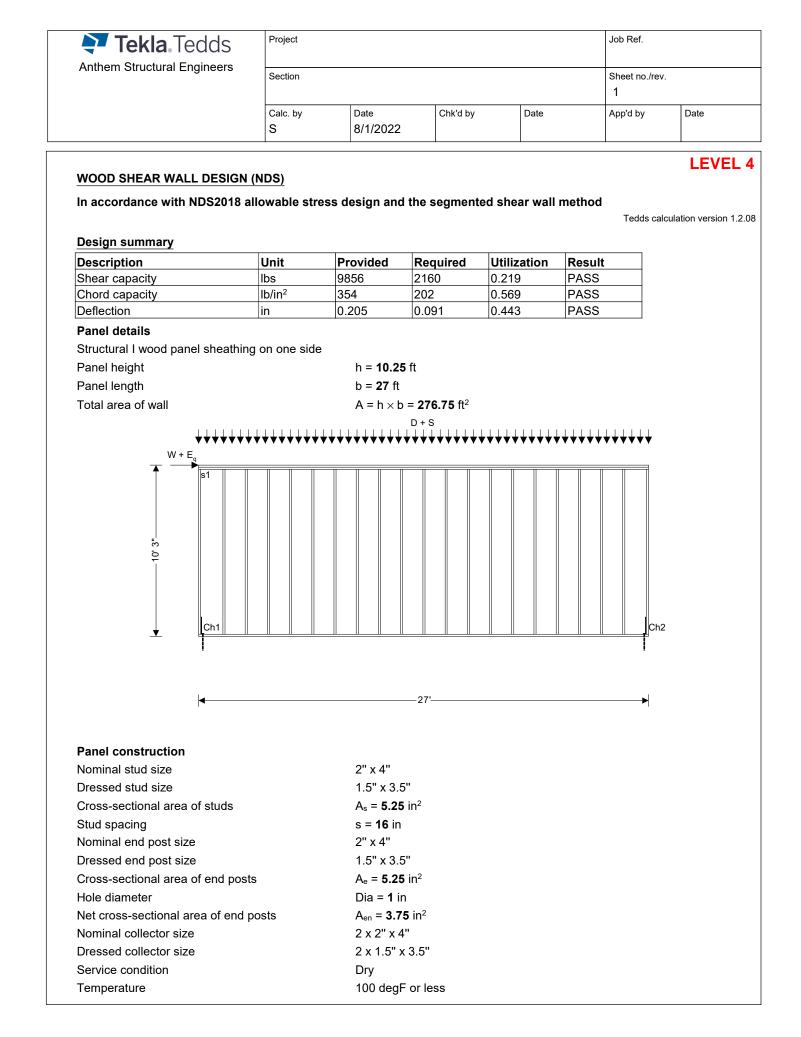
| Maximum applied compressive stress | $f_c = P_{coll} / (2 \times A_s) = 189 \text{ lb/in}^2$ | LEVEL 1 |
|------------------------------------|---|--------------------|
| Column stability factor | C _P = 1.00 | |
| Design compressive stress | $\textbf{F_c'} = \textbf{F_c} \times \textbf{C_D} \times \textbf{C_{Mc}} \times \textbf{C_{tc}} \times \textbf{C_{Fc}} \times \textbf{C_i} \times \textbf{C_P} = \textbf{2376} \text{ lb/in}^2$ | |
| | f _c / F _c ' = 0.080 | |
| P | ASS - Design compressive stress exceeds maximum applied o | compressive stress |
| Hold down force | | |

| Hold down lorce | |
|-----------------|------------------------------------|
| Chord 3 | T ₃ = 5.777 kips |
| Chord 4 | T ₄ = 5.777 kips |
| Chord 5 | T₅ = 5.629 kips |
| Chord 6 | T ₆ = 5.629 kips |

N-S WALLS TYPICAL INTERIOR WALL (SEGMENTED METHOD - TEDDS)



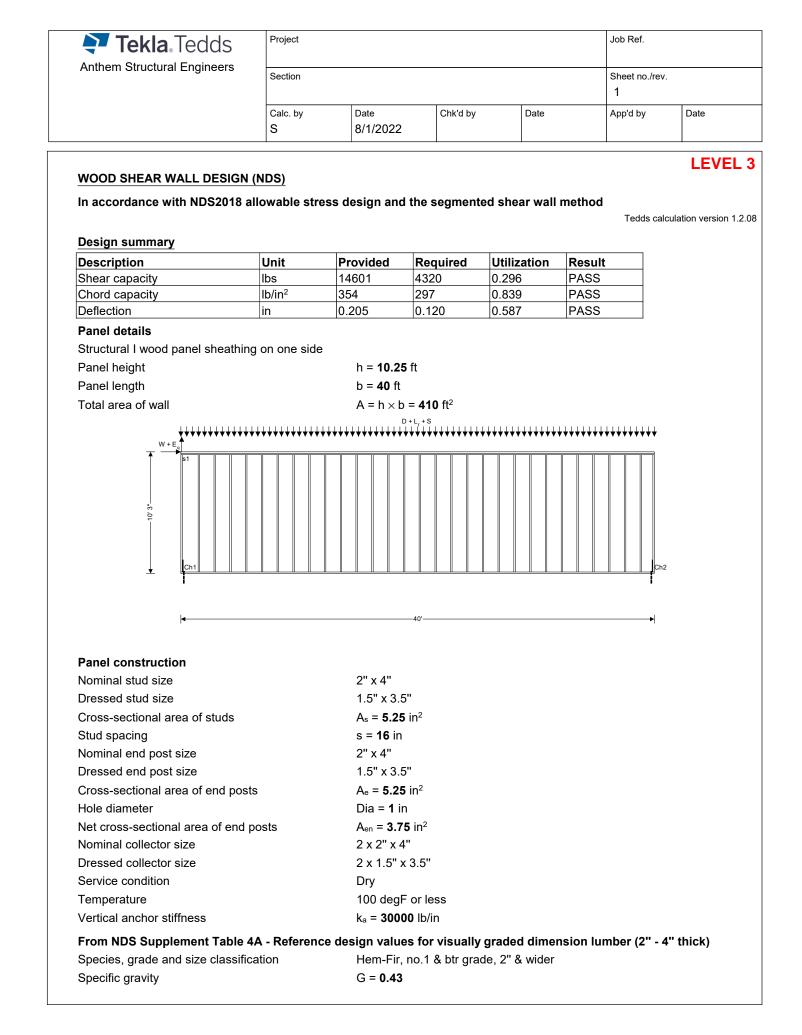




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| Vert | ical anchor stif | fness | | ka = 30000 | lb/in | | | LEVEL 4 | | | |
| | | | | - | | raded dimens | ion lumber (2' | " - 4" thick) | | | |
| Spe | cies, grade and | d size classifica | ation | Hem-Fir, n | o.1 & btr grade | e, 2" & wider | | | | | |
| Spe | cific gravity | | | G = 0.43 | | | | | | | |
| Ten | sion parallel to | grain | | Ft = 725 lb, | /in² | | | | | | |
| Com | npression paral | llel to grain | | Fc = 1350 | b/in² | | | | | | |
| Mod | lulus of elastici | ty | | E = 15000 | 00 lb/in ² | | | | | | |
| Mini | mum modulus | of elasticity | | E _{min} = 5500 |)00 lb/in² | | | | | | |
| She | athing details | | | | | | | | | | |
| She | athing materi | al | | 7/16" woo | d panel struc | tural I oriente | d strandboard | d sheathing | | | |
| | tener type | | | | on nails at 6" | | | U | | | |
| | | | | | | | | | | | |
| | | | | - | | • Shear Walls ·).5 - G), 1] = 5 | | Panels | | | |
| Nor | ninal unit she | ar capacity fo | r wind design | $v_{w} = 785 \text{ r}$ | $f \times min[1 - ()$ | 0.5 - G), 1] = 7 | ' 30.1 lb/ft | | | | |
| | arent shear w | | - | G _a = 16 ki | - ` | | | | | | |
| Арр | | vali shear sun | 11655 | | p5/11 | | | | | | |
| Loa | ding details | | | | | | | | | | |
| Dea | d load acting o | n top of panel | | D = 256 lb/ | ′ft | | | | | | |
| Sno | w load acting c | on top of panel | | S = 1280 II | o/ft | | | | | | |
| Self | weight of pane | el | | S _{wt} = 10 lb/ | ′ft² | | | | | | |
| In pl | lane wind load | acting at head | of panel | W = 3600 | lbs | | | | | | |
| Win | d load servicea | ability factor | | f _{Wserv} = 1.0 | 0 | | | | | | |
| - | lane seismic lo ion spectral res | - | - | E _q = 400 lb ods S _{DS} = 0.33 | | | | | | | |
| | | | - | | • | | | | | | |
| | ord forces fron | | | | | | | | | | |
| Chord | W _{ch[i]} (lbs) | Eq_ch[i] (Ibs) | Dc_ch[i] (Ibs) | DT_ch[i] (Ibs) | L _{f_ch[i]} (Ibs) | Lr_ch[i] (Ibs) | S _{ch[i]} (lbs) | R _{ch[i]} (Ibs) | | | |
| Ch1 | 0; | 0; | 0; | 0; | 0; | 0; | 0; | 0; | | | |
| Ch2 | 0; | 0; | 0; | 0; | 0; | 0; | 0; | 0; | | | |
| Fro | m IBC 2021 cl. | 1605.1 Basic | load combinat | tions from AS | CE 7, section | 2.4 | | | | | |
| Loa | d combination | no.1 | | D + 0.6W | | | | | | | |
| Loa | d combination | no.2 | | D + 0.7E | | | | | | | |
| | d combination | | | D + 0.75L _f | D + 0.75Lf + 0.45W + 0.75(Lr or S or R) | | | | | | |
| Loa | d combination | no.4 | | D + 0.75L _f | + 0.525E + 0.7 | 75S | | | | | |
| Loa | d combination | no.5 | | 0.6D + 0.6 | W | | | | | | |
| Loa | d combination | no.6 | | 0.6D + 0.7 | E | | | | | | |
| Adjı | ustment factor | rs | | | | | | | | | |
| Loa | d duration facto | or – Table 2.3.2 | | C _D = 1.60 | | | | | | | |
| Size | e factor for tens | ion – Table 4A | | C _{Ft} = 1.50 | | | | | | | |
| Size | factor for com | pression – Tab | le 4A | CFc = 1.15 | | | | | | | |
| Wet | service factor | for tension – Ta | able 4A | C _{Mt} = 1.00 | | | | | | | |
| Wet | service factor | for compressic | n – Table 4A | C _{Mc} = 1.00 | | | | | | | |
| Wet | service factor | for modulus of | elasticity – Tab | ole 4A | | | | | | | |
| | | | | C _{ME} = 1.00 | | | | | | | |
| Tem | perature factor | r for tension – | Table 2.3.3 | C _{tt} = 1.00 | | | | | | | |

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| Temperature factor for compress | ion – Table 2.3 | 3.3 | | | I | LEVEI |
| | | C _{tc} = 1.00 | | | | |
| Temperature factor for modulus of | of elasticity – 1 | | | | | |
| Incident factor of 4.2.0 | | $C_{tE} = 1.00$ | | | | |
| Incising factor – cl.4.3.8 Buckling stiffness factor – cl.4.4.2 |) | Ci = 1.00 C⊤ = 1.00 | | | | |
| Adjusted modulus of elasticity | - | | $C_{\text{ME}} 	imes C_{\text{tE}} 	imes C_{\text{i}}$ | × CT = 55000 | 0 psi | |
| Critical buckling design value | | | < E _{min} ' / (h / d) ² | | e poi | |
| Reference compression design v | alue | | $\times C_{Mc} \times C_{tc} \times C_{tc}$ | - | 1 nsi | |
| For sawn lumber | aluc | c = 0.8 | | | P p si | |
| Column stability factor – eqn.3 | 3.7-1 | | F _{cF} / Fc*)) / (2) | × c) – √([(1 + | (F _{cE} / F _c *)) / (2 | \times c)] ² - (F _{CE} / |
| | | F_{c}^{*}) / c) = 0 | | | (1 02 / 1 0)) / (2 | , o)] (i ce i |
| From CDDWC Table 4.2.4 Mavin | | , , | | | | |
| From SDPWS Table 4.3.4 Maxim Maximum shear wall aspect ratio | | 3.5 | 5 | | | |
| Shear wall length | | b = 27 ft | | | | |
| Shear wall aspect ratio | | h / b = 0.38 | | | | |
| Segmented shear wall capacity | 1 | | | | | |
| Maximum shear force under w | | $V_{w max} = 0.6$ | δ × W = 2.16 k | ips | | |
| Shear capacity for wind loadin | • | _ | / 2 = 9.856 kip | | | |
| offical capacity for white loadin | 9 | V _w – V _w × D V _{w max} / V _w = | | 55 | | |
| | | _ | | for wind load | l exceeds maxi | mum shear fo |
| Maximum shear force under s | eismic loadir | | ′ × E _q = 0.28 k | | | |
| Shear capacity for seismic loa | ding | | / 2 = 7.031 kip | | | |
| | 0 | $V_{s max} / V_{s} =$ | • | | | |
| | | — | | r seismic load | l exceeds maxi | mum shear fo |
| Chord capacity for chord 1 | | | | | | |
| Shear wall aspect ratio Load combination 5 | | h / b = 0.38 | | | | |
| Shear force for maximum tension | 1 | $V = 0.6 \times W$ | = 2.16 kips | | | |
| Axial force for maximum tensi | on | P = (0.6 × (| D + S _{wt} × h)) > | × b ₁ / 2 = 2.9 | 04 kips | |
| Maximum tensile force in chord | | $T = V \times h / (I$ | o) - P = <mark>-2.084</mark> | kips 🔶 NE | GATIVE. NO | |
| Maximum applied tensile stress | | $f_t = T / A_{en} =$ | | | ERTURNING | |
| Design tensile stress | | F_t = $F_t \times C_D$ | $	imes C_{Mt} 	imes C_{tt} 	imes C_{F}$ | t × Ci = 1740 | b/in² | |
| | | f _t / F _t ' = -0.3 1 | | | | |
| | | PASS - Des | ign tensile str | ess exceeds | maximum appli | ed tensile str |
| Load combination 1 | | | 0.401 | | | |
| Shear force for maximum compre | | $V = 0.6 \times W$ | - | | | |
| Axial force for maximum comp | | | wt×h))×s / 2 | | | |
| Maximum compressive force in c | | | b) + P = <mark>1.059</mark> | kips | | |
| Wayimum applied compressive s | tress | $f_c = C / A_e =$ | 202 ID/In- | | | |
| Maximum applied compressive st Design compressive stress | | | \times C _{Mc} \times C _{tc} \times C | | 2EA lb/in? | |

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| | PASS - L | Design compres | sive stress ex | ceeds maxim | um applied com | LEVEL | | | |
| Chord capacity for chord 2 | | | | | | | | | |
| Load combination 5 | | | | | | | | | |
| Shear force for maximum tensior | | $V = 0.6 \times W$ | - | | | | | | |
| Axial force for maximum tensi | on | | $D + S_{wt} \times h)) >$ | | - | | | | |
| Maximum tensile force in chord | | | o) - P = <mark>-2.084</mark> | | GATIVE. NO ERTURNING | | | | |
| Maximum applied tensile stress | | $f_t = T / A_{en} =$ | | | | | | | |
| Design tensile stress | | | $\times C_{Mt} \times C_{tt} \times C_{Ft}$ | $1 \times C_i = 1740$ lb | 0/In² | | | | |
| | | $f_t / F_t' = -0.31$ | | ass avcads n | naximum applie | d toncilo stro | | | |
| Load combination 1 | | PASS - Des | iyii terisile sire | ess exceeds n | пахіпійті аррпе | u lensne stre | | | |
| Shear force for maximum compre | ession | $V = 0.6 \times W$ | = 2.16 kips | | | | | | |
| Axial force for maximum com | | | - | = 0 239 kips | | | | | |
| Maximum compressive force in c | | P = ((D + S _{wt} × h)) × s / 2 = 0.239 kips C = V × h / (b) + P = 1.059 kips | | | | | | | |
| Maximum applied compressive s | | $f_c = C / A_e =$ | , | | | | | | |
| 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 | | | | | | | |
| 200.g.t compressive en coc | | f _c / F _c ' = 0.569 | | | | | | | |
| | PASS - L | Design compres | sive stress ex | ceeds maxim | um applied com | pressive stre | | | |
| Wind load deflection | | | | | | | | | |
| Design shear force | | $V_{\delta w} = f_{Wserv}$ | × W = 3.6 kips | 3 | | | | | |
| Deflection limit | | $\Delta_{w_{allow}} = h / 600 = 0.205$ in | | | | | | | |
| Induced unit shear | | $v_{\delta w} = V_{\delta w} / b = 133.33 \text{ lb/ft}$ | | | | | | | |
| Anchor tension force | | $T_{\delta} = max(0 \text{ kips}, v_{\delta w} \times h - 0.6 \times (D + S_{wt} \times h) \times b / 2) = 0.000 \text{ kips}$ | | | | | | | |
| 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 | | | | | | | |
| | | δ_{sww} / Δ_{w_allow} | = 0.443 | | | | | | |
| | | | PASS - She | ar wall deflect | tion is less than | deflection lir | | | |
| Seismic deflection | | | | | | | | | |
| Design shear force | | $V_{\delta s} = E_q = 0$ | .4 kips | | | | | | |
| Deflection limit | | $\Delta s_{allow} = 0.0$ | 20 × h = 2.46 i | in | | | | | |
| Induced unit shear | | $v_{\delta s} = V_{\delta s} / b =$ | = 14.81 lb/ft | | | | | | |
| Anchor tension force | | T_{δ} = max(0 k | ips, $v_{\delta s} 	imes h$ - (0.6 | $6 - 0.2 	imes S_{DS}) 	imes$ | $(D + S_{wt} \times h) \times h$ | o / 2) = 0.000 | | | |
| | | kips | | | | | | | |
| Shear wall elastic deflection - Ec | ın. 4.3-1 | $\delta_{swse} = 2 \times v_{\delta}$ | $_{ m s}$ $	imes$ h ³ / (3 $	imes$ E $	imes$ | $(A_e \times b) + v_{\delta s} \times b$ | $h / (G_a) + h \times T_{\delta}$ | / (k _a ×b) = 0.0 | | | |
| | | in | | | | | | | |
| Deflection ampification factor | | $C_{d\delta} = 4$ | | | | | | | |
| Seismic importance factor | am 10.0.15 | le = 1.25 | | i | | | | | |
| Amp. seis. deflection – ASCE7 E | qn. 12.8-15 | | oswse / le = 0.032 | in | | | | | |
| | | δ_{sws} / Δ_{s_allow} | | | diam in taxa di | | | | |
| | | | PASS - She | ar wall deflect | tion is less than | deflecti | | | |

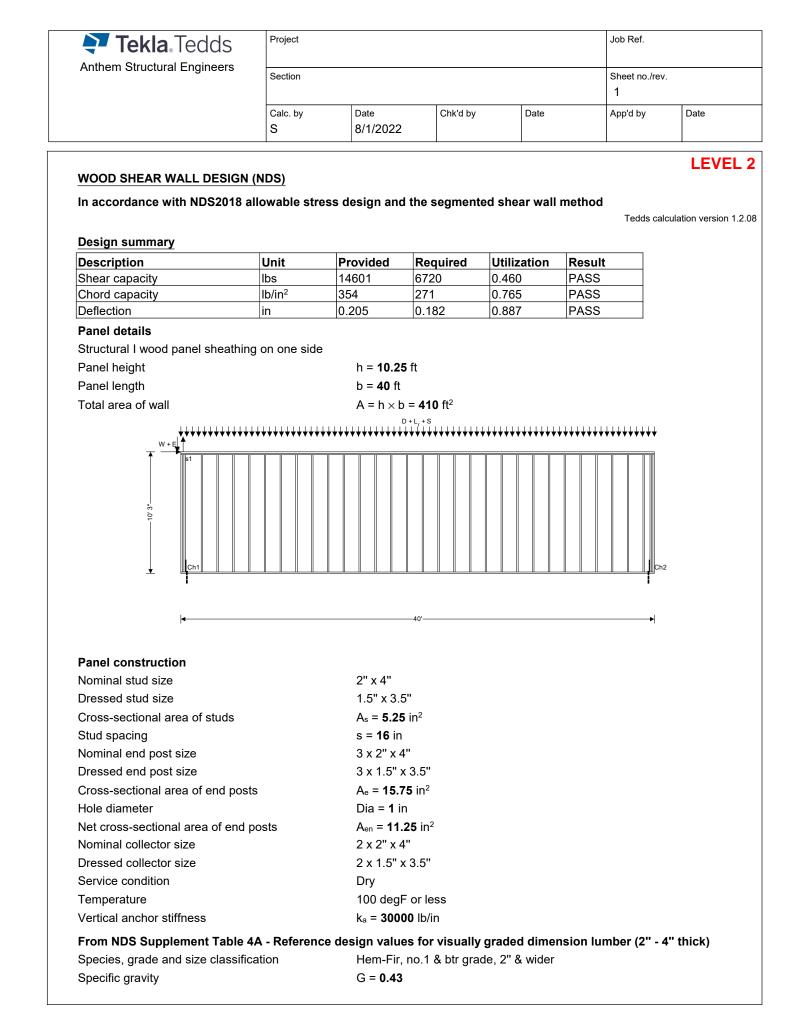


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| Ten | sion parallel to g | grain | | Ft = 725 lb/ | /in ² | | | LEVEL | 3 | | |
| Com | npression parall | el to grain | | Fc = 1350 l | b/in² | | | | | | |
| Mod | lulus of elasticit | у | | E = 15000 |)0 lb/in ² | | | | | | |
| Mini | imum modulus o | of elasticity | | E _{min} = 5500 | E _{min} = 550000 lb/in ² | | | | | | |
| She | athing details | | | | | | | | | | |
| | eathing materia | al | | 7/16" woo | d panel struc | tural I oriente | d strandboard | d sheathing | | | |
| | stener type | | | | on nails at 6" | | | J. J | | | |
| | | la 4 2 A Nomin | al Unit Chaor | Consoltion for | Wood Frame | | Wood bood | Denelo | | | |
| | m SDPWS Tabl | | | - | | | | Paneis | | | |
| | ninal unit shea | | | | | <i>,</i> - | | | | | |
| | ninal unit shea | | 0 | | | 0.5 - G), 1] = 7 | 730.1 lb/ft | | | | |
| Арр | parent shear w | all shear stiff | ness | Ga = 16 ki | ps/in | | | | | | |
| Loa | ding details | | | | | | | | | | |
| Dea | id load acting or | n top of panel | | D = 512 lb/ | ft | | | | | | |
| Floo | or live load actin | g on top of pa | nel | L _f = 640 lb/ | ft | | | | | | |
| Sno | w load acting or | n top of panel | | S = 1280 lk | o/ft | | | | | | |
| Self | weight of panel | l | | S _{wt} = 10 lb/ | ft² | | | | | | |
| In pl | lane wind load a | acting at head | of panel | W = 7200 | bs | | | | | | |
| Win | d load serviceal | bility factor | | f _{Wserv} = 1.0 | D | | | | | | |
| - | lane seismic loa | - | - | E _q = 1000 | | | | | | | |
| Des | ign spectral res | ponse accel. p | par., short perio | ds S _{DS} = 0.33 | 3 | | | | | | |
| Cho | ord forces from | shear walls | above | | | | | | | | |
| Chord | W _{ch[i]} (Ibs) | Eq_ch[i] (Ibs) | Dc_ch[i] (Ibs) | D _{T_ch[i]} (Ibs) | L _{f_ch[i]} (Ibs) | Lr_ch[i] (Ibs) | S _{ch[i]} (Ibs) | R _{ch[i]} (Ibs) | | | |
| Ch1 | -1392; | 0; | 0; | 0; | 0; | 0; | 0; | 0; | | | |
| Ch2 | 0; | 0; | 0; | 0; | 0; | 0; | 0; | 0; | | | |
| From | m IBC 2021 cl.1 | 1605.1 Basic | load combinat | ions from AS | CE 7, section | 2.4 | | | | | |
| Load | d combination n | 10.1 | | D + 0.6W | | | | | | | |
| Load | d combination n | 10.2 | | D + 0.7E | | | | | | | |
| Load | d combination n | 10.3 | | D + 0.75L _f | + 0.45W + 0.7 | ′5(L _r or S or R) | | | | | |
| Load | d combination n | 10.4 | | D + 0.75L _f | + 0.525E + 0.1 | 75S | | | | | |
| | d combination n | - | | 0.6D + 0.6 | N | | | | | | |
| Load | d combination n | 10.6 | | 0.6D + 0.7 | Ξ | | | | | | |
| Adju | ustment factor | s | | | | | | | | | |
| Load | d duration facto | r – Table 2.3.2 | 2 | C _D = 1.60 | | | | | | | |
| Size | e factor for tensi | on – Table 4A | | C _{Ft} = 1.50 | | | | | | | |
| Size | e factor for comp | pression – Tab | le 4A | C _{Fc} = 1.15 | | | | | | | |
| Wet | t service factor f | or tension – Ta | able 4A | C _{Mt} = 1.00 | | | | | | | |
| | t service factor f | • | | C _{Mc} = 1.00 | | | | | | | |
| Wet | t service factor f | or modulus of | elasticity – Tab | | | | | | | | |
| _ | | . | | C _{ME} = 1.00 | | | | | | | |
| | perature factor | | | Ctt = 1.00 | | | | | | | |
| Iem | perature factor | ior compressi | on – Table 2.3. | | | | | | | | |
| Tom | perature factor | for modulus o | f elasticity To | Ctc = 1.00 | | | | | | | |
| Tell | | | | 510 2.0.0 | | | | | | | |

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| | | C _{tE} = 1.00 | | | | LEVE |
| Incising factor – cl.4.3.8 | | C _i = 1.00 | | | | |
| Buckling stiffness factor – cl.4.4.2 | | C⊤ = 1.00 | | | | |
| Adjusted modulus of elasticity | | E_{min} ' = $E_{min} \times C$ | $C_{ME} \times C_{tE} \times C_{i}$ | < C⊤ = 550000 | psi | |
| Critical buckling design value | | F_{cE} = 0.822 $	imes$ | E _{min} ' / (h / d)² = | = 366 psi | | |
| Reference compression design valu | le | $F_{c}^{*} = F_{c} \times C_{D} \times$ | $\mathbf{K} \mathbf{C}_{Mc} \times \mathbf{C}_{tc} \times \mathbf{C}_{tc}$ | Fc × Ci = 2484 | psi | |
| For sawn lumber | | c = 0.8 | | | | |
| Column stability factor - eqn.3.7 | -1 | $C_{P} = (1 + (F_{cl}))$ | _E / F _c *)) / (2 × | <pre>c c) – √([(1 + (</pre> | F _{cE} / F _c *)) / (2 × | c c)] ² - (F _{cE} |
| | | F _c *) / c) = 0.1 | 4 | | | |
| From SDPWS Table 4.3.4 Maximu | ım 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 win | d loading | $V_{w_{max}} = 0.6$ | < W = 4.32 ki | ps | | |
| Shear capacity for wind loading | | $V_w = v_w \times b / $ | 2 = 14.601 ki | ps | | |
| | | $V_{w_{max}} / V_{w} = 0$ | | | | |
| | | | | | exceeds maxim | um shear f |
| Maximum shear force under seis | Ũ | $V_{s_max} = 0.7 \times$ | • | | | |
| Shear capacity for seismic loading | ng | $V_s = v_s \times b / 2$ | 2 = 10.416 kip | DS | | |
| | | $V_{s_{max}} / V_{s} = 0.$ | | a a ia mia la a d | avaa da maxim | |
| . | | PASS - Shear | capacity for | seismic Ioad | exceeds maxim | um snear f |
| Chord capacity for chord 1 | | h / b = 0.256 | | | | |
| Shear wall aspect ratio Load combination 5 | | n/b - 0.256 | | | | |
| Shear force for maximum tension | | V = 0.6 × W = | 4.32 kips | | | |
| Axial force for maximum tension | 1 | | • | (b ₁ /2+0.6× | ≪W _{ch1} = 6.539 k | ips |
| Maximum tensile force in chord | | $T = V \times h / (b)$ | ,, | | | 1 |
| Maximum applied tensile stress | | $f_t = T / A_{en} = -1$ | | | RTURNING | |
| Design tensile stress | | | | × Ci = 1740 lb/ | ′in² | |
| C C | | f _t / F _t ' = -0.832 | | | | |
| | | PASS - Desig | n tensile stre | ess exceeds m | naximum applie | d tensile st |
| Load combination 1 | | | | | | |
| Shear force for maximum compress | | V = 0.6 × W = | • | | | |
| Axial force for maximum compre | ession | $P = ((D + S_{wt}$ | \times h)) \times s / 2 | + -1 \times 0.6 \times V | V _{ch1} = 1.245 kip | s |
| Maximum compressive force in cho | rd | $C = V \times h / (b)$ | + P = <mark>2.352 k</mark> | tips | | |
| Maximum applied compressive stre | SS | $f_c = C / A_e = 44$ | 18 lb/in ² | | | |
| Design compressive stress | | $F_c' = F_c \times C_D \times$ | | $F_{C} \times C_{i} \times C_{P} = 3$ | 54 lb/in ² | |
| | | f _c / F _c ' = 1.264 | | | | |

Chord capacity for chord 2 Load combination 5

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| Shear force for maximum tension | | V = 0.6 × W | = 4.32 kips | | | LEVE | | | |
| Axial force for maximum tension | on | P = (0.6 × (I | P = $(0.6 \times (D + S_{wt} \times h)) \times b_1 / 2 = 7.374$ kips T = V × h / (b) - P = -6.267 kips ← NEGATIVE. NO | | | | | | |
| Maximum tensile force in chord | | | | | | | | | |
| | | $T = V \times h / (b) - P = -6.267 \text{ kips} \longleftarrow \text{NEGATIVE. NO}$ | | | | | | | |
| Maximum applied tensile stress Design tensile stress | | $F_t' = F_t \times C_D$ | $\mathbf{K} \mathbf{C}_{Mt} \times \mathbf{C}_{tt} \times \mathbf{C}_{Ft}$ | × C _i = 1740 | lb/in ² | | | | |
| | | f _t / F _t ' = -0.96 | 0 | | | | | | |
| | | PASS - Desi | gn tensile stre | ss exceeds | s maximum appl | lied tensile st | | | |
| Load combination 3 | | | | | | | | | |
| Shear force for maximum compre | ssion | $V = 0.45 \times W$ | ′ = 3.24 kips | | | | | | |
| Axial force for maximum comp | oression | P = ((D + S ₁ | _{vt} × h) + 0.75 > | < L _f) × s / 2 | = 0.73 kips | | | | |
| Maximum compressive force in cl | nord | $C = V \times h / (k$ | o) + P = <mark>1.560</mark> k | ips | | | | | |
| Maximum applied compressive st | ress | $f_c = C / A_e = 2$ | 2 97 lb/in² | | | | | | |
| Design compressive stress | | $F_c' = F_c \times C_D$ | \times C _{Mc} \times C _{tc} \times C | $c_{\rm c} \times C_{\rm i} \times C_{\rm P}$ | = 354 lb/in² | | | | |
| | | f _c / F _c ' = 0.83 | | | | | | | |
| | PASS - | Design compress | sive stress exc | eeds maxi | mum applied co | mpressive st | | | |
| Wind load deflection | | | | | | | | | |
| Design shear force | | $V_{\delta w} = f_{Wserv}$ | < W = 7.2 kips | | | | | | |
| Deflection limit | | $\Delta_{w_{allow}} = h / b$ | 600 = 0.205 in | | | | | | |
| Induced unit shear | | $v_{\delta w} = V_{\delta w} / b =$ | = 180 lb/ft | | | | | | |
| Anchor tension force | | $T_{\delta} = max(0 k)$ | $ps, v_{\delta w} 	imes h - 0.6$ | \times (D + S _{wt} \times | < h) × b / 2 + | | | | |
| | | max(abs(W _{ct} | 1),abs(W _{ch2}))) = | 0.000 kips | | | | | |
| Shear wall deflection - Eqn. 4.3-1 | | δ_{sww} = 2 × $v_{\delta w}$ | imes h ³ / (3 $	imes$ E $	imes$ | $A_e \times b$) + $v_{\delta v}$ | $_{a} \times h / (G_{a}) + h \times $ | $T_{\delta} / (k_a \times b) = C$ | | | |
| | | in | | | | | | | |
| | | δ_{sww} / Δ_{w_allow} | | | | | | | |
| | | | PASS - Shea | r wall defle | ection is less the | an deflection | | | |
| Seismic deflection | | | | | | | | | |
| Design shear force | | $V_{\delta s} = E_q = 1$ | kips | | | | | | |
| Deflection limit | | Δ_{s_allow} = 0.02 | 20 × h = 2.46 i | n | | | | | |
| Induced unit shear | | $v_{\delta s} = V_{\delta s} / b =$ | 25 lb/ft | | | | | | |
| Anchor tension force | | $T_{\delta} = max(0 k)$ | $ips, v_{\delta s} 	imes h$ - (0.6 | - $0.2 \times S_{\text{DS}}$ | $) \times (D + S_{wt} \times h) >$ | < b / 2) = 0.000 | | | |
| | | kips | | | | | | | |
| Shear wall elastic deflection – Eq | n. 4.3-1 | δ _{swse} = 2 × v _{δs} 0.017 in | $_{3} \times h^{3} / (3 \times E \times$ | $A_e \times b$) + $v_{\delta e}$ | $_{s} \times h / (G_{a}) + h \times $ | $T_{\delta} / (k_a \times b) =$ | | | |
| Deflection ampification factor | | $C_{d\delta} = 4$ | | | | | | | |
| Seismic importance factor | | l _e = 1.25 | | | | | | | |
| Amp. seis. deflection – ASCE7 Ed | qn. 12.8-15 | | _{swse} / I _e = 0.053 | in | | | | | |
| | | δ_{sws} / $\Delta_{s_{allow}}$ = | = 0.022 | | | | | | |
| | | | | | | | | | |

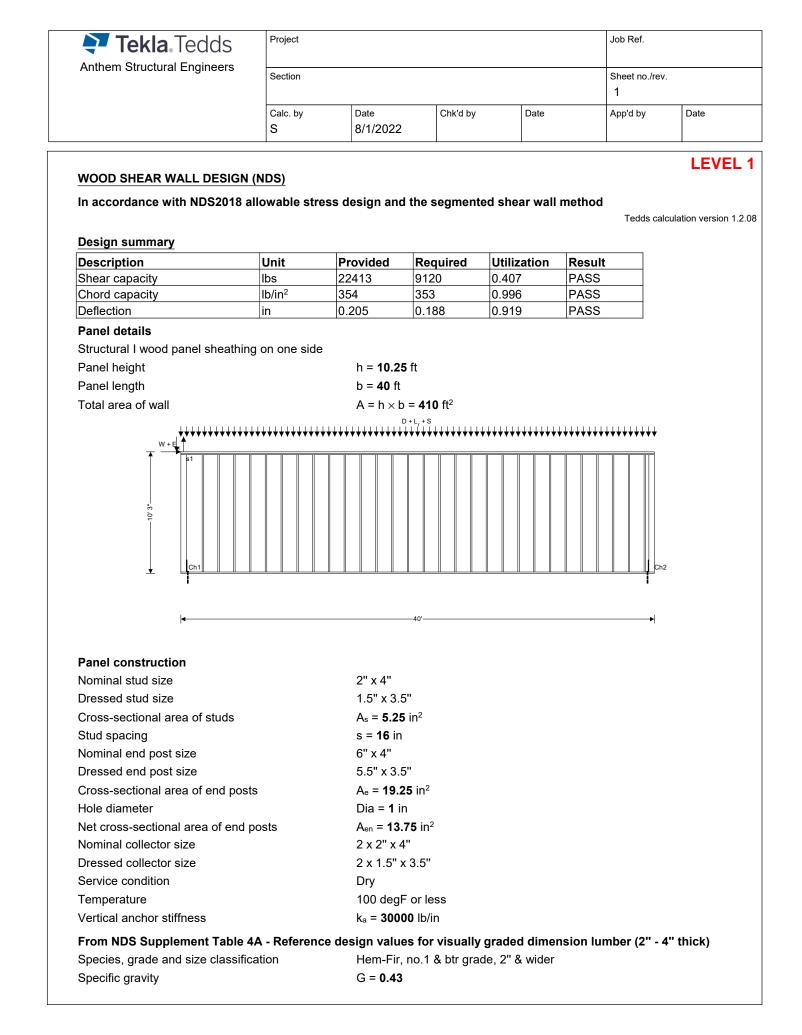


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| Tens | sion parallel to g | prain | | Ft = 725 lb/ | ín² | | | LEVEL 2 | | | |
| | npression paralle | - | | F _c = 1350 | | | | | | | |
| | Iulus of elasticity | - | | E = 150000 |)0 lb/in ² | | | | | | |
| Mini | mum modulus c | of elasticity | | E _{min} = 5500 | E _{min} = 550000 lb/in ² | | | | | | |
| She | athing details | | | | | | | | | | |
| | athing materia | al | | 7/16'' woo | d panel struc | ctural I oriente | d strandboard | d sheathing | | | |
| | tener type | | | | on nails at 6" | | | g | | | |
| | | | | | | | | | | | |
| | m SDPWS Tabl | | | - | | | | l Panels | | | |
| | ninal unit shea | | | • • | | | | | | | |
| Non | ninal unit shea | r capacity fo | r wind design | v _w = 785 p | $f \times min[1 - (0)]$ | 0.5 - G), 1] = 7 | 730.1 lb/ft | | | | |
| Арр | parent shear wa | all shear stiff | ness | Ga = 16 ki | ps/in | | | | | | |
| Loa | ding details | | | | | | | | | | |
| Dea | d load acting on | n top of panel | | D = 774 lb/ | ft | | | | | | |
| Floo | or live load acting | g on top of pa | nel | L _f = 512 lb/ | ft | | | | | | |
| Sno | w load acting or | n top of panel | | S = 1280 lk | o/ft | | | | | | |
| Self | weight of panel | | | S _{wt} = 10 lb/ | ft² | | | | | | |
| In pl | lane wind load a | acting at head | of panel | W = 11200 | lbs | | | | | | |
| Win | d load serviceat | oility factor | | f _{Wserv} = 1.0 |) | | | | | | |
| - | lane seismic loa ign spectral resi | - | - | E _q = 1600 | | | | | | | |
| | | | - | us obs - 0.33. | | | | | | | |
| | ord forces from | | | | | | 0 (11) | | | | |
| Chord | W _{ch[i]} (lbs) | Eq_ch[i] (lbs) | Dc_ch[i] (lbs) | DT_ch[i] (Ibs) | Lf_ch[i] (lbs) | Lr_ch[i] (lbs) | S _{ch[i]} (Ibs) | R _{ch[i]} (lbs) | | | |
| Ch1 | -3272; | 0; | 0; | 0; | 0; | 0; | 0; | 0; | | | |
| Ch2 | 3272; | 0; | 0; | 0; | 0; | 0; | 0; | 0; | | | |
| | m IBC 2021 cl.1 | | load combinat | | CE 7, section | 2.4 | | | | | |
| | d combination n | | | D + 0.6W | | | | | | | |
| | d combination n | | | D + 0.7E | | | | | | | |
| | d combination n | | | | | '5(L _r or S or R) | | | | | |
| | d combination n | | | | + 0.525E + 0.7 | 758 | | | | | |
| | d combination n | | | 0.6D + 0.6 | | | | | | | |
| Load | d combination n | 0.0 | | 0.6D + 0.7I | = | | | | | | |
| - | ustment factors | | | | | | | | | | |
| | d duration factor | | | C _D = 1.60 | | | | | | | |
| | e factor for tensio | | | C _{Ft} = 1.50 | | | | | | | |
| | e factor for comp | | | C _{Fc} = 1.15 | | | | | | | |
| | service factor for | | | C _{Mt} = 1.00 | | | | | | | |
| | service factor fo | • | | C _{Mc} = 1.00 | | | | | | | |
| vvet | service factor for | or modulus of | | | | | | | | | |
| Tom | perature factor | for tension | Table 2 2 2 | C _{ME} = 1.00 C _{tt} = 1.00 | | | | | | | |
| | perature factor | | | | | | | | | | |
| | | | IGDIC 2.0. | C _{tc} = 1.00 | | | | | | | |
| Tem | perature factor | for modulus o | f elasticity – Ta | | | | | | | | |

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| S | 6 | 8/1/2022 | | | | | |
| | | C _{tE} = 1.00 | | | | | LEVE |
| 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 | | $E_{min}' = E_{min} \times C$ | | | 000 psi | | |
| Critical buckling design value | | $F_{cE} = 0.822 \times E$ | = _{min} ' / (h / d) ² = | = 366 psi | | | |
| Reference compression design value | | $F_{c}^{*} = F_{c} \times C_{D} \times$ | $C_{\text{Mc}} \times C_{\text{tc}} \times C$ | $F_c \times C_i = 24$ | 484 psi | | |
| For sawn lumber | | c = 0.8 | | | | | |
| Column stability factor - eqn.3.7-1 | | $C_{P} = (1 + (F_{cE}))$ | = / F _c *)) / (2 × | c) – √([(1 | 1 + (F _{cE} / F _c * |)) / (2 × | c)] ² - (F _{cE} |
| | | F _c *) / c) = 0.1 4 | 4 | | | | |
| From SDPWS Table 4.3.4 Maximum Maximum shear wall aspect ratio | n Shear Wall | Aspect Ratios 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 ki | ps | | | |
| Shear capacity for wind loading | - | $V_w = v_w \times b / 2$ | 2 = 14.601 kii | os | | | |
| | | $V_{w_{max}} / V_w = 0$ | .46 | | and avanada | movimu | um abaar f |
| Maximum shear force under seisr | nic looding | $V_{s_{max}} = 0.7 \times$ | ear capacity f | | Dad exceeds | maximu | ini shear i |
| Shear capacity for seismic loading | Ũ | $V_s = V_s \times b / 2$ | | | | | |
| Shear capacity for seisific loading | 1 | | • | 15 | | | |
| | | V _{s_max} / V _s = 0. PASS - Shear | | seismic lo | oad exceeds | maximu | ım shear f |
| Chord capacity for chord 1 | | | | | | | |
| Shear wall aspect ratio Load combination 5 | | h / b = 0.256 | | | | | |
| Shear force for maximum tension | | V = 0.6 × W = | 6.72 kips | | | | |
| Axial force for maximum tension | | P = (0.6 × (D | + S _{wt} \times h)) \times | b ₁ / 2 + 0 | $0.6 \times W_{ch1} =$ | 8.555 ki | ps |
| Maximum tensile force in chord | | $T = V \times h / (b)$ | - P = <mark>-6.833</mark> k | ips ┥ 🗕 | NEGATIVE. N | 0 | |
| Maximum applied tensile stress | | f _t = T / A _{en} = -6 | 07 lb/in ² | | OVERTURNIN | ١Ġ | |
| Design tensile stress | | F_t = $F_t \times C_D \times C_D$ | $C_{Mt} 	imes C_{tt} 	imes C_{Ft}$ | × Ci = 174 | 10 lb/in ² | | |
| | | f _t / F _t ' = -0.349 | | | | | |
| | | PASS - Desig | n tensile stre | ss exceed | ds maximum | applied | l tensile st |
| Load combination 1 | | | • =• · · | | | | |
| Shear force for maximum compression | | V = 0.6 × W = | • | | | | |
| Axial force for maximum compres | | $P = ((D + S_{wt}$ | | | $\delta \times W_{ch1} = 2.$ | 548 kips | 5 |
| Maximum compressive force in chord | | $C = V \times h / (b)$ | | ips | | | |
| Maximum applied compressive stress | 3 | $f_c = C / A_e = 27$ | | | | | |
| Design compressive stress | | $F_{c}' = F_{c} \times C_{D} \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_{i} \times C_{P} = 354 \text{ lb/in}^{2}$ | | | | | |
| | | fc / Fc' = 0.765 | | | | | |

Chord capacity for chord 2 Load combination 5

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| Shear force for maximum tension | | V = 0.6 × W | = 6.72 kips | | | LEVE | | |
| Axial force for maximum tension | 'n | | | $h_1/2 + -1$ | $\times 0.6 \times W_{ch2} = 8$ | 555 kins | | |
| | '''' | $P = (0.6 \times (D + S_{wt} \times h)) \times b_1 / 2 + -1 \times 0.6 \times W_{ch2} = 8.555 \text{ kips}$ T = V × h / (b) - P = -6.833 kips VEGATIVE. NO | | | | | | |
| Maximum tensile force in chord Maximum applied tensile stress Design tensile stress | | $T = V \times h / (b) - P = -6.833 \text{ kips} \bullet \text{NEGATIVE. NO}$ $f_t = T / A_{en} = -607 \text{ lb/in}^2 \bullet \text{OVERTURNING}$ | | | | | | |
| | | | × Смt × Ctt × CFt | × Ci = 1740 | b/in² | | | |
| 5 | | f _t / F _t ' = -0.34 | | | | | | |
| | | | | ss exceeds | maximum appli | ed tensile s | | |
| Load combination 1 | | | - | | | | | |
| Shear force for maximum compre | ssion | $V = 0.6 \times W$ | = 6.72 kips | | | | | |
| Axial force for maximum comp | ression | P = ((D + S | _{wt} × h)) × s / 2 | + $0.6 	imes W_{ch2}$ | 2 = 2.548 kips | | | |
| Maximum compressive force in ch | ord | $C = V \times h / (t)$ | o) + P = <mark>4.270</mark> k | ips | | | | |
| Maximum applied compressive st | | $f_{c} = C / A_{e} = 2$ | | | | | | |
| Design compressive stress | | $F_c' = F_c \times C_D$ | $	imes C_{Mc} 	imes C_{tc} 	imes C$ | $c_{c} \times C_{i} \times C_{P} =$ | 354 lb/in ² | | | |
| - | | f _c / F _c ' = 0.76 | 5 | | | | | |
| | PASS - | Design compres | sive stress exc | eeds maxin | num applied con | npressive s | | |
| Wind load deflection | | | | | | | | |
| Design shear force | | V _{δw} = f _{Wserv} : | × W = 11.2 kip | s | | | | |
| Deflection limit | | | , 600 = 0.205 in | | | | | |
| Induced unit shear | | $v_{\delta w} = V_{\delta w} / b$ | | | | | | |
| Anchor tension force | | | ips,v _{ðw} × h - 0.6 | \times (D + S _{ut} \times | h) × h / 2 + | | | |
| | | | • | • | | | | |
| Shear wall deflection – Eqn. 4.3-1 | | $\begin{split} &\max(abs(W_{ch1}),abs(W_{ch2}))) = \textbf{0.000} \text{ kips} \\ &\delta_{sww} = 2 \times v_{\delta w} \times h^3 / (3 \times E \times A_e \times b) + v_{\delta w} \times h / (G_a) + h \times T_{\delta} / (k_a \times b) = \\ &\textbf{0.182} \text{ in} \end{split}$ | | | | | | |
| | | δ_{sww} / Δ_{w_allow} | = 0.887 | | | | | |
| | | | PASS - Shea | r wall defled | ction is less tha | n deflection | | |
| Seismic deflection | | | | | | | | |
| Design shear force | | $V_{\delta s} = E_q = 1$ | . 6 kips | | | | | |
| Deflection limit | | $\Delta_{s allow} = 0.02$ | 20 × h = 2.46 i | า | | | | |
| Induced unit shear | | | | | | | | |
| Anchor tension force | | T _δ = max(0 k kips | ips, $v_{\delta s} 	imes h$ - (0.6 | - $0.2 \times S_{DS}$) | \times (D + S _{wt} \times h) \times | b / 2) = 0.00 | | |
| Shear wall elastic deflection – Eq | า. 4.3-1 | δ _{swse} = 2 × v _δ 0.026 in | $_{s}$ $	imes$ h ³ / (3 $	imes$ E $	imes$ | $A_e \times b$) + $v_{\delta s}$: | imes h / (G _a) + h $	imes$ Ta | $_{\delta}$ / (k _a × b) = | | |
| Deflection ampification factor | | $C_{d\delta} = 4$ | | | | | | |
| Seismic importance factor | | l _e = 1.25 | | | | | | |
| Amp. seis. deflection – ASCE7 Ec | n. 12.8-15 | $\delta_{\text{sws}} = C_{\text{d}\delta} \times \delta$ | _{swse} / I _e = 0.083 | in | | | | |
| | | δ_{sws} / Δ_{s_allow} = | = 0.034 | | | | | |
| | | | | | | | | |



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| | | | | | | | 2 | | | | | |
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| Ten | sion parallel to g | grain | | Ft = 725 lb/ | in ² | | | LEVE | L 1 | | | |
| | npression paralle | - | | F₀ = 1350 I | o/in² | | | | | | | |
| Mod | lulus of elasticity | / | | E = 150000 | 0 lb/in ² | | | | | | | |
| Mini | mum modulus c | of elasticity | | E _{min} = 5500 | E _{min} = 550000 lb/in ² | | | | | | | |
| She | athing details | | | | | | | | | | | |
| | athing materia | al | | 7/16'' woo | d panel struc | tural I oriente | d strandboard | d sheathing | | | | |
| | tener type | | | | on nails at 4" | | | J. J | | | | |
| | | o 4 2 A Nomir | al Unit Shoor | | | | Wood based | Banala | | | | |
| | m SDPWS Tabl | | | - | | | | Paneis | | | | |
| | ninal unit shea | | | | | , - | | | | | | |
| | ninal unit shea | | • | | | (0.5 - G), 1] = | 1120.6 lb/ft | | | | | |
| Арр | parent shear wa | all shear stiff | ness | G _a = 21 ki | os/in | | | | | | | |
| Loa | ding details | | | | | | | | | | | |
| Dea | d load acting on | top of panel | | D = 1024 lk | o/ft | | | | | | | |
| Floc | or live load acting | g on top of pa | nel | L _f = 1920 lk | o/ft | | | | | | | |
| Sno | w load acting or | n top of panel | | S = 1280 lb | /ft | | | | | | | |
| Self | weight of panel | | | S _{wt} = 10 lb/ | ft ² | | | | | | | |
| | lane wind load a | - | of panel | W = 15200 | lbs | | | | | | | |
| Win | d load serviceat | oility factor | | f _{Wserv} = 1.00 |) | | | | | | | |
| - | lane seismic loa | - | - | Eq = 2250 | | | | | | | | |
| Des | ign spectral resp | ponse accel. p | oar., short perio | ds $S_{DS} = 0.333$ | 3 | | | | | | | |
| | ord forces from | | | | | | | , | | | | |
| Chord | W _{ch[i]} (Ibs) | Eq_ch[i] (Ibs) | Dc_ch[i] (Ibs) | D _{T_ch[i]} (Ibs) | L _{f_ch[i]} (Ibs) | Lr_ch[i] (Ibs) | S _{ch[i]} (Ibs) | R _{ch[i]} (Ibs) | | | | |
| Ch1 | -6177; | 0; | 0; | 0; | 0; | 0; | 0; | 0; | | | | |
| Ch2 | 6177; | 0; | 0; | 0; | 0; | 0; | 0; | 0; | | | | |
| Fro | m IBC 2021 cl.1 | 605.1 Basic | load combinat | ions from ASC | CE 7, section | 2.4 | | | | | | |
| | d combination n | | | D + 0.6W | | | | | | | | |
| | d combination n | | | D + 0.7E | | | | | | | | |
| | d combination n | | | | | ′5(L _r or S or R) | | | | | | |
| | d combination n | | | | + 0.525E + 0. | 75S | | | | | | |
| | d combination n | | | 0.6D + 0.6\ | | | | | | | | |
| Loa | d combination n | 0.6 | | 0.6D + 0.7I | = | | | | | | | |
| Adju | ustment factors | 5 | | | | | | | | | | |
| Loa | d duration factor | r – Table 2.3.2 | 2 | C _D = 1.60 | | | | | | | | |
| Size | e factor for tensio | on – Table 4A | | C _{Ft} = 1.50 | | | | | | | | |
| | e factor for comp | | | C _{Fc} = 1.15 | | | | | | | | |
| | service factor for | | | C _{Mt} = 1.00 | | | | | | | | |
| | service factor for | • | | C _{Mc} = 1.00 | | | | | | | | |
| Wet | service factor for | or modulus of | elasticity – Tab | | | | | | | | | |
| – | noroting ft- | for tonnian - | | $C_{ME} = 1.00$ | | | | | | | | |
| | nperature factor | | | Ctt = 1.00 | | | | | | | | |
| 101 | | ior compressi | on - Table 2.3. | C _{tc} = 1.00 | | | | | | | | |
| Tem | perature factor | for modulus o | f elasticity – Ta | | | | | | | | | |
| | | | | | | | | | | | | |

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| | | CtE = 1.00 | | | | LEVE | | | |
| Incising factor – cl.4.3.8 | | C _i = 1.00 | | | | | | | |
| Buckling stiffness factor – cl.4.4.2 | | C⊤ = 1.00 | | | | | | | |
| Adjusted modulus of elasticity | $E_{min}' = E_{min} \times C$ | | | psi | | | | | |
| Critical buckling design value | | $F_{cE} = 0.822 \times E$ | = _{min} ' / (h / d) ² = | = 366 psi | | | | | |
| Reference compression design value | e | $F_{c^*} = F_c \times C_D \times$ | $C_{\text{Mc}} \times C_{\text{tc}} \times C$ | _{Fc} × 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 × | : c) – √([(1 + (| (F _{cE} / F _c *)) / (2 > | ≺ c)]² - (F _{cE} | | | |
| | | F _c *) / c) = 0.1 4 | 4 | | | | | | |
| From SDPWS Table 4.3.4 Maximum | n Shear Wall | • | | | | | | | |
| Maximum shear wall aspect ratio Shear wall length | | 3.5 b = 40 ft | | | | | | | |
| Shear wall length Shear wall aspect ratio | | b = 40 π h / b = 0.256 | | | | | | | |
| Segmented shear wall capacity | | II / D = 0.230 | | | | | | | |
| Maximum shear force under wind | loading | $V_{w max} = 0.6 \times$ | /// = 9 12 ki | ne | | | | | |
| | loauling | - | | • | | | | | |
| Shear capacity for wind loading | | $V_w = v_w \times b / 2$ | | ps | | | | | |
| | | $V_{w_{max}} / V_{w} = 0$ | | for wind lood | avaaada maxim | um abaar f | | | |
| Maximum shear force under seisi | mic loading | $V_{s_{max}} = 0.7 \times$ | | | exceeds maxim | iuiii Shedi i | | | |
| | U U | _ | | • | | | | | |
| Shear capacity for seismic loading | g | $V_s = v_s \times b / 2$ | • | DS | | | | | |
| | | V _{s_max} / V _s = 0. PASS - Shear | | seismic load | exceeds maxim | num shear f | | | |
| 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 × (D | + S _{wt} \times h)) \times | b ₁ /2+0.6> | < W _{ch1} = 9.812 k | kips | | | |
| Maximum tensile force in chord | | $T = V \times h / (b)$ | - P = <mark>-7.475</mark> k | ips 🔶 NE | GATIVE. NO | | | | |
| Maximum applied tensile stress | | ft = T / A _{en} = -5 | 44 lb/in ² | OV | ERTURNING | | | | |
| Design tensile stress | | F_t = $F_t \times C_D \times C_D$ | $C_{Mt} 	imes C_{tt} 	imes C_{Ft}$ | × Ci = 1740 lb | /in² | | | | |
| | | f _t / F _t ' = -0.312 | | | | | | | |
| | | PASS - Desig | n tensile stre | ss exceeds n | naximum applie | d tensile st | | | |
| Load combination 1 | | | | | | | | | |
| Shear force for maximum compression | on | $V = 0.6 \times W =$ | 9.12 kips | | | | | | |
| Axial force for maximum compres | ssion | $P = ((D + S_{wt}))$ | \times h)) \times s / 2 | + -1 × 0.6 × \ | N _{ch1} = 4.457 kip | S | | | |
| Maximum compressive force in chord | b | $C = V \times h / (b)$ | + P = <mark>6.794</mark> k | tips | | | | | |
| | s | C = V × h / (b) + P = <mark>6.794 kips</mark> f _c = C / A _e = 353 lb/in ² | | | | | | | |
| Maximum applied compressive stres | | | | $F_{c}' = F_{c} \times C_{D} \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_{i} \times C_{P} = 354 \text{ lb/in}^{2}$ | | | | | |
| Maximum applied compressive stres Design compressive stress | | F_c ' = $F_c \times C_D \times$ | $C_{\text{Mc}} 	imes C_{\text{tc}} 	imes C_{\text{F}}$ | $F_{C} \times C_{i} \times C_{P} = 3$ | 354 lb/in ² | | | | |

Chord capacity for chord 2 Load combination 5

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| Shear force for maximum tension | | V = 0.6 × W | = 9.12 kips | | | LEVE | | |
| Axial force for maximum tensior | ı | P = (0.6 × (| $(D + S_{wt} \times h)) \times$ | b ₁ /2+-1 | $\times 0.6 \times W_{ch2} = 9$ | .812 kips | | |
| Maximum tensile force in chord | | $P = (0.6 \times (D + S_{wt} \times h)) \times b_1 / 2 + -1 \times 0.6 \times W_{ch2} = 9.812 \text{ kips}$ $T = V \times h / (b) - P = -7.475 \text{ kips} \bullet \qquad \text{NEGATIVE. NO}$ $f_t = T / A_{en} = -544 \text{ lb/in}^2 \bullet \qquad \text{OVERTURNING}$ | | | | | | |
| | | T = V × h / (b) - P = -7.475 kips | | | | | | |
| Maximum applied tensile stress Design tensile stress | | $F_t' = F_t \times C_D$ | $\mathbf{K} \mathbf{C}_{Mt} \times \mathbf{C}_{tt} \times \mathbf{C}_{Ft}$ | × Ci = 1740 | lb/in² | | | |
| | | f _t / F _t ' = -0.31 | 2 | | | | | |
| | | PASS - Desi | gn tensile stre | ss exceeds | maximum appli | ed tensile s | | |
| Load combination 1 | | | | | | | | |
| Shear force for maximum compres | sion | $V = 0.6 \times W$ | = 9.12 kips | | | | | |
| Axial force for maximum compre | ession | P = ((D + S, | $_{vt} \times h)) \times s / 2$ | + $0.6 \times W_{ch}$ | ₂ = 4.457 kips | | | |
| Maximum compressive force in cho | ord | $C = V \times h / (k$ | o) + P = <mark>6.794</mark> k | ips | | | | |
| Maximum applied compressive stre | ss | $f_c = C / A_e = C$ | 353 lb/in ² | | | | | |
| Design compressive stress | | $F_c' = F_c \times C_D$ | $	imes C_{Mc} 	imes C_{tc} 	imes C_{tc}$ | $c_{c} \times C_{i} \times C_{P} =$ | 354 lb/in ² | | | |
| | | fc / Fc' = 0.99 | 6 | | | | | |
| | PASS - | Design compres | sive stress exc | eeds maxin | num applied con | npressive s | | |
| Wind load deflection | | | | | | | | |
| Design shear force | | V _{δw} = f _{Wserv} > | < W = 15.2 kip | s | | | | |
| Deflection limit | | $\Delta_{\rm w}$ allow= h / | 600 = 0.205 in | | | | | |
| Induced unit shear | | $v_{\delta w} = V_{\delta w} / b =$ | = 380 lb/ft | | | | | |
| Anchor tension force | | T _δ = max(0 k | ips, $v_{\delta w} 	imes h - 0.6$ | \times (D + S _{wt} \times | h)×b/2+ | | | |
| | | max(abs(W _{ct} | 1),abs(W _{ch2}))) = | 0.000 kips | | | | |
| Shear wall deflection – Eqn. 4.3-1 | | δ_{sww} = 2 × $v_{\delta w}$ | imes h ³ / (3 $	imes$ E $	imes$. | $A_e \times b$) + $v_{\delta w}$ | imes h / (G _a) + h $	imes$ Ta | $_{\delta}$ / (k _a × b) = | | |
| | | 0.188 in | | | | | | |
| | | δ_{sww} / Δ_{w_allow} | = 0.919 | | | | | |
| | | | PASS - Shea | r wall defle | ction is less tha | n deflection | | |
| Seismic deflection | | | | | | | | |
| Design shear force | | $V_{\delta s} = E_q = 2$ | . 25 kips | | | | | |
| Deflection limit | | $\Delta_{s_{allow}} = 0.02$ | 20 × h = 2.46 i | า | | | | |
| Induced unit shear | | $v_{\delta s} = V_{\delta s} / b =$ | 56.25 lb/ft | | | | | |
| Anchor tension force | | $T_{\delta} = max(0 k)$ | ips, $v_{\delta s} 	imes h$ - (0.6 | - $0.2 \times S_{DS}$) | imes (D + S _{wt} $	imes$ h) $	imes$ | b / 2) = 0.00 | | |
| | | kips | | | | | | |
| Shear wall elastic deflection - Eqn. | 4.3-1 | $\delta_{swse} = 2 \times v_{\delta s}$ | $_{\rm s} 	imes {\rm h}^{\rm 3}$ / (3 $	imes$ E $	imes$ | $A_e \times b$) + $v_{\delta s}$ | imes h / (G _a) + h $	imes$ Ta | $_{\delta}$ / (k _a × b) = | | |
| | | 0.028 in | | | | | | |
| Deflection ampification factor | | $C_{d\delta} = 4$ | | | | | | |
| Seismic importance factor | | l _e = 1.25 | | | | | | |
| Amp. seis. deflection – ASCE7 Eqr | . 12.8-15 | | swse / I _e = 0.089 | in | | | | |
| | | δ_{sws} / Δ_{s_allow} = | | | | | | |
| | | | PASS - Shea | r wall defle | ction is less thai | n deflection | | |

<u>N-S WALLS</u> EAST, WEST ELEVATIONS (PERFORATED METHOD - TEDDS)

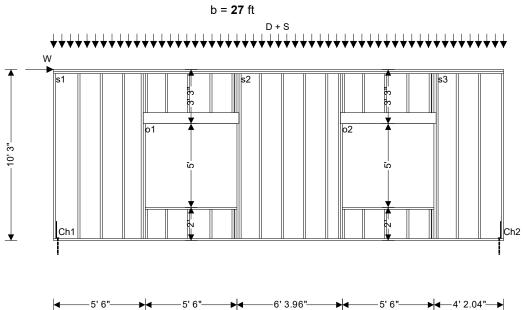




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| WOOD SHEAR WALL DESIGN | <u> </u> | ess design and | d the perforate | ed shear wall i | method | | r, LEVEI |
| In accordance with NDS2018 a Design summary | allowable stro | | - | | | Tedds calcula | T, LEVEI |
| In accordance with NDS2018 a Design summary | <u> </u> | ess design and Provided | Required | ed shear wall i Utilization | method Result | Tedds calcula | |
| In accordance with NDS2018 a Design summary Description | allowable stro | | - | | | Tedds calcula | |
| | allowable stro | Provided | Required | Utilization | Result | Tedds calcula | |
| In accordance with NDS2018 a Design summary Description Shear capacity | Unit | Provided 4790 | Required | Utilization 0.225 | Result | Tedds calcula | |

Structural I wood panel sheathing on one side

Panel height Panel length



h = **10.25** ft

| 4 | 5' 6" | | | | ── • 4 ′ 2.04 |
|---|-------|-----|--------|-----|-----------------------------|
| - | 5 0 | 5.0 | 0 0.00 | 5.0 | 4 4 2.04 |
| | | | | | |

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" |
| | |

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|---|--|---|-----------------------|-----------------|--------------------------|------------------|--|--|
| Anthem Structural Engineers | Section | | | | Sheet no./rev. 2 | | | |
| | Calc. by S | Date 8/1/2022 | Chk'd by | Date | App'd by | Date | | |
| Cross-sectional area of studs | | A _s = 8.25 in | 2 | | WES | ST, LEVE | | |
| 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 post | S | A _e = 8.25 in | 2 | | | | | |
| Hole diameter | | Dia = 1 in | | | | | | |
| Net cross-sectional area of end p | osts | A _{en} = 6.75 ir | 1 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 o | ^r less | | | | | |
| Vertical anchor stiffness | | k _a = 30000 | b/in | | | | | |
| From NDS Supplement Table 4 | A - Referenc | e design values | for visually gra | ded dimensio | on lumber (2" - 4 | " thick) | | |
| Species, grade and size classific | ation | Hem-Fir, no | .2 grade, 2" & v | wider | | | | |
| Specific gravity | | G = 0.43 | | | | | | |
| Tension parallel to grain | | Ft = 525 lb/i | n² | | | | | |
| Compression parallel to grain | | F_c = 1300 lb | /in² | | | | | |
| Modulus of elasticity | | E = 130000 | 0 lb/in ² | | | | | |
| Minimum modulus of elasticity | | E _{min} = 4700 | 00 lb/in ² | | | | | |
| Sheathing details | | | | | | | | |
| Sheathing material | | 7/16'' wood | l panel structu | Iral I oriented | strandboard sh | eathing | | |
| Fastener type | | 8d commo | n nails at 6"ce | enters | | | | |
| From SDPWS Table 4.3A Nomi | nal Unit Shea | ar Capacities for | Wood-Frame S | Shear Walls - | Wood-based Pa | nels | | |
| Nominal unit shear capacity for | or seismic de | sign v _s = min(56 | 60 plf · min[1 · | - (0.5 - G), 1] | , 1740 plf) = 520 |).8 lb/ft | | |
| Nominal unit shear capacity for | or wind desig | n $v_w = min(73)$ | 85 plf · min[1 | - (0.5 - G), 1] | l, 2435 plf) = 73 | 0.1 lb/ft | | |
| Apparent shear wall shear stif | fness | G _a = 16 kip | s/in | | | | | |
| Loading details | | | | | | | | |
| Dead load acting on top of panel | | D = 160 lb/f | | | | | | |
| Snow load acting on top of panel | | S = 800 lb/f | | | | | | |
| Self weight of panel | | S _{wt} = 10 lb/f | | | | | | |
| In plane wind load acting at head | of panel | W = 1800 lb | | | | | | |
| Wind load serviceability factor | | f _{Wserv} = 1.00 | | | | | | |
| From IBC 2021 cl.1605.1 Basic | load combin | | E 7, section 2. | 4 | | | | |
| Load combination no.1 | | D + 0.6W | | | | | | |
| Load combination no.2 | | D + 0.7E | A 4 - 1 | . · | | | | |
| Load combination no.3 | | | · 0.45W + 0.75(| . , | | | | |
| Load combination no.4 | | | 0.525E + 0.75 | S | | | | |
| Load combination no.5 | | 0.6D + 0.6V | | | | | | |
| <i></i> - | | 0.6D + 0.7E | | | | | | |
| Load combination no.6 | | | | | | | | |
| Adjustment factors | 2 | o (••• | | | | | | |
| Adjustment factors Load duration factor – Table 2.3.2 | | C _D = 1.60 | | | | | | |
| Load combination no.6 Adjustment factors Load duration factor – Table 2.3.3 Size factor for tension – Table 4A Size factor for compression – Table | L Contraction of the second seco | C _D = 1.60 C _{Ft} = 1.30 C _{Fc} = 1.10 | | | | | | |

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| | | | | | 3 | |
| | Calc. by S | Date 8/1/2022 | Chk'd by | Date | App'd by | Date |
| Wet service factor for tension – T | able 4A | C _{Mt} = 1.00 | | | WES | ST, LEVEL |
| Wet service factor for compression | on – Table 4A | C _{Mc} = 1.00 | | | | |
| Wet service factor for modulus of | elasticity – Ta | ble 4A | | | | |
| | | C _{ME} = 1.00 | | | | |
| Temperature factor for tension – | | C _{tt} = 1.00 | | | | |
| Temperature factor for compress | ion – Table 2.3 | 3.3 | | | | |
| | | C _{tc} = 1.00 | | | | |
| Temperature factor for modulus of | of elasticity – T | | | | | |
| | | C _{tE} = 1.00 | | | | |
| Incising factor – cl.4.3.8 | | C _i = 1.00 | | | | |
| Buckling stiffness factor – cl.4.4.2 | 2 | C⊤ = 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 · | E _{min} ' / (h / d) ² | = 772 psi | | |
| Reference compression design v | alue | $F_c^* = F_c \cdot C_c$ | $\cdot C_{Mc} \cdot C_{tc} \cdot C$ | CFc · Ci = 2288 | osi | |
| For sawn lumber | | c = 0.8 | | | | |
| Column stability factor - eqn.3 | .7-1 | $C_{P} = (1 + (F_{P}))$ | F _{cE} / F _c *)) / (2 · | c) – Ӥ[(1 + (| F _{cE} / F _c [*])) / (2 · | c)] ² - (F _{cE} / |
| , i | | F_{c}^{*}) / c) = 0 | <i>,,</i> , , | , (1) (| | |
| Shear wall aspect ratio Perforated wall length Shear wall aspect ratio | | h / b ₁ = 1.86 b ₂ = 6.33 ft h / b ₂ = 1.61 | | | | |
| Perforated wall length | | b ₃ = 4.17 ft | | | | |
| | | | | | | |
| - | | h / b ₃ = 2.45 | 8 | | | |
| Shear wall aspect ratio | or - cl 4 3 3 5 | h / b₃ = 2.45 | 8 | | | |
| Shear wall aspect ratio Shear capacity adjustment fact | | h / b ₃ = 2.45 | | h = 15 223 ft | | |
| Shear wall aspect ratio Shear capacity adjustment fact Sum of perforated shear wall leng | gths | h / b ₃ = 2.45 SL _i = b ₁ + b ₂ | + b ₃ · 2 · b _s / ł | | | |
| Shear wall aspect ratio Shear capacity adjustment fact Sum of perforated shear wall leng Total length of perforated shear w | gths | $h / b_3 = 2.45$ $SL_i = b_1 + b_2$ $L_{tot} = b_1 + w_c$ | + b ₃ · 2 · b _s / ł ₁ + b ₂ + w _{o2} + b | 93 = 27 ft | | |
| Shear wall aspect ratio Shear capacity adjustment fact Sum of perforated shear wall leng Total length of perforated shear w Total area of openings | gths | $h / b_3 = 2.45$ $SL_i = b_1 + b_2$ $L_{tot} = b_1 + w_c$ $A_o = w_{o1} \cdot h_c$ | + $b_3 \cdot 2 \cdot b_s / h_1$ + $b_2 + w_{o2} + b_1$ + $w_{o2} \cdot h_{o2} = {$ | ₉₃ = 27 ft 55 ft ² | | |
| Shear wall aspect ratio Shear capacity adjustment fact Sum of perforated shear wall leng Total length of perforated shear w Total area of openings Sheathing area ratio (eqn. 4.3-6) | gths <i>v</i> all | h / b ₃ = 2.45 SL _i = b ₁ + b ₂ L _{tot} = b ₁ + w _c A _o = w _{o1} · h _c r = 1 / (1 + A | + b ₃ · 2 · b _s / ł ₁ + b ₂ + w _{o2} + b | ₉₃ = 27 ft 55 ft ² | | |
| Shear wall aspect ratio Shear capacity adjustment fact Sum of perforated shear wall leng Total length of perforated shear w Total area of openings Sheathing area ratio (eqn. 4.3-6) Shear capacity adjustment factor | gths <i>v</i> all | $h / b_3 = 2.45$ $SL_i = b_1 + b_2$ $L_{tot} = b_1 + w_c$ $A_o = w_{o1} \cdot h_c$ | + $b_3 \cdot 2 \cdot b_s / h_1$ + $b_2 + w_{o2} + b_1$ + $w_{o2} \cdot h_{o2} = {$ | ₉₃ = 27 ft 55 ft ² | | |
| Shear wall aspect ratio Shear capacity adjustment fact Sum of perforated shear wall leng Total length of perforated shear w Total area of openings Sheathing area ratio (eqn. 4.3-6) Shear capacity adjustment factor Perforated shear wall capacity | gths vall (eqn. 4.3-5) | h / b ₃ = 2.45 $SL_i = b_1 + b_2$ $L_{tot} = b_1 + w_0$ $A_0 = w_{01} \cdot h_0$ r = 1 / (1 + A) $C_0 = 0.862$ | $\begin{array}{c} + b_{3} \cdot 2 \cdot b_{s} / h_{1} \\ + b_{2} + w_{o2} + b_{1} \\ + w_{o2} \cdot h_{o2} = b_{0} \\ - ((h \cdot SL_{i})) = 0 \end{array}$ | n3 = 27 ft 55 ft ² .739 | | |
| Shear wall aspect ratio Shear capacity adjustment fact Sum of perforated shear wall leng Total length of perforated shear w Total area of openings Sheathing area ratio (eqn. 4.3-6) Shear capacity adjustment factor Perforated shear wall capacity Maximum shear force under w | yths vall (eqn. 4.3-5) vind loading | h / b ₃ = 2.45 SL _i = b ₁ + b ₂ L _{tot} = b ₁ + w ₀ A ₀ = w ₀₁ · h ₀ r = 1 / (1 + A C ₀ = 0.862 $V_{w_max} = 0.6$ | $ + b_{3} \cdot 2 \cdot b_{s} / h_{1} + b_{2} + w_{o2} + b_{1} + w_{o2} \cdot h_{o2} = b_{1} + w_{o2} \cdot h_{o2} = b_{0} / (h \cdot SL_{i})) = 0 $ | n₃ = 27 ft 55 ft² .739 ips | | |
| Shear wall aspect ratio Shear capacity adjustment fact Sum of perforated shear wall leng Total length of perforated shear w Total area of openings Sheathing area ratio (eqn. 4.3-6) Shear capacity adjustment factor Perforated shear wall capacity | yths vall (eqn. 4.3-5) vind loading | h / b ₃ = 2.45 SL _i = b ₁ + b ₂ L _{tot} = b ₁ + w _o A _o = w _{o1} · h _o r = 1 / (1 + A C _o = 0.862 $V_{w_max} = 0.6$ | $\begin{array}{c} + b_{3} \cdot 2 \cdot b_{s} / h_{1} \\ + b_{2} + w_{o2} + b_{1} \\ + w_{o2} \cdot h_{o2} = b_{0} \\ - ((h \cdot SL_{i})) = 0 \end{array}$ | n₃ = 27 ft 55 ft² .739 ips | | |
| Shear wall aspect ratio Shear capacity adjustment fact Sum of perforated shear wall leng Total length of perforated shear w Total area of openings Sheathing area ratio (eqn. 4.3-6) Shear capacity adjustment factor Perforated shear wall capacity Maximum shear force under w | yths vall (eqn. 4.3-5) vind loading | h / b ₃ = 2.45 SL _i = b ₁ + b ₂ L _{tot} = b ₁ + w _o A _o = w _{o1} · h _o r = 1 / (1 + A C _o = 0.862 V _{w_max} = 0.6 V _w = v _w · C V _{w_max} / V _w = | + $b_3 \cdot 2 \cdot b_s / h_1$ + $b_2 + w_{o2} + b_{o2}$ + $w_{o2} \cdot h_{o2} = \frac{1}{2}$ (h · SLi)) = 0 · W = 1.08 ki · SLi / 2 = 4. 0.225 | n3 = 27 ft 55 ft ² .739 ips 79 kips | | |
| Shear wall aspect ratio Shear capacity adjustment fact Sum of perforated shear wall leng Total length of perforated shear w Total area of openings Sheathing area ratio (eqn. 4.3-6) Shear capacity adjustment factor Perforated shear wall capacity Maximum shear force under w | yths vall (eqn. 4.3-5) vind loading | h / b ₃ = 2.45 SL _i = b ₁ + b ₂ L _{tot} = b ₁ + w _o A _o = w _{o1} · h _o r = 1 / (1 + A C _o = 0.862 V _{w_max} = 0.6 V _w = v _w · C V _{w_max} / V _w = | + $b_3 \cdot 2 \cdot b_s / h_1$ + $b_2 + w_{o2} + b_{o2}$ + $w_{o2} \cdot h_{o2} = \frac{1}{2}$ (h · SLi)) = 0 · W = 1.08 ki · SLi / 2 = 4. 0.225 | n3 = 27 ft 55 ft ² .739 ips 79 kips | exceeds maxim | num shear fo |
| Shear wall aspect ratio Shear capacity adjustment fact Sum of perforated shear wall leng Total length of perforated shear w Total area of openings Sheathing area ratio (eqn. 4.3-6) Shear capacity adjustment factor Perforated shear wall capacity Maximum shear force under w | oths vall (eqn. 4.3-5) vind loading g | h / b ₃ = 2.45 SL _i = b ₁ + b ₂ L _{tot} = b ₁ + w _o A _o = w _{o1} · h _o r = 1 / (1 + A C _o = 0.862 V _{w_max} = 0.6 V _w = v _w · C V _{w_max} / V _w = | + $b_3 \cdot 2 \cdot b_s / h_1$ + $b_2 + w_{o2} + b_{o2}$ + $w_{o2} \cdot h_{o2} = \frac{1}{2}$ (h · SLi)) = 0 · W = 1.08 ki · SLi / 2 = 4. 0.225 | n3 = 27 ft 55 ft ² .739 ips 79 kips | exceeds maxim | num shear foi |
| Shear wall aspect ratio Shear capacity adjustment fact Sum of perforated shear wall leng Total length of perforated shear w Total area of openings Sheathing area ratio (eqn. 4.3-6) Shear capacity adjustment factor Perforated shear wall capacity Maximum shear force under w Shear capacity for wind loadin Chord capacity for chords 1 an | oths vall (eqn. 4.3-5) vind loading g d 2 | h / b ₃ = 2.45 SL _i = b ₁ + b ₂ L _{tot} = b ₁ + w _o A _o = w _{o1} · h _o r = 1 / (1 + A C _o = 0.862 V _{w_max} = 0.6 V _w = v _w · C V _{w_max} / V _w = | + $b_3 \cdot 2 \cdot b_s / h_1$ + $b_2 + w_{o2} + b_{o2}$ + $w_{o2} \cdot h_{o2} = 0$ ($h \cdot SL_i$)) = 0 W = 1.08 ki · SL_i / 2 = 4. 0.225 hear capacity | n3 = 27 ft 55 ft ² .739 ips 79 kips | exceeds maxim | num shear foi |
| Shear wall aspect ratio Shear capacity adjustment fact Sum of perforated shear wall leng Total length of perforated shear w Total area of openings Sheathing area ratio (eqn. 4.3-6) Shear capacity adjustment factor Perforated shear wall capacity Maximum shear force under w Shear capacity for wind loadin Chord capacity for chords 1 an Load combination 5 | yths vall (eqn. 4.3-5) rind loading g d 2 | h / b ₃ = 2.45 $SL_i = b_1 + b_2$ $L_{tot} = b_1 + w_0$ $A_0 = w_{01} \cdot h_0$ r = 1 / (1 + A) $C_0 = 0.862$ $V_{w_max} = 0.6$ $V_w = v_w \cdot C$ $V_{w_max} / V_w = PASS - S$ $V = 0.6 \cdot W$ | + $b_3 \cdot 2 \cdot b_s / h_1$ + $b_2 + w_{o2} + b_{o2}$ + $b_{o2} \cdot b_{o2} = 4$ + $(h \cdot SL_i) = 0$ + $W = 1.08 k_i$ + $O \cdot SL_i / 2 = 4$. - 0.225 - $hear capacity$ = $1.08 kips$ | n3 = 27 ft 55 ft ² .739 ips 79 kips | | num shear foi |
| Shear wall aspect ratio Shear capacity adjustment fact Sum of perforated shear wall leng Total length of perforated shear w Total area of openings Sheathing area ratio (eqn. 4.3-6) Shear capacity adjustment factor Perforated shear wall capacity Maximum shear force under w Shear capacity for wind loadin Chord capacity for chords 1 and Load combination 5 Shear force for maximum tension | yths vall (eqn. 4.3-5) rind loading g d 2 | h / b ₃ = 2.45 $SL_i = b_1 + b_2$ $L_{tot} = b_1 + w_0$ $A_o = w_{o1} \cdot h_0$ r = 1 / (1 + A) $C_o = 0.862$ $V_{w_max} = 0.6$ $V_w = v_w \cdot C$ $V_{w_max} / V_w = PASS - S$ $V = 0.6 \cdot W$ $P = (0.6 \cdot (1))$ | + $b_3 \cdot 2 \cdot b_s / h_1$ + $b_2 + w_{o2} + b_{o2}$ + $w_{o2} \cdot h_{o2} = 4$ ($h \cdot SL_i$)) = 0 - $(H \cdot SL_i)$ = 0 - $SL_i / 2 = 4$. 0.225 hear capacity = 1.08 kips D + S _{wt} · h)) · | a = 27 ft 55 ft² .739 ips 79 kips for wind load of b / 2 = 2.126 | | |
| Shear wall aspect ratio Shear capacity adjustment fact Sum of perforated shear wall leng Total length of perforated shear w Total area of openings Sheathing area ratio (eqn. 4.3-6) Shear capacity adjustment factor Perforated shear wall capacity Maximum shear force under w Shear capacity for wind loadin Chord capacity for chords 1 an Load combination 5 Shear force for maximum tension Axial force for maximum tension | yths vall (eqn. 4.3-5) rind loading g d 2 | h / b ₃ = 2.45 $SL_i = b_1 + b_2$ $L_{tot} = b_1 + w_0$ $A_o = w_{o1} \cdot h_0$ r = 1 / (1 + A) $C_o = 0.862$ $V_{w_max} = 0.6$ $V_w = v_w \cdot C$ $V_{w_max} / V_w = PASS - S$ $V = 0.6 \cdot W$ $P = (0.6 \cdot (1))$ | + $b_3 \cdot 2 \cdot b_s / h_1$ + $b_2 + w_{o2} + b_{o2}$ + $w_{o2} \cdot h_{o2} = 4$ o /($h \cdot SL_i$)) = 0. 5 · $W = 1.08 k_i$ o · $SL_i / 2 = 4$. 0.225 hear capacity = 1.08 kips D + S _{wt} · h)) · Co · SL_i)) - P = | a = 27 ft 55 ft² .739 ips 79 kips for wind load of b / 2 = 2.126 | kips | NO |

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$f_t / F_t' = -0.174$

WEST, LEVEL 4

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 1 Shear force for maximum compression

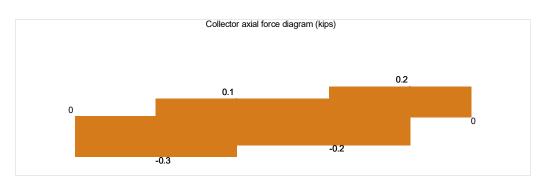
Axial force for maximum compression

Maximum compressive force in chord

Maximum applied compressive stress Design compressive stress $V = 0.6 \cdot W = 1.08 \text{ kips}$ $P = ((D + S_{wt} \cdot h)) \cdot s / 2 = 0.175 \text{ kips}$ $C = V \cdot h / ((C_o \cdot SL_i)) + P = 1.019 \text{ kips}$ $f_c = C / A_e = 123 \text{ lb/in}^2$ $F_c' = F_c \cdot C_D \cdot C_{Mc} \cdot C_{tc} \cdot C_{Fc} \cdot C_i \cdot C_P = 709 \text{ lb/in}^2$ $f_c / F_c' = 0.174$

PASS - Design compressive stress exceeds maximum applied compressive stress

Collector capacity



Maximum shear force on wall Uniform shear applied to wall Shear resisted by wall segments Maximum force in collector Maximum applied tensile stress Design tensile stress

Maximum applied compressive stress Column stability factor Design compressive stress

 $\begin{array}{l} V_{max} = V_{w_max} = \textbf{1.08 kips} \\ v_a = V_{max} / \left((C_o \cdot SL_i) \right) = \textbf{82.3 plf} \\ v_b = v_a \cdot b / (b_1 + b_2 + b_3) = \textbf{138.9 plf} \\ P_{coll} = \textbf{0.311 kips} \\ f_t = P_{coll} / (2 \cdot A_s) = \textbf{19 lb/in}^2 \\ F_t' = F_t \cdot C_D \cdot C_{Mt} \cdot C_{tt} \cdot C_{Ft} \cdot C_i = \textbf{1092 lb/in}^2 \\ f_t / F_t' = \textbf{0.017} \\ \textbf{PASS - Design tensile stress exceeds maximum applied tensile stress} \\ f_c = P_{coll} / (2 \cdot A_s) = \textbf{19 lb/in}^2 \\ C_P = \textbf{1.00} \\ F_c' = F_c \cdot C_D \cdot C_{Mc} \cdot C_{tc} \cdot C_{Fc} \cdot C_i \cdot C_P = \textbf{2288 lb/in}^2 \\ f_c / F_c' = \textbf{0.008} \end{array}$

Wind load deflection

Design shear force Deflection limit Induced unit shear Anchor tension force
$$\begin{split} V_{dw} &= f_{Wserv} \cdot \ W = \textbf{1.8 kips} \\ D_{w_allow} &= h \ / \ 500 = \textbf{0.246 in} \\ v_{dw_max} &= V_{dw} \ / \ (C_o \cdot \ SL_i) = \textbf{137.16 lb/ft} \\ T_d &= max(0 \ kips, v_{dw_max} \cdot \ h - 0.6 \cdot \ (D + S_{wt} \cdot \ h) \cdot \ b \ / \ 2) = \textbf{0.000 kips} \end{split}$$

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| Shear wall deflection – Eqn. 4.3-1 | | d _{sww} = 2 · v _{dw} | ,_ _{_max} · h ³ / (3 · | E · A _e · SL _i) + v _{dw_} | ₩ ES1 _{max} · h / (G _a) + | , LEVEL 4 h · Td / (ka · |

SLi) = **0.095** in

 d_{sww} / $D_{w_{allow}}$ = 0.387

PASS - Shear wall deflection is less than deflection limit

| Tekla. Tedds | Project | Project | | | | | | |
|---|--------------------|------------------|---------------------|---------------|-----------------|-------------|---------|--|
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| WOOD SHEAR WALL DESIGN | | ess design and | d the perforate | ed shear wall | method | | T, LEVE | |
| 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 sheathir | ng on one side | h = 10.2 | - 4 | | | | | |
| Panel height | | | | | | | | |
| Panel length | | b = 27 ft | D + S | | | | | |
| | ↓↓↓↓↓↓ | ↓ ↓↓↓↓↓↓↓ | | **** | ↓ ↓↓↓↓↓↓ | ↓ ↓ ↓ | | |
| | | | | | s2 | | | |

| V | Ch1 | | | | | | | 5 | | | | Ch2 |
|---|----------|--|---------|---|------|-------------|---|---------|-------------------|------|---|-----|
| | | | -17' 6' | n | | | • | -5' 6"- | - > | —4'- | ► | |

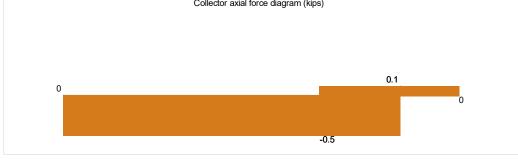
Panel opening details

| 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" |
| | |

| | Tekla [®] | | Project | | | | Job Ref. | | |
|--|--|--|---|--|--|--|--|--|-----|
| Anth | hem Structural | Engineers | Section | | | | Sheet no 2 | o./rev. | |
| | | | Calc. by S | Date 8/1/2022 | Chk'd by | Date | App'd by | Date | |
| Cros | ss-sectional are | ea of end posts | 3 | A _e = 16.5 ir | 1 ² | | V | VEST, LEV | /EL |
| Hole | e diameter | | | Dia = 1 in | | | | | |
| Net | cross-sectiona | l area of end p | osts | A _{en} = 13.5 | in² | | | | |
| Nom | ninal collector s | size | | 2 x 2" x 6" | | | | | |
| Dres | ssed collector s | size | | 2 x 1.5" x 5 | 5.5" | | | | |
| Serv | vice condition | | | Dry | | | | | |
| Tem | nperature | | | 100 degF o | or less | | | | |
| Verti | tical anchor stif | fness | | k _a = 30000 | lb/in | | | | |
| Fror | m NDS Supple | ement Table 4 | A - Reference | design values | for visually g | raded dimens | ion lumber (2 | " - 4" thick) | |
| | cies, grade and | | | - | o.1 & btr grade | | Ŷ | , | |
| Spe | cific gravity | | | G = 0.43 | - | | | | |
| - | sion parallel to | grain | | Ft = 725 lb/ | /in² | | | | |
| Corr | npression paral | llel to grain | | F _c = 1350 l | b/in² | | | | |
| Mod | lulus of elastici | ty | | E = 15000 | 00 lb/in ² | | | | |
| Mini | imum modulus | of elasticity | | E _{min} = 5500 |)00 lb/in ² | | | | |
| She | athing details | | | | | | | | |
| | eathing materi | | | 7/16" woo | d panel struc | tural I oriente | d strandboard | d sheathing | |
| 0110 | • | | | | | | a offantaboard | a onoaaning | |
| Fast | stener type | | | 8d commo | on nails at 6"o | centers | | | |
| | tener type | | | | on nails at 6" | | | Damala | |
| Fror | m SDPWS Tab | | | Capacities for | r Wood-Frame | Shear Walls | | | |
| Fror Nom | m SDPWS Tab minal unit shea | ar capacity fo | r seismic des | Capacities for ign v _s = min(5 | Wood-Frame 60 plf × min[′ | Shear Walls 1 - (0.5 - G), 1 |], 1740 plf) = | 520.8 lb/ft | |
| Fror Nom | m SDPWS Tab | ar capacity fo | r seismic des | Capacities for ign v _s = min(5 | Wood-Frame 60 plf × min[′ | Shear Walls |], 1740 plf) = | 520.8 lb/ft | |
| Fror Non Non | m SDPWS Tab minal unit shea | ar capacity fo ar capacity fo | r seismic des r wind design | Capacities for ign v _s = min(5 | r Wood-Frame 60 plf × min[785 plf × min[| Shear Walls 1 - (0.5 - G), 1 |], 1740 plf) = | 520.8 lb/ft | |
| Fror Non Non App | m SDPWS Tab minal unit she minal unit she parent shear w | ar capacity fo ar capacity fo | r seismic des r wind design | Capacities for ign v _s = min(5 v _w = min(7 | r Wood-Frame 60 plf × min[785 plf × min[| Shear Walls 1 - (0.5 - G), 1 |], 1740 plf) = | 520.8 lb/ft | |
| Fror Non Non App Load | m SDPWS Tab minal unit shea minal unit shea parent shear w ding details | ar capacity fo ar capacity fo vall shear stiff | r seismic des r wind design | Capacities for ign v _s = min(5 v _w = min(7 | r Wood-Frame 60 plf × min[785 plf × min[ps/in | Shear Walls 1 - (0.5 - G), 1 |], 1740 plf) = | 520.8 lb/ft | |
| Fror Non Non App Load | m SDPWS Tab minal unit shea minal unit shea minal unit shear w ding details id load acting o | ar capacity fo ar capacity fo vall shear stiff on top of panel | r seismic des r wind design fness | Capacities for ign v_s = min(5 v_w = min(7 G_a = 16 ki | r Wood-Frame 60 plf × min[[*] 785 plf × min[ps/in ft | Shear Walls 1 - (0.5 - G), 1 |], 1740 plf) = | 520.8 lb/ft | |
| Fror Non App Load Snov | m SDPWS Tab minal unit shea minal unit shea parent shear w ding details | ar capacity fo ar capacity fo vall shear stiff on top of panel on top of panel | r seismic des r wind design fness | Capacities for ign $v_s = min(5)$ $v_w = min(7)$ $G_a = 16$ ki D = 460 lb/ | r Wood-Frame 60 plf × min[785 plf × min[785 nlf ps/in ft | Shear Walls 1 - (0.5 - G), 1 |], 1740 plf) = | 520.8 lb/ft | |
| Fror Non App Load Snov Self | m SDPWS Tab minal unit shea minal unit shea parent shear w ding details id load acting o w load acting o | ar capacity fo ar capacity fo vall shear stiff on top of panel on top of panel | r seismic des r wind design fness | Capacities for ign $v_s = min(5 v_w = min(7 G_a = 16 ki))$ D = 460 lb/ S = 800 lb/ | r Wood-Frame 60 plf × min[785 plf × min[785 nlf ft ft ft | Shear Walls 1 - (0.5 - G), 1 |], 1740 plf) = | 520.8 lb/ft | |
| Fron Non App Load Snov Self | m SDPWS Tab minal unit shea minal unit shea parent shear w ding details id load acting o w load acting o weight of pane | ar capacity fo ar capacity fo vall shear stiff on top of panel on top of panel el acting at head | r seismic des r wind design fness | Capacities for ign $v_s = min(5)$ $v_w = min(7)$ $G_a = 16$ ki D = 460 lb/ S = 800 lb/ $S_{wt} = 10$ lb/ | r Wood-Frame 60 plf × min[785 plf × min[785 n ft ft ft ft | Shear Walls 1 - (0.5 - G), 1 |], 1740 plf) = | 520.8 lb/ft | |
| Fron Non App Load Dead Snov Self In pl | m SDPWS Tab minal unit shea parent shear w ding details id load acting o w load acting o weight of pane lane wind load | ar capacity fo ar capacity fo vall shear stiff on top of panel on top of panel acting at head ability factor | r seismic des r wind design fness of panel | Capacities for ign $v_s = min(5 v_w = min(7 G_a = 16 ki))$ D = 460 lb/ S = 800 lb/ S _{wt} = 10 lb/ W = 3800 l | r Wood-Frame 60 plf × min[785 plf × min[785 n ft ft ft ft | Shear Walls 1 - (0.5 - G), 1 |], 1740 plf) = | 520.8 lb/ft | |
| Fror Non App Load Dead Snov Self In pl Wind Cho | m SDPWS Tab minal unit shea parent shear w ding details id load acting o w load acting o weight of pane lane wind load d load servicea | ar capacity fo ar capacity fo vall shear stiff on top of panel on top of panel acting at head ability factor | r seismic des r wind design fness of panel | Capacities for ign $v_s = min(5 v_w = min(7 G_a = 16 ki))$ D = 460 lb/ S = 800 lb/ S _{wt} = 10 lb/ W = 3800 l | r Wood-Frame 60 plf × min[785 plf × min[785 n ft ft ft ft | Shear Walls 1 - (0.5 - G), 1 |], 1740 plf) = | 520.8 lb/ft | |
| Fror Non App Load Snov Self In pl Wind Cho | m SDPWS Tab minal unit shea parent shear w ding details id load acting o w load acting o w load acting o weight of pane lane wind load d load servicea ord forces from | ar capacity fo ar capacity fo vall shear stiff on top of panel on top of panel el acting at head ability factor n shear walls | r seismic des r wind design fness of panel above | Capacities for ign $v_s = min(5 v_w = min(7 G_a = 16 ki))$ D = 460 lb/ S = 800 lb/ S _{wt} = 10 lb/ W = 3800 l f _{Wserv} = 1.00 | r Wood-Frame 60 plf × min[785 plf × min[785 plf × min[ft ft ft ft bs 0 | e Shear Walls 1 - (0.5 - G), 1 1 - (0.5 - G), 1 |], 1740 plf) = I], 2435 plf) = | 520.8 lb/ft 730.1 lb/ft | |
| Fror Non App Load Dead Snov Self In pl Wind Cho | m SDPWS Tab minal unit shea minal unit shea parent shear w ding details id load acting o w load acting o w load acting o w load acting o w load acting o d load servicea ord forces from Wch[] (Ibs) | ar capacity fo ar capacity fo vall shear stiff on top of panel on top of panel el acting at head ability factor n shear walls E q_ch[] (lbs) | r seismic des r wind design fness of panel above Dc_ch[i] (Ibs) | Capacities for ign v _s = min(5 v _w = min(7 G _a = 16 ki D = 460 lb/ S = 800 lb/ S _{wt} = 10 lb/ W = 3800 l f _{Wserv} = 1.00 | r Wood-Frame 60 plf \times min[' 785 plf \times min[ps/in ft ft ft ft Lft ² bs 0 Lf_ch[i] (lbs) | e Shear Walls 1 - (0.5 - G), 1 1 - (0.5 - G), 7 Lr_ch[ī] (Ibs) |], 1740 plf) = I], 2435 plf) = Schii] (Ibs) | 520.8 lb/ft 730.1 lb/ft Rch[i] (lbs) | |
| Fror Non App Load Dead Snov Self In pl Wind Cho nord h1 h2 | m SDPWS Tab minal unit shear parent shear w ding details ad load acting of weight of pane lane wind load d load servicear ord forces from Wch[i] (lbs) -697; 697; | ar capacity fo ar capacity fo vall shear stiff on top of panel on top of panel acting at head ability factor n shear walls $E_{q_ch[1]}(lbs)$ 0; 0; | r seismic des r wind design fness of panel above Dc_ch[i] (Ibs) 0; 0; | Capacities for ign $v_s = min(5 v_w = min(7 G_a = 16 ki))$ $D = 460 lb/S = 800 lb/S = 800 lb/S v_w = 10 lb/W = 3800 lb/S v_w = 1.000$ $D_{T_ch[1]}(lbs)$ 0; 0; 0; | r Wood-Frame 60 plf × min[785 plf × min[ps/in ft ft ft ft ft^2 bs 0 L _{f_ch[1]} (lbs) 0; 0; | e Shear Walls 1 - (0.5 - G), 1 1 - (0.5 - G), 1 1 - (0.5 - G), 1 0; 0; |], 1740 plf) = I], 2435 plf) = S ch[ī] (Ibs) 0; | 520.8 lb/ft 730.1 lb/ft R ch[i] (lbs) 0; | |
| From Nom App Load Dead Snow Self In pl: Wind Cho nord in1 in2 | m SDPWS Tab minal unit shear parent shear w ding details ad load acting of weight of pane lane wind load d load servicear ord forces from Wch[i] (lbs) -697; 697; | ar capacity for ar capacity for vall shear stiff on top of panel on top of panel acting at head ability factor n shear walls $E_{q_ch[i]}$ (Ibs) 0; 0; 1605.1 Basic | r seismic des r wind design fness of panel above Dc_ch[i] (Ibs) 0; 0; | Capacities for ign v _s = min(5 v _w = min(7 G _a = 16 ki D = 460 lb/ S = 800 lb/ S = 800 lb/ W = 3800 l f _{Wserv} = 1.00 DT_ch[i] (lbs) 0; 0; tions from ASC | r Wood-Frame 60 plf × min[785 plf × min[ps/in ft ft ft ft ft^2 bs 0 L _{f_ch[1]} (lbs) 0; 0; | e Shear Walls 1 - (0.5 - G), 1 1 - (0.5 - G), 1 1 - (0.5 - G), 1 0; 0; |], 1740 plf) = I], 2435 plf) = S ch[ī] (Ibs) 0; | 520.8 lb/ft 730.1 lb/ft R ch[i] (lbs) 0; | |
| From Nom App Load Snow Self In pla Wind Cho nord in1 in2 From Load | m SDPWS Tab minal unit shear minal unit shear parent shear w ding details id load acting o w load acting o m load services for forces from -697; 697; m IBC 2021 cl. | ar capacity fo ar capacity fo vall shear stiff on top of panel on top of panel acting at head ability factor n shear walls $E_{q_ch[]}$ (lbs) 0; 0; .1605.1 Basic no.1 | r seismic des r wind design fness of panel above Dc_ch[i] (Ibs) 0; 0; | Capacities for ign $v_s = min(5 v_w = min(7 G_a = 16 ki))$ $D = 460 lb/S = 800 lb/S = 800 lb/S v_w = 10 lb/W = 3800 lb/S v_w = 1.000$ $D_{T_ch[1]}(lbs)$ 0; 0; 0; | r Wood-Frame 60 plf × min[785 plf × min[ps/in ft ft ft ft ft^2 bs 0 L _{f_ch[1]} (lbs) 0; 0; | e Shear Walls 1 - (0.5 - G), 1 1 - (0.5 - G), 1 1 - (0.5 - G), 1 0; 0; |], 1740 plf) = I], 2435 plf) = S ch[ī] (Ibs) 0; | 520.8 lb/ft 730.1 lb/ft R ch[i] (lbs) 0; | |
| Fror Non App Load Dead Snov Self In pla Wind Cho nord in1 in2 Fror Load | m SDPWS Tab minal unit shear minal unit shear parent shear w ding details id load acting o w load acting o m load acting o for ces from a forces from -697; m IBC 2021 cl. | ar capacity fo ar capacity fo vall shear stiff on top of panel on top of panel acting at head ability factor n shear walls $E_{q_ch[1]}$ (Ibs) 0; 0; 1605.1 Basic no.1 no.2 | r seismic des r wind design fness of panel above Dc_ch[i] (Ibs) 0; 0; | Capacities for ign $v_s = min(5)$ $v_w = min(7)$ $G_a = 16 ki$ D = 460 lb/ S = 800 lb/ S = 800 lb/ $S_wt = 10 lb/$ W = 3800 l $f_{Wserv} = 1.00$ $D_{T_ch[1]}(lbs)$ 0; tions from ASC D + 0.6W D + 0.7E | r Wood-Frame 60 plf × min[785 plf × min[785 plf × min[ps/in ft ft ft ft ft ft ft ft ft ft ft ft ft | e Shear Walls 1 - (0.5 - G), 1 1 - (0.5 - G), 1 1 - (0.5 - G), 1 0; 0; |], 1740 plf) = I], 2435 plf) = S ch[ī] (Ibs) 0; | 520.8 lb/ft 730.1 lb/ft R ch[i] (lbs) 0; | |
| From Nom App Load Dead Snow Self In pl: Wind Cho Ord in1 in2 From Load Load | m SDPWS Tab minal unit shear parent shear w ding details ad load acting of weight of pane lane wind load d load servicear ord forces from Wch[i] (lbs) -697; 697; m IBC 2021 cl. d combination in d combination in | ar capacity fo ar capacity fo vall shear stiff on top of panel on top of panel acting at head ability factor n shear walls E q_ch[1] (Ibs) 0; 0; 1605.1 Basic no.1 no.2 no.3 | r seismic des r wind design fness of panel above Dc_ch[i] (Ibs) 0; 0; | Capacities for ign $v_s = min(5 v_w = min(7 G_a = 16 ki))$ D = 460 lb/S = 800 lb/S = 800 lb/S wt = 10 lb/W = 3800 lb/W = 3800 lb/W = 3800 lb/W = 3800 lb/W = 0.000 lb/W = 0.0000 lb/W = 0.000 lb/W = 0.000 lb/W = 0.0000 lb/W | r Wood-Frame 60 plf × min[785 plf × min[785 plf × min[ps/in ft ft ft ft ft ft ft ft ft ft ft ft ft | Shear Walls 1 - (0.5 - G), 1 1 - (0.5 - G), 1 1 - (0.5 - G), 1 L _{r_ch[]} (Ibs) 0; 0; 2.4 5(L _r or S or R) |], 1740 plf) = I], 2435 plf) = S ch[ī] (Ibs) 0; | 520.8 lb/ft 730.1 lb/ft R ch[i] (lbs) 0; | |
| From Nom App Load Snov Self In pla Wind Cho Ord h1 h2 From Load Load Load | m SDPWS Tab minal unit shear parent shear w ding details ad load acting o w load acting o m load services for forces from -697; 697; m IBC 2021 cl. d combination of d combination of | ar capacity fo ar capacity fo vall shear stiff on top of panel on top of panel acting at head ability factor n shear walls $E_{q_ch[1]}(lbs)$ 0; 0; 1605.1 Basic no.1 no.2 no.3 no.4 | r seismic des r wind design fness of panel above Dc_ch[i] (Ibs) 0; 0; | Capacities for ign $v_s = min(5 v_w = min(7 G_a = 16 ki))$ D = 460 lb/S = 800 lb/S = 10 lb/S wt = 10 lb/S | r Wood-Frame 60 plf × min[* 785 plf × min[785 plf × min[ps/in ft ft ft ft ft ft ft Characteristics CE 7, section + 0.45W + 0.7 + 0.525E + 0.7 | Shear Walls 1 - (0.5 - G), 1 1 - (0.5 - G), 1 1 - (0.5 - G), 1 L _{r_ch[]} (Ibs) 0; 0; 2.4 5(L _r or S or R) |], 1740 plf) = I], 2435 plf) = S ch[ī] (Ibs) 0; | 520.8 lb/ft 730.1 lb/ft R ch[i] (lbs) 0; | |
| Fror Non App Load Dead Snov Self In pl Wind Cho Cho nord in1 in2 Fror Load Load Load | m SDPWS Tab minal unit shear minal unit shear parent shear w ding details id load acting o w load acting o m load servicea ord forces from d load servicea ord forces from here for ces | ar capacity fo ar capacity fo vall shear stiff on top of panel on top of panel acting at head ability factor n shear walls $E_{q_ch[1]}(lbs)$ 0; 0; 1605.1 Basic no.1 no.2 no.3 no.4 no.5 | r seismic des r wind design fness of panel above Dc_ch[i] (Ibs) 0; 0; | Capacities for ign $v_s = min(5 v_w = min(7 G_a = 16 ki))$ D = 460 lb/S = 800 lb/S = 800 lb/S = 10 lb/ | r Wood-Frame 60 plf × min[785 plf × min[7 | Shear Walls 1 - (0.5 - G), 1 1 - (0.5 - G), 1 1 - (0.5 - G), 1 L _{r_ch[]} (Ibs) 0; 0; 2.4 5(L _r or S or R) |], 1740 plf) = I], 2435 plf) = S ch[ī] (Ibs) 0; | 520.8 lb/ft 730.1 lb/ft R ch[i] (lbs) 0; | |
| From Nom App Load Snov Self In pla Wind Cho Ord h1 h2 From Load Load Load Load | m SDPWS Tab minal unit shear minal unit shear minal unit shear oarent shear w ding details id load acting o w load acting o in load servicear ord forces from Wch(1) (lbs) -697; m IBC 2021 cl. d combination of d combina | ar capacity fo ar capacity fo vall shear stiff on top of panel on top of panel acting at head ability factor n shear walls $E_{q_ch[1]}(lbs)$ 0; 0; 1605.1 Basic no.1 no.2 no.3 no.4 no.5 no.6 | r seismic des r wind design fness of panel above Dc_ch[i] (Ibs) 0; 0; | Capacities for ign $v_s = min(5)$ $v_w = min(7)$ $G_a = 16 ki$ D = 460 lb/ S = 800 lb/ S = 800 lb/ W = 3800 lb/ $O_{T_ch[1]}(lbs)$ 0; 0; tions from ASC D + 0.6W $D + 0.75L_f$ $D + 0.75L_f$ 0.6D + 0.61 | r Wood-Frame 60 plf × min[785 plf × min[7 | Shear Walls 1 - (0.5 - G), 1 1 - (0.5 - G), 1 1 - (0.5 - G), 1 L _{r_ch[]} (Ibs) 0; 0; 2.4 5(L _r or S or R) |], 1740 plf) = I], 2435 plf) = S ch[ī] (Ibs) 0; | 520.8 lb/ft 730.1 lb/ft R ch[i] (lbs) 0; | |
| Fror Non App Load Dead Snov Self In pl Wind Cho Cho Cho In ord Enord Load Load Load Load Load Load | m SDPWS Tab ninal unit shea parent shear w ding details ad load acting o w load acting o m load servicea ord forces from Wch[i] (lbs) -697; m IBC 2021 cl. d combination of d combination of d combination of d combination of d combination of d combination of d | ar capacity for ar capacity for vall shear stiff on top of panel on top of panel acting at head ability factor n shear walls $E_{q_ch[1]}$ (Ibs) 0; 0; 1605.1 Basic no.1 no.2 no.3 no.4 no.5 no.6 rs | of panel above Dc_ch[i] (Ibs) 0; Ioad combinat | Capacities for ign $v_s = min(5)$ $v_w = min(7)$ $G_a = 16 ki$ D = 460 lb/ S = 800 lb/ S = 800 lb/ $S_wt = 10 lb/$ W = 3800 lb/ W = 3800 lb/ W = 3800 lb/ W = 3800 lb/ W = 3800 lb/ $G_{r} = 10 lb/$ W = 3800 lb/ $G_{r} = 10 lb/$ $G_{r} = 1.00$ $G_{r} = 1.00$ $G_{r} = 0.75L_{f}$ $G_{r} = 0.75L_{f}$ $G_{r} = 0.75L_{f}$ $G_{r} = 0.71$ $G_{r} = 0.71$ | r Wood-Frame 60 plf × min[785 plf × min[7 | Shear Walls 1 - (0.5 - G), 1 1 - (0.5 - G), 1 1 - (0.5 - G), 1 L _{r_ch[]} (Ibs) 0; 0; 2.4 5(L _r or S or R) |], 1740 plf) = I], 2435 plf) = S ch[ī] (Ibs) 0; | 520.8 lb/ft 730.1 lb/ft R ch[i] (lbs) 0; | |
| Fror Non App Load Dead Snov Self In pl: Wind Cho Cho hord Cho Cho Cho Cho Cho Cho Cho Cho Cho Cho | m SDPWS Tab minal unit shear minal unit shear minal unit shear oarent shear w ding details id load acting o w load acting o in load servicear ord forces from Wch(1) (lbs) -697; m IBC 2021 cl. d combination of d combina | ar capacity for ar capacity for vall shear stiff on top of panel on top of panel acting at head ability factor n shear walls $E_{q_ch[i]}$ (lbs) 0; 0; 1605.1 Basic no.1 no.2 no.3 no.4 no.5 no.6 rs or – Table 2.3.2 | r seismic des r wind design fness of panel above Dc_ch[i] (Ibs) 0; 0; Ioad combinat | Capacities for ign $v_s = min(5)$ $v_w = min(7)$ $G_a = 16 ki$ D = 460 lb/ S = 800 lb/ S = 800 lb/ W = 3800 lb/ $O_{T_ch[1]}(lbs)$ 0; 0; tions from ASC D + 0.6W $D + 0.75L_f$ $D + 0.75L_f$ 0.6D + 0.61 | r Wood-Frame 60 plf × min[785 plf × min[7 | Shear Walls 1 - (0.5 - G), 1 1 - (0.5 - G), 1 1 - (0.5 - G), 1 L _{r_ch[]} (Ibs) 0; 0; 2.4 5(L _r or S or R) |], 1740 plf) = I], 2435 plf) = S ch[ī] (Ibs) 0; | 520.8 lb/ft 730.1 lb/ft R ch[i] (lbs) 0; | |

| Tekla Tedds | Project | | | | Job Ref. | |
|--|-------------------|--|--|--|--|-----------------------------|
| Anthem Structural Engineers | Section | | | | Sheet no./rev | |
| | | | 3 | | | |
| | Calc. by S | Date 8/1/2022 | Chk'd by | Date | App'd by | Date |
| | 0 | 0, 1/2022 | | | | |
| Wet service factor for tension – 1 Wet service factor for compression | | C _{Mt} = 1.00 C _{Mc} = 1.00 | | | WE | ST, LEVEL |
| Wet service factor for modulus o | | | | | | |
| | relationly re | Сме = 1.00 | | | | |
| Temperature factor for tension – | Table 2.3.3 | C _{tt} = 1.00 | | | | |
| Temperature factor for compress | | | | | | |
| | | C _{tc} = 1.00 | | | | |
| Temperature factor for modulus of | of elasticity – T | able 2.3.3 | | | | |
| | | CtE = 1.00 | | | | |
| Incising factor – cl.4.3.8 | | C _i = 1.00 | | | | |
| Buckling stiffness factor - cl.4.4.2 | 2 | C⊤ = 1.00 | | | | |
| Adjusted modulus of elasticity | | E _{min} ' = E _{min} : | $< C_{ME} \times C_{tE} \times C_{i}$ | × C _T = 550000 | psi | |
| Critical buckling design value | | F _{cE} = 0.822 | imes E _{min} ' / (h / d) ² | = 904 psi | | |
| Reference compression design v | alue | $F_{c^*} = F_{c} \times C$ | $D 	imes C_{Mc} 	imes C_{tc} 	imes C_{tc}$ | C _{Fc} × C _i = 2376 | psi | |
| For sawn lumber | | c = 0.8 | | | | |
| Column stability factor - eqn.3 | 3.7-1 | C _P = (1 + (| F _{cE} / F _c *)) / (2 > | × c) – √([(1 + | (F _{cE} / F _c *)) / (2 | × c)]² - (F _{cE} / |
| | | F _c *) / c) = (| .34 | | | |
| From SDPWS Table 4.3.4 Maxin | mum Shear W | /all Aspect Ratio | S | | | |
| Maximum shear wall aspect ratio | | 3.5 | - | | | |
| Perforated wall length | | b₁ = 17.5 ft | | | | |
| Shear wall aspect ratio | | h / b1 = 0.5 8 | 6 | | | |
| Perforated wall length | | b ₂ = 4 ft | | | | |
| Shear wall aspect ratio | | h / b ₂ = 2.56 | 3 | | | |
| Shear capacity adjustment fact | tor – cl.4.3.3.5 | 5 | | | | |
| Sum of perforated shear wall leng | gths | $\Sigma L_i = b_1 + b_2$ | $2 \times 2 \times b_s / h = 2$ | 0.622 ft | | |
| Total length of perforated shear v | wall | $L_{tot} = b_1 + w$ | _{p1} + 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 | r (eqn. 4.3-5) | C _o = 0.942 | | | | |
| Perforated shear wall capacity | | | | | | |
| Maximum shear force under v | vind loading | $V_{w_{max}} = 0.$ | 6 × W = 2.28 k | ips | | |
| Shear capacity for wind loadin | g | $V_w = v_w \times 0$ | $C_{o} \times \Sigma L_{i} / 2 = 7.$ | . 089 kips | | |
| | - | V _{w max} / V _w = | | | | |
| | | - | | for wind load | exceeds maxin | num shear for |
| Chord capacity for chord 1 | | | | | | |
| Load combination 5 | | | | | | |
| Shear force for maximum tensior | ı | $V = 0.6 \times W$ | = 2.28 kips | | | |
| Axial force for maximum tensi | on | | · | × b / 2 + 0.6 × | : W _{ch1} = 4.138 k | ips |
| Maximum tensile force in chord | | • | | | ← NEGATIVE. | • |
| Maximum applied tensile stress | | f _t = T / A _{en} = | | | OVERTURN | ING |
| | | F_t = $F_t \times C_D$ | \times C _{Mt} \times C _{tt} \times C _{Et} | t × Ci = 1508 lb | /in² | |
| Design tensile stress | | | | | | |
| Design tensile stress | | ft / Ft' = -0.1 | | | | |

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| Load combination 1 | | | | | WE | ST, LEVE |
| Shear force for maximum compre | ession | $V = 0.6 \times W$ | = 2.28 kips | | | |
| Axial force for maximum com | pression | P = ((D + S | $S_{wt} \times h) \times s / 2$ | + -1 × 0.6 × | W _{ch1} = 0.793 kip | os |
| Maximum compressive force in c | hord | $C = V \times h / ($ | $((C_o \times \Sigma L_i)) + P$ | = 1.997 kips | | |
| Maximum applied compressive s | | $f_c = C / A_e =$ | | | | |
| Design compressive stress | | $F_{c}' = F_{c} \times C_{c}$ | $0 \times C_{Mc} \times C_{tc} \times C_{tc}$ | $C_{Fc} \times C_i \times C_P =$ | 818 lb/in ² | |
| | | fc / Fc' = 0.1 4 | 48 | | | |
| | PASS - | Design compres | ssive stress ex | ceeds maxin | num applied con | npressive s |
| Chord capacity for chord 2 Load combination 5 | | | | | | |
| Shear force for maximum tensior | | V = 0.6 × W | - 2 28 kins | | | |
| | | | | | | 120 kino |
| Axial force for maximum tensi | on | | . ,, | | $0.6 \times W_{ch2} = 4.1$ | • |
| Maximum tensile force in chord | | | | -2.935 KIPS | NEGATIVE. OVERTURN | |
| Maximum applied tensile stress | | ft = T / A _{en} = | | | h /in2 | |
| Design tensile stress | | $F_t = F_t \times C_D$ $f_t / F_t' = -0.14$ | $\times C_{Mt} \times C_{tt} \times C_{F}$ | t × Ci = 1508 I | D/IN- | |
| | | | | ass avraads | maximum applie | od tonsilo s |
| Load combination 1 | | 7 400 - 203 | ingir tensile str | | | |
| Shear force for maximum compre | ession | V = 0.6 × W | = 2.28 kips | | | |
| Axial force for maximum com | | | $S_{wt} \times h) \times s / 2$ | $+0.6 \times W_{cb'}$ | a = 0 793 kins | |
| Maximum compressive force in c | | | $((C_o \times \Sigma L_i)) + P$ | | | |
| Maximum applied compressive s | | $f_c = C / A_e =$ | | | | |
| Design compressive stress | | | $X \simeq C_{Mc} \times C_{tc} \times C_{tc}$ | $C_{Fc} \times C_i \times C_P =$ | 818 lb/in ² | |
| g., | | f _c / F _c ' = 0.1 | | | •••• | |
| | PASS - | Design compres | | ceeds maxin | num applied con | pressive s |
| Collector capacity | | | | | | - |
| | | Collector axial force dia | ıgram (kips) | | | |
| | | | | | | |



Maximum shear force on wall Uniform shear applied to wall Shear resisted by wall segments Maximum force in collector
$$\begin{split} V_{max} &= V_{w_max} = \textbf{2.28 kips} \\ v_a &= V_{max} / ((C_o \times \Sigma L_i)) = \textbf{117.4 plf} \\ v_b &= v_a \times b / (b_1 + b_2) = \textbf{147.4 plf} \\ P_{coll} &= \textbf{0.526 kips} \end{split}$$

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| Maximum applied tensile stress | | $f_t = P_{coll} / (2$ | × As) = 32 lb/in ² | 2 | WE | ST, LEVEL | | | |
| Design tensile stress | | F_t = $F_t \times C_D$ | imes C _{Mt} $	imes$ C _{tt} $	imes$ C _F | t × Ci = 1508 | b/in ² | | | | |
| | | ft / Ft' = 0.02 | :1 | | | | | | |
| | | PASS - Des | sign tensile str | ess exceeds | maximum applie | ed tensile stre | | | |
| Maximum applied compressive s | stress | $f_c = P_{coll} / (2$ | × As) = 32 lb/in ² | 2 | | | | | |
| Column stability factor | | C _P = 1.00 | | | | | | | |
| Design compressive stress | | $\textbf{F_c'} = \textbf{F_c} \times \textbf{C_D} \times \textbf{C_{Mc}} \times \textbf{C_{tc}} \times \textbf{C_{Fc}} \times \textbf{C_i} \times \textbf{C_P} = \textbf{2376} \text{ Ib/in}^2$ | | | | | | | |
| | | f _c / F _c ' = 0.013 | | | | | | | |
| | PASS - | Design compres | ssive stress ex | ceeds maxim | um applied con | pressive stre | | | |
| Wind load deflection | | | | | | | | | |
| Design shear force | | $V_{\delta w} = f_{Wserv}$ | × W = 3.8 kips | s | | | | | |
| Deflection limit | | $\Delta_{w_{allow}}$ = h / | 500 = 0.246 ir | า | | | | | |
| Induced unit shear | | $v_{\delta w_{max}} = V_{\delta w}$ | , / (C _o ×ΣL _i) = 1 | 95.67 lb/ft | | | | | |
| Anchor tension force | | $T_{\delta} = max(0)$ | kips,v $_{\delta w_max} \times h$ · | - 0.6 × (D + S _w | $_{t} \times$ h) \times b / 2 + | | | | |
| | | max(abs(W | ch1), abs(W ch2))) | = 0.000 kips | | | | | |
| Shear wall deflection - Eqn. 4.3- | ·1 | $\delta_{sww} = 2 \times v_{\delta}$ | $_{m_{max}} 	imes h^3$ / (3 $	imes$ | $E \times A_e \times \Sigma L_i$) + | ⊦ v _{δw_max} × h / (Ga) |) + h $	imes$ T $_{\delta}$ / (k _a > | | | |
| | | ΣLi) = 0.129 | in | | | | | | |
| | | $\delta_{ m sww}$ / $\Delta_{ m w_allow}$ | , = 0.523 | | | | | | |
| | | | PASS - She | ar wall deflec | tion is less than | deflection li | | | |

PASS - Shear wall deflection is less than deflection limit

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|---|---|---|--------------|------------|------------|----------|----------------|--|--|--|
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| WOOD SHEAR WALL DESIGN | <u> </u> | design and t | he perforate | d shear wa | all method | | T, LEVE | | | |
| Design summary Description | Unit F | Provided | Required | Utilizatio | n Result | : | | | | |
| Shear capacity | lbs 7 | | 3600 | 0.502 | PASS | | | | | |
| Chord capacity | lb/in ² 8 | 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 sheathin Panel height | g on one side | h = 10.25 fi b = 27 ft | t | | | | | | | |
| Panel length | | | | | | | | | | |
| Panel length | · • • • • • • • • • • • • • • • • • • • | | D+S | ·+++++++ | ×++++++++ | | | | | |

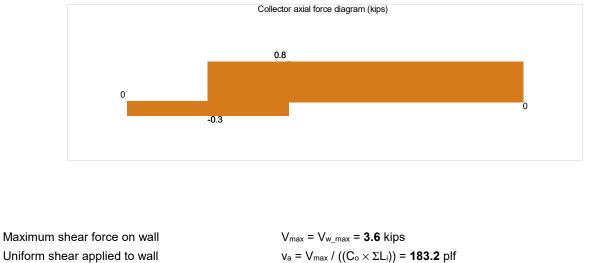
| • | Ch1 | | | Ch2 |
|----------|-----------------|---------------|---|-------|
| | ⊲ 5' 6"▶ | ⊲ 5'6" | → | ► |

| 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 | I _{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" |

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|--|--|---|--|--|--|---|--|---|--|--|
| Antl | hem Structural | Engineers | Section | | | | | Sheet no./rev. | | |
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| Cros | ss-sectional are | ea of end posts | ; | A _e = 16.5 ii | n ² | | v | VEST, LEVEL | | |
| Hole diameter | | | | Dia = 1 in | Dia = 1 in | | | | | |
| Net | cross-sectiona | l area of end p | osts | A _{en} = 13.5 | in ² | | | | | |
| Nom | ninal collector s | size | | 2 x 2" x 6" | | | | | | |
| Dres | ssed collector s | size | | 2 x 1.5" x 5 | 5.5" | | | | | |
| Serv | vice condition | | | Dry | | | | | | |
| Temperature | | | | 100 degF o | or less | | | | | |
| Vert | Vertical anchor stiffness | | | | lb/in | | | | | |
| From | m NDS Supple | ment Table 4 | A - Reference | design values | for visually o | raded dimone | ion lumber (? | " - 4" thick) | | |
| | cies, grade and | | | - | o.1 & btr grade | | | - + theky | | |
| | cific gravity | | | G = 0.43 | or a ba grade | , <u></u> a maoi | | | | |
| | sion parallel to | orain | | G = 0.43 Ft = 725 lb/ | /in ² | | | | | |
| | npression paral | - | | $F_c = 1350$ | | | | | | |
| | lulus of elastici | • | | | | | | | | |
| | imum modulus | • | | | E = 1500000 lb/in ² E _{min} = 550000 lb/in ² | | | | | |
| | | - | | | | | | | | |
| | athing details | | | | | | | | | |
| | eathing materi | al | | | 7/16" wood panel structural I oriented strandboard sheathing | | | | | |
| Fas | stener type | | | 8d commo | on nails at 6" | centers | | | | |
| Fro | m SDPWS Tab | le 4 3A Nomir | hal Unit Shear | Canacities for | | | M/ | | | |
| | | | | oupuonnoo ioi | r wood-Frame | e Shear Walls | - wood-based | l Panels | | |
| | ninal unit she | | | - | | | | | | |
| Non | | ar capacity fo | r seismic des | ign v _s = min(5 | 60 plf × min[| 1 - (0.5 - G), 1 |], 1740 plf) = | 520.8 lb/ft | | |
| Non Non | ninal unit she | ar capacity fo ar capacity fo | r seismic des r wind design | ign v _s = min(5 v _w = min(7 | i60 plf × min[785 plf × min[| |], 1740 plf) = | 520.8 lb/ft | | |
| Non Non | | ar capacity fo ar capacity fo | r seismic des r wind design | ign v _s = min(5 | i60 plf × min[785 plf × min[| 1 - (0.5 - G), 1 |], 1740 plf) = | 520.8 lb/ft | | |
| Non Non App | ninal unit she | ar capacity fo ar capacity fo | r seismic des r wind design | ign v _s = min(5 v _w = min(7 | i60 plf × min[785 plf × min[| 1 - (0.5 - G), 1 |], 1740 plf) = | 520.8 lb/ft | | |
| Non Non App Loa | minal unit shea parent shear w | ar capacity fo ar capacity fo vall shear stiff | r seismic des r wind design | ign v _s = min(5 v _w = min(7 | 60 plf × min[785 plf × min[ps/in | 1 - (0.5 - G), 1 |], 1740 plf) = | 520.8 lb/ft | | |
| Non Non App Loa Dea | minal unit shea parent shear w ding details | ar capacity fo ar capacity fo vall shear stiff n top of panel | r seismic des r wind design | ign v _s = min(5 v _w = min(7 G _a = 16 ki | i60 plf × min[785 plf × min[ps/in ′ft | 1 - (0.5 - G), 1 |], 1740 plf) = | 520.8 lb/ft | | |
| Non Non App Loa Dea Sno | ninal unit shea parent shear w ding details nd load acting o | ar capacity fo ar capacity fo vall shear stiff n top of panel on top of panel | r seismic des r wind design | ign v _s = min(5 v _w = min(7 G _a = 16 ki D = 720 lb/ | 560 plf × min[785 plf × min[ps/in /ft | 1 - (0.5 - G), 1 |], 1740 plf) = | 520.8 lb/ft | | |
| Non Non App Loa Dea Sno Self | minal unit shea parent shear w ding details id load acting o w load acting c | ar capacity fo ar capacity fo vall shear stiff n top of panel on top of panel | r seismic des r wind design ness | ign v _s = min(5 v _w = min(7 G _a = 16 ki D = 720 lb/ S = 800 lb/ | 60 plf × min[785 plf × min[ps/in /ft /ft | 1 - (0.5 - G), 1 |], 1740 plf) = | 520.8 lb/ft | | |
| Non Non App Loa Dea Sno Self In pl | ninal unit shear parent shear w ding details id load acting o w load acting o weight of pane | ar capacity fo ar capacity fo vall shear stiff n top of panel on top of panel el acting at head | r seismic des r wind design ness | ign v _s = min(5 v _w = min(7 G _a = 16 ki D = 720 lb/ S = 800 lb/ S _{wt} = 10 lb/ | 60 plf × min[785 plf × min[ps/in /ft /ft /ft² | 1 - (0.5 - G), 1 |], 1740 plf) = | 520.8 lb/ft | | |
| Non App Loa Dea Sno Self In pl Win | ninal unit shea parent shear w ding details id load acting o w load acting o weight of pane lane wind load | ar capacity fo ar capacity fo vall shear stiff n top of panel on top of panel acting at head ability factor | r seismic des r wind design ness of panel | ign $v_s = min(5)$ $v_w = min(7)$ $G_a = 16 ki$ D = 720 lb/ S = 800 lb/ $S_{wt} = 10 lb/$ W = 6000 lb/ | 60 plf × min[785 plf × min[ps/in /ft /ft /ft² | 1 - (0.5 - G), 1 |], 1740 plf) = | 520.8 lb/ft | | |
| Non App Loa Dea Sno Self In pl Win | minal unit shear barent shear w ding details id load acting o w load acting o w load acting o w load acting o weight of pane lane wind load d load servicea ord forces from | ar capacity fo ar capacity fo vall shear stiff on top of panel on top of panel el acting at head ability factor n shear walls a | r seismic des r wind design ness of panel | ign v _s = min(5 v _w = min(7 G _a = 16 ki D = 720 lb/ S = 800 lb/ S _{wt} = 10 lb/ W = 6000 l f _{Wserv} = 1.00 | 60 plf × min[785 plf × min[ps/in /ft /ft /ft² | 1 - (0.5 - G), 1 |], 1740 plf) = I], 2435 plf) = | 520.8 lb/ft 730.1 lb/ft | | |
| Non Non App Dea Sno Self In pl Win | minal unit shear barent shear w ding details id load acting o w load acting o w load acting o w load acting o w load acting o lane wind load d load servicea ord forces from Wchiji (lbs) | ar capacity fo ar capacity fo vall shear stiff on top of panel on top of panel el acting at head ability factor n shear walls of $E_{q_ch(I)}$ (Ibs) | r seismic des r wind design ness of panel above Dc_chii (lbs) | ign $v_s = min(5)$ $v_w = min(7)$ $G_a = 16 ki$ D = 720 lb/ S = 800 lb/ $S_{wt} = 10 lb/$ W = 6000 lb/ | 60 plf × min[785 plf × min[ps/in /ft ft ft2 lbs 0 Lf_ch[1] (Ibs) | 1 - (0.5 - G), 1 1 - (0.5 - G), 1 |], 1740 plf) = | 520.8 lb/ft 730.1 lb/ft Rch[i] (lbs) | | |
| Non Non App Dea Sno Self In pl Win Chord | minal unit shear barent shear w ding details id load acting o w load acting o w load acting o w load acting o weight of pane lane wind load d load servicea ord forces from | ar capacity fo ar capacity fo vall shear stiff on top of panel on top of panel el acting at head ability factor n shear walls a | r seismic des r wind design ness of panel above | ign v _s = min(5 v _w = min(7 G _a = 16 ki D = 720 lb/ S = 800 lb/ S _{wt} = 10 lb/ W = 6000 l f _{Wserv} = 1.00 | 60 plf × min[785 plf × min[ps/in /ft /ft lbs 0 | 1 - (0.5 - G), 1 1 - (0.5 - G), <i>1</i> Lr_ch[i] (Ibs) |], 1740 plf) = I], 2435 plf) = S ch[i] (Ibs) | 520.8 lb/ft 730.1 lb/ft | | |
| Non Non App Loa Dea Sno Self In pl Win Cho Chord Chord Ch1 Ch2 | minal unit shear barent shear w ding details id load acting o w load acting o | ar capacity fo ar capacity fo vall shear stiff on top of panel on top of panel acting at head ability factor n shear walls $E_{q_ch[1]}$ (Ibs) 0; 0; | r seismic des r wind design ness of panel above Dc_ch[i] (Ibs) 0; 0; | ign $v_s = min(5)$ $v_w = min(7)$ $G_a = 16 ki$ D = 720 lb/ S = 800 lb/ S = 800 lb/ $S_{wt} = 10 lb/$ W = 6000 l $f_{Wserv} = 1.00$ $D_{T_ch[i]}(lbs)$ 0; 0; | 60 plf × min[785 plf × min[ps/in /ft ft /ft² lbs 0 L _{f_ch[1]} (lbs) 0; 0; | 1 - (0.5 - G), 1 1 - (0.5 - G), 7 1 - (0.5 - G), 7 0; 0; |], 1740 plf) = I], 2435 plf) = S ch[i] (Ibs) 0; | 520.8 lb/ft 730.1 lb/ft R ch[i] (lbs) 0; | | |
| Non Non App Loa Dea Sno Self In pl Win Cho Chord Ch1 Ch1 Ch2 Fror | minal unit shear barent shear w ding details id load acting o w load acting o m IBC 2021 cl. | ar capacity fo ar capacity fo vall shear stiff on top of panel on top of panel acting at head ability factor n shear walls a $E_{q_ch[]}$ (Ibs) 0; 0; 1605.1 Basic | r seismic des r wind design ness of panel above Dc_ch[i] (Ibs) 0; 0; | ign v _s = min(5 v _w = min(7 G _a = 16 ki D = 720 lb/ S = 800 lb/ S _{wt} = 10 lb/ W = 6000 l f _{Wserv} = 1.00 DT_ch[i] (lbs) 0; 0; tions from ASC | 60 plf × min[785 plf × min[ps/in /ft ft /ft² lbs 0 L _{f_ch[1]} (lbs) 0; 0; | 1 - (0.5 - G), 1 1 - (0.5 - G), 7 1 - (0.5 - G), 7 0; 0; |], 1740 plf) = I], 2435 plf) = S ch[i] (Ibs) 0; | 520.8 lb/ft 730.1 lb/ft R ch[i] (lbs) 0; | | |
| Non Non App Loa Dea Sno Self In pl Win Cho Chord Chord Ch1 Ch2 | minal unit shear parent shear w ding details id load acting o w load acting o w load acting o weight of pane lane wind load d load servicea ord forces from Wch[i] (lbs) -1570; 1570; m IBC 2021 cl. d combination | ar capacity fo ar capacity fo vall shear stiff on top of panel on top of panel acting at head ability factor n shear walls a $E_{q_ch[1]}$ (Ibs) 0; 0; 1605.1 Basic no.1 | r seismic des r wind design ness of panel above Dc_ch[i] (Ibs) 0; 0; | ign v _s = min(5 v _w = min(7 G _a = 16 ki D = 720 lb/ S = 800 lb/ S _{wt} = 10 lb/ W = 6000 l f _{Wserv} = 1.00 D _{T_ch[1]} (lbs) 0; 0; tions from ASC D + 0.6W | 60 plf × min[785 plf × min[ps/in /ft ft /ft² lbs 0 L _{f_ch[1]} (lbs) 0; 0; | 1 - (0.5 - G), 1 1 - (0.5 - G), 7 1 - (0.5 - G), 7 0; 0; |], 1740 plf) = I], 2435 plf) = S ch[i] (Ibs) 0; | 520.8 lb/ft 730.1 lb/ft R ch[i] (lbs) 0; | | |
| Non Non App Loa Dea Sno Self In pl Win Cho Chord Chord Chord Ch1 Ch2 Froi Load | minal unit shear parent shear w ding details ad load acting o w load servicea lane wind load d load servicea ord forces from Wchttl (lbs) -1570; 1570; m IBC 2021 cl. d combination of d combination of | ar capacity fo ar capacity fo vall shear stiff on top of panel on top of panel acting at head ability factor n shear walls $E_{q_ch[1]}$ (Ibs) 0; 0; 1605.1 Basic no.1 no.2 | r seismic des r wind design ness of panel above Dc_ch[i] (Ibs) 0; 0; | ign v _s = min(5 v _w = min(7 G _a = 16 ki D = 720 lb/ S = 800 lb/ S _{wt} = 10 lb/ W = 6000 l f _{Wserv} = 1.00 D _{T_ch[i]} (lbs) 0; 0; tions from ASC D + 0.6W D + 0.7E | 60 plf × min[785 plf × min[ps/in /ft ft /ft² lbs 0 Lf_ch[i] (lbs) 0; 0; 0; CE 7, section | 1 - (0.5 - G), 1 1 - (0.5 - G), 7 1 - (0.5 - G), 7 0; 0; 0; 2.4 |], 1740 plf) = I], 2435 plf) = S ch[i] (Ibs) 0; | 520.8 lb/ft 730.1 lb/ft R ch[i] (lbs) 0; | | |
| Non Non App Loa Dea Sno Self In pl Win Cho Chord Ch1 Ch2 Fror Load Load | minal unit shear parent shear w ding details ad load acting o w load servicea ord forces from wch(i) (lbs) -1570; 1570; m IBC 2021 cl. d combination of d combination of d combination of | ar capacity fo ar capacity fo vall shear stiff on top of panel on top of panel acting at head ability factor n shear walls a $E_{q_ch[]}$ (Ibs) 0; 0; 1605.1 Basic no.1 no.2 no.3 | r seismic des r wind design ness of panel above Dc_ch[i] (Ibs) 0; 0; | $\begin{array}{l} \text{ign } v_{s} = \min(5) \\ v_{w} = \min(7) \\ G_{a} = 16 \text{ ki} \\ D = 720 \text{ lb/} \\ S = 800 \text{ lb/} \\ S_{wt} = 10 \text{ lb/} \\ W = 6000 \text{ l} \\ f_{Wserv} = 1.00 \\ \hline \\ \textbf{D}_{T_ch[1]}(\textbf{lbs}) \\ 0; \\ 0; \\ \hline \\ \textbf{tions from ASC} \\ D + 0.6W \\ D + 0.7E \\ D + 0.75L_f \end{array}$ | 60 plf × min[785 plf × min[785 plf × min] ps/in /ft /ft /ft /ft /ft /ft /ft /ft /ft /ft | 1 - (0.5 - G), 1 1 - (0.5 - G), 1 0; 0; 0; 2.4 |], 1740 plf) = I], 2435 plf) = S ch[i] (Ibs) 0; | 520.8 lb/ft 730.1 lb/ft R ch[i] (lbs) 0; | | |
| Non Non App Loa Dea Sno Self In pl Win Cho Chord Ch1 Ch2 Fron Load Load | minal unit shear parent shear w ding details id load acting o w load acting o w load acting o weight of pane lane wind load d load servicea ord forces from Wch[i] (lbs) -1570; 1570; m IBC 2021 cl. d combination id d combination id d combination id | ar capacity fo ar capacity fo vall shear stiff on top of panel on top of panel acting at head ability factor n shear walls a $E_{q_ch(I)}$ (Ibs) 0; 0; 1605.1 Basic no.1 no.2 no.3 no.4 | r seismic des r wind design ness of panel above Dc_ch[i] (Ibs) 0; 0; | $\begin{array}{l} \text{ign } v_{s} = \min(5) \\ v_{w} = \min(7) \\ G_{a} = 16 \text{ ki} \\ D = 720 \text{ lb/} \\ S = 800 \text{ lb/} \\ S_{wt} = 10 \text{ lb/} \\ W = 6000 \text{ l} \\ f_{Wserv} = 1.00 \\ \hline \end{array}$ $\begin{array}{l} \textbf{D}_{T_{c}ch[i]}(\textbf{lbs}) \\ 0; \\ 0; \\ \hline \end{array}$ $\begin{array}{l} \textbf{D}_{T_{c}ch[i]}(\textbf{lbs}) \\ 0; \\ 0; \\ \hline \end{array}$ $\begin{array}{l} \textbf{D}_{T_{c}ch[i]}(\textbf{lbs}) \\ 0; \\ 0; \\ \hline \end{array}$ $\begin{array}{l} \textbf{D}_{T_{c}ch[i]}(\textbf{lbs}) \\ 0; \\ 0; \\ \hline \end{array}$ $\begin{array}{l} \textbf{D}_{T_{c}ch[i]}(\textbf{lbs}) \\ 0; \\ 0; \\ \hline \end{array}$ $\begin{array}{l} \textbf{D}_{T_{c}ch[i]}(\textbf{lbs}) \\ 0; \\ 0; \\ \hline \end{array}$ | 60 plf × min[785 plf × min[785 plf × min] ps/in ft ft ft ft ft ft ft ft ft ft ft ft ft | 1 - (0.5 - G), 1 1 - (0.5 - G), 1 0; 0; 0; 2.4 |], 1740 plf) = I], 2435 plf) = S ch[i] (Ibs) 0; | 520.8 lb/ft 730.1 lb/ft Rch[i] (lbs) 0; | | |
| Non Non App Loa Dea Sno Self In pl Win Cho Chord Chord Ch1 Ch2 Fror Load Load Load | minal unit shear parent shear w ding details ad load acting o w load servicea ord forces from Wchttl (lbs) -1570; 1570; m IBC 2021 cl. d combination of d combination of d combination of d combination of d combination of d combination of d combination of | ar capacity fo ar capacity fo vall shear stiff on top of panel on top of panel acting at head ability factor n shear walls $E_{q_ch[1]}$ (Ibs) 0; 0; 1605.1 Basic no.1 no.2 no.3 no.4 no.5 | r seismic des r wind design ness of panel above Dc_ch[i] (Ibs) 0; 0; | ign $v_s = min(5)$ $v_w = min(7)$ $G_a = 16$ ki D = 720 lb/ S = 800 lb/ S = 800 lb/ S = 800 lb/ W = 6000 l $f_{Wserv} = 1.00$ $D_{T_ch[i]}$ (lbs) 0; 0; tions from ASC D + 0.6W D + 0.7E $D + 0.75L_f$ $0.6D + 0.6^{10}$ | 60 plf × min[785 plf × min[785 plf × min[ps/in /ft ft ft ft ft ft ft ft ft ft ft ft ft f | 1 - (0.5 - G), 1 1 - (0.5 - G), 1 0; 0; 0; 2.4 |], 1740 plf) = I], 2435 plf) = S ch[i] (Ibs) 0; | 520.8 lb/ft 730.1 lb/ft Rch[i] (lbs) 0; | | |
| Non Non App Loa Dea Sno Self In pl Win Cho Chord Chord Ch1 Ch2 Fror Load Load Load | minal unit shear parent shear w ding details id load acting o w load acting o w load acting o weight of pane lane wind load d load servicea ord forces from Wch[i] (lbs) -1570; 1570; m IBC 2021 cl. d combination id d combination id d combination id | ar capacity fo ar capacity fo vall shear stiff on top of panel on top of panel acting at head ability factor n shear walls $E_{q_ch[1]}$ (Ibs) 0; 0; 1605.1 Basic no.1 no.2 no.3 no.4 no.5 | r seismic des r wind design ness of panel above Dc_ch[i] (Ibs) 0; 0; | $\begin{array}{l} \text{ign } v_{s} = \min(5) \\ v_{w} = \min(7) \\ G_{a} = 16 \text{ ki} \\ D = 720 \text{ lb/} \\ S = 800 \text{ lb/} \\ S_{wt} = 10 \text{ lb/} \\ W = 6000 \text{ l} \\ f_{Wserv} = 1.00 \\ \hline \end{array}$ $\begin{array}{l} \textbf{D}_{T_{c}ch[i]}(\textbf{lbs}) \\ 0; \\ 0; \\ \hline \end{array}$ $\begin{array}{l} \textbf{D}_{T_{c}ch[i]}(\textbf{lbs}) \\ 0; \\ 0; \\ \hline \end{array}$ $\begin{array}{l} \textbf{D}_{T_{c}ch[i]}(\textbf{lbs}) \\ 0; \\ 0; \\ \hline \end{array}$ $\begin{array}{l} \textbf{D}_{T_{c}ch[i]}(\textbf{lbs}) \\ 0; \\ 0; \\ \hline \end{array}$ $\begin{array}{l} \textbf{D}_{T_{c}ch[i]}(\textbf{lbs}) \\ 0; \\ 0; \\ \hline \end{array}$ $\begin{array}{l} \textbf{D}_{T_{c}ch[i]}(\textbf{lbs}) \\ 0; \\ 0; \\ \hline \end{array}$ | 60 plf × min[785 plf × min[785 plf × min[ps/in /ft ft ft ft ft ft ft ft ft ft ft ft ft f | 1 - (0.5 - G), 1 1 - (0.5 - G), 1 0; 0; 0; 2.4 |], 1740 plf) = I], 2435 plf) = S ch[i] (Ibs) 0; | 520.8 lb/ft 730.1 lb/ft Rch[i] (lbs) 0; | | |
| Non Non App Loa Dea Sno Self In pl Win Cho Chord Chord Chord Ch1 Ch2 Fror Load Load Load Load Load Adju | minal unit shear parent shear w ding details ad load acting o w load servicea ord forces from wch[] (lbs) -1570; 1570; 1570; m IBC 2021 cl. d combination of d combination of d combination of d combination of d combi | ar capacity fo ar capacity fo vall shear stiff on top of panel on top of panel acting at head ability factor n shear walls $E_{q_ch[1]}$ (Ibs) 0; 0; 1605.1 Basic no.1 no.2 no.3 no.4 no.5 no.6 rs | r seismic des r wind design ness of panel above Dc_ch[i] (Ibs) 0; 0; load combina | ign $v_s = min(5)$ $v_w = min(7)$ $G_a = 16$ ki D = 720 lb/ S = 800 lb/ S = 800 lb/ S = 800 lb/ W = 6000 l $f_{Wserv} = 1.00$ $D_{T_ch[1]}$ (lbs) 0; 0; 1005 $D_{T_ch[1]}$ (lbs) 0; 0; 1005 $D_{T_ch[1]}$ (lbs) 0; 0; 1005 $D_{T_ch[1]}$ (lbs) 0; 0; 1007 $D_{T_ch[1]}$ (lbs) 0; 0; 0; 1007 $D_{T_ch[1]}$ (lbs) 0; 0; 0; 0; 0; 0; 0; 0; 0; 0; | 60 plf × min[785 plf × min[785 plf × min[ps/in /ft ft ft ft ft ft ft ft ft ft ft ft ft f | 1 - (0.5 - G), 1 1 - (0.5 - G), 1 0; 0; 0; 2.4 |], 1740 plf) = I], 2435 plf) = S ch[i] (Ibs) 0; | 520.8 lb/ft 730.1 lb/ft Rch[i] (lbs) 0; | | |
| Non Non App Loa Dea Sno Self In pl Win Cho Chord Chord Chord Ch1 Ch2 Froi Load Load Load Load Load Adju | minal unit shear parent shear w ding details id load acting o w load acting o d load servicea ord forces from Wch[i] (lbs) -1570; 1570; m IBC 2021 cl. d combination id d combination id d combination id d combination id d combination id d combination id d combination id | ar capacity fo ar capacity fo vall shear stiff on top of panel on top of panel acting at head ability factor n shear walls $E_{q_ch[1]}$ (Ibs) 0; 0; 1605.1 Basic no.1 no.2 no.3 no.4 no.5 no.6 rs | r seismic des r wind design ness of panel above Dc_ch[i] (Ibs) 0; 0; load combina | ign $v_s = min(5)$ $v_w = min(7)$ $G_a = 16$ ki D = 720 lb/ S = 800 lb/ S = 800 lb/ $S_{wt} = 10$ lb/ W = 6000 l $f_{Wserv} = 1.00$ $D_{T_ch[1]}$ (lbs) 0; tions from ASC D + 0.6W D + 0.7E $D + 0.75L_f$ $D + 0.75L_f$ $0.6D + 0.6^{10}$ 0.6D + 0.71 | 60 plf × min[785 plf × min[785 plf × min[ps/in /ft ft ft ft ft ft ft ft ft ft ft ft ft f | 1 - (0.5 - G), 1 1 - (0.5 - G), 1 0; 0; 0; 2.4 |], 1740 plf) = I], 2435 plf) = S ch[i] (Ibs) 0; | 520.8 lb/ft 730.1 lb/ft Rch[i] (lbs) 0; | | |
| Non Non App Loa Dea Sno Self In pl Win Cho Chord Ch1 Ch2 Fror Load Load Load Load Load Load Load | minal unit shear parent shear w ding details ad load acting o w load servicea ord forces from wch[] (lbs) -1570; 1570; 1570; m IBC 2021 cl. d combination of d combination of d combination of d combination of d combi | ar capacity fo ar capacity fo vall shear stiff on top of panel on top of panel acting at head ability factor n shear walls $E_{q_ch(I)}$ (Ibs) 0; 0; 1605.1 Basic no.1 no.2 no.3 no.4 no.5 no.6 rs or – Table 2.3.2 ion – Table 4A | r seismic des r wind design ness of panel above Dc_ch[i] (Ibs) 0; 0; Ioad combina | ign $v_s = min(5)$ $v_w = min(7)$ $G_a = 16$ ki D = 720 lb/ S = 800 lb/ S = 800 lb/ S = 800 lb/ W = 6000 l $f_{Wserv} = 1.00$ $D_{T_ch[1]}$ (lbs) 0; 0; 1005 $D_{T_ch[1]}$ (lbs) 0; 0; 1005 $D_{T_ch[1]}$ (lbs) 0; 0; 1005 $D_{T_ch[1]}$ (lbs) 0; 0; 1007 $D_{T_ch[1]}$ (lbs) 0; 0; 0; 1007 $D_{T_ch[1]}$ (lbs) 0; 0; 0; 0; 0; 0; 0; 0; 0; 0; | 60 plf × min[785 plf × min[785 plf × min[ps/in /ft ft ft ft ft ft ft ft ft ft ft ft ft f | 1 - (0.5 - G), 1 1 - (0.5 - G), 1 0; 0; 0; 2.4 |], 1740 plf) = I], 2435 plf) = S ch[i] (Ibs) 0; | 520.8 lb/ft 730.1 lb/ft Rch[i] (lbs) 0; | | |

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| | | | | | 3 | |
| | Calc. by S | Date 8/1/2022 | Chk'd by | Date | App'd by | Date |
| Wet service factor for tension – | Table 44 | C _{Mt} = 1.00 | I | I | WF | ST, LEVEL |
| Wet service factor for compressi | | С _{мс} = 1.00 | | | | |
| Wet service factor for modulus o | | | | | | |
| | · · · · · · · · · · · · · · · · · · · | Сме = 1.00 | | | | |
| Temperature factor for tension – | Table 2.3.3 | C _{tt} = 1.00 | | | | |
| Temperature factor for compress | sion – Table 2. | 3.3 | | | | |
| | | C _{tc} = 1.00 | | | | |
| Temperature factor for modulus | of elasticity – 1 | 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⊤ = 1.00 | | | | |
| Adjusted modulus of elasticity | | $E_{min}' = E_{min}$ | $	imes C_{\text{ME}} 	imes C_{\text{tE}} 	imes C_{\text{i}}$ | × C _T = 550000 |) psi | |
| Critical buckling design value | | F _{cE} = 0.822 | imes E _{min} ' / (h / d) ² | = 904 psi | | |
| Reference compression design v | /alue | $F_{c}^{*} = F_{c} \times C$ | $D 	imes C_{Mc} 	imes C_{tc} 	imes C$ | C _{Fc} × 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 > | × c) – √([(1 + | (F _{cE} / F _c *)) / (2 | × c)] ² - (F _{cE} / |
| | | F_{c}^{*}) / c) = (| <i>,,</i> , , | | | |
| Perforated wall length Shear wall aspect ratio Perforated wall length Shear wall aspect ratio | ton | b ₁ = 5.5 ft h / b ₁ = 1.8 6 b ₂ = 16 ft h / b ₂ = 0.6 4 | | | | |
| Shear capacity adjustment fac | | | 04 5 4 | | | |
| Sum of perforated shear wall len | - | $\Sigma L_i = b_1 + b_1$ | | | | |
| Total length of perforated shear | wali | | _{o1} + b ₂ = 27 ft | | | |
| Total area of openings | | | l₀1 = 27.5 ft ² | | | |
| Sheathing area ratio (eqn. 4.3-6) Shear capacity adjustment factor | | r = 1 / (1 +) C₀ = 0.914 | $A_o /(h \times \Sigma L_i)) = 0$ | 0.003 | | |
| | | 00 - 0.314 | | | | |
| Perforated shear wall capacity | | \/ ^ | C = C + C | | | |
| Maximum shear force under v | 0 | - | 6 × W = 3.6 kip | | | |
| Shear capacity for wind loadir | ng | | $C_o \times \Sigma L_i / 2 = 7.$ | .171 kips | | |
| | | $V_{w_{max}} / V_{w}$ | | | | |
| | | PASS - S | Snear capacity | tor wind load | exceeds maxin | num shear for |
| Chord capacity for chord 1 Load combination 5 | | | | | | |
| Shear force for maximum tension | n | V = 0.6 × W | / = 3.6 kips | | | |
| Axial force for maximum tens | | | | ×b/2+06× | ≪W _{ch1} = 5.72 ki | os |
| Maximum tensile force in chord | • • | | | | | |
| | | f _t = T / A _{en} = | | | OVERTURN | |
| Maximum applied tensile stress | | | - | - | | |
| Maximum applied tensile stress Design tensile stress | | F _t ' = F _t × C _□ f _t / F _t ' = -0.1 | $X \times C_{Mt} \times C_{tt} \times C_{F}$ | _{it} × C _i = 1508 lk | o/in ² | |

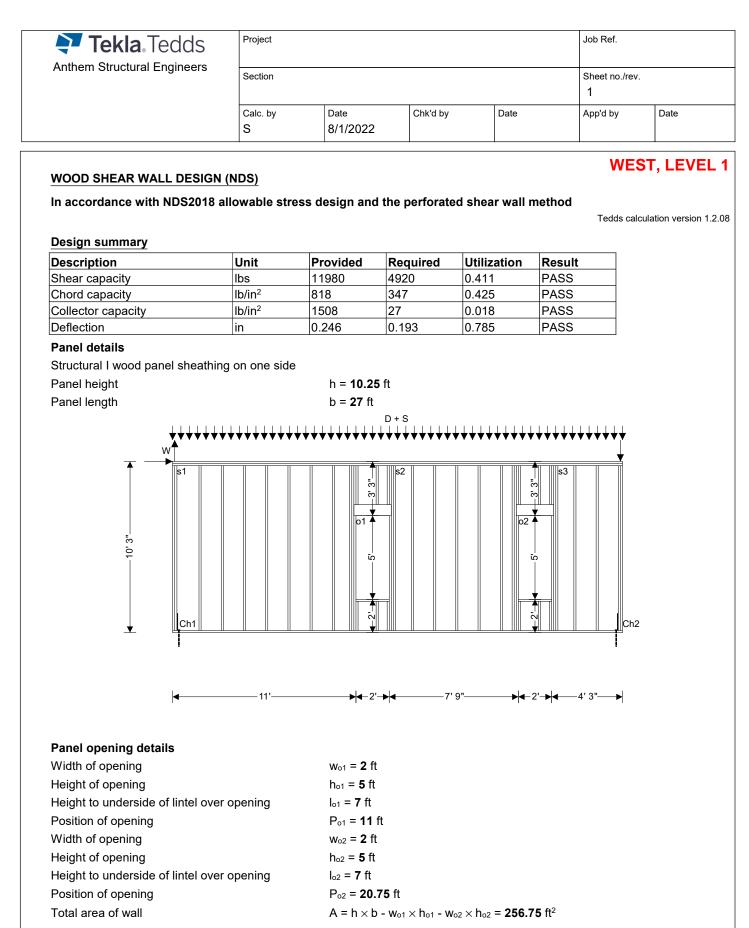
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| Load combination 1 | | | | | WES | ST, LEVI |
| Shear force for maximum compression | | $V = 0.6 \times W$ | ′ = 3.6 kips | | | |
| Axial force for maximum compression | | P = ((D + S | $S_{wt} \times h)) \times s / 2$ | + -1 × 0.6 × W | / _{ch1} = 1.49 kips | |
| Maximum compressive force in c | hord | $C = V \times h / f$ | $((C_o \times \Sigma L_i)) + P =$ | 3.369 kips | | |
| Maximum applied compressive s | tress | $f_c = C / A_e =$ | 204 lb/in ² | | | |
| Design compressive stress | | $F_c' = F_c \times C_l$ | $0 \times C_{Mc} \times C_{tc} \times C_{tc}$ | $C_{Fc} \times C_i \times C_P = 8$ | 18 lb/in² | |
| | | f _c / F _c ' = 0.2 | 50 | | | |
| | PASS - | Design compres | ssive stress ex | ceeds maximu | m applied com | pressive s |
| Chord capacity for chord 2 Load combination 5 | | | | | | |
| Shear force for maximum tensior | ı | $V = 0.6 \times W$ | ′ = 3.6 kips | | | |
| Axial force for maximum tensi | on | P = (0.6× | $(D + S_{wt} \times h)) >$ | < b / 2 + -1 × 0 | $.6 \times W_{ch2} = 5.7$ | 2 kips |
| Maximum tensile force in chord | | $T = V \times h / ($ | $(C_o \times \Sigma L_i)) - P =$ | -3.842 kips - | — NEGATIVE. I | NO |
| Maximum applied tensile stress | | f _t = T / A _{en} = | -285 lb/in ² | | OVERTURNI | NG |
| Design tensile stress | | F_t ' = $F_t \times C_D$ | $\times C_{\text{Mt}} \times C_{\text{tt}} \times C_{\text{F}}$ | t × Ci = 1508 lb/ | in² | |
| | | ft / Ft' = -0.1 | 89 | | | |
| | | PASS - Des | sign tensile str | ess exceeds m | aximum applie | d tensile s |
| Load combination 1 | | | | | | |
| Shear force for maximum compre | ession | $V = 0.6 \times W$ | • | | | |
| Axial force for maximum com | pression | P = ((D + S | $S_{wt} \times h)) \times s / 2$ | + $0.6 \times W_{ch2}$ = | = 1.49 kips | |
| Maximum compressive force in c | hord | $C = V \times h / $ | $((C_o \times \Sigma L_i)) + P =$ | 3.369 kips | | |
| Maximum applied compressive s | tress | $f_c = C / A_e =$ | 204 lb/in ² | | | |
| Design compressive stress | | $F_c' = F_c \times C_c$ | $C \times C_{Mc} \times C_{tc} \times C_{tc}$ | $F_{\rm Fc} \times C_{\rm i} \times C_{\rm P} = 8$ | 18 lb/in² | |
| | | f _c / F _c ' = 0.2 | 50 | | | |
| | _ | Design compres | | | | |



Uniform shear applied to wall Shear resisted by wall segments Maximum force in collector
$$\begin{split} V_{max} &= V_{w_max} = \textbf{3.6 kips} \\ v_a &= V_{max} \ / \ ((C_o \times \Sigma L_i)) = \textbf{183.2 plf} \\ v_b &= v_a \times b \ / \ (b_1 + b_2) = \textbf{230.1 plf} \\ P_{coll} &= \textbf{0.75 kips} \end{split}$$

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| Maximum applied tensile stress | | $f_t = P_{coll} / (2)$ | × As) = 45 lb/in ² | 2 | WE | ST, LEVEL | | |
| Design tensile stress | | F_t = $F_t \times C_D$ | \times C _{Mt} \times C _{tt} \times C _F | t × Ci = 1508 I | b/in² | | | |
| | | ft / Ft' = 0.03 | 0 | | | | | |
| | | PASS - Design tensile stress exceeds maximum applied tensile stre | | | | | | |
| Maximum applied compressive s | tress | $f_c = P_{coll} / (2 \times A_s) = 45 \text{ lb/in}^2$ | | | | | | |
| Column stability factor | | C _P = 1.00 | | | | | | |
| Design compressive stress | | $F_c' = F_c \times C_c$ | $0 \times C_{Mc} \times C_{tc} \times C_{tc}$ | $C_{Fc} \times C_i \times C_P =$ | 2376 lb/in ² | | | |
| | | f _c / F _c ' = 0.019 | | | | | | |
| | PASS - | Design compres | sive stress ex | ceeds maxim | num applied con | pressive stre | | |
| Wind load deflection | | | | | | | | |
| Design shear force | | $V_{\delta w} = f_{Wserv}$ | \times W = 6 kips | | | | | |
| Deflection limit | | Δ_{w_allow} = h / | 500 = 0.246 ir | n | | | | |
| Induced unit shear | | | | | | | | |
| 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(abs(W | ch1),abs(Wch2))) | = 0.000 kips | | | | |
| Shear wall deflection - Eqn. 4.3- | 1 | δ_{sww} = 2 \times V $_{\delta}$ | $_{ m w_max} 	imes h^3$ / (3 $	imes$ | $E \times A_e \times \Sigma L_i) \cdot \\$ | + $v_{\delta w_{max}} \times h$ / (Ga |) + h \times T $_{\delta}$ / (ka | | |
| | | ΣL _i) = 0.201 | in | | | | | |
| | | $\delta_{ m sww}$ / $\Delta_{ m w_allow}$ | = 0.815 | | | | | |

PASS - Shear wall deflection is less than deflection limit



Panel construction

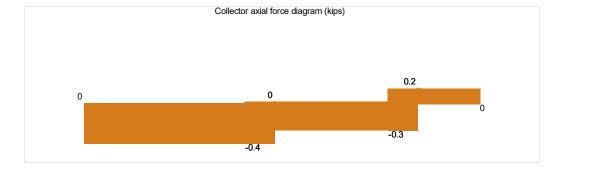
Nominal stud size Dressed stud size

2" x 6" 1.5" x 5.5"

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| | | Calc. by S | Date 8/1/2022 | Chk'd by | Date | App'd by | Date |
| Cross-sectional area o | of studs | | A _s = 8.25 ir | ו ² | | N | /EST, LEVE |
| 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 o | of end posts | ; | A _e = 16.5 in | 1 ² | | | |
| Hole diameter | | | Dia = 1 in | | | | |
| Net cross-sectional are | ea of end p | osts | 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 o | | | | |
| Vertical anchor stiffnes | SS | | k _a = 30000 | lb/in | | | |
| From NDS Suppleme | nt Table 4 | A - Reference | design values | for visually g | raded dimens | ion lumber (2' | " - 4" thick) |
| Species, grade and size | ze classifica | ation | Hem-Fir, n | o.1 & btr grade | e, 2'' & wider | | |
| Specific gravity | | | G = 0.43 | | | | |
| Tension parallel to gra | in | | Ft = 725 lb/ | 'in² | | | |
| Compression parallel | to grain | | F _c = 1350 l | b/in² | | | |
| Modulus of elasticity | | | E = 15000 |)0 lb/in ² | | | |
| Minimum modulus of e | elasticity | | E _{min} = 5500 | 00 lb/in ² | | | |
| Sheathing details | | | | | | | |
| Sheathing material | | | 7/16" woo | d panel struc | tural I oriented | d strandboard | sheathing |
| Fastener type | | | 8d commo | on nails at 4" | centers | | |
| From SDPWS Table 4 | 4.3A Nomir | nal Unit Shear | Capacities for | Wood-Frame | e Shear Walls · | - Wood-based | Panels |
| Nominal unit shear o | apacity fo | r seismic des | ign v₅ = min(8 | 60 plf × min[| 1 - (0.5 - G), 1 |], 1740 plf) = | 799.8 lb/ft |
| Nominal unit shear o | apacity fo | r wind design | v _w = min(1 | 205 plf \times mir | n[1 - (0.5 - G), | 1], 2435 plf) | = 1120.6 lb/ft |
| Apparent shear wall | shear stiff | ness | G _a = 21 ki | ps/in | | | |
| Loading details | | | | | | | |
| Dead load acting on to | op of panel | | D = 980 lb/ | ft | | | |
| | op of panel | | S = 800 lb/ | ft | | | |
| Snow load acting on to | | | S _{wt} = 10 lb/ | ft ² | | | |
| Self weight of panel | | of nonal | W = 8200 | bs | | | |
| Self weight of panel In plane wind load acti | • | or panel | | | | | |
| Self weight of panel | • | or paner | f _{Wserv} = 1.0 | | | | |
| Self weight of panel In plane wind load acti Wind load serviceabili Chord forces from sh | ty factor near walls | above | f _{Wserv} = 1.0 |) | 1 | C | |
| Self weight of panel In plane wind load acti Wind load serviceabili Chord forces from sh hord W _{ch[1]} (lbs) E | ty factor near walls a _{q_ch[i]} (Ibs) | above Dc_ch[i] (Ibs) | f _{Wserv} = 1.00 D _{T_ch[i]} (Ibs) |) L _{f_ch[i]} (Ibs) | L _{r_ch[i]} (Ibs) | Sch[i] (Ibs) | R _{ch[i]} (Ibs) |
| Self weight of panel In plane wind load acti Wind load serviceabili Chord forces from st hord Wchtil (Ibs) E. Ch1 -4418; | ty factor near walls a q_ch[i] (lbs) 0; | above Dc_ch[i] (Ibs) 0; | f _{Wserv} = 1.00 Dτ_ch[i] (Ibs) 0; |) L _{f_ch[i]} (Ibs) 0; | 0; | 0; | 0; |
| Self weight of panelIn plane wind load actiWind load serviceabilityChord forces from sthordWch[1] (lbs)Eh1-4418;Ch24418; | ty factor near walls a q_ch[] (lbs) 0; 0; | above Dc_ch[i] (Ibs) 0; 0; | f _{Wserv} = 1.00 DT_ch[i] (Ibs) 0; 0; | D Lf_ch[i] (Ibs) 0; 0; | 0; 0; | | |
| Self weight of panel In plane wind load acti Wind load serviceabilit Chord forces from st hord Weht (1bs) E Ch1 -4418; Ch2 4418; From IBC 2021 cl.160 | ty factor near walls a n_ch[i] (lbs) 0; 0; 0; 0; 0; | above Dc_ch[i] (Ibs) 0; 0; | f _{Wserv} = 1.00 D _{T_ch[i]} (Ibs) 0; 0; tions from AS0 | D Lf_ch[i] (Ibs) 0; 0; | 0; 0; | 0; | 0; |
| Self weight of panel In plane wind load acti Wind load serviceability Chord forces from st hord Wch[1] (lbs) Eh1 -4418; Ch2 4418; From IBC 2021 cl.160 Load combination no.7 | ty factor near walls : a_ch[i] (lbs) 0; 0; 0; 0; 05.1 Basic | above Dc_ch[i] (Ibs) 0; 0; | f _{Wserv} = 1.0 D _{T_ch[i]} (Ibs) 0; 0; tions from ASC D + 0.6W | D Lf_ch[i] (Ibs) 0; 0; | 0; 0; | 0; | 0; |
| Self weight of panel In plane wind load acti Wind load serviceabilit Chord forces from sh hord Wch[1] (Ibs) E Ch1 -4418; Ch2 4418; From IBC 2021 cl.160 Load combination no.2 | ty factor near walls a <u>a_ch[]</u> (Ibs) 0; 0; 0; 0; 0; 0; 0; 0; 0; 0; | above Dc_ch[i] (Ibs) 0; 0; | f _{Wserv} = 1.0 DT_ch[i] (lbs) 0; 0; tions from ASC D + 0.6W D + 0.7E | D L _{f_ch[i]} (Ibs) 0; 0; CE 7, section | 0; 0; 2.4 | 0; | 0; |
| Self weight of panel In plane wind load acti Wind load serviceabilit Chord forces from sh hord W _{ch[1]} (Ibs) E. Ch1 -4418; Ch2 4418; From IBC 2021 cl.160 Load combination no.2 Load combination no.3 | ty factor near walls a n_ch[i] (Ibs) 0; 0; 0; 0; 0; 0; 0; 0; 0; 0; | above Dc_ch[i] (Ibs) 0; 0; | f _{Wserv} = 1.0 D _{T_ch[i]} (Ibs) 0; 0; tions from ASC D + 0.6W D + 0.7E D + 0.75Lf | D Lf_ch[i] (Ibs) 0; 0; CE 7, section + 0.45W + 0.7 | 0; 0; 2.4 5(Lr or S or R) | 0; | 0; |
| Self weight of panel In plane wind load acti Wind load serviceabilit Chord forces from st hord Wch[i] (Ibs) E Ch1 -4418; Ch2 4418; From IBC 2021 cl.160 Load combination no.2 | ty factor near walls : a_ch[i] (Ibs) 0; 0; 0; 0; 0; 0; 0; 0; 0; 0; | above Dc_ch[i] (Ibs) 0; 0; | f _{Wserv} = 1.0 D _{T_ch[i]} (Ibs) 0; 0; tions from ASC D + 0.6W D + 0.7E D + 0.75Lf | L _{f_ch[i]} (Ibs) 0; 0; CE 7, section + 0.45W + 0.7 + 0.525E + 0.7 | 0; 0; 2.4 5(Lr or S or R) | 0; | 0; |

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| | S | 8/1/2022 | | | | |
| Adjustment factors | | | | | WES | ST, LEVE |
| Load duration factor - Table 2.3.2 | 2 | C _D = 1.60 | | | | |
| Size factor for tension – Table 4A | | C _{Ft} = 1.30 | | | | |
| Size factor for compression – Tab | ole 4A | C _{Fc} = 1.10 | | | | |
| Wet service factor for tension – T | able 4A | C _{Mt} = 1.00 | | | | |
| Wet service factor for compression | | C _{Mc} = 1.00 | | | | |
| Wet service factor for modulus of | f elasticity – Tal | ble 4A | | | | |
| | | C _{ME} = 1.00 | | | | |
| Temperature factor for tension – Temperature factor for compress | | C _{tt} = 1.00 .3 | | | | |
| | | C _{tc} = 1.00 | | | | |
| Temperature factor for modulus of | of elasticity – Ta | | | | | |
| | | C _{tE} = 1.00 | | | | |
| Incising factor – cl.4.3.8 | | C _i = 1.00 | | | | |
| Buckling stiffness factor – cl.4.4.2 | 2 | C⊤ = 1.00 | | | | |
| Adjusted modulus of elasticity | | E_{min} ' = $E_{min} \times$ | $C_{\text{ME}} 	imes C_{\text{tE}} 	imes C_{\text{i}}$ | × C _T = 550000 | psi | |
| Critical buckling design value | | F_{cE} = 0.822 × | < E _{min} ' / (h / d) ² | = 904 psi | | |
| Reference compression design v | alue | $F_{c}^{*} = F_{c} \times C_{D}$ | imes C _{Mc} $	imes$ C _{tc} $	imes$ C | C _{Fc} × C _i = 2376 | psi | |
| For sawn lumber | | c = 0.8 | | | | |
| Column stability factor - eqn.3 | 8.7-1 | C _P = (1 + (F | _{сЕ} / F _c *)) / (2 > | < c) – √([(1 + | (F _{cE} / F _c *)) / (2 > | < c)] ² - (F _{cE} / |
| | | | <u> </u> | | | |
| | | F_{c}^{*}) / c) = 0 . | <i>,,</i> , , , | | | |
| From SDPWS Table 4.3.4 Maxir | num Shear Wa | $F_{c}^{*}) / c) = 0.$ | 34 | , u | | |
| From SDPWS Table 4.3.4 Maxir Maximum shear wall aspect ratio | | $F_{c}^{*}) / c) = 0.$ | 34 | | | |
| | | F_{c}^{*}) / c) = 0. all Aspect Ratios | 34 | , | | |
| Maximum shear wall aspect ratio | | $F_c^*) / c) = 0.$ all Aspect Ratios 3.5 | 34 | | | |
| Maximum shear wall aspect ratio Perforated wall length | | F _c *) / c) = 0. all Aspect Ratios 3.5 b ₁ = 11 ft | 34 | | | |
| Maximum shear wall aspect ratio Perforated wall length Shear wall aspect ratio | | F_{c}^{*}) / c) = 0. all Aspect Ratios 3.5 $b_1 = 11 \text{ ft}$ $h / b_1 = 0.932$ | 34 2 | , | | |
| Maximum shear wall aspect ratio Perforated wall length Shear wall aspect ratio Perforated wall length | | F _c *) / c) = 0. all Aspect Ratios 3.5 b ₁ = 11 ft h / b ₁ = 0.932 b ₂ = 7.75 ft | 34 2 | , | | |
| Maximum shear wall aspect ratio Perforated wall length Shear wall aspect ratio Perforated wall length Shear wall aspect ratio | | F_{c}^{*}) / c) = 0. all Aspect Ratios 3.5 $b_1 = 11$ ft $h / b_1 = 0.932$ $b_2 = 7.75$ ft $h / b_2 = 1.323$ | 34 2 3 | | | |
| Maximum shear wall aspect ratio Perforated wall length Shear wall aspect ratio Perforated wall length Shear wall aspect ratio Perforated wall length | | F_{c}^{*}) / c) = 0. all Aspect Ratios 3.5 $b_1 = 11 \text{ ft}$ $h / b_1 = 0.932$ $b_2 = 7.75 \text{ ft}$ $h / b_2 = 1.323$ $b_3 = 4.25 \text{ ft}$ | 34 2 3 | | | |
| Maximum shear wall aspect ratio Perforated wall length Shear wall aspect ratio Perforated wall length Shear wall aspect ratio Perforated wall length Shear wall aspect ratio | or – cl.4.3.3.5 | $F_c^*) / c) = 0.$ all Aspect Ratios 3.5 $b_1 = 11 \text{ ft}$ $h / b_1 = 0.932$ $b_2 = 7.75 \text{ ft}$ $h / b_2 = 1.323$ $b_3 = 4.25 \text{ ft}$ $h / b_3 = 2.412$ | 34 2 3 | | | |
| Maximum shear wall aspect ratio Perforated wall length Shear wall aspect ratio Perforated wall length Shear wall aspect ratio Perforated wall length Shear wall aspect ratio Shear capacity adjustment fact | :or – cl.4.3.3.5 gths | $F_c^*) / c) = 0.$ all Aspect Ratios 3.5 $b_1 = 11$ ft $h / b_1 = 0.932$ $b_2 = 7.75$ ft $h / b_2 = 1.323$ $b_3 = 4.25$ ft $h / b_3 = 2.412$ $\Sigma L_i = b_1 + b_2$ | 34 2 3 2 | h = 22.274 ft | | |
| Maximum shear wall aspect ratio Perforated wall length Shear wall aspect ratio Perforated wall length Shear wall aspect ratio Perforated wall length Shear wall aspect ratio Shear capacity adjustment fact Sum of perforated shear wall lengt Total length of perforated shear wall | :or – cl.4.3.3.5 gths | $F_c^*) / c) = 0.$ all Aspect Ratios 3.5 $b_1 = 11 \text{ ft}$ $h / b_1 = 0.932$ $b_2 = 7.75 \text{ ft}$ $h / b_2 = 1.323$ $b_3 = 4.25 \text{ ft}$ $h / b_3 = 2.412$ $\Sigma L_i = b_1 + b_2$ $L_{tot} = b_1 + w_{or}$ | 34 2 3 2 4 b b b b b b c b c c c c c c c c | h = 22.274 ft ₁₃ = 27 ft | | |
| Maximum shear wall aspect ratio Perforated wall length Shear wall aspect ratio Perforated wall length Shear wall aspect ratio Perforated wall length Shear wall aspect ratio Shear capacity adjustment fact Sum of perforated shear wall lengt Total length of perforated shear w Total area of openings | t or – cl.4.3.3.5 gths vall | F_{c}^{*}) / c) = 0. all Aspect Ratios 3.5 $b_1 = 11$ ft $h / b_1 = 0.932$ $b_2 = 7.75$ ft $h / b_2 = 1.323$ $b_3 = 4.25$ ft $h / b_3 = 2.412$ $\Sigma L_i = b_1 + b_2$ $L_{tot} = b_1 + w_{o}$ $A_o = w_{o1} \times h_o$ | 34 34 2 3 2 4 $b_3 \times 2 \times b_8 / 1$ $b_1 + b_2 + w_{02} + b_1$ $b_1 + w_{02} \times h_{02} = 1$ | h = 22.274 ft ₁₃ = 27 ft 20 ft ² | | |
| Maximum shear wall aspect ratio Perforated wall length Shear wall aspect ratio Perforated wall length Shear wall aspect ratio Perforated wall length Shear wall aspect ratio Shear capacity adjustment fact Sum of perforated shear wall lengt Total length of perforated shear w Total area of openings Sheathing area ratio (eqn. 4.3-6) | c or – cl.4.3.3.5 gths vall | $F_{c}^{*}) / c) = 0.$ all Aspect Ratios 3.5 $b_1 = 11 \text{ ft}$ $h / b_1 = 0.932$ $b_2 = 7.75 \text{ ft}$ $h / b_2 = 1.323$ $b_3 = 4.25 \text{ ft}$ $h / b_3 = 2.412$ $\Sigma L_i = b_1 + b_2$ $L_{tot} = b_1 + w_0$ $A_0 = w_{01} \times h_0$ $r = 1 / (1 + A_0)$ | 34 2 3 2 4 b b b b c b c c c c c c c c | h = 22.274 ft ₁₃ = 27 ft 20 ft ² | | |
| Maximum shear wall aspect ratio Perforated wall length Shear wall aspect ratio Perforated wall length Shear wall aspect ratio Perforated wall length Shear wall aspect ratio Shear capacity adjustment fact Sum of perforated shear wall lengt Total length of perforated shear w Total area of openings Sheathing area ratio (eqn. 4.3-6) Shear capacity adjustment factor | c or – cl.4.3.3.5 gths vall | F_{c}^{*}) / c) = 0. all Aspect Ratios 3.5 $b_1 = 11$ ft $h / b_1 = 0.932$ $b_2 = 7.75$ ft $h / b_2 = 1.323$ $b_3 = 4.25$ ft $h / b_3 = 2.412$ $\Sigma L_i = b_1 + b_2$ $L_{tot} = b_1 + w_{o}$ $A_o = w_{o1} \times h_o$ | 34 34 2 3 2 4 $b_3 \times 2 \times b_8 / 1$ $b_1 + b_2 + w_{02} + b_1$ $b_1 + w_{02} \times h_{02} = 1$ | h = 22.274 ft ₁₃ = 27 ft 20 ft ² | | |
| Maximum shear wall aspect ratio Perforated wall length Shear wall aspect ratio Perforated wall length Shear wall aspect ratio Perforated wall length Shear wall aspect ratio Shear capacity adjustment fact Sum of perforated shear wall lengt Total length of perforated shear w Total area of openings Sheathing area ratio (eqn. 4.3-6) Shear capacity adjustment factor Perforated shear wall capacity | c or – cl.4.3.3.5 gths vall (eqn. 4.3-5) | $F_{c}^{*}) / c) = 0.$ all Aspect Ratios 3.5 b ₁ = 11 ft h / b ₁ = 0.932 b ₂ = 7.75 ft h / b ₂ = 1.323 b ₃ = 4.25 ft h / b ₃ = 2.412 $\Sigma L_{i} = b_{1} + b_{2}$ $L_{tot} = b_{1} + w_{o}$ $A_{o} = w_{o1} \times h_{o}$ $r = 1 / (1 + A_{o})$ $C_{o} = 0.96$ | 34 34 34 3 2 3 2 3 4 4 $b_3 \times 2 \times b_5 / 1$ $a_1 + b_2 + w_{02} + b_1$ $a_1 + w_{02} \times h_{02} = 1$ $b_1 / (h \times \Sigma L_i)) = 0$ | h = 22.274 ft 3 = 27 ft 20 ft ² .919 | | |
| Maximum shear wall aspect ratio Perforated wall length Shear wall aspect ratio Perforated wall length Shear wall aspect ratio Perforated wall length Shear wall aspect ratio Shear capacity adjustment fact Sum of perforated shear wall length Total length of perforated shear w Total area of openings Sheathing area ratio (eqn. 4.3-6) Shear capacity adjustment factor Perforated shear wall capacity Maximum shear force under w | t or – cl.4.3.3.5 gths vall (eqn. 4.3-5) <i>v</i> ind loading | $F_{c}^{*}) / c) = 0.$ all Aspect Ratios 3.5 b ₁ = 11 ft h / b ₁ = 0.932 b ₂ = 7.75 ft h / b ₂ = 1.323 b ₃ = 4.25 ft h / b ₃ = 2.412 $\Sigma L_{i} = b_{1} + b_{2}$ $L_{tot} = b_{1} + w_{0}$ $A_{o} = w_{o1} \times h_{o}$ $r = 1 / (1 + A_{o})$ $C_{o} = 0.96$ $V_{w_max} = 0.6$ | 34 34 2 2 3 2 4 $b_3 \times 2 \times b_8 / 1$ $a_1 + b_2 + w_{02} + b_1$ $a_1 + w_{02} \times h_{02} = 1$ $b_2 / (h \times \Sigma L_i)) = 0$ $\times W = 4.92 \text{ k}$ | h = 22.274 ft ¹³ = 27 ft 20 ft ² .919 ips | | |
| Maximum shear wall aspect ratio Perforated wall length Shear wall aspect ratio Perforated wall length Shear wall aspect ratio Perforated wall length Shear wall aspect ratio Shear capacity adjustment fact Sum of perforated shear wall lengt Total length of perforated shear w Total area of openings Sheathing area ratio (eqn. 4.3-6) Shear capacity adjustment factor Perforated shear wall capacity | t or – cl.4.3.3.5 gths vall (eqn. 4.3-5) <i>v</i> ind loading | $F_{c}^{*}) / c) = 0.$ all Aspect Ratios 3.5 b ₁ = 11 ft h / b ₁ = 0.932 b ₂ = 7.75 ft h / b ₂ = 1.323 b ₃ = 4.25 ft h / b ₃ = 2.412 $\Sigma L_{i} = b_{1} + b_{2}$ $L_{tot} = b_{1} + w_{0}$ $A_{o} = w_{o1} \times h_{o}$ $r = 1 / (1 + A_{o})$ $C_{o} = 0.96$ $V_{w_max} = 0.6$ | 34 34 34 3 2 3 2 3 4 4 $b_3 \times 2 \times b_5 / 1$ $a_1 + b_2 + w_{02} + b_1$ $a_1 + w_{02} \times h_{02} = 1$ $b_1 / (h \times \Sigma L_i)) = 0$ | h = 22.274 ft ¹³ = 27 ft 20 ft ² .919 ips | | |
| Maximum shear wall aspect ratio Perforated wall length Shear wall aspect ratio Perforated wall length Shear wall aspect ratio Perforated wall length Shear wall aspect ratio Shear capacity adjustment fact Sum of perforated shear wall length Total length of perforated shear w Total area of openings Sheathing area ratio (eqn. 4.3-6) Shear capacity adjustment factor Perforated shear wall capacity Maximum shear force under w | t or – cl.4.3.3.5 gths vall (eqn. 4.3-5) <i>v</i> ind loading | F_{c}^{*}) / c) = 0. all Aspect Ratios 3.5 b ₁ = 11 ft h / b ₁ = 0.932 b ₂ = 7.75 ft h / b ₂ = 1.323 b ₃ = 4.25 ft h / b ₃ = 2.412 $\Sigma L_{i} = b_{1} + b_{2}$ $L_{tot} = b_{1} + w_{0}$ $A_{o} = w_{o1} \times h_{o}$ $r = 1 / (1 + A_{c})$ $C_{o} = 0.96$ $V_{w_{max}} = 0.6$ $V_{w_{max}} / V_{w} =$ | 34 34 34 34 34 34 34 34 3 2 3 4 4 4 4 5 4 5 4 5 4 5 4 5 5 5 6 7 1 1 4 5 5 5 6 7 1 1 4 5 5 7 1 1 4 5 5 7 1 1 4 5 5 7 1 1 4 5 5 7 1 1 4 5 7 1 1 4 5 7 1 1 4 5 7 1 1 4 5 7 1 1 4 5 7 1 1 4 5 7 1 1 4 5 7 1 1 4 5 7 1 1 4 5 7 1 1 4 5 7 1 1 4 5 7 1 1 4 5 7 1 1 4 5 7 1 1 4 5 7 1 1 4 5 7 1 1 4 5 7 1 1 1 1 1 1 1 1 1 1 1 1 1 | h = 22.274 ft 3 = 27 ft 20 ft ² .919 ips 1.98 kips | exceeds maxim | um shear fo |
| Maximum shear wall aspect ratio Perforated wall length Shear wall aspect ratio Perforated wall length Shear wall aspect ratio Perforated wall length Shear wall aspect ratio Shear capacity adjustment fact Sum of perforated shear wall length Total length of perforated shear w Total area of openings Sheathing area ratio (eqn. 4.3-6) Shear capacity adjustment factor Perforated shear wall capacity Maximum shear force under w | t or – cl.4.3.3.5 gths vall (eqn. 4.3-5) <i>v</i> ind loading | F_{c}^{*}) / c) = 0. all Aspect Ratios 3.5 b ₁ = 11 ft h / b ₁ = 0.932 b ₂ = 7.75 ft h / b ₂ = 1.323 b ₃ = 4.25 ft h / b ₃ = 2.412 $\Sigma L_{i} = b_{1} + b_{2}$ $L_{tot} = b_{1} + w_{0}$ $A_{o} = w_{o1} \times h_{o}$ $r = 1 / (1 + A_{c})$ $C_{o} = 0.96$ $V_{w_{max}} = 0.6$ $V_{w_{max}} / V_{w} =$ | 34 34 34 34 34 34 34 34 3 2 3 4 4 4 4 5 4 5 4 5 4 5 4 5 5 5 6 7 1 1 4 5 5 5 6 7 1 1 4 5 5 7 1 1 4 5 5 7 1 1 4 5 5 7 1 1 4 5 5 7 1 1 4 5 7 1 1 4 5 7 1 1 4 5 7 1 1 4 5 7 1 1 4 5 7 1 1 4 5 7 1 1 4 5 7 1 1 4 5 7 1 1 4 5 7 1 1 4 5 7 1 1 4 5 7 1 1 4 5 7 1 1 4 5 7 1 1 4 5 7 1 1 4 5 7 1 1 4 5 7 1 1 1 1 1 1 1 1 1 1 1 1 1 | h = 22.274 ft 3 = 27 ft 20 ft ² .919 ips 1.98 kips | exceeds maxim | um shear fo |

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| | Calc. by S | Date 8/1/2022 | Chk'd by | Date | App'd by | Date | |
| Axial force for maximum tensi | on | P = (0.6 × (| D + S _{wt} × h)) > | × b / 2 + 0.6 > | WE Wch1 = 6.117 k | ST, LEVE | |
| Maximum tensile force in chord | | $T = V \times h / (($ | $C_o \times \Sigma L_i$)) - P = | -3.759 kips | | . NO | |
| Maximum applied tensile stress | | ft = T / A _{en} = | -278 lb/in ² | | OVERTURI | NING | |
| Design tensile stress | | $F_t' = F_t \times C_D$ | \times C _{Mt} \times C _{tt} \times C _F | $t \times C_i = 1508$ lk | p/in² | | |
| Ŭ | | f _t / F _t ' = -0.18 | 5 | | | | |
| | | PASS - Desi | ign tensile str | ess exceeds l | maximum applie | ed tensile str | |
| Load combination 1 | | | | | | | |
| Shear force for maximum compre | ession | $V = 0.6 \times W$ | = 4.92 kips | | | | |
| Axial force for maximum comp | pression | P = ((D + S | _{wt} × h)) × s / 2 | + -1 $	imes$ 0.6 $	imes$ | W _{ch1} = 3.372 kip | os | |
| Maximum compressive force in c | hord | C = V × h / ((| $(C_o \times \Sigma L_i)) + P$ | 5.731 kips | | | |
| Maximum applied compressive s | tress | $f_c = C / A_e = 3$ | 347 lb/in ² | | | | |
| Design compressive stress | | $F_c' = F_c \times C_D$ | $	imes C_{Mc} 	imes C_{tc} 	imes C$ | $C_{Fc} \times C_i \times C_P =$ | 818 lb/in ² | | |
| | | f _c / F _c ' = 0.425 | | | | | |
| | PASS - | Design compres | sive stress ex | ceeds maxim | um applied com | pressive str | |
| Chord capacity for chord 2 | | | | | | | |
| | | | | | | | |
| Load combination 5 | | | | | | | |
| Shear force for maximum tension | | $V = 0.6 \times W$ | • | | | | |
| - | | P = (0.6 × (| D + S _{wt} × h)) > | | $0.6	imes W_{ch2}$ = 6.1 | | |
| Shear force for maximum tension Axial force for maximum tensi Maximum tensile force in chord | | P = (0.6 × () T = V × h / ((| $D + S_{wt} \times h)) > C_o \times \Sigma L_i)) - P =$ | | - NEGATIVE. | NO | |
| Shear force for maximum tension Axial force for maximum tensi Maximum tensile force in chord Maximum applied tensile stress | | P = $(0.6 \times (M_{T} = V \times h / ((M_{f_{t}} = T / A_{en} = M_{en})))$ | D + S _{wt} × h)) × C _o × ΣL _i)) - P = -278 lb/in ² | -3.759 kips | NEGATIVE. OVERTURN | NO | |
| Shear force for maximum tension Axial force for maximum tensi Maximum tensile force in chord | | $P = (0.6 \times (10^{-1} \text{ C}) + (0.6 \times \text{ C}))$ $T = V \times \text{ h} / ((10^{-1} \text{ C}) + (0.6 \times \text{ C}))$ $f_t = T / A_{en} = F_t = F_t \times C_D \times C_D$ | D + S _{wt} × h)) > C _o × ΣLi)) - P = - 278 lb/in ² × C _{Mt} × C _{tt} × C _F | -3.759 kips | NEGATIVE. OVERTURN | NO | |
| Shear force for maximum tension Axial force for maximum tensi Maximum tensile force in chord Maximum applied tensile stress | | $P = (0.6 \times (10^{-5} \text{ Cm}) + (0.6 \times 10^{-5} \text{ Cm}) + (0.6 \times 10^{-5}$ | $D + S_{wt} \times h)) > C_o \times \Sigma L_i)) - P =$ -278 lb/in ² $\times C_{Mt} \times C_{tt} \times C_F$ -5 | <mark>-3.759 kips</mark> t × Ci = 1508 l t | NEGATIVE. OVERTURN 0/in ² | NO IING | |
| Shear force for maximum tension Axial force for maximum tensi Maximum tensile force in chord Maximum applied tensile stress Design tensile stress | | $P = (0.6 \times (10^{-5} \text{ Cm}) + (0.6 \times 10^{-5} \text{ Cm}) + (0.6 \times 10^{-5}$ | $D + S_{wt} \times h)) > C_o \times \Sigma L_i)) - P =$ -278 lb/in ² $\times C_{Mt} \times C_{tt} \times C_F$ -5 | <mark>-3.759 kips</mark> t × Ci = 1508 l t | NEGATIVE. OVERTURN | NO IING | |
| Shear force for maximum tension Axial force for maximum tensi Maximum tensile force in chord Maximum applied tensile stress Design tensile stress Load combination 1 | on | $P = (0.6 \times (10^{-5} \text{ Cm}) + (0.6 \times 10^{-5} \text{ Cm}) + (0.6 \times 10^{-5}$ | $D + S_{wt} \times h)) > C_{o} \times \Sigma L_{i})) - P =$ -278 lb/in ² $\times C_{Mt} \times C_{tt} \times C_{F}$ ign tensile stre | <mark>-3.759 kips</mark> t × Ci = 1508 l t | NEGATIVE. OVERTURN D/in ² | NO IING | |
| Shear force for maximum tension Axial force for maximum tensi Maximum tensile force in chord Maximum applied tensile stress Design tensile stress Load combination 1 Shear force for maximum compre | on | $P = (0.6 \times (10^{-5} \times 10^{-5} \times 10$ | $D + S_{wt} \times h)) > C_{o} \times \Sigma L_{i})) - P =$ -278 lb/in ² $\times C_{Mt} \times C_{tt} \times C_{F}$ 5 ign tensile structure = 4.92 kips | - 3.759 kips t × Ci = 1508 lt ess exceeds i | NEGATIVE. OVERTURN D/in ² maximum applie | NO IING | |
| Shear force for maximum tension Axial force for maximum tension Maximum tensile force in chord Maximum applied tensile stress Design tensile stress Load combination 1 Shear force for maximum comprese Axial force for maximum comprese | on ession pression | $P = (0.6 \times (0.5 \times 10^{-5} \text{ Cm})^{-5} $ | $D + S_{wt} \times h)) \times C_{o} \times \Sigma L_{i})) - P =$ -278 lb/in ² $\times C_{Mt} \times C_{tt} \times C_{F}$ ign tensile structure $= 4.92 \text{ kips}$ wt × h)) × s / 2 | -3.759 kips t × Ci = 1508 lk ess exceeds i + 0.6 × Wch2 | NEGATIVE. OVERTURN D/in ² maximum applie | NO IING | |
| Shear force for maximum tension Axial force for maximum tensi Maximum tensile force in chord Maximum applied tensile stress Design tensile stress Load combination 1 Shear force for maximum compressive force in c | on ession pression hord | $P = (0.6 \times (10^{-5} \times 10^{-5} \times 10$ | $D + S_{wt} \times h)) \times C_{o} \times \Sigma L_{i})) - P =$ -278 lb/in ² $\times C_{Mt} \times C_{tt} \times C_{F}$ 5 ign tensile structure = 4.92 kips wt × h)) × s / 2 (C_{o} × \Sigma L_{i})) + P = | -3.759 kips t × Ci = 1508 lk ess exceeds i + 0.6 × Wch2 | NEGATIVE. OVERTURN D/in ² maximum applie | NO IING | |
| Shear force for maximum tension Axial force for maximum tension Maximum tensile force in chord Maximum applied tensile stress Design tensile stress Load combination 1 Shear force for maximum compre- Axial force for maximum compre- Maximum compressive force in c Maximum applied compressive s | on ession pression hord | $P = (0.6 \times (l + 1) + 1) + (0.6 \times l + 1) + (0$ | $D + S_{wt} \times h)) \times C_{o} \times \Sigma L_{i})) - P =$ -278 lb/in ² $\times C_{Mt} \times C_{tt} \times C_{F}$ 5 ign tensile structure = 4.92 kips $Mt \times h)) \times s / 2$ i(C_{o} × \Sigma L_{i})) + P = 347 lb/in ² | -3.759 kips t × Ci = 1508 lk ess exceeds i + 0.6 × W _{ch2} = 5.731 kips | NEGATIVE. OVERTURN b/in ² maximum applie = 3.372 kips | NO IING | |
| Shear force for maximum tension Axial force for maximum tensi Maximum tensile force in chord Maximum applied tensile stress Design tensile stress Load combination 1 Shear force for maximum compressive force in c | on ession pression hord | $P = (0.6 \times (10^{-5} \times 10^{-5} \times 10$ | $D + S_{wt} \times h)) \times C_{o} \times \Sigma L_{i})) - P =$ -278 lb/in ² $\times C_{Mt} \times C_{tt} \times C_{F}$ 5 ign tensile structure = 4.92 kips $Mt \times h)) \times s / 2$ $(C_{o} \times \Sigma L_{i})) + P =$ 347 lb/in ² $\times C_{Mc} \times C_{tc} \times C_{tc} \times C_{tc}$ | -3.759 kips t × Ci = 1508 lk ess exceeds i + 0.6 × W _{ch2} = 5.731 kips | NEGATIVE. OVERTURN b/in ² maximum applie = 3.372 kips | NO IING | |
| Shear force for maximum tension Axial force for maximum tension Maximum tensile force in chord Maximum applied tensile stress Design tensile stress Load combination 1 Shear force for maximum compre- Axial force for maximum compre- Maximum compressive force in c Maximum applied compressive s | on ession pression hord tress | $P = (0.6 \times (1 + 1)^{2})^{2}$ $T = V \times h / ((1 + 1)^{2})^{2}$ $F_{t} = T / A_{en} = F_{t} = T / A_{en} = F_{t} = F_{t} \times C_{D}$ $F_{t} / F_{t} = -0.18$ $PASS - Dest$ $V = 0.6 \times W$ $P = ((D + S_{e})^{2})^{2}$ $C = V \times h / (0 + S_{e})^{2}$ $F_{c} = C / A_{e} = F_{c}$ $F_{c} = F_{c} \times C_{D}$ $f_{c} / F_{c} = 0.42$ | $D + S_{wt} \times h)) \times$ $C_{o} \times \Sigma L_{i})) - P =$ -278 lb/in ² $\times C_{Mt} \times C_{tt} \times C_{F}$ 5 ign tensile structure = 4.92 kips $Mt \times h)) \times s / 2$ $(C_{o} \times \Sigma L_{i})) + P =$ 347 lb/in ² $\times C_{Mc} \times C_{tc} \times C_{T}$ 5 | - 3.759 kips t × Ci = 1508 lk ess exceeds i + 0.6 × W _{ch2} = 5.731 kips C _{Fc} × Ci × CP = | NEGATIVE. OVERTURN b/in ² maximum applie = 3.372 kips 818 lb/in ² | NO NNG | |
| Shear force for maximum tension Axial force for maximum tension Maximum tensile force in chord Maximum applied tensile stress Design tensile stress Load combination 1 Shear force for maximum compre- Axial force for maximum compre- Maximum compressive force in c Maximum applied compressive s | on ession pression hord tress | $P = (0.6 \times (10^{-5} \times 10^{-5} \times 10$ | $D + S_{wt} \times h)) \times$ $C_{o} \times \Sigma L_{i})) - P =$ -278 lb/in ² $\times C_{Mt} \times C_{tt} \times C_{F}$ 5 ign tensile structure = 4.92 kips $Mt \times h)) \times s / 2$ $(C_{o} \times \Sigma L_{i})) + P =$ 347 lb/in ² $\times C_{Mc} \times C_{tc} \times C_{T}$ 5 | - 3.759 kips t × Ci = 1508 lk ess exceeds i + 0.6 × W _{ch2} = 5.731 kips C _{Fc} × Ci × CP = | NEGATIVE. OVERTURN b/in ² maximum applie = 3.372 kips 818 lb/in ² | NO NING | |

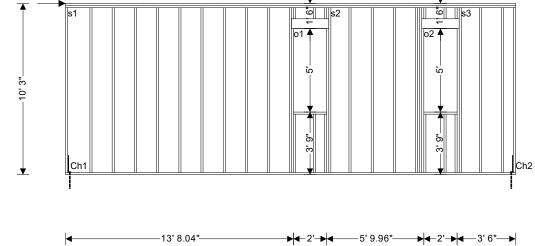


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WEST, LEVEL 1

| Maximum shear force on wall | $V_{max} = V_{w_max} = 4.92$ kips |
|------------------------------------|---|
| Uniform shear applied to wall | $v_a = V_{max} / ((C_o \times \Sigma L_i)) = 230.1 \text{ plf}$ |
| Shear resisted by wall segments | $v_b = v_a \times b / (b_1 + b_2 + b_3) = 270.1 \text{ plf}$ |
| Maximum force in collector | P _{coll} = 0.44 kips |
| Maximum applied tensile stress | $f_t = P_{coll} / (2 \times A_s) = 27 \text{ Ib/in}^2$ |
| Design tensile stress | $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i = \textbf{1508} \ lb/in^2$ |
| | f _t / F _t ' = 0.018 |
| | PASS - Design tensile stress exceeds maximum applied tensile stress |
| Maximum applied compressive stress | $f_c = P_{coll} / (2 \times A_s) = 27 \text{ lb/in}^2$ |
| Column stability factor | C _P = 1.00 |
| Design compressive stress | $F_{c}' = F_{c} \times C_{D} \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_{i} \times C_{P} = 2376 \text{ lb/in}^{2}$ |
| | f _c / F _c ' = 0.011 |
| PASS | - Design compressive stress exceeds maximum applied compressive stress |
| Wind load deflection | |
| Design shear force | $V_{\delta w} = f_{Wserv} \times W = 8.2 \text{ kips}$ |
| Deflection limit | $\Delta_{w_{allow}} = h / 500 = 0.246$ in |
| Induced unit shear | $v_{\delta w_max} = V_{\delta w} / (C_o \times \Sigma L_i) = 383.52 \text{ lb/ft}$ |
| Anchor tension force | T_{δ} = max(0 kips,v _{δw_max} × h - 0.6 × (D + S _{wt} × h) × b / 2 + |
| | max(abs(W _{ch1}),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 L_i) + v_{\delta w_max} \times h / (G_a) + h \times T_\delta / (k_a \times L_i) + v_{\delta w_max} \times h / (G_a) + h \times T_\delta / (k_a \times L_i) + v_{\delta w_max} \times h / (G_a) + h \times T_\delta / (k_a \times L_i) + v_{\delta w_max} \times h / (G_a) + h \times T_\delta / (k_a \times L_i) + v_{\delta w_max} \times h / (G_a) + h \times T_\delta / (k_a \times L_i) + v_{\delta w_max} \times h / (G_a) + h \times T_\delta / (k_a \times L_i) + v_{\delta w_max} \times h / (G_a) + h \times T_\delta / (k_a \times L_i) + v_{\delta w_max} \times h / (G_a) + h \times T_\delta / (k_a \times L_i) + v_{\delta w_max} \times h / (G_a) + h \times T_\delta / (k_a \times L_i) + v_{\delta w_max} \times h / (G_a) + h \times T_\delta / (k_a \times L_i) + v_{\delta w_max} \times h / (G_a) + h \times T_\delta / (k_a \times L_i) + v_{\delta w_max} \times h / (G_a) + h \times T_\delta / (k_a \times L_i) + v_{\delta w_max} \times h / (G_a) + h \times T_\delta / (k_a \times L_i) + v_{\delta w_max} \times h / (G_a) + h \times T_\delta / (k_a \times L_i) + v_{\delta w_max} \times h / (G_a) + h \times T_\delta / (k_a \times L_i) + v_{\delta w_max} \times h / (G_a) + h \times T_\delta / (k_a \times L_i) + v_{\delta w_max} \times h / (G_a) + h \times T_\delta / (k_a \times L_i) + v_{\delta w_max} \times h / (G_a) + h \times T_\delta / (k_a \times L_i) + v_{\delta w_max} \times h / (G_a) + h \times T_\delta / (k_a \times L_i) + v_{\delta w_max} \times h / (G_a) + h \times T_\delta / (k_a \times L_i) + v_{\delta w_max} \times h / (G_a) + h \times T_\delta / (k_a \times L_i) + v_{\delta w_max} \times h / (G_a) + h \times T_\delta / (k_a \times L_i) + v_{\delta w_max} \times h / (G_a) + h \times T_\delta / (k_a \times L_i) + v_{\delta w_max} \times h / (G_a) + h \times T_\delta / (k_a \times L_i) + v_{\delta w_max} \times h / (G_a) + h \times T_\delta / (k_a \times L_i) + v_{\delta w_max} \times h / (G_a) + h \times T_\delta / (k_a \times L_i) + v_{\delta w_max} \times h / (G_a) + h \times T_\delta / (k_a \times L_i) + v_{\delta w_max} \times h / (G_a) + h \times T_\delta / (k_a \times L_i) + v_{\delta w_max} \times h / (G_a) + h \times T_\delta / (k_a \times L_i) + v_{\delta w_max} \times h / (G_a) + h \times T_\delta / (k_a \times L_i) + v_{\delta w_max} \times h / (G_a) + h \times T_\delta / (k_a \times L_i) + v_{\delta w_max} \times h / (G_a) + h \times T_\delta / (k_a \times L_i) + v_{\delta w_max} \times h / (G_a) + h \times T_\delta / (k_a \times L_i) + v_{\delta w_max} \times h / (G_a) + h \times T_\delta / (k_a \times L_i) + v_{\delta w_max} \times h / (G_a) + h \times T_\delta / (k_a \times L_i) + v_{\delta w_max} \times h / (K_a \times L_i) + v_{\delta w_max} \times h / (K_a \times L_i) + v_{\delta w_max} \times h / (K_a \times L_i) + v_{\delta w_max} \times h / (K_a \times L_i) + v_{\delta w_max} \times h / (K_a \times L_i) + v_{\delta w_max} \times h / (K_a \times L$ |
| | ΣL _i) = 0.193 in |
| | $\delta_{sww} / \Delta_{w_allow} = 0.785$ |
| | PASS - Shear wall deflection is less than deflection limit |
| | |

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| In accordance with NDS2018 | allowable stre | ess design and | I the perforat | ed shear wall | method | | |
| Design summary | | | | | | Tedds calcu | lation version 1. |
| Design summary Description | Unit | Provided | Required | Utilization | Result | Tedds calcu | llation version 1 |
| | lbs | 7776 | 1080 | 0.139 | Result PASS | Tedds calcu | llation version 1 |
| Description | | | | | | Tedds calcu | llation version 1 |
| Description Shear capacity | lbs | 7776 | 1080 | 0.139 | PASS | Tedds calcu | Ilation version 1 |
| Description Shear capacity Chord capacity | lbs lb/in² | 7776 818 | 1080 84 | 0.139 0.103 | PASS PASS | Tedds calcu | Ilation version 1 |
| Description Shear capacity Chord capacity Collector capacity | lbs lb/in ² lb/in ² in | 7776 818 1508 0.246 | 1080 84 7 0.057 | 0.139 0.103 0.005 | PASS PASS PASS | Tedds calcu | Ilation version 1 |
| Description Shear capacity Chord capacity Collector capacity Deflection Panel details | lbs lb/in ² lb/in ² in | 7776 818 1508 0.246 | 1080 84 7 0.057 | 0.139 0.103 0.005 | PASS PASS PASS | Tedds calcu | ilation version 1 |
| Description Shear capacity Chord capacity Collector capacity Deflection Panel details Structural I wood panel sheathir | lbs lb/in ² lb/in ² in | 7776 818 1508 0.246 | 1080 84 7 0.057 | 0.139 0.103 0.005 | PASS PASS PASS | Tedds calcu | ilation version 1 |



| Panel opening of | letails | |
|------------------|---------|--|
|------------------|---------|--|

| Width of opening | w _{o1} = 2 ft |
|--|--|
| Height of opening | h _{o1} = 5 ft |
| Height to underside of lintel over opening | I _{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" |
| | |

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| Cross-sectional area of studs | | A _s = 8.25 in ² | | | EAS | T, LEVE |
| 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 | 3 | A _e = 8.25 in ² | | | | |
| Hole diameter | | Dia = 1 in | | | | |
| Net cross-sectional area of end p | osts | 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 | ess | | | |
| Vertical anchor stiffness | | k _a = 30000 lb | /in | | | |
| From NDS Supplement Table 4 | A - Reference | design values fo | or visually gra | ided dimensio | on lumber (2" - 4 | ' thick) |
| Species, grade and size classification | ation | Hem-Fir, no. | l & btr grade, 2 | 2" & wider | | |
| Specific gravity | | G = 0.43 | | | | |
| Tension parallel to grain | | Ft = 725 lb/in | 2 | | | |
| Compression parallel to grain | | F _c = 1350 lb/ | n² | | | |
| Modulus of elasticity | | E = 1500000 | lb/in ² | | | |
| Minimum modulus of elasticity | | E _{min} = 55000 |) lb/in² | | | |
| Sheathing details | | | | | | |
| Sheathing material | | | | | strandboard she | eathing |
| Fastener type | | 8d common | nails at 6"ce | enters | | |
| From SDPWS Table 4.3A Nomin | nal Unit Shear | r Capacities for V | /ood-Frame S | Shear Walls - V | Nood-based Par | nels |
| Nominal unit shear capacity fo | r seismic des | sign v _s = min(560 |) plf $	imes$ min[1 - | - (0.5 - G), 1], | 1740 plf) = 520 | .8 lb/ft |
| Nominal unit shear capacity fo | r wind desigr | $v_w = min(78)$ | 5 plf $	imes$ min[1 \cdot | - (0.5 - G), 1], | , 2435 plf) = 730 | .1 lb/ft |
| Apparent shear wall shear stift | ness | 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 | i | | | |
| | | | | | | |
| Wind load serviceability factor | | f _{Wserv} = 1.00 | | | | |
| | load combina | itions from ASCE | 7, section 2.4 | 4 | | |
| Wind load serviceability factor From IBC 2021 cl.1605.1 Basic Load combination no.1 | load combina | tions from ASCE D + 0.6W | 7, section 2.4 | 4 | | |
| Wind load serviceability factor From IBC 2021 cl.1605.1 Basic Load combination no.1 Load combination no.2 | load combina | tions from ASCE D + 0.6W D + 0.7E | | | | |
| Wind load serviceability factor From IBC 2021 cl.1605.1 Basic Load combination no.1 | load combina | tions from ASCE D + 0.6W D + 0.7E D + 0.75L _f + | 0.45W + 0.75(| L _r or S or R) | | |
| Wind load serviceability factor From IBC 2021 cl.1605.1 Basic Load combination no.1 Load combination no.2 | load combina | tions from ASCE D + 0.6W D + 0.7E D + 0.75L _f + | | L _r or S or R) | | |
| Wind load serviceability factor From IBC 2021 cl.1605.1 Basic Load combination no.1 Load combination no.2 Load combination no.3 | load combina | tions from ASCE D + 0.6W D + 0.7E D + 0.75L _f + | 0.45W + 0.75(| L _r or S or R) | | |
| Wind load serviceability factor From IBC 2021 cl.1605.1 Basic Load combination no.1 Load combination no.2 Load combination no.3 Load combination no.4 | load combina | tions from ASCE D + 0.6W D + 0.7E D + 0.75Lf + D + 0.75Lf + | 0.45W + 0.75(| L _r or S or R) | | |
| Wind load serviceability factor From IBC 2021 cl.1605.1 Basic Load combination no.1 Load combination no.2 Load combination no.3 Load combination no.4 Load combination no.5 Load combination no.6 Adjustment factors | | tions from ASCE D + 0.6W D + 0.7E $D + 0.75L_{f} + 0.75L_{f} + 0.6D + 0.6W$ 0.6D + 0.6W 0.6D + 0.7E | 0.45W + 0.75(| L _r or S or R) | | |
| Wind load serviceability factor From IBC 2021 cl.1605.1 Basic Load combination no.1 Load combination no.2 Load combination no.3 Load combination no.4 Load combination no.5 Load combination no.6 Adjustment factors Load duration factor – Table 2.3.2 | 2 | tions from ASCE D + 0.6W D + 0.7E $D + 0.75L_f + 0.6D + 0.75L_f + 0.6D + 0.6W$ 0.6D + 0.6W 0.6D + 0.7E $C_D = 1.60$ | 0.45W + 0.75(| L _r or S or R) | | |
| Wind load serviceability factor From IBC 2021 cl.1605.1 Basic Load combination no.1 Load combination no.2 Load combination no.3 Load combination no.4 Load combination no.5 Load combination no.6 Adjustment factors | 2 | tions from ASCE D + 0.6W D + 0.7E $D + 0.75L_{f} + 0.75L_{f} + 0.6D + 0.6W$ 0.6D + 0.6W 0.6D + 0.7E | 0.45W + 0.75(| L _r or S or R) | | |

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| Wet service factor for tension – Ta | able 4A | C _{Mt} = 1.00 | | | EAS | T, LEVEL | | | |
| Wet service factor for compression | on – Table 4A | C _{Mc} = 1.00 | | | | | | | |
| Wet service factor for modulus of | elasticity – Ta | ble 4A | | | | | | | |
| | | C _{ME} = 1.00 | | | | | | | |
| Temperature factor for tension – | | C _{tt} = 1.00 | | | | | | | |
| Temperature factor for compressi | on – Table 2.3 | | | | | | | | |
| | | C _{tc} = 1.00 | | | | | | | |
| Temperature factor for modulus of | f elasticity – T | | | | | | | | |
| | | C _{tE} = 1.00 | | | | | | | |
| Incising factor – cl.4.3.8 | | C _i = 1.00 | | | | | | | |
| Buckling stiffness factor – cl.4.4.2 | | C⊤ = 1.00 | | ~ | | | | | |
| Adjusted modulus of elasticity | | | | < Ст = 550000 р | SI | | | | |
| Critical buckling design value | | F_{cE} = 0.822 \times | $E_{min}' / (h / d)^2 =$ | = 904 psi | | | | | |
| Reference compression design va | alue | $F_{c}^{*} = F_{c} \times C_{D}$ | $\times C_{Mc} \times C_{tc} \times C_{tc}$ | _{Fc} × Ci = 2376 p | si | | | | |
| For sawn lumber | | c = 0.8 | c = 0.8 | | | | | | |
| Column stability factor - eqn.3 | .7-1 | $C_{P} = (1 + (F_{c}))$ | $C_{P} = (1 + (F_{cE} / F_{c}^{*})) / (2 \times c) - \sqrt{([(1 + (F_{cE} / F_{c}^{*})) / (2 \times c)]^{2} - (F_{cE} / C_{cE})^{2}}$ | | | | | | |
| | | F _c *) / c) = 0.3 | 34 | | | | | | |
| Perforated wall length | | b ₁ = 13.67 ft | | | | | | | |
| Shear wall aspect ratio Perforated wall length | | h / b ₁ = 0.75 b ₂ = 5.83 ft | | | | | | | |
| Shear wall aspect ratio Perforated wall length Shear wall aspect ratio | | h / b ₁ = 0.75 b ₂ = 5.83 ft h / b ₂ = 1.758 | | | | | | | |
| Shear wall aspect ratio Perforated wall length Shear wall aspect ratio Perforated wall length | | h / b ₁ = 0.75 b ₂ = 5.83 ft h / b ₂ = 1.758 b ₃ = 3.5 ft | | | | | | | |
| Shear wall aspect ratio Perforated wall length Shear wall aspect ratio Perforated wall length Shear wall aspect ratio | | h / b ₁ = 0.75 b ₂ = 5.83 ft h / b ₂ = 1.758 b ₃ = 3.5 ft h / b ₃ = 2.929 | | | | | | | |
| Shear wall aspect ratio Perforated wall length Shear wall aspect ratio Perforated wall length Shear wall aspect ratio Shear capacity adjustment fact | | h / b ₁ = 0.75 b ₂ = 5.83 ft h / b ₂ = 1.758 b ₃ = 3.5 ft h / b ₃ = 2.929 | | <i>(</i> | | | | | |
| Shear wall aspect ratio Perforated wall length Shear wall aspect ratio Perforated wall length Shear wall aspect ratio Shear capacity adjustment fact Sum of perforated shear wall length | Iths | h / b ₁ = 0.75 b ₂ = 5.83 ft h / b ₂ = 1.758 b ₃ = 3.5 ft h / b ₃ = 2.929 $\Sigma L_i = b_1 + b_2 + b_3$ | - $b_3 	imes 2 	imes b_s$ / h | | | | | | |
| Shear wall aspect ratio Perforated wall length Shear wall aspect ratio Perforated wall length Shear wall aspect ratio Shear capacity adjustment fact Sum of perforated shear wall length Total length of perforated shear wall | Iths | h / b ₁ = 0.75 b ₂ = 5.83 ft h / b ₂ = 1.758 b ₃ = 3.5 ft h / b ₃ = 2.929 $\Sigma L_i = b_1 + b_2 + L_{tot} = b_1 + w_{o1}$ | - b ₃ × 2 × b _s / h + b ₂ + w _{o2} + b ₃ | a = 27 ft | | | | | |
| Shear wall aspect ratio Perforated wall length Shear wall aspect ratio Perforated wall length Shear wall aspect ratio Shear capacity adjustment fact Sum of perforated shear wall lengt Total length of perforated shear wall | Iths | $h / b_{1} = 0.75$ $b_{2} = 5.83 \text{ ft}$ $h / b_{2} = 1.758$ $b_{3} = 3.5 \text{ ft}$ $h / b_{3} = 2.929$ $\Sigma L_{i} = b_{1} + b_{2} + L_{tot} = b_{1} + w_{o1}$ $A_{o} = w_{o1} \times h_{o1}$ | $b_3 \times 2 \times b_s / h_1 + b_2 + w_{o2} + b_3 + w_{o2} \times h_{o2} = 2$ | s = 27 ft 2 0 ft ² | | | | | |
| Shear wall aspect ratio Perforated wall length Shear wall aspect ratio Perforated wall length Shear wall aspect ratio Shear capacity adjustment fact Sum of perforated shear wall lengt Total length of perforated shear w Total area of openings Sheathing area ratio (eqn. 4.3-6) | iths /all | $h / b_{1} = 0.75$ $b_{2} = 5.83 \text{ ft}$ $h / b_{2} = 1.758$ $b_{3} = 3.5 \text{ ft}$ $h / b_{3} = 2.929$ $\Sigma L_{i} = b_{1} + b_{2} + 1$ $L_{tot} = b_{1} + w_{o1}$ $A_{o} = w_{o1} \times h_{o1}$ $r = 1 / (1 + A_{o})$ | - b ₃ × 2 × b _s / h + b ₂ + w _{o2} + b ₃ | s = 27 ft 2 0 ft ² | | | | | |
| Shear wall aspect ratio Perforated wall length Shear wall aspect ratio Perforated wall length Shear wall aspect ratio Shear capacity adjustment fact Sum of perforated shear wall lengt Total length of perforated shear w Total area of openings Sheathing area ratio (eqn. 4.3-6) Shear capacity adjustment factor | iths /all | $h / b_{1} = 0.75$ $b_{2} = 5.83 \text{ ft}$ $h / b_{2} = 1.758$ $b_{3} = 3.5 \text{ ft}$ $h / b_{3} = 2.929$ $\Sigma L_{i} = b_{1} + b_{2} + L_{tot} = b_{1} + w_{o1}$ $A_{o} = w_{o1} \times h_{o1}$ | $b_3 \times 2 \times b_s / h_1 + b_2 + w_{o2} + b_3 + w_{o2} \times h_{o2} = 2$ | s = 27 ft 2 0 ft ² | | | | | |
| Shear wall aspect ratio Perforated wall length Shear wall aspect ratio Perforated wall length Shear wall aspect ratio Shear capacity adjustment fact Sum of perforated shear wall length Total length of perforated shear wall Sheathing area ratio (eqn. 4.3-6) Shear capacity adjustment factor Perforated shear wall capacity | yths vall (eqn. 4.3-5) | $h / b_{1} = 0.75$ $b_{2} = 5.83 \text{ ft}$ $h / b_{2} = 1.758$ $b_{3} = 3.5 \text{ ft}$ $h / b_{3} = 2.929$ $\Sigma L_{i} = b_{1} + b_{2} + 1$ $L_{tot} = b_{1} + w_{o1}$ $A_{o} = w_{o1} \times h_{o1}$ $r = 1 / (1 + A_{o})$ $C_{o} = 0.973$ | $b_3 \times 2 \times b_s / h$ + b_2 + w_{o2} + b_3 + $w_{o2} \times h_{o2}$ = 2 /($h \times \Sigma L_i$)) = 0 .9 | e = 27 ft 20 ft ² 918 | | | | | |
| Shear wall aspect ratio Perforated wall length Shear wall aspect ratio Perforated wall length Shear wall aspect ratio Shear capacity adjustment fact Sum of perforated shear wall lengt Total length of perforated shear w Total area of openings Sheathing area ratio (eqn. 4.3-6) Shear capacity adjustment factor | yths vall (eqn. 4.3-5) | $h / b_{1} = 0.75$ $b_{2} = 5.83 \text{ ft}$ $h / b_{2} = 1.758$ $b_{3} = 3.5 \text{ ft}$ $h / b_{3} = 2.929$ $\Sigma L_{i} = b_{1} + b_{2} + 1$ $L_{tot} = b_{1} + w_{o1}$ $A_{o} = w_{o1} \times h_{o1}$ $r = 1 / (1 + A_{o})$ $C_{o} = 0.973$ | $b_3 \times 2 \times b_s / h_1 + b_2 + w_{o2} + b_3 + w_{o2} \times h_{o2} = 2$ | e = 27 ft 20 ft ² 918 | | | | | |
| Shear wall aspect ratio Perforated wall length Shear wall aspect ratio Perforated wall length Shear wall aspect ratio Shear capacity adjustment fact Sum of perforated shear wall length Total length of perforated shear wall Sheathing area ratio (eqn. 4.3-6) Shear capacity adjustment factor Perforated shear wall capacity | iths /all (eqn. 4.3-5) ind loading | $h / b_{1} = 0.75$ $b_{2} = 5.83 \text{ ft}$ $h / b_{2} = 1.758$ $b_{3} = 3.5 \text{ ft}$ $h / b_{3} = 2.929$ $\Sigma L_{i} = b_{1} + b_{2} + b_{1}$ $L_{tot} = b_{1} + w_{01}$ $A_{o} = w_{o1} \times h_{o1}$ $r = 1 / (1 + A_{o})$ $C_{o} = 0.973$ $V_{w_{max}} = 0.6$ | $b_3 \times 2 \times b_s / h$ + b_2 + w_{o2} + b_3 + $w_{o2} \times h_{o2}$ = 2 /($h \times \Sigma L_i$)) = 0 .9 | s = 27 ft 10 ft ² 918 ps | | | | | |
| Shear wall aspect ratio Perforated wall length Shear wall aspect ratio Perforated wall length Shear wall aspect ratio Shear capacity adjustment fact Sum of perforated shear wall length Total length of perforated shear wall Total area of openings Sheathing area ratio (eqn. 4.3-6) Shear capacity adjustment factor Perforated shear wall capacity Maximum shear force under wall | iths /all (eqn. 4.3-5) ind loading | h / b ₁ = 0.75 b ₂ = 5.83 ft h / b ₂ = 1.758 b ₃ = 3.5 ft h / b ₃ = 2.929 $\Sigma L_i = b_1 + b_2 + b_1 + b_2 + b_1 + b_2 + b_1 + b_2 + b_1 + b_1 + b_1 + b_2 + b_1 + b_$ | $b_{3} \times 2 \times b_{s} / h$ $b_{2} + w_{o2} + b_{3}$ $+ w_{o2} \times h_{o2} = 2$ $/(h \times \Sigma L_{i})) = 0.4$ $\times W = 1.08 \text{ ki}$ $\times \Sigma L_{i} / 2 = 7.7$ 0.139 | s = 27 ft 20 ft ² 918 ps 776 kips | | | | | |
| Shear wall aspect ratio Perforated wall length Shear wall aspect ratio Perforated wall length Shear wall aspect ratio Shear capacity adjustment fact Sum of perforated shear wall length Total length of perforated shear wall Total area of openings Sheathing area ratio (eqn. 4.3-6) Shear capacity adjustment factor Perforated shear wall capacity Maximum shear force under wall | iths /all (eqn. 4.3-5) ind loading | h / b ₁ = 0.75 b ₂ = 5.83 ft h / b ₂ = 1.758 b ₃ = 3.5 ft h / b ₃ = 2.929 $\Sigma L_i = b_1 + b_2 + b_1 + b_2 + b_1 + b_2 + b_1 + b_2 + b_1 + b_1 + b_1 + b_2 + b_1 + b_$ | $b_{3} \times 2 \times b_{s} / h$ $b_{2} + w_{o2} + b_{3}$ $+ w_{o2} \times h_{o2} = 2$ $/(h \times \Sigma L_{i})) = 0.4$ $\times W = 1.08 \text{ ki}$ $\times \Sigma L_{i} / 2 = 7.7$ 0.139 | s = 27 ft 20 ft ² 918 ps 776 kips | xceeds maximi | um shear fo | | | |
| Shear wall aspect ratio Perforated wall length Shear wall aspect ratio Perforated wall length Shear wall aspect ratio Shear capacity adjustment fact Sum of perforated shear wall length Total length of perforated shear wall Total area of openings Sheathing area ratio (eqn. 4.3-6) Shear capacity adjustment factor Perforated shear wall capacity Maximum shear force under wall | iths /all (eqn. 4.3-5) ind loading | h / b ₁ = 0.75 b ₂ = 5.83 ft h / b ₂ = 1.758 b ₃ = 3.5 ft h / b ₃ = 2.929 $\Sigma L_i = b_1 + b_2 + b_1 + b_2 + b_1 + b_2 + b_1 + b_2 + b_1 + b_1 + b_1 + b_2 + b_1 + b_$ | $b_{3} \times 2 \times b_{s} / h$ $b_{2} + w_{o2} + b_{3}$ $+ w_{o2} \times h_{o2} = 2$ $/(h \times \Sigma L_{i})) = 0.4$ $\times W = 1.08 \text{ ki}$ $\times \Sigma L_{i} / 2 = 7.7$ 0.139 | s = 27 ft 20 ft ² 918 ps 776 kips | xceeds maxim | um shear fo | | | |
| Shear wall aspect ratio Perforated wall length Shear wall aspect ratio Perforated wall length Shear wall aspect ratio Shear capacity adjustment fact Sum of perforated shear wall lengt Total length of perforated shear w Total area of openings Sheathing area ratio (eqn. 4.3-6) Shear capacity adjustment factor Perforated shear wall capacity Maximum shear force under w Shear capacity for wind loading Chord capacity for chords 1 an | yths /all (eqn. 4.3-5) ind loading g | h / b ₁ = 0.75 b ₂ = 5.83 ft h / b ₂ = 1.758 b ₃ = 3.5 ft h / b ₃ = 2.929 $\Sigma L_i = b_1 + b_2 + b_1 + b_2 + b_1 + b_2 + b_1 + b_2 + b_1 + b_1 + b_1 + b_2 + b_1 + b_$ | $b_{3} \times 2 \times b_{s} / h$ + $b_{2} + w_{o2} + b_{3}$ + $w_{o2} \times h_{o2} = 2$ /($h \times \Sigma L_{i}$)) = 0.4 × $W = 1.08$ kig × $\Sigma L_{i} / 2 = 7.7$ 0.139 lear capacity f | s = 27 ft 20 ft ² 918 ps 776 kips | xceeds maximi | um shear foi | | | |
| Shear wall aspect ratio Perforated wall length Shear wall aspect ratio Perforated wall length Shear wall aspect ratio Shear capacity adjustment fact Sum of perforated shear wall lengt Total length of perforated shear w Total area of openings Sheathing area ratio (eqn. 4.3-6) Shear capacity adjustment factor Perforated shear wall capacity Maximum shear force under w Shear capacity for wind loading Chord capacity for chords 1 an Load combination 5 | ths /all (eqn. 4.3-5) ind loading g | h / b ₁ = 0.75 b ₂ = 5.83 ft h / b ₂ = 1.758 b ₃ = 3.5 ft h / b ₃ = 2.929 $\Sigma L_i = b_1 + b_2 + b_1 + b_2 + b_1 + b_0 + b_1 + b_0 + b_1 + b_0 + b_1 + b_0 + b_$ | $b_{3} \times 2 \times b_{s} / h$ + $b_{2} + w_{o2} + b_{3}$ + $w_{o2} \times h_{o2} = 2$ /($h \times \Sigma L_{i}$)) = 0.4 × $W = 1.08 \text{ kig}$ × $\Sigma L_{i} / 2 = 7.7$ 0.139 ear capacity f | s = 27 ft 20 ft ² 918 ps 776 kips | | um shear foi | | | |
| Shear wall aspect ratio Perforated wall length Shear wall aspect ratio Perforated wall length Shear wall aspect ratio Shear capacity adjustment fact Sum of perforated shear wall lengt Total length of perforated shear wall Total area of openings Sheathing area ratio (eqn. 4.3-6) Shear capacity adjustment factor Perforated shear wall capacity Maximum shear force under wa Shear capacity for wind loading Chord capacity for chords 1 and Load combination 5 Shear force for maximum tension | ths /all (eqn. 4.3-5) ind loading g | h / b ₁ = 0.75 b ₂ = 5.83 ft h / b ₂ = 1.758 b ₃ = 3.5 ft h / b ₃ = 2.929 $\Sigma L_i = b_1 + b_2 + 1$ $L_{tot} = b_1 + w_{o1}$ $A_o = w_{o1} \times h_{o1}$ $r = 1 / (1 + A_o)$ $C_o = 0.973$ $V_{w_max} = 0.6$ $V_w = V_w \times C_o$ $V_{w_max} / V_w = 0$ PASS - Sh $V = 0.6 \times W =$ $P = (0.6 \times (D_{w_m}))$ | $b_{3} \times 2 \times b_{s} / h$ + $b_{2} + w_{o2} + b_{3}$ + $w_{o2} \times h_{o2} = 2$ /($h \times \Sigma L_{i}$)) = 0.4 × $W = 1.08 \text{ kig}$ × $\Sigma L_{i} / 2 = 7.7$ 0.139 rear capacity f | e = 27 ft 10 ft ² 918 976 kips 776 kips 776 kips 776 kips | | | | | |
| Shear wall aspect ratio Perforated wall length Shear wall aspect ratio Perforated wall length Shear wall aspect ratio Shear capacity adjustment fact Sum of perforated shear wall lengt Total length of perforated shear wall Total area of openings Sheathing area ratio (eqn. 4.3-6) Shear capacity adjustment factor Perforated shear wall capacity Maximum shear force under wall Shear capacity for wind loading Chord capacity for chords 1 and Load combination 5 Shear force for maximum tension Axial force for maximum tension | ths /all (eqn. 4.3-5) ind loading g | h / b ₁ = 0.75 b ₂ = 5.83 ft h / b ₂ = 1.758 b ₃ = 3.5 ft h / b ₃ = 2.929 $\Sigma L_i = b_1 + b_2 + 1$ $L_{tot} = b_1 + w_{o1}$ $A_o = w_{o1} \times h_{o1}$ $r = 1 / (1 + A_o)$ $C_o = 0.973$ $V_{w_max} = 0.6$ $V_w = V_w \times C_o$ $V_{w_max} / V_w = 0$ PASS - Sh $V = 0.6 \times W =$ $P = (0.6 \times (D_{w_m}))$ | $b_{3} \times 2 \times b_{s} / h$ + $b_{2} + w_{o2} + b_{3}$ + $w_{o2} \times h_{o2} = 2$ /($h \times \Sigma L_{i}$)) = 0.4 × $W = 1.08 \text{ kig}$ × $\Sigma L_{i} / 2 = 7.7$ 0.139 ear capacity f = 1.08 kips () + S _{wt} × h)) × $\Sigma_{o} \times \Sigma L_{i}$)) - P = [| e = 27 ft 10 ft ² 918 976 kips 776 kips 776 kips 776 kips | kips | 10 | | | |

| Anthem Structural Engineers | Project | | Job Ref. | Job Ref. | | | |
|-----------------------------|---------------|------------------|----------|----------|---------------------|------|--|
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$f_t / F_t' = -0.158$

EAST, LEVEL 4

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 1

Shear force for maximum compression Axial force for maximum compression

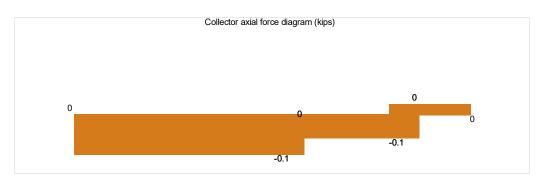
Maximum compressive force in chord Maximum applied compressive stress

Design compressive stress

 $\begin{array}{l} \mathsf{V} = 0.6 \times \mathsf{W} = \textbf{1.08 kips} \\ \mathsf{P} = ((\mathsf{D} + \mathsf{S}_{wt} \times \mathsf{h})) \times \mathsf{s} \ / \ 2 = \textbf{0.175 kips} \\ \mathsf{C} = \mathsf{V} \times \mathsf{h} \ / \ ((\mathsf{C}_o \times \Sigma \mathsf{L}_i)) + \mathsf{P} = \boxed{\textbf{0.695 kips}} \\ \mathsf{f}_c = \mathsf{C} \ / \ \mathsf{A}_e = \textbf{84 lb} / \mathsf{in}^2 \\ \mathsf{F}_c' = \mathsf{F}_c \times \mathsf{C}_\mathsf{D} \times \mathsf{C}_\mathsf{Mc} \times \mathsf{C}_\mathsf{tc} \times \mathsf{C}_\mathsf{Fc} \times \mathsf{C}_\mathsf{i} \times \mathsf{C}_\mathsf{P} = \textbf{818 lb} / \mathsf{in}^2 \\ \mathsf{f}_c \ / \ \mathsf{F}_c' = \textbf{0.103} \end{array}$

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 | $v_b = v_a \times b / (b_1 + b_2 + b_3) = 59.5 \text{ plf}$ |
| Maximum force in collector | P _{coll} = 0.121 kips |
| Maximum applied tensile stress | $f_t = P_{coll} / (2 \times A_s) = 7 \text{ lb/in}^2$ |
| Design tensile stress | $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i = 1508 \text{ lb/in}^2$ |
| | f _t / F _t ' = 0.005 |
| | PASS - Design tensile stress exceeds maximum applied tensile stress |
| Maximum applied compressive stress | $f_c = P_{coll} / (2 \times A_s) = 7 \text{ lb/in}^2$ |
| Column stability factor | C _P = 1.00 |
| Design compressive stress | $F_{c}' = F_{c} \times C_{D} \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_{i} \times C_{P} = 2376 \text{ lb/in}^{2}$ |
| | f _c / F _c ' = 0.003 |
| PASS - | Design compressive stress exceeds maximum applied compressive stress |

Wind load deflection

| Design shear force |
|----------------------|
| Deflection limit |
| Induced unit shear |
| Anchor tension force |

$$\begin{split} V_{\delta w} &= f_{Wserv} \times W = \textbf{1.8 kips} \\ \Delta_{w_allow} &= h \ / \ 500 = \textbf{0.246 in} \\ v_{\delta w_max} &= V_{\delta w} \ / \ (C_o \times \Sigma L_i) = \textbf{84.49 lb/ft} \\ T_\delta &= max(0 \ kips, v_{\delta w_max} \times h \ - \ 0.6 \times (D + S_{wt} \times h) \times b \ / \ 2) = \textbf{0.000 kips} \end{split}$$

| Tekla Tedds Anthem Structural Engineers | Project | | | | Job Ref. | |
|--|---------------|------------------|----------|------|---------------------|------|
| | Section | | | | Sheet no./rev. 5 | |
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Shear wall deflection – Eqn. 4.3-1

 $\delta_{sww} = 2 \times v_{\delta w_max} \times h^3 \ / \ (3 \times E \times A_e \times \Sigma L_i) + v_{\delta w_max} \times h \ F_{\bullet} \times F_{\bullet$

ΣL_i) = **0.057** in

 δ_{sww} / Δ_{w_allow} = 0.231

PASS - Shear wall deflection is less than deflection limit

| Tekla , Tedds | Project | | | | Job | Ref. | |
|--|--------------------|------------------------|---|---------------|----------|-------------|-----------------|
| Anthem Structural Engineers | Section | | | | She 1 | et no./rev. | |
| | Calc. by S | Date 8/1/2022 | Chk'd by | Date | | 'd by | Date |
| WOOD SHEAR WALL DESIGN In accordance with NDS2018 a | <u> </u> | ss design and | I the perforate | ed shear wall | | | ST, LEVE |
| 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 | | |
| Structural I wood panel sheathin Panel height Panel length ↓↓↓↓↓↓↓ | g on one side | h = 10.28 b = 27 ft | D+S | ┟┼┿┽┿┿┿┿┿ | ┟┵┵┵┵┵┵ | | |
| w the second sec | ↓ | | ↓ | | | , i ↓ | |
| s1 | | | 53 → → → → → → → → → → → → → | | | | |

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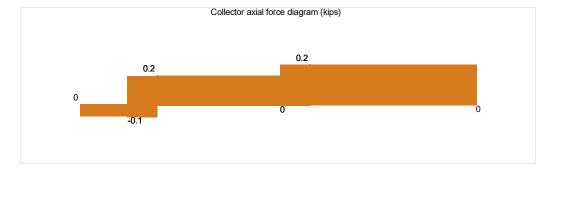
Panel opening detailsWidth of opening $w_{o1} = 2 \text{ ft}$ Height of opening $h_{o1} = 5 \text{ ft}$ Height to underside of lintel over opening $l_{o1} = 8.75 \text{ ft}$ Position of opening $P_{o1} = 3.25 \text{ ft}$ Width of opening $w_{o2} = 2 \text{ ft}$ Height of opening $h_{o2} = 5 \text{ ft}$

| 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" |

| | | | Project | | | | Job Ref. | | | |
|-----------------------|------------------------------------|-------------------|----------------|---|---|-----------------|--------------------------|--------------------------|--|--|
| Anth | nem Structural | Engineers | Section | | | | Sheet no 2 | Sheet no./rev. 2 | | |
| | | | Calc. by S | Date 8/1/2022 | Chk'd by | Date | App'd by | Date | | |
| Cros | s-sectional are | a of studs | | A _s = 8.25 i | n ² | | E | AST, LEVE | | |
| Stud | l spacing | | | s = 16 in | | | | | | |
| | ninal end post s | size | | 2" x 6" | | | | | | |
| Dres | sed end post s | size | | 1.5" x 5.5" | | | | | | |
| Cros | s-sectional are | a of end posts | 6 | A _e = 8.25 i | n² | | | | | |
| Hole | diameter | | | Dia = 1 in | | | | | | |
| Net | cross-sectional | area of end p | osts | A _{en} = 6.75 | in ² | | | | | |
| Nom | ninal collector s | ize | | 2 x 2" x 6" | | | | | | |
| Dres | sed collector s | ize | | 2 x 1.5" x 5 | 5.5" | | | | | |
| Serv | vice condition | | | Dry | | | | | | |
| Tem | perature | | | 100 degF | or less | | | | | |
| Vert | ical anchor stiff | ness | | k _a = 30000 | lb/in | | | | | |
| Fror | n NDS Supple | ment Table 4 | A - Reference | design values | for visually g | raded dimens | ion lumber (2' | " - 4" thick) | | |
| Spe | cies, grade and | l size classifica | ation | Hem-Fir, n | o.2 grade, 2" & | k wider | - | - | | |
| Spe | cific gravity | | | G = 0.43 | - | | | | | |
| Tens | sion parallel to | grain | | Ft = 525 lb | $F_{t} = 525 \text{ lb/in}^{2}$ | | | | | |
| | pression paral | - | | F _c = 1300 | F _c = 1300 lb/in ² | | | | | |
| Mod | ulus of elasticit | y | | E = 1300000 lb/in ² | | | | | | |
| Mini | mum modulus | of elasticity | | E _{min} = 470000 lb/in ² | | | | | | |
| She | athing details | | | | | | | | | |
| She | athing materia | al | | 7/16'' woo | d panel struc | tural I oriente | d strandboard | d sheathing | | |
| Fas | tener type | | | 8d comm | 8d common nails at 6"centers | | | | | |
| Fror | n SDPWS Tab | le 4.3A Nomii | nal Unit Shear | Capacities fo | r Wood-Frame | Shear Walls | - Wood-based | Panels | | |
| | ninal unit shea | | | - | | | | | | |
| | ninal unit shea | | | | | · · · | | | | |
| | | | - | | v_w = min(785 plf × min[1 - (0.5 - G), 1], 2435 plf) = 730.1 lb/ft G _a = 16 kips/in | | | | | |
| | arent shear w | all snear stin | ness | Ga = 16 K | ps/in | | | | | |
| | ding details | | | | | | | | | |
| | d load acting o | | | D = 460 lb | | | | | | |
| | w load acting o | | | S = 800 lb/ | | | | | | |
| | weight of pane | | -f | S _{wt} = 10 lb | | | | | | |
| - | ane wind load a | - | of panel | W = 3800 | | | | | | |
| | d load servicea | - | | f _{Wserv} = 1.0 | U | | | | | |
| | rd forces from | | i | | | | 0 (11-2) | | | |
| nord | W _{ch[i]} (lbs) | Eq_ch[i] (lbs) | Dc_ch[i] (lbs) | DT_ch[i] (Ibs) | L _{f_ch[i]} (lbs) | Lr_ch[i] (Ibs) | S _{ch[i]} (lbs) | R _{ch[i]} (lbs) | | |
| h1 | -697; | 0; | 0; | 0; | 0; | 0; | 0; | 0; | | |
| h2 | 697; | 0; | 0; | 0; | 0; | 0; | 0; | 0; | | |
| | n IBC 2021 cl. | | load combinat | | CE 7, section | 2.4 | | | | |
| | d combination r | | | D + 0.6W | | | | | | |
| | d combination r | | | D + 0.7E | o (=) · · · | | | | | |
| Load combination no.3 | | | | | + 0.45W + 0.7 | , | | | | |
| | Load combination no.4 | | | | D + 0.75L _f + 0.525E + 0.75S | | | | | |
| Load | d combination r d combination r | | | D + 0.75Lf 0.6D + 0.6 | | 55 | | | | |

| Tekla Tedds | Project | | | | Job Ref. | | | | |
|---|---|---|--|---|---------------|--------------|--|--|--|
| Anthem Structural Engineers | Section | | | Sheet no./rev. 3 | | | | | |
| | Calc. by S | Date 8/1/2022 | Chk'd by | Date | App'd by | Date | | | |
| Adjustment factors | | | | | EAS | ST, LEVE | | | |
| Load duration factor – Table 2.3.2 | 2 | C _D = 1.60 | | | | | | | |
| Size factor for tension – Table 4A | | C _{Ft} = 1.30 | | | | | | | |
| Size factor for compression – Tak | ole 4A | C _{Fc} = 1.10 | | | | | | | |
| Wet service factor for tension - T | able 4A | C _{Mt} = 1.00 | | | | | | | |
| Wet service factor for compression | on – Table 4A | C _{Mc} = 1.00 | | | | | | | |
| Wet service factor for modulus of | f elasticity – Tal | ble 4A | | | | | | | |
| | | C _{ME} = 1.00 | | | | | | | |
| Temperature factor for tension – Temperature factor for compress | | C _{tt} = 1.00 .3 | | | | | | | |
| | | C _{tc} = 1.00 | | | | | | | |
| Temperature factor for modulus of | of elasticity – Ta | | | | | | | | |
| | | C _{tE} = 1.00 | | | | | | | |
| Incising factor – cl.4.3.8 | | C _i = 1.00 | | | | | | | |
| Buckling stiffness factor – cl.4.4.2 | 2 | C _T = 1.00 | | | | | | | |
| Adjusted modulus of elasticity | | $\begin{split} &E_{min}\texttt{'} = E_{min} \times C_{ME} \times C_{tE} \times C_{i} \times C_{T} = \textbf{470000} \; psi \\ &F_{cE} = 0.822 \times E_{min}\texttt{'} \; / \; (h \; / \; d)^2 = \textbf{772} \; psi \end{split}$ | | | | | | | |
| Critical buckling design value | | | | | | | | | |
| Reference compression design v | alue | $F_{c}^{*} = F_{c} \times C_{D} \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_{i} = \textbf{2288 psi}$ | | | | | | | |
| For sawn lumber | | c = 0.8 | | | | | | | |
| Column stability factor - eqn.3 | 3.7-1 | C _P = (1 + (F | C_{P} = (1 + (F _{cE} / F _c [*])) / (2 × c) - $\sqrt{([(1 + (F_{cE} / F_{c}^{*})) / (2 × c)]^{2} - (F_{cE} / C_{CE})^{2})}$ | | | | | | |
| | | F _c *) / c) = 0 | .31 | | | | | | |
| From SDPWS Table 4.3.4 Maxii | mum Shear W | all Aspect Ratio | S | | | | | | |
| Maximum shear wall aspect ratio | | 3.5 | | | | | | | |
| Perforated wall length | | b ₁ = 3.25 ft | | | | | | | |
| | | | | | | | | | |
| Shear wall aspect ratio | | h / b₁ = 3.15 | 4 | | | | | | |
| Shear wall aspect ratio Perforated wall length | | h / b ₁ = 3.15 b ₂ = 8.42 ft | 4 | | | | | | |
| | | | | | | | | | |
| Perforated wall length | | b ₂ = 8.42 ft | 7 | | | | | | |
| Perforated wall length Shear wall aspect ratio | | b ₂ = 8.42 ft h / b ₂ = 1.21 | 7 | | | | | | |
| Perforated wall length Shear wall aspect ratio Perforated wall length | or – cl.4.3.3.5 | b ₂ = 8.42 ft h / b ₂ = 1.21 b ₃ = 11.33 ft h / b ₃ = 0.90 | 7 | | | | | | |
| Perforated wall length Shear wall aspect ratio Perforated wall length Shear wall aspect ratio | | b ₂ = 8.42 ft h / b ₂ = 1.21 b ₃ = 11.33 ft h / b ₃ = 0.90 | 7 | ₃ = 21.811 ft | | | | | |
| Perforated wall length Shear wall aspect ratio Perforated wall length Shear wall aspect ratio Shear capacity adjustment fact | gths | $b_2 = 8.42$ ft h / $b_2 = 1.21$ $b_3 = 11.33$ ft h / $b_3 = 0.90$ $\Sigma L_i = b_1 \times 2$ | 7 5 | | | | | | |
| Perforated wall length Shear wall aspect ratio Perforated wall length Shear wall aspect ratio Shear capacity adjustment fact Sum of perforated shear wall length | gths | $b_2 = 8.42 \text{ ft}$ $h / b_2 = 1.21$ $b_3 = 11.33 \text{ ft}$ $h / b_3 = 0.90$ $\Sigma L_i = b_1 \times 2 \times 2$ $L_{tot} = b_1 + w_c$ | 7 5 < b _s / h + b ₂ + b | 93 = 27 ft | | | | | |
| Perforated wall length Shear wall aspect ratio Perforated wall length Shear wall aspect ratio Shear capacity adjustment fact Sum of perforated shear wall length Total length of perforated shear wall | gths vall | $b_2 = 8.42 \text{ ft}$ $h / b_2 = 1.21$ $b_3 = 11.33 \text{ ft}$ $h / b_3 = 0.90$ $\Sigma L_i = b_1 \times 2 \times 2$ $L_{tot} = b_1 + w_{ct}$ $A_0 = w_{o1} \times h_{ct}$ | 7 5 × b _s / h + b ₂ + b 1 + b ₂ + w _{o2} + b | ₂₃ = 27 ft 20 ft ² | | | | | |
| Perforated wall length Shear wall aspect ratio Perforated wall length Shear wall aspect ratio Shear capacity adjustment fact Sum of perforated shear wall length Total length of perforated shear wall Total area of openings | gths vall | $b_2 = 8.42 \text{ ft}$ $h / b_2 = 1.21$ $b_3 = 11.33 \text{ ft}$ $h / b_3 = 0.90$ $\Sigma L_i = b_1 \times 2 \times 2$ $L_{tot} = b_1 + w_{ct}$ $A_0 = w_{o1} \times h_{ct}$ | 7 5 $(x + b_s) / h + b_2 + b_1 + b_2 + w_{02} + b_{01} + w_{02} \times h_{02} = 3$ | ₂₃ = 27 ft 20 ft ² | | | | | |
| Perforated wall length Shear wall aspect ratio Perforated wall length Shear wall aspect ratio Shear capacity adjustment fact Sum of perforated shear wall length Total length of perforated shear wall Total area of openings Sheathing area ratio (eqn. 4.3-6) Shear capacity adjustment factor | gths wall ⁻ (eqn. 4.3-5) | $b_2 = 8.42$ ft $h / b_2 = 1.21$ $b_3 = 11.33$ ft $h / b_3 = 0.90$ $\Sigma L_i = b_1 \times 2 \times$ $L_{tot} = b_1 + w_{ct}$ $A_o = w_{o1} \times h_{ct}$ $r = 1 / (1 + A_{tot})$ | 7 5 $(x + b_s) / h + b_2 + b_1 + b_2 + w_{02} + b_{01} + w_{02} \times h_{02} = 3$ | ₂₃ = 27 ft 20 ft ² | | | | | |
| Perforated wall length Shear wall aspect ratio Perforated wall length Shear wall aspect ratio Shear capacity adjustment fact Sum of perforated shear wall length Total length of perforated shear wall Total area of openings Sheathing area ratio (eqn. 4.3-6) Shear capacity adjustment factor Perforated shear wall capacity | gths vall ⁻ (eqn. 4.3-5) | $b_{2} = 8.42 \text{ ft}$ $h / b_{2} = 1.21$ $b_{3} = 11.33 \text{ ft}$ $h / b_{3} = 0.90$ $\Sigma L_{i} = b_{1} \times 2 \times 2$ $L_{tot} = b_{1} + w_{c}$ $A_{o} = w_{o1} \times h_{c}$ $r = 1 / (1 + A)$ $C_{o} = 0.976$ | 7 5 $x b_s / h + b_2 + b_{11} + b_2 + w_{02} + b_{01} + w_{02} \times h_{02} = 3$ $y_{00} / (h \times \Sigma L_i)) = 0$ | ¹³ = 27 ft 20 ft ² .918 | | | | | |
| Perforated wall length Shear wall aspect ratio Perforated wall length Shear wall aspect ratio Shear capacity adjustment fact Sum of perforated shear wall length Total length of perforated shear wall Total area of openings Sheathing area ratio (eqn. 4.3-6) Shear capacity adjustment factor Perforated shear wall capacity Maximum shear force under wall | gths wall ⁻ (eqn. 4.3-5) vind loading | $b_{2} = 8.42 \text{ ft}$ $h / b_{2} = 1.21$ $b_{3} = 11.33 \text{ ft}$ $h / b_{3} = 0.90$ $\Sigma L_{i} = b_{1} \times 2 \times 2$ $L_{tot} = b_{1} + w_{c}$ $A_{o} = w_{o1} \times h_{c}$ $r = 1 / (1 + A)$ $C_{o} = 0.976$ $V_{w_{max}} = 0.6$ | 7 5 $(b_s / h + b_2 + b_1 + b_2 + w_{02} + b_{01} + w_{02} \times h_{02} = 3$ $(h \times \Sigma L_i) = 0$ $(b \times W = 2.28 \text{ k})$ | n₃ = 27 ft 20 ft² .918 ips | | | | | |
| Perforated wall length Shear wall aspect ratio Perforated wall length Shear wall aspect ratio Shear capacity adjustment fact Sum of perforated shear wall length Total length of perforated shear wall Total area of openings Sheathing area ratio (eqn. 4.3-6) Shear capacity adjustment factor Perforated shear wall capacity | gths wall ⁻ (eqn. 4.3-5) vind loading | $b_{2} = 8.42 \text{ ft}$ $h / b_{2} = 1.21$ $b_{3} = 11.33 \text{ ft}$ $h / b_{3} = 0.90$ $\Sigma L_{i} = b_{1} \times 22$ $L_{tot} = b_{1} + w_{o}$ $A_{o} = w_{o1} \times h_{o}$ $r = 1 / (1 + A)$ $C_{o} = 0.976$ $V_{w} = v_{w} \times C$ | 7 5 $(x b_s / h + b_2 + b_1 + b_2 + w_{02} + b_{01} + w_{02} \times h_{02} = 3$ $(y_{00} / (h \times \Sigma L_i)) = 0$ $(b \times W = 2.28 \text{ k})$ $(b \times \Sigma L_i / 2 = 7.28 \text{ k})$ | n₃ = 27 ft 20 ft² .918 ips | | | | | |
| Perforated wall length Shear wall aspect ratio Perforated wall length Shear wall aspect ratio Shear capacity adjustment fact Sum of perforated shear wall length Total length of perforated shear wall Total area of openings Sheathing area ratio (eqn. 4.3-6) Shear capacity adjustment factor Perforated shear wall capacity Maximum shear force under wall | gths wall ⁻ (eqn. 4.3-5) vind loading | $b_{2} = 8.42 \text{ ft}$ $h / b_{2} = 1.21$ $b_{3} = 11.33 \text{ ft}$ $h / b_{3} = 0.90$ $\Sigma L_{i} = b_{1} \times 2 \times 2$ $L_{tot} = b_{1} + w_{o}$ $A_{o} = w_{o1} \times h_{o}$ $r = 1 / (1 + A)$ $C_{o} = 0.976$ $V_{w_{max}} = 0.6$ $V_{w_{max}} / V_{w} = 0.6$ | 7 5 $(x + b_s / h + b_2 + b_1 + b_2 + w_{02} + b_{01} + w_{02} \times h_{02} = 3$ $(x + b_0 + w_{02} + b_{02} + b_{01} + w_{02} \times h_{02} = 3$ $(x + b_0 + w_{02} + b_{02} + b_$ | n₃ = 27 ft 20 ft² .918 ips 77 kips | exceeds maxin | num shear fo | | | |
| Perforated wall length Shear wall aspect ratio Perforated wall length Shear wall aspect ratio Shear capacity adjustment fact Sum of perforated shear wall length Total length of perforated shear wall Total area of openings Sheathing area ratio (eqn. 4.3-6) Shear capacity adjustment factor Perforated shear wall capacity Maximum shear force under wall | gths wall ⁻ (eqn. 4.3-5) vind loading | $b_{2} = 8.42 \text{ ft}$ $h / b_{2} = 1.21$ $b_{3} = 11.33 \text{ ft}$ $h / b_{3} = 0.90$ $\Sigma L_{i} = b_{1} \times 2 \times 2$ $L_{tot} = b_{1} + w_{o}$ $A_{o} = w_{o1} \times h_{o}$ $r = 1 / (1 + A)$ $C_{o} = 0.976$ $V_{w_{max}} = 0.6$ $V_{w_{max}} / V_{w} = 0.6$ | 7 5 $(x + b_s / h + b_2 + b_1 + b_2 + w_{02} + b_{01} + w_{02} \times h_{02} = 3$ $(x + b_0 + w_{02} + b_{02} + b_{01} + w_{02} \times h_{02} = 3$ $(x + b_0 + w_{02} + b_{02} + b_$ | n₃ = 27 ft 20 ft² .918 ips 77 kips | exceeds maxin | num shear fo | | | |

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| | | | | | 4 | | | |
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| Axial force for maximum tensi | on | P = (0.6 × (| D + S _{wt} × h)) > | × b / 2 + 0.6 > | EA Wch1 = 4.138 | ST, LEVE | | |
| Maximum tensile force in chord | | T = V × h / ((| $C_o \times \Sigma L_i)$ - P = | -3.040 kips | | E. NO | | |
| Maximum applied tensile stress | | $f_t = T / A_{en} =$ | -450 lb/in ² | | OVERTUR | NING | | |
| Design tensile stress | | $F_t' = F_t \times C_D$ | \times C _{Mt} \times C _{tt} \times C _F | t × Ci = 1092 II | b/in ² | | | |
| | | f _t / F _t ' = -0.41 | 2 | | | | | |
| | | PASS - Des | ign tensile str | ess exceeds | maximum appli | ed tensile sti | | |
| Load combination 1 | | | | | | | | |
| Shear force for maximum compre | ession | $V = 0.6 \times W$ | = 2.28 kips | | | | | |
| Axial force for maximum comp | pression | P = ((D + S | _{wt} × h)) × s / 2 | 2 + -1 $	imes$ 0.6 $	imes$ | W _{ch1} = 0.793 ki | ps | | |
| Maximum compressive force in c | hord | $C = V \times h / (0$ | $(C_o \times \Sigma L_i)) + P$ | = 1.891 kips | | | | |
| Maximum applied compressive s | tress | f _c = C / A _e = 229 lb/in ² | | | | | | |
| Design compressive stress | | $F_c' = F_c \times C_D$ | imes C _{Mc} $	imes$ C _{tc} $	imes$ C | $C_{Fc} \times C_i \times C_P =$ | 709 lb/in ² | | | |
| | | f _c / F _c ' = 0.323 | | | | | | |
| | PASS - | Design compres | sive stress ex | ceeds maxim | um applied cor | npressive sti | | |
| Chord capacity for chord 2 | | | | | | | | |
| Load combination 5 | | | | | | | | |
| Shear force for maximum tensior | 1 | V = 0.6 × W = 2.28 kips | | | | | | |
| Axial force for maximum tensi | on | 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 × h / ((C _o × ΣL _i)) - P = -3.040 kips - NEGATIVE. NO | | | | | | |
| Maximum applied tensile stress | | $f_t = T / A_{en} = -450 \text{ lb/in}^2$ OVERTURNING | | | | | | |
| Design tensile stress | | $F_t' = F_t \times C_D$ | $\times C_{Mt} \times C_{tt} \times C_{F}$ | =t × Ci = 1092 Ⅱ | b/in ² | | | |
| | | ft / Ft' = -0.41 | 2 | | | | | |
| | | PASS - Des | ign tensile str | ess exceeds | maximum appli | ed tensile sti | | |
| Load combination 1 | | | | | | | | |
| Shear force for maximum compre | ession | V = 0.6 × W = 2.28 kips | | | | | | |
| Axial force for maximum comp | pression | $P = ((D + S_{wt} \times h)) \times s / 2 + 0.6 \times W_{ch2} = 0.793 \text{ kips}$ | | | | | | |
| Maximum compressive force in c | hord | $C = V \times h / ((C_o \times \Sigma L_i)) + P = 1.891 kips$ | | | | | | |
| Maximum applied compressive s | tress | f _c = C / A _e = 229 lb/in ² | | | | | | |
| | | $F_c' = F_c \times C_D$ | \times C _{Mc} \times C _{tc} \times C | $C_{Fc} \times C_i \times C_P =$ | 709 lb/in ² | | | |
| Design compressive stress | | | | | | | | |
| Design compressive stress | | f _c / F _c ' = 0.32 Design compres | | | | | | |



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EAST, LEVEL 3

| Maximum shear force on wall | V _{max} = V _{w_max} = 2.28 kips |
|------------------------------------|---|
| Uniform shear applied to wall | $v_a = V_{max} / ((C_o \times \Sigma L_i)) = 107.1 \text{ plf}$ |
| Shear resisted by wall segments | $v_b = v_a \times b / (b_1 + b_2 + b_3) = 125.7 \text{ plf}$ |
| Maximum force in collector | P _{coll} = 0.211 kips |
| Maximum applied tensile stress | $f_t = P_{coll} / (2 \times A_s) = 13 \text{ lb/in}^2$ |
| Design tensile stress | $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i = \textbf{1092} \text{ lb/in}^2$ |
| | ft / Ft' = 0.012 |
| | PASS - Design tensile stress exceeds maximum applied tensile stress |
| Maximum applied compressive stress | $f_c = P_{coll} / (2 \times A_s) = 13 \text{ lb/in}^2$ |
| Column stability factor | C _P = 1.00 |
| Design compressive stress | $F_{c}' = F_{c} \times C_{D} \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_{i} \times C_{P} = 2288 \text{ lb/in}^{2}$ |
| | fc / Fc' = 0.006 |
| PASS | S - Design compressive stress exceeds maximum applied compressive stress |
| Wind load deflection | |
| Design shear force | $V_{\delta w} = f_{Wserv} \times W = 3.8 \text{ kips}$ |
| Deflection limit | $\Delta_{w_allow} = h / 500 = 0.246$ in |
| Induced unit shear | $v_{\delta w_max} = V_{\delta w} / (C_o \times \Sigma L_i) = 178.51 \text{ lb/ft}$ |
| Anchor tension force | T_{δ} = max(0 kips,v_{\delta w_{max}} \times h - 0.6 \times (D + S_{wt} \times h) \times b / 2 + |
| | max(abs(W _{ch1}),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 L_b) + k_b \times L_b $ |
| | ΣL _i) = 0.121 in |
| | $\delta_{sww} / \Delta_{w_allow} = 0.492$ |
| | PASS - Shear wall deflection is less than deflection limit |

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| | | | | | | | |
| WOOD SHEAR WALL DESIGN | <u> </u> | ess design and | the perforate | ed shear wall i | nethod | EAS | T, LEVEL 2 |
| In accordance with NDS2018 a | <u> </u> | ess design and | the perforate | ed shear wall ı | nethod | | T, LEVEL 2 |
| | <u> </u> | ess design and | the perforate | ed shear wall ı Utilization | nethod | Tedds calc | |

250

64

0.245

PASS

PASS

PASS

►

0.305

0.043

0.997

| Collector capacity | |
|--------------------|--|
| Deflection | |
| | |

Panel details

Chord capacity

Structural I wood panel sheathing on one side

lb/in²

lb/in²

in

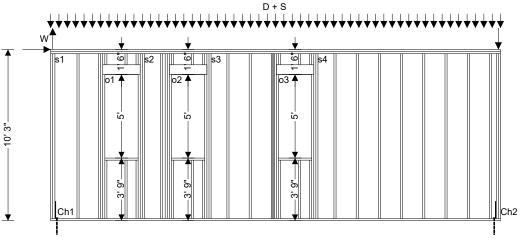
Panel height Panel length

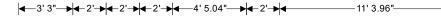


818

1508

0.246





| 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 | l₀₃ = 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 |

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| Anti | hem Structural | Engineers | Section | | | | Sheet no | o./rev. | | |
| | | | Calc. by S | Date 8/1/2022 | Chk'd by | Date | App'd by | Date | | |
| Pan | el constructio | 'n | | | | | E | EAST, LEVE | | |
| | ninal stud size | | | 2" x 6" | | | | | | |
| | ssed stud size | | | 1.5" x 5.5" | | | | | | |
| | ss-sectional are | ea of studs | | As = 8.25 i | 1 ² | | | | | |
| Stuc | d spacing | | | s = 16 in | | | | | | |
| | ninal end post s | size | | 2 x 2" x 6" | | | | | | |
| | ssed end post s | | | 2 x 1.5" x 5 | 5.5" | | | | | |
| | ss-sectional are | | 6 | A₀ = 16.5 i | n² | | | | | |
| | e diameter | · | | Dia = 1 in | | | | | | |
| Net | cross-sectiona | l area of end p | osts | A _{en} = 13.5 | in ² | | | | | |
| | ninal collector s | • | | 2 x 2" x 6" | | | | | | |
| Dres | ssed collector s | size | | 2 x 1.5" x 5 | 5.5" | | | | | |
| Ser | vice condition | | | Dry | | | | | | |
| Tem | nperature | | | 100 degF o | or less | | | | | |
| Vert | tical anchor stif | fness | | k _a = 69646 | lb/in | | | | | |
| Fro | m NDS Supple | ement Table 4 | A - Reference | design values | for visually g | raded dimens | ion lumber (2 | " - 4" thick) | | |
| | cies, grade and | | | - | o.1 & btr grade | | | | | |
| - | cific gravity | | | G = 0.43 Ft = 725 lb/in ² | | | | | | |
| - | sion parallel to | grain | | | | | | | | |
| | npression paral | - | | F _c = 1350 | b/in² | | | | | |
| Mod | lulus of elastici | ty | | E = 15000 | 00 lb/in ² | | | | | |
| Mini | imum modulus | of elasticity | | E _{min} = 550 |)00 lb/in² | | | | | |
| She | athing details | | | | | | | | | |
| | athing materi | | | 7/16" woo | d panel struc | tural I oriente | d strandboard | d sheathing | | |
| | tener type | | | | on nails at 6" | | | L'enouting | | |
| | | | | | | | | Damala | | |
| | | | | | | e Shear Walls 1 - (0.5 - G), 1 | | | | |
| | | | | | | | | | | |
| | ninal unit she | | - | | | 1 - (0.5 - G), 1 | i j, 2435 plt) = | : 7 30.1 lb/ft | | |
| Арр | parent shear w | vall shear stiff | ness | Ga = 16 ki | ps/in | | | | | |
| Loa | ding details | | | | | | | | | |
| Dea | d load acting o | n top of panel | | D = 720 lb. | ′ft | | | | | |
| Sno | w load acting o | on top of panel | | S = 800 lb/ | ft | | | | | |
| Self | weight of pane | el | | S _{wt} = 10 lb, | ′ft² | | | | | |
| In pl | lane wind load | acting at head | of panel | W = 6000 | lbs | | | | | |
| Win | d load servicea | ability factor | | f _{Wserv} = 1.0 | 0 | | | | | |
| Cho | ord forces fron | n shear walls | above | | | | | | | |
| Chord | W _{ch[i]} (lbs) | Eq_ch[i] (Ibs) | Dc_ch[i] (Ibs) | D _{T_ch[i]} (lbs) | L _{f_ch[i]} (Ibs) | Lr_ch[i] (Ibs) | S _{ch[i]} (Ibs) | R _{ch[i]} (Ibs) | | |
| Ch1 | -2152; | 0; | 0; | 0; | 0; | 0; | 0; | 0; | | |
| Ch2 | 2152; | 0; | 0; | 0; | 0; | 0; | 0; | 0; | | |
| Fro | m IBC 2021 cl. | 1605 1 Basic | load combinat | ions from ΔS | CE 7. section | 2.4 | L | | | |
| | d combination | | | D + 0.6W | , | | | | | |
| | d combination | | | D + 0.7E | | | | | | |
| | | | | | | | | | | |

 $D + 0.75L_f + 0.45W + 0.75(L_r \text{ or } S \text{ or } R)$

Load combination no.3

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| | | | | | | |
| Load combination no.4 | | | + 0.525E + 0.75 | S | EA | ST, LEVEI |
| Load combination no.5 | | 0.6D + 0.6V | | | | |
| Load combination no.6 | | 0.6D + 0.7E | - | | | |
| Adjustment factors | | | | | | |
| Load duration factor – Table 2.3. | | C _D = 1.60 | | | | |
| Size factor for tension – Table 4A | | C _{Ft} = 1.30 | | | | |
| Size factor for compression – Tal | | $C_{Fc} = 1.10$ | | | | |
| Wet service factor for tension – 1 | | C _{Mt} = 1.00 | | | | |
| Wet service factor for compression | | C _{Mc} = 1.00 | | | | |
| Wet service factor for modulus or | r elasticity – Ta | | | | | |
| Tanan analysis factor for torsian | | C _{ME} = 1.00 C _{tt} = 1.00 | | | | |
| Temperature factor for tension – Temperature factor for compress | | | | | | |
| remperature factor for compress | | C _{tc} = 1.00 | | | | |
| Temperature factor for modulus of | of electicity _ T | | | | | |
| | | CtE = 1.00 | | | | |
| Incising factor – cl.4.3.8 | | Ct⊧ = 1.00 Ci = 1.00 | | | | |
| Buckling stiffness factor – cl.4.4.2 | 2 | C⊤ = 1.00 | | | | |
| Adjusted modulus of elasticity | - | | \times Cme \times Cte \times Ci | × CT = 55000 | 0 nsi | |
| Critical buckling design value | | | $\times E_{min}' / (h / d)^2$ | | P | |
| | alua | | . , | - | 3 poi | |
| Reference compression design v | alue | $r_{c} - r_{c} \times C$ $c = 0.8$ | $_{\rm D} 	imes C_{\rm Mc} 	imes C_{\rm tc} 	imes C$ | JFc X Ui - 23/0 | b psi | |
| For sawn lumber | | | | | | |
| Column stability factor – eqn.3 | 3.7-1 | | <i>,,</i> , , , , , , , , , , , , , , , , , , | × c) – √([(1 + | (F _{cE} / F _c *)) / (2 | × C)] ² - (F _{cE} / |
| | | F _c *) / c) = (|).34 | | | |
| From SDPWS Table 4.3.4 Maxie | mum Shear W | all Aspect Ratio | os | | | |
| Maximum shear wall aspect ratio | | 3.5 | | | | |
| Perforated wall length | | b₁ = 3.25 ft | | | | |
| Shear wall aspect ratio | | h / b1 = 3.1 | 54 | | | |
| Perforated wall length | | b ₂ = 2 ft | | | | |
| Shear wall aspect ratio | | h / b ₂ = 5.1 2 | 25 | | | |
| Perforated wall length | | b ₃ = 4.42 ft | | | | |
| Shear wall aspect ratio | | h / b ₃ = 2.3 | | | | |
| Perforated wall length | | b ₄ = 11.33 t | | | | |
| Shear wall aspect ratio | | h / b ₄ = 0.9 |)5 | | | |
| Shear capacity adjustment fact | | 5 | | | | |
| Sum of perforated shear wall length | - | | imes b _s / h + b ₃ $	imes$ 2 | | | |
| Total length of perforated shear v | wall | $L_{tot} = b_1 + w$ | $v_{o1} + w_{o2} + b_3 + v_{o1}$ | v _{o3} + b ₄ = 25 ft | t | |
| Total area of openings | | $A_o = w_{o1} \times h$ | h_{o1} + W_{o2} × h_{o2} + | $w_{03} \times h_{03} = 30$ | ft² | |
| 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 | | | | | | |
| | بنام مالم ماليم م | V - 0 | c = M - c c ki | 06 | | |
| Maximum shear force under v | vind loading | V w_max - 0. | 6 × W = 3.6 kip | P 3 | | |

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| | | V _{w_max} / V _w | = 0.609 | | EAS | ST, LEVE |
| | | PASS - | Shear capacity | for wind load | exceeds maxim | um shear |
| 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 tensior | 1 | P = (0.6× | $(D + S_{wt} \times h)) >$ | < b / 2 + 0.6 × | W _{ch1} = 5.371 ki | ps |
| Maximum tensile force in chord | | $T = V \times h /$ | ((C _o × ΣL _i)) - P = | -3.092 kips ┥ | — NEGATIVE. N | 10 |
| Maximum applied tensile stress | | f _t = T / A _{en} = | | | OVERTURNI | |
| Design tensile stress | | F_t = $F_t \times C_t$ | $C \times C_{Mt} \times C_{tt} \times C_{F}$ | _t × C _i = 1508 lb/ | in² | |
| | | f _t / F _t ' = -0.1 | | | | |
| | | PASS - De | sign tensile stro | ess exceeds m | aximum applie | d tensile s |
| Load combination 1 | | | | | | |
| Shear force for maximum compress | | $V = 0.6 \times W$ | - | | | |
| Axial force for maximum compre | | | $S_{wt} \times h) \times s / 2$ | | V _{ch1} = 1.84 kips | |
| Maximum compressive force in cho | | | $((C_o \times \Sigma L_i)) + P =$ | 4.118 kips | | |
| Maximum applied compressive stre | SS | f _c = C / A _e = | | | | |
| Design compressive stress | | | $D \times C_{Mc} \times C_{tc} \times C$ | $C_{Fc} \times C_i \times C_P = 8$ | 18 lb/in ² | |
| | DASS | f _c / F _c ' = 0.3 Design compre | | coode maximu | m applied com | nroccivo c |
| | FA33 - | Design compre | 331VE 311E33 EX | | in applied com | pressive s |
| Chord capacity for chord 2 Load combination 5 | | | | | | |
| Shear force for maximum tension | | V = 0.6 × V | / = 3.6 kins | | | |
| Axial force for maximum tensior | 1 | | $(D + S_{wt} \times h)) >$ | | 6 × Waha = 5 3 | 71 kins |
| Maximum tensile force in chord | 1 | • | $(C_{o} \times \Sigma L_{i})) - P =$ | | | • |
| Maximum applied tensile stress | | f _t = T / A _{en} = | | 0.002 1105 | OVERTURN | |
| Design tensile stress | | | $\times C_{Mt} \times C_{tt} \times C_{F}$ | t × Ci = 1508 lb/ | in ² | |
| 5 | | f _t / F _t ' = -0.1 | | | | |
| | | PASS - De | sign tensile stre | ess exceeds m | aximum applie | d tensile s |
| Load combination 1 | | | | | | |
| Shear force for maximum compress | sion | $V = 0.6 \times W$ | / = 3.6 kips | | | |
| Axial force for maximum compre | ession | P = ((D + 3 | $S_{wt} \times h$)) \times s / 2 | + $0.6 \times W_{ch2}$ = | = 1.84 kips | |
| Maximum compressive force in cho | ord | $C = V \times h /$ | $((C_o \times \Sigma L_i)) + P =$ | 4.118 kips | | |
| Maximum applied compressive stre | SS | $f_c = C / A_e =$ | 250 lb/in ² | | | |
| Design compressive stress | | $F_c' = F_c \times C$ | $_{\text{D}} 	imes C_{\text{Mc}} 	imes C_{\text{tc}} 	imes C$ | $C_{Fc} \times C_i \times C_P = 8$ | 18 lb/in ² | |
| | | f _c / F _c ' = 0.3 | 05 | | | |



| Maximum shear force on wall | $v_{max} - v_{w_max} - 3.6$ klps |
|------------------------------------|--|
| Uniform shear applied to wall | $v_a = V_{max} / ((C_o \times \Sigma L_i)) = 222.3 \text{ plf}$ |
| Shear resisted by wall segments | v _b = v _a × b / (b ₁ + b ₃ + b ₄) = 315.9 plf |
| Maximum force in collector | P _{coll} = 1.061 kips |
| Maximum applied tensile stress | $f_t = P_{coll} / (2 \times A_s) = 64 \text{ lb/in}^2$ |
| Design tensile stress | $F_{t}' = F_{t} \times C_{D} \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_{i} = 1508 \text{ lb/in}^{2}$ |
| | f _t / F _t ' = 0.043 |
| | PASS - Design tensile stress exceeds maximum applied tensile stress |
| Maximum applied compressive stress | $f_c = P_{coll} / (2 \times A_s) = 64 \text{ lb/in}^2$ |
| Column stability factor | C _P = 1.00 |
| Design compressive stress | $F_{c}' = F_{c} \times C_{D} \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_{i} \times C_{P} = 2376 \text{ lb/in}^{2}$ |
| | f _c / F _c ' = 0.027 |
| PA | ASS - Design compressive stress exceeds maximum applied compressive stress |
| | |

Wind load deflection

Design shear force Deflection limit Induced unit shear Anchor tension force

Shear wall deflection - Eqn. 4.3-1

$$\begin{split} &V_{\delta w} = f_{Wserv} \times W = 6 \text{ kips} \\ &\Delta_{w_allow} = h \ / \ 500 = 0.246 \text{ in} \\ &v_{\delta w_max} = V_{\delta w} \ / \ (C_o \times \Sigma L_i) = 370.51 \text{ lb/ft} \\ &T_\delta = max(0 \text{ kips}, v_{\delta w_max} \times h - 0.6 \times (D + S_{wt} \times h) \times b \ / \ 2 + \\ &max(abs(W_{ch1}), abs(W_{ch2}))) = 0.000 \text{ kips} \\ &\delta_{sww} = 2 \times v_{\delta w_max} \times h^3 \ / \ (3 \times E \times A_e \times \Sigma L_i) + v_{\delta w_max} \times h \ / \ (G_a) + h \times T_\delta \ / \ (k_a \times \Sigma L_i) = 0.245 \text{ in} \\ &\delta_{sww} \ / \ \Delta_{w_allow} = 0.997 \end{split}$$

PASS - Shear wall deflection is less than deflection limit

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| WOOD SHEAR WALL DESIGN In accordance with NDS2018 a | <u> </u> | ess design and | I the perforate | ed shear wall r | | | T, LEVE |
| 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 | | |
| Structural I wood panel sheathin Panel height Panel length ↓↓↓↓↓↓ | | h = 10.2 b = 27 ft | D + S | ++++++++++ | ┟┿┿┿┿┿┿┿ | ·↓↓↓ | |
| | _ | | | | | | |
| ν | | s2 | | | | | |



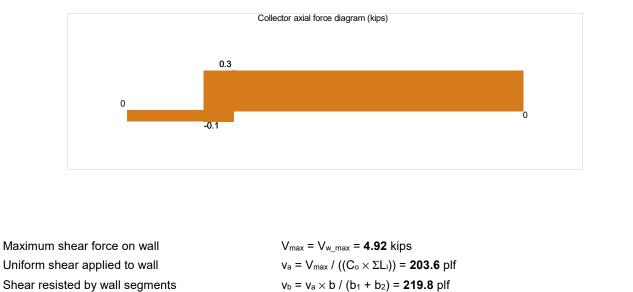
| Panel opening details | |
|--|---|
| Width of opening | w _{o1} = 2 ft |
| Height of opening | h _{o1} = 5 ft |
| Height to underside of lintel over opening | I _{o1} = 7.5 ft |
| Position of opening | P _{o1} = 5.25 ft |
| Total area of wall | A = $h \times b$ - $w_{o1} \times h_{o1}$ = 266.75 ft 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" |

| 구 Te | kla _® T | edds | Project | | | | Job Ref. | | |
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| Cross-section | onal are | a of end posts | 3 | A _e = 16.5 i | n ² | | E | EAST, LEV | ΈL |
| Hole diamet | ter | | | Dia = 1 in | | | | | |
| Net cross-se | ectional | area of end p | osts | A _{en} = 13.5 | in² | | | | |
| Nominal col | llector si | ze | | 2 x 2" x 6" | | | | | |
| Dressed col | llector si | ze | | 2 x 1.5" x 5 | 5.5" | | | | |
| Service con | ndition | | | Dry | | | | | |
| Temperatur | e | | | 100 degF o | or less | | | | |
| Vertical anc | chor stiff | ness | | k _a = 30000 | lb/in | | | | |
| From NDS | Suppler | ment Table 4 | A - Reference | design values | for visually o | raded dimens | ion lumber (2' | " - 4" thick) | |
| | | size classifica | | - | o.1 & btr grade | | | , | |
| Specific gra | | | | G = 0.43 | 0 | | | | |
| Tension par | - | grain | | Ft = 725 lb | /in² | | | | |
| Compressio | - | - | | F _c = 1350 | b/in ² | | | | |
| Modulus of | - | - | | E = 15000 | | | | | |
| Minimum m | - | | | E _{min} = 550 (| | | | | |
| Sheathing | | | | | | | | | |
| Sheathing | | al | | 7/16" woo | d nanel struc | tural I oriente | d strandboard | d sheathing | |
| - | materia | 11 | | | | | u stranubuart | a sheathing | |
| Eastonar t | VIDO | | | Od comm | on naile at 6"/ | ontoro | | | |
| Fastener ty | | | | | on nails at 6" | | | | |
| - | | e 4.3A Nomir | nal Unit Shear | 8d commo | | | - Wood-based | l Panels | |
| From SDPV | WS Tabl | | | | r Wood-Frame | Shear Walls | | | |
| From SDPV Nominal ur | WS Tabl nit shea | r capacity fo | | Capacities fo ign v _s = min(5 | r Wood-Frame 660 plf × min[* | Shear Walls |], 1740 plf) = | 520.8 lb/ft | |
| From SDPV Nominal ur Nominal ur | WS Tabl nit shea nit shea | r capacity fo r capacity fo | r seismic desi r wind design | Capacities fo ign v _s = min(5 | r Wood-Frame 560 plf × min[[:] 785 plf × min[| • Shear Walls • 1 - (0.5 - G), 1 |], 1740 plf) = | 520.8 lb/ft | |
| From SDPV Nominal ur Nominal ur Apparent s | WS Tabl nit shea nit shea shear wa | r capacity fo | r seismic desi r wind design | Capacities fo ign v _s = min(5 v _w = min(7 | r Wood-Frame 560 plf × min[[:] 785 plf × min[| • Shear Walls • 1 - (0.5 - G), 1 |], 1740 plf) = | 520.8 lb/ft | |
| From SDPV Nominal ur Nominal ur Apparent s Loading de | WS Tabl nit shea nit shea shear wa etails | r capacity fo r capacity fo all shear stiff | r seismic desi r wind design | Capacities for ign $v_s = min(5)$ $v_w = min(7)$ $G_a = 16$ ki | r Wood-Frame 560 plf × min[⁻ 785 plf × min[ps/in | • Shear Walls • 1 - (0.5 - G), 1 |], 1740 plf) = | 520.8 lb/ft | |
| From SDPV Nominal ur Nominal ur Apparent s Loading de Dead load a | WS Tabl nit shea nit shea shear wa etails acting or | r capacity fo r capacity fo all shear stiff top of panel | r seismic desi r wind design | Capacities for ign $v_s = min(5)$ $v_w = min(7)$ $G_a = 16$ ki D = 980 lb/ | r Wood-Frame 560 plf × min[[*] 785 plf × min[ps/in ⁄ft | • Shear Walls • 1 - (0.5 - G), 1 |], 1740 plf) = | 520.8 lb/ft | |
| From SDPV Nominal ur Nominal ur Apparent s Loading de Dead load a Snow load a | WS Tabl nit shea nit shea shear wa shear wa stails acting or acting or | r capacity fo r capacity fo all shear stiff n top of panel n top of panel | r seismic desi r wind design | Capacities for ign $v_s = min(5)$ $v_w = min(7)$ $G_a = 16 ki$ D = 980 lb/ S = 800 lb/ | r Wood-Frame 560 plf × min[785 plf × min[785/in /ft | • Shear Walls • 1 - (0.5 - G), 1 |], 1740 plf) = | 520.8 lb/ft | |
| From SDPV Nominal un Nominal un Apparent s Loading de Dead load a Snow load a Self weight | WS Tabl nit shea nit shea shear wa etails acting or of panel | r capacity fo r capacity fo all shear stiff n top of panel n top of panel | r seismic desi r wind design ness | Capacities for ign v _s = min(5 v _w = min(7 G _a = 16 ki D = 980 lb/ S = 800 lb/ S _{wt} = 10 lb/ | r Wood-Frame 560 plf × min[785 plf × min[785 in /ft /ft | • Shear Walls • 1 - (0.5 - G), 1 |], 1740 plf) = | 520.8 lb/ft | |
| From SDPV Nominal ur Nominal ur Apparent s Loading de Dead load a Snow load a Self weight In plane win | WS Tabl nit shea nit shea shear wa etails acting or acting or of panel nd load a | ar capacity fo ar capacity fo all shear stiff a top of panel a top of panel acting at head | r seismic desi r wind design ness | Capacities for ign $v_s = min(5)$ $v_w = min(7)$ $G_a = 16 ki$ D = 980 lb/ S = 800 lb/ $S_{wt} = 10 lb/$ W = 8200 lb/ | r Wood-Frame 560 plf × min[785 plf × min[785 nl 785 plf × min[785 ml 785 ml | • Shear Walls • 1 - (0.5 - G), 1 |], 1740 plf) = | 520.8 lb/ft | |
| From SDPV Nominal un Nominal un Apparent s Loading de Dead load a Snow load a Self weight In plane win Wind load s | WS Tabl nit shea nit shea shear wa etails acting or acting or of panel nd load a serviceal | ar capacity fo ar capacity fo all shear stiff top of panel top of panel acting at head pility factor | r seismic desi r wind design ness of panel | Capacities for ign v _s = min(5 v _w = min(7 G _a = 16 ki D = 980 lb/ S = 800 lb/ S _{wt} = 10 lb/ | r Wood-Frame 560 plf × min[785 plf × min[785 nl 785 plf × min[785 ml 785 ml | • Shear Walls • 1 - (0.5 - G), 1 |], 1740 plf) = | 520.8 lb/ft | |
| From SDPV Nominal ur Nominal ur Apparent s Loading de Dead load a Snow load a Self weight In plane win Wind load s Chord force | WS Tabl nit shea nit shea shear wa etails acting or acting or of panel nd load a serviceal ces from (lbs) | ar capacity fo ar capacity fo all shear stiff a top of panel a top of panel acting at head | r seismic desi r wind design ness of panel | Capacities for ign $v_s = min(5)$ $v_w = min(7)$ $G_a = 16 ki$ D = 980 lb/ S = 800 lb/ $S_{wt} = 10 lb/$ W = 8200 lb/ | r Wood-Frame 560 plf × min[785 plf × min[785 nl 785 plf × min[785 ml 785 ml | • Shear Walls • 1 - (0.5 - G), 1 |], 1740 plf) = | 520.8 lb/ft | |
| From SDPV Nominal ur Nominal ur Apparent s Loading de Dead load a Snow load a Self weight In plane win Wind load s Chord force hord Wchtij Ch1 -44 | WS Tabl nit shea nit shea nit shea shear wa etails acting or acting or of panel nd load a serviceal ces from (lbs) | ar capacity for all shear stiff a top of panel a top of panel acting at head pility factor shear walls $E_{q_ch[i]}$ (lbs) 0; | r seismic desi r wind design ness of panel above D c_ch[i] (Ibs) 0; | Capacities for ign v _s = min(5 v _w = min(7 G _a = 16 ki D = 980 lb/ S = 800 lb/ S _{wt} = 10 lb/ W = 8200 l f _{Wserv} = 1.0 | r Wood-Frame 560 plf × min[785 plf × min[785 nf 785 nf 77 77 77 77 77 77 77 77 77 77 77 77 77 | e Shear Walls 1 - (0.5 - G), 1 1 - (0.5 - G), 1 1 - (0.5 - G), 1 <u>Lr_ch[i]</u> (Ibs) 0; |], 1740 plf) =], 2435 plf) =], 2635 plf) = , 2435 plf) = | 520.8 lb/ft 730.1 lb/ft Rch[i] (lbs) 0; | |
| From SDPV Nominal ur Nominal ur Apparent s Loading de Dead load a Snow load a Self weight In plane win Wind load s Chord force hord Wchtij Ch1 -44 | WS Tabl nit shea nit shea shear wa etails acting or acting or of panel nd load a serviceal ces from (lbs) | r capacity fo ar capacity fo all shear stiff top of panel top of panel acting at head pility factor shear walls E q_ch(i) (Ibs) | r seismic desi r wind design ness of panel above Dc_ch[i] (Ibs) | Capacities for ign v _s = min(5 v _w = min(7 G _a = 16 ki D = 980 lb/ S = 800 lb/ S _{wt} = 10 lb/ W = 8200 l f _{Wserv} = 1.0 | r Wood-Frame 560 plf × min[785 plf × min[785 n 785 plf × min[ps/in 785 785 785 785 785 785 785 785 785 785 | Shear Walls 1 - (0.5 - G), 1 1 - (0.5 - G), 1 Lr_ch[i] (Ibs) |], 1740 plf) =], 2435 plf) = | 520.8 lb/ft 730.1 lb/ft Rch[i] (lbs) | |
| From SDPV Nominal ur Nominal ur Apparent s Loading de Dead load a Snow load a Self weight In plane win Wind load s Chord force hord Wchtt Ch1 -44 Ch2 44 | WS Tabl nit shea nit shea shear wa etails acting or acting or of panel nd load a serviceal es from (lbs) -18; -18; -18; | ar capacity for ar capacity for all shear stiff a top of panel acting at head pility factor shear walls $E_{q_ch[1]}(lbs)$ 0; 0; | r seismic desi r wind design ness of panel above Dc_ch[i] (Ibs) 0; 0; | Capacities for ign v _s = min(5 v _w = min(7 G _a = 16 ki D = 980 lb/ S = 800 lb/ S _{wt} = 10 lb/ W = 8200 l f _{Wserv} = 1.0 | r Wood-Frame 560 plf \times min[785 plf \times min[ps/in /ft /ft /ft ft ft ft^2 lbs 0 L _{f_ch[i]} (lbs) 0; 0; | Shear Walls 1 - (0.5 - G), 1 1 - (0.5 - G), 1 1 - (0.5 - G), 1 0; 0; |], 1740 plf) =], 2435 plf) =], 2635 plf) = , 2435 plf) = | 520.8 lb/ft 730.1 lb/ft Rch[i] (lbs) 0; | |
| From SDPV Nominal ur Nominal ur Apparent s Loading de Dead load a Snow load a Self weight In plane win Wind load s Chord force hord Wchtt Ch1 -44 Ch2 44 | WS Tabl nit shea nit shea shear wa etails acting or acting or of panel nd load a serviceal ces from (Ibs) 18; 18; 2021 cl.1 | ar capacity fo ar capacity fo all shear stiff top of panel top of panel acting at head bility factor shear walls C c c c c c f g c h i i i i i i i i i i | r seismic desi r wind design ness of panel above Dc_ch[i] (Ibs) 0; 0; | Capacities for ign $v_s = min(5)$ $v_w = min(7)$ $G_a = 16 ki$ D = 980 lb/ S = 800 lb/ S = 800 lb/ $S_{wt} = 10 lb/$ W = 8200 lb/ $f_{Wserv} = 1.0$ $D_{T_ch[i]}(lbs)$ 0; 0; | r Wood-Frame 560 plf \times min[785 plf \times min[ps/in /ft /ft /ft ft ft ft^2 lbs 0 L _{f_ch[i]} (lbs) 0; 0; | Shear Walls 1 - (0.5 - G), 1 1 - (0.5 - G), 1 1 - (0.5 - G), 1 0; 0; |], 1740 plf) =], 2435 plf) =], 2635 plf) = , 2435 plf) = | 520.8 lb/ft 730.1 lb/ft Rch[i] (lbs) 0; | |
| From SDPV Nominal ur Nominal ur Apparent s Loading de Dead load a Snow load a Self weight In plane win Wind load s Chord force hord Wchttl Ch1 -44 Ch2 442 | WS Tabl nit shea nit shea shear wa etails acting or acting or of panel nd load a serviceal ces from (Ibs) 18; 18; 2021 cl.1 | ar capacity fo ar capacity fo all shear stiff top of panel top of panel acting at head pility factor shear walls Eq_ch[] (Ibs) 0; 0; 0; 1605.1 Basic | r seismic desi r wind design ness of panel above Dc_ch[i] (Ibs) 0; 0; | Capacities for ign v _s = min(5 v _w = min(7 G _a = 16 ki D = 980 lb/ S = 800 lb/ S _{wt} = 10 lb/ W = 8200 l f _{Wserv} = 1.0 DT_ch[1] (lbs) 0; 0; tions from AS | r Wood-Frame 560 plf \times min[785 plf \times min[ps/in /ft /ft /ft ft ft ft^2 lbs 0 L _{f_ch[i]} (lbs) 0; 0; | Shear Walls 1 - (0.5 - G), 1 1 - (0.5 - G), 1 1 - (0.5 - G), 1 0; 0; |], 1740 plf) =], 2435 plf) =], 2635 plf) = , 2435 plf) = | 520.8 lb/ft 730.1 lb/ft Rch[i] (lbs) 0; | |
| From SDPV Nominal ur Nominal ur Apparent s Loading de Dead load a Snow load a Self weight In plane win Wind load s Chord force hord Wcht Ch1 -44 Ch2 44 From IBC 2 Load combi | WS Tabl nit shea nit shea nit shea shear wa etails acting or of panel nd load a serviceal ces from (lbs) 18; 18; 2021 cl.1 ination n ination n | ar capacity fo ar capacity fo all shear stiff top of panel top of panel acting at head pility factor shear walls E q_ch[] (lbs) 0; 0; 0; 0; 1605.1 Basic 0.1 0.2 | r seismic desi r wind design ness of panel above Dc_ch[i] (Ibs) 0; 0; | Capacities for ign v _s = min(5 v _w = min(7 G _a = 16 ki D = 980 lb/ S = 800 lb/ S = 800 lb/ S = 800 lb/ W = 8200 lb/ W = 8200 lb/ W = 8200 lb/ W = 8200 lb/ O; DT_ch[i] (lbs) O; tions from ASC D + 0.6W D + 0.7E | r Wood-Frame 560 plf \times min[785 plf \times min[ps/in /ft /ft /ft ft ft ft^2 lbs 0 L _{f_ch[i]} (lbs) 0; 0; | Shear Walls 1 - (0.5 - G), 1 1 - (0.5 - G), 1 1 - (0.5 - G), 1 0; 0; 0; 2.4 |], 1740 plf) =], 2435 plf) =], 2635 plf) = , 2435 plf) = | 520.8 lb/ft 730.1 lb/ft Rch[i] (lbs) 0; | |
| From SDPV Nominal ur Nominal ur Apparent s Loading de Dead load a Snow load a Self weight In plane win Wind load s Chord force hord Wchtt Ch1 -44 Ch2 442 From IBC 2 Load combi | WS Tabl nit shea nit shea shear wa etails acting or acting or of panel nd load a serviceal ces from (Ibs) 18; 18; 2021 cl.1 ination n ination n | ar capacity fo ar capacity fo all shear stiff top of panel top of panel acting at head pility factor shear walls E q_ch[] (Ibs) 0; 0; I605.1 Basic 0.1 0.2 0.3 | r seismic desi r wind design ness of panel above Dc_ch[i] (Ibs) 0; 0; | Capacities for ign $v_s = min(5)$ $v_w = min(7)$ $G_a = 16 ki$ D = 980 lb/ S = 800 lb/ $S_{wt} = 10 lb/$ W = 8200 lb/ W = 8200 lb/ $U = 0.75 L_f$ | r Wood-Frame 560 plf × min[785 plf × min[785 plf × min[ps/in /ft /ft /ft /ft /ft 2 lbs 0 L _{f_ch[i]} (lbs) 0; 0; 0; CE 7, section | Shear Walls 1 - (0.5 - G), 1 1 - (0.5 - G), 1 1 - (0.5 - G), 1 0; 0; 0; 0; 0; 0; 0; 0; 0; 0; 0; 0; 0; 0; 0; 0; 0; 0; 0; 2.4 |], 1740 plf) =], 2435 plf) =], 2635 plf) = , 2435 plf) = | 520.8 lb/ft 730.1 lb/ft Rch[i] (lbs) 0; | |
| From SDPV Nominal ur Nominal ur Apparent s Loading de Dead load a Snow load a Self weight In plane win Wind load s Chord force hord Wchtti Ch1 -44 Ch2 442 From IBC 2 Load combi Load combi | WS Tabl nit shea nit shea nit shea nit shea shear wa etails acting or acting or of panel nd load a serviceal ces from (Ibs) 18; 18; 2021 cl.1 ination n ination n ination n | ar capacity fo ar capacity fo all shear stiff top of panel top of panel acting at head pility factor shear walls E q_ch[] (lbs) 0; 0; 0; 0; 1605.1 Basic 0.1 0.2 0.3 0.4 | r seismic desi r wind design ness of panel above Dc_ch[i] (Ibs) 0; 0; | Capacities for ign $v_s = min(5)$ $v_w = min(7)$ $G_a = 16 ki$ D = 980 lb/ S = 800 lb/ $S_{wt} = 10 lb/$ W = 8200 lb/ W = 8200 lb/ $U = 0.75 L_f$ | r Wood-Frame 560 plf × min[785 plf × min[785 plf × min[ps/in /ft /ft /ft /ft 2 lbs 0 Lf_ch[i] (lbs) 0; CE 7, section + 0.45W + 0.7 + 0.525E + 0.7 | Shear Walls 1 - (0.5 - G), 1 1 - (0.5 - G), 1 1 - (0.5 - G), 1 0; 0; 0; 0; 0; 0; 0; 0; 0; 0; 0; 0; 0; 0; 0; 0; 0; 0; 0; 2.4 |], 1740 plf) =], 2435 plf) =], 2635 plf) = , 2435 plf) = | 520.8 lb/ft 730.1 lb/ft Rch[i] (lbs) 0; | |
| From SDPV Nominal ur Nominal ur Apparent s Loading de Dead load a Snow load a Self weight In plane win Wind load s Chord force hord Wchtil Ch1 -44 Ch2 44 ² From IBC 2 Load combi Load combi Load combi | WS Tabl nit shea nit shea shear wa etails acting or acting or of panel nd load a serviceal ces from (Ibs) 18; 18; 2021 cl.1 ination n ination n ination n ination n | ar capacity for an capacity for all shear stiff a top of panel acting at head pility factor shear walls $E_{q_ch[1]}(lbs)$ 0; 0; 0; 1605.1 Basic 0.1 0.2 0.3 0.4 0.5 | r seismic desi r wind design ness of panel above Dc_ch[i] (Ibs) 0; 0; | Capacities for ign $v_s = min(5)$ $v_w = min(7)$ $G_a = 16 ki$ D = 980 lb/ S = 800 lb/ S = 800 lb/ $S_{wt} = 10 lb/$ W = 8200 lb/ W = 8200 lb/ W = 8200 lb/ W = 8200 lb/ W = 8200 lb/ $G_{wserv} = 1.0$ $D_{T_cch[i]}$ (lbs) 0; 0; tions from ASC D + 0.6W $D + 0.75L_f$ $D + 0.75L_f$ | r Wood-Frame 560 plf × min[785 plf × min[ps/in /ft (ft (ft (ft (ft 2 lbs 0 Lf_ch[i] (lbs) 0; 0; CE 7, section + 0.45W + 0.7 + 0.525E + 0.7 W | Shear Walls 1 - (0.5 - G), 1 1 - (0.5 - G), 1 1 - (0.5 - G), 1 0; 0; 0; 0; 0; 0; 0; 0; 0; 0; 0; 0; 0; 0; 0; 0; 0; 0; 0; 2.4 |], 1740 plf) =], 2435 plf) =], 2635 plf) = , 2435 plf) = | 520.8 lb/ft 730.1 lb/ft Rch[i] (lbs) 0; | |
| From SDPV Nominal ur Nominal ur Apparent s Loading de Dead load a Snow load a Self weight In plane win Wind load s Chord force hord Wcht Ch1 -44 Ch2 44 ⁻ From IBC 2 Load combi Load combi Load combi Load combi | WS Tabl nit shea nit shea nit shea nit shea shear wa etails acting or acting or of panel nd load a serviceal ces from (lbs) 18; 2021 cl.1 ination n ination n ination n ination n ination n | ar capacity for all shear stiff at top of panel top of panel acting at head polity factor shear walls $E_{q_ch[1]}(lbs)$ 0; 0; 0; 0; 1605.1 Basic 0.1 0.2 0.3 0.4 0.5 0.6 | r seismic desi r wind design ness of panel above Dc_ch[i] (Ibs) 0; 0; | Capacities for ign $v_s = min(5)$ $v_w = min(7)$ $G_a = 16 ki$ D = 980 lb/ S = 800 lb/ S = 800 lb/ $S_{wt} = 10 lb/$ W = 8200 lb/ W = 8200 lb/ W = 8200 lb/ W = 8200 lb/ W = 8200 lb/ $G_{wserv} = 1.0$ $D_{T_ch[1]}(lbs)$ 0; 0; tions from ASC D + 0.6W $D + 0.75L_f$ $D + 0.75L_f$ 0.6D + 0.6 | r Wood-Frame 560 plf × min[785 plf × min[ps/in /ft (ft (ft (ft (ft 2 lbs 0 Lf_ch[i] (lbs) 0; 0; CE 7, section + 0.45W + 0.7 + 0.525E + 0.7 W | Shear Walls 1 - (0.5 - G), 1 1 - (0.5 - G), 1 1 - (0.5 - G), 1 0; 0; 0; 0; 0; 0; 0; 0; 0; 0; 0; 0; 0; 0; 0; 0; 0; 0; 0; 2.4 |], 1740 plf) =], 2435 plf) =], 2635 plf) = , 2435 plf) = | 520.8 lb/ft 730.1 lb/ft Rch[i] (lbs) 0; | |
| From SDPV Nominal ur Nominal ur Apparent s Loading de Dead load a Snow load a Self weight In plane win Wind load s Chord force hord Wchtt Ch1 -44 Ch2 442 From IBC 2 Load combi Load combi Load combi Load combi Load combi | WS Tabl nit shea nit shea shear wa etails acting or acting or of panel nd load a serviceal ces from (Ibs) 18; 18; 2021 cl.1 ination n ination n ination n ination n ination n ination n | ar capacity for all shear stiff at top of panel top of panel acting at head polity factor shear walls $E_{q_ch[1]}(lbs)$ 0; 0; 0; 0; 1605.1 Basic 0.1 0.2 0.3 0.4 0.5 0.6 | r seismic desi r wind design ness of panel above Dc_ch[i] (Ibs) 0; 0; load combinat | Capacities for ign $v_s = min(5)$ $v_w = min(7)$ $G_a = 16 ki$ D = 980 lb/ S = 800 lb/ S = 800 lb/ $S_{wt} = 10 lb/$ W = 8200 lb/ W = 8200 lb/ $w_{serv} = 1.0$ $D_{T_ch[1]}(lbs)$ 0; 0; tions from ASC D + 0.6W D + 0.7E $D + 0.75L_f$ 0.6D + 0.6' 0.6D + 0.7' | r Wood-Frame 560 plf × min[785 plf × min[ps/in /ft (ft (ft (ft (ft 2 lbs 0 Lf_ch[i] (lbs) 0; 0; CE 7, section + 0.45W + 0.7 + 0.525E + 0.7 W | Shear Walls 1 - (0.5 - G), 1 1 - (0.5 - G), 1 1 - (0.5 - G), 1 0; 0; 0; 0; 0; 0; 0; 0; 0; 0; 0; 0; 0; 0; 0; 0; 0; 0; 0; 2.4 |], 1740 plf) =], 2435 plf) =], 2635 plf) = , 2435 plf) = | 520.8 lb/ft 730.1 lb/ft Rch[i] (lbs) 0; | |
| From SDPV Nominal ur Nominal ur Apparent s Loading de Dead load a Snow load a Self weight In plane win Wind load s Chord force Mord Wch(1) Ch1 -44 Ch2 44 ² From IBC 2 Load combi Load combi Load combi Load combi Load combi Load combi | WS Tabl nit shea nit shea nit shea shear wa etails acting or acting or of panel nd load a serviceal ces from (Ibs) 18; 2021 cl.1 ination n ination n ination n ination n ination n ination n ination n on factors | ar capacity for all shear stiff a top of panel b top of panel acting at head bility factor shear walls $E_{q_ch[1]}(lbs)$ 0; 0; 0; 0; 1605.1 Basic 0.1 0.2 0.3 0.4 0.5 0.6 s | r seismic desi r wind design ness of panel above Dc_ch[i] (Ibs) 0; 0; Ioad combinat | Capacities for ign $v_s = min(5)$ $v_w = min(7)$ $G_a = 16 ki$ D = 980 lb/ S = 800 lb/ S = 800 lb/ $S_{wt} = 10 lb/$ W = 8200 lb/ W = 8200 lb/ W = 8200 lb/ W = 8200 lb/ W = 8200 lb/ $G_{wserv} = 1.0$ $D_{T_ch[1]}(lbs)$ 0; 0; tions from ASC D + 0.6W $D + 0.75L_f$ $D + 0.75L_f$ 0.6D + 0.6 | r Wood-Frame 560 plf × min[785 plf × min[ps/in /ft (ft (ft (ft (ft 2 lbs 0 Lf_ch[i] (lbs) 0; 0; CE 7, section + 0.45W + 0.7 + 0.525E + 0.7 W | Shear Walls 1 - (0.5 - G), 1 1 - (0.5 - G), 1 1 - (0.5 - G), 1 0; 0; 0; 0; 0; 0; 0; 0; 0; 0; 0; 0; 0; 0; 0; 0; 0; 0; 0; 2.4 |], 1740 plf) =], 2435 plf) =], 2635 plf) = , 2435 plf) = | 520.8 lb/ft 730.1 lb/ft Rch[i] (lbs) 0; | |

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|--|-------------------|---|---|--|--|--|
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| | Calc. by S | Date 8/1/2022 | Chk'd by | Date | App'd by | Date |
| Wet service factor for tension – T | able 4A | C _{Mt} = 1.00 | | | EA | ST, LEVEL |
| Wet service factor for compression | | | | | | |
| Wet service factor for modulus of | f elasticity – Ta | | | | | |
| Taura anatura faatan fan tanaian | | Сме = 1.00 | | | | |
| Temperature factor for tension – Temperature factor for compress | | Ctt = 1.00 | | | | |
| | | C _{tc} = 1.00 | | | | |
| Temperature factor for modulus of | of elasticity – | | | | | |
| · | , | C _{tE} = 1.00 | | | | |
| Incising factor – cl.4.3.8 | | C _i = 1.00 | | | | |
| Buckling stiffness factor - cl.4.4.2 | 2 | C⊤ = 1.00 | | | | |
| Adjusted modulus of elasticity | | Emin' = Emin > | $\langle C_{ME} \times C_{tE} \times C_{i} \rangle$ | × C _T = 550000 |) psi | |
| Critical buckling design value | | $F_{cE} = 0.822$ | imes E _{min} ' / (h / d) ² | = 904 psi | | |
| Reference compression design v | alue | $F_{c}^{*} = F_{c} \times C$ | $C \times C_{Mc} \times C_{tc} \times C_{tc}$ | C _{Fc} × C _i = 2376 | psi | |
| For sawn lumber | | c = 0.8 | | | | |
| Column stability factor - eqn.3 | 3.7-1 | C _P = (1 + (| F _{cE} / F _c *)) / (2 > | ≺ c) – √([(1 + | (F _{cE} / F _c *)) / (2 | \times c)] ² - (F _{cE} / |
| | | F _c *) / c) = (| .34 | | | |
| Perforated wall length Shear wall aspect ratio Perforated wall length Shear wall aspect ratio | | b ₁ = 5.25 ft h / b ₁ = 1.95 b ₂ = 19.75 f h / b ₂ = 0.5 1 | t | | | |
| Shear capacity adjustment fact | | | 0 T (| | | |
| Sum of perforated shear wall leng | - | $\Sigma L_i = b_1 + b_2$ | | | | |
| Total length of perforated shear v Total area of openings | vali | $L_{tot} = D_1 + W$ $A_o = W_{o1} \times h$ | _{b1} + b ₂ = 27 ft | | | |
| Sheathing area ratio (eqn. 4.3-6) | | | $\Lambda_0 / (h \times \Sigma L_i)) = 0$ | 962 | | |
| Shear capacity adjustment factor | | C₀ = 0.967 | (11 × 2L)) - U | .502 | | |
| Perforated shear wall capacity | (| | | | | |
| Maximum shear force under v | ind loading | V = 0 | 6 × W = 4.92 k | ins | | |
| Shear capacity for wind loadin | Ũ | - | $S_{o} \times \Sigma L_{i} / 2 = 8.$ | | | |
| Shear capacity for wind loadin | y | $V_{W} - V_{W} \times C$ $V_{w max} / V_{w} =$ | | 023 KIPS | | |
| | | - | | for wind load | exceeds maxin | num shear foi |
| Chord capacity for chord 1 | | | ······ | | | |
| Load combination 5 | | | | | | |
| Shear force for maximum tensior | 1 | $V = 0.6 \times W$ | = 4.92 kips | | | |
| Axial force for maximum tensi | | | - | < b / 2 + 0.6 × | ≪W _{ch1} = 6.117 k | ips |
| Maximum tensile force in chord | | | . ,, | | ← NEGATIVE. | - |
| Maximum applied tensile stress | | ft = T / A _{en} = | -299 lb/in ² | | OVERTURN | |
| | | | ~ ~ ~ | V C - 4500 H | /in ² | |
| Design tensile stress | | Ft' = Ft × CD ft / Ft' = -0.1 | $\times C_{Mt} \times C_{tt} \times C_{Ft}$ | t X Gi – 1500 II. | ,,,,, | |

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|--|---------------|--|---|----------------------------------|-------------------------------------|---------------|--|--|
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| | Calc. by S | Date 8/1/2022 | Chk'd by | Date | App'd by | Date | | |
| Load combination 1 | | | | | EAS | ST, LEVE | | |
| Shear force for maximum compre | ssion | V = 0.6 × W | = 4.92 kips | | | | | |
| Axial force for maximum comp | ression | P = ((D + S | 5wt×h))×s / 2 | + -1 × 0.6 × | W _{ch1} = 3.372 kip | s | | |
| Maximum compressive force in ch | ord | $C = V \times h / ($ | $(C_o \times \Sigma L_i)) + P =$ | 5.459 kips | | | | |
| Maximum applied compressive st | ress | $f_c = C / A_e =$ | | | | | | |
| 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 | | | | | | |
| | | fc / Fc' = 0.4 |)4 | | | | | |
| | PASS - | Design compres | sive stress ex | ceeds maxim | um applied com | pressive str | | |
| 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 | on | P = (0.6 × | (D + S _{wt} × h)) > | × b / 2 + -1 × | $0.6 \times W_{ch2} = 6.1$ | 17 kips | | |
| Maximum tensile force in chord | | $T = V \times h / ((C_o \times \Sigma L_i)) - P = 4.031 \text{ kips} - \text{NEGATIVE. NO}$ | | | | | | |
| Maximum applied tensile stress | | f _t = T / A _{en} = -299 lb/in ² OVERTURNING | | | | | | |
| Design tensile stress | | F_t = $F_t \times C_D$ | imes C _{Mt} $	imes$ C _{tt} $	imes$ C _F | $t_t 	imes C_i = 1508$ lk | o/in² | | | |
| | | ft / Ft' = -0.1 9 | 98 | | | | | |
| | | PASS - Des | ign tensile str | ess exceeds l | maximum applie | d tensile str | | |
| Load combination 1 | | | | | | | | |
| Shear force for maximum compre | | $V = 0.6 \times W$ | • | | | | | |
| Axial force for maximum comp | | | $S_{wt} \times h)) \times s / 2$ | | = 3.372 kips | | | |
| Maximum compressive force in ch | | | $(C_o \times \Sigma L_i)) + P =$ | 5.459 kips | | | | |
| Maximum applied compressive st | ress | $f_c = C / A_e =$ | | | | | | |
| Design compressive stress | | | $0 \times C_{Mc} \times C_{tc} \times C_{tc}$ | $C_{Fc} \times C_i \times C_P =$ | 818 lb/in ² | | | |
| | | f _c / F _c ' = 0.4 |)4 | | | | | |





Maximum force in collector

 $v_b = v_a \times b / (b_1 + b_2) = 219.8 \text{ plf}$ P_{coll} = **0.322** kips

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|--|---|--|--------------------|---------------------|----------------|---------------|--|--|--|
| | Section | | Sheet no./rev 5 | Sheet no./rev. 5 | | | | | |
| | Calc. by S | Date 8/1/2022 | Chk'd by | Date | App'd by | Date | | | |
| Maximum applied tensile stress | $f_t = P_{coll} / (2 \times A_s) = 19 \text{ lb/in}^2$ EAST, LEVEL | | | | | | | | |
| Design tensile stress | | $F_{t}' = F_{t} \times C_{D} \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_{i} = 1508 \text{ lb/in}^{2}$ | | | | | | | |
| | | f _t / F _t ' = 0.013 | | | | | | | |
| | | PASS - Design tensile stress exceeds maximum applied tensile stre | | | | | | | |
| Maximum applied compressive s | stress | $f_{c} = P_{coll} / (2 \times A_{s}) = 19 \text{ lb/in}^{2}$ $C_{P} = 1.00$ | | | | | | | |
| Column stability factor | | | | | | | | | |
| Design compressive stress | | $F_{c}' = F_{c} \times C_{D} \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_{i} \times C_{P} = 2376 \text{ lb/in}^{2}$ | | | | | | | |
| | | f _c / F _c ' = 0.008 | | | | | | | |
| | PASS - | Design compres | ssive stress ex | ceeds maxim | um applied con | pressive stre | | | |
| Wind load deflection | | | | | | | | | |
| Design shear force | | $V_{\delta w} = f_{Wserv} \times W = 8.2 \text{ kips}$ | | | | | | | |
| Deflection limit | | $\Delta_{w_{allow}} = h / 500 = 0.246$ in | | | | | | | |
| Induced unit shear | | $v_{\delta w_{max}} = V_{\delta w} / (C_o \times \Sigma L_i) = 339.26 \text{ Ib/ft}$ | | | | | | | |
| Anchor tension force | | T_{δ} = max(0 kips, v_{\delta w_max} \times h - 0.6 \times (D + S_{wt} \times h) \times b / 2 + 0.6 \times (D + S_{wt} \times h) \times | | | | | | | |
| | max(abs(W _{ch1}),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 E_b) + v_{\delta w_max} \times h / (G_a) + h \times T_\delta / (K_a \times E_b) + v_{\delta w_max} \times h / (G_a) + h \times T_\delta / (K_a \times E_b) + v_{\delta w_max} \times h / (G_a) + h \times T_\delta / (K_a \times E_b) + v_{\delta w_max} \times h / (G_a) + h \times T_\delta / (K_a \times E_b) + v_{\delta w_max} \times h / (G_a) + h \times T_\delta / (K_a \times E_b) + v_{\delta w_max} \times h / (G_a) + h \times T_\delta / (K_a \times E_b) + v_{\delta w_max} \times h / (G_a) + h \times T_\delta / (K_a \times E_b) + v_{\delta w_max} \times h / (G_a) + h \times T_\delta / (K_a \times E_b) + v_{\delta w_max} \times h / (G_a) + h \times T_\delta / (K_a \times E_b) + v_{\delta w_max} \times h / (G_a) + h \times T_\delta / (K_a \times E_b) + v_{\delta w_max} \times h / (G_a) + h \times T_\delta / (K_a \times E_b) + v_{\delta w_max} \times h / (G_a) + h \times T_\delta / (K_a \times E_b) + v_{\delta w_max} \times h / (G_a) + h \times T_\delta / (K_a \times E_b) + v_{\delta w_max} \times h / (G_a) + h \times T_\delta / (K_a \times E_b) + v_{\delta w_max} \times h / (G_a) + h \times T_\delta / (K_a \times E_b) + v_{\delta w_max} \times h / (G_a) + h \times T_\delta / (K_a \times E_b) + v_{\delta w_max} \times h / (G_a) + h \times T_\delta / (K_a \times E_b) + v_{\delta w_max} \times h / (G_a) + h \times T_\delta / (K_a \times E_b) + v_{\delta w_max} \times h / (K_a \times E_b) + v_{\delta w_max} \times h / (K_a \times E_b) + v_{\delta w_max} \times h / (K_a \times E_b) + v_{\delta w_max} \times h / (K_a \times E_b) + v_{\delta w_max} \times h / (K_a \times E_b) + v_{\delta w_max} \times h / (K_a \times E_b) + v_{\delta w_max} \times h / (K_a \times E_b) + v_{\delta w_max} \times h / (K_a \times E_b) + v_{\delta w_max} \times h / (K_a \times E_b) + v_{\delta w_max} \times h / (K_a \times E_b) + v_{\delta w_max} \times h / (K_a \times E_b) + v_{\delta w_max} \times h / (K_a \times E_b) + v_{\delta w_max} \times h / (K_a \times E_b) + v_{\delta w_max} \times h / (K_a \times E_b) + v_{\delta w_max} \times h / (K_a \times E_b) + v_{\delta w_max} \times h / (K_a \times E_b) + v_{\delta w_max} \times h / (K_a \times E_b) + v_{\delta w_max} \times h / (K_a \times E_b) + v_{\delta w_max} \times h / (K_a \times E_b) + v_{\delta w_max} \times H / (K_a \times E_b) + v_{\delta w_max} \times h / (K_a \times E_b) + v_{\delta w_max} \times h / (K_b \times E_b) + v_{\delta w_max} \times h / (K_b \times E_b) + v_{\delta w_max} \times h / (K_b \times E_b) + v_{\delta w_max} \times h / (K_b \times E_b) + v_{\delta w_max} \times h / (K_b \times E_b) + v_{\delta w_max} \times h / (K_b \times E_b) + v_{\delta w_max} \times h / (K_b \times E_b) + v_{\delta w_max} \times h / (K_b \times E_b) + v_{\delta w_max} \times h / (K_b \times E_b) + v_{\delta w_max} \times h / (K_b \times E_b) + v_{\delta w_max} \times h / (K_b \times E_b) + v_{\delta w_ma$ | | | | | | | | |
| | | ΣL _i) = 0.222 in | | | | | | | |
| | | $\delta_{ m sww}$ / $\Delta_{ m w}$ allov | v = 0.903 | | | | | | |
| | | PASS - Shear wall deflection is less than deflection lin | | | | | | | |

PASS - Shear wall deflection is less than deflection limit

REFERENCES



HDU/DTT

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 | | | Di | mensio (in.) | ns | | | Fasteners (in.) | Minimum Wood | All | Code | | | | | | | | | | | | |
|----|----------------|-----|------|----------------------|-----------------|----------------|-----------|------------------------------|--------------------|-------------------------|----------|--------|--|--------|--|--|--|--|----------------|--------|-------|-------|-------|--|
| | No. | Ga. | w | H | в | CL | S0 | Anchor Bolt Dia. (in.) | It Dia. Eastoners | Member Size (in.) | DF/SP | SPF/HF | Deflection at Allowable Load (in.) | Ref. | | | | | | | | | | |
| | | | | | | | | | (6) #9 x 1 ½" SD | | 840 | 840 | 0.17 | | | | | | | | | | | |
| | DTT1Z | 14 | 1½ | 71⁄8 | 17/16 | 3⁄4 | 3∕16 | 3⁄8 | (6) 0.148 x 1 ½ | 1½x5½ | 910 | 640 | 0.167 | | | | | | | | | | | |
| | | | | | | | | | (8) 0.148 x 1 ½ | | 910 | 850 | 0.167 | | | | | | | | | | | |
| SS | DTT2Z | | 31⁄4 | 6 ¹⁵ /16 | | | 3∕16 | | (8) ¼ x 1½ SDS | 1½x3½ | 1,825 | 1,800 | 0.105 | | | | | | | | | | | |
| | DITZZ | 14 | | | 1% | 13/16 | | 3∕16 | 1/2 | (8) ¼ x 1½ SDS | 3 x 31⁄2 | 2,145 | 1,835 | 0.128 | | | | | | | | | | |
| SS | DTT2Z-SDS2.5 | | | | | | | | | | | | | | | | | | (8) ¼ x 2½ SDS | 3 x 3½ | 2,145 | 2,105 | 0.128 | |
| | HDU2-SDS2.5 | 14 | 3 | 811/16 | 3¼ | 15⁄16 | 1% | 5%8 | (6) ¼ x 2½ SDS | 3 x 31⁄2 | 3,075 | 2,215 | 0.088 | IBC, | | | | | | | | | | |
| | HDU4-SDS2.5 | 14 | 3 | 10 ¹⁵ /16 | 3¼ | 1 <i>⁵</i> ⁄i6 | 1% | 5⁄8 | (10) ¼ x 2 ½ SDS | 3 x 31⁄2 | 4,565 | 3,285 | 0.114 | FL, LA | | | | | | | | | | |
| | HDU5-SDS2.5 | 14 | 3 | 13¾6 | 3¼ | 15⁄16 | 1% | 5∕8 | (14) ¼ x 2½ SDS | 3 x 31⁄2 | 5,645 | 4,340 | 0.115 | | | | | | | | | | | |
| | | | | | | | | | | 3 x 3½ | 6,765 | 5,820 | 0.11 | | | | | | | | | | | |
| | HDU8-SDS2.5 10 | 10 | 3 | 16% | 3½ | 1% | 1½ | 7∕8 | (20) ¼ x 2½ SDS | 31⁄2 x 31⁄2 | 6,970 | 5,995 | 0.116 | | | | | | | | | | | |
| | | | | | | | | | | 31⁄2 x 41⁄2 | 7,870 | 6,580 | 0.113 | | | | | | | | | | | |
| | HDU11-SDS2.5 | 10 | 3 | 221/4 | 31/2 | 13% | 1½ | 1 | (20) 16 × 216 CDC | 31⁄2 x 51⁄2 | 9,535 | 8,030 | 0.137 | | | | | | | | | | | |
| | NUTI-3032.3 | 10 | 3 | 22.74 | 372 | 178 | 1 72 | ' I | (30) ¼ x 2½ SDS | 31⁄2 x 71⁄4 | 11,175 | 9,610 | 0.137 | | | | | | | | | | | |
| | | | | | | | | | | 3½ x 5½ | 10,770 | 9,260 | 0.122 | — | | | | | | | | | | |
| | HDU14-SDS2.5 | 7 | 3 | 25 ¹ %6 | 3½ | 1%6 | 1%6 | 1 | (36) ¼ x 2½ SDS | 3½x7¼ | 14,390 | 12,375 | 0.177 | IBC, | | | | | | | | | | |
| | | | | | | | | | | 51⁄2 x 51⁄2 | 14,445 | 12,425 | 0.172 | FL, LA | | | | | | | | | | |

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.

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

SIMPSON Strong-Tie

Strap Ties (cont.)

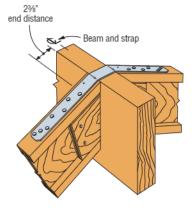
Codes: See p. 11 for Code Reference Key Chart

These products are available with additional corrosion protection. For more information, see p. 14.

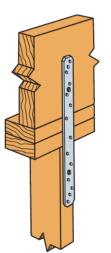
SS For stainless-steel fasteners, see p.21.

Many of these products are approved for installation SD with Strong-Drive[®] SD Connector screws. See pp. 348–352 for more information.

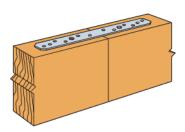
| Model No. | | Ga. | | nsions n.) | Fasteners (Total) (in.) | Allowable Tension Loads (DF/SP) | Allowable Tension Loads (SPF/HF) | Code Ref. | | |
|--------------|---------|-----|-------|---------------|-------------------------------|---------------------------------------|--|--------------|--|--|
| | | | W | L | | (160) | (160) | | | |
| ľ | ST2115 | | 3⁄4 | 16% | (10) 0.162 x 21⁄2 | 660 | 660 | | | |
| | LSTA9 | | 11⁄4 | 9 | (8) 0.148 x 21⁄2 | 740 | 635 | | | |
| | LSTA12 | 1 | 11⁄4 | 12 | (10) 0.148 x 21⁄2 | 925 | 795 | | | |
| | LSTA15 | 20 | 11⁄4 | 15 | (12) 0.148 x 21⁄2 | 1,110 | 955 | | | |
| | LSTA18 | 1 | 11⁄4 | 18 | (14) 0.148 x 21⁄2 | 1,235 | 1,115 | | | |
| [| LSTA21 | | 1 1⁄4 | 21 | (16) 0.148 x 21⁄2 | 1,235 | 1,235 | | | |
| [| LSTA24 | | 11⁄4 | 24 | (18) 0.148 x 21⁄2 | 1,235 | 1,235 | | | |
| | LSTA30 | | 11⁄4 | 30 | (22) 0.148 x 21⁄2 | 1,640 | 1,640 | | | |
| | LSTA36 | | 11⁄4 | 36 | (24) 0.148 x 21⁄2 | 1,640 | 1,640 | | | |
| | MSTA9 | | 11⁄4 | 9 | (8) 0.148 x 21⁄2 | 750 | 650 | | | |
| 3 [| MSTA12 | 18 | 11⁄4 | 12 | (10) 0.148 x 21⁄2 | 940 | 810 | | | |
| | MSTA15 | 10 | 11⁄4 | 15 | (12) 0.148 x 21⁄2 | 1,130 | 970 | | | |
| 9 [| MSTA18 | | 11⁄4 | 18 | (14) 0.148 x 21⁄2 | 1,315 | 1,135 | | | |
| | MSTA21 | | 11⁄4 | 21 | (16) 0.148 x 21⁄2 | 1,505 | 1,295 | | | |
| 3 [| MSTA24 | | 11⁄4 | 24 | (18) 0.148 x 21⁄2 | 1,640 | 1,460 | | | |
| | MSTA30 | | 11⁄4 | 30 | (22) 0.148 x 21⁄2 | 2,050 | 1,825 | | | |
| s [| MSTA36 | | 11⁄4 | 36 | (26) 0.148 x 21⁄2 | 2,050 | 2,050 | | | |
| [| MSTA49 | | 11⁄4 | 49 | (26) 0.148 x 21⁄2 | 2,020 | 2,020 | | | |
| [| ST9 | 16 | 11⁄4 | 9 | (8) 0.162 x 21⁄2 | 885 | 765 | | | |
| [| ST12 | | 11⁄4 | 11% | (10) 0.162 x 21⁄2 | 1,105 | 955 | | | |
| [| ST18 | | 11⁄4 | 173⁄4 | (14) 0.162 x 21⁄2 | 1,420 | 1,335 | IBC, | | |
| [| ST22 | | 11⁄4 | 21% | (18) 0.162 x 21⁄2 | 1,420 | 1,420 | FL, LA | | |
| | HRS6 | | 1% | 6 | (6) 0.148 x 21⁄2 | 605 | 530 | | | |
| | HRS8 | 12 | 1% | 8 | (10) 0.148 x 21⁄2 | 1,010 | 880 | | | |
| | HRS12 | | 1% | 12 | (14) 0.148 x 21⁄2 | 1,415 | 1,230 | | | |
| [| ST292 | | 21/16 | 95/16 | (12) 0.162 x 21⁄2 | 1,260 | 1,120 | | | |
| [| ST2122 | 20 | 21/16 | 12 13/16 | (16) 0.162 x 21⁄2 | 1,530 | 1,510 | | | |
| | ST2215 | | 21⁄16 | 16% | (20) 0.162 x 21⁄2 | 1,875 | 1,875 | | | |
| [| ST6215 | 16 | 21/16 | 16% | (20) 0.162 x 21⁄2 | 2,090 | 1,910 | | | |
| [| ST6224 | 10 | 21⁄16 | 235/16 | (28) 0.162 x 21⁄2 | 2,535 | 2,535 | | | |
| [| ST6236 | 14 | 21⁄16 | 3313/16 | (40) 0.162 x 21⁄2 | 3,845 | 3,845 | | | |
| • [| MSTI26 | | 21⁄16 | 26 | (26) 0.148 x 11⁄2 | 2,745 | 2,380 | | | |
| | MSTI36 | | 21⁄16 | 36 | (36) 0.148 x 11⁄2 | 3,800 | 3,295 | | | |
| | MSTI48 | 12 | 21⁄16 | 48 | (48) 0.148 x 11⁄2 | 5,070 | 4,390 | | | |
| | MSTI60 | | 21⁄16 | 60 | (60) 0.148 x 11⁄2 | 5,070 | 5,070 | | | |
| | MSTI72 | | 21⁄16 | 72 | (72) 0.148 x 11⁄2 | 5,070 | 5,070 | | | |
| | HTP37Z | | 3 | 7 | (20) 0.148 x 11⁄2 | 900 | 690 | | | |
| [| MSTC28 | 16 | 3 | 281⁄4 | (36) 0.148 x 31⁄4 | 3,460 | 2,990 | | | |
| [| MSTC40 | 10 | 3 | 401⁄4 | (52) 0.148 x 31⁄4 | 4,735 | 4,315 | | | |
| Ì | MSTC52 | | 3 | 521⁄4 | (62) 0.148 x 31⁄4 | 4,735 | 4,735 | | | |
| Ĩ | MSTC66 | 14 | 3 | 65¾ | (76) 0.148 x 31⁄4 | 5,850 | 5,850 | | | |
| Ì | MSTC78 | 14 | 3 | 77¾ | (76) 0.148 x 31⁄4 | 5,850 | 5,850 | | | |
| | HRS416Z | 12 | 31⁄4 | 16 | (16) 1/4 x 1 1/2 SDS | 2,835 | 2,305 | | | |
| | LSTI49 | 40 | 3¾ | 49 | (32) 0.148 x 11/2 | 2,970 | 2,560 | IBC, | | |
| Ì | LSTI73 | 18 | 3¾ | 73 | (48) 0.148 x 11/2 | 4,205 | 3,840 | FL, LA | | |



Typical LSTA Installation (hanger not shown) Bend strap one time only, max. 12/12 joist pitch.



Typical LSTA18 Installation



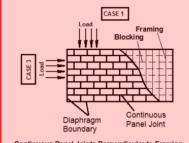
Typical MSTA15 Installation

1. See pp. 266-267 for Straps and Ties General Notes.

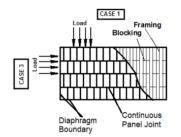
Fasteners: Nail dimensions are listed diameter by length. SDS screws are Simpson Strong-Tie[®] Strong-Drive SDS Heavy-Duty Connector screws. See pp. 21–22 for fastener information.

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 (pif)^{12,3,4,5,6,7,8,9}

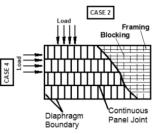
| NOMINAL NAIL | | | BLOCKED DIAPHRAGMS | | | | | | | | UNBLOCKED DIAPHRAGMS | | | | | |
|--|--|---------------------|---|----------------------------------|--------------|--------------|--------------------|---|---------------------------|--------------|--|--|-----------------------------------|------------|--|--|
| DIAMETER (inch) or | MINIMUM | MINIMUM WIDTH OF | 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) | | | | | | | | | FASTENERS SPACED 6" MAX. AT SUPPORTED EDGES | | | | |
| STAPLE GAGE | FASTENER | FRAMING | | 6 | 4 | l I | 2 | 1/2 | : | 2 | | 1 (No d edges or | | other | | |
| Nails must be smooth or deformed and must be carbon | LENGTH | MEMBER | | Nail spacing at other panel edge | | | | | edges (Cases 1, 2, 3 & 4) | | | | configurations (Cases 2, 3, 4, | | | |
| steel (bright or galvanized). | (inches) | (inches) | | 6 | 6 | 5 | 4 | | 3 | | continuous joints parallel to load) | | 5 & 6) | | | |
| | | | Seismic | Wind | Seismic | Wind | Seismic | Wind | Seismic | Wind | Seismic | Wind | Seismic | Wind | | |
| | | | | ³ /8-i | inch Nominal | Panel Thick | iness | | | | | | | | | |
| 0.131 | 13/4 | 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 | 13/4 | 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 Nomina | Panel Thic | kness | | | | | | | | | |
| 0.131 | 2 | 23 | 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 | 23 | 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 Nomina | I Panel Thic | kness | | | | | | | | | |
| 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 | 11/2 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 -i | inch Nominal | Panel Thick | ness ¹⁰ | | | | | | | | | |
| 0.148 | 21/4 | 2 | 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 | 21/4 | 2 3 | 285 320 | 395 450 | 375 425 | 525 595 | | STANDARD ALLOWABLE DIAPHRAGM VALUE (84 GUN NAILS) REDUCE BY SPECIFIC GRAVITY FACTOR FOR HEM FIR 395 215 | | | | | | 265 295 | | |
| 0.131 | 21/4 | 23 | 270 305 | 375 425 | 360 405 | 500 565 | (PER TA HEM FIF | (PER TABLE 6, FOOTNOTE 4) HEM FIR, G = 0.43 100 255 | | | | | | | | |
| 0.120 | 21/4 | 23 | 235 260 | 325 365 | 310 350 | 435 490 | | (1-(0.5-0.43)) = 0.93 255 PLF (0.93) = 237.2 PLF 5 322 175 5 322 175 | | | | | | 220 245 | | |
| 0.113 | 21/4 | 23 | 210 240 | 295 335 | 280 315 | 395 445 | 42.0 | 000 | 400 | | | 265 295 | 140 160 | 200 220 | | |
| 14, 15, 16 Gage | 1 ¹ / ₂ Leg Length | 2 | 175 200 | 245 280 | 235 265 | 330 370 | REDUCE | STANDARD ALLOWABLE DIAPHRAGM VALUE (8d GUN NAILS) 235 100 220 REDUCE BY SPECIFIC GRAVITY FACTOR FOR HEM FIR 5 215 115 160 (PER TABLE 6, FOOTNOTE 4) 5 245 130 180 | | | | | | | | |



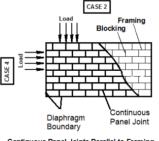
Continuous Panel Joints Perpendicular to Framing Long Panel Direction Perpendicular to Support



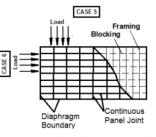
Continuous Panel Joints Perpendicular to Framing Long Panel Direction Parallel to Supports



Continuous Panel Joints Parallel to Framing Long Panel Direction Perpendicular to Supports

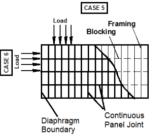


Continuous Panel Joints Parallel to Framing Long Panel Direction Parallel to Supports



195 PLF (0.93) = 181.4 PLF

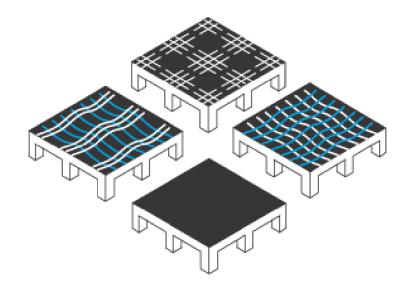
Continuous Panel Joints Perpendicular and Parallel To Framing Long Panel Direction Perpendicular to Supports



Continuous Panel Joints Perpendicular and Parallel To Framing Long Panel Direction Parallel to Supports

FOUNDATION DESIGN

STRUCTURAL SLAB



Basecamp TH1 Slab and Foundation.cpt 9/6/2022

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Materials

Concrete Mix

| <i>Mix Name</i> | Density | Density For | f'ci (nci) | f'c (nci) | fcui (nci) | fcu (nci) | Poissons | Thermal Ex Coeff | r | User Eci | User Ec |
|---------------------|---------|-------------|---------------|--------------|---------------|--------------|----------|---------------------|---------|----------|---------|
| Name | (pcf) | Loads (pcf) | (psi) | (psi) | (psi) | (psi) | Ratio | COEII | Ec Calc | (psi) | (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 | Туре | Aps (in²) | Eps (ksi) | fse (ksi) | fpy (ksi) | fpu (ksi) | Duct Width (inches) | Strands Per Duct | Min Radius (feet) |
|----------------|----------|--------------|--------------|--------------|--------------|--------------|------------------------|---------------------|----------------------|
| 1/2" Unbonded | unbonded | 0.153 | 28000 | 175 | 243 | 270 | 0.5 | 1 | 6 |
| 1/2" 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) |
|----------------|-------------------------|--------------------------|--------------------|-----------------------------|---------------------------------|---------------------------|
| 1/2" Unbonded | 216 | 0.25 | 0 | 0.0014 | 0.07 | 22 |
| 1/2" 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

| | | - | - | | <i>c</i> | 0011 | 100 11 1 |
|---------------------|-------------|-------------|-------------|---------|---------------------------|------------------|-------------------|
| <i>Bar Name</i> | As (in²) | Es (ksi) | Fy (ksi) | Coating | <i>Straight Ld/Db</i> | 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 | o-S0ze217 | 1.096 | 0.5906 | None | 72.52 | 0.03937 | 2 | Rail |
| Ancon Shearfix 10 r | nn0.1217 | 1.096 | 0.5906 | None | 72.52 | 0.03937 | 2 | Rail |
| Ancon Shearfix 12 r | mm0.1753 | 1.578 | 0.5906 | None | 72.52 | 0.03937 | 2 | Rail |
| Ancon Shearfix 14 r | nn0.2386 | 2.147 | 0.5906 | None | 72.52 | 0.03937 | 2 | Rail |
| Ancon Shearfix 16 r | nn0.3116 | 2.805 | 0.5906 | None | 72.52 | 0.03937 | 2 | Rail |
| Ancon Shearfix 20 r | nn 0. 4869 | 4.383 | 0.5906 | None | 72.52 | 0.03937 | 2 | Rail |
| Ancon Shearfix 24 r | mm0.7012 | 6.311 | 0.5906 | None | 72.52 | 0.03937 | 2 | Rail |

Basecamp TH1 Slab and Foundation.cpt - 9/6/2022

Loadings

| Loading Name | Туре | 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 Designal 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 Designation 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 Design 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 Designal 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 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 | 0.75 | 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 Design

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 |

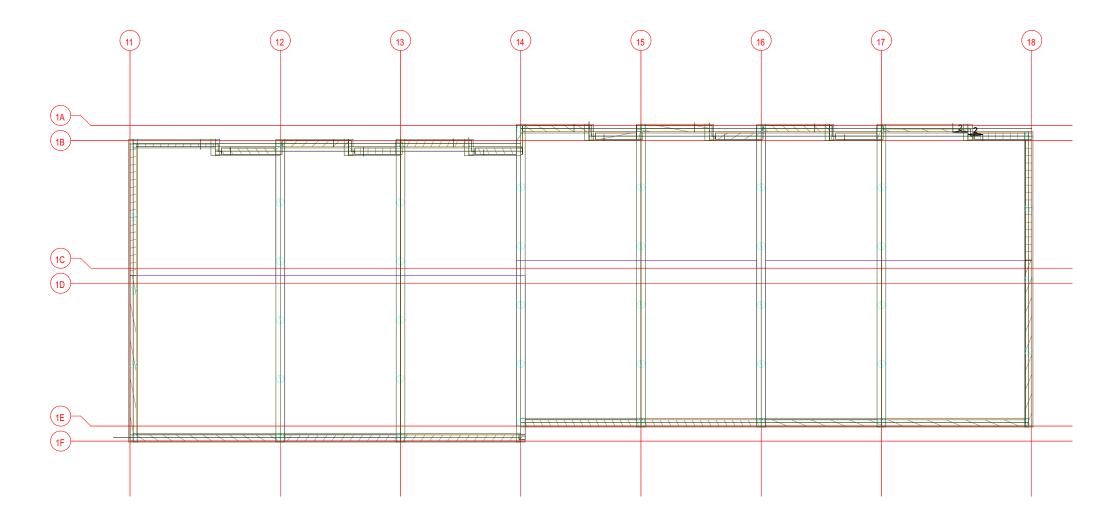
Load Combinations (6)

Factored LC: 1.2D + f1L + 1.6S

Active Design Criteria: User Minimum Design, Code Minimum Design, Strength Design, Ductility Des Analysis: Linear

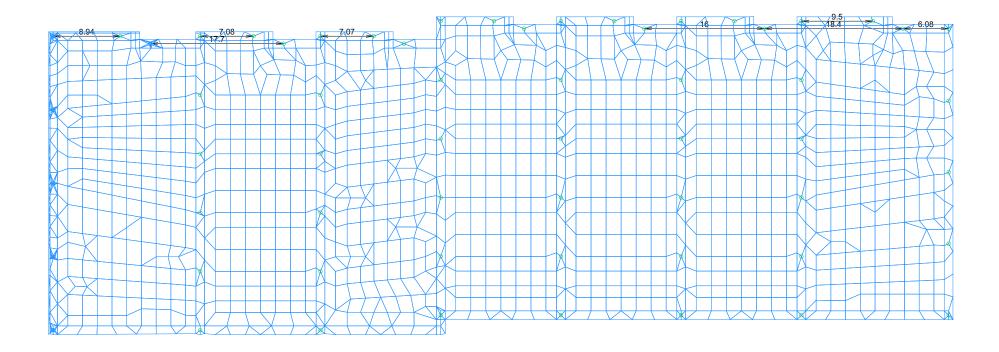
| Loading | Standard Factor | Alt. Envelope Factor |
|----------------------------|-----------------|----------------------|
| Self-Dead Loading | 1.2 | 0.9 |
| Hyperstatic Loading | 1 | 1 |
| Other Dead Loading | 1.2 | 0.9 |
| Live (Reducible) Loading | 0.5 | 0 |
| Live (Unreducible) Loading | 1 | 0 |
| 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; Scale = 1:153



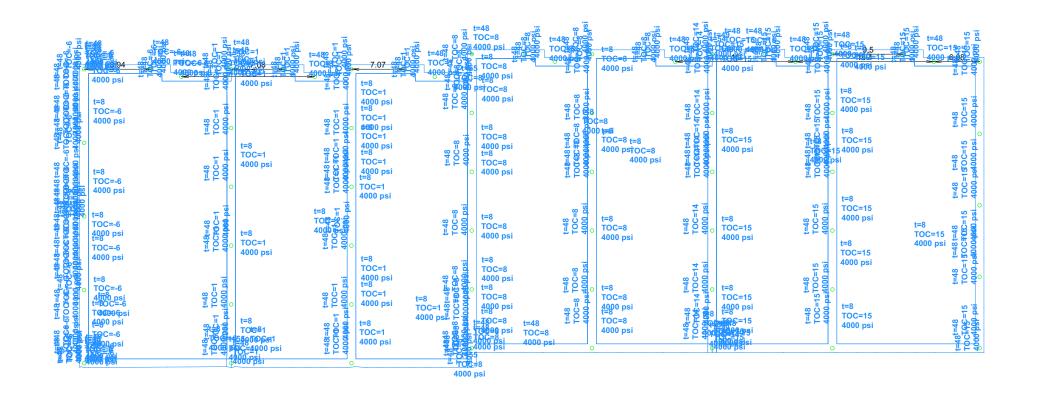
Element: Standard Plan

Element: User Lines; User Notes; User Dimensions; Wall Elements Below; Wall Elements Above; Wall Element Groups Above; Wall Element Group Axes; Column Elements Below; Column Elements Above; Slab Elements; Point Springs; Point Spring Icons; Line Spring Icons; Area Spring Scale = 1:153



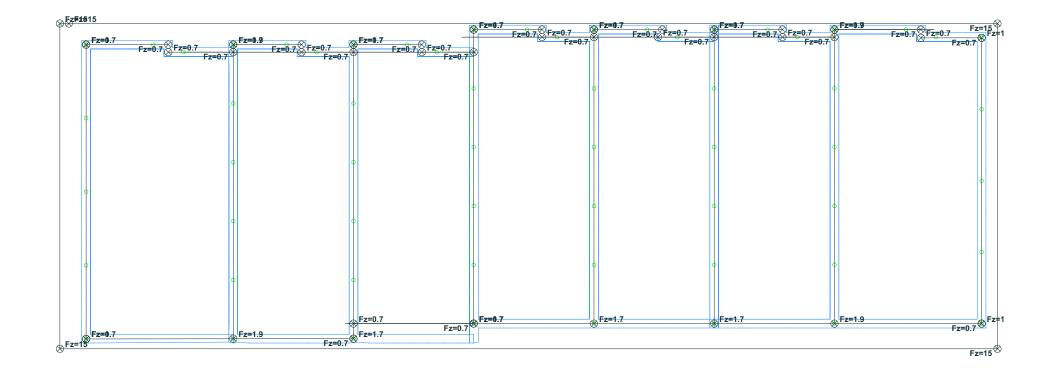
Element: Slab Summary Plan

Element: User Lines; User Notes; User Dimensions; Wall Elements Below; Wall Elements Above; Column Elements Above; Point Springs; Point Springs; Line Springs; Line Springs; Slab Elements; Slab Element Outline Only; Slab Element Thicknesses; Slab Element Elevations; Slab Element Constructions; Slab Elements; Slab Element State Element State Element Elevations; Slab Element State Elements; Slab Elements; Slab Element State Element State Element Elevations; Slab Element State Elements; Slab Element State Elements; Slab Element State Element Stat



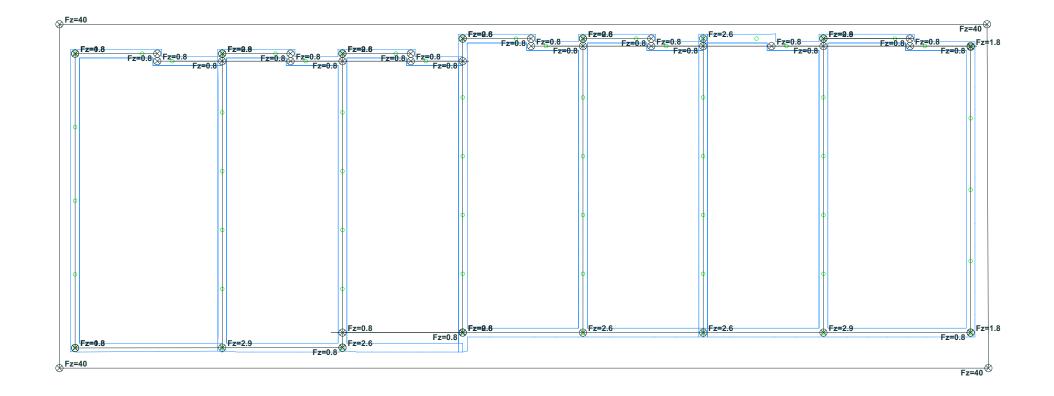
Other Dead Loading: All Loads Plan

Other Dead Loading: User Lines; User Notes; User Dimensions; Point Loads; Point Load Icons; Point Load Values; Line Load Icons; Line Load Icons; Line Load Values; Area Load Icons; Area Load Icons; Area Load Values; Element: Wall Elements Below; Wall Elements Above; Wall Element Outline Only; Column Elements Below; Column Elements Above; Slab Element; Slab Element Outline Only; Scale = 1:153



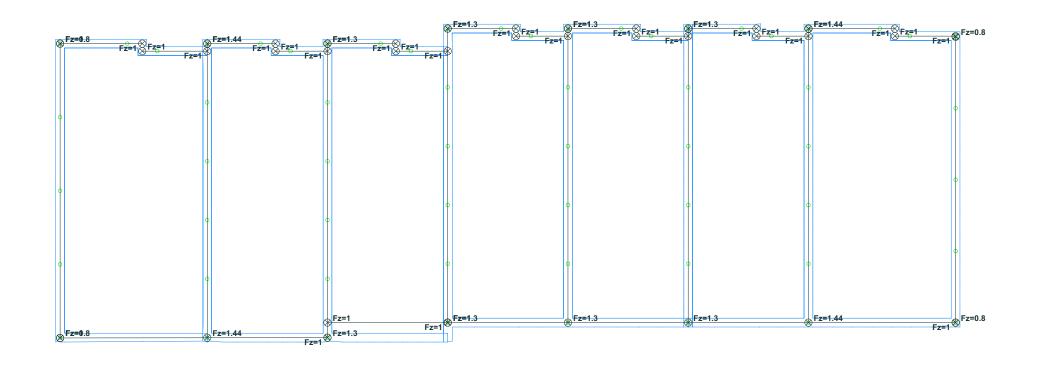
Live (Unreducible) Loading: All Loads Plan

Live (Unreducible) Loading: User Lines; User Notes; User Dimensions; Point Loads; Point Load Icons; Point Load Values; Line Load S; Line Load Icons; Line Load Values; Area Load S; Area Load Icons; 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; Scale = 1:153

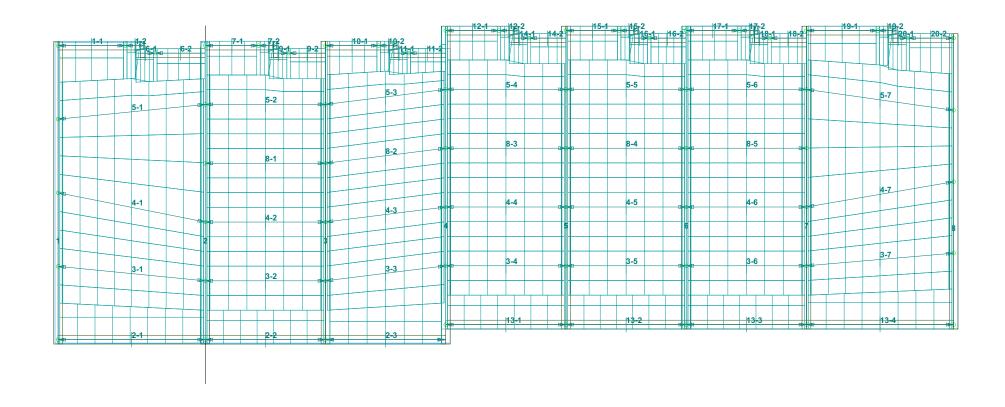


Snow Loading: All Loads Plan

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 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; Scale = 1:153

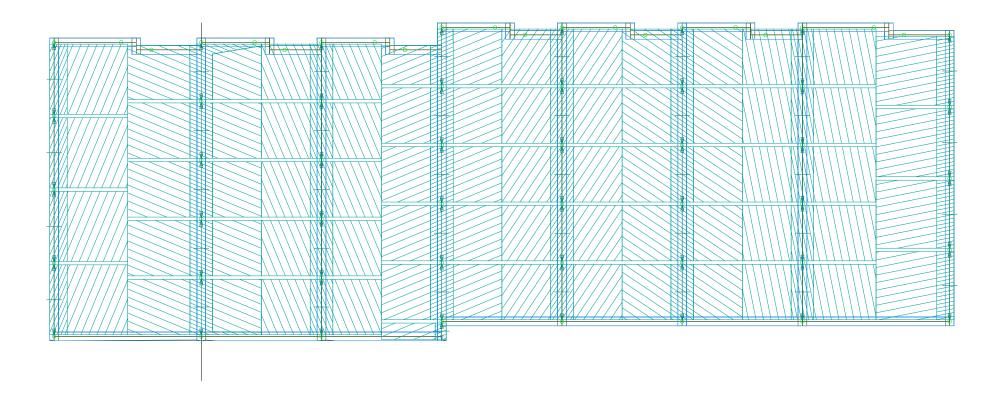


Design Strip: Latitude Span Boundaries; Latitude SSS; SS Numbers; Latitude DSS; DS Numbers; Latitude Strip Boundaries; Latitude SSSS; SSS Internal Sections; User Notes; User Lines; User Dimensions; Mesh Input: Beams; Slab Areas; Columns Below; Scale = 1:153



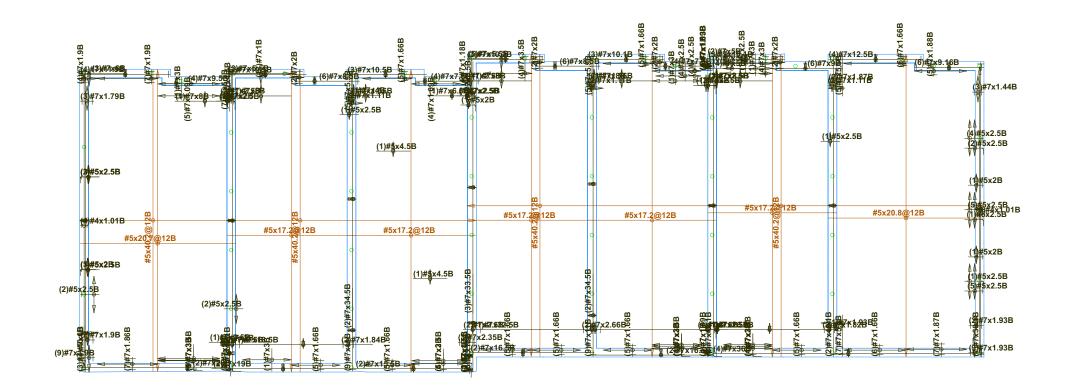
Design Strip: Longitude Design Spans Plan Design Strip: Longitude Span Boundaries; Longitude SS; Longitude DS; Longitude Strip Boundaries; Longitude SSS; SSS Hatching; User Notes; User Lines; User Dimensions;

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; Scale = 1:153



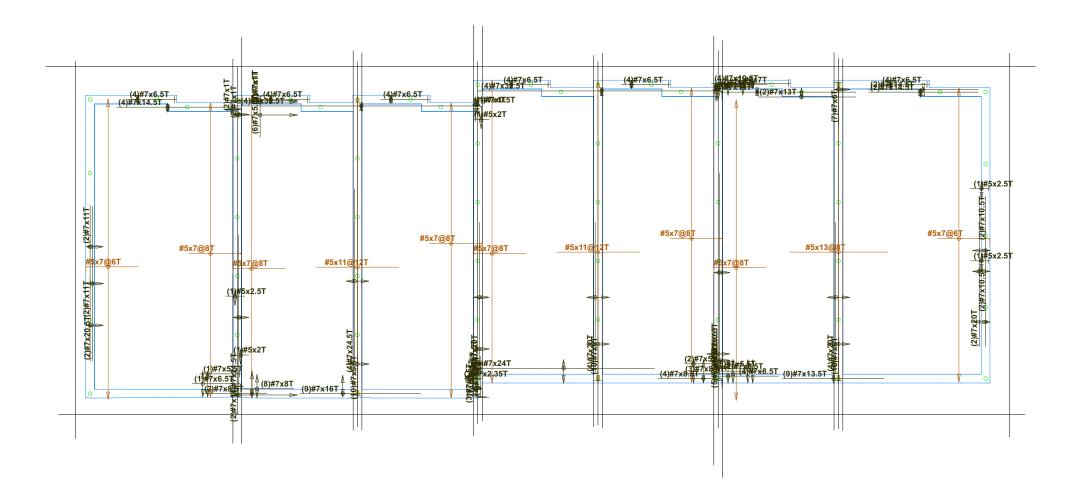
Reinforcement: Bottom Bars Plan

Reinforcement: Latitude User Concentrated Reinf.; Longitude User Concentrated Reinf.; Latitude Program Concentrated Reinf.; Bottom Face Concentrated Reinf.; Latitude User Distributed Reinf.; Longitude User Distributed Reinf.; Concentrated R



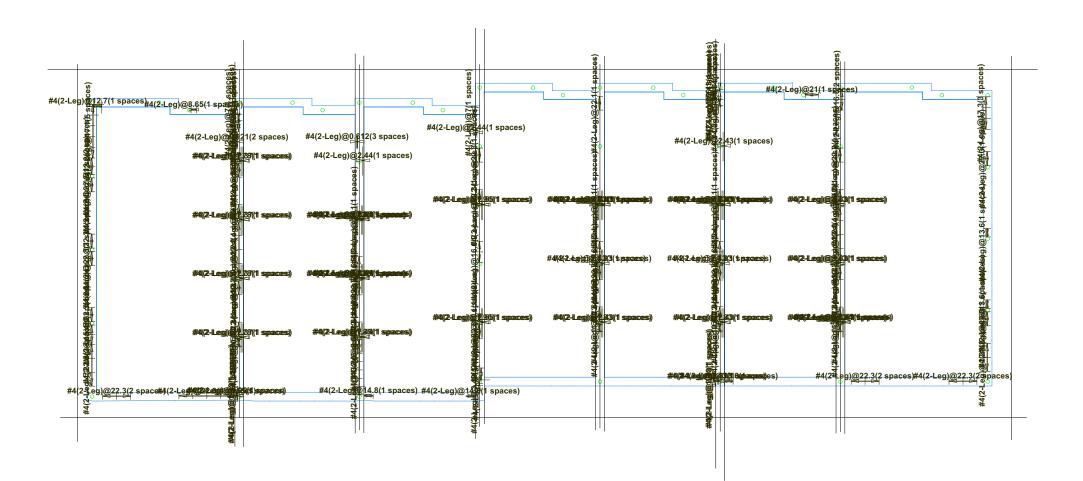
Reinforcement: Top Bars Plan

Reinforcement: Latitude User Concentrated Reinf.; Longitude User Concentrated Reinf.; Latitude Program Concentrated Reinf.; Top Face Concentrated Reinf.; Concentrated Reinf.; Longitude User Distributed Reinf.; Longitude User Distributed Reinf.; Longitude User Distributed Reinf.; Concentrated Reinf.; Concentrated Reinf.; Concentrated Reinf.; Concentrated Reinf. Descriptions; Concentrated Reinf. Extent; Latitude User Distributed Reinf.; Longitude User Distributed Reinf.; Longitude User Distributed Reinf.; Concentrated Reinf.; Longitude User Distributed Reinf.; Concentrated Reinf.; Concentrate



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 Bars; Latitude Program Individual Transverse Bars; Longitude User Transverse Reinf.; Longitude Program Transverse Reinf.; Longitude User Individual Transverse Bars; Latitude Program Individual Transverse Bars; Longitude User Transverse Reinf.; Longitude Program Transverse Reinf.; Longitude User Individual Transverse Bars; Latitude Program Transverse Reinf.; Longitude User Individual Transverse Bars; Latitude Program Individual Transverse Bars; Longitude User Transverse Reinf.; Longitude User Individual Transverse Bars; Latitude Program Individual Transverse Bars; Latitude Program Individual Transverse Bars; Longitude User Transverse Reinf.; Longitude User Individual Transverse Bars; Latitude Program Individual Transverse Bars; Latitude Program Individual Transverse Bars; Latitude Program Individual Transverse Bars; Latitude User Transverse Reinf.; Longitude User Individual Transverse Bars; Latitude Program Individual Transverse Bars; Latitude Prog



Design Status: Status Plan

Design Status: Latitude Span Designs; Longitude Span Design Numbers; Span Design Numbers; Span Design Status; Latitude DS Designs; Longitude DS Designs; DS Design Numbers; DS Design Status; 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 SSS; SSS Internal Sections;

Scale = 1:153

| 21C-4 0K | 1C-1 0K 1R-1 0K 7 2 0 1 1 1 1 1 1 1 1 1 1 1 1 1 | | 6C-2 OK SK -2 SK -2 S | 7C-1 OK 7R-1 0K % V 0 N | 0K 50-2 | 9C-2 OK 0R-2 OK US 0K S 0K S 0K S 0K S 0 S 0 S 0 S 0 S 0 | | 10C-2 OK-1 105/47 5L-357 OR 5C-3 OK | 11L-2 OK 11C-2 11 0 2 0 2 | 12C-1 OK 12R-1 OK | | 14C-2 OK 14R-2 OK 20 X 20 X | 15C-1 OK 15R-1 OK 22 × 25 O 21 | 15C-2 016C-1 15D 0K 5L 51 K 0K 5G-5 0K | 16L-2 16X52 0K 16R-2 0K 22 0K 23 0K 23 0K 23 0K 24 0K 25 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 | 17C-1 0K 17R-1 0K 200 0 0 | 5C-6 OK | 18C-2 OK-2 18R-2 OK9 20 7 | 19C-1 OK 19R-1 40K 70K | 19C-2 044C-1 946 94 044 51-7 04 52-7 04 55 55 57 04 55 55 75 75 75 75 75 75 75 75 | 20C-2 OK 20R-2 OK 4 X 8 0 |
|---------------|--|---|---|--|--|---|-------------|---|--|----------------------------|---|--|--|--|---|---|--|--|------------------------------------|--|---|
| 21C-3 OK 0 | 21R.3 0K | 5E-1 5E-1 0K 5E-1 0K 0K 0K 1 -1 | 22C-4 OK | 22 <u>8 4</u> 0K | OK 5R-2 8L-1 1 OK 8 8C-1 OK | 23C-4 | 238 4 OK | 5R-3 8L-2-1-20 OK 8C-2 OK 8R-2 0K 8R-2 | 24C-5 | 248-5 0K-5 | 5R-4 8L-37 OK 50 8C-3 OK 8R-3 | 25C-4 0K | 25R4 0K | 5R-5 0K-4 8L-4-1 7 0 80-4 0 0 K 8R-4 8R-4 | 26C-4 | 26R 4 0K | 5R-6 0K 8L-57 0K K 0K 0K 8R-5 | 27C-4 | 27R-4 | -5 R-7 OK 41 <u>-7 ° ×</u> OK [®] 0 OK [®] 0 | 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 |
| 21C-2 2 | 0K-2 | OK 4C-1 127 OK 4R-1 OK | 22 6 -3 0K | 22 8-3 0K | 8R-1 0K 4L-27 0K 7 0K 0K 4C-2 0K 4R-2 | • • • | 238-3 OK | 4L-3 0K 4R-3 | 3 24C-4 | | OK 47 OK 57 OK 57 OK 4C-4 OK 4R-4 OK 4R-4 | 25C-3 | 258.3 0K | QLK357 0K37 4C-5 0K 4R-5 3L-57 | 26C-3 | 26R-3 0K | | 2 27C-3 | 2 27R-3 | 4C-7 OK 4R-7 OK 7 × 3L-7 80 OK | 28C-2 |
| ok c | N | 3L-1 OK 4 2 3C-1 OK 3R-1 | 22G-2 | 0 7 7 7 7 7 7 7 7 7 7 7 7 7 7 7 7 7 7 7 | 4R-2 OK 3L-2N X OK 0K 0K 3C-2 OK 3R-2 | 23C-2 | 238-2 OK | 36-3 OK 3R-3 N | -2 24C-3 | 4 4 5 | OK ⊡ O 3C-4 OK 3R-4 19K-1 | | 258 0K | OK 🤯 Ö 36-5 OK 3R-5 19K2 | | 260 0 0 0 0 0 0 0 0 0 0 | 3C-6 OK 3R-6 19 ^K 3- | 1 | 278 | 0K 3C-7 0K 3R-7 13K4 + 13K4 + 2K = 0 0K = 0 | <u>.</u> |
| 21C-1 | 21R-1 | £ <u>K</u> → K 2¢-1 → K | 22¢-1 | 0 X 1 7 7 7 7 7 | 22 ¹⁶ 2 | 530 0 0 0 0 0 | 238-1 OK | OK-140 2L-370 OK 2C-31- OK 470 OK 470 | 24C-1 24C-: OK OK | | ок , 5 13С-1 ОК | 25C-1 | | ок 13С-2 ОК | | | ОК 5 13С-3 ОК | | 27R | 13C-4 | 0 3 |