KL&A
Engineers & Builders

Steamboat Basecamp Phase I
1901 Curve Plaza
Steamboat Springs, CO 80487

# Structural Calculations

COLORADO LICENSED
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| | |
|---|---|
| Prepared by: | Sina Erturk, PE |
| Date: | 11/15/2021 |

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# 100. BASIS OF DESIGN

**Calculation Package**

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## Calculation Package

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**100. NARRATIVE**

The following sections provide detailed calculations and descriptions of the structural systems for this project.

This project was designed in accordance with the 2018 International Building Code and Routt County 2018 ICC Building Code Adoption and Policies. The following is an overview of the loading used in the design of the structure and the key parameters used to derive the loads.

A. Dead Loads
Detailed information regarding self-weight and superimposed dead load (e.g. MEP, Finishes, insulation, etc.) for each loading case can be found in Section 101. A graphical summary of the extents of applied dead loads can be found in the load keys in the drawing documents.

B. Live Loads
Detailed information regarding live loads for each loading case can be found in Section 101. A graphical summary of the extents of applied live loads can be found in the load keys in the drawing documents. The following typical live loads were used in the design of the structure.

| | |
|---|---|
| Gymnasiums | 100 psf |
| Roof | 20 psf |

C. Snow Loads
Detailed information regarding snow loading, including flat roof snow loading, drifting, and sliding can be found in Section 102. All snow loads on the structure have been calculated in accordance with the 2018 International Building Code and ASCE7-16. The ground snow load value of 105psf used for design is in accordance with the Routt County Regional Building Department 2018 IBC Code Amendments. The loads keys in the drawing documents show a graphical summary of the design snow loading used for the project.

D. Wind Loads:
Detailed information regarding wind loading, including MWFRS loads as well as Components and Cladding pressures can be found in Section 103. Ultimate level wind speed for design is in accordance with the Routt County Regional Building Department 2018 IRC Code Adoption. Wind loads on the structure have been calculated in accordance with the 2018 International Building Code and ASCE 7-16 based on the following criteria:

| | |
|---|---|
| Enclosure classification | Enclosed Building |

# Calculation Package

| | |
|---|---|
| Risk category | II |
| Wind speed | 115 mph |
| Wind directionality factor, Kd | 1.0 |
| Exposure category | C |
| Topographic factor, Kzt | 1 |
| Building flexibility | Flexible |
| Gust effect factor, G | 0.85 |
| Internal pressure coefficient | 0.18 |

Resulting wind loads:

Wind base shear for Infill:

| | |
|---|---|
| East/West | 14.7 kips |
| North/South | 0 kips |

E. Seismic Loads:

F. Detailed information regarding seismic loads can be found in Section 104. Seismic loads on the structure have been calculated based on the Routt County Building Department 2018 IBC Code Amendments. The Routt County Building Department has specified design parameters, Sds = 0.333 and Sd1 = 0.133, designating all of Routt County as Seismic Design Category C. The seismic loads have been determined based on following criteria:

| | |
|---|---|
| $S_S$ | 0.333 |
| $S_1$ | 0.133 |
| Site class | D |
| Period | 0.274 seconds |
| Period determination | Approx per ASCE 7 |
| Long period, $T_L$ | 4 seconds |
| Seismic force resisting system | Light frame wood walls with structural wood shear panels |
| Response modification factor, R | 6.5 |
| Overstrength factor, $\Omega_o$ | 2.5 |
| Deflection amplification factor | 4 |
| Importance factor, $I_e$ | 1.0 |

Resulting Seismic Base Shear:

| | |
|---|---|
| East/West | 7.8 kips |
| North/South | 7.8 kips |

G. Soil Loads and Capacities

Geotechnical criteria for design is based on the report provided by Brian D. Len, PE (PE#25750) of NWCC, Inc, dated 03/15/2021. A complete copy of the geotechnical report can be found in appendix A-100 of this calculation package. Minimum frost depth per the

# Calculation Package

Routt County Building Department 2018 IBC Code Amendments is 48 inches. The following is a summary of the geotechnical design parameters:

Spread Footings:

| | |
|---|---|
| Allowable bearing pressure | 3 ksf |

# Calculation Package

## 101. DEAD AND LIVE LOADS

| Title | Basecamp Phase 1 | Date | 11/11/21 | Job no. | 21304 |
|---|---|---|---|---|---|
| Subject | Load Keys | By | SYE | Sheet | of |

**Load Key**

| Load # | Description of Load | Self Weight | Superimposed Loads | | | Live Load Reduction? | Notes |
|---|---|---|---|---|---|---|---|
| | | | Dead Load | Live Load | Snow Load | | |
| 1 | Roof | 10 | 15 | 20 | 75 | No | |
| 2 | Mezzanine | 11 | 30 | 100 | 0 | No | |
| 3 | Roof w/RTU | 10 | 55 | 20 | 75 | No | |

| | | | | | |
|---|---|---|---|---|---|
| Title | Basecamp Phase 1 | Date | 11/11/21 | Job no. | 21304 |
| Subject | Load Keys | By | SYE | Sheet | of |

## Roof

**Total Superimposed Dead Load:** 15 PSF
**Total Self Weight Dead Load:** 10 PSF

**Live Load:** Roof = 20 PSF

**Snow Load:**
Balanced Snow Load or Roof Live Load: 75 PSF
Do you want to use the snow load to calculate the seismic story weight? yes
Seismic Snow: 15

**Special Load:** = PSF
Note - this does not appear on the Load Key Summary

### Superimposed Dead Load:

| | Category | Material | Thickness (in) | PSF |
|---|---|---|---|---|
| | Ceiling Finishes | Typical Mechanical Duct Allowance | | 4.0 |
| | | | | 0.0 |
| | | | | 0.0 |
| | | | | 0.0 |
| | | | | 0.0 |
| User Input Load | Misc | | | 11.0 |
| User Input Load | | | | |

### Deck/Slab Self Weight:

| | Category | Type | Thickness (in) | PSF |
|---|---|---|---|---|
| | Wood | Plywood Floor Sheathing | 0.625 | 1.9 |
| | | | | 0.0 |
| User Input Load | | | | |

### Framing Self Weight:

| | Category | Member | Spacing (in) | PSF |
|---|---|---|---|---|
| | Open Web Truss | TJM Open Web Truss | 12 | 8.5 |
| | | | | 0.0 |
| User Input Load | | | | |

### Custom Self Weight:

| Description | PSF |
|---|---|
| | |

| Title | Basecamp Phase 1 | Date | 11/11/21 | Job no. | 21304 |
|---|---|---|---|---|---|
| Subject | Load Keys | By | SYE | Sheet | of |

## Mezzanine

**Total Superimposed Dead Load:** 30 PSF
**Total Self Weight Dead Load:** 11 PSF

**Live Load:** Gymnazium = 100 PSF

**Snow Load:**
Balanced Snow Load or Roof Live Load — PSF
Do you want to use the snow load to calculate the seismic story weight?
Seismic Snow 0

**Special Load:** = PSF
Note - this does not appear on the Load Key Summary

**Superimposed Dead Load:**

| | Category | Material | Thickness (in) | PSF |
|---|---|---|---|---|
| | Floor Finishes | Typical Mechanical Duct Allowance | | 4.0 |
| | Topping Slabs | :rete (was 104 pcf-> 120 pcf in recent submit | 1.5 | 15.0 |
| | | | | 0.0 |
| | | | | 0.0 |
| | | | | 0.0 |
| User Input Load | Misc | | | 11.0 |
| User Input Load | | | | |

**Deck/Slab Self Weight:**

| | Category | Type | Thickness (in) | PSF |
|---|---|---|---|---|
| | Wood | Plywood Floor Sheathing | 0.75 | 2.3 |
| | | | | 0.0 |
| User Input Load | | | | |

**Framing Self Weight:**

| | Category | Member | Spacing (in) | PSF |
|---|---|---|---|---|
| | Open Web Truss | TJM Open Web Truss | 12 | 8.5 |
| | | | | 0.0 |
| User Input Load | | | | |

**Custom Self Weight:**

| Description | PSF |
|---|---|
| | |

| Title | Basecamp Phase 1 | Date | 11/11/21 | Job no. | 21304 |
|---|---|---|---|---|---|
| Subject | Load Keys | By | SYE | Sheet | of |

## Roof w/RTU

**Total Superimposed Dead Load:** 55 PSF
**Total Self Weight Dead Load:** 10 PSF

**Live Load:** Roof = 20 PSF

**Snow Load:**
Balanced Snow Load or Roof Live Load: 75 PSF
Do you want to use the snow load to calculate the seismic story weight? yes
Seismic Snow: 15

**Special Load:** = PSF
Note - this does not appear on the Load Key Summary

**Superimposed Dead Load:**

| | Category | Material | Thickness (in) | PSF |
|---|---|---|---|---|
| | Ceiling Finishes | Typical Mechanical Duct Allowance | | 4.0 |
| | | | | 0.0 |
| | | | | 0.0 |
| | | | | 0.0 |
| | | | | 0.0 |
| User Input Load | Misc | | | 11.0 |
| User Input Load | RTU | | | 40.0 |

**Deck/Slab Self Weight:**

| | Category | Type | Thickness (in) | PSF |
|---|---|---|---|---|
| | Wood | Plywood Floor Sheathing | 0.625 | 1.9 |
| | | | | 0.0 |
| User Input Load | | | | |

**Framing Self Weight:**

| | Category | Member | Spacing (in) | PSF |
|---|---|---|---|---|
| | Open Web Truss | TJM Open Web Truss | 12 | 8.5 |
| | | | | 0.0 |
| User Input Load | | | | |

**Custom Self Weight:**

| Description | PSF |
|---|---|
| | |

| | | | | | |
|---|---|---|---|---|---|
| Title | Basecamp Phase 1 | Date | 11/11/21 | Job no. | 21304 |
| Subject | Load Keys | By | SYE | Sheet | of |

**Wall Summary**

| Load # | Description of Load | Self Weight (psf) | Superimposed Dead Load (psf) | Notes |
|---|---|---|---|---|
| 1 | 2x6 Wall | 3 | 8 | |

| | | | | | |
|---|---|---|---|---|---|
| Title | Basecamp Phase 1 | Date | 11/11/21 | Job no. | 21304 |
| Subject | Load Keys | By | SYE | Sheet | of |

## 2x6 Wall

**Total Superimposed Dead Load:** 8
**Total Self Weight Dead Load:** 3

### Superimposed Dead Load:

| | Category | Material | Thickness (in) | PSF |
|---|---|---|---|---|
| | Covered Finishes | Rigid Insulation | 2 | 3.0 |
| | Covered Finishes | Gypsum Board | 0.625 | 2.8 |
| | | | | 0.0 |
| | | | | 0.0 |
| | | | | 0.0 |
| User Input Load | Misc | | | 2.2 |
| User Input Load | | | | |

### Framing Self Weight:

| | Category | Member | Spacing (in) | PSF |
|---|---|---|---|---|
| | DFL | 2x6 | 16 | 1.7 |
| | | | | 0.0 |
| User Input Loads | | | | |

### Solid Wall/Sheathing Self Weight:

| Category | Type | Thickness (in) | PSF |
|---|---|---|---|
| Wood Sheathing | Wood Sheathing | 0.5 | 1.5 |

| | Type | Grout Spacing | Block Size | PSF |
|---|---|---|---|---|
| CMU | | | | 0.0 |

| | Category | Type | Thickness (in) | PSF |
|---|---|---|---|---|
| User Input Loads | | | | |

### Custom Self Weight:

| Description | PSF |
|---|---|
| | |

## 102. SNOW LOADS

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| | | | |
|---|---|---|---|
| **JOB TITLE** | Basecamp Phase 1 | | |
| **JOB NO.** | 21304 | **SHEET NO.** | |
| **CALCULATED BY** | Sina Erturk | **DATE** | 6/18/21 |
| **CHECKED BY** | Jake Sujansky | **DATE** | |

## Snow Loads : ASCE 7- 16

Nominal Snow Forces

| | | | |
|---|---|---|---|
| Roof slope | | = | 14.0 deg |
| Horiz. eave to ridge dist (W) | | = | 136.0 ft |
| Roof length parallel to ridge (L) | | = | 96.0 ft |
| Type of Roof | | | Monoslope |
| Ground Snow Load | Pg | = | 105.0 psf |
| Risk Category | | = | II |
| Importance Factor | I | = | 1.0 |
| Thermal Factor | Ct | = | 1.00 |
| Exposure Factor | Ce | = | 1.0 |
| Pf = 0.7*Ce*Ct*I*Pg | | = | 73.5 psf |
| Unobstructed Slippery Surface | | | no |
| Sloped-roof Factor | Cs | = | 1.00 |
| Balanced Snow Load | | **=** | **73.5 psf** |
| Rain on Snow Surcharge Angle | | | 2.72 deg |
| Code Maximum Rain Surcharge | | | 5.0 psf |
| Rain on Snow Surcharge | | = | 0.0 psf |
| Ps plus rain surcharge | | = | 73.5 psf |
| Minimum Snow Load | Pm | = | 20.0 psf |
| Uniform Roof Design Snow Load | | = | **73.5 psf** use 75.0 |

Near ground level surface balanced snow load = **105.0 psf**

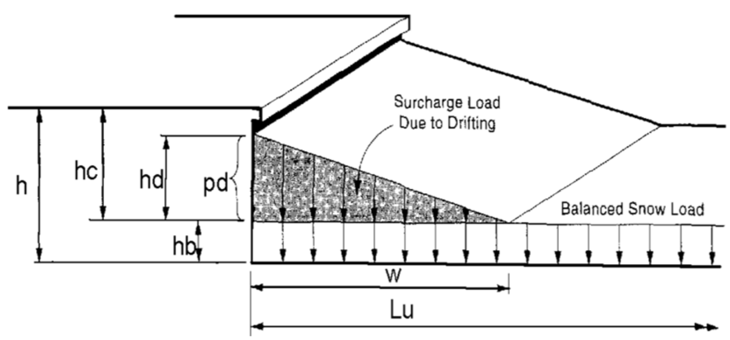
NOTE: Alternate spans of continuous beams shall be loaded with half the design roof snow load so as to produce the greatest possible effect - see code for loading diagrams and exceptions for gable roofs..

**Windward Snow Drifts 1 -** Against walls, parapets, etc

| | | | |
|---|---|---|---|
| Upwind fetch | lu | = | 29.3 ft |
| Projection height | h | = | 4.0 ft |
| Snow density | g | = | 27.7 pcf |
| Balanced snow height | hb | = | 2.66 ft |
| | hd | = | 2.13 ft |
| | hc | = | 1.34 ft |
| hc/hb >0.2 = 0.5 | **Therefore, design for drift** | | |
| Drift height (hc) | | = | 1.34 ft |
| Drift width | w | = | 10.73 ft |
| Surcharge load: | pd = γ*hd | = | **37.1 psf** |
| Balanced Snow load: | | = | 73.5 psf |
| | | | 110.6 psf |

**Windward Snow Drifts 2 -** Against walls, parapets, etc

| | | | |
|---|---|---|---|
| Upwind fetch | lu | = | 52.0 ft |
| Projection height | h | = | 10.0 ft |
| Snow density | g | = | 27.7 pcf |
| Balanced snow height | hb | = | 2.66 ft |
| | hd | = | 2.82 ft |
| | hc | = | 7.34 ft |
| hc/hb >0.2 = 2.8 | **Therefore, design for drift** | | |
| Drift height (hd) | | = | 2.82 ft |
| Drift width | w | = | 11.27 ft |
| Surcharge load: | pd = γ*hd | = | **77.9 psf** |
| Balanced Snow load: | | = | 73.5 psf |
| | | | 151.4 psf |




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

## 103. WIND LOADS

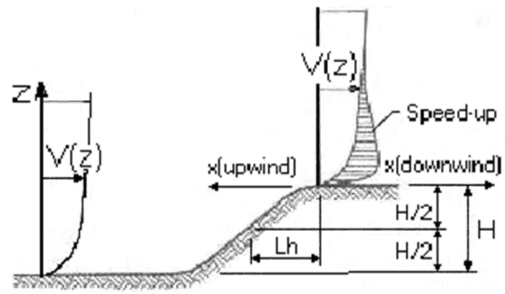
## Wind Loads : ASCE 7- 16

| | |
|---|---|
| Ultimate Wind Speed | 115 mph |
| Nominal Wind Speed | 89.1 mph |
| Risk Category | II |
| Exposure Category | C |
| Enclosure Classif. | Enclosed Building |
| Internal pressure | +/-0.18 |
| Directionality (Kd) | 0.85 |
| Kh case 1 | 0.939 |
| Kh case 2 | 0.939 |
| Type of roof | Gable |

Topographic Factor (Kzt)

| | |
|---|---|
| Topography | Flat |
| Hill Height (H) | 10.0 ft |
| Half Hill Length (Lh) | 10.0 ft |
| Actual H/Lh = | 0.00 |
| Use H/Lh = | 0.00 |
| Modified Lh = | 10.0 ft |
| From top of crest: x = | 10.0 ft |
| Bldg up/down wind? | downwind |

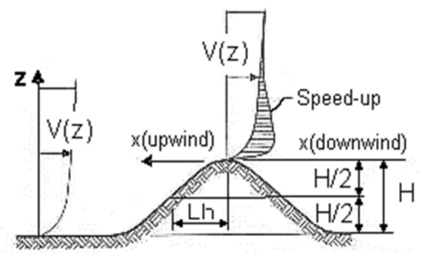
H< 15ft;exp C
∴ Kzt=1.0



**ESCARPMENT**

| | |
|---|---|
| H/Lh= 0.00 | $K_1$ = 0.000 |
| x/Lh = 1.00 | $K_2$ = 0.333 |
| z/Lh = 2.43 | $K_3$ = 1.000 |

At Mean Roof Ht:
$Kzt = (1+K_1K_2K_3)^2 = 1.00$



**2D RIDGE or 3D AXISYMMETRICAL HILL**

## Gust Effect Factor

| | |
|---|---|
| h = | 24.3 ft |
| B = | 136.0 ft |
| /z (0.6h) = | 15.0 ft |

Flexible structure if natural frequency < 1 Hz (T > 1 second).
If building h/B>4 then may be flexible and should be investigated.
h/B = 0.18 Rigid structure (low rise bldg)

**G = 0.85** Using rigid structure default

**Rigid Structure**

| | |
|---|---|
| ē = | 0.20 |
| ℓ = | 500 ft |
| $z_{min}$ = | 15 ft |
| c = | 0.20 |
| $g_Q$, $g_v$ = | 3.4 |
| $L_z$ = | 427.1 ft |
| Q = | 0.86 |
| $I_z$ = | 0.23 |
| G = | 0.85 use G = 0.85 |

**Flexible or Dynamically Sensitive Structure**

| | | | | | |
|---|---|---|---|---|---|
| 34 ıcy ($\eta_1$) = | 0.0 Hz | | | | |
| Damping ratio (β) = | 0 | | | | |
| /b = | 0.65 | | | | |
| /α = | 0.15 | | | | |
| Vz = | 97.1 | | | | |
| $N_1$ = | 0.00 | | | | |
| $R_n$ = | 0.000 | | | | |
| $R_h$ = | 28.282 | η = | 0.000 | h = | 24.3 ft |
| $R_B$ = | 28.282 | η = | 0.000 | | |
| $R_L$ = | 28.282 | η = | 0.000 | | |
| $g_R$ = | 0.000 | | | | |
| R = | 0.000 | | | | |
| Gf = | 0.000 | | | | |

## Enclosure Classification

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

**Test for Open Building:** All walls are at least 80% open.
Ao ≥ 0.8Ag

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

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

Conditions to qualify as Partially Enclosed Building. Must satisfy all of the following:
- Ao ≥ 1.1Aoi
- Ao > smaller of 4' or 0.01 Ag
- Aoi / Agi ≤ 0.20

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

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

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

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

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

## Ground Elevation Factor (Ke)

| | | | | | |
|---|---|---|---|---|---|
| Grd level above sea level = | 6680.0 ft | | | Ke = | 0.7852 |
| Constant = | 0.00256 | Adj Constant = | 0.00201 | | |

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| | | | |
|---|---|---|---|
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| **JOB NO.** | 21304 | **SHEET NO.** | |
| **CALCULATED BY** | Sina Erturk | **DATE** | 6/18/21 |
| **CHECKED BY** | Jake Sujansky | **DATE** | |

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

| | | | | | |
|---|---|---|---|---|---|
| Kh (case 2) = | 0.94 | h = | 24.3 ft | GCpi = | +/-0.18 |
| Base pressure ($q_h$) = | **21.2 psf** | ridge ht = | 32.8 ft | G = | 0.85 |
| Roof Angle (θ) = | 14.0 deg | L = | 96.0 ft | qi = qh | |
| Roof tributary area - (h/2)*L: | 1164 sf | B = | 136.0 ft | | |
| (h/2)*B: | 1649 sf | | | | |

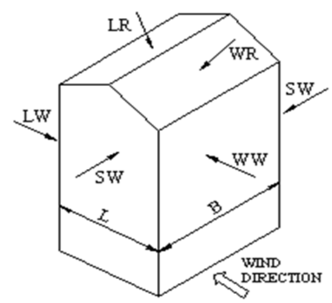
**Ultimate Wind Surface Pressures (psf)**

| | **Wind Normal to Ridge** | | | | **Wind Parallel to Ridge** | | | | |
|---|---|---|---|---|---|---|---|---|---|
| | B/L = 1.42 | | h/L = 0.18 | | | L/B = 0.71 | | h/L = 0.25 | |
| **Surface** | Cp | $q_hGC_p$ | w/+$q_iGC_{pi}$ | w/-$q_hGC_{pi}$ | Dist.* | Cp | $q_hGC_p$ | w/ +$q_iGC_{pi}$ | w/ -$q_hGC_{pi}$ |
| Windward Wall (WW) | 0.80 | 14.4 | see table below | | | 0.80 | 14.4 | see table below | |
| Leeward Wall (LW) | -0.42 | -7.5 | -11.3 | -3.7 | | -0.50 | -9.0 | -12.8 | -5.2 |
| Side Wall (SW) | -0.70 | -12.6 | -16.4 | -8.8 | | -0.70 | -12.6 | -16.4 | -8.8 |
| Leeward Roof (LR) | -0.46 | -8.3 | -12.1 | -4.5 | | Included in windward roof | | | |
| Neg Windward Roof pressure | -0.54 | -9.7 | -13.5 | -5.9 | 0 to h/2* | -0.90 | -16.2 | -20.1 | -12.4 |
| Pos/min Windward Roof press. | -0.03 | -0.6 | -4.4 | 3.2 | h/2 to h* | -0.90 | -16.2 | -20.1 | -12.4 |
| | | | | | h to 2h* | -0.50 | -9.0 | -12.8 | -5.2 |
| | | | | | > 2h* | -0.30 | -5.4 | -9.2 | -1.6 |
| | | | | | Min press. | -0.18 | -3.2 | -7.1 | 0.6 |

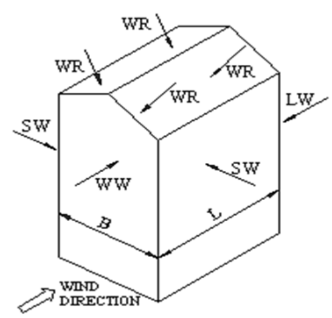
*Horizontal distance from windward edge

**Windward Wall Pressures at "z" (psf)**

| | | | | Windward Wall | | | Combined WW + LW | |
|---|---|---|---|---|---|---|---|---|
| | z | Kz | Kzt | $q_zGC_p$ | w/+$q_iGC_{pi}$ | w/-$q_hGC_{pi}$ | Normal to Ridge | Parallel to Ridge |
| | 0 to 15' | 0.85 | 1.00 | 13.0 | 9.2 | 16.9 | 20.6 | 22.1 |
| h= | 24.3 ft | 0.94 | 1.00 | 14.4 | 10.6 | 18.3 | 21.9 | 23.5 |
| h= | 24.3 ft | 0.94 | 1.00 | 14.4 | 10.6 | 18.3 | 21.9 | 23.5 |
| ridge = | 32.8 ft | 1.00 | 1.00 | 15.4 | 11.6 | 19.2 | 22.9 | 24.4 |

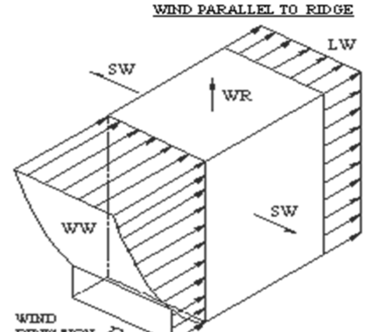



WIND NORMAL TO RIDGE




WIND PARALLEL TO RIDGE

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




TYPICAL WIND LOADING

**Parapet**

| z | Kz | Kzt | qp (psf) |
|---|---|---|---|
| 29.0 ft | 0.98 | 1.00 | 22.0 |

Windward parapet: 33.1 psf (GCpn = +1.5)
Leeward parapet: -22.0 psf (GCpn = -1.0)

Windward roof overhangs ( add to windward roof pressure ) : 14.4 psf (upward)

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Ultimate Wind Pressures

## Wind Loads - Components & Cladding : h ≤ 60'

| | | | |
|---|---|---|---|
| Kh (case 1) = | 0.94 | h = | 24.3 ft |
| Base pressure (qh) = | **21.2 psf** | a = | 9.6 ft |
| Minimum parapet ht = | 4.8 ft | GCpi = | +/-0.18 |
| Roof Angle (θ) = | 14.0 deg | qi = qh = | 21.2 psf |
| Type of roof = | Gable | | |

### Roof

| | Surface Pressure (psf) | | | | | | | | User input | |
|---|---|---|---|---|---|---|---|---|---|---|
| Area | 2 sf | 10 sf | 20 sf | 50 sf | 75 sf | 100 sf | 200 sf | 250 sf | 50 sf | 20 sf |
| Negative Zone 1 & 2e | -46.3 | -46.3 | -46.3 | -28.1 | -20.1 | -16.0 | -16.0 | -16.0 | -28.1 | -46.3 |
| Negative Zone 2n, 2r &3e | -67.5 | -67.5 | -58.3 | -46.3 | -40.9 | -37.1 | -28.0 | -25.0 | -46.3 | -58.3 |
| Negative Zone 3r | -80.2 | -80.2 | -68.7 | -53.5 | -46.8 | -42.0 | -42.0 | -42.0 | -53.5 | -68.7 |
| Positive All Zones | 16 | 16 | 16 | 16 | 16.0 | 16.0 | 16.0 | 16.0 | 16.0 | 16.0 |
| Overhang Zone 1 & 2e | -49.2 | -49.2 | -49.2 | -37.2 | -31.8 | -28.0 | -28.0 | -28.0 | -37.2 | -49.2 |
| Overhang Zone 2n & 2r | -74.3 | -70.5 | -63.6 | -54.5 | -50.5 | -47.7 | -40.8 | -38.6 | -54.5 | -63.6 |
| Overhang Zone 3e | -87 | -87 | -75.1 | -59.4 | -52.5 | -47.5 | -35.7 | -31.8 | -59.4 | -75.1 |
| Overhang Zone 3r | -99.7 | -99.7 | -84.4 | -64.1 | -55.2 | -48.8 | -48.8 | -48.8 | -64.1 | -84.4 |

Overhang pressures in the table above assume an internal pressure coefficient (Gcpi) of 0.0
Overhang soffit pressure equals adj wall pressure (which includes internal pressure of 3.8 psf)

### Parapet

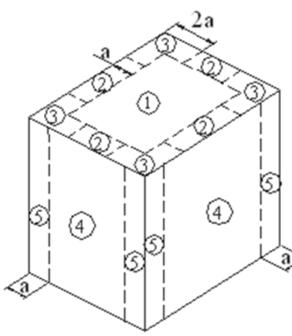
qp = 22.0 psf

| | Surface Pressure (psf) | | | | | | User input |
|---|---|---|---|---|---|---|---|
| Solid Parapet Pressure | 10 sf | 20 sf | 50 sf | 100 sf | 250 sf | 500 sf | 20 sf |
| CASE A: Zone 2e : | 66.1 | 64.9 | 44.6 | 29.2 | 27.6 | 26.4 | 64.9 |
| Zone 2n, 2r & 3e : | 88.1 | 77.5 | 63.4 | 52.7 | 38.6 | 37.5 | 77.5 |
| Zone 3r : | 101.4 | 88.3 | 70.9 | 57.8 | 56.3 | 55.1 | 88.3 |
| CASE B : Interior zone : | -46.3 | -43.9 | -40.8 | -38.5 | -35.4 | -33.1 | -43.9 |
| Corner zone : | -52.9 | -49.4 | -44.7 | -41.2 | -36.6 | -33.1 | -49.4 |

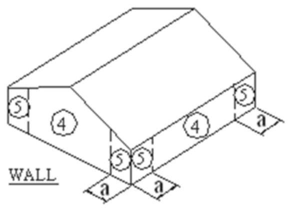
### Walls

| | GCp +/- GCpi | | | | Surface Pressure at h | | | | User input | |
|---|---|---|---|---|---|---|---|---|---|---|
| Area | 10 sf | 100 sf | 200 sf | 500 sf | 10 sf | 100 sf | 200 sf | 500 sf | 20 sf | 20 sf |
| Negative Zone 4 | -1.28 | -1.10 | -1.05 | -0.98 | -22.9 | -23.4 | -22.3 | -20.8 | -26.0 | -26.0 |
| Negative Zone 5 | -1.58 | -1.23 | -1.12 | -0.98 | -42.0 | -26.0 | -23.8 | -20.8 | -31.3 | -31.3 |
| Positive Zone 4 & 5 | 1.18 | 1.00 | 0.95 | 0.88 | 22.9 | 21.3 | 20.2 | 18.7 | 23.9 | 23.9 |

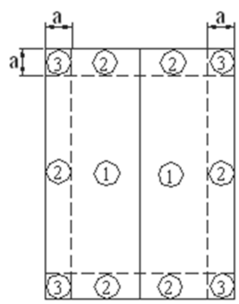
## Location of C&C Wind Pressure Zones - ASCE 7-10 & earlier



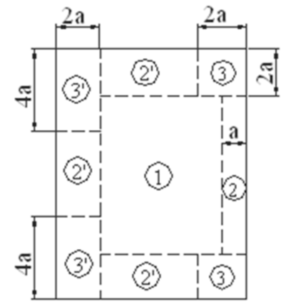
Roofs w/ θ ≤ 10°
and all walls
**h > 60'**



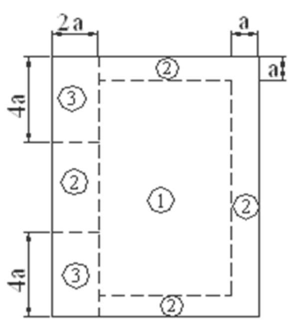
**Walls h ≤ 60'**
**& alt design h<90'**



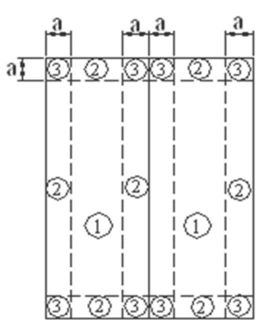
Gable, Sawtooth and
Multispan Gable θ ≤ 7 degrees &
Monoslope ≤ 3 degrees
**h ≤ 60' & alt design h<90'**



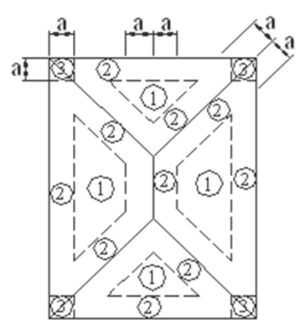
Monoslope roofs
3° < θ ≤ 10°
**h ≤ 60' & alt design h<90'**



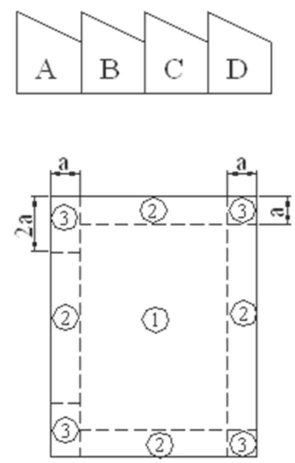
Monoslope roofs
10° < θ ≤ 30°
**h ≤ 60' & alt design h<90'**



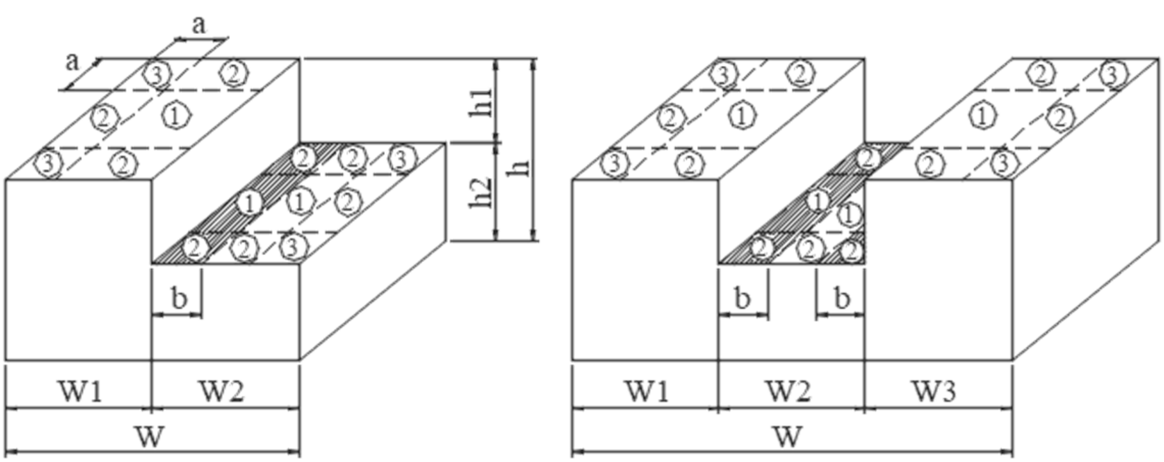
Multispan Gable &
Gable 7° < θ ≤ 45°



Hip 7° < θ ≤ 27°



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



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

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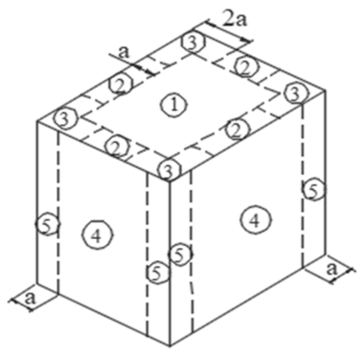
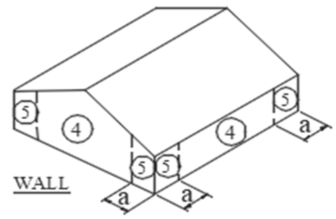
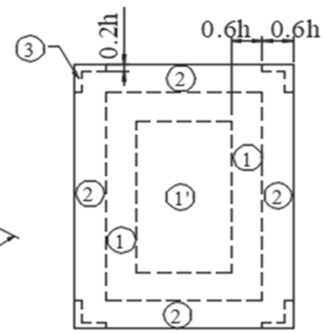
## Location of C&C Wind Pressure Zones - ASCE 7-16



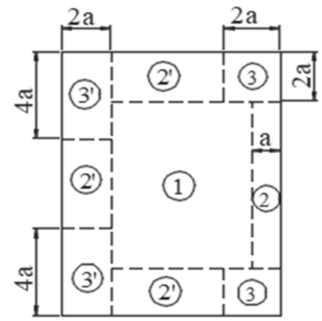
Roofs w/ θ ≤ 10°
and all walls
**h > 60'**



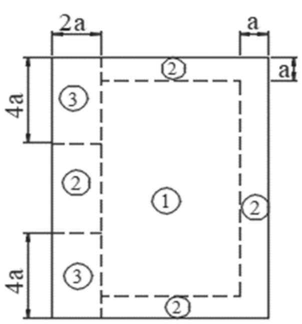

**Walls h ≤ 60'**
**& alt design h<90'**



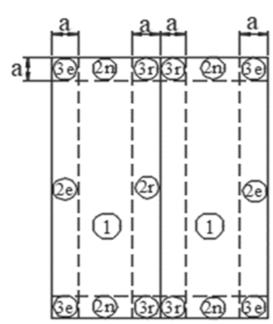
Gable, Sawtooth and
Multispan Gable θ ≤ 7 degrees &
Monoslope ≤ 3 degrees
**h ≤ 60' & alt design h<90'**



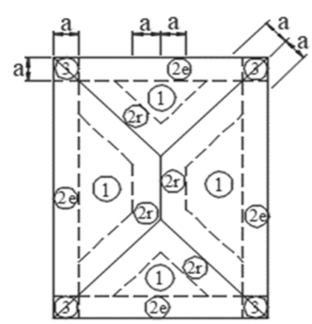
Monoslope roofs
3° < θ ≤ 10°
**h ≤ 60' & alt design h<90'**



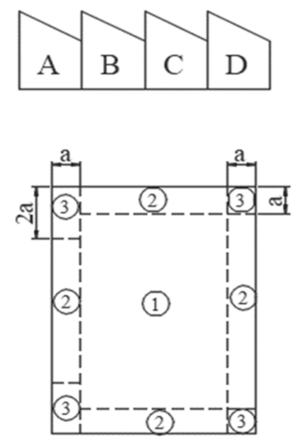
Monoslope roofs
10° < θ ≤ 30°
**h ≤ 60' & alt design h<90'**



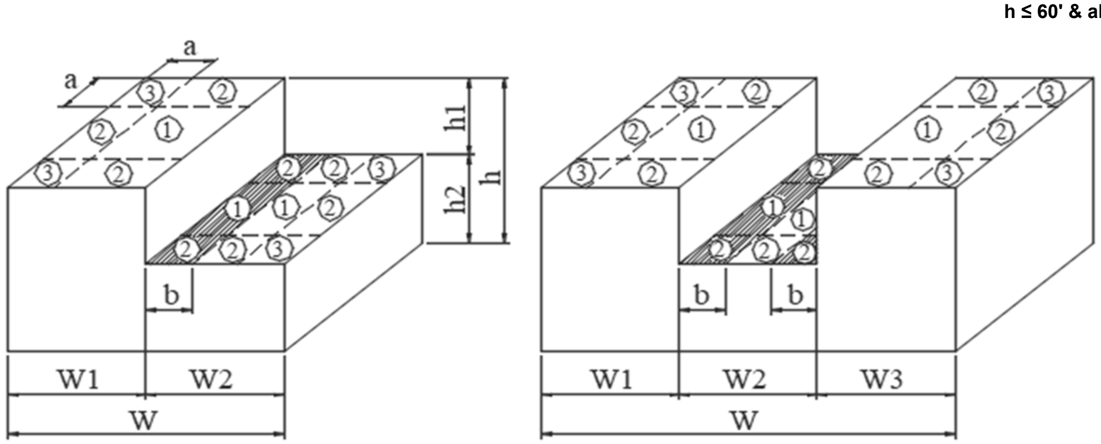
Multispan Gable &
Gable 7° < θ ≤ 45°



Hip 7° < θ ≤ 27°




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




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

## 104. SEISMIC LOADS

# Hazards by Location

## Search Information


**Address:** 1901 Curve Plaza, Steamboat Springs, CO 80487, USA

**Coordinates:** 40.5005622, -106.8562806

**Elevation:** 6673 ft

**Timestamp:** 2021-06-18T17:49:19.019Z

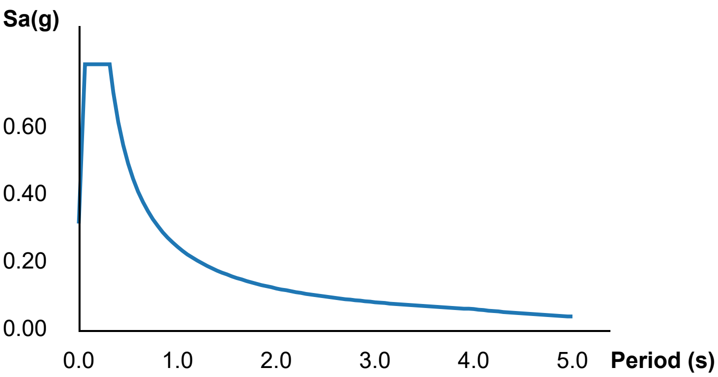
**Hazard Type:** Seismic

**Reference Document:** ASCE7-16

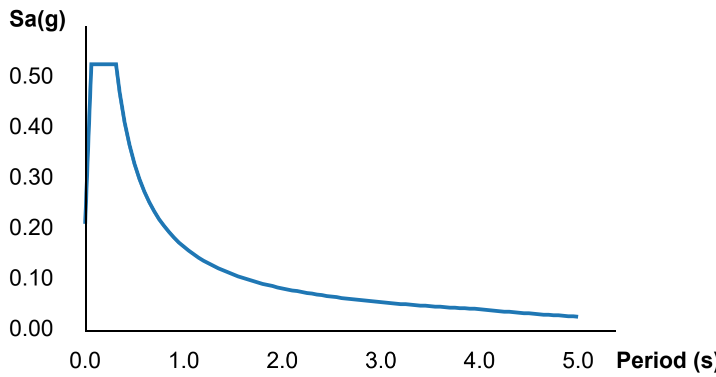
**Risk Category:** II

**Site Class:** D-default

## MCER Horizontal Response Spectrum




## Design Horizontal Response Spectrum




## Basic Parameters

| Name | Value | Description |
|---|---|---|
| $S_S$ | 0.596 | $MCE_R$ ground motion (period=0.2s) |
| $S_1$ | 0.103 | $MCE_R$ ground motion (period=1.0s) |
| $S_{MS}$ | 0.789 | Site-modified spectral acceleration value |
| $S_{M1}$ | 0.247 | Site-modified spectral acceleration value |
| $S_{DS}$ | 0.526 | Numeric seismic design value at 0.2s SA |
| $S_{D1}$ | 0.165 | Numeric seismic design value at 1.0s SA |

## Additional Information

| Name | Value | Description |
|---|---|---|
| SDC | D | Seismic design category |
| $F_a$ | 1.323 | Site amplification factor at 0.2s |
| $F_v$ | 2.394 | Site amplification factor at 1.0s |

| | | |
|---|---|---|
| $CR_S$ | 0.906 | Coefficient of risk (0.2s) |
| $CR_1$ | 0.946 | Coefficient of risk (1.0s) |
| PGA | 0.418 | $MCE_G$ peak ground acceleration |
| $F_{PGA}$ | 1.2 | Site amplification factor at PGA |
| $PGA_M$ | 0.502 | Site modified peak ground acceleration |
| $T_L$ | 4 | Long-period transition period (s) |
| SsRT | 0.596 | Probabilistic risk-targeted ground motion (0.2s) |
| SsUH | 0.658 | Factored uniform-hazard spectral acceleration (2% probability of exceedance in 50 years) |
| SsD | 1.5 | Factored deterministic acceleration value (0.2s) |
| S1RT | 0.103 | Probabilistic risk-targeted ground motion (1.0s) |
| S1UH | 0.109 | Factored uniform-hazard spectral acceleration (2% probability of exceedance in 50 years) |
| S1D | 0.6 | Factored deterministic acceleration value (1.0s) |
| PGAd | 0.5 | Factored deterministic acceleration value (PGA) |

*The results indicated here DO NOT reflect any state or local amendments to the values or any delineation lines made during the building code adoption process. Users should confirm any output obtained from this tool with the local Authority Having Jurisdiction before proceeding with design.*

## Disclaimer

Hazard loads are provided by the U.S. Geological Survey Seismic Design Web Services.

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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

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Golden, Colorado 80401
303-384-9910



## Seismic Loads:   IBC 2018   Strength Level Forces

Risk Category : II
Importance Factor (I) : 1.00
Site Class :) - code defaul

Ss (0.2 sec) = 32.50 %g
S1 (1.0 sec) = 8.30 %g

| | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| Fa = | 1.540 | Sms = | 0.501 | $S_{DS}$ = | 0.334 | Design Category = | C |
| Fv = | 2.400 | Sm1 = | 0.199 | $S_{D1}$ = | 0.133 | Design Category = | B |

Seismic Design Category = **C**

Redundancy Coefficient ρ = 1.00
Number of Stories: 1
Structure Type: Light Frame
Horizontal Struct Irregularities:No plan Irregularity
Vertical Structural Irregularities:No vertical Irregularity
Flexible Diaphragms: Yes
Building System: **Bearing Wall Systems**
Seismic resisting system: **Light frame (wood) walls with structural wood shear panels**
System Structural Height Limit: **Height not limited**
Actual Structural Height (hn) =32.8 ft

## DESIGN COEFFICIENTS AND FACTORS

Response Modification Coefficient (R) = 6.5
Over-Strength Factor (Ωo) = 2.5
Deflection Amplification Factor (Cd) = 4
$S_{DS}$ = 0.334
$S_{D1}$ = 0.133

Seismic Load Effect (E) = Eh +/-Ev = ρ $Q_E$ +/- 0.2$S_{DS}$ D = Qe +/- 0.067D   $Q_E$ = horizontal seismic force
Special Seismic Load Effect (Em) = Emh +/- Ev = Ωo $Q_E$ +/- 0.2$S_{DS}$ D = 2.5Qe +/- 0.067D   D = dead load

## PERMITTED ANALYTICAL PROCEDURES

**Simplified Analysis** - Use Equivalent Lateral Force Analysis

**Equivalent Lateral-Force Analysis** - Permitted
Building period coef. ($C_T$) = 0.020   Cu = 1.63
Approx fundamental period (Ta) = $C_T h_n$^ = 0.274 sec   x= 0.75   Tmax = CuTa = 0.448
User calculated fundamental period (T) = sec   Use T = 0.274
Long Period Transition Period (TL) = ASCE7 map = 4
Seismic response coef. (Cs) = $S_{DS}$I/R = 0.051
need not exceed Cs = Sd1 I /RT = 0.075
but not less than Cs = 0.044SdsI = 0.015
USE Cs = 0.051
Design Base Shear V = 0.051W

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

## ALLOWABLE STORY DRIFT

Structure Type: All other structures
Allowable story drift Δa = 0.020hsx   where hsx is the story height below level x

# 200. INFILL ADDITION

**Calculation Package**

# 200 Infill Addition Calculation Index:

# 200. INFILL ADDITION

## Wood Infill Existing Structure Loading Calculations

Prepared by: Sina Erturk
Date: 10/07/2021

| | DL | LL | SL | ASD_Live | ASD_Snow | ASD_Live + Snow |
|---|---|---|---|---|---|---|
| Roof | 65 | 0 | 75 | 65 | 140 | 121.25 |
| Mazzanine | 41 | 100 | 0 | 141 | 41 | 116 |
| (E) Roof 1 | 25 | 0 | 75 | 25 | 100 | 81.25 |
| (E) Roof 2 | 25 | 0 | 225 | 25 | 250 | 193.75 |
| (N) Drift 1 | 0 | 0 | 78 | 0 | 78 | 58.5 |
| (N) Drift 3 | 0 | 0 | 37 | 0 | 37 | 27.75 |

Story Width 11.3 ft

| | | Beam 1 | | |
|---|---|---|---|---|
| | | Length | 14.2 | ft |
| | | Trib Width | 5.63 | ft |
| Reaction | Roof | ASD_Live | 2.59 | k |
| | | ASD_Snow | 5.58 | k |
| | | ASD_L+S | 4.831055 | k |
| | Mazzanine | ASD_Live | 5.62 | k |
| | | ASD_Snow | 1.63 | k |
| | | ASD_L+S | 4.621875 | |
| | Total | ASD_Live | 8.21 | k |
| | | ASD_Snow | 7.21 | k |
| | | ASD_L+S | 9.45293 | k |

| | | Beam 2 | | |
|---|---|---|---|---|
| | | Length | 25 | ft |
| | | Trib Width | 5.63 | ft |
| Reaction | Roof | ASD_Live | 4.57 | k |
| | | ASD_Snow | 9.84 | k |
| | | ASD_L+S | 8.525391 | k |
| | Mazzanine | ASD_Live | 9.91 | k |
| | | ASD_Snow | 2.88 | k |
| | | ASD_L+S | 8.15625 | k |
| | Total | ASD_Live | 14.48 | k |
| | | ASD_Snow | 12.73 | k |
| | | ASD_L+S | 16.68164 | k |

| | | Beam 3 | | |
|---|---|---|---|---|
| | | Length | 14.2 | ft |
| | | Trib Width | 5.63 | ft |
| Reaction | Roof | ASD_Live | 2.59 | k |
| | | ASD_Snow | 5.58 | k |
| | | ASD_L+S | 4.831055 | k |
| | Mazzanine | ASD_Live | 5.62 | k |
| | | ASD_Snow | 1.63 | k |
| | | ASD_L+S | 4.621875 | k |
| | Total | ASD_Live | 8.21 | k |
| | | ASD_Snow | 7.21 | k |
| | | ASD_L+S | 9.45 | k |

| Column 1 | HSS6x6x1/4, PL3/4x12x1'-0" , C | | | | |
|---|---|---|---|---|---|
| Drift Area (sqf) | 40 | | | | |
| (E) Area (sqf) | 275 | Addition | Drift | Existing | Total |
| | ASD_Live (k) | 8.21 | 0 | 6.88 | 15.08 |
| Total | ASD_Snow (k) | 7.21 | 3.12 | 27.50 | 37.83 |
| | ASD_L+S (k) | 9.45 | 2.34 | 22.34 | 34.14 |
| | ASD (k) | 9.45 | 3.12 | 27.50 | 37.83 |

| Column 2 | HSS7x7x1/4, PL7/8x13x1'-1", D | | | | |
|---|---|---|---|---|---|
| Drift Area (sqf) | 40 | | | | |
| (E) Area (sqf) | 79 | Addition | Drift | Existing | Total |
| | ASD_Live (k) | 22.69 | 0 | 1.98 | 24.67 |
| Total | ASD_Snow (k) | 19.94 | 1.48 | 7.90 | 29.32 |
| | ASD_L+S (k) | 26.13 | 1.11 | 6.42 | 33.66 |
| | ASD (k) | 26.13 | 1.48 | 7.90 | 33.66 |

| Column 3 | HSS7x7x1/4 PL7/8x13x1'-1", D | | | | |
|---|---|---|---|---|---|
| Drift Area (sqf) | 40 | | | | |
| (E) Area (sqf) | 79 | Addition | Drift | Existing | Total |
| | ASD_Live (k) | 22.69 | 0 | 1.98 | 24.67 |
| Total | ASD_Snow (k) | 19.94 | 1.48 | 7.90 | 29.32 |
| | ASD_L+S (k) | 26.13 | 1.11 | 6.42 | 33.66 |
| | ASD (k) | 26.13 | 1.48 | 7.90 | 33.66 |

| Column 4 | HSS7x7x1/4 PL7/8x13x1'-1", C | | | | |
|---|---|---|---|---|---|
| Drift Area (sqf) | 40 | | | | |
| (E) Area (sqf) | 352 | Addition | Drift | Existing | Total |
| | ASD_Live (k) | 8.21 | 0 | 8.80 | 17.01 |
| Total | ASD_Snow (k) | 7.21 | 3.12 | 88.00 | 98.33 |
| | ASD_L+S (k) | 9.45 | 2.34 | 68.20 | 79.99 |
| | ASD (k) | 9.45 | 3.12 | 88.00 | 98.33 |

## Design Calculations

HSS6x6x1/4

| | | | |
|---|---|---|---|
| Pasd | 38.0 | k | |
| Height | 19 | ft | |
| P/Ω | 76.2 | k | from AISC Table 4-4 (p4-59) |
| Capacity | 50% | | |

HSS7x7x1/4

| | | | |
|---|---|---|---|
| Pasd | 99.0 | k | |
| Height | 19 | ft | |
| P/Ω | 107 | k | from AISC Table 4-4 (p4-58) |
| Capacity | 93% | | |

Footing C

| | | |
|---|---|---|
| Pasd | 38.0 | k |
| P/Ω | 85 | k |
| Capacity | 45% | |

Footing D

| | | |
|---|---|---|
| Pasd | 98.0 | k |
| P/Ω | 108 | k |
| Capacity | 91% | |

| | | | | |
|---|---|---|---|---|
| | | Project Name: | Basecamp | Engineer: SYE |
| | | Project Number: | 21304 | Date: 11/11/2021 |

## Existing Base Plates Check

## Column Base Plates for Axial Compression

per AISC Steel Manual, 13th Ed. Or 14th Ed, p 14-4, LRFD Analysis

### General Input

| | | | |
|---|---|---|---|
| | C1 | | Base Plate Design |
| Column Section | **HSS6X6X1/4** | | HSS Column |
| $B$ | **12** | in | Width Base Plate |
| $N$ | **12** | in | Length of Base Plate |
| $f'_c$ | **4000** | psi | 28-day Compressive Strength of Concrete |
| $P_u$ | **58.5** | kips | Required Compressive Strength |
| $F_y$ | **36** | ksi | Yield Strength of Base Plate |
| $B$ | 6.00 | in | Width of HSS |
| $H$ | 6.00 | in | Depth of HSS |

### Calculations

| | | | |
|---|---|---|---|
| $A_1$ | 144 | $in^2$ | Area of steel concentrically bearing on concrete support |
| $A_2$ | 0 | $in^2$ | area of the lower base of the largest frustum of a pyramid, contained wholly within the support and having for its upper base the loaded areas, and having side slopes of 1 vertical to 2 horizontal (Enter 0 to ignore bearing load multiplier) |
| $\mathrm{sqrt}(A_2/A_1) <= 2.0$ | 1.000 | | Bearing load multiplier |
| $\phi P_p$ | 318 | kips | Design bearing load on concrete ($\phi$=0.65 per ACI 318-02) |
| $X$ | 0.18 | | |
| $\lambda$ | 0.45 | | |
| $\lambda n'$ | 0.68 | in | Cantilever dimension for base plate |
| $n$ | 3.60 | in | Cantilever dimension for base plate |
| $m$ | 3.15 | in | Cantilever dimension for base plate |
| $l$ | 3.60 | in | Critical cantilever dimension for base plate |
| $t_{min}$ | 0.57 | in | Minimum base plate thickness |
| | 5/8 | in | Practical base plate thickness |

| | | |
|---|---|---|
| base plate dimensions | **12"x12"x0.625"** | |
| weight of baseplate | 25.56 | lb |
| base plate mark | **0.570087713** | |

Comments: (E) Column 200-6

| | | | |
|---|---|---|---|
| Project Name: | Basecamp | Engineer: | SYE |
| Project Number: | 21304 | Date: | 11/11/2021 |

## Column Base Plates for Axial Compression (HSS Columns)

per AISC Steel Manual, 13th Ed. Or 14th Ed, p 14-4, LRFD Analysis

### General Input

| | | | |
|---|---|---|---|
| | **C2** | | Base Plate Design |
| Column Section | **HSS7X7X1/4** | | HSS Column |
| $B$ | **13** | in | Width Base Plate |
| $N$ | **13** | in | Length of Base Plate |
| $f'_c$ | **4000** | psi | 28-day Compressive Strength of Concrete |
| $P_u$ | **147** | kips | Required Compressive Strength |
| $F_y$ | **36** | ksi | Yield Strength of Base Plate |
| $B$ | 7.00 | in | Width of HSS |
| $H$ | 7.00 | in | Depth of HSS |

### Calculations

| | | | |
|---|---|---|---|
| $A_1$ | 169 | $in^2$ | Area of steel concentrically bearing on concrete support |
| $A_2$ | | $in^2$ | area of the lower base of the largest frustum of a pyramid, contained wholly within the support and having for its upper base the loaded areas, and having side slopes of 1 vertical to 2 horizontal (Enter 0 to ignore bearing load multiplier) |
| $\mathrm{sqrt}(A_2/A_1) <= 2.0$ | 1.000 | | Bearing load multiplier |
| $\phi P_p$ | 373 | kips | Design bearing load on concrete ($\phi$=0.65 per ACI 318-14) |
| | detmine minimum baseplate bearing area | | |
| A reqd | 67 | $in^2$ | minimum area for bearing conconrete |
| $X$ | 0.39 | | |
| $\lambda$ | 0.71 | | |
| $\lambda n'$ | 1.23 | in | Cantilever dimension for base plate |
| $n$ | 3.70 | in | Cantilever dimension for base plate |
| $m$ | 3.18 | in | Cantilever dimension for base plate |
| $l$ | 3.70 | in | Critical cantilever dimension for base plate |
| $t_{min}$ | 0.86 | in | Minimum base plate thickness |
| | 7/8 | in | Practical base plate thickness |
| base plate dimensions | **13"x13"x0.875"** | | |
| weight of baseplate | 42.00 | lb | |
| base plate mark | **B1A** | | |

## Column Base Plates for Axial Compression (HSS Columns)

per AISC Steel Manual, 13th Ed. Or 14th Ed, p 14-4, LRFD Analysis

| Design | Mark | Column Section | B (in) | N (in) | $f'_c$ (psi) | $P_u$ (kips) | $F_y$ (ksi) | $A_2$ ($in^2$) | $t_{min}$ | Recommended BP | Comments |
|---|---|---|---|---|---|---|---|---|---|---|---|
| C1 | 0.570088 | HSS6X6X1/4 | 12 | 12 | 4000 | 58.5 | 36 | | 0.57 in | 12"x12"x0.625" | (E) Column |
| C2 | 0.857353 | HSS7X7X1/4 | 13 | 13 | 4000 | 147 | 36 | | 0.86 in | 13"x13"x0.875" | (E) Column |

Project Name: Basecamp
Project Numbe 21304

Engineer: SYE
Date: 11/11/2021

KL&A Engineers & Builders

| | | | | | |
|---|---|---|---|---|---|
| Title | Basecamp | Date | 10/19/21 | Job No. | 21304 |
| Subject | Wood Infill Existing Footings | By | SYE | Sheet | |

**Project Settings**

| | | | | | |
|---|---|---|---|---|---|
| Concrete Strength, $f'_c$ | 4 | ksi | Alowable Soil Bearing, q | 3 | ksf |
| Reinf Cover | 3.00 | in | Soil Density, $\gamma$ | 100 | pcf |
| Reinf Yeild Strength, $f_y$ | 60 | ksi | | | |

**FOOTING DESIGN SUMMARY** V2-2

*Limit States*

| | Footing Mark | Minimum Col Size | | Design Capacity | | | Footing Size | | | Bot Reinf | | Top Reinf | | Soil Height | | Soil Brg | Punching | BM Shear | Flexure | Uplift | Neg Flx |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | B (in) | N (in) | $\phi P_n$ (LRFD) (kip) | $P/\Omega$ (ASD) (kip) | $P_{uplift}$ (ASD) (kip) | b (in) | L (in) | h (in) | Qty | Bar | Qty | Bar | min (ft) | max (ft) | D/C | D/C | D/C | D/C | D/C | D/C |
| PASS | F1 | 12.0 | 12.0 | 128 | 85 | 0 | 64 | 64 | 12 | (6) | #5 | () | #5 | 1.0 | 4.0 | 1.00 | 0.84 | 0.66 | 0.80 | 0.00 | 0.00 |
| PASS | F2 | 12.0 | 12.0 | 158 | 105 | 0 | 84 | 84 | 14 | (7) | #5 | () | #5 | 1.0 | 4.0 | 0.71 | 0.79 | 0.56 | 0.99 | 0.00 | 0.00 |
| | F3 | | | | | | | | | | | | | | | | | | | | |
| | F4 | | | | | | | | | | | | | | | | | | | | |
| | F5 | | | | | | | | | | | | | | | | | | | | |
| | F6 | | | | | | | | | | | | | | | | | | | | |
| | F7 | | | | | | | | | | | | | | | | | | | | |
| | F8 | | | | | | | | | | | | | | | | | | | | |
| | F9 | | | | | | | | | | | | | | | | | | | | |
| | F10 | | | | | | | | | | | | | | | | | | | | |
| | F11 | | | | | | | | | | | | | | | | | | | | |
| | F12 | | | | | | | | | | | | | | | | | | | | |
| | F13 | | | | | | | | | | | | | | | | | | | | |
| | F14 | | | | | | | | | | | | | | | | | | | | |
| | F15 | | | | | | | | | | | | | | | | | | | | |
| | F16 | | | | | | | | | | | | | | | | | | | | |
| | F17 | | | | | | | | | | | | | | | | | | | | |
| | F18 | | | | | | | | | | | | | | | | | | | | |
| | F19 | | | | | | | | | | | | | | | | | | | | |
| | F20 | | | | | | | | | | | | | | | | | | | | |

| Title | Basecamp | Date | 6/25/14 | Job No. | 21304 |
|---|---|---|---|---|---|
| Subject | Wood Infill Existing Footings | By | SYE | Sheet | |

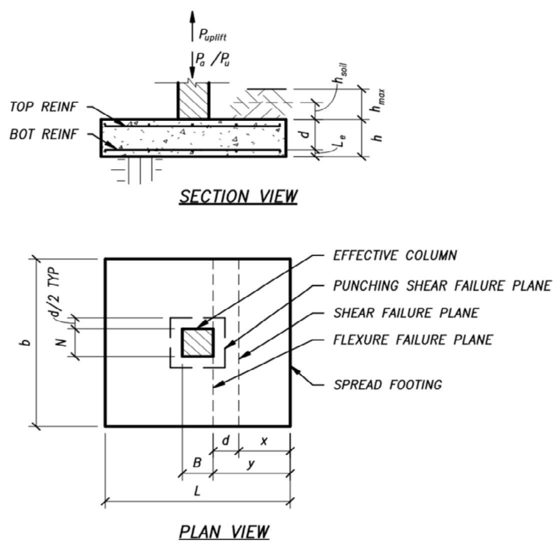
## CONCENTRIC SPREAD FOOTING DESIGN

Governing Code: ACI 318-14 V2-2

**FOOTING NUMBER** = F1

**DESIGN CAPACITY**

| | | | |
|---|---|---|---|
| Axial (ASD), $P/\Omega$ | = | 85 | kip |
| Axial (LRFD), $\phi P_n$ | = | 128 | kip |
| Net Uplift (ASD), $P_{uplift}$ | = | 0 | kip |




**FOOTING CRITERIA**

| | | | |
|---|---|---|---|
| Concrete Strength, $f'_c$ | = | 4 | ksi |
| Footing Width, b | = | 64 | in |
| Footing Length, L | = | 64 | in |
| Footing Thickness, h | = | 12 | in |

**BOTTOM REINFORCMENT**

| | | | |
|---|---|---|---|
| Quantity | = | (6) | |
| Reinforcing bar | = | #5 | |
| Reinforcing strength, $f_y$ | = | 60 | ksi |
| Reinforcing bar diam, $d_b$ | = | 0.63 | in |
| Reinforcing bar Area, $A_b$ | = | 0.31 | $in^2$ |
| Reinf Clear Cover, $L_e$ | = | 3 | in |

**TOP REINFORCMENT**

| | | | |
|---|---|---|---|
| Quantity | = | () | |
| Reinforcing bar | = | #5 | |
| Reinforcing bar diam | = | 0.63 | in |
| Reinforcing bar Area | = | 0.31 | $in^2$ |

**SOIL CRITERIA**

| | | | |
|---|---|---|---|
| Soil Bearing, q | = | 3 | ksf |
| Soil Density, $\gamma$ | = | 100 | pcf |
| Min Soil Height, $h_{soil}$ | = | 1.0 | ft |
| Max Soil Height, $h_{max}$ | = | 4.0 | ft |

**MINIMUM COLUMN SIZE**

| | | | |
|---|---|---|---|
| Depth, B | = | 12.0 | in |
| Width, N | = | 12.0 | in |

**DESIGN SUMMARY**

Footing 5' - 4" x 5' - 4" x 12 " w/ (6) #5B EW

**LIMIT STATE SUMMARY**

| | D/C | | Limit |
|---|---|---|---|
| Soil Bearing | 1.00 | ≤ | 1.0 |
| FTG Punching Shear | 0.84 | ≤ | 1.0 |
| FTG Beam Shear | 0.66 | ≤ | 1.0 |
| FTG Flexure | 0.80 | ≤ | 1.0 |
| Uplift | 0.00 | ≤ | 1.0 |
| FTG Negative Flexure | N/A | > | 1.0 |

## CONCENTRIC SPREAD FOOTING DESIGN

Governing Code: ACI 318-14 V2-2

**FOOTING NUMBER** = F1

### SOIL BEARING

| | | | | | |
|---|---|---|---|---|---|
| Service Load, $P_a$ | | | = | 85.3 | kip |
| Ultimate Load, $P_u$ | | | = | 128.0 | kip |
| Footing Width, b | | | = | 64 | in |
| Footing Length, L | | | = | 64 | in |
| Area of Footing, A | = | bL | = | 4096 | $in^2$ |
| Soil Allowable | | | = | 3 | ksf |
| Applied Soil Pressure, $q_a$ | = | $P_a/A$ | = | 3.0 | ksf |
| Ultimate Soil Rxn, $q_u$ | = | $P_u/A$ | = | 0.031 | ksi |

**$q_a/q$**

**1 ≤ 1.0 OK**

### FOOTING PUNCHING SHEAR

| | | | | | | |
|---|---|---|---|---|---|---|
| Resistance Factor, ϕ | | | = | 0.75 | | |
| Depth of Reinforcing, d | = | $h - L_e - d_b/2$ | = | 8.7 | in | |
| Effective Col Width, $B_e$ | = | B | = | 12.0 | in | |
| Effective Col Depth, $N_e$ | = | N | = | 12.0 | in | |
| Perimeter, $b_o$ | = | $2(N_e+d)+2(B_e+d)$ | = | 83 | in | |
| $\alpha_s$ | | | = | 40 | | |
| β | = | $N_e/B_e$ | = | 1.0 | | |
| Punching Force, $R_u$ | = | $P_u - q_u (N_e+d) (B_e+d)$ | = | 114.6 | kip | |
| *Punching Shear Strength, $V_c$* | | | | | | |
| Eqn (11-33) | = | $(2+4/\beta)(f'_c)$^0.5 $b_o$ d | = | 273 | kip | |
| Eqn (11-34) | = | $[(\alpha_s\, d)/b_o + 2)(f'_c)$^0.5 $b_o$ d | = | 282 | kip | |
| Eqn (11-35) | = | $4(f'_c)$^0.5 $b_o$ d | = | 182 | kip | *Controls* |
| Punching Shear Design Strength, $\phi R_n$ | | | = | 136 | kip | |

**$R_u / \phi R_n$**

**0.84 ≤ 1.0 OK**

## CONCENTRIC SPREAD FOOTING DESIGN

Governing Code: ACI 318-14 V2-2

**FOOTING NUMBER** = F1

### FOOTING BEAM SHEAR

| | | | | | |
|---|---|---|---|---|---|
| Resistance Factor, $\phi$ | | | = | 0.75 | |
| Distance to Shear Plane, x | = | $b/2 - [\min(B_e, N_e)/2 + d]$ | = | 17.3 | in |
| Shear Force, $R_u$ | = | $q_u(x)L$ | = | 34.6 | kip |
| Shear Strength, $V_c$ | = | $2\,(f'_c)$^0.5 b d | = | 70.3 | kip |
| Beam Shear Design Strength, $\phi R_n$ | | | = | 52.7 | kip |

**$R_u$ / $\phi R_n$**
**0.66 ≤ 1.0 OK**

### FOOTING FLEXURE

| | | | | | |
|---|---|---|---|---|---|
| Resistance Factor, $\phi$ | | | = | 0.9 | |
| Distance to Moment, y | = | $b/2 - \min(B_e, N_e)/2$ | = | 26.0 | in |
| Moment, $M_u$ | = | $q_u\, b\, y^2/2$ | = | 676.0 | k-in |
| Minimum Steel, $A_{s,min}$ | = | 0.0018*b*h | = | 1.38 | $in^2$ |
| Bars Used, $n_b$ | | | = | (6) | |
| Area of Steel, $A_s$ | = | $n_b{}^*A_b$ | = | 1.86 | $in^2$ |
| $A_{s,min}/A_s$ | | | = | 0.74 | |
| a | = | $A_s\, F_y/(0.85\, f'_c\, b)$ | = | 0.51 | in |
| Moment Capacity, $M_n$ | = | $A_s\, F_y\, (d - a/2)$ | = | 941 | k-in |
| Design Moment Capacity,$\phi M_n$ | | | = | 847 | k-in |

**$R_u$ / $\phi R_n$**
**0.80 ≤ 1.0 OK**

### UPLIFT

| | | | | | |
|---|---|---|---|---|---|
| Uplift Load, $P_{uplift}$ | | | = | 0 | kip |
| Volume of Concrete, $V_{ftg}$ | = | bLh | = | 28 | cuft |
| Volume of Soil, $V_{soil}$ | = | $bLh_{soil}$ | = | 28 | cuft |
| Dead Load, DL | = | $V_{ftg}{}^*\gamma_{conc}+V_{soil}{}^*\gamma$ | = | 7 | kip |
| Effective Dead Load, 0.6DL | | | = | 4 | kip |

**D/C**
**0.00 ≤ 1.0 OK**

### FOOTING NEGATIVE FLEXURE

| | | | | | |
|---|---|---|---|---|---|
| Soil weight, $q_{soil}$ | = | $h_{max}\gamma$ | = | 2.8E-03 | ksi |
| Footing Self Weight, $q_{ftg}$ | | | = | 1.0E-03 | ksi |
| Moment, $M_u$ | = | $q_{soil}\, b\, y^2/2$ | = | 83 | k-in |
| Bars Used, $n_b$ | | | = | 0 | |
| Area of Steel, $A_s$ | = | $n_b{}^*A_b$ | = | 0 | $in^2$ |
| a | = | $A_s\, F_y/(0.85\, f'_c\, b)$ | = | 0.00 | in |
| Moment Capacity, $M_n$ | = | $A_s\, F_y\, (d - a/2)$ | = | 0 | k-in |
| Design Moment Capacity,$\phi M_n$ | | | = | 0 | k-in |

**$R_u$ / $\phi R_n$**
**N/A > 1.0 NG**

| | | | | | |
|---|---|---|---|---|---|
| Title | Basecamp | Date | 6/25/14 | Job No. | 21304 |
| Subject | Wood Infill Existing Footings | By | SYE | Sheet | |

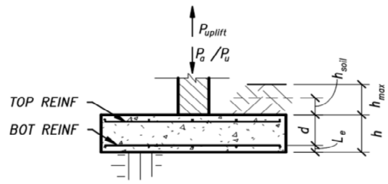
## CONCENTRIC SPREAD FOOTING DESIGN

Governing Code: ACI 318-14 V2-2

**FOOTING NUMBER** = F2

**DESIGN CAPACITY**

| | | | |
|---|---|---|---|
| Axial (ASD), P/Ω | = | 105 | kip |
| Axial (LRFD), $\phi P_n$ | = | 158 | kip |
| Net Uplift (ASD), $P_{uplift}$ | = | 0 | kip |



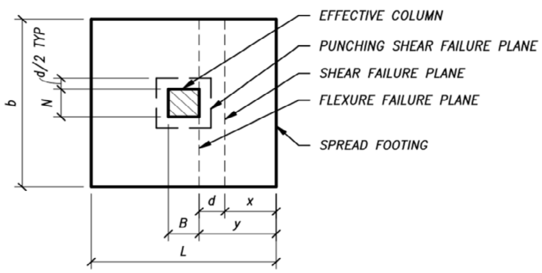

SECTION VIEW

**FOOTING CRITERIA**

| | | | |
|---|---|---|---|
| Concrete Strength, $f'_c$ | = | 4 | ksi |
| Footing Width, b | = | 84 | in |
| Footing Length, L | = | 84 | in |
| Footing Thickness, h | = | 14 | in |

**BOTTOM REINFORCMENT**

| | | | |
|---|---|---|---|
| Quantity | = | (7) | |
| Reinforcing bar | = | #5 | |
| Reinforcing strength, $f_y$ | = | 60 | ksi |
| Reinforcing bar diam, $d_b$ | = | 0.63 | in |
| Reinforcing bar Area, $A_b$ | = | 0.31 | $in^2$ |
| Reinf Clear Cover, $L_e$ | = | 3 | in |




PLAN VIEW

**TOP REINFORCMENT**

| | | | |
|---|---|---|---|
| Quantity | = | () | |
| Reinforcing bar | = | #5 | |
| Reinforcing bar diam | = | 0.63 | in |
| Reinforcing bar Area | = | 0.31 | $in^2$ |

**SOIL CRITERIA**

| | | | |
|---|---|---|---|
| Soil Bearing, q | = | 3 | ksf |
| Soil Density, γ | = | 100 | pcf |
| Min Soil Height, $h_{soil}$ | = | 1.0 | ft |
| Max Soil Height, $h_{max}$ | = | 4.0 | ft |

**MINIMUM COLUMN SIZE**

| | | | |
|---|---|---|---|
| Depth, B | = | 12.0 | in |
| Width, N | = | 12.0 | in |

**DESIGN SUMMARY**

Footing 7' - 0" x 7' - 0" x 14 " w/ (7) #5B EW

**LIMIT STATE SUMMARY**

| | D/C | | Limit |
|---|---|---|---|
| Soil Bearing | 0.71 | ≤ | 1.0 |
| FTG Punching Shear | 0.79 | ≤ | 1.0 |
| FTG Beam Shear | 0.56 | ≤ | 1.0 |
| FTG Flexure | 0.99 | ≤ | 1.0 |
| Uplift | 0.00 | ≤ | 1.0 |
| FTG Negative Flexure | N/A | > | 1.0 |

## CONCENTRIC SPREAD FOOTING DESIGN

Governing Code: ACI 318-14 V2-2

**FOOTING NUMBER** = F2

### SOIL BEARING

| | | | | | |
|---|---|---|---|---|---|
| Service Load, $P_a$ | | | = | 105.0 | kip |
| Ultimate Load, $P_u$ | | | = | 157.5 | kip |
| Footing Width, b | | | = | 84 | in |
| Footing Length, L | | | = | 84 | in |
| Area of Footing, A | = | bL | = | 7056 | $in^2$ |
| Soil Allowable | | | = | 3 | ksf |
| Applied Soil Pressure, $q_a$ | = | $P_a/A$ | = | 2.1 | ksf |
| Ultimate Soil Rxn, $q_u$ | = | $P_u/A$ | = | 0.022 | ksi |

**$q_a/q$**

**0.71 ≤ 1.0 OK**

### FOOTING PUNCHING SHEAR

| | | | | | | |
|---|---|---|---|---|---|---|
| Resistance Factor, $\phi$ | | | = | 0.75 | | |
| Depth of Reinforcing, d | = | $h - L_e - d_b/2$ | = | 10.7 | in | |
| Effective Col Width, $B_e$ | = | B | = | 12.0 | in | |
| Effective Col Depth, $N_e$ | = | N | = | 12.0 | in | |
| Perimeter, $b_o$ | = | $2(N_e+d)+2(B_e+d)$ | = | 91 | in | |
| $\alpha_s$ | | | = | 40 | | |
| $\beta$ | = | $N_e/B_e$ | = | 1.0 | | |
| Punching Force, $R_u$ | = | $P_u - q_u (N_e+d) (B_e+d)$ | = | 146.0 | kip | |
| *Punching Shear Strength, $V_c$* | | | | | | |
| Eqn (11-33) | = | $(2+4/\beta)(f'_c)$^0.5 $b_o$ d | = | 368 | kip | |
| Eqn (11-34) | = | $[(\alpha_s d)/b_o + 2)(f'_c)$^0.5 $b_o$ d | = | 412 | kip | |
| Eqn (11-35) | = | $4(f'_c)$^0.5 $b_o$ d | = | 245 | kip | *Controls* |
| Punching Shear Design Strength, $\phi R_n$ | | | = | 184 | kip | |

**$R_u$ / $\phi R_n$**

**0.79 ≤ 1.0 OK**

## CONCENTRIC SPREAD FOOTING DESIGN

Governing Code: ACI 318-14 V2-2

**FOOTING NUMBER** = F2

### FOOTING BEAM SHEAR

| | | | | | |
|---|---|---|---|---|---|
| Resistance Factor, $\phi$ | | | = | 0.75 | |
| Distance to Shear Plane, x | = | $b/2 - [\min(B_e, N_e)/2 + d]$ | = | 25.3 | in |
| Shear Force, $R_u$ | = | $q_u(x)L$ | = | 47.5 | kip |
| Shear Strength, $V_c$ | = | 2 ($f'_c$)^0.5 b d | = | 113.6 | kip |
| Beam Shear Design Strength, $\phi R_n$ | | | = | 85.2 | kip |

**$R_u$ / $\phi R_n$**
**0.56 ≤ 1.0 OK**

### FOOTING FLEXURE

| | | | | | |
|---|---|---|---|---|---|
| Resistance Factor, $\phi$ | | | = | 0.9 | |
| Distance to Moment, y | = | $b/2 - \min(B_e, N_e)/2$ | = | 36.0 | in |
| Moment, $M_u$ | = | $q_u\, b\, y^2/2$ | = | 1215.0 | k-in |
| Minimum Steel, $A_{s,min}$ | = | 0.0018*b*h | = | 2.12 | $in^2$ |
| Bars Used, $n_b$ | | | = | (7) | |
| Area of Steel, $A_s$ | = | $n_b$*$A_b$ | = | 2.17 | $in^2$ |
| $A_{s,min}/A_s$ | | | = | 0.98 | |
| a | = | $A_s F_y/(0.85 f'_c b)$ | = | 0.46 | in |
| Moment Capacity, $M_n$ | = | $A_s F_y (d - a/2)$ | = | 1362 | k-in |
| Design Moment Capacity,$\phi M_n$ | | | = | 1226 | k-in |

**$R_u$ / $\phi R_n$**
**0.99 ≤ 1.0 OK**

### UPLIFT

| | | | | | |
|---|---|---|---|---|---|
| Uplift Load, $P_{uplift}$ | | | = | 0 | kip |
| Volume of Concrete, $V_{ftg}$ | = | bLh | = | 57 | cuft |
| Volume of Soil, $V_{soil}$ | = | $bLh_{soil}$ | = | 49 | cuft |
| Dead Load, DL | = | $V_{ftg}$*$\gamma_{conc}$+$V_{soil}$*$\gamma$ | = | 13 | kip |
| Effective Dead Load, 0.6DL | | | = | 8 | kip |

**D/C**
**0.00 ≤ 1.0 OK**

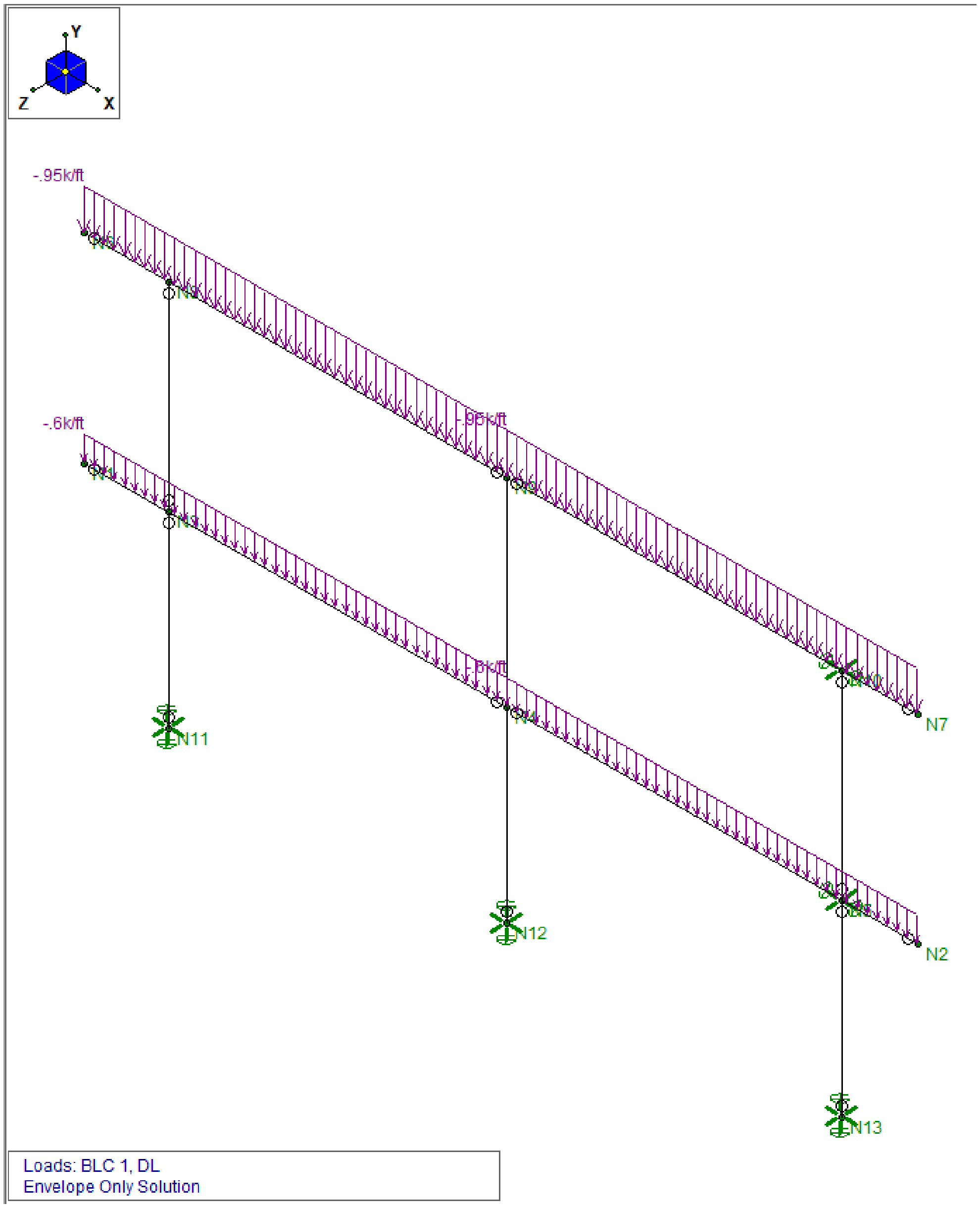
### FOOTING NEGATIVE FLEXURE

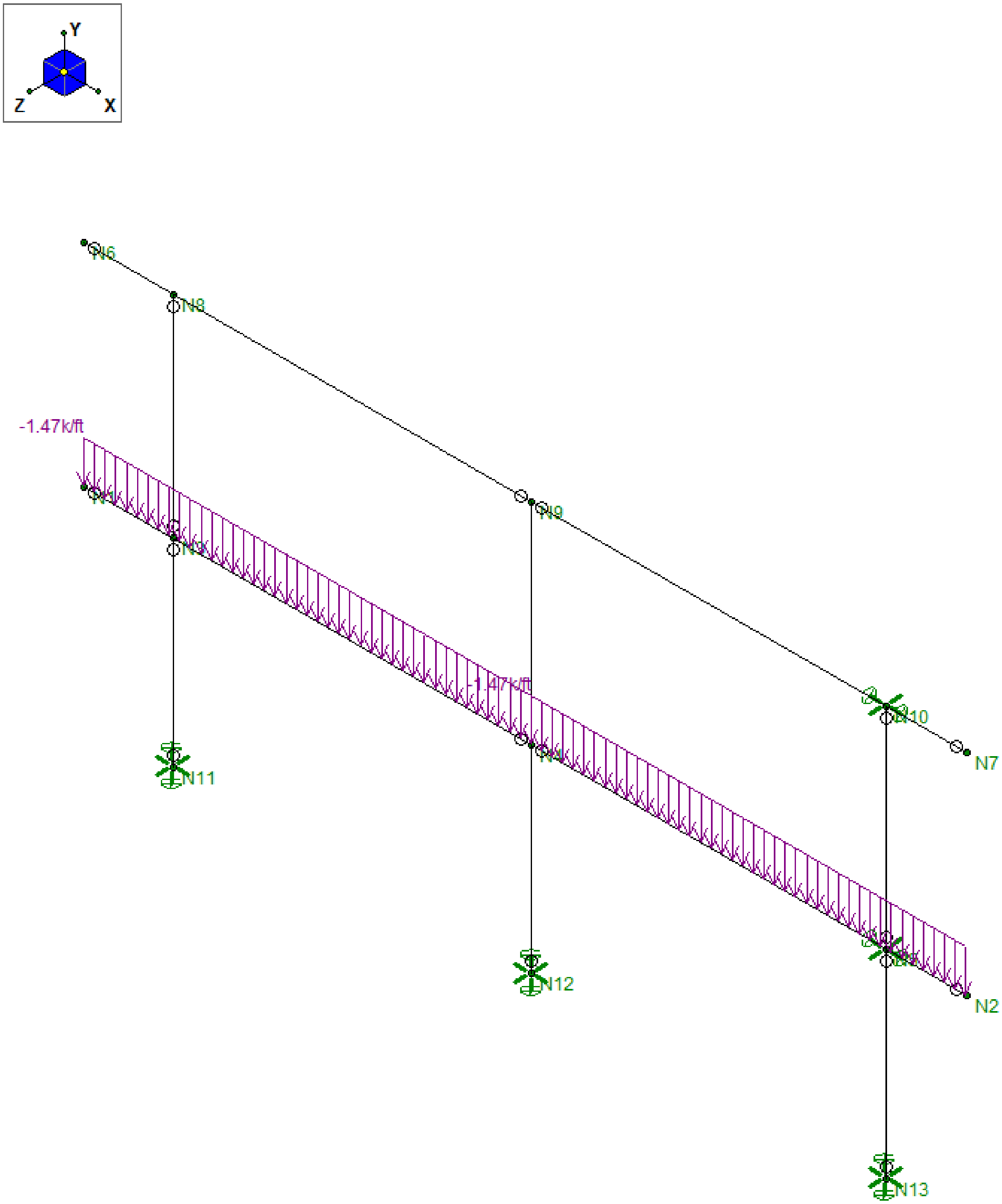
| | | | | | |
|---|---|---|---|---|---|
| Soil weight, $q_{soil}$ | = | $h_{max}\gamma$ | = | 2.8E-03 | ksi |
| Footing Self Weight, $q_{ftg}$ | | | = | 1.2E-03 | ksi |
| Moment, $M_u$ | = | $q_{soil}\, b\, y^2/2$ | = | 217 | k-in |
| Bars Used, $n_b$ | | | = | 0 | |
| Area of Steel, $A_s$ | = | $n_b$*$A_b$ | = | 0 | $in^2$ |
| a | = | $A_s F_y/(0.85 f'_c b)$ | = | 0.00 | in |
| Moment Capacity, $M_n$ | = | $A_s F_y (d - a/2)$ | = | 0 | k-in |
| Design Moment Capacity,$\phi M_n$ | | | = | 0 | k-in |

**$R_u$ / $\phi R_n$**
**N/A > 1.0 NG**

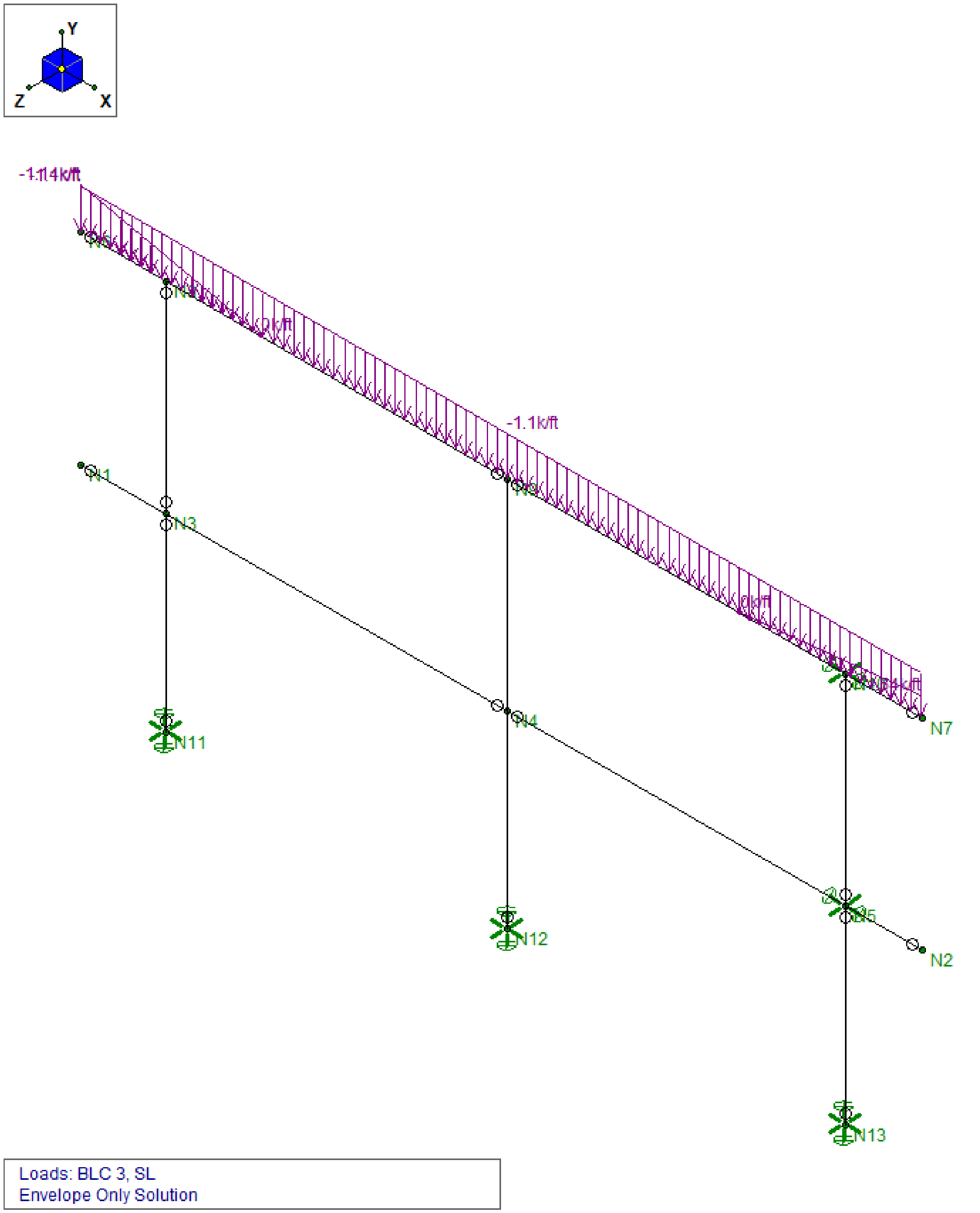
## 201. GRAVITY DESIGN

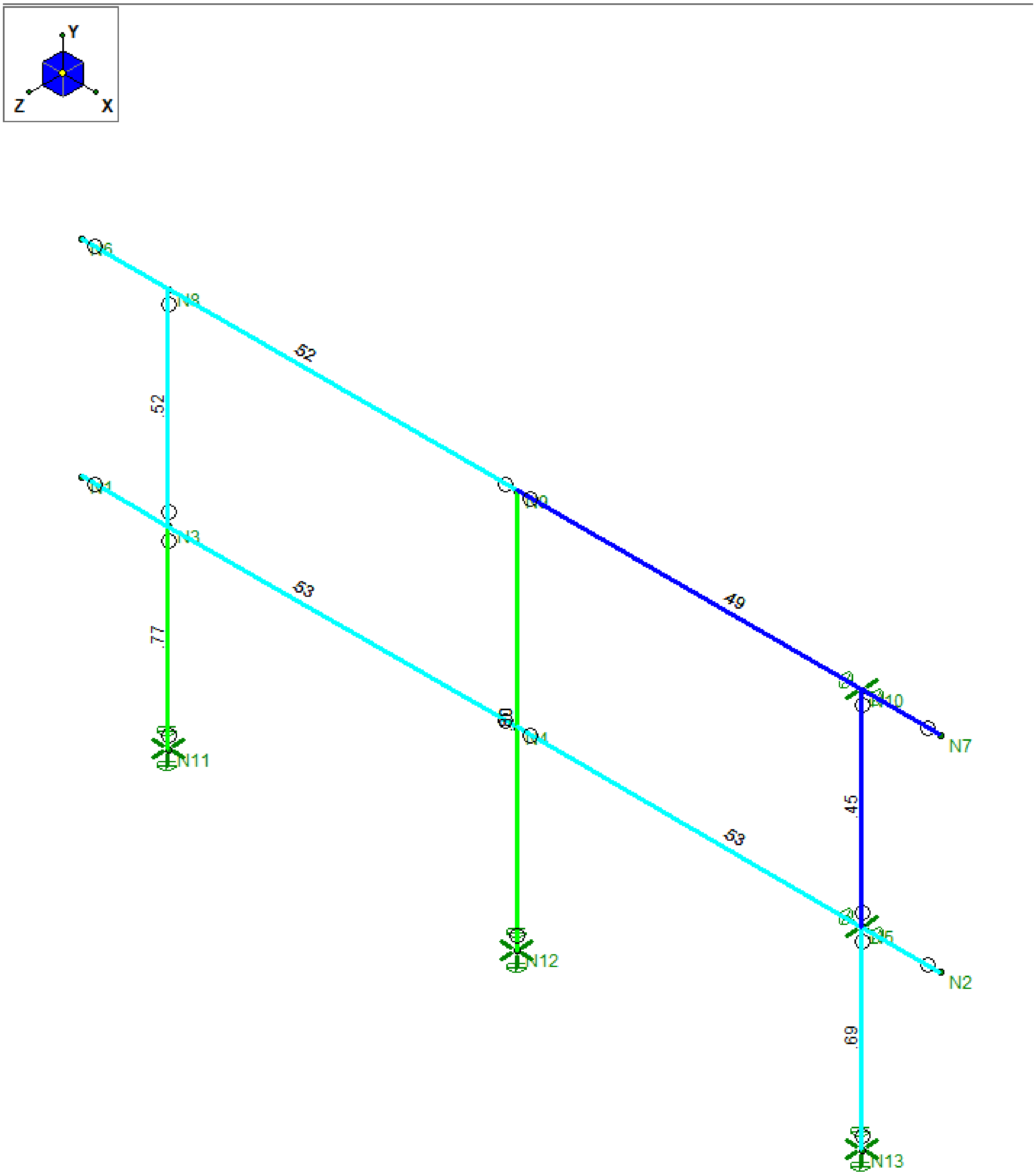
# Steel Framing Design Calculations






Loads: BLC 2, LL
Envelope Only Solution


Y
Z
X
-1.1k/ft
N1
N3
N4
N7
N11
N12
N5
N2
N13
Loads: BLC 3, SL
Envelope Only Solution




Member Code Checks Displayed (Enveloped)
Envelope Only Solution

| Wide Flange or Channel Shape | W14x22 | | |
|---|---|---|---|
| $F_y$ | 50 | ksi | Yield strength |
| $E$ | 29000 | ksi | Modulus of Elasticity |
| $G$ | 11200 | ksi | Shear Modulus of Elasticity |
| $A$ | 6.49 | $in^2$ | Cross Sectional Area of beam |
| $I_x$ | 199.00 | $in^4$ | Moment of Inertia about Major Axis |
| $S_x$ | 29.00 | $in^3$ | Section Modulus about Major Axis |
| $Z_x$ | 33.20 | $in^3$ | Plastic Modulus about Major Axis |
| $I_y$ | 7.00 | $in^4$ | Moment of Inertia about Minor Axis |
| $r_y$ | 1.04 | in | Radius of Gyration about Minor Axis |
| $J$ | 0.21 | $in^4$ | Polar Moment of Inertia |
| $C_w$ | 314 | $in^6$ | Warping Constant |
| $d$ | 13.70 | in | Depth |
| $t_w$ | 0.23 | in | Web Thickness |
| $t_f$ | 0.34 | in | Flange Thickness |
| $b_f$ | 5.00 | in | Flange Width |
| $k$ | 0.74 | in | Fillet Dimension |
| $h$ | 11.56 | in | Distance flanges less the fillet |
| $h_0$ | 13.37 | in | Distance between centroids of flanges |
| $A_w$ | 3.15 | $in^2$ | Area of web |
| | N | *Use LTB modification factor? (Y/N)* | |
| $C_b$ | 1.00 | | Modification Factor for Nonuniform Moment |
| $L_b$ | 2 | ft | Distance between lateral braces |

| Service Loads | | |
|---|---|---|
| Dead | 65 | psf |
| Live | 0 | psf |
| Snow or Live Roof | 112 | psf |
| Garage/Assembly? | N | (Y/N) |

| Service Load Combinations | | |
|---|---|---|
| Combo 1: D + L | 65 | psf |
| Combo 2: D + S | 177 | psf |
| Combo 3: D + 0.75L + 0.75S | 149 | psf |
| Governing Combination | **177** | psf |

| Geometry | | |
|---|---|---|
| Simple Span | 14.2 | ft |
| Tributary Width | 14.65 | ft |
| Beam Slope | 0 | /12 |
| | 0.00 | degrees |
| | 0.0000 | radians |
| Slope Factor | 1.000 | |
| Sloped Length | 14.20 | ft |

| Factored Load Combinations | | |
|---|---|---|
| Combo 1: 1.4D | 91 | psf |
| Combo 2: 1.2D + 1.6L + 0.5($L_r$ or S) | 134 | psf |
| Combo 3: 1.2D + 1.6(Lr or S) + 0.5L | 257 | psf |
| Governing Combination | **257** | psf |
| Avg. Load Factor for Governing Load Case | 1.453 | |

| Governing Limit State | |
|---|---|
| Flexure | 0.76 |

| Reactions | $R_1$ | $R_2$ |
|---|---|---|
| Total Service, kips | 18.41 | 18.41 |
| Total Factored, kips | 26.75 | 26.75 |

**13th Ed AISC Manual, LRFD Design**

| Beam Flexural Strength | | | | |
|---|---|---|---|---|
| **Check for Lateral Torsional Buckling (LTB)** | | | | |
| $L_b$ | 2 | ft | | Distance between lateral braces |
| $L_p$ | 3.67 | ft | (F2-5) | Limiting unbraced length for limit state of yielding |
| $L_r$ | 10.4 | ft | (F2-6) | Limiting unbraced length for limit state of inelastic LTB |
| $r_{ts}^2$ | 1.62 | in | (F2-7) | Square of effective radius of gyration |
| $c$ | 1.00 | | (F2-8a,b) | LTB constant |
| $F_{cr}$ | 809.29 | ksi | (F2-4) | Elastic LTB stress |
| $M_p = (M_n)_1$ | 138.3 | k*ft | (F2-1) | if $L_b <= L_p$ -- Plastic moment |
| $(M_n)_2$ | 138.3 | k*ft | (F2-2) | if $L_p < L_b <= L_r$--Inelastic buckling moment |
| $(M_n)_3$ | 138.3 | k*ft | (F2-3) | if $L_b > L_r$ -- Elastic buckling moment |
| **Check for Flange Local Buckling** | | | | |
| $\lambda$ | 7.46 | | | Width-thickness ratio for flange |
| $\lambda_p$ | 9.15 | | Table B4.1-1 | Limiting slenderness for compact flange |
| $\lambda_r$ | 24.08 | | | Limiting slenderness for non-compact flange |
| | **Compact** | | | Slenderness Check |
| $k_c$ | 0.56 | | F3.2 | Coefficient for slender, unstiffened elements |
| $M_n$ | 138.3 | k*ft | (F3-1) | Nominal flexural strength |
| **Flexural Strength Summary** | | | | |
| $\phi_b$ | 0.9 | | | Resistance factor for bending |
| $M_n$ | 138.3 | k*ft | | Nominal flexural strength |
| $\phi M_n$ | 124.5 | k*ft | | Design flexural strength |
| $M_u$ | 94.97 | k*ft | | Required flexural strength |
| D/C | 0.76 | | | Demand/Capacity ratio |

| Beam Shear Strength | | | | |
|---|---|---|---|---|
| $\phi_v$ | 1.0 | | | Resistance factor for shear |
| $a$ | 0 | in | | Transverse stiffener clear spacing (0 = no stiffeners) |
| $k_v$ | 5.000 | | G2.1(ii) | Web plate shear buckling coefficient |
| $(h/t_w)/\sqrt{k_v E/F_y}$ | 0.933 | | | Web shear coefficient (formula limits) |
| $C_v$ | 1.00 | | (G2-2) | Web shear coefficient |
| $V_n$ | 94.5 | kips | (G2-1) | Nominal shear strength |
| $\phi V_n$ | 94.5 | kips | | Design shear strength |
| $V_u$ | 26.75 | kips | | Required shear strength |
| D/C | 0.28 | | | Demand/Capacity ratio |

| Beam Deflection | | | |
|---|---|---|---|
| L / 240 | 0.710 | in | Total load deflection criteria |
| L / 360 | 0.473 | in | Live load deflection criteria |
| $\Delta_{TL}$ | 0.411 | in | Maximum deflection under total load |
| $\Delta_{LL}$ | 0.260 | in | Maximum deflection under live load |
| $I_x$ | 199.00 | $in^4$ | Moment of Inertia about Major Axis |
| $I_{min}$ | 115.21 | $in^4$ | Minimum Moment of Inertia |
| D/C | 0.58 | | |

| Wide Flange or Channel Shape | W16x31 | | |
|---|---|---|---|
| $F_y$ | 50 | ksi | Yield strength |
| $E$ | 29000 | ksi | Modulus of Elasticity |
| $G$ | 11200 | ksi | Shear Modulus of Elasticity |
| $A$ | 9.13 | $in^2$ | Cross Sectional Area of beam |
| $I_x$ | 375.00 | $in^4$ | Moment of Inertia about Major Axis |
| $S_x$ | 47.20 | $in^3$ | Section Modulus about Major Axis |
| $Z_x$ | 54.00 | $in^3$ | Plastic Modulus about Major Axis |
| $I_y$ | 12.40 | $in^4$ | Moment of Inertia about Minor Axis |
| $r_y$ | 1.17 | in | Radius of Gyration about Minor Axis |
| $J$ | 0.46 | $in^4$ | Polar Moment of Inertia |
| $C_w$ | 739 | $in^6$ | Warping Constant |
| $d$ | 15.90 | in | Depth |
| $t_w$ | 0.28 | in | Web Thickness |
| $t_f$ | 0.44 | in | Flange Thickness |
| $b_f$ | 5.53 | in | Flange Width |
| $k$ | 0.84 | in | Fillet Dimension |
| $h$ | 13.34 | in | Distance flanges less the fillet |
| $h_0$ | 15.46 | in | Distance between centroids of flanges |
| $A_w$ | 4.37 | $in^2$ | Area of web |
| | N | *Use LTB modification factor? (Y/N)* | |
| $C_b$ | 1.00 | | Modification Factor for Nonuniform Moment |
| $L_b$ | 2 | ft | Distance between lateral braces |

| Service Loads | | |
|---|---|---|
| Dead | 41 | psf |
| Live | 100 | psf |
| Snow or Live Roof | 0 | psf |
| Garage/Assembly? | N | (Y/N) |

| Service Load Combinations | | |
|---|---|---|
| Combo 1: D + L | 141 | psf |
| Combo 2: D + S | 41 | psf |
| Combo 3: D + 0.75L + 0.75S | 116 | psf |
| Governing Combination | **141** | psf |

| Geometry | | |
|---|---|---|
| Simple Span | 25 | ft |
| Tributary Width | 6.00 | ft |
| Beam Slope | 0 | /12 |
| | 0.00 | degrees |
| | 0.0000 | radians |
| Slope Factor | 1.000 | |
| Sloped Length | 25.00 | ft |

| Factored Load Combinations | | |
|---|---|---|
| Combo 1: 1.4D | 57 | psf |
| Combo 2: 1.2D + 1.6L + 0.5($L_r$ or S) | 209 | psf |
| Combo 3: 1.2D + 1.6(Lr or S) + 0.5L | 99 | psf |
| Governing Combination | **209** | psf |
| Avg. Load Factor for Governing Load Case | 1.484 | |

| Reactions | $R_1$ | $R_2$ |
|---|---|---|
| Total Service, kips | 10.58 | 10.58 |
| Total Factored, kips | 15.69 | 15.69 |

| Governing Limit State | |
|---|---|
| Deflection | 0.58 |

## 13th Ed AISC Manual, LRFD Design

**Beam Flexural Strength**

| Check for Lateral Torsional Buckling (LTB) | | | | |
|---|---|---|---|---|
| $L_b$ | 2 | ft | | Distance between lateral braces |
| $L_p$ | 4.13 | ft | (F2-5) | Limiting unbraced length for limit state of yielding |
| $L_r$ | 11.9 | ft | (F2-6) | Limiting unbraced length for limit state of inelastic LTB |
| $r_{ts}^2$ | 2.03 | in | (F2-7) | Square of effective radius of gyration |
| $c$ | 1.00 | | (F2-8a,b) | LTB constant |
| $F_{cr}$ | 1014.81 | ksi | (F2-4) | Elastic LTB stress |
| $M_p = (M_n)_1$ | 225.0 | k*ft | (F2-1) | if $L_b \le L_p$ -- Plastic moment |
| $(M_n)_2$ | 225.0 | k*ft | (F2-2) | if $L_p < L_b \le L_r$--Inelastic buckling moment |
| $(M_n)_3$ | 225.0 | k*ft | (F2-3) | if $L_b > L_r$ -- Elastic buckling moment |
| **Check for Flange Local Buckling** | | | | |
| $\lambda$ | 6.28 | | | Width-thickness ratio for flange |
| $\lambda_p$ | 9.15 | | Table B4.1-1 | Limiting slenderness for compact flange |
| $\lambda_r$ | 24.08 | | | Limiting slenderness for non-compact flange |
| | **Compact** | | | Slenderness Check |
| $k_c$ | 0.57 | | F3.2 | Coefficient for slender, unstiffened elements |
| $M_n$ | 225.0 | k*ft | (F3-1) | Nominal flexural strength |
| **Flexural Strength Summary** | | | | |
| $\phi_b$ | 0.9 | | | Resistance factor for bending |
| $M_n$ | 225.0 | k*ft | | Nominal flexural strength |
| $\phi M_n$ | 202.5 | k*ft | | Design flexural strength |
| $M_u$ | 98.06 | k*ft | | Required flexural strength |
| D/C | 0.48 | | | Demand/Capacity ratio |

**Beam Shear Strength**

| | | | | |
|---|---|---|---|---|
| $\phi_v$ | 1.0 | | | Resistance factor for shear |
| $a$ | 0 | in | | Transverse stiffener clear spacing (0 = no stiffeners) |
| $k_v$ | 5.000 | | G2.1(ii) | Web plate shear buckling coefficient |
| $(h/t_w)/\sqrt{k_v E/F_y}$ | 0.901 | | | Web shear coefficient (formula limits) |
| $C_v$ | 1.00 | | (G2-2) | Web shear coefficient |
| $V_n$ | 131.2 | kips | (G2-1) | Nominal shear strength |
| $\phi V_n$ | 131.2 | kips | | Design shear strength |
| $V_u$ | 15.69 | kips | | Required shear strength |
| D/C | 0.12 | | | Demand/Capacity ratio |

**Beam Deflection**

| | | | |
|---|---|---|---|
| L / 240 | 1.250 | in | Total load deflection criteria |
| L / 360 | 0.833 | in | Live load deflection criteria |
| $\Delta_{TL}$ | 0.684 | in | Maximum deflection under total load |
| $\Delta_{LL}$ | 0.485 | in | Maximum deflection under live load |
| $I_x$ | 375.00 | $in^4$ | Moment of Inertia about Major Axis |
| $I_{min}$ | 218.21 | $in^4$ | Minimum Moment of Inertia |
| D/C | 0.58 | | |

KL&A Engineers & Builders

| Title | Date | 11/11/21 | Job No. |
|---|---|---|---|
| Subject | By | | Sheet |

| Project Settings | | | | | |
|---|---|---|---|---|---|
| Concrete Strength, $f'_c$ | 3 | ksi | Alowable Soil Bearing, q | 3 | ksf |
| Reinf Cover | 2.50 | in | Soil Density, $\gamma$ | 100 | pcf |
| Reinf Yeild Strength, $f_y$ | 60 | ksi | | | |

**FOOTING DESIGN SUMMARY** V2-2

*Limit States*

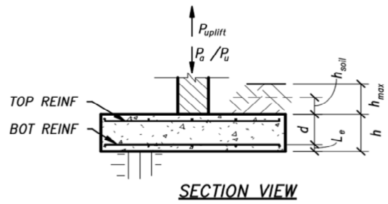
| | Footing Mark | Minimum Col Size | | Design Capacity | | | Footing Size | | | Bot Reinf | | Top Reinf | | Soil Height | | Soil Brg | Punching | BM Shear | Flexure | Uplift | Neg Flx |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | B (in) | N (in) | $\phi P_n$ (LRFD) (kip) | $P/\Omega$ (ASD) (kip) | $P_{uplift}$ (ASD) (kip) | b (in) | L (in) | h (in) | Qty | Bar | Qty | Bar | min (ft) | max (ft) | D/C | D/C | D/C | D/C | D/C | D/C |
| PASS | F1 | 12.0 | 12.0 | 128 | 85 | 0 | 64 | 64 | 12 | (6) | #5 | () | #5 | 1.0 | 4.0 | 1.00 | 0.89 | 0.70 | 0.76 | 0.00 | 0.00 |
| PASS | F2 | 12.0 | 12.0 | 162 | 108 | 0 | 72 | 72 | 14 | (7) | #5 | () | #5 | 1.0 | 4.0 | 1.00 | 0.85 | 0.64 | 0.84 | 0.00 | 0.00 |
| PASS | F3 | 12.0 | 12.0 | 162 | 108 | 0 | 72 | 72 | 14 | (7) | #5 | () | #5 | 1.0 | 4.0 | 1.00 | 0.85 | 0.64 | 0.84 | 0.00 | 0.00 |
| PASS | F4 | 12.0 | 12.0 | 113 | 75 | 0 | 60 | 60 | 12 | (6) | #5 | () | #5 | 1.0 | 4.0 | 1.00 | 0.77 | 0.61 | 0.70 | 0.00 | 0.00 |
| PASS | F5 | 12.0 | 12.0 | 72 | 48 | 0 | 48 | 48 | 12 | (5) | #5 | () | #5 | 1.0 | 4.0 | 1.00 | 0.45 | 0.36 | 0.67 | 0.00 | 0.00 |
| | F6 | | | | | | | | | | | | | | | | | | | | |
| | F7 | | | | | | | | | | | | | | | | | | | | |
| | F8 | | | | | | | | | | | | | | | | | | | | |
| | F9 | | | | | | | | | | | | | | | | | | | | |
| | F10 | | | | | | | | | | | | | | | | | | | | |
| | F11 | | | | | | | | | | | | | | | | | | | | |
| | F12 | | | | | | | | | | | | | | | | | | | | |
| | F13 | | | | | | | | | | | | | | | | | | | | |
| | F14 | | | | | | | | | | | | | | | | | | | | |
| | F15 | | | | | | | | | | | | | | | | | | | | |
| | F16 | | | | | | | | | | | | | | | | | | | | |
| | F17 | | | | | | | | | | | | | | | | | | | | |
| | F18 | | | | | | | | | | | | | | | | | | | | |
| | F19 | | | | | | | | | | | | | | | | | | | | |
| | F20 | | | | | | | | | | | | | | | | | | | | |

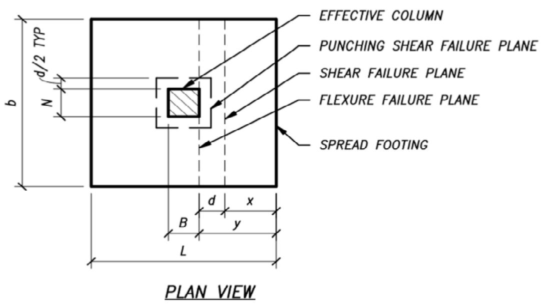
| Title | Date 6/25/14 | Job No. |
|---|---|---|
| Subject | By | Sheet |

## CONCENTRIC SPREAD FOOTING DESIGN

Governing Code: ACI 318-11 V2-2

| | | | |
|---|---|---|---|
| **FOOTING NUMBER** | = | F1 | |
| **DESIGN CAPACITY** | | | |
| Axial (ASD), $P/\Omega$ | = | 85 | kip |
| Axial (LRFD), $\phi P_n$ | = | 128 | kip |
| Net Uplift (ASD), $P_{uplift}$ | = | 0 | kip |
| **FOOTING CRITERIA** | | | |
| Concrete Strength, $f'_c$ | = | 3 | ksi |
| Footing Width, b | = | 64 | in |
| Footing Length, L | = | 64 | in |
| Footing Thickness, h | = | 12 | in |
| **BOTTOM REINFORCMENT** | | | |
| Quantity | = | (6) | |
| Reinforcing bar | = | #5 | |
| Reinforcing strength, $f_y$ | = | 60 | ksi |
| Reinforcing bar diam, $d_b$ | = | 0.63 | in |
| Reinforcing bar Area, $A_b$ | = | 0.31 | $in^2$ |
| Reinf Clear Cover, $L_e$ | = | 2.5 | in |




| | | | |
|---|---|---|---|
| **TOP REINFORCMENT** | | | |
| Quantity | = | () | |
| Reinforcing bar | = | #5 | |
| Reinforcing bar diam | = | 0.63 | in |
| Reinforcing bar Area | = | 0.31 | $in^2$ |

| | | | |
|---|---|---|---|
| **SOIL CRITERIA** | | | |
| Soil Bearing, q | = | 3 | ksf |
| Soil Density, $\gamma$ | = | 100 | pcf |
| Min Soil Height, $h_{soil}$ | = | 1.0 | ft |
| Max Soil Height, $h_{max}$ | = | 4.0 | ft |

| | | | |
|---|---|---|---|
| **MINIMUM COLUMN SIZE** | | | |
| Depth, B | = | 12.0 | in |
| Width, N | = | 12.0 | in |

**DESIGN SUMMARY**

Footing 5' - 4" x 5' - 4" x 12 " w/ (6) #5B EW

**LIMIT STATE SUMMARY**

| | D/C | | Limit |
|---|---|---|---|
| Soil Bearing | 1.00 | ≤ | 1.0 |
| FTG Punching Shear | 0.89 | ≤ | 1.0 |
| FTG Beam Shear | 0.70 | ≤ | 1.0 |
| FTG Flexure | 0.76 | ≤ | 1.0 |
| Uplift | 0.00 | ≤ | 1.0 |
| FTG Negative Flexure | N/A | > | 1.0 |

# CONCENTRIC SPREAD FOOTING DESIGN

Governing Code: ACI 318-11 V2-2

**FOOTING NUMBER** = F1

## SOIL BEARING

| | | | | | |
|---|---|---|---|---|---|
| Service Load, $P_a$ | | | = | 85.3 | kip |
| Ultimate Load, $P_u$ | | | = | 128.0 | kip |
| Footing Width, b | | | = | 64 | in |
| Footing Length, L | | | = | 64 | in |
| Area of Footing, A | = | bL | = | 4096 | $in^2$ |
| Soil Allowable | | | = | 3 | ksf |
| Applied Soil Pressure, $q_a$ | = | $P_a/A$ | = | 3.0 | ksf |
| Ultimate Soil Rxn, $q_u$ | = | $P_u/A$ | = | 0.031 | ksi |

**$q_a/q$**

**1 ≤ 1.0 OK**

## FOOTING PUNCHING SHEAR

| | | | | | | |
|---|---|---|---|---|---|---|
| Resistance Factor, $\phi$ | | | = | 0.75 | | |
| Depth of Reinforcing, d | = | $h - L_e - d_b/2$ | = | 9.2 | in | |
| Effective Col Width, $B_e$ | = | B | = | 12.0 | in | |
| Effective Col Depth, $N_e$ | = | N | = | 12.0 | in | |
| Perimeter, $b_o$ | = | $2(N_e+d)+2(B_e+d)$ | = | 85 | in | |
| $\alpha_s$ | | | = | 40 | | |
| $\beta$ | = | $N_e/B_e$ | = | 1.0 | | |
| Punching Force, $R_u$ | = | $P_u - q_u\,(N_e+d)\,(B_e+d)$ | = | 114.0 | kip | |
| *Punching Shear Strength, $V_c$* | | | | | | |
| Eqn (11-33) | = | $(2+4/\beta)(f'_c)$^0.5 $b_o$ d | = | 256 | kip | |
| Eqn (11-34) | = | $[(\alpha_s\ d)/b_o + 2)(f'_c)$^0.5 $b_o$ d | = | 270 | kip | |
| Eqn (11-35) | = | $4(f'_c)$^0.5 $b_o$ d | = | 171 | kip | *Controls* |
| Punching Shear Design Strength, $\phi R_n$ | | | = | 128 | kip | |

**$R_u$ / $\phi R_n$**

**0.89 ≤ 1.0 OK**

## CONCENTRIC SPREAD FOOTING DESIGN

Governing Code: ACI 318-11 V2-2

**FOOTING NUMBER** = F1

### FOOTING BEAM SHEAR

| | | | | | |
|---|---|---|---|---|---|
| Resistance Factor, $\phi$ | | | = | 0.75 | |
| Distance to Shear Plane, x | = | $b/2 - [\min(B_e, N_e)/2 + d]$ | = | 16.8 | in |
| Shear Force, $R_u$ | = | $q_u(x)L$ | = | 33.6 | kip |
| Shear Strength, $V_c$ | = | $2\,(f'_c)$^0.5 b d | = | 64.4 | kip |
| Beam Shear Design Strength, $\phi R_n$ | | | = | 48.3 | kip |

**$R_u$ / $\phi R_n$**
**0.70 ≤ 1.0 OK**

### FOOTING FLEXURE

| | | | | | |
|---|---|---|---|---|---|
| Resistance Factor, $\phi$ | | | = | 0.9 | |
| Distance to Moment, y | = | $b/2 - \min(B_e, N_e)/2$ | = | 26.0 | in |
| Moment, $M_u$ | = | $q_u\, b\, y^2/2$ | = | 676.0 | k-in |
| Minimum Steel, $A_{s,min}$ | = | 0.0018*b*h | = | 1.38 | $in^2$ |
| Bars Used, $n_b$ | | | = | (6) | |
| Area of Steel, $A_s$ | = | $n_b$*$A_b$ | = | 1.86 | $in^2$ |
| $A_{s,min}/A_s$ | | | = | 0.74 | |
| a | = | $A_s\, F_y/(0.85\, f'_c\, b)$ | = | 0.68 | in |
| Moment Capacity, $M_n$ | = | $A_s\, F_y\, (d - a/2)$ | = | 987 | k-in |
| Design Moment Capacity, $\phi M_n$ | | | = | 888 | k-in |

**$R_u$ / $\phi R_n$**
**0.76 ≤ 1.0 OK**

### UPLIFT

| | | | | | |
|---|---|---|---|---|---|
| Uplift Load, $P_{uplift}$ | | | = | 0 | kip |
| Volume of Concrete, $V_{ftg}$ | = | bLh | = | 28 | cuft |
| Volume of Soil, $V_{soil}$ | = | $bLh_{soil}$ | = | 28 | cuft |
| Dead Load, DL | = | $V_{ftg}$*$\gamma_{conc}$+$V_{soil}$*$\gamma$ | = | 7 | kip |
| Effective Dead Load, 0.6DL | | | = | 4 | kip |

**D/C**
**0.00 ≤ 1.0 OK**

### FOOTING NEGATIVE FLEXURE

| | | | | | |
|---|---|---|---|---|---|
| Soil weight, $q_{soil}$ | = | $h_{max}\gamma$ | = | 2.8E-03 | ksi |
| Footing Self Weight, $q_{ftg}$ | | | = | 1.0E-03 | ksi |
| Moment, $M_u$ | = | $q_{soil}\, b\, y^2/2$ | = | 83 | k-in |
| Bars Used, $n_b$ | | | = | 0 | |
| Area of Steel, $A_s$ | = | $n_b$*$A_b$ | = | 0 | $in^2$ |
| a | = | $A_s\, F_y/(0.85\, f'_c\, b)$ | = | 0.00 | in |
| Moment Capacity, $M_n$ | = | $A_s\, F_y\, (d - a/2)$ | = | 0 | k-in |
| Design Moment Capacity, $\phi M_n$ | | | = | 0 | k-in |

**$R_u$ / $\phi R_n$**
**N/A > 1.0 NG**

KL&A Engineers & Builders

| Title | | Date | 6/25/14 | Job No. |
|---|---|---|---|---|
| Subject | | By | | Sheet |

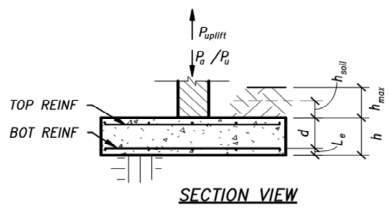
## CONCENTRIC SPREAD FOOTING DESIGN

Governing Code: ACI 318-11 V2-2

**FOOTING NUMBER** = F2

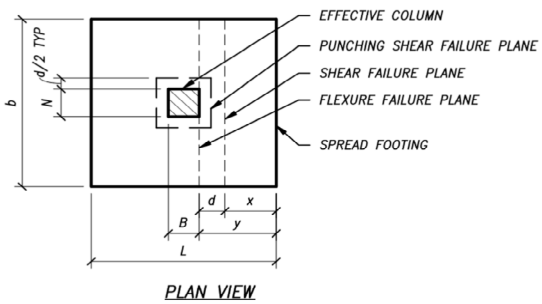
**DESIGN CAPACITY**

| | | | |
|---|---|---|---|
| Axial (ASD), $P/\Omega$ | = | 108 | kip |
| Axial (LRFD), $\phi P_n$ | = | 162 | kip |
| Net Uplift (ASD), $P_{uplift}$ | = | 0 | kip |




**FOOTING CRITERIA**

| | | | |
|---|---|---|---|
| Concrete Strength, $f'_c$ | = | 3 | ksi |
| Footing Width, b | = | 72 | in |
| Footing Length, L | = | 72 | in |
| Footing Thickness, h | = | 14 | in |




**BOTTOM REINFORCMENT**

| | | | |
|---|---|---|---|
| Quantity | = | (7) | |
| Reinforcing bar | = | #5 | |
| Reinforcing strength, $f_y$ | = | 60 | ksi |
| Reinforcing bar diam, $d_b$ | = | 0.63 | in |
| Reinforcing bar Area, $A_b$ | = | 0.31 | $in^2$ |
| Reinf Clear Cover, $L_e$ | = | 2.5 | in |

**TOP REINFORCMENT**

| | | | |
|---|---|---|---|
| Quantity | = | () | |
| Reinforcing bar | = | #5 | |
| Reinforcing bar diam | = | 0.63 | in |
| Reinforcing bar Area | = | 0.31 | $in^2$ |

**SOIL CRITERIA**

| | | | |
|---|---|---|---|
| Soil Bearing, q | = | 3 | ksf |
| Soil Density, $\gamma$ | = | 100 | pcf |
| Min Soil Height, $h_{soil}$ | = | 1.0 | ft |
| Max Soil Height, $h_{max}$ | = | 4.0 | ft |

**MINIMUM COLUMN SIZE**

| | | | |
|---|---|---|---|
| Depth, B | = | 12.0 | in |
| Width, N | = | 12.0 | in |

**DESIGN SUMMARY**

Footing 6' - 0" x 6' - 0" x 14 " w/ (7) #5B EW

**LIMIT STATE SUMMARY**

| | D/C | | Limit |
|---|---|---|---|
| Soil Bearing | 1.00 | ≤ | 1.0 |
| FTG Punching Shear | 0.85 | ≤ | 1.0 |
| FTG Beam Shear | 0.64 | ≤ | 1.0 |
| FTG Flexure | 0.84 | ≤ | 1.0 |
| Uplift | 0.00 | ≤ | 1.0 |
| FTG Negative Flexure | N/A | > | 1.0 |

KL&A Engineers & Builders

| Title | Date 6/25/14 | Job No. |
|---|---|---|
| Subject | By | Sheet |

## CONCENTRIC SPREAD FOOTING DESIGN

Governing Code: ACI 318-11 V2-2

**FOOTING NUMBER** = F2

### SOIL BEARING

| | | | | |
|---|---|---|---|---|
| Service Load, $P_a$ | | = | 108.0 | kip |
| Ultimate Load, $P_u$ | | = | 162.0 | kip |
| Footing Width, b | | = | 72 | in |
| Footing Length, L | | = | 72 | in |
| Area of Footing, A | = bL | = | 5184 | $in^2$ |
| Soil Allowable | | = | 3 | ksf |
| Applied Soil Pressure, $q_a$ | = $P_a/A$ | = | 3.0 | ksf |
| Ultimate Soil Rxn, $q_u$ | = $P_u/A$ | = | 0.031 | ksi |

**$q_a/q$**
**1 ≤ 1.0 OK**

### FOOTING PUNCHING SHEAR

| | | | | | |
|---|---|---|---|---|---|
| Resistance Factor, $\phi$ | | = | 0.75 | | |
| Depth of Reinforcing, d | = $h - L_e - d_b/2$ | = | 11.2 | in | |
| Effective Col Width, $B_e$ | = B | = | 12.0 | in | |
| Effective Col Depth, $N_e$ | = N | = | 12.0 | in | |
| Perimeter, $b_o$ | = $2(N_e+d)+2(B_e+d)$ | = | 93 | in | |
| $\alpha_s$ | | = | 40 | | |
| $\beta$ | = $N_e/B_e$ | = | 1.0 | | |
| Punching Force, $R_u$ | = $P_u - q_u (N_e+d) (B_e+d)$ | = | 145.2 | kip | |
| *Punching Shear Strength, $V_c$* | | | | | |
| Eqn (11-33) | = $(2+4/\beta)(f'_c)$^0.5 $b_o$ d | = | 341 | kip | |
| Eqn (11-34) | = $[(\alpha_s d)/b_o + 2)(f'_c)$^0.5 $b_o$ d | = | 388 | kip | |
| Eqn (11-35) | = $4(f'_c)$^0.5 $b_o$ d | = | 227 | kip | *Controls* |
| Punching Shear Design Strength, $\phi R_n$ | | = | 171 | kip | |

**$R_u / \phi R_n$**
**0.85 ≤ 1.0 OK**

# CONCENTRIC SPREAD FOOTING DESIGN

Governing Code: ACI 318-11 V2-2

**FOOTING NUMBER** = F2

## FOOTING BEAM SHEAR

| | | | | |
|---|---|---|---|---|
| Resistance Factor, $\phi$ | | = | 0.75 | |
| Distance to Shear Plane, x | = $b/2 - [\min(B_e, N_e)/2 + d]$ | = | 18.8 | in |
| Shear Force, $R_u$ | = $q_u(x)L$ | = | 42.3 | kip |
| Shear Strength, $V_c$ | = $2\ (f'_c)$^0.5 b d | = | 88.2 | kip |
| Beam Shear Design Strength, $\phi R_n$ | | = | 66.2 | kip |

**$R_u$ / $\phi R_n$**
**0.64 ≤ 1.0 OK**

## FOOTING FLEXURE

| | | | | |
|---|---|---|---|---|
| Resistance Factor, $\phi$ | | = | 0.9 | |
| Distance to Moment, y | = $b/2 - \min(B_e, N_e)/2$ | = | 30.0 | in |
| Moment, $M_u$ | = $q_u\ b\ y^2/2$ | = | 1012.5 | k-in |
| Minimum Steel, $A_{s,min}$ | = 0.0018*b*h | = | 1.81 | $in^2$ |
| Bars Used, $n_b$ | | = | (7) | |
| Area of Steel, $A_s$ | = $n_b*A_b$ | = | 2.17 | $in^2$ |
| $A_{s,min}/A_s$ | | = | 0.84 | |
| a | = $A_s\ F_y/(0.85\ f'_c\ b)$ | = | 0.71 | in |
| Moment Capacity, $M_n$ | = $A_s\ F_y\ (d - a/2)$ | = | 1410 | k-in |
| Design Moment Capacity,$\phi M_n$ | | = | 1269 | k-in |

**$R_u$ / $\phi R_n$**
**0.84 ≤ 1.0 OK**

## UPLIFT

| | | | | |
|---|---|---|---|---|
| Uplift Load, $P_{uplift}$ | | = | 0 | kip |
| Volume of Concrete, $V_{ftg}$ | = bLh | = | 42 | cuft |
| Volume of Soil, $V_{soil}$ | = $bLh_{soil}$ | = | 36 | cuft |
| Dead Load, DL | = $V_{ftg}*\gamma_{conc}+V_{soil}*\gamma$ | = | 10 | kip |
| Effective Dead Load, 0.6DL | | = | 6 | kip |

**D/C**
**0.00 ≤ 1.0 OK**

## FOOTING NEGATIVE FLEXURE

| | | | | |
|---|---|---|---|---|
| Soil weight, $q_{soil}$ | = $h_{max}\gamma$ | = | 2.8E-03 | ksi |
| Footing Self Weight, $q_{ftg}$ | | = | 1.2E-03 | ksi |
| Moment, $M_u$ | = $q_{soil}\ b\ y^2/2$ | = | 129 | k-in |
| Bars Used, $n_b$ | | = | 0 | |
| Area of Steel, $A_s$ | = $n_b*A_b$ | = | 0 | $in^2$ |
| a | = $A_s\ F_y/(0.85\ f'_c\ b)$ | = | 0.00 | in |
| Moment Capacity, $M_n$ | = $A_s\ F_y\ (d - a/2)$ | = | 0 | k-in |
| Design Moment Capacity,$\phi M_n$ | | = | 0 | k-in |

**$R_u$ / $\phi R_n$**
**N/A > 1.0 NG**

**Column Base Plates for Axial Compression (HSS Columns)**
per AISC Steel Manual, 13th Ed. Or 14th Ed, p 14-4, LRFD Analysis

| Design | Mark | Column Section | B (in) | N (in) | $f'_c$ (psi) | $P_u$ (kips) | $F_y$ (ksi) | $A_2$ (in$^2$) | $t_{min}$ | Recommended BP | Comments |
|---|---|---|---|---|---|---|---|---|---|---|---|
| C1 | 0.570088 | HSS4X4X1/2 | 10 | 10 | 4000 | 98 | 50 | | 0.71 in | 10"x10"x0.75" | (E) Column |
| C2 | 0.857353 | HSS6X6X1/2 | 12 | 12 | 4000 | 104 | 50 | | 0.64 in | 12"x12"x0.75" | (E) Column |
| C3 | 1.176239 | HSS4X4X1/2 | 5.5 | 12 | 4000 | 88 | 50 | | 1.00 in | 5.5"x12"x1" | |

Project Name: Basecamp
Project Number: 21304
Engineer: SYE
Date: 11/11/2021

| | | | |
|---|---|---|---|
| Project Name: | Basecamp | Engineer: | SYE |
| Project Number: | 21304 | Date: | 11/11/2021 |

## Column Base Plates for Axial Compression

per AISC Steel Manual, 13th Ed. Or 14th Ed, p 14-4, LRFD Analysis

### General Input

| | | | |
|---|---|---|---|
| | C1 | | Base Plate Design |
| Column Section | **HSS4X4X1/2** | | HSS Column |
| *B* | **10** | in | Width Base Plate |
| *N* | **10** | in | Length of Base Plate |
| $f'_c$ | **4000** | psi | 28-day Compressive Strength of Concrete |
| $P_u$ | **98** | kips | Required Compressive Strength |
| $F_y$ | **50** | ksi | Yield Strength of Base Plate |
| *B* | 4.00 | in | Width of HSS |
| *H* | 4.00 | in | Depth of HSS |

### Calculations

| | | | |
|---|---|---|---|
| $A_1$ | 100 | $in^2$ | Area of steel concentrically bearing on concrete support |
| $A_2$ | 0 | $in^2$ | area of the lower base of the largest frustum of a pyramid, contained wholly within the support and having for its upper base the loaded areas, and having side slopes of 1 vertical to 2 horizontal (Enter 0 to ignore bearing load multiplier) |
| $\sqrt{A_2/A_1}$ <=2.0 | 1.000 | | Bearing load multiplier |
| $\phi P_p$ | 221 | kips | Design bearing load on concrete ($\phi$=0.65 per ACI 318-02) |
| *X* | 0.44 | | |
| $\lambda$ | 0.76 | | |
| $\lambda n'$ | 0.76 | in | Cantilever dimension for base plate |
| *n* | 3.40 | in | Cantilever dimension for base plate |
| *m* | 3.10 | in | Cantilever dimension for base plate |
| *l* | 3.40 | in | Critical cantilever dimension for base plate |
| $t_{min}$ | 0.71 | in | Minimum base plate thickness |
| | 3/4 | in | Practical base plate thickness |

| | | |
|---|---|---|
| base plate dimensions | **10"x10"x0.75"** | |
| weight of baseplate | 21.30 | lb |
| base plate mark | **0.570087713** | |

Comments: (E) Column

201-16

# Column Base Plates for Axial Compression (HSS Columns)

per AISC Steel Manual, 13th Ed. Or 14th Ed, p 14-4, LRFD Analysis

## General Input

| | | | |
|---|---|---|---|
| | **C3** | | Base Plate Design |
| Column Section | **HSS4X4X1/2** | | HSS Column |
| $B$ | **5.5** | in | Width Base Plate |
| $N$ | **12** | in | Length of Base Plate |
| $f'_c$ | **4000** | psi | 28-day Compressive Strength of Concrete |
| $P_u$ | **88** | kips | Required Compressive Strength |
| $F_y$ | **36** | ksi | Yield Strength of Base Plate |
| $B$ | 4.00 | in | Width of HSS |
| $H$ | 4.00 | in | Depth of HSS |

## Calculations

| | | | |
|---|---|---|---|
| $A_1$ | 66 | $in^2$ | Area of steel concentrically bearing on concrete support |
| $A_2$ | | $in^2$ | area of the lower base of the largest frustum of a pyramid, contained wholly within the support and having for its upper base the loaded areas, and having side slopes of 1 vertical to 2 horizontal (Enter 0 to ignore bearing load multiplier) |
| $\sqrt{A_2/A_1} \le 2.0$ | 1.000 | | Bearing load multiplier |
| $\phi P_p$ | 146 | kips | Design bearing load on concrete ($\phi$=0.65 per ACI 318-14) |
| | detmine minimum baseplate bearing area | | |
| A reqd | 40 | $in^2$ | minimum area for bearing conconrete |
| $X$ | 0.60 | | |
| $\lambda$ | 0.95 | | |
| $\lambda n'$ | 0.95 | in | Cantilever dimension for base plate |
| $n$ | 1.15 | in | Cantilever dimension for base plate |
| $m$ | 4.10 | in | Cantilever dimension for base plate |
| $l$ | 4.10 | in | Critical cantilever dimension for base plate |
| $t_{min}$ | 1.18 | in | Minimum base plate thickness |
| | 1 1/4 | in | Practical base plate thickness |
| base plate dimensions | **5.5"x12"x1.25"** | | |
| weight of baseplate | 23.43 | lb | |
| base plate mark | **B1A** | | |

| | | | |
|---|---|---|---|
| Title | Basecamp Infill | Job No. | 21304 |
| | | Date | 11/2/2021 |
| Description | | By | SYE |
| | Column/Beam Connection | | |

## Web Check at Concentrated Load per AISC 360 15th Edition - LRFD Design

**Shape** **W18x40**

| | | | |
|---|---|---|---|
| $F_{yw}$ | 50 | ksi | Yield strength of web |
| $F_{uw}$ | 65 | ksi | Tensile strength of web |
| E | 29000 | ksi | Modulus of Elasticity |
| $t_w$ | 0.32 | in | Web Thickness |
| $t_f$ | 0.53 | in | Flange Thickness |
| k | 0.93 | in | Fillet Dimension |
| d | 17.90 | in | Depth |
| h | 16.05 | in | Web Height |
| $b_f$ | 6.02 | in | Flange Width |
| $l_b$ | 11 | in | Length of Bearing |
| $Q_f$ | 1 | | See definition in Section J10.3 |
| $C_r$ | 960000 | ksi | See definition in Section J10.4 |

**Concentrated Load Input**

| | | |
|---|---|---|
| Concentrated Load, $P_u$ | 60 | kips |
| Location of Load from Member End | 60 | in |
| Compression Flange Restrained for Sidesway Buckling | Yes | |
| Largest laterally unbraced length along either flange at the point of load, $L_b$ | 16 | in |
| Compression load occurs on both sides of beam? | Yes | |

**Stiffener Input**

| | | |
|---|---|---|
| Stiffener Type | Full | Depth |
| Stiffener width, b | 6 | in |
| Stiffener thickness, $t_s$ | 0.5 | in |
| Yield strength of stiffener, $F_{ys}$ | 36 | ksi |
| Yield strength of stiffener, $F_{us}$ | 58 | ksi |
| Stiffener Weld Thickness, D | 1/4 | in |

**Limit State Summary**

| | $R_u$ / $\phi R_n$ | | Limit | |
|---|---|---|---|---|
| Web Local Yielding | 0.24 | ≤ | 1.0 | |
| Web Local Crippling | 0.35 | ≤ | 1.0 | |
| Web Sidesway Buckling | N/A | ≤ | 1.0 | |
| Web Compression Buckling | 1.18 | > | 1.0 | Web Compression Buckling OK, Stiffener Provided |
| Stiffener Shear | 0.03 | ≤ | 1.0 | |
| Stiffener Weld | 0.02 | ≤ | 1.0 | |
| Stiffener Buckling | 0.03 | ≤ | 1.0 | |

**Web Local Yielding per J10.2**

$\phi$ 1.0

| | | | | |
|---|---|---|---|---|
| D/C | **0.24** | $\phi R_n$ | 246.3 kips | When the concentrated force to be resisted is applied at a distance from the member end that is greater than the depth of the member, *d* |
| D/C | **0.29** | $\phi R_n$ | 209.8 kips | When the concentrated force to be resisted is applied at a distance from the member end that is less than or equal to the depth of the member, *d* |

**Web Local Crippling per J10.3**

$\phi$ 0.75

| | | | | |
|---|---|---|---|---|
| D/C | **0.35** | $\phi R_n$ | 171.9 kips | When the concentrated force to be resisted is applied at a distance from the member end that is greater than or equal to *d* / 2 |
| | | | | When the concentrated force to be resisted is applied at a distance from the member end that is less than *d* / 2 |
| D/C | **0.70** | $\phi R_n$ | 85.9 kips | For *lb* / *d* ≤ 0.2 |

| | | | | |
|---|---|---|---|---|
| D/C | **0.63** | $\phi R_n$ | 94.8 kips | For *lb / d* > 0.2 |

## Web Sidesway Buckling per J10.4

$\phi$ 0.85

Compression flange is restrained against rotation

| | | | | |
|---|---|---|---|---|
| | $(h/t_w)/(L_b/b_f)$ | | 19.17 | compactness parameter |
| D/C | N/A | $\phi R_n$ n/a | kips | Sidesway web buckling will not control if the compactness parameter is > 2.3 |

Compression flange is *not* restrained against rotation

| | | | | |
|---|---|---|---|---|
| | $(h/t_w)/(L_b/b_f)$ | | 19.17 | compactness parameter |
| D/C | N/A | $\phi R_n$ n/a | kips | Sidesway web buckling will not control if the compactness parameter is > 1.7 |

## Web Compression Buckling per J10.5

$\phi$ 0.9

| | | | | |
|---|---|---|---|---|
| D/C | 1.18 | $\phi R_n$ | 50.7 kips | When pair of concentrated forces are applied at a distance from the member end that is greater than or equal to half the depth of the member, *d / 2* |
| D/C | 2.37 | $\phi R_n$ | 25.3 kips | When pair of concentrated forces are applied at a distance from the member end that is less than half the depth of the member, *d / 2* |

**Partial Height Stiffener Design** (assumes identical stiffeners, one on each side of web. Such stiffeners are appropriate for web yielding, web crippling, and web sidesway.)

| | | | | | |
|---|---|---|---|---|---|
| $\phi R_n$ | 171.9 | kips | Governing design strength (from above) for partial length stiffener design. | | |
| $R_u$ | 60 | kips | Required strength | | |
| **Stiffener Definition** | | | | | |
| $b_f$ | 6.02 | in | Flange width | | |
| $b_{min}$ | 1.85 | in | Minimum stiffener width per J10.8 | | |
| b | 6 | in | Stiffener width | | |
| $t_{s\_min}$ | 0.375 | in | Minimum stiffener thickness | | |
| $t_s$ | 0.5 | in | Stiffener thickness | | |
| $t_w$ | 0.315 | in | Beam web thickness | | |
| $L_{min}$ | 8.95 | in | Minimum stiffener length (half the depth of the web) | | |
| $L_s$ | 0 | in | Actual stiffener length | | |
| $A_{vs}$ | 0.00 | $in^2$ | Shear area of single stiffener | | |
| **Shear Check per J4.2** | | | | | |
| $V_u$ | -55.9 | kips | Required shear strength per stiffener = $(Ru - \phi Rn) / 2$ | | |
| $\phi$ | 1.0 | | Resistance factor for shear | | |
| $\phi V_n$ | 0.0 | kips | Stiffener shear design strength | D/C | N/A |
| **Weld Check per J2.4** | | | | | |
| $V_u$ | -55.9 | kips | Required shear strength per stiffener = $(Ru - \phi Rn) / 2$ | | |
| $\phi$ | 0.75 | | Resistance factor for shear | | |
| $D_{max}$ | 0.29 | in | Maximum Effective Weld Size | | |
| D | 0.25 | in | Weld Size | | |
| $l_w$ | -1.81 | in | Weld Length | | |
| $\phi V_n$ | 0.0 | kips | Weld shear design strength | D/C | N/A |

**Full Height Stiffener Design** (assumes identical stiffeners, one on each side of web. Such stiffeners are appropriate if Compression Buckling of Web is an issue.)

| | | | |
|---|---|---|---|
| $\phi R_n$ | 50.7 | kips | Input governing design strength (from above) for full length stiffener design. |
| $R_u$ | 60 | kips | Required strength |
| **Stiffener Definition** | | | |
| $b_f$ | 6.02 | in | Flange width |
| $b_{min}$ | 1.85 | in | Minimum stiffener width per J10.8 |
| b | 6 | in | Stiffener width |

| | | | | | |
|---|---|---|---|---|---|
| $t_{s\_min}$ | 0.375 | in | Minimum stiffener thickness | | |
| $t_s$ | 0.5 | in | Stiffener thickness | | |
| $t_w$ | 0.315 | in | Beam web thickness | | |
| $L_s$ | 16.85 | in | Stiffener length | | |
| $L_{seff}$ | 13.23 | in | Effective Stiffener Length accounting for beam k and clearances required | | |
| $A_{e\_end}$ | 7.191 | $in^2$ | Effective area of end stiffener column | | |
| $A_{e\_interior}$ | 8.481 | $in^2$ | Effective area of interior stiffener column | | |
| $I_e$ | 77.819 | $in^4$ | Effective moment of inertia of stiffeners about major axis | | |
| $r_{end}$ | 3.290 | in | Radius of gyration for end stiffeners | | |
| $r_{interior}$ | 3.029 | in | Radius of gyration for interior stiffeners | | |
| $A_v$ | 6.61 | $in^2$ | Shear area of single stiffener | | |

**Shear Check per J4.2**

| | | | | | |
|---|---|---|---|---|---|
| $V_u$ | 4.7 | kips | Required shear strength per stiffener = (*Ru* -ϕ*Rn* ) / 2 | | |
| ϕ | 1.0 | | Resistance factor for shear | | |
| $\phi V_n$ | 142.8 | kips | Stiffener shear design strength | D/C | **0.03** |

**Weld Check per J2.4**

| | | | | | |
|---|---|---|---|---|---|
| $V_u$ | 4.7 | kips | Required shear strength per stiffener = (*Ru* -ϕ*Rn* ) / 2 | | |
| ϕ | 0.75 | | Resistance factor for shear | | |
| $D_{max}$ | 0.29 | in | Maximum Effective Weld Size | | |
| D | 0.25 | in | Weld Size | | |
| $l_w$ | 13.23 | in | Weld Length | | |
| $\phi V_n$ | 294.5 | kips | Weld shear design strength | D/C | **0.02** |

**Buckling Check**

**Compact Compression per J10.4**

| | | | |
|---|---|---|---|
| $V_u$ | 9.3 | kips | Required compressive strength = *Ru* -ϕ*Rn* |
| | | ϕ | 0.90 |
| K | 0.75 | | Effective Length Factor |
| $L_c$ | 12.0345 | in | Effective Length if Section per J10.8 |

Stiffener is at the Interior of the Beam, > $12.5t_w$

| | | | | |
|---|---|---|---|---|
| | | $L_c/r$ | 3.97 | Compactness Parameter - Interior |
| D/C | **0.03** | $\phi R_n$ | 274.772 kips | When is stiffener section is determined to be compact per J10.4 and is at the interior of the beam |

Stiffener is at the End of the Beam, ≤ $12.5t_w$

| | | | | |
|---|---|---|---|---|
| | | $L_c/r$ | 3.66 | Compactness Parameter - End |
| D/C | **0.04** | $\phi R_n$ | 233.0 kips | When is stiffener section is determined to be compact per J10.4 and is at the end of the beam |

**Nonslender Compression per E3**

| | | |
|---|---|---|
| ϕ | 0.90 | |
| λ | 12 | Slenderness Parameter |

Stiffener is at the Interior of the Beam, > $12.5t_w$

| | | | | |
|---|---|---|---|---|
| | | $L_c/r$ | 3.97 | Compactness Parameter - Interior |
| | | $F_e$ | 18134.2 ksi | Elastic Buckling Stress |
| | | $F_{cr}$ | 36.0 | Critical Buckling Stress |
| D/C | **0.03** | $\phi R_n$ | 274.5 kips | When is stiffener section is determined to be nonslender per B4.1 and E3 and is at the interior of the beam |

Stiffener is at the End of the Beam, ≤ $12.5t_w$

| | | | | |
|---|---|---|---|---|
| | | $L_c/r$ | 3.66 | Compactness Parameter - End |
| | | $F_e$ | 21387.3 ksi | Elastic Buckling Stress |
| | | $F_{cr}$ | 36.0 ksi | Critical Buckling Stress |
| D/C | **0.04** | $\phi R_n$ | 232.8 ksi | When is stiffener section is determined to be nonslender per B4.1 and E3 and is at the interior of the beam |

**Slender Compression per E7**

| | | | |
|---|---|---|---|
| | $\phi$ | 0.90 | |
| | $\lambda$ | 12 | Slenderness Parameter |
| | $\lambda_r$ | 12.77 | Limiting Width-to-Thickness Ratio |

Stiffener is at the Interior of the Beam, > 12.5$t_w$

| | | | |
|---|---|---|---|
| | $F_{el}$ | 90.5 ksi | Elastic Buckling Stess - Slender |
| | $b_e$ | 6.00 in | Effective Width of Slender Stiffener |
| | $A_{e\_interior}$ | 8.48 $in^2$ | Effective area of interior stiffener column |
| D/C **0.03** | $\phi R_n$ | 274.5 kips | When is stiffener section is determined to be slender per B4.1 and E7 and is at the interior of the beam |

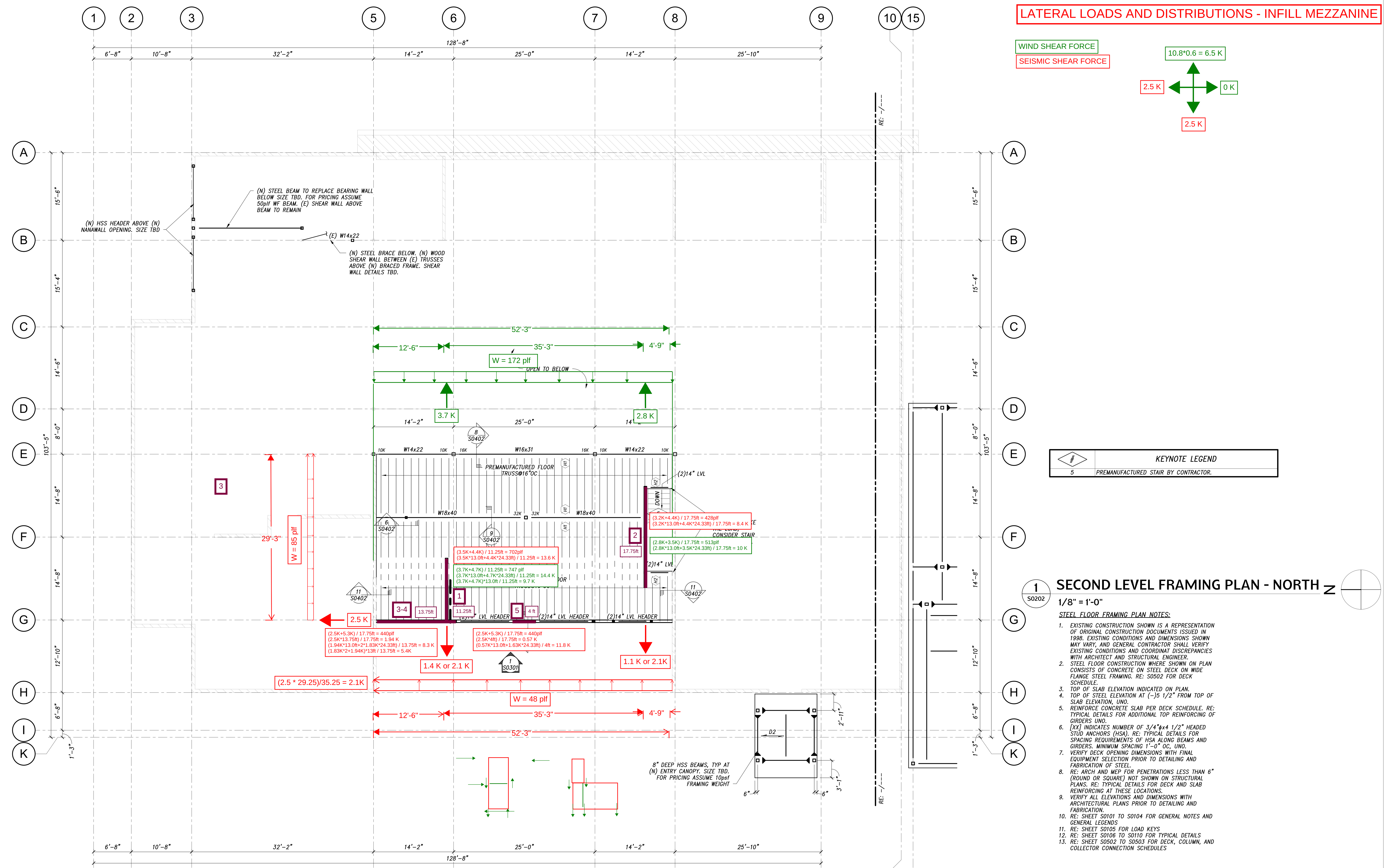
Stiffener is at the End of the Beam, ≤ 12.5$t_w$

| | | | |
|---|---|---|---|
| | $F_{el}$ | 90.5 ksi | Elastic Buckling Stess - Slender |
| | $b_e$ | 6.00 in | Effective Width of Slender Stiffener |
| | $A_{e\_end}$ | 7.19 $in^2$ | Effective area of interior stiffener column |
| D/C **0.04** | $\phi R_n$ | 232.8 kips | When is stiffener section is determined to be slender per B4.1 and E7 and is at the end of the beam |

## 202. LATERAL DESIGN

10/22/2021 4:41:30 PM

# LATERAL LOADS AND DISTRIBUTIONS - INFILL MEZZANINE




| KEYNOTE LEGEND | |
|---|---|
| 5 | PREMANUFACTURED STAIR BY CONTRACTOR. |

## 1 S0202 SECOND LEVEL FRAMING PLAN - NORTH

1/8" = 1'-0"

*STEEL FLOOR FRAMING PLAN NOTES:*

1. EXISTING CONSTRUCTION SHOWN IS A REPRESENTATION OF ORIGINAL CONSTRUCTION DOCUMENTS ISSUED IN 1998. EXISTING CONDITIONS AND DIMENSIONS SHOWN MAY VARY, AND GENERAL CONTRACTOR SHALL VERIFY EXISTING CONDITIONS AND COORDINAT DISCREPANCIES WITH ARCHITECT AND STRUCTURAL ENGINEER.
2. STEEL FLOOR CONSTRUCTION WHERE SHOWN ON PLAN CONSISTS OF CONCRETE ON STEEL DECK ON WIDE FLANGE STEEL FRAMING. RE: S0502 FOR DECK SCHEDULE.
3. TOP OF SLAB ELEVATION INDICATED ON PLAN.
4. TOP OF STEEL ELEVATION AT (–)5 1/2" FROM TOP OF SLAB ELEVATION, UNO.
5. REINFORCE CONCRETE SLAB PER DECK SCHEDULE. RE: TYPICAL DETAILS FOR ADDITIONAL TOP REINFORCING OF GIRDERS UNO.
6. [XX] INDICATES NUMBER OF 3/4"ϕx4 1/2" HEADED STUD ANCHORS (HSA). RE: TYPICAL DETAILS FOR SPACING REQUIREMENTS OF HSA ALONG BEAMS AND GIRDERS. MINIMUM SPACING 1'–0" OC, UNO.
7. VERIFY DECK OPENING DIMENSIONS WITH FINAL EQUIPMENT SELECTION PRIOR TO DETAILING AND FABRICATION OF STEEL.
8. RE: ARCH AND MEP FOR PENETRATIONS LESS THAN 6" (ROUND OR SQUARE) NOT SHOWN ON STRUCTURAL PLANS. RE: TYPICAL DETAILS FOR DECK AND SLAB REINFORCING AT THESE LOCATIONS.
9. VERIFY ALL ELEVATIONS AND DIMENSIONS WITH ARCHITECTURAL PLANS PRIOR TO DETAILING AND FABRICATION.
10. RE: SHEET S0101 TO S0104 FOR GENERAL NOTES AND GENERAL LEGENDS
11. RE: SHEET S0105 FOR LOAD KEYS
12. RE: SHEET S0106 TO S0110 FOR TYPICAL DETAILS
13. RE: SHEET S0502 TO S0503 FOR DECK, COLUMN, AND COLLECTOR CONNECTION SCHEDULES

APPROVAL STAMPS:

NOT FOR CONSTRUCTION

| No. | Date | Description |
|---|---|---|

SUBMISSIONS & REVISIONS

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Boulder, CO 80302
T. 303.444.6038

INTERIOR DESIGNER:
**JOHNSON NATHAN STROHE**
1600 Wynkoop St., Suite 100
Denver, CO 80202
T.303.892.7062

PROJECT LOCATION
STEAMBOAT BASECAMP
1901 CURVE PLAZA
STEAMBOAT SPRINGS, CO 80487

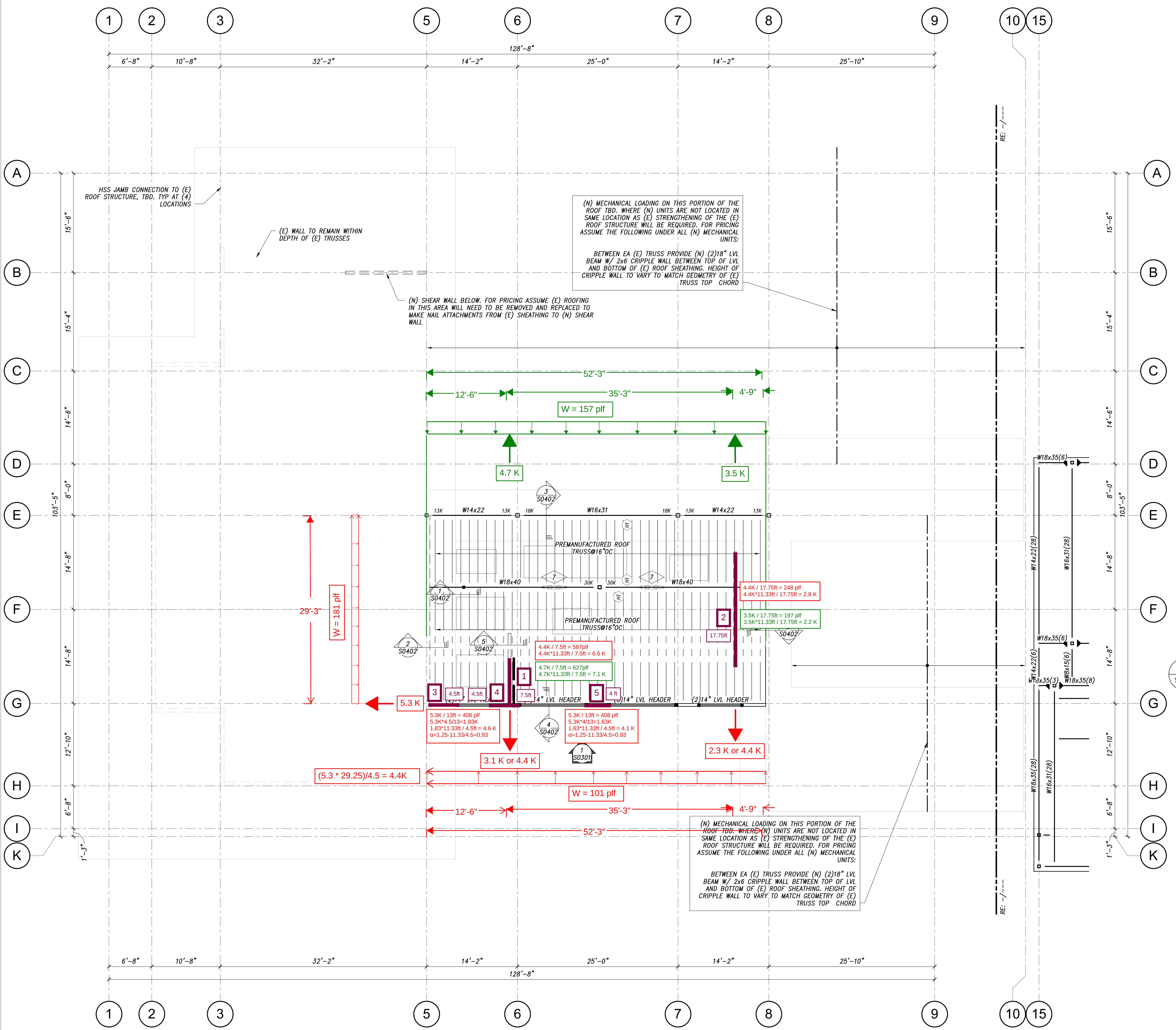
DRAWING TITLE
SECOND LEVEL FRAMING PLAN

SEAL

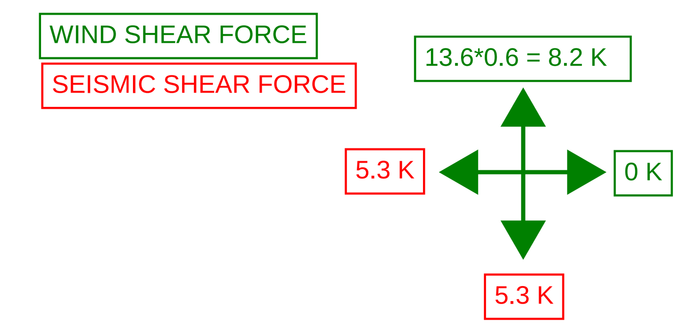
DATE: 07/14/21
DRAWN BY: SYE/JMS
CHECKED BY: PMK
PROJECT NO:

DRAWING NO:
S0202

10/22/2021 4:41:30 PM




# LATERAL LOADS AND DISTRIBUTIONS - INFILL ROOF




| KEYNOTE LEGEND | |
|---|---|
| 7 | FULL HEIGHT BLOCKING BETWEEN TRUSSES, ATTACH SHEATHING TO BLOCKING W/ (3)8d NAILS |

## 1 S0203 ROOF FRAMING PLAN - NORTH

1/8" = 1'-0"

### STEEL ROOF FRAMING PLAN NOTES:

1. EXISTING CONSTRUCTION SHOWN IS A REPRESENTATION OF ORIGINAL CONSTRUCTION DOCUMENTS ISSUED IN 1998. EXISTING CONDITIONS AND DIMENSIONS SHOWN MAY VARY, AND GENERAL CONTRACTOR SHALL VERIFY EXISTING CONDITIONS AND COORDINAT DISCREPANCIES WITH ARCHITECT AND STRUCTURAL ENGINEER.
2. ROOF CONSTRUCTION CONSISTS OF ROOF DECK ON WIDE FLANGE STEEL FRAMING. RE: S0502 FOR DECK SCHEDULE.
3. BOTTOM OF DECK ELEVATION INDICATED ON PLAN.
4. PROVIDE CONSTANT SLOPE OF DECK AND FRAMING BETWEEN ELEVATIONS GIVEN ON PLAN.
5. TOP OF STEEL ELEVATION = BOTTOM OF DECK
6. VERIFY DECK OPENING DIMENSIONS WITH FINAL EQUIPMENT SELECTION PRIOR TO DETAILING AND FABRICATION OF STEEL.
7. RE: ARCH AND MEP FOR PENETRATIONS LESS THAN 6" (ROUND OR SQUARE) NOT SHOWN ON STRUCTURAL PLANS. RE: TYPICAL DETAILS FOR DECK REINFORCING AT THESE LOCATIONS.
8. VERIFY ALL ELEVATIONS AND DIMENSIONS WITH ARCHITECTURAL PLANS PRIOR TO DETAILING AND FABRICATION.
9. RE: SHEET S0101 TO S0104 FOR GENERAL NOTES AND GENERAL LEGENDS
10. RE: SHEET S0105 FOR LOAD KEYS
11. RE: SHEET S0106 TO S0110 FOR TYPICAL DETAILS
12. RE: SHEET S0502 TO S0503 FOR DECK AND COLLECTOR CONNECTION SCHEDULES

N

APPROVAL STAMPS:

NOT FOR CONSTRUCTION

| No. | Date | Description |
|---|---|---|

SUBMISSIONS & REVISIONS

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T. 303.444.6038

INTERIOR DESIGNER:
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Denver, CO 80202
T.303.892.7062

PROJECT LOCATION
STEAMBOAT BASECAMP
1901 CURVE PLAZA
STEAMBOAT SPRINGS, CO 80487

DRAWING TITLE
ROOF LEVEL FRAMING PLAN

SEAL

DATE: 06/28/21
DRAWN BY: SYE/JMS
CHECKED BY: PMK
PROJECT NO:

DRAWING NO:
**S0203**

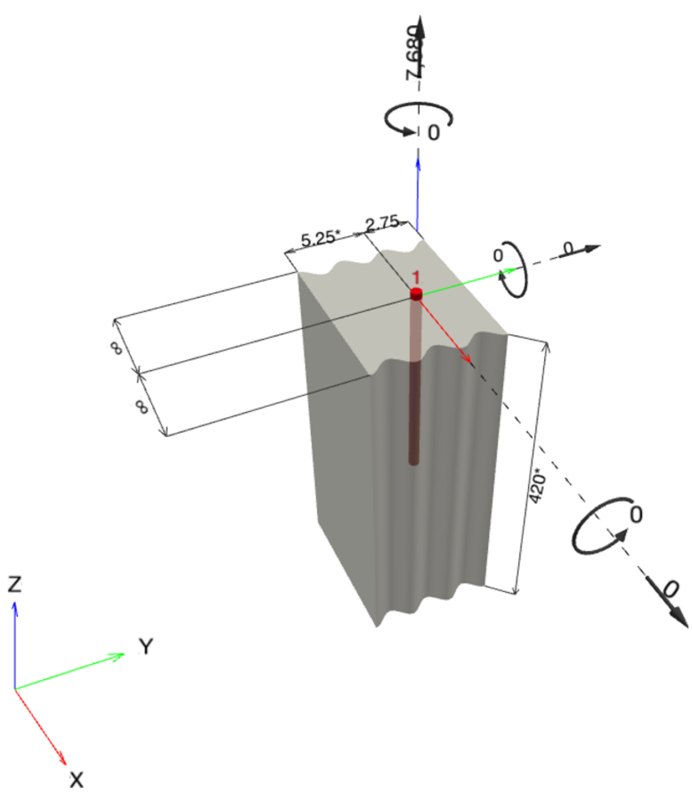
| | | | |
|---|---|---|---|
| Company: | | Page: | 1 |
| Address: | | Specifier: | |
| Phone I Fax: | \| | E-Mail: | |
| Design: | Drafts_211102_HD1_SYE | Date: | 11/12/2021 |
| Fastening point: | | | |

**Specifier's comments:**

# 1 Input data

| | |
|---|---|
| **Anchor type and diameter:** | **Hex Head ASTM F 1554 GR. 36 5/8** |
| Item number: | not available |
| Effective embedment depth: | $h_{ef}$ = 12.000 in. |
| Material: | ASTM F 1554 |
| Evaluation Service Report: | Hilti Technical Data |
| Issued I Valid: | - \| - |
| Proof: | Design Method ACI 318-14 / CIP |
| Stand-off installation: | |
| Profile: | |
| Base material: | cracked concrete, 4000, $f_c'$ = 4,000 psi; h = 420.000 in. |
| Reinforcement: | tension: condition A, shear: condition B; anchor reinforcement: tension |
| | edge reinforcement: none or < No. 4 bar |

**Geometry [in.] & Loading [lb, in.lb]**



| | | | |
|---|---|---|---|
| Company: | | Page: | 2 |
| Address: | | Specifier: | |
| Phone I Fax: | \| | E-Mail: | |
| Design: | Drafts_211102_HD1_SYE | Date: | 11/12/2021 |
| Fastening point: | | | |

**1.1 Design results**

| Case | Description | Forces [lb] / Moments [in.lb] | Seismic | Max. Util. Anchor [%] |
|---|---|---|---|---|
| 1 | Combination 1 | N = 7,680; $V_x$ = 0; $V_y$ = 0;<br>$M_x$ = 0; $M_y$ = 0; $M_z$ = 0; | no | 79 |

## 2 Load case/Resulting anchor forces

**Anchor reactions [lb]**

Tension force: (+Tension, -Compression)

| Anchor | Tension force | Shear force | Shear force x | Shear force y |
|---|---|---|---|---|
| 1 | 7,680 | 0 | 0 | 0 |

max. concrete compressive strain: - [‰]
max. concrete compressive stress: - [psi]
resulting tension force in (x/y)=(0.000/0.000): 0 [lb]
resulting compression force in (x/y)=(0.000/0.000): 0 [lb]

## 3 Tension load

| | Load $N_{ua}$ [lb] | Capacity $\phi$ $N_n$ [lb] | Utilization $\beta_N = N_{ua}/\phi\ N_n$ | Status |
|---|---|---|---|---|
| Steel Strength* | 7,680 | 9,831 | 79 | OK |
| Pullout Strength* | 7,680 | 10,170 | 76 | OK |
| Concrete Breakout Failure**[1] | N/A | N/A | N/A | N/A |
| Concrete Side-Face Blowout, direction y+** | 7,680 | 14,063 | 55 | OK |

* highest loaded anchor **anchor group (anchors in tension)

[1] Tension Anchor Reinforcement has been selected!

| | | | |
|---|---|---|---|
| Company: | | Page: | 3 |
| Address: | | Specifier: | |
| Phone I Fax: | \| | E-Mail: | |
| Design: | Drafts_211102_HD1_SYE | Date: | 11/12/2021 |
| Fastening point: | | | |

### 3.1 Steel Strength

$N_{sa} = A_{se,N}\, f_{uta}$ ACI 318-14 Eq. (17.4.1.2)

$\phi\ N_{sa} \geq N_{ua}$ ACI 318-14 Table 17.3.1.1

**Variables**

| $A_{se,N}$ [in.$^2$] | $f_{uta}$ [psi] |
|---|---|
| 0.23 | 58,000 |

**Calculations**

| $N_{sa}$ [lb] |
|---|
| 13,108 |

**Results**

| $N_{sa}$ [lb] | $\phi_{steel}$ | $\phi\ N_{sa}$ [lb] | $N_{ua}$ [lb] |
|---|---|---|---|
| 13,108 | 0.750 | 9,831 | 7,680 |

### 3.2 Pullout Strength

$N_{pN} = \psi_{c,p}\, N_p$ ACI 318-14 Eq. (17.4.3.1)

$N_p = 8\, A_{brg}\, f'_c$ ACI 318-14 Eq. (17.4.3.4)

$\phi\ N_{pN} \geq N_{ua}$ ACI 318-14 Table 17.3.1.1

**Variables**

| $\psi_{c,p}$ | $A_{brg}$ [in.$^2$] | $\lambda_a$ | $f'_c$ [psi] |
|---|---|---|---|
| 1.000 | 0.45 | 1.000 | 4,000 |

**Calculations**

| $N_p$ [lb] |
|---|
| 14,528 |

**Results**

| $N_{pn}$ [lb] | $\phi_{concrete}$ | $\phi\ N_{pn}$ [lb] | $N_{ua}$ [lb] |
|---|---|---|---|
| 14,528 | 0.700 | 10,170 | 7,680 |

### 3.3 Concrete Side-Face Blowout, direction y+

$N_{sb} = 160\ \alpha_{corner}\ c_{a1}\ \sqrt{A_{brg}}\ \lambda_a\ \sqrt{f'_c}$ ACI 318-14 Eq. (17.4.4.1)

$\phi\ N_{sb} \geq N_{ua}$ ACI 318-14 Table 17.3.1.1

$\alpha_{corner} = \dfrac{\left(1 + \dfrac{c_{a2}}{c_{a1}}\right)}{4}$ see ACI 318-14, Section 17.4.4.1

**Variables**

| $c_{a1}$ [in.] | $c_{a2}$ [in.] | $A_{brg}$ [in.$^2$] | $\lambda_a$ | $f'_c$ [psi] | s [in.] |
|---|---|---|---|---|---|
| 2.750 | - | 0.45 | 1.000 | 4,000 | - |

**Calculations**

| $\alpha_{corner}$ | $N_{sb}$ [lb] |
|---|---|
| 1.000 | 18,750 |

**Results**

| $N_{sb}$ [lb] | $\phi_{concrete}$ | $\phi\ N_{sb}$ [lb] | $N_{ua}$ [lb] |
|---|---|---|---|
| 18,750 | 0.750 | 14,063 | 7,680 |

## 4 Shear load

| | Load $V_{ua}$ [lb] | Capacity $\phi$ $V_n$ [lb] | Utilization $\beta_V = V_{ua}/\phi$ $V_n$ | Status |
|---|---|---|---|---|
| Steel Strength* | N/A | N/A | N/A | N/A |
| Steel failure (with lever arm)* | N/A | N/A | N/A | N/A |
| Pryout Strength* | N/A | N/A | N/A | N/A |
| Concrete edge failure in direction ** | N/A | N/A | N/A | N/A |

* highest loaded anchor **anchor group (relevant anchors)

## 5 Warnings

- The anchor design methods in PROFIS Engineering require rigid anchor plates per current regulations (AS 5216:2021, ETAG 001/Annex C, EOTA TR029 etc.). This means load re-distribution on the anchors due to elastic deformations of the anchor plate are not considered - the anchor plate is assumed to be sufficiently stiff, in order not to be deformed when subjected to the design loading. PROFIS Engineering calculates the minimum required anchor plate thickness with CBFEM to limit the stress of the anchor plate based on the assumptions explained above. The proof if the rigid anchor plate assumption is valid is not carried out by PROFIS Engineering. Input data and results must be checked for agreement with the existing conditions and for plausibility!
- Condition A applies where the potential concrete failure surfaces are crossed by supplementary reinforcement proportioned to tie the potential concrete failure prism into the structural member. Condition B applies where such supplementary reinforcement is not provided, or where pullout or pryout strength governs.
- For additional information about ACI 318 strength design provisions, please go to https://submittals.us.hilti.com/PROFISAnchorDesignGuide/
- The design of Anchor Reinforcement is beyond the scope of PROFIS Engineering. Refer to ACI 318-14, Section 17.4.2.9 for information about Anchor Reinforcement.
- Anchor Reinforcement has been selected as a design option, calculations should be compared with PROFIS Engineering calculations.

**Fastening meets the design criteria!**

| | | | |
|---|---|---|---|
| Company: | | Page: | 6 |
| Address: | | Specifier: | |
| Phone I Fax: | \| | E-Mail: | |
| Design: | Drafts_211102_HD1_SYE | Date: | 11/12/2021 |
| Fastening point: | | | |

## 6 Installation data

Profile: -

Hole diameter in the fixture: -

Plate thickness (input): -

Anchor type and diameter: Hex Head ASTM F 1554 GR. 36 5/8

Item number: not available

Maximum installation torque: -

Hole diameter in the base material: - in.

Hole depth in the base material: 12.000 in.

Minimum thickness of the base material: 12.922 in.

Hilti Hex Head headed stud anchor with 12 in embedment, 5/8, Steel galvanized, installation per instruction for use

**Coordinates Anchor in.**

| Anchor | x | y | $c_{-x}$ | $c_{+x}$ | $c_{-y}$ | $c_{+y}$ |
|---|---|---|---|---|---|---|
| 1 | 0.000 | 0.000 | - | - | 5.250 | 2.750 |

## 7 Remarks; Your Cooperation Duties

- Any and all information and data contained in the Software concern solely the use of Hilti products and are based on the principles, formulas and security regulations in accordance with Hilti's technical directions and operating, mounting and assembly instructions, etc., that must be strictly complied with by the user. All figures contained therein are average figures, and therefore use-specific tests are to be conducted prior to using the relevant Hilti product. The results of the calculations carried out by means of the Software are based essentially on the data you put in. Therefore, you bear the sole responsibility for the absence of errors, the completeness and the relevance of the data to be put in by you. Moreover, you bear sole responsibility for having the results of the calculation checked and cleared by an expert, particularly with regard to compliance with applicable norms and permits, prior to using them for your specific facility. The Software serves only as an aid to interpret norms and permits without any guarantee as to the absence of errors, the correctness and the relevance of the results or suitability for a specific application.
- You must take all necessary and reasonable steps to prevent or limit damage caused by the Software. In particular, you must arrange for the regular backup of programs and data and, if applicable, carry out the updates of the Software offered by Hilti on a regular basis. If you do not use the AutoUpdate function of the Software, you must ensure that you are using the current and thus up-to-date version of the Software in each case by carrying out manual updates via the Hilti Website. Hilti will not be liable for consequences, such as the recovery of lost or damaged data or programs, arising from a culpable breach of duty by you.

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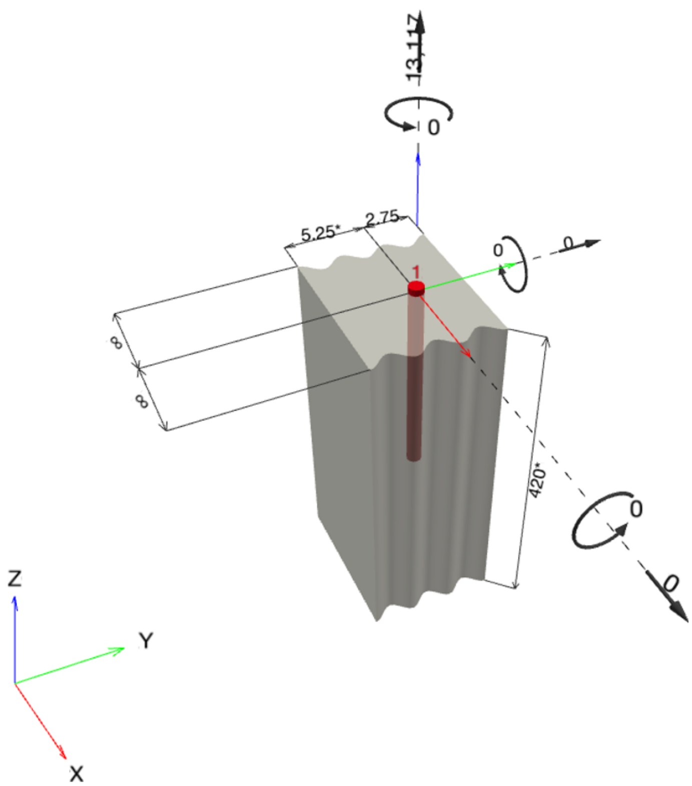
| | | | |
|---|---|---|---|
| Company: | | Page: | 1 |
| Address: | | Specifier: | |
| Phone I Fax: | \| | E-Mail: | |
| Design: | Drafts_211102_HD2_SYE | Date: | 11/12/2021 |
| Fastening point: | | | |


**Specifier's comments:**

## 1 Input data

| | |
|---|---|
| **Anchor type and diameter:** | **Hex Head ASTM F 1554 GR. 36 7/8** |
| Item number: | not available |
| Effective embedment depth: | $h_{ef}$ = 12.000 in. |
| Material: | ASTM F 1554 |
| Evaluation Service Report: | Hilti Technical Data |
| Issued I Valid: | - \| - |
| Proof: | Design Method ACI 318-14 / CIP |
| Stand-off installation: | |
| Profile: | |
| Base material: | cracked concrete, 4000, $f_c'$ = 4,000 psi; h = 420.000 in. |
| Reinforcement: | tension: condition A, shear: condition B; anchor reinforcement: tension |
| | edge reinforcement: none or < No. 4 bar |

**Geometry [in.] & Loading [lb, in.lb]**



### 1.1 Design results

| Case | Description | Forces [lb] / Moments [in.lb] | Seismic | Max. Util. Anchor [%] |
|---|---|---|---|---|
| 1 | Combination 1 | N = 13,117; $V_x$ = 0; $V_y$ = 0; $M_x$ = 0; $M_y$ = 0; $M_z$ = 0; | no | 67 |

## 2 Load case/Resulting anchor forces

**Anchor reactions [lb]**

Tension force: (+Tension, -Compression)

| Anchor | Tension force | Shear force | Shear force x | Shear force y |
|---|---|---|---|---|
| 1 | 13,117 | 0 | 0 | 0 |

max. concrete compressive strain: - [‰]
max. concrete compressive stress: - [psi]
resulting tension force in (x/y)=(0.000/0.000): 0 [lb]
resulting compression force in (x/y)=(0.000/0.000): 0 [lb]

## 3 Tension load

| | Load $N_{ua}$ [lb] | Capacity $\phi$ $N_n$ [lb] | Utilization $\beta_N = N_{ua}/\phi$ $N_n$ | Status |
|---|---|---|---|---|
| Steel Strength* | 13,117 | 20,097 | 66 | OK |
| Pullout Strength* | 13,117 | 19,958 | 66 | OK |
| Concrete Breakout Failure**[1] | N/A | N/A | N/A | N/A |
| Concrete Side-Face Blowout, direction y+** | 13,117 | 19,701 | 67 | OK |

* highest loaded anchor **anchor group (anchors in tension)

[1] Tension Anchor Reinforcement has been selected!

### 3.1 Steel Strength

$N_{sa} = A_{se,N}\, f_{uta}$ ACI 318-14 Eq. (17.4.1.2)

$\phi\ N_{sa} \geq N_{ua}$ ACI 318-14 Table 17.3.1.1

**Variables**

| $A_{se,N}$ [in.$^2$] | $f_{uta}$ [psi] |
|---|---|
| 0.46 | 58,000 |

**Calculations**

| $N_{sa}$ [lb] |
|---|
| 26,796 |

**Results**

| $N_{sa}$ [lb] | $\phi_{steel}$ | $\phi\ N_{sa}$ [lb] | $N_{ua}$ [lb] |
|---|---|---|---|
| 26,796 | 0.750 | 20,097 | 13,117 |

### 3.2 Pullout Strength

$N_{pN} = \psi_{c,p}\, N_p$ ACI 318-14 Eq. (17.4.3.1)

$N_p = 8\, A_{brg}\, f'_c$ ACI 318-14 Eq. (17.4.3.4)

$\phi\ N_{pN} \geq N_{ua}$ ACI 318-14 Table 17.3.1.1

**Variables**

| $\psi_{c,p}$ | $A_{brg}$ [in.$^2$] | $\lambda_a$ | $f'_c$ [psi] |
|---|---|---|---|
| 1.000 | 0.89 | 1.000 | 4,000 |

**Calculations**

| $N_p$ [lb] |
|---|
| 28,512 |

**Results**

| $N_{pn}$ [lb] | $\phi_{concrete}$ | $\phi\ N_{pn}$ [lb] | $N_{ua}$ [lb] |
|---|---|---|---|
| 28,512 | 0.700 | 19,958 | 13,117 |

### 3.3 Concrete Side-Face Blowout, direction y+

$N_{sb} = 160\ \alpha_{corner}\ c_{a1}\ \sqrt{A_{brg}}\ \lambda_a\ \sqrt{f'_c}$ ACI 318-14 Eq. (17.4.4.1)

$\phi\ N_{sb} \geq N_{ua}$ ACI 318-14 Table 17.3.1.1

$\alpha_{corner} = \dfrac{\left(1 + \dfrac{c_{a2}}{c_{a1}}\right)}{4}$ see ACI 318-14, Section 17.4.4.1

**Variables**

| $c_{a1}$ [in.] | $c_{a2}$ [in.] | $A_{brg}$ [in.$^2$] | $\lambda_a$ | $f'_c$ [psi] | s [in.] |
|---|---|---|---|---|---|
| 2.750 | - | 0.89 | 1.000 | 4,000 | - |

**Calculations**

| $\alpha_{corner}$ | $N_{sb}$ [lb] |
|---|---|
| 1.000 | 26,268 |

**Results**

| $N_{sb}$ [lb] | $\phi_{concrete}$ | $\phi\ N_{sb}$ [lb] | $N_{ua}$ [lb] |
|---|---|---|---|
| 26,268 | 0.750 | 19,701 | 13,117 |

## 4 Shear load

| | Load $V_{ua}$ [lb] | Capacity $\phi$ $V_n$ [lb] | Utilization $\beta_V = V_{ua}/\phi$ $V_n$ | Status |
|---|---|---|---|---|
| Steel Strength* | N/A | N/A | N/A | N/A |
| Steel failure (with lever arm)* | N/A | N/A | N/A | N/A |
| Pryout Strength* | N/A | N/A | N/A | N/A |
| Concrete edge failure in direction ** | N/A | N/A | N/A | N/A |

* highest loaded anchor    **anchor group (relevant anchors)

## 5 Warnings

- The anchor design methods in PROFIS Engineering require rigid anchor plates per current regulations (AS 5216:2021, ETAG 001/Annex C, EOTA TR029 etc.). This means load re-distribution on the anchors due to elastic deformations of the anchor plate are not considered - the anchor plate is assumed to be sufficiently stiff, in order not to be deformed when subjected to the design loading. PROFIS Engineering calculates the minimum required anchor plate thickness with CBFEM to limit the stress of the anchor plate based on the assumptions explained above. The proof if the rigid anchor plate assumption is valid is not carried out by PROFIS Engineering. Input data and results must be checked for agreement with the existing conditions and for plausibility!
- Condition A applies where the potential concrete failure surfaces are crossed by supplementary reinforcement proportioned to tie the potential concrete failure prism into the structural member. Condition B applies where such supplementary reinforcement is not provided, or where pullout or pryout strength governs.
- For additional information about ACI 318 strength design provisions, please go to https://submittals.us.hilti.com/PROFISAnchorDesignGuide/
- The design of Anchor Reinforcement is beyond the scope of PROFIS Engineering. Refer to ACI 318-14, Section 17.4.2.9 for information about Anchor Reinforcement.
- Anchor Reinforcement has been selected as a design option, calculations should be compared with PROFIS Engineering calculations.

## Fastening meets the design criteria!

| | | | |
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| Phone I Fax: | \| | E-Mail: | |
| Design: | Drafts_211102_HD2_SYE | Date: | 11/12/2021 |
| Fastening point: | | | |

## 6 Installation data

Profile: -

Hole diameter in the fixture: -

Plate thickness (input): -

Anchor type and diameter: Hex Head ASTM F 1554 GR. 36 7/8

Item number: not available

Maximum installation torque: -

Hole diameter in the base material: - in.

Hole depth in the base material: 12.000 in.

Minimum thickness of the base material: 13.052 in.

Hilti Hex Head headed stud anchor with 12 in embedment, 7/8, Steel galvanized, installation per instruction for use

**Coordinates Anchor in.**

| Anchor | x | y | $c_{-x}$ | $c_{+x}$ | $c_{-y}$ | $c_{+y}$ |
|---|---|---|---|---|---|---|
| 1 | 0.000 | 0.000 | - | - | 5.250 | 2.750 |

## 7 Remarks; Your Cooperation Duties

- Any and all information and data contained in the Software concern solely the use of Hilti products and are based on the principles, formulas and security regulations in accordance with Hilti's technical directions and operating, mounting and assembly instructions, etc., that must be strictly complied with by the user. All figures contained therein are average figures, and therefore use-specific tests are to be conducted prior to using the relevant Hilti product. The results of the calculations carried out by means of the Software are based essentially on the data you put in. Therefore, you bear the sole responsibility for the absence of errors, the completeness and the relevance of the data to be put in by you. Moreover, you bear sole responsibility for having the results of the calculation checked and cleared by an expert, particularly with regard to compliance with applicable norms and permits, prior to using them for your specific facility. The Software serves only as an aid to interpret norms and permits without any guarantee as to the absence of errors, the correctness and the relevance of the results or suitability for a specific application.
- You must take all necessary and reasonable steps to prevent or limit damage caused by the Software. In particular, you must arrange for the regular backup of programs and data and, if applicable, carry out the updates of the Software offered by Hilti on a regular basis. If you do not use the AutoUpdate function of the Software, you must ensure that you are using the current and thus up-to-date version of the Software in each case by carrying out manual updates via the Hilti Website. Hilti will not be liable for consequences, such as the recovery of lost or damaged data or programs, arising from a culpable breach of duty by you.

| | | | |
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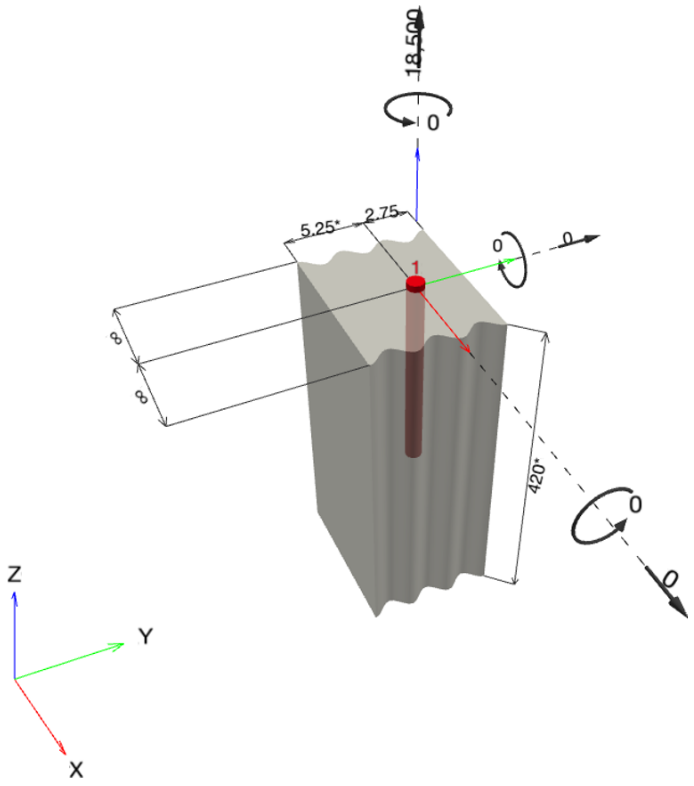
| Phone I Fax: | \| | E-Mail: | |
| Design: | Drafts_211102_HD3_SYE | Date: | 11/12/2021 |
| Fastening point: | | | |

**Specifier's comments:**

# 1 Input data

| | |
|---|---|
| **Anchor type and diameter:** | **Hex Head ASTM F 1554 GR. 36 1** |
| Item number: | not available |
| Effective embedment depth: | $h_{ef}$ = 12.000 in. |
| Material: | ASTM F 1554 |
| Evaluation Service Report: | Hilti Technical Data |
| Issued I Valid: | - \| - |
| Proof: | Design Method ACI 318-14 / CIP |
| Stand-off installation: | |
| Profile: | |
| Base material: | cracked concrete, 4000, $f_c'$ = 4,000 psi; h = 420.000 in. |
| Reinforcement: | tension: condition A, shear: condition B; anchor reinforcement: tension |
| | edge reinforcement: none or < No. 4 bar |

**Geometry [in.] & Loading [lb, in.lb]**



| | | | |
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| Phone I Fax: | \| | E-Mail: | |
| Design: | Drafts_211102_HD3_SYE | Date: | 11/12/2021 |
| Fastening point: | | | |

**1.1 Design results**

| Case | Description | Forces [lb] / Moments [in.lb] | Seismic | Max. Util. Anchor [%] |
|---|---|---|---|---|
| 1 | Combination 1 | $N = 18{,}500$; $V_x = 0$; $V_y = 0$; $M_x = 0$; $M_y = 0$; $M_z = 0$; | no | 83 |

## 2 Load case/Resulting anchor forces

**Anchor reactions [lb]**

Tension force: (+Tension, -Compression)

| Anchor | Tension force | Shear force | Shear force x | Shear force y |
|---|---|---|---|---|
| 1 | 18,500 | 0 | 0 | 0 |

max. concrete compressive strain: - [‰]
max. concrete compressive stress: - [psi]
resulting tension force in (x/y)=(0.000/0.000): 0 [lb]
resulting compression force in (x/y)=(0.000/0.000): 0 [lb]

## 3 Tension load

| | Load $N_{ua}$ [lb] | Capacity $\phi$ $N_n$ [lb] | Utilization $\beta_N = N_{ua}/\phi$ $N_n$ | Status |
|---|---|---|---|---|
| Steel Strength* | 18,500 | 26,361 | 71 | OK |
| Pullout Strength* | 18,500 | 26,051 | 72 | OK |
| Concrete Breakout Failure**[1] | N/A | N/A | N/A | N/A |
| Concrete Side-Face Blowout, direction y+** | 18,500 | 22,508 | 83 | OK |

* highest loaded anchor **anchor group (anchors in tension)

[1] Tension Anchor Reinforcement has been selected!

### 3.1 Steel Strength

$N_{sa} = A_{se,N}\, f_{uta}$ ACI 318-14 Eq. (17.4.1.2)

$\phi\ N_{sa} \geq N_{ua}$ ACI 318-14 Table 17.3.1.1

**Variables**

| $A_{se,N}$ [in.$^2$] | $f_{uta}$ [psi] |
|---|---|
| 0.61 | 58,000 |

**Calculations**

| $N_{sa}$ [lb] |
|---|
| 35,148 |

**Results**

| $N_{sa}$ [lb] | $\phi_{steel}$ | $\phi\ N_{sa}$ [lb] | $N_{ua}$ [lb] |
|---|---|---|---|
| 35,148 | 0.750 | 26,361 | 18,500 |

### 3.2 Pullout Strength

$N_{pN} = \psi_{c,p}\, N_p$ ACI 318-14 Eq. (17.4.3.1)

$N_p = 8\, A_{brg}\, f'_c$ ACI 318-14 Eq. (17.4.3.4)

$\phi\ N_{pN} \geq N_{ua}$ ACI 318-14 Table 17.3.1.1

**Variables**

| $\psi_{c,p}$ | $A_{brg}$ [in.$^2$] | $\lambda_a$ | $f'_c$ [psi] |
|---|---|---|---|
| 1.000 | 1.16 | 1.000 | 4,000 |

**Calculations**

| $N_p$ [lb] |
|---|
| 37,216 |

**Results**

| $N_{pn}$ [lb] | $\phi_{concrete}$ | $\phi\ N_{pn}$ [lb] | $N_{ua}$ [lb] |
|---|---|---|---|
| 37,216 | 0.700 | 26,051 | 18,500 |

### 3.3 Concrete Side-Face Blowout, direction y+

$N_{sb} = 160\ \alpha_{corner}\ c_{a1}\ \sqrt{A_{brg}}\ \lambda_a\ \sqrt{f'_c}$ ACI 318-14 Eq. (17.4.4.1)

$\phi\ N_{sb} \geq N_{ua}$ ACI 318-14 Table 17.3.1.1

$\alpha_{corner} = \dfrac{\left(1 + \dfrac{c_{a2}}{c_{a1}}\right)}{4}$ see ACI 318-14, Section 17.4.4.1

**Variables**

| $c_{a1}$ [in.] | $c_{a2}$ [in.] | $A_{brg}$ [in.$^2$] | $\lambda_a$ | $f'_c$ [psi] | s [in.] |
|---|---|---|---|---|---|
| 2.750 | - | 1.16 | 1.000 | 4,000 | - |

**Calculations**

| $\alpha_{corner}$ | $N_{sb}$ [lb] |
|---|---|
| 1.000 | 30,010 |

**Results**

| $N_{sb}$ [lb] | $\phi_{concrete}$ | $\phi\ N_{sb}$ [lb] | $N_{ua}$ [lb] |
|---|---|---|---|
| 30,010 | 0.750 | 22,508 | 18,500 |

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| | | | |
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| Fastening point: | | | |

## 4 Shear load

| | Load $V_{ua}$ [lb] | Capacity $\phi$ $V_n$ [lb] | Utilization $\beta_V = V_{ua}/\phi$ $V_n$ | Status |
|---|---|---|---|---|
| Steel Strength* | N/A | N/A | N/A | N/A |
| Steel failure (with lever arm)* | N/A | N/A | N/A | N/A |
| Pryout Strength* | N/A | N/A | N/A | N/A |
| Concrete edge failure in direction ** | N/A | N/A | N/A | N/A |

* highest loaded anchor  **anchor group (relevant anchors)

## 5 Warnings

- The anchor design methods in PROFIS Engineering require rigid anchor plates per current regulations (AS 5216:2021, ETAG 001/Annex C, EOTA TR029 etc.). This means load re-distribution on the anchors due to elastic deformations of the anchor plate are not considered - the anchor plate is assumed to be sufficiently stiff, in order not to be deformed when subjected to the design loading. PROFIS Engineering calculates the minimum required anchor plate thickness with CBFEM to limit the stress of the anchor plate based on the assumptions explained above. The proof if the rigid anchor plate assumption is valid is not carried out by PROFIS Engineering. Input data and results must be checked for agreement with the existing conditions and for plausibility!
- Condition A applies where the potential concrete failure surfaces are crossed by supplementary reinforcement proportioned to tie the potential concrete failure prism into the structural member. Condition B applies where such supplementary reinforcement is not provided, or where pullout or pryout strength governs.
- For additional information about ACI 318 strength design provisions, please go to https://submittals.us.hilti.com/PROFISAnchorDesignGuide/
- The design of Anchor Reinforcement is beyond the scope of PROFIS Engineering. Refer to ACI 318-14, Section 17.4.2.9 for information about Anchor Reinforcement.
- Anchor Reinforcement has been selected as a design option, calculations should be compared with PROFIS Engineering calculations.

## Fastening meets the design criteria!

| | | | |
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| Design: | Drafts_211102_HD3_SYE | Date: | 11/12/2021 |
| Fastening point: | | | |

# 6 Installation data

Profile: -

Hole diameter in the fixture: -

Plate thickness (input): -

Anchor type and diameter: Hex Head ASTM F 1554 GR. 36 1

Item number: not available

Maximum installation torque: -

Hole diameter in the base material: - in.

Hole depth in the base material: 12.000 in.

Minimum thickness of the base material: 13.172 in.

Hilti Hex Head headed stud anchor with 12 in embedment, 1, Steel galvanized, installation per instruction for use

**Coordinates Anchor in.**

| Anchor | x | y | $c_{-x}$ | $c_{+x}$ | $c_{-y}$ | $c_{+y}$ |
|---|---|---|---|---|---|---|
| 1 | 0.000 | 0.000 | - | - | 5.250 | 2.750 |

## 7 Remarks; Your Cooperation Duties

- Any and all information and data contained in the Software concern solely the use of Hilti products and are based on the principles, formulas and security regulations in accordance with Hilti's technical directions and operating, mounting and assembly instructions, etc., that must be strictly complied with by the user. All figures contained therein are average figures, and therefore use-specific tests are to be conducted prior to using the relevant Hilti product. The results of the calculations carried out by means of the Software are based essentially on the data you put in. Therefore, you bear the sole responsibility for the absence of errors, the completeness and the relevance of the data to be put in by you. Moreover, you bear sole responsibility for having the results of the calculation checked and cleared by an expert, particularly with regard to compliance with applicable norms and permits, prior to using them for your specific facility. The Software serves only as an aid to interpret norms and permits without any guarantee as to the absence of errors, the correctness and the relevance of the results or suitability for a specific application.
- You must take all necessary and reasonable steps to prevent or limit damage caused by the Software. In particular, you must arrange for the regular backup of programs and data and, if applicable, carry out the updates of the Software offered by Hilti on a regular basis. If you do not use the AutoUpdate function of the Software, you must ensure that you are using the current and thus up-to-date version of the Software in each case by carrying out manual updates via the Hilti Website. Hilti will not be liable for consequences, such as the recovery of lost or damaged data or programs, arising from a culpable breach of duty by you.

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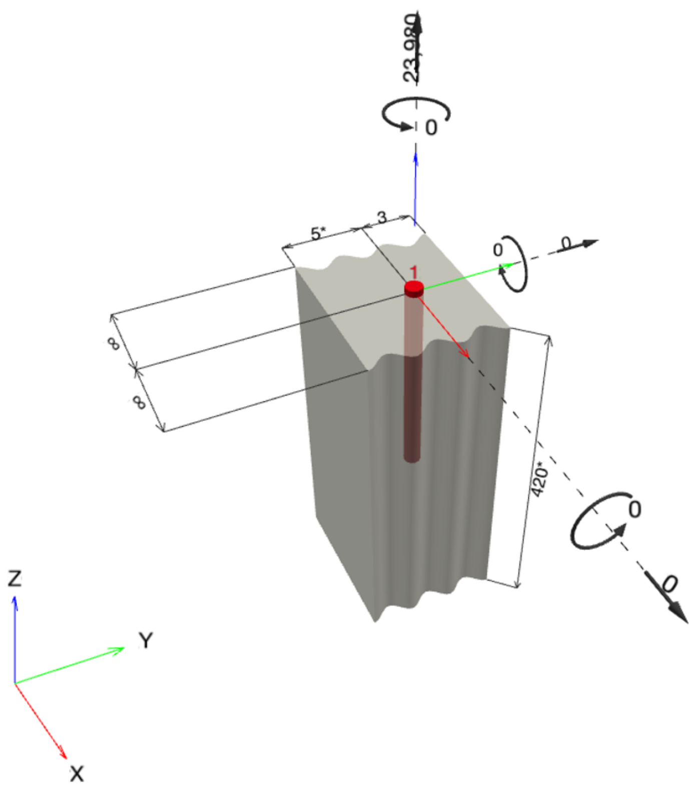
| Design: | Drafts_211102_HD4_SYE | Date: | 11/12/2021 |
| Fastening point: | | | |

**Specifier's comments:**

## 1 Input data

| | |
|---|---|
| **Anchor type and diameter:** | **Hex Head ASTM F 1554 GR. 36 1** |
| Item number: | not available |
| Effective embedment depth: | $h_{ef}$ = 12.000 in. |
| Material: | ASTM F 1554 |
| Evaluation Service Report: | Hilti Technical Data |
| Issued I Valid: | - \| - |
| Proof: | Design Method ACI 318-14 / CIP |
| Stand-off installation: | |
| Profile: | |
| Base material: | cracked concrete, 4000, $f_c'$ = 4,000 psi; h = 420.000 in. |
| Reinforcement: | tension: condition A, shear: condition B; anchor reinforcement: tension |
| | edge reinforcement: none or < No. 4 bar |

**Geometry [in.] & Loading [lb, in.lb]**



### 1.1 Design results

| Case | Description | Forces [lb] / Moments [in.lb] | Seismic | Max. Util. Anchor [%] |
|---|---|---|---|---|
| 1 | Combination 1 | $N = 23{,}980$; $V_x = 0$; $V_y = 0$; $M_x = 0$; $M_y = 0$; $M_z = 0$; | no | 98 |

## 2 Load case/Resulting anchor forces

**Anchor reactions [lb]**

Tension force: (+Tension, -Compression)

| Anchor | Tension force | Shear force | Shear force x | Shear force y |
|---|---|---|---|---|
| 1 | 23,980 | 0 | 0 | 0 |

max. concrete compressive strain: - [‰]
max. concrete compressive stress: - [psi]
resulting tension force in (x/y)=(0.000/0.000): 0 [lb]
resulting compression force in (x/y)=(0.000/0.000): 0 [lb]

## 3 Tension load

| | Load $N_{ua}$ [lb] | Capacity $\phi$ $N_n$ [lb] | Utilization $\beta_N = N_{ua}/\phi$ $N_n$ | Status |
|---|---|---|---|---|
| Steel Strength* | 23,980 | 26,361 | 91 | OK |
| Pullout Strength* | 23,980 | 26,051 | 93 | OK |
| Concrete Breakout Failure**[1] | N/A | N/A | N/A | N/A |
| Concrete Side-Face Blowout, direction y+** | 23,980 | 24,554 | 98 | OK |

* highest loaded anchor **anchor group (anchors in tension)

[1] Tension Anchor Reinforcement has been selected!

### 3.1 Steel Strength

$N_{sa} = A_{se,N}\ f_{uta}$ ACI 318-14 Eq. (17.4.1.2)

$\phi\ N_{sa} \geq N_{ua}$ ACI 318-14 Table 17.3.1.1

**Variables**

| $A_{se,N}$ [in.$^2$] | $f_{uta}$ [psi] |
|---|---|
| 0.61 | 58,000 |

**Calculations**

| $N_{sa}$ [lb] |
|---|
| 35,148 |

**Results**

| $N_{sa}$ [lb] | $\phi_{steel}$ | $\phi\ N_{sa}$ [lb] | $N_{ua}$ [lb] |
|---|---|---|---|
| 35,148 | 0.750 | 26,361 | 23,980 |

### 3.2 Pullout Strength

$N_{pN} = \psi_{c,p}\ N_p$ ACI 318-14 Eq. (17.4.3.1)

$N_p = 8\ A_{brg}\ f'_c$ ACI 318-14 Eq. (17.4.3.4)

$\phi\ N_{pN} \geq N_{ua}$ ACI 318-14 Table 17.3.1.1

**Variables**

| $\psi_{c,p}$ | $A_{brg}$ [in.$^2$] | $\lambda_a$ | $f'_c$ [psi] |
|---|---|---|---|
| 1.000 | 1.16 | 1.000 | 4,000 |

**Calculations**

| $N_p$ [lb] |
|---|
| 37,216 |

**Results**

| $N_{pn}$ [lb] | $\phi_{concrete}$ | $\phi\ N_{pn}$ [lb] | $N_{ua}$ [lb] |
|---|---|---|---|
| 37,216 | 0.700 | 26,051 | 23,980 |

### 3.3 Concrete Side-Face Blowout, direction y+

$N_{sb} = 160\ \alpha_{corner}\ c_{a1}\ \sqrt{A_{brg}}\ \lambda_a\ \sqrt{f'_c}$ ACI 318-14 Eq. (17.4.4.1)

$\phi\ N_{sb} \geq N_{ua}$ ACI 318-14 Table 17.3.1.1

$\alpha_{corner} = \dfrac{\left(1 + \dfrac{c_{a2}}{c_{a1}}\right)}{4}$ see ACI 318-14, Section 17.4.4.1

**Variables**

| $c_{a1}$ [in.] | $c_{a2}$ [in.] | $A_{brg}$ [in.$^2$] | $\lambda_a$ | $f'_c$ [psi] | s [in.] |
|---|---|---|---|---|---|
| 3.000 | - | 1.16 | 1.000 | 4,000 | - |

**Calculations**

| $\alpha_{corner}$ | $N_{sb}$ [lb] |
|---|---|
| 1.000 | 32,739 |

**Results**

| $N_{sb}$ [lb] | $\phi_{concrete}$ | $\phi\ N_{sb}$ [lb] | $N_{ua}$ [lb] |
|---|---|---|---|
| 32,739 | 0.750 | 24,554 | 23,980 |

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| Fastening point: | | | |


## 4 Shear load

| | Load $V_{ua}$ [lb] | Capacity $\phi$ $V_n$ [lb] | Utilization $\beta_V = V_{ua}/\phi$ $V_n$ | Status |
|---|---|---|---|---|
| Steel Strength* | N/A | N/A | N/A | N/A |
| Steel failure (with lever arm)* | N/A | N/A | N/A | N/A |
| Pryout Strength* | N/A | N/A | N/A | N/A |
| Concrete edge failure in direction ** | N/A | N/A | N/A | N/A |

* highest loaded anchor    **anchor group (relevant anchors)

## 5 Warnings

- The anchor design methods in PROFIS Engineering require rigid anchor plates per current regulations (AS 5216:2021, ETAG 001/Annex C, EOTA TR029 etc.). This means load re-distribution on the anchors due to elastic deformations of the anchor plate are not considered - the anchor plate is assumed to be sufficiently stiff, in order not to be deformed when subjected to the design loading. PROFIS Engineering calculates the minimum required anchor plate thickness with CBFEM to limit the stress of the anchor plate based on the assumptions explained above. The proof if the rigid anchor plate assumption is valid is not carried out by PROFIS Engineering. Input data and results must be checked for agreement with the existing conditions and for plausibility!
- Condition A applies where the potential concrete failure surfaces are crossed by supplementary reinforcement proportioned to tie the potential concrete failure prism into the structural member. Condition B applies where such supplementary reinforcement is not provided, or where pullout or pryout strength governs.
- For additional information about ACI 318 strength design provisions, please go to https://submittals.us.hilti.com/PROFISAnchorDesignGuide/
- The design of Anchor Reinforcement is beyond the scope of PROFIS Engineering. Refer to ACI 318-14, Section 17.4.2.9 for information about Anchor Reinforcement.
- Anchor Reinforcement has been selected as a design option, calculations should be compared with PROFIS Engineering calculations.

## Fastening meets the design criteria!

Company:
Address:
Phone I Fax: |
Design: Drafts_211102_HD4_SYE
Fastening point:



Specifier:
E-Mail:
Date: 11/12/2021

## 6 Installation data

Profile: -

Hole diameter in the fixture: -

Plate thickness (input): -

Anchor type and diameter: Hex Head ASTM F 1554 GR. 36 1

Item number: not available

Maximum installation torque: -

Hole diameter in the base material: - in.

Hole depth in the base material: 12.000 in.

Minimum thickness of the base material: 13.172 in.

Hilti Hex Head headed stud anchor with 12 in embedment, 1, Steel galvanized, installation per instruction for use

**Coordinates Anchor in.**

| Anchor | x | y | $c_{-x}$ | $c_{+x}$ | $c_{-y}$ | $c_{+y}$ |
|---|---|---|---|---|---|---|
| 1 | 0.000 | 0.000 | - | - | 5.000 | 3.000 |

| | | | |
|---|---|---|---|
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| Phone I Fax: | \| | E-Mail: | |
| Design: | Drafts_211102_HD4_SYE | Date: | 11/12/2021 |
| Fastening point: | | | |

## 7 Remarks; Your Cooperation Duties

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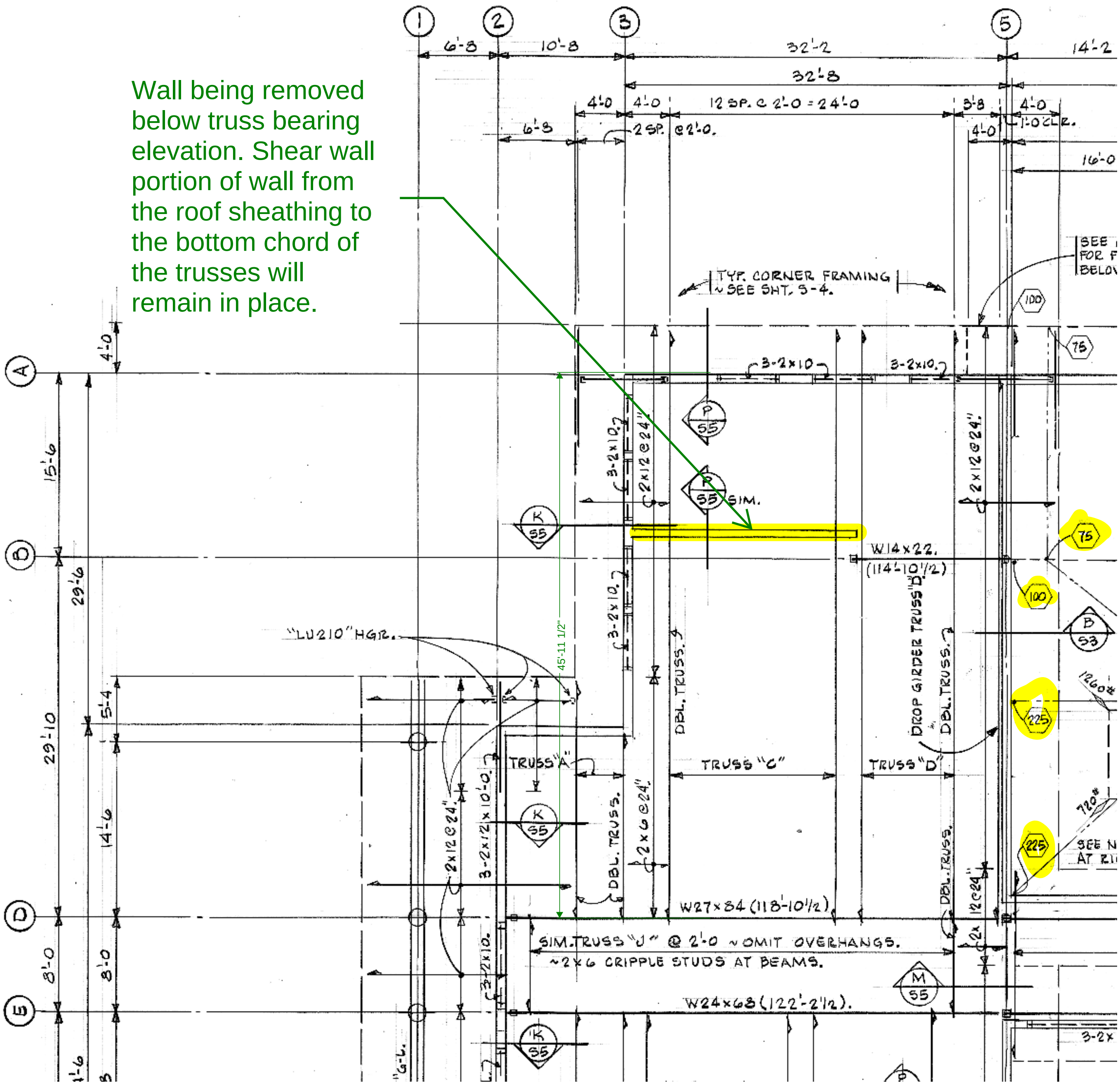
- You must take all necessary and reasonable steps to prevent or limit damage caused by the Software. In particular, you must arrange for the regular backup of programs and data and, if applicable, carry out the updates of the Software offered by Hilti on a regular basis. If you do not use the AutoUpdate function of the Software, you must ensure that you are using the current and thus up-to-date version of the Software in each case by carrying out manual updates via the Hilti Website. Hilti will not be liable for consequences, such as the recovery of lost or damaged data or programs, arising from a culpable breach of duty by you.

# 300. EXISTING WOOD SHEAR WALL/BEARING WALL REMOVAL

# Existing Wood Shear Wall/Bearing Wall Removal

-The following calculations are for the removal of the wall highlighted below. A new glulam beam spanning to wood columns will replace the gravity load path of the wall. A new shear wall to the east of the existing wall will replace the lateral load path.



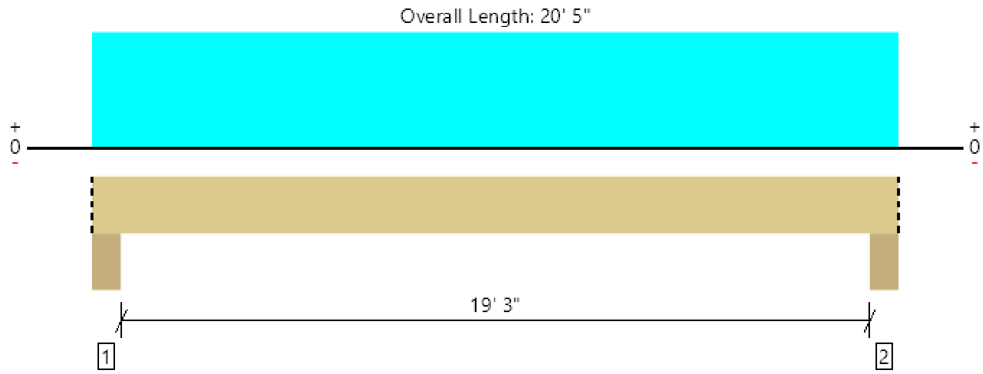
FORTE WEB

MEMBER REPORT

PASSED

# Glulam beam to support roof trusses at removed bearing wall

Level 1, Roof: Drop Beam
1 piece(s) 8 3/4" x 27" 24F-V4 DF Glulam




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) | 38357 @ 5 1/2" | 39813 (7.00") | Passed (96%) | -- | 1.0 D + 1.0 S (All Spans) |
| Shear (lbs) | 27711 @ 2' 10" | 47998 | Passed (58%) | 1.15 | 1.0 D + 1.0 S (All Spans) |
| Pos Moment (Ft-lbs) | 178595 @ 10' 2 1/2" | 215318 | Passed (83%) | 1.15 | 1.0 D + 1.0 S (All Spans) |
| Live Load Defl. (in) | 0.315 @ 10' 2 1/2" | 0.975 | Passed (L/743) | -- | 1.0 D + 1.0 S (All Spans) |
| Total Load Defl. (in) | 0.473 @ 10' 2 1/2" | 1.300 | Passed (L/495) | -- | 1.0 D + 1.0 S (All Spans) |

System : Roof
Member Type : Drop Beam
Building Use : Commercial
Building Code : IBC 2015
Design Methodology : ASD
Member Pitch : 0/12

- Deflection criteria: LL (L/240) and TL (L/180).
- Allowed moment does not reflect the adjustment for the beam stability factor.
- Critical positive moment adjusted by a volume factor of 0.88 that was calculated using length L = 19' 6".
- The effects of positive or negative camber have not been accounted for when calculating deflection.
- The specified glulam is assumed to have its strong laminations at the bottom of the beam. Install with proper side up as indicated by the manufacturer.
- Applicable calculations are based on NDS.

| Supports | Bearing Length | | | Loads to Supports (lbs) | | | | Accessories |
|---|---|---|---|---|---|---|---|---|
| | Total | Available | Required | Dead | Roof Live | Snow | Total | |
| 1 - Column - DF | 7.00" | 7.00" | 6.74" | 12836 | 204 | 25521 | 38561 | Blocking |
| 2 - Column - DF | 7.00" | 7.00" | 6.74" | 12836 | 204 | 25521 | 38561 | Blocking |

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

| Lateral Bracing | Bracing Intervals | Comments |
|---|---|---|
| Top Edge (Lu) | 20' 5" o/c | |
| Bottom Edge (Lu) | 20' 5" o/c | |

•Maximum allowable bracing intervals based on applied load.

| Vertical Loads | Location (Side) | Tributary Width | Dead (0.90) | Roof Live (non-snow: 1.25) | Snow (1.15) | Comments |
|---|---|---|---|---|---|---|
| 0 - Self Weight (PLF) | 0 to 20' 5" | N/A | 57.4 | -- | -- | |
| 1 - Uniform (PLF) | 0 to 20' 5" | N/A | 1200.0 | 20.0 | 2500.0 | Default Load |

## Weyerhaeuser Notes



| ForteWEB Software Operator | Job Notes |
|---|---|
| Jake Sujansky<br>KL&A, Inc.<br>(303) 384-9910<br>jsujansky@klaa.com | |

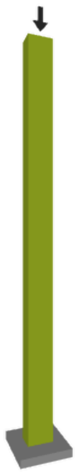
300-3

Weyerhaeuser

# Column support on both sides of Glulam beam

Level 1, Free Standing Post
1 piece(s) 7" x 7" 1.8E Parallam® PSL

Post Height: 13'



Drawing is Conceptual

| Design Results | Actual | Allowed | Result | LDF | Load: Combination |
|---|---|---|---|---|---|
| Slenderness | 22 | 50 | Passed (45%) | -- | -- |
| Compression (lbs) | 38357 | 67883 | Passed (57%) | 1.15 | 1.0 D + 1.0 S |
| Base Bearing (lbs) | 38357 | 1455300 | Passed (3%) | -- | 1.0 D + 1.0 S |
| Bending/Compression | 0.94 | 1 | Passed (94%) | 1.15 | 1.0 D + 1.0 S |

- Input axial load eccentricity for this design is 16.67% of applicable member side dimension.
- Applicable calculations are based on NDS.

| Supports | Type | Material |
|---|---|---|
| Base | Plate | Steel |

Member Type : Free Standing Post
Building Code : IBC 2015
Design Methodology : ASD

| Max Unbraced Length | Comments |
|---|---|
| Full Member Length | No bracing assumed. |

| Vertical Load | Dead (0.90) | Roof Live (non-snow: 1.25) | Snow (1.15) | Comments |
|---|---|---|---|---|
| 1 - Point (lb) | 12836 | 204 | 25521 | Linked from: Roof: Drop Beam, Support 1 |

## Weyerhaeuser Notes



| ForteWEB Software Operator | Job Notes |
|---|---|
| Jake Sujansky<br>KL&A, Inc.<br>(303) 384-9910<br>jsujansky@klaa.com | |

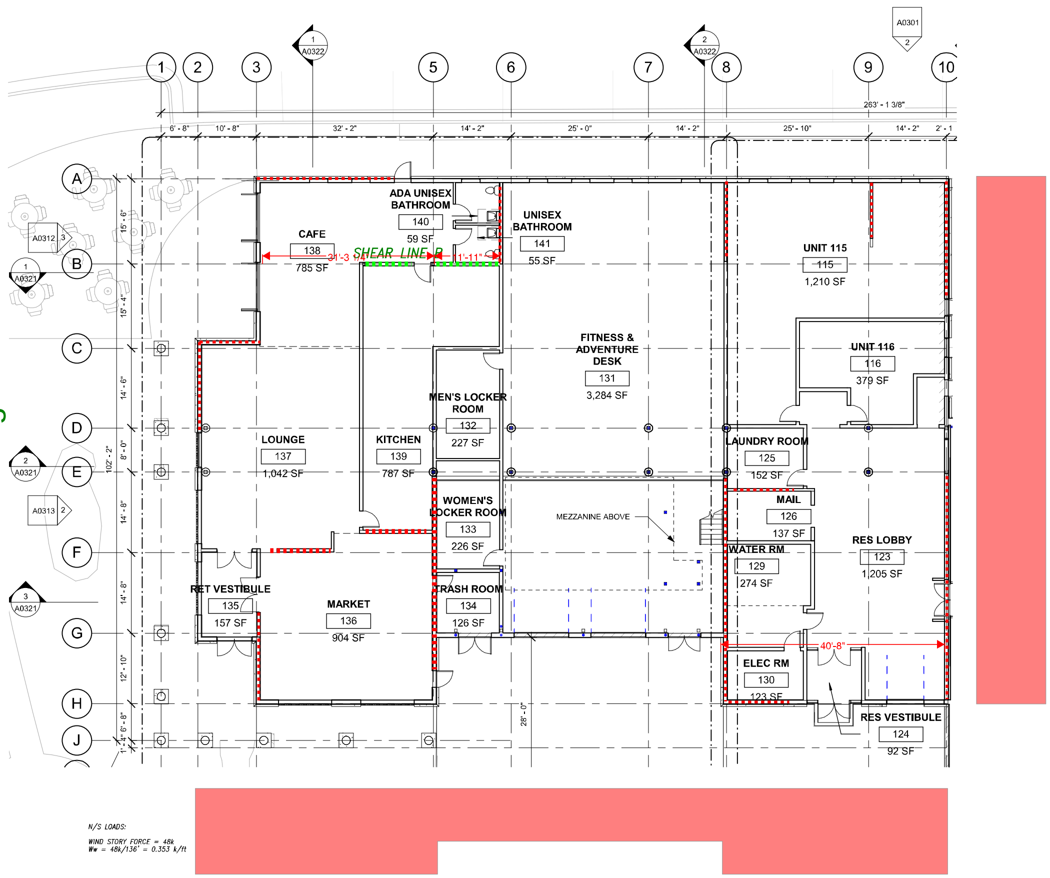
7/24/2021 12:22:00 AM

# Lateral load rundown to shear wall that is being removed

LATERAL LOADS ON (E) BUILDING BASED ON R = 6.5 (LIGHT FRAMED SHEAR WALLS). ALL LOADS ARE ALLOWABLE LEVEL.

ROOF DIAPHRAGM IS FLEXIBLE, THEREFORE DETERMINATION OF LATEARL LOAD ON CRITICAL LINES OF RESISTANCE CAN BE DETERMINED BY TRIBUTARY AREA

LINES OF RESISTANCE WHERE SHEAR WALLS ARE ALTERED AND NEED TO BE RETROFITTED TO CURRENT IBC LEVEL LOADS ARE HIGHLIGHTED IN GREEN




E/W LOADS:

WIND STORY FORCE = 21.6k
$W_w$ = 21.6k/95' = 0.228 k/ft

SEISMIC STORY FORCE = 20.3k
$W_E$ = 20.3k/95' = 0.214 k/ft

N/S LOADS:

WIND STORY FORCE = 48k
Ww = 48k/136' = 0.353 k/ft

N/S LOADS (ZONE 1):

SEISMIC STORY FORCE = 15.4k
$W_E$ = 15.4k/10,500sf*95 = 0.140 k/ft

N/S LOADS (ZONE 2):

SEISMIC STORY FORCE = 15.4k
$W_E$ = 15.4k/10,500sf*56 = 0.083 k/ft

N/S LOADS (ZONE 2):

SEISMIC STORY FORCE = 15.4k
$W_E$ = 15.4k/10,500sf*95 = 0.140 k/ft

## SHEAR LINE B REACTION

$P_W$ = 0.228k/ft * (15.5'+15.5')/2 = 3.5k

$P_E$ (WOOD WALL) = 0.214 * (15.5'+15.5')/2 = 3.4k

Shear wall Length = 14.5'
Applied shear = 3,500plf / 14.5' = 242 plf
Use type B shear wall (capacity = 600 plf)

Holdown forces
T/C = 3,500 lbs * 15' / 14.5' = 3,621 lbs

1 GROUND LEVEL PLAN
A0301/A0201 3/32" = 1'-0"

APPROVAL STAMPS:

NOT FOR CONSTRUCTION

| No. | Date | Description |
|---|---|---|
| 1 | 07.23.21 | DESIGN DEVELOPMENT |

SUBMISSIONS & REVISIONS

OWNER
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M.E.P. & F.P. ENGINEERS
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Boulder, CO 80302
T. 303.444.6038

INTERIOR DESIGNER:
**JOHNSON NATHAN STROHE**
1600 Wynkoop St., Suite 100
Denver, CO 80202
T.303.892.7062

PROJECT LOCATION
STEAMBOAT BASECAMP
1901 CURVE PLAZA
STEAMBOAT SPRINGS, CO 80487

DRAWING TITLE
GROUND LEVEL PLAN

SEAL

DATE: 07/23/21
DRAWN BY:
CHECKED BY:
PROJECT NO:

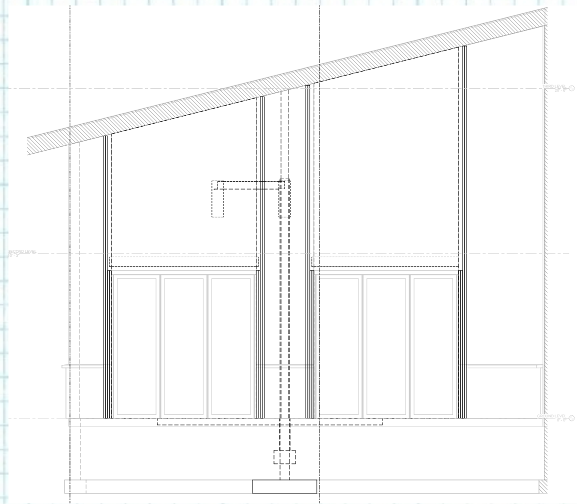
DRAWING NO:
A0201

30-5

# 400. FOLDING DOOR ROUGH OPENING SUPPORT

Design Wood Framing For Support of new
Rough Openings at New Folding Doors

Design header for out of plane wind load:
Wind Pressure = 29.2 psf (Ultimate), 17.5psf (Nominal)
Trib = 8.75ft*17.5psf = 154 plf
Span = 9ft
Use **(2)2x8 DFL-No2 flat orientation** for out of plane
-->see attached spreadsheet for capacity (374plf)



Roof Gravity Loads:
Dead = 35psf --> results in 280 plf line load to hdr
Snow = 75psf --> results in 600 plf line load to hdr

Wall Dead Load = 20psf*13ft = 260 plf

Point Load From Glulam Supporting Ceiling diaphragm beyond:
Dead = 900 lbs

Wu = 280plf+600plf+260plf = 1,140 plf
Mu = [1,140plf * (9ft)^2 / 8] + 900 lbs * 6.5ft*2.5ft/9ft
Mu = 13,168 lb-ft
Vu = [1,140plf * 9ft / 2] + 900 lbs * 6.5ft/9ft
Vu = 5,780 lbs

Use **(4)14"LVL** -->Ma = 55,796 ft-lb

Check perp grain crushing of (2)2x8 flat header for Vu from vert header
Allowable load = 625psi*3"*7" = 13,125 lbs > 5,780 lbs

Therefore **(2) trimmer studs** are acceptable for bearing.

Design King Studs For Bending
Height = 18ft
Pu from 2x8 header = 154plf*9ft/2 = 694 lbs
Pu occurs at 9.5ft from ground
Wu = 2ft trib * 17.5psf = 35 plf full height
Mu = [35plf * (18ft)^2 / 8] + 694lbs*9.5ft*8.5ft/18ft
Mu = 4,531 lb-ft
Vu = [35plf * 18ft / 2] + 694 lbs * 9.5ft/18ft
Vu = 682 lbs

Use **(3)2x8 DFL-No2 King studs**, Ma = 3*1,892 lb-ft = 5,676 lb-ft

Attach top and bottom of king studs with (2)A34 clips
Conn capacity = 2*545 lbs = 1,090 lbs

KL&A
Engineers & Builders

Project: Basecamp Phase 1
Proj #:
Engineer: JMS
Date:

# Dimensional Timber Beams

**Input:**

| | | |
|---|---|---|
| Material: (Matl-Grade) | | dfl-No2 |
| Load Duration: | | Wind |
| Repetitive? | | No |
| TL Deflection Criteria | L / | 180 |
| LL Deflection Criteria | L / | 240 |
| Flat Member Use? | | No |

**Material Options**

| | |
|---|---|
| ASP | Aspen |
| DFL | Doug Fir-Larch |
| DFLN | Doug Fir-Larch (North) |
| DFS | Doug Fir (South) |
| HF | Hem Fir |
| HFN | Hem Fir (North) |
| MO | Mixed Oak |
| RW | Redwood |
| SPF | Spruce-Pine-Fire |
| WC | Western Cedars |

**Grade Options**

| | |
|---|---|
| SS | Select Structural |
| No1 | Number 1 |
| No2 | Number 2 |
| No3 | Number 3 |

**Load Duration Options**

Normal
Snow
Wind
Seismic
Impact

**Repetitive Options**

No = Beams
Yes = Joists

**Flat Member Use Options**

No = Beams/Joists
Yes = Decking

**Dimension Lumber:**

| | | |
|---|---|---|
| Fb = | 900 | psi |
| Fv = | 180 | psi |
| E= | 1600 | ksi |
| Cd | 1.60 | |
| Cr | 1.00 | |

**Timbers:**

| | | |
|---|---|---|
| Fb = | 875 | psi |
| Fv = | 170 | psi |
| E= | 1300 | ksi |
| Cfu | 1.00 | |
| E'= | 1300 | ksi |

187 allowable load for single 2x8

| | |
|---|---|
| | Shear |
| | Moment |
| | Deflection |

**MAXIMUM UNIFORM LOAD (lb/ft)**
Minimum OF M,V OR Delta

| Nominal | | Actual | | | | | Max | Max | Length of beam (ft) | | | | | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| w | d | w | d | S | A | I | M | V | 3 | | 4 | | 5 | | 6 | | 7 | | 8 | | 9 | | 10 | | 11 | | 12 | |
| (in) | (in) | (in) | (in) | ($in^3$) | ($in^2$) | ($in^4$) | ft-lbs | lbs | TL | LL | TL | LL | TL | LL | TL | LL | TL | LL | TL | LL | TL | LL | TL | LL | TL | LL | TL | LL |
| 2 | 4 | 1 1/2 | 3 1/2 | 3.1 | 5.3 | 5 | 551 | 1,008 | 490 | | 276 | | 176 | 152 | 118 | 88 | 74 | 56 | 50 | 37 | 35 | 26 | 25 | 19 | 19 | 14 | 15 | 11 |
| 2 | 6 | 1 1/2 | 5 1/2 | 7.6 | 8.3 | 21 | 1,180 | 1,584 | 1,049 | | 590 | | 378 | | 262 | | 193 | | 147 | 144 | 117 | 101 | 94 | 74 | 74 | 56 | 57 | 43 |
| 2 | 8 | 1 1/2 | 7 1/4 | 13.1 | 10.9 | 48 | 1,892 | 2,088 | 1,682 | | 946 | | 606 | | 421 | | 309 | | 237 | | 187 | | 151 | | 125 | | 105 | 98 |
| 2 | 10 | 1 1/2 | 9 1/4 | 21.4 | 13.9 | 99 | 2,824 | 2,664 | 2,510 | | 1,412 | | 904 | | 627 | | 461 | | 353 | | 279 | | 226 | | 187 | | 157 | |
| 2 | 12 | 1 1/2 | 11 1/4 | 31.6 | 16.9 | 178 | 3,797 | 3,240 | 3,375 | | 1,898 | | 1,215 | | 844 | | 620 | | 475 | | 375 | | 304 | | 251 | | 211 | |
| 3 | 6 | 2 1/2 | 5 1/2 | 12.6 | 13.8 | 35 | 1,966 | 2,640 | 1,748 | | 983 | | 629 | | 437 | | 321 | | 246 | 241 | 194 | 169 | 157 | 123 | 123 | 93 | 95 | 71 |
| 3 | 8 | 2 1/2 | 7 1/4 | 21.9 | 18.1 | 79 | 3,154 | 3,480 | 2,803 | | 1,577 | | 1,009 | | 701 | | 515 | | 394 | | 311 | | 252 | | 209 | | 175 | 163 |
| 3 | 10 | 2 1/2 | 9 1/4 | 35.7 | 23.1 | 165 | 4,706 | 4,440 | 4,183 | | 2,353 | | 1,506 | | 1,046 | | 768 | | 588 | | 465 | | 376 | | 311 | | 261 | |
| 3 | 12 | 2 1/2 | 11 1/4 | 52.7 | 28.1 | 297 | 6,328 | 5,400 | 5,625 | | 3,164 | | 2,025 | | 1,406 | | 1,033 | | 791 | | 625 | | 506 | | 418 | | 352 | |
| 3 | 14 | 2 1/2 | 13 1/4 | 73.2 | 33.1 | 485 | 7,900 | 6,360 | | | 3,950 | | 2,528 | | 1,756 | | 1,290 | | 988 | | 780 | | 632 | | 522 | | 439 | |
| 3 | 16 | 2 1/2 | 15 1/4 | 96.9 | 38.1 | 739 | 10,465 | 7,320 | | | 5,233 | | 3,349 | | 2,326 | | 1,709 | | 1,308 | | 1,034 | | 837 | | 692 | | 581 | |
| 4 | 4 | 3 1/2 | 3 1/2 | 7.1 | 12.3 | 13 | 1,286 | 2,352 | 1,143 | | 643 | | 412 | 356 | 274 | 206 | 173 | 130 | 116 | 87 | 81 | 61 | 59 | 44 | 45 | 33 | 34 | 26 |
| 4 | 6 | 3 1/2 | 5 1/2 | 17.6 | 19.3 | 49 | 2,753 | 3,696 | 2,447 | | 1,376 | | 881 | | 612 | | 449 | | 344 | 337 | 272 | 237 | 220 | 173 | 173 | 130 | 133 | 100 |
| 4 | 8 | 3 1/2 | 7 1/4 | 30.7 | 25.4 | 111 | 4,783 | 4,872 | 4,252 | | 2,392 | | 1,531 | | 1,063 | | 781 | | 598 | | 472 | | 383 | | 316 | 297 | 266 | 229 |
| 4 | 10 | 3 1/2 | 9 1/4 | 49.9 | 32.4 | 231 | 7,187 | 6,216 | 6,389 | | 3,594 | | 2,300 | | 1,597 | | 1,173 | | 898 | | 710 | | 575 | | 475 | | 399 | |
| 4 | 12 | 3 1/2 | 11 1/4 | 73.8 | 39.4 | 415 | 9,745 | 7,560 | 8,663 | | 4,873 | | 3,119 | | 2,166 | | 1,591 | | 1,218 | | 963 | | 780 | | 644 | | 541 | |
| 4 | 14 | 3 1/2 | 13 1/4 | 102.4 | 46.4 | 678 | 12,289 | 8,904 | | | 6,145 | | 3,933 | | 2,731 | | 2,006 | | 1,536 | | 1,214 | | 983 | | 813 | | 683 | |
| 4 | 16 | 3 1/2 | 15 1/4 | 135.7 | 53.4 | 1034 | 16,279 | 10,248 | | | 8,140 | | 5,209 | | 3,618 | | 2,658 | | 2,035 | | 1,608 | | 1,302 | | 1,076 | | 904 | |

# A-100 GEOTECHNICAL REPORT

NorthWest Colorado Consultants, Inc.
Geotechnical / Environmental Engineering • Materials Testing

March 15, 2021

FV Basecamp, LLC
c/o May Riegler Properties
2201 Wisconsin Ave., Suite 200
Washington DC 20007

Attn: Gaby Riegler

Job Number: 20-11961

Subject: Supplemental Subsurface Investigation and Geotechnical Recommendations, Steamboat Basecamp, Lots 1 and 2, Worldwest Subdivision, Steamboat Springs, Colorado.

Gaby,

As requested, NWCC, Inc. (NWCC) has completed this Supplemental Subsurface Investigation and Geotechnical Recommendations report for the proposed Steamboat Basecamp to be constructed within Lots 1 and 2 of the Worldwest Subdivision in Steamboat Springs, Colorado. A Subsoil Investigation report was prepared for a proposed retail building under NWCC's Job Number 95-2241, dated June 27, 1995 within Lot 3, Block 1 of the Curve Subdivision. A Subsoil and Foundation Investigation report was prepared for a proposed building in the northeast corner of the property under NWCC's Job Number 07-7550, dated July 27, 2007. These reports were used in the preparation of this report as well as information from a Supplemental Subsurface Investigation completed at the site on November 11, 2020.

**Proposed Construction**: Based on our discussions with the client and Jake Mielke with Steamboat Engineering and Design, NWCC understands the project will consist of the renovation of the existing building and the construction of several multi-family residential and mixed use buildings, as well as several garage buildings in three phases.

The existing building will be remodeled for mixed commercial/residential use in Phase I of the project. A two-story addition will be constructed in the southeast portion of the building. Isolated interior pads will be constructed under the existing concrete slab to support the upper floors. A small addition will also be constructed in the northwest portion of the building. We have assumed that the addition will be constructed with a concrete slab-on-grade floor system placed near the existing ground surface.

Phase 2 of the project will consist of the construction of a multi-family building with 21 units and a commercial area in the northwest corner. Phase 3 of the project will consist of two multi-family units with separate garage structures within Lot 2 of the Worldwest Subdivision. We understand the buildings will consist of one to four story wood framed structures with the lower levels constructed with concrete slab on grade floor systems and/or crawlspaces. NWCC has assumed the lower levels of the buildings will be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2580 Copper Ridge Drive • Steamboat Springs, CO 80487

constructed at or above the existing ground surface. NWCC has assumed site grading will include minor unretained cuts and fills of less than 6 feet in height.

**Subsurface Conditions:** To investigate the subsurface conditions at the site, six test holes were advanced in the area of the existing building for the investigation completed in 1995. Three additional test holes were drilled within Lot 2 for the investigation completed in 2007. Four test pits were advanced within Phase 2 and 3 on November 11, 2020 with a Komatsu PC238 trackhoe. A site plan showing the approximate test hole and pit locations is presented in Figure #1.

The subsurface conditions encountered in the test holes and test pits were variable and generally consisted of variable layers of fill materials overlying natural topsoil and organic materials, natural clays and natural sands and gravels to the maximum depth investigated, 15 feet below the existing ground surface (bgs). It should be noted that practical rig refusal was encountered in the test holes on cobbles at depths ranging from 8 to 15 feet bgs. Graphic logs of the exploratory test holes and test pits are presented in Figures #2, #3 and #4. The associated Legend and Notes are presented in Figure #5.

A thin layer of topsoil and organic fill materials was encountered at the ground surface in Test Pits 1 through 4 and was approximately 3 to 6 inches in thickness. Sand and gravel fill materials were encountered below the topsoil fill materials in Test Pits 1 and 2 and were approximately 12 to 30 inches in thickness. The sand and gravel fill materials were silty to clayey, low to non-plastic, medium dense, moist and brown in color. Clay fill materials were encountered at the existing ground surface in Test Holes 3, 4, 5, 7, 8 and 9; below the topsoil fill materials in Test Pits 3 and 4; and below the sand and gravel fill materials in Test Pit 1. The clay fill materials ranged from approximately 1 to 6 feet in thickness. The clay fill materials were sandy with occasional gravels and debris, low to highly plastic, medium stiff to soft, moist and brown in color.

A layer of natural topsoil and organic materials was encountered at the ground surface in Test Holes 1, 2, and 6 and below the fill materials in Test Holes 3, 4, and 5 and Test Pits 2 and 3. The layer of natural topsoil and organic materials was approximately 3 to 18 inches in thickness.

Natural clays were encountered below the natural topsoil materials in Test Holes 1, 3, 4, 5, 6 and Test Pits 2 and 3, and below the clay fill materials in Test Holes 8 and 9, and Test Pits 1 and 4 at depths ranging from 1 ½ to 5 feet bgs. The natural clays extended to depths ranging from 2 ½ to 12 feet bgs. The natural clays were slightly sandy to sandy, moderately to highly plastic, stiff, moist and brown in color. Samples of the natural clays classified as CL to CH soils in accordance with the Unified Soil Classification System (USCS).

Natural sands and gravels were encountered below the natural topsoil and organic materials in Test Hole 2, below the fill materials in Test Hole 7 and below the natural clays in Test Holes 1, 3, 4, 5, 6, 8 and 9, and Test Pits 1, 2, 3 and 4. The natural sands and gravels were encountered at depths ranging from 6 inches to 12 feet bgs in all of the test holes and test pits. The sands and gravels extended to the maximum depth investigated in each test hole and test pit. The natural sands and gravels were silty to slightly clayey, fine to coarse grained with cobbles and small boulders, very low to non-plastic, dense, moist to wet and brown to gray in color. Samples of the natural gravels classified as SM to GM soils in accordance with the USCS.

Swell-consolidation tests conducted on samples of the natural clays from the previous investigations indicate these materials exhibited a moderate to high swell potential when wetted under a constant load.

Groundwater was encountered at depths ranging from 7 to 10 feet bgs in the Test Holes 1 through 6 at the time of drilling. These test holes were drilled on May 25, 1995, which was likely near the seasonal high groundwater table. Groundwater seepage was not encountered in any of the other test holes or pits at the time of drilling/excavation. It should be noted that the groundwater conditions at the site can be expected to fluctuate with changes in precipitation and runoff and flows in the Yampa River, located approximately 1,000 feet to the south.

During construction of the existing building all of the existing fill materials, topsoil and organic materials and natural clays were removed from under the foundations. Structural fill materials consisting of ¾-inch washed rock materials were compacted under the footings. The structural fill materials were compacted to a minimum of 80 percent of the maximum relative density of the materials.

**Foundation Recommendations**: Based on a review of the Subsoil and Foundation (NWCC, 2007) and Subsoil Investigation (NWCC, 1995) reports, the subsurface conditions encountered in the recent test pits NWCC anticipates that the natural sands and gravels will be encountered from 5 to 12 feet below the existing ground surface. Due to the highly variable depth of the existing fill materials and the swell potential of the natural clays, NWCC believes the most economically feasible building foundation systems will consist of footings placed on properly compacted structural fill materials placed over the natural sands and gravels after all of the existing fill materials and underlying topsoil and organic materials, and natural clays are removed. Due to the moderate to high swell potential of the clays, NWCC recommends these materials be removed from beneath the footings.

NWCC recommends the footings placed on the natural sands and gravels or on properly compacted, structural fill materials placed over the natural sands and gravels be designed using a maximum allowable soil bearing pressure of 3,000 psf. NWCC recommends structural fill materials placed under the footings consist of a non-expansive granular soil approved by this office. Footings placed on the natural sands and gravels or on non-expansive granular fill placed over the natural sands and gravels will not require a minimum dead load.

Structural fill materials should be uniformly placed in 6 to 8 inch loose lifts and compacted to at least 100 percent of the maximum standard Proctor density, within 2 percent of the optimum moisture content as determined by ASTM D-698. Structural fill materials should extend out from the edge of the footings on a 1(horizontal) to 1(vertical) or flatter slope.

NWCC recommends a **Site Class C** designation be used in structural design calculations in accordance with Table 20.3-1 in Chapter 20 of ASCE 7.

**Alternate Foundation Recommendations:** If the removal of all of the existing fill materials and natural clays is not economically feasible, an alternative deep foundation system for the buildings would be to place the buildings on deep foundation systems consisting of helical screw piles advanced into the natural sands and gravels. High capacity helical piles or pile groups with pile caps will most likely be required for the buildings due to anticipated loadings. The helical screw pile foundations will place the bottom of the foundations in a zone of relatively stable bearing soils and eliminate the risk of foundation movement from swell and/or consolidation of the existing fill materials and natural clays.

Utilizing this type of foundation, each column is supported on a single or group of screw piles and the structures are founded on grade beams or pier caps that are supported by a series of piles. Load applied to the piles is transmitted to the natural sands and gravels through the end bearing pressure at the helices of the screw pile. Foundation movement should be less than ½-inch if the following design and construction conditions are observed.

The helical screw pile foundation system should be designed by a qualified engineer, using industry standards and be installed by a licensed/certified installer. If pile groups are required, we recommend a minimum pile spacing of 3 times the largest helix to achieve the maximum capacity of each individual pile. Lateral loads should be resisted using battered piles or tiebacks or through passive soil pressures against foundation walls or grade beams.

We strongly recommend that at least two test piles be advanced at each building site so that the torque versus depth relationships can be established and the proper shaft and helix size and type can be determined. In addition, load testing of the helical screw piles is strongly recommended to verify the design capacity of the piles. Difficult installation should be anticipated due to the presence of cobbles and boulders in the fill materials.

A representative of this office should observe the test piles/load test and helical screw pile installations.

NWCC also recommends the following:

- Minimum 10-inch diameter helix;
- Minimum installation torque of 4,000 ft-lbs;
- Full-time installation observation by a qualified special inspector;
- Review of the Contractor's quality control plan regarding instrumentation calibration and testing, materials QC, and pile installation procedures.

An alternative deep foundation system would consist of rammed aggregate piers (RAP). The rammed aggregate piers are typically constructed to bridge poor bearing soils, such as the existing fill materials and natural clays encountered at this site, extending down to a suitable bearing layer, the underlying natural sands and gravels. A RAP foundation system should develop an end bearing pressure of at least 4,000 psf for aggregate piers founded in the sand and gravels. A RAP foundation system has the advantage of not only supporting shallow foundation elements, but also supporting floor slab areas and improving the engineering characteristics of the existing fill materials and native soils between the piers, thus decreasing the potential for floor slab movement and eliminating the need for structural slabs or structural floors over crawlspaces.

RAP foundation elements are designed as proprietary foundation systems. If a RAP foundation system is selected, NWCC should be contacted to coordinate with the RAP contractor/design team during foundation design.

**Floor Slabs:** NWCC has assumed the lower levels of the buildings will most likely be constructed with concrete slab-on-grade floor systems placed near the existing grades. The natural soils, excluding the existing fill materials and topsoil and organic materials, are capable of supporting slab-on-grade construction. However, floor slabs present a very difficult problem where swelling materials are present near floor slab elevation because sufficient dead load cannot be imposed on them to resist the uplift pressure generated when the materials are wetted and expand. Based on the moisture-volume change characteristics of the natural clays encountered at this site, NWCC believes slab-on-grade construction may be used, provided the risk of distress resulting from slab movement is recognized and special design precautions are followed.

The following measures must be taken to reduce damage, which could result from movement should the underslab clays are subjected to moisture changes.

1) Floor slabs must be separated from all bearing walls; columns and their foundation support with a positive slip joint. NWCC recommends the use of ½-inch thick cellotex or impregnated felt.

2) Interior non-bearing partition walls resting on the floor slabs must be provided with a slip joint, preferably at the bottom, so in the event, the floor slab moves this movement is not transmitted to the upper structure. This detail is also important for wallboard and doorframes and is shown in Figure #6. This detail can be omitted if all of the clays are removed from beneath the floor slabs.

3) A minimum 6-inch gravel layer must be provided beneath all floor slabs to act as a capillary break and to help distribute pressures. Prior to placing the gravel, excavation should be shaped so that if water does get under the slab, it will flow to the low point of the excavation. In addition, all existing fill materials and topsoil and organic materials should be removed prior to placement of the underslab gravels or new granular fill materials. If the removal of all of the existing fill materials and topsoil and organics and replacing with granular fill materials is not economically feasible, we recommend the lower levels be constructed on structural floor systems over a crawlspace.

4) Floor slabs must be provided with control joints placed a maximum of 10 to 12 feet on center in each direction, depending on slab configurations, to help control shrinkage cracking. Locations of the joints should be carefully checked to assure that natural, unavoidable cracking will be controlled. Depth of the control joints should be a minimum of ¼ the thickness of the slab.

5) Underslab soils must be kept as close as possible to their in-situ moisture content. Excessive wetting or drying of these soils prior to placement of floor slab could result in differential movement after slabs are constructed.

6) It has been NWCC's experience that the risk of floor slab movement can be reduced by removing at least 2 feet of the expansive soils and replacing them with a well compacted, non-expansive fill. If this is done or if fills are required to bring underslab areas to the desired grade, the fill should consist of non-expansive, granular materials. Fill should be uniformly placed and compacted in 6 to 8-inch lifts to at least 95% of the maximum standard Proctor density at or near the optimum moisture content, as determined by ASTM D-698.

Following the above precautions and recommendations will not prevent floor slab movement in the event the clays beneath the floor slabs undergo moisture changes. However, they should reduce the amount of damage if such movement occurs. The only way to eliminate the risk of all floor slab movement is to construct a structural floor over a well-vented crawl space or void form materials or remove all of the expansive clays and replace them with non-expansive granular fill materials.

**Foundation Walls and Retaining Structures:** Foundation walls and retaining structures, which are laterally supported and can be expected to undergo only a moderate amount of deflection, may be designed for a lateral earth pressure computed on the basis of an equivalent fluid unit weight of 45 pcf for imported, free draining granular backfill and 55 pcf for on-site soils.

Cantilevered retaining structures at the site can be expected to deflect sufficiently to mobilize full active earth pressure condition. Therefore, cantilevered structures may be designed for a lateral earth pressure computed on the basis of an equivalent fluid unit weight of 35 pcf for imported, free draining granular backfill and 45 pcf for on-site soils.

Foundation walls and retaining structures should be designed for appropriate hydrostatic and surcharge pressures such as adjacent buildings, traffic and construction materials. An upward sloping backfill and/or natural slope will also significantly increase earth pressures on foundation walls and retaining structures and the structural engineer should carefully evaluate these additional lateral loads when designing foundation and retaining walls.

Lateral resistance of retaining wall foundations placed on undisturbed natural soils at the site will be a combination of sliding resistance of the footings on the foundation materials and passive pressure against the sides of footings. Sliding friction can be taken as 0.4 times the vertical dead load. Passive pressure against the sides of the footing can be calculated using an equivalent fluid pressure of 250 pcf. Fill placed against the sides of footings to resist lateral loads should be compacted to at least 100% of the maximum standard Proctor density and near the optimum moisture content.

NWCC recommends imported granular soils for backfilling foundation walls and retaining structures because their use results in lower lateral earth pressures. Imported granular materials should be placed to within 2 to 3 feet of the ground surface. Imported granular soils should be free draining and have less than 7 percent passing the No. 200 sieve. Granular soils placed behind foundation and retaining walls should be sloped from the base of the wall at an angle of at least 45 degrees from the vertical. The upper 2 to 3 feet of fill should be a relatively impervious soil or pavement structure to prevent surface water infiltration into the backfill.

Wall backfill should be carefully placed in uniform lifts and compacted to at least 95 percent of the maximum standard Proctor density and near the optimum moisture content. Care should be taken not to overcompact backfill since this could cause excessive lateral pressure on the walls. Some settlement of deep foundation wall backfill materials will occur even if materials are placed correctly.

**Underdrain System:** Any floor levels constructed below the existing or finished ground surfaces and the foundations should be protected by underdrain systems to help reduce the problems associated with surface and subsurface drainage during high runoff periods. If any level is placed within 2 feet of the seasonal high groundwater table, a permanent/full-time dewatering system may be required in the lower level. NWCC must be consulted to provide or review the design of a dewatering system.

Localized perched water or runoff can infiltrate the lower levels of the structures at the foundation levels. This water can be one of the primary causes of differential foundation and slab movement. Especially, when expansive soils are encountered. Excessive moisture in crawl space areas or lower level can also lead to rotting and mildewing of wooden structural members and the formation of mold and mold spores. Formation of mold and mold spores could have detrimental effects on the air quality in these areas, which in turn can lead to potential adverse health effects.

Drains should be located around the entire perimeter of the lower levels and be placed and at least 12 inches below any floor slab or crawl space levels and at least 6 inches below the bottom of the foundation walls or footings. NWCC recommends the use of perforated PVC pipe for the drainpipe, which meets or exceeds ASTM D-3034/SDR 35 requirements, to minimize the potential for pipe crushing during backfill operations. Holes in the drainpipe should be oriented down between 4 o'clock and 8 o'clock to promote rapid runoff of water. Drainpipe should be surrounded with at least 12 inches of free-draining gravel and should be protected from contamination by a filter covering of Mirafi 140N subsurface drainage fabric or an equivalent product. Drains should have a minimum slope of 1/8 inch per foot and be daylighted at positive outfalls protected from freezing or be led to sumps from which water can be pumped. The use of interior laterals, multiple daylights, or sumps will likely be required for the proposed structure. Caution should be taken when backfilling so as not to damage or disturb the installed underdrain. NWCC recommends the drainage system include a cleanout every 100 feet, be protected against intrusion by animals at outfalls, and be tested prior to backfilling. NWCC also recommends the client retain our firm to observe the underdrain systems during construction to verify that they are being installed in accordance with recommendations provided in this report and observe a flow test prior to backfilling the system.

In addition, NWCC recommends an impervious barrier be constructed to keep water from infiltrating under the footings and/or foundation walls. The barrier should be constructed of an impervious material, which is approved by this office and placed below the perimeter drain and up against the sides of the foundation walls. A typical perimeter/underdrain detail is shown in Figure #7.

**Surface Drainage:** Proper surface drainage at this site is of paramount importance for minimizing infiltration of surface drainage into wall backfill and bearing soils, which could result in increased wall pressures, differential foundation, and slab movement. The following drainage precautions should be observed during construction and at all times after the structures have been completed:

1) Ground surface surrounding structures should be sloped (minimum of 1.0 inch per foot) to drain away from structures in all directions to a minimum of 10 feet. Ponding must be avoided. If necessary, raising the top of foundation walls to achieve a better surface grade is advisable.

2) Non-structural backfill placed around structures should be compacted to at least 95% of the maximum standard Proctor density at or near the optimum moisture content to minimize future settlement of the fill. Backfill should be placed immediately after the braced foundation walls are able to structurally support the fill. Puddling or sluicing must be avoided.

3) Top 2 to 3 feet of soil placed within 10 feet of foundations should be impervious in nature to minimize infiltration of surface water into wall backfill.

4) Roof downspouts and drains should discharge well beyond the limits of all backfill. Roof overhangs, which project two to three feet beyond foundation walls, should be considered if gutters are not used.

5) Landscaping, which requires excessive watering and lawn sprinkler heads, should be located a minimum of 10 feet from the foundation walls of the structures or any permanent, unretained cuts. Additionally, large piles of man-made or natural snow should be removed prior to melting within 10 feet of the foundation walls of the structures or any permanent, unretained cuts.

6) Plastic membranes should not be used to cover the ground surface adjacent to foundation walls.

**Site Grading:** Temporary cuts for foundation construction should be constructed to OSHA standards for temporary excavations. Permanent, unretained cuts should be kept as shallow as possible and should not exceed a 3(Horizontal) to 1(Vertical) configuration for the existing fill materials and natural clays.

We recommend permanent, unretained cuts be limited to 10 feet in height or less. The risk of slope instability will be significantly increased if groundwater seepage is encountered in the cuts. NWCC office should be notified immediately to evaluate the site if seepage is encountered or deeper cuts are planned and determine if additional investigations and/or stabilization measures are warranted.

Excavating during periods of low runoff at the site can reduce potential slope instability during excavation. Excavations should not be attempted during the spring or early summer when seasonal runoff and groundwater levels are typically high.

Fills up to 10 feet in height can be constructed at the site and should be constructed to a 2(Horizontal) to 1(Vertical) or flatter configuration. The fill areas should be prepared by stripping any existing fill materials and topsoil and organics, scarification, and compaction to at least 95% of the maximum standard Proctor density and within 2% of optimum moisture content as determined by ASTM D698. The fills should be properly benched/keyed into the natural hillsides after the existing fill materials, natural topsoil, and organic materials, silts, and clays have been removed. The fill materials should consist of the on-site soils (exclusive of topsoil, organics, or silts) and be uniformly placed and compacted in 6 to 8-inch loose lifts to the minimum density value and moisture content range indicated above.

Proper surface drainage features should be provided around all permanent cuts and fills and steep natural slopes to direct surface runoff away from these areas. Cuts, fills, and other stripped areas should be protected against erosion by revegetation or other methods. Areas of concentrated drainage should be avoided and may require the use of riprap for erosion control. NWCC recommends that a maximum of 4 inches of topsoil be placed over the new cut and fill slopes. It should be noted that the newly placed topsoil materials may slough/slide off the slopes during the spring runoff seasons until the root zone in the vegetated cover establishes.

A qualified engineer experienced in this area should prepare site grading and drainage plans. The contractor must provide a construction sequencing plan for excavation, wall construction, and bracing and backfilling for the steeper and more sensitive portions of the site prior to starting the excavations or construction.

**Pavement Section Recommendations:** Pavement section alternatives presented below are based on anticipated soil conditions, assumed traffic loadings indicated below, pavement design procedures presented in the AASHTO Guide for Design of Pavement Structures, and our experience with similar sites and conditions in this part of Steamboat Springs. AASHTO pavement design procedures have been adopted and are used by the Colorado Department of Transportation (CDOT). NWCC has assumed the proposed pavement areas will be subjected to automobiles with occasional delivery trucks and with regular trash truck service.

Based on the results of the field and laboratory investigations and our understanding of the proposed construction, it appears the materials to be encountered at proposed pavement subgrade elevations will most likely consist of existing fill materials or natural clays. We have assumed the fill materials will generally classify as CL soils in accordance with the USCS, which is the worst-case scenario. NWCC recommends the pavement areas subjected to both truck and automobile traffic, such as at the entrances and roadways through the facility be constructed with a minimum of 4 inches of hot mix asphalt (HMA) overlying a minimum of 4 inches of CDOT class 6 aggregate base course (ABC) and a minimum of 8 inches of subbase aggregates (Pit Run). The pavement areas subjected to automobiles only, such as the parking stalls, can be paved with a minimum of 3 inches of HMS, 4 inches of CDOT class 6 aggregate base course (ABC), and a minimum of 6 inches of subbase aggregates (subbase).

NWCC recommends the areas subjected to heavy truck turning movements, such as the pads in front of the trash dumpsters or loading docks be paved with a rigid pavement section consisting of at least 8 inches of Portland cement concrete (PCC).

NWCC recommends the asphalt pavement material (HMA) consist of an approved "Superpave" mix designed by a qualified, registered engineer. The mix design should be designed using the SX gradation and mixed with PG 58-28 oil or other performance graded asphaltic materials. The mix should be produced and placed by a qualified contractor and should be compacted to between 92 and 96 percent of the maximum theoretical (Rice) density or at least 92 percent of the maximum Rice density. Quality control activities should be conducted on paving materials at the time of placement.

Base course materials (ABC) should consist of a well-graded aggregate base course material that meets CDOT Class 6 ABC grading and durability requirements and the subbase should consist of well-graded aggregate materials that meet CDOT Class 2 ABC grading and durability requirements.

ABC and subbase materials should be uniformly placed and compacted in 4 to 6-inch loose lifts to at least 95 % of the maximum modified Proctor density and within +/- 2 % of the optimum moisture content as determined by ASTM D1557.

Concrete pavement materials shall be based on a mix design established by a qualified engineer. Concrete should have a minimum 28-day compressive strength of 4,500 psi, be air-entrained with approximately 6 percent air, and have a maximum water/cement ratio of 0.42. Concrete should have a maximum slump of 4 inches and should contain control joints no greater than 10 to 12 feet on center, depending on slab configurations. The depth of the control joints should be at least ¼ of the slab thickness.

Prior to placement of subbase materials, NWCC recommends that all of the existing fill materials be removed, any debris removed and the materials moisture conditioned and compacted. Prior to placement of the subgrade fill materials the natural clays should be scarified and recompacted to a depth of 8 inches. The scarified natural clays and subgrade materials should be compacted in 6 to 8 inch lifts to at least 95 % of the maximum standard Proctor density and within +/- 2 % of the optimum moisture content as determined by ASTM D698. The finished subgrade surface, after recompaction, should also be sloped at least 1 percent to avoid ponding and to reduce the potential for wetting and expansion of the subgrade soils. The finished subgrade surface should be proof rolled with a loaded tandem dump truck or loaded water truck and any areas deflecting or rutting should be removed and or stabilized prior to placing the subbase aggregates.

The collection and diversion of surface and subsurface drainage away from the paved areas is extremely important to the satisfactory performance of the pavement. The design of the surface and subsurface drainage features should be carefully considered to remove all water from paved areas and to prevent ponding of water on and adjacent to paved areas.

**Limitations:** The recommendations provided in this report are based on the subsurface conditions encountered at this site and our understanding of the proposed construction. We believe that this information gives a high degree of reliability for anticipating the behavior of the proposed structures; however, our recommendations are professional opinions and cannot control nature, nor can they assure the soils profiles beneath those or adjacent to those observed. No warranties expressed or implied are given on the content of this report.

Expansive soils were encountered at this site. These soils are not prone to volume changes at their natural moisture content but can consolidate or swell with changes in moisture and loading. The behavior of expansive soils is not fully understood. The swell and/or consolidation potential of any particular site can change erratically both in lateral and vertical extent. Moisture changes also occur erratically, resulting in conditions, which cannot always be predicted. The recommendations presented in this report are based on the current state of the art for foundations and floor slabs on swelling/consolidating soils. The owner should be aware that there is a risk in construction on these types of materials. Performance of the structures will depend on following the recommendations and in proper maintenance after construction is complete. As water is the main cause for volume change in the soils, it is necessary that the changes in moisture content be kept to a minimum; therefore, positive surface drainage should be maintained away from the structures. Any distress noted in the structures should be brought to the attention of this office.

This report is based on the investigation at the described site and on the specific anticipated construction as stated herein. If either of these conditions is changed, the results would also most likely change. Therefore, NWCC strongly recommends that our firm be contacted prior to finalizing the construction plans so that we can verify that our recommendations are being properly incorporated into the construction plans. Man-made or natural changes in the conditions of a property can also occur over a period of time. In addition, changes in requirements due to state of the art knowledge and/or legislation do from time to time occur. As a result, the findings of this report may become invalid due to these changes. Therefore, this report is subject to review and not considered valid after a period of 3 years or if conditions as stated above are altered.

It is the responsibility of the owner or his representative to ensure information in this report is incorporated into the plans and/or specifications and construction of the project. It is advisable that a contractor familiar with construction details typically used to dealing with the local subsoils and climatic conditions be retained to build the structures.

If you have any questions regarding this report or if we may be of further service, please do not hesitate to contact us.

Sincerely,
**NWCC, INC.,**

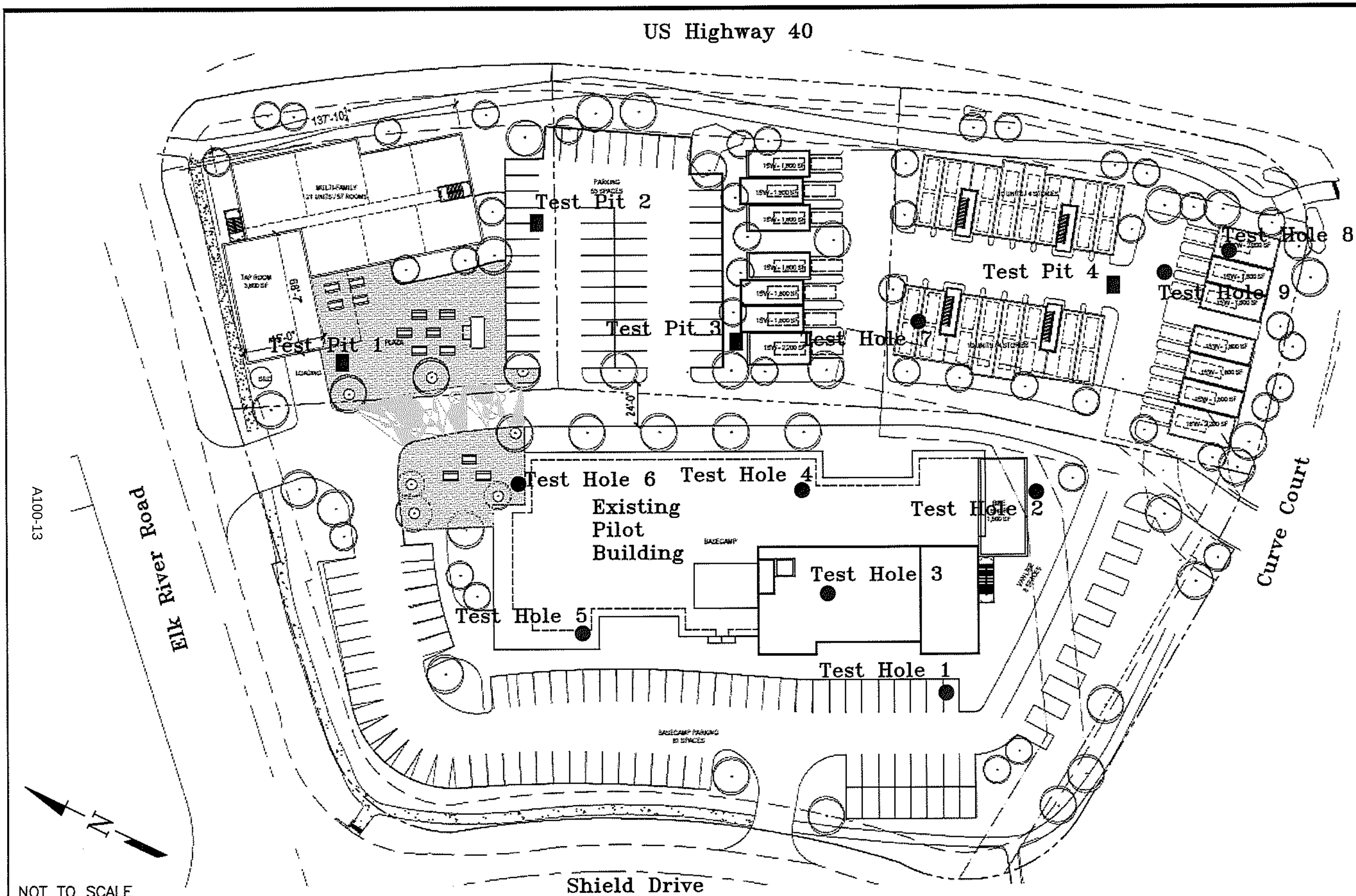
Timothy S. Travis, P.E.
Senior Project Engineer

COLORADO LICENSED BRIAN D. LEN 25750 PROFESSIONAL ENGINEER

Reviewed by Brian D. Len, P.E.
Principal Engineer

3/16/21

cc: Jake Mielke, SEAD

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|---|---|---|---|
| **Title:** SITE PLAN–LOCATION OF TEST HOLES AND PITS | **Date:** 2/2/2021 | | |
| **Job Name:** Steamboat Basecamp | **Job No.** 20–11961 | | |
| **LOCATION:** Worldwest Subdivision, Steamboat Springs, Colorado | **Figure** #1 | | |


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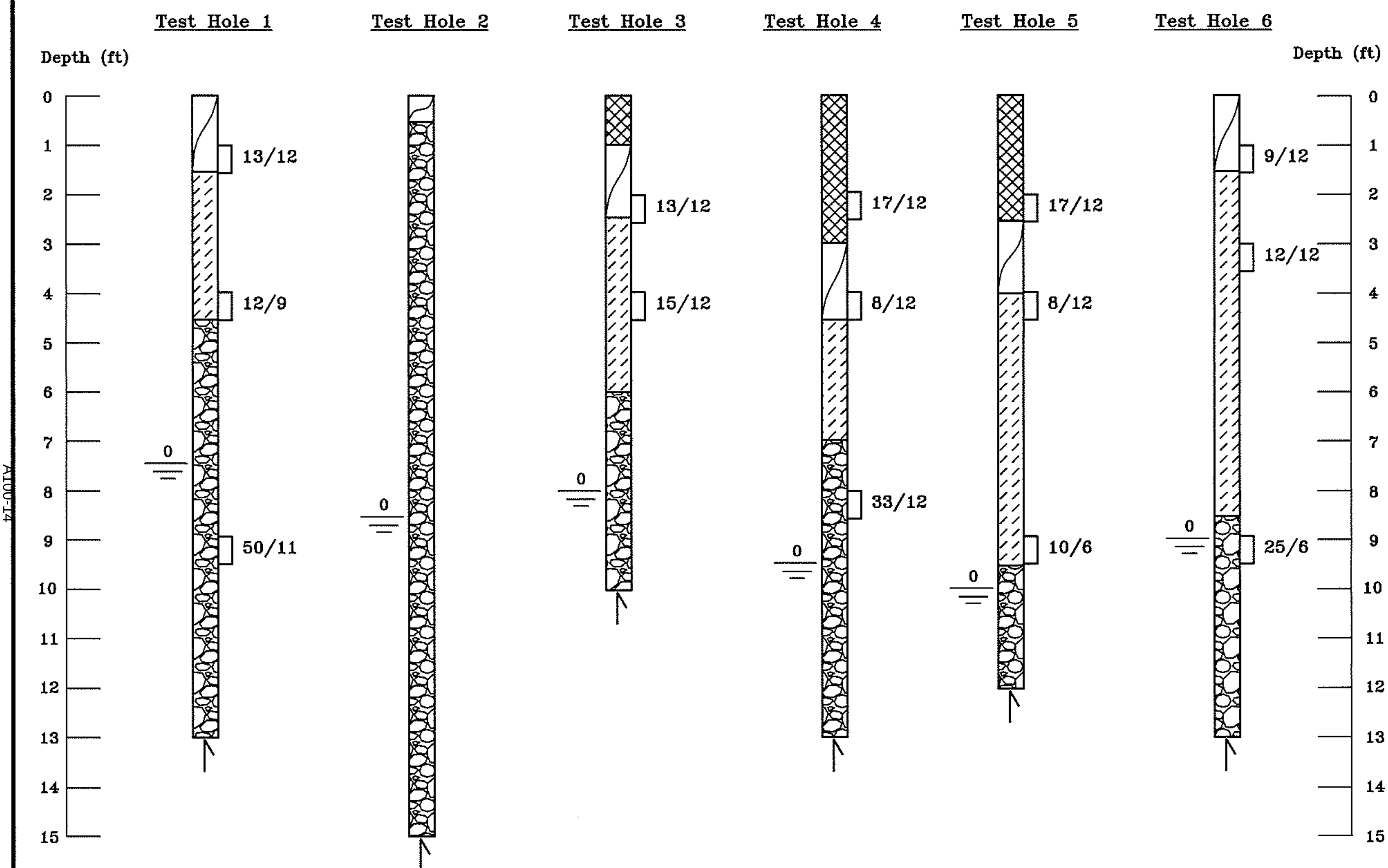


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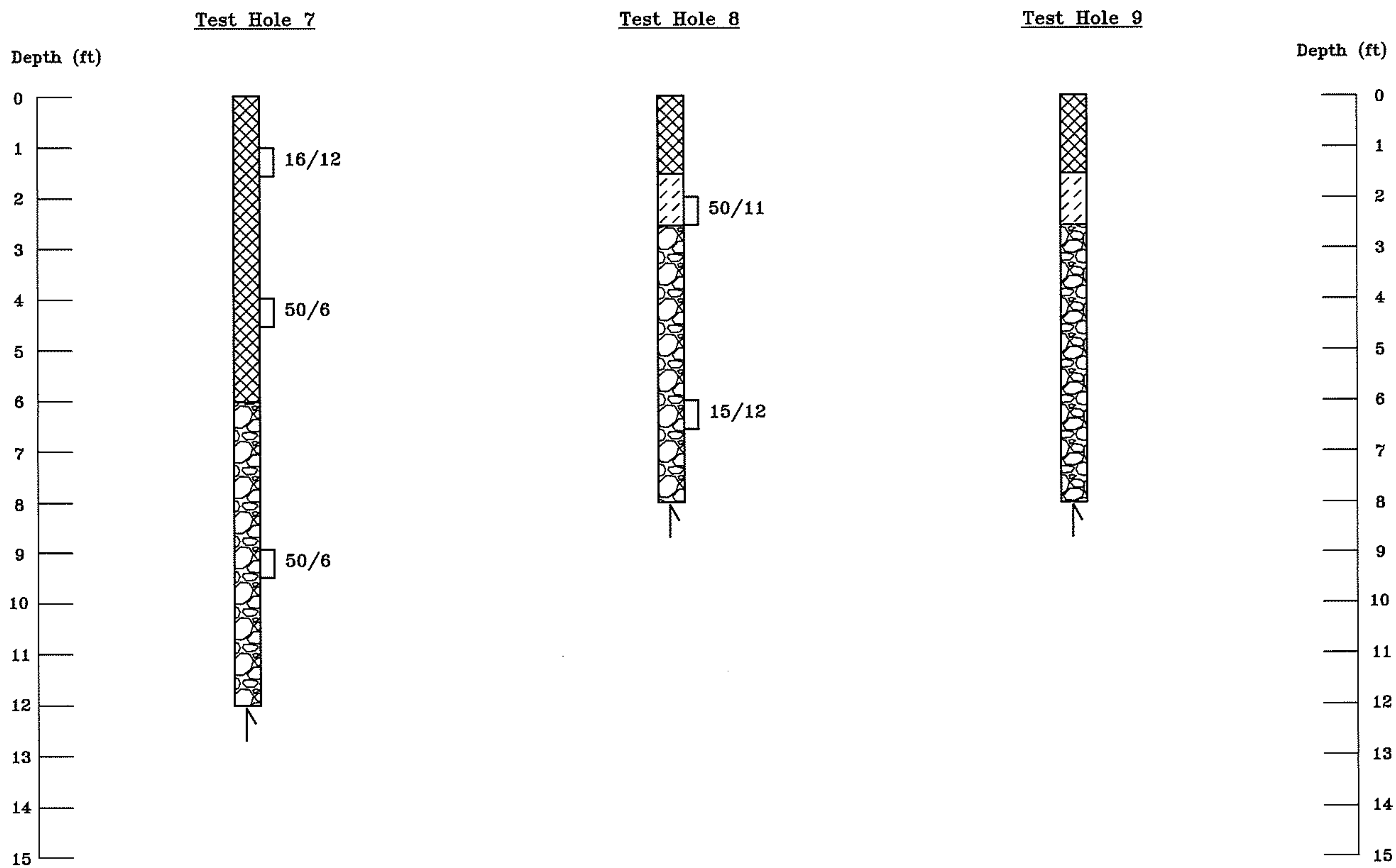
|---|---|---|---|
| Title: | LOGS OF EXPLORATORY TEST HOLES | Date: 3/5/2021 | NWCC North West Colorado Consultants Inc. Geotechnical / Environmental Engineering · Materials Testing (970)879-7888 · Fax (970)879-7891 2580 Copper Ridge Drive Steamboat Springs, Colorado 80487 |
| Job Name: | Steamboat Basecamp | Job No. 20-11961 | |
| LOCATION: | Worldwest Subdivision, Steamboat Springs, Colorado | Figure #2 | |

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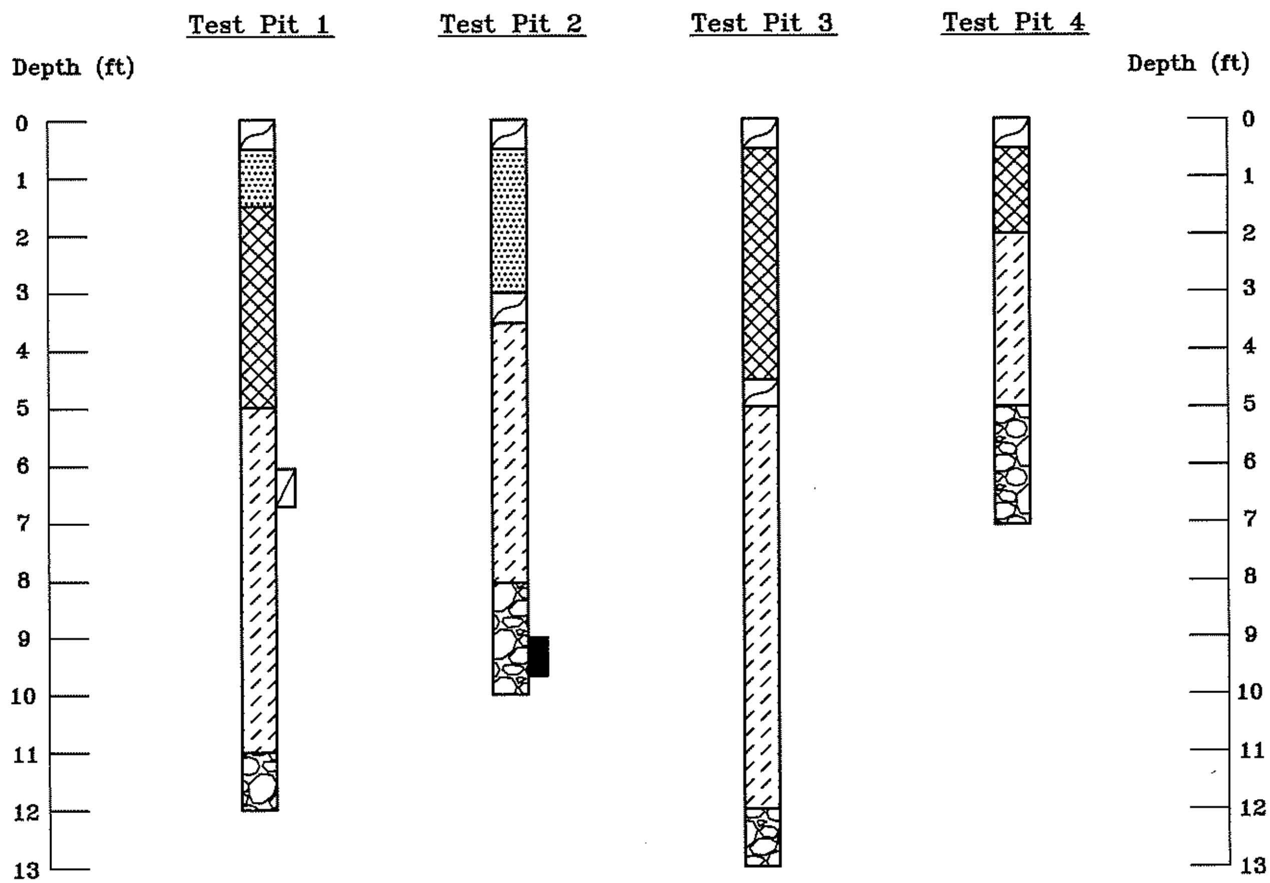
| Title: | LOGS OF EXPLORATORY TEST HOLES | Date: 3/5/2021 | NWCC North West Colorado Consultants, Inc. Geotechnical / Environmental Engineering · Materials Testing (970)879-7888 · Fax (970)879-7891 2580 Copper Ridge Drive Steamboat Springs, Colorado 80487 |
| Job Name: | Steamboat Basecamp | Job No. 20-11961 | |
| LOCATION: | Worldwest Subdivision, Steamboat Springs, Colorado | Figure #3 | |

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|---|---|---|---|
| Title: | LOGS OF EXPLORATORY TEST PITS | Date: | 3/5/2021 |
| Job Name: | Steamboat Basecamp | Job No. | 20-11961 |
| LOCATION: | Worldwest Subdivision, Steamboat Springs, Colorado | Figure | #4 |


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## LEGEND:

Topsoil and organics.

SAND AND GRAVEL FILL: Silty to clayey, low to non–plastic, medium dense, moist and brown in color.

CLAY FILL: Sandy with occasional gravels and debris, low to highly plastic, medium stiff to soft, moist and brown in color.

CLAY: Slightly sandy to sandy, moderately to highly plastic, stiff, moist and brown in color.

SANDS AND GRAVELS: Silty to slightly clayey, fine to coarse grained with cobbles and small boulders, very low to non–plastic, dense, moist to wet and brown to gray in color.

Drive Sample, 2–inch I.D. California Liner Sampler.

Hand Drive Sample–California Liner.

Small Disturbed Sample.

Indicates depth of practical rig refusal on cobbles.

13/12 Drive Sample Blow Count, indicates 13 blows of a 140–pound hammer falling 30 inches were required to drive the sampler 12 inches.

Indicates depth at which groundwater was encountered at the time of drilling.

## NOTES:

1) Test Holes 1 through 6 were drilled on May 25, 1995 and Test Holes 7 through 9 were drilled on May 10, 2007 with a truck–mounted drill rig using 4–inch diameter continuous flight power augers. Test Pits 1 through 4 were excavated on November 11, 2020 with a Cat trackhoe.

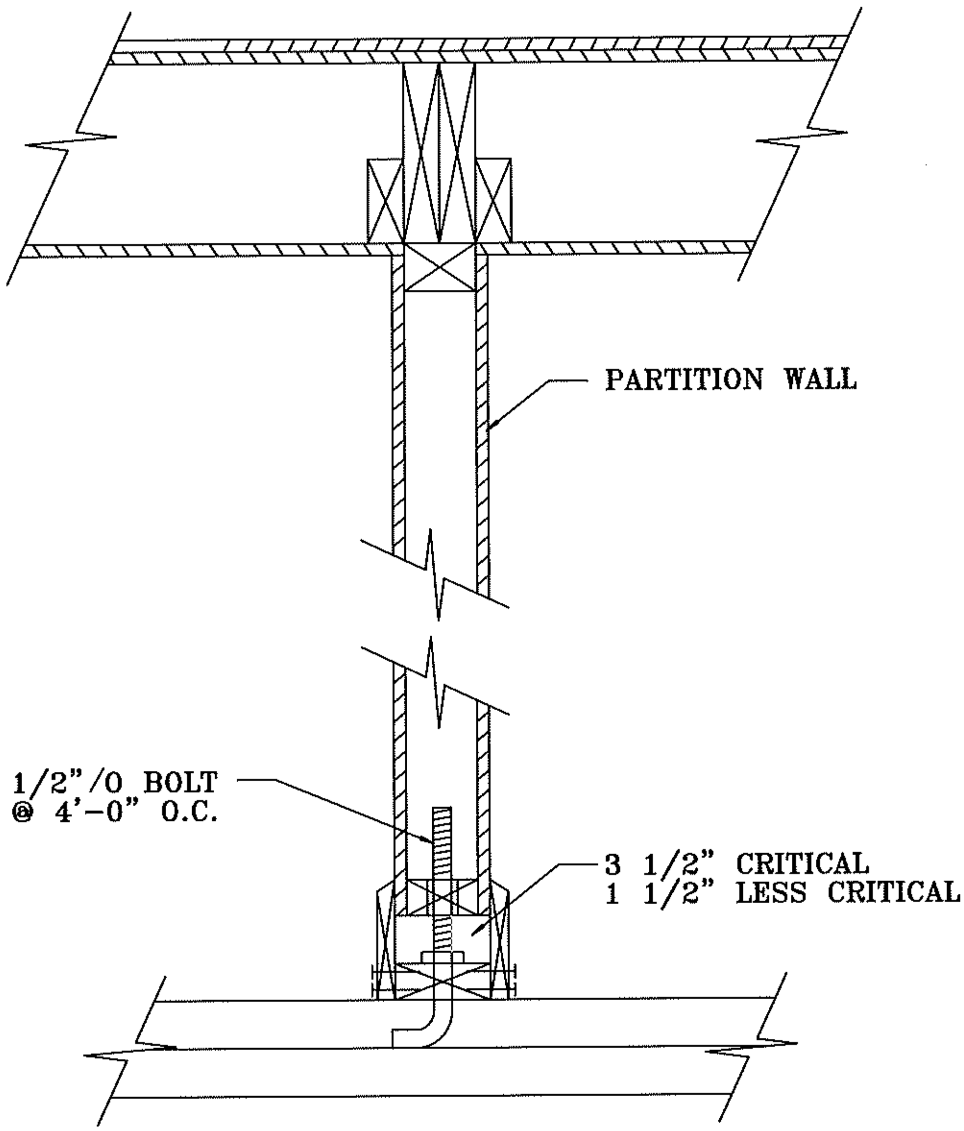
2) Locations of the test holes and test pits were determined in the field by pacing from the existing structure.

3) Elevations of the test holes were not measured and logs are drawn to the depths investigated.

4) The lines between materials shown on the logs represent the approximate boundaries between material types and transitions may be gradual.

5) The water level readings shown on the logs were made at the time and under the conditions indicated. Fluctuations in the water levels will probably occur with time.

| | | |
|---|---|---|
| Title: LEGEND AND NOTES | Date: 3/5/2021 | North West Colorado Consultants, Inc. Geotechnical / Environmental Engineering · Materials Testing |
| Job Name: Steamboat Basecamp | Job No. 20–11961 | (970)879-7888 · Fax (970)879-7891 |
| Location: Worldwest Subdivision, Steamboat Springs, Colorado | Figure #5 | 2580 Copper Ridge Drive Steamboat Springs, Colorado 80487 |

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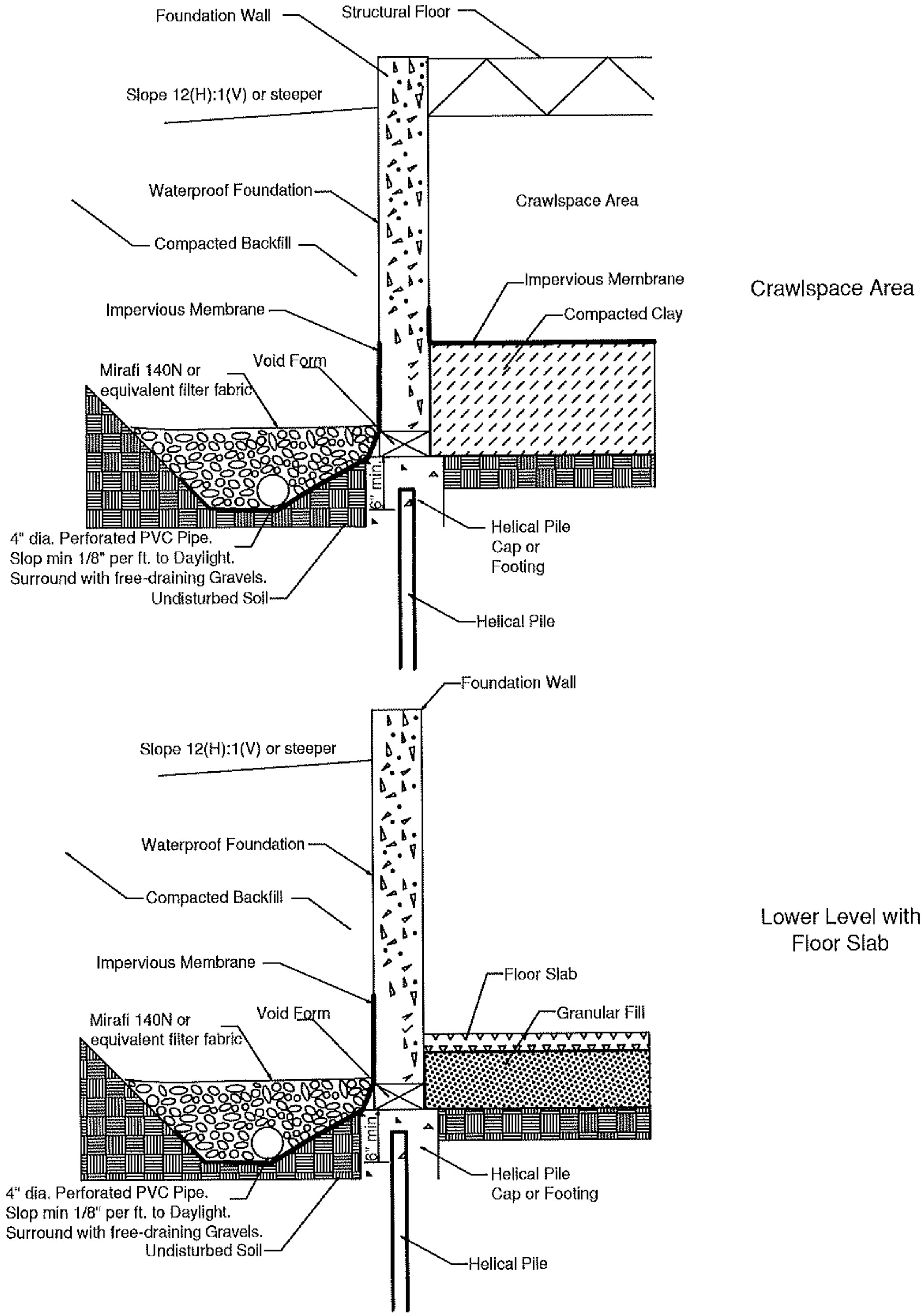
|---|---|---|
| Title: HUNG PARTITION WALL DETAIL | Date: 3/5/2021 | NWCC North West Colorado Consultants, Inc. Geotechnical / Environmental Engineering · Materials Testing (970)879-7888 · Fax (970)879-7891 2580 Copper Ridge Drive Steamboat Springs, Colorado 80487 |
| Job Name: Steamboat Basecamp | Job No. 20-11961 | |
| Location: Worldwest Subdivision, Steamboat Springs, Colorado | Figure #6 | |

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|---|---|
| Title: PERIMETER/UNDERDRAIN DETAIL | Date: 3/5/2021 |
| Job Name: Steamboat Basecamp | Job No. 20-11961 |
| Location: Worldwest Subdivision, Steamboat Springs, Colorado | Figure #7 |

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