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ANDRZEJ ROBERT TATKOWSKI
STATE OF WASHINGTON
23021482
STRUCTURAL ENGINEER
PROFESSIONAL ENGINEER
2/27/2025

STRUCTURAL CALCULATIONS

To
City of Puyallup

Project

Puyallup Public Safety Building

Submitted
March 4, 2025

Project Number
2170269.07

MACKENZIE
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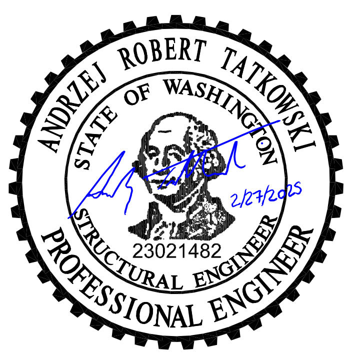
STRUCTURAL SYSTEMS NARRATIVE

The following calculations are for the Puyallup Public Safety Building Tenant Improvement in Puyallup, WA. The existing building is being upgraded structurally to a Risk Category IV building by another engineer and permit. Structural scope for the TI includes two exterior steel-framed freestanding canopies, non-bearing interior CMU partitions, an operable partition, and other free standing landscape elements.



ROOF LOADING

Reference Standards: IBC 2021, ASCE 7-16

Input Result Check/Confirm

LIVE LOAD:

$L_{rf} = 20$ psf — Roof Live Load

Refer to Snow Loading for Snow Load and Drift

ROOF DEAD LOAD:

R = 23 psf — Laminated Glass and Adjustable Stand-Off Connectors (per Manufacturer)

$P = \frac{20 \text{ plf}}{3 \text{ ft}} = 6.67$ psf — Purlins - HSS4X4X3/8

$G = \frac{57 \text{ plf}}{15 \text{ ft}} = 3.8$ psf — Girders - Tapered WT20-WT7

Misc = 0.33 psf — Miscellaneous

Total Roof Dead Load to Girders:

$DL_{rf} = R + P + Misc = 30.00$ psf

Total Roof Dead Load to Columns:

$DL_{rf,COL} = R + P + G + Misc = 33.80$ psf

ROOF SEISMIC DEAD LOAD:

$h_{rf} = 12\ ft$	Roof Height
$A_{rf} = 36\ ft \cdot 18\ ft = 648\ ft^2$	Roof Area
$RF = DL_{rf,COL} \cdot A_{rf} = 21.90\ kip$	Roof Load by Area
$ST = 82.2\ pcf \cdot 2.5\ in \cdot 24\ in \cdot 4 \cdot h_{rf} \cdot 3 = 4.93\ kip$	Stone Veneer - 3in Thick x 2ft Wide x 4 sides
$COL = 106\ plf \cdot 9\ ft \cdot 3 = 2.86\ kip$	Columns

Seismic Dead Load:

$DL_{rf,seis} = RF = 21.90\ kip$

$DL_{col,seis} = ST = 4.93\ kip$

Total Seismic Dead Load:

$DL_{seis} = RF + ST + COL = 29.69\ kip$

SNOW DRIFT GEOMETRY AND LOADING

Purpose Statement: Snow drift calculation for lower roofs, adjacent structures, roof top projections and parapets. Calculates snow drift and balanced snow load only. Ground snow load and minimum roof snow load to be determined outside of this program.

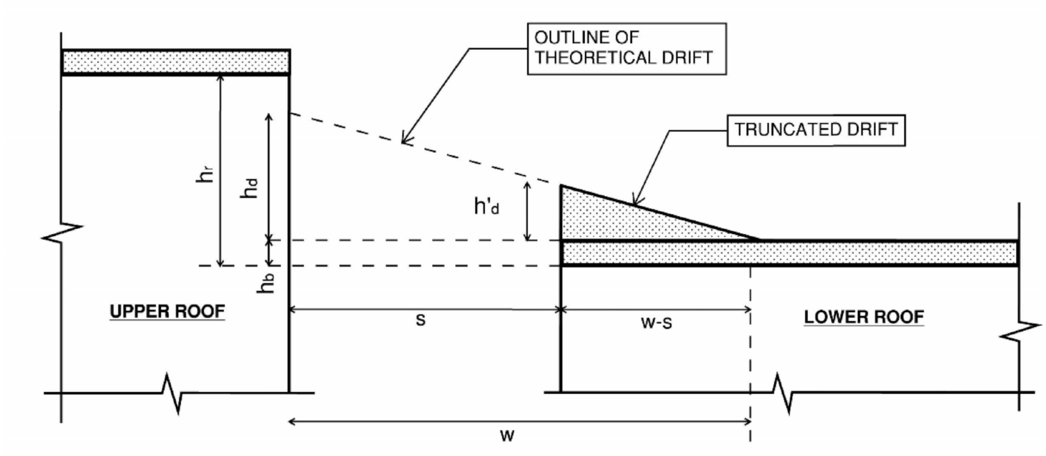
Reference Standards: IBC 2021, ASCE 7-16,

Input *Result* *Check/Confirm*

BASIC SNOW LOAD INPUT

p_g = 20 psf	Basic Ground Snow Load
Surface = B	Terrain Exposure Category - ASCE 7-16 26.7
Exposure on Roof = Partially Exposed	Snow Exposure Factor - ASCE 7-16 Table 7.3-1
$C_e = 1$	
Therm Factor = Unheated and open air structures	Thermal Factor - ASCE 7-16 Table 7.3-2
$C_t = 1.2$	
Occupancy Category = IV	Snow Importance Factor - ASCE 7-16 Table 1.5-2
$I_s = 1.2$	
$C_s = 1$	Sloped roof factor - ASCE 7-16 Figure 7.4-1
$p_f = 0.7 \cdot C_e \cdot C_t \cdot I_s \cdot p_g = 20.16$ psf	Flat roof snow load - ASCE 7-16 Eq. 7.3-1
$p_s = C_s \cdot p_f = 20.16$ psf	Sloped roof snow load - ASCE 7-16, Eq. 7.4-1
$\gamma = \mathrm{Min}\left(30\text{ pcf}, \frac{0.13 \cdot p_g}{1\text{ ft}} + 14\text{ pcf}\right) = 16.6\text{ pcf}$	Density of Snow - ASCE 7-16 Eq. 7.7-1
$h_b = \frac{p_s}{\gamma} = 1.21$ ft	Height of balanced snow load - ASCE 7-16 7.7.1

DRIFT LOAD FOR ROOF OF AN ADJACENT LOWER STRUCTURE:



$L_{upper} = 225\ ft$ — Horizontal length of upper roof (8 bays)

$L_{lower} = 18\ ft$ — Horizontal length of lower roof

$h_r = 40.5\ ft - 12.5\ ft = 28\ ft$ — Height difference between upper and lower roof

$s = 0.25\ ft$ — Horizontal separation between roofs

$h_c = h_r - h_b = 26.79\ ft$ — Adjacent difference in roof height

$$Check_{7,7,1a} = \begin{cases} \text{"Drift does not apply"} & \text{if } \dfrac{h_c}{SnowLoad.h_b} < 0.2 \\ \text{"Drift applies"} & \text{otherwise} \end{cases} = \text{Drift applies}$$

$$L_{upwind} = \begin{cases} L_{upper} & \text{if } L_{upper} > 20\ ft \\ 20\ ft & \text{otherwise} \end{cases} = 225\ ft$$ — Length of roof upwind

Leeward Drift Height and Width:

$$h_{dl} = Min\left(\sqrt{I_s} \cdot \left(0.43 \cdot \left(\frac{L_{upwind}}{1\ ft}\right)^{\left(\frac{1}{3}\right)} \cdot \left(\frac{p_g}{1\ psf} + 10\right)^{\left(\frac{1}{4}\right)} - 1.5\right) \cdot 1\ ft, \frac{6 \cdot h_r - s}{6}\right) = 5.06\ ft$$

$$w_l = Min\left(6 \cdot h_{dl}, 6 \cdot h_r - s\right) = 30.37\ ft$$

Windward Drift Height and Width

$$h_{dw} = 0.75 \cdot \sqrt{I_s} \cdot \left(0.43 \cdot \left(\frac{L_{lower}}{1\ ft}\right)^{\left(\frac{1}{3}\right)} \cdot \left(\frac{p_g}{1\ psf} + 10\right)^{\left(\frac{1}{4}\right)} - 1.5\right) \cdot 1\ ft = 0.934\ ft$$

$$w_w = \text{Min}\left(\left\| \begin{array}{ll} 4 \cdot h_{dw} & \text{if } h_{dw} \le h_c \\ \dfrac{4 \cdot h_{dw}^{\ 2}}{h_c} & \text{otherwise} \end{array}, 8 \cdot h_c\right) = 3.74\ ft$$

Design Drift Loading and Width:

$$Check_{7,7,2} = \left| \begin{array}{ll} \text{"Leeward Drift"} & \text{if And}\left(s < 20\ ft, s < 6 \cdot h_r\right) \\ \text{"Windward Drift"} & \text{otherwise} \end{array} \right. = \text{Leeward Drift}$$

$$h_{d`} = \left| \begin{array}{ll} h_{dl} & \text{if } Check_{7,7,2} = \text{"Leeward Drift"} \\ h_{dw} & \text{otherwise} \end{array} \right. = 5.06\ ft$$ Calculated theoretical drift height

$$w` = \left| \begin{array}{ll} w_l & \text{if } Check_{7,7,2} = \text{"Leeward Drift"} \\ w_w & \text{otherwise} \end{array} \right. = 30.37\ ft$$ Calculated theoretical drift width

$$w = \left| \begin{array}{ll} 0\ ft & \text{if } w` \le s \\ w` - s & \text{otherwise} \end{array} \right. = 30.12\ ft$$ Design drift width from edge of low roof

$$h_d = \left| \begin{array}{ll} 0\ ft & \text{if } w = 0\ ft \\ h_{d`} & \text{if } w` \le h_{d`} \\ \left(\dfrac{h_{d`}}{w`}\right) \cdot \left(w` - s\right) & \text{otherwise} \end{array} \right. = 5.02\ ft$$ Truncated design drift height at edge of low roof

Maximum Intensity of Snow Load:

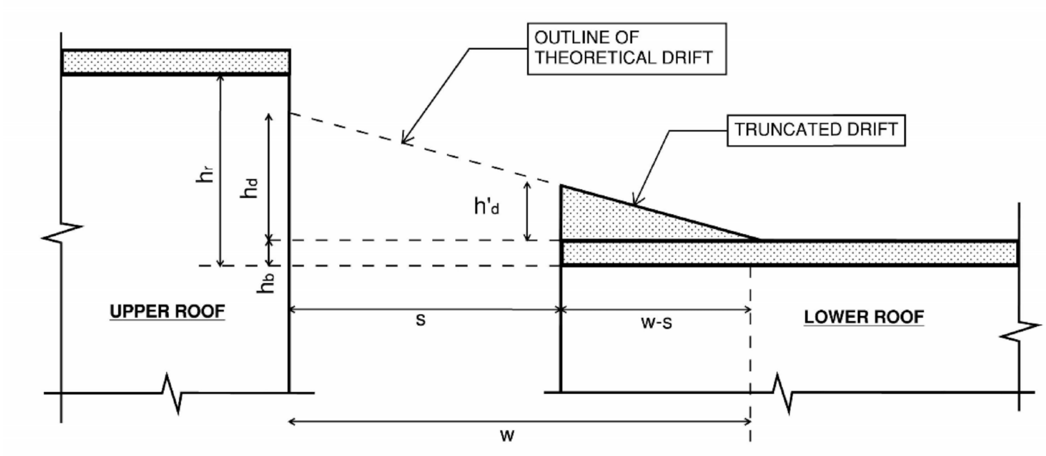
$$p_{max} = h_d \cdot \gamma + p_f = 103.50\ psf$$

$$p_{d,MAX} = p_{max} - p_f = 83.34\ psf$$ Drift surcharge load at peak

$$w_{CHNG} = 1 - \frac{17.5\ ft}{w} = 0.419$$

$$p_{d,MIN} = p_{d,MAX} \cdot w_{CHNG} = 34.92\ psf$$ Drift surcharge load at peak

N-S DIRECTION: DRIFT LOAD FOR ROOF OF AN ADJACENT LOWER STRUCTURE



$L_{upper} = 452\ \text{ft}$ — Horizontal length of upper roof (8 bays)

$L_{lower} = 18\ \text{ft}$ — Horizontal length of lower roof

$h_r = 40.5\ \text{ft} - 12.5\ \text{ft} = 28\ \text{ft}$ — Height difference between upper and lower roof

$s = 0.25\ \text{ft}$ — Horizontal separation between roofs

$h_c = h_r - h_b = 26.79\ \text{ft}$ — Adjusted difference in roof height

$$Check_{7,7,1a} = \begin{cases} \text{"Drift does not apply"} & \text{if } \dfrac{h_c}{SnowLoad.h_b} < 0.2 \\ \text{"Drift applies"} & \text{otherwise} \end{cases} = \text{Drift applies}$$

$$L_{upwind} = \begin{cases} L_{upper} & \text{if } L_{upper} > 20\ \text{ft} \\ 20\ \text{ft} & \text{otherwise} \end{cases} = 452\ \text{ft}$$ Length of roof upwind

Leeward Drift Height and Width:

$$h_{dl} = \text{Min}\left(\sqrt{I_s} \cdot \left(0.43 \cdot \left(\frac{L_{upwind}}{1\ \text{ft}}\right)^{\left(\frac{1}{3}\right)} \cdot \left(\frac{p_g}{1\ \text{psf}} + 10\right)^{\left(\frac{1}{4}\right)} - 1.5\right) \cdot 1\ \text{ft}, \frac{6 \cdot h_r - s}{6}\right) = 6.82\ \text{ft}$$

$$w_l = \text{Min}\left(6 \cdot h_{dl}, 6 \cdot h_r - s\right) = 40.90\ \text{ft}$$

Windward Drift Height and Width

$$h_{dw} = 0.75 \cdot \sqrt{I_s} \cdot \left(0.43 \cdot \left(\frac{L_{lower}}{1\ \text{ft}}\right)^{\left(\frac{1}{3}\right)} \cdot \left(\frac{p_g}{1\ \text{psf}} + 10\right)^{\left(\frac{1}{4}\right)} - 1.5\right) \cdot 1\ \text{ft} = 0.934\ \text{ft}$$

$$w_w = \text{Min}\left(\left|\begin{array}{ll} 4 \cdot h_{dw} & \text{if } h_{dw} \le h_c \\ \dfrac{4 \cdot h_{dw}^{\ 2}}{h_c} & \text{otherwise} \end{array}\right., 8 \cdot h_c\right) = 3.74\ \text{ft}$$

Design Drift Loading and Width:

$$Check_{7,7,2} = \left|\begin{array}{ll} \text{"Leeward Drift"} & \text{if And}\left(s < 20\ \text{ft}, s < 6 \cdot h_r\right) \\ \text{"Windward Drift"} & \text{otherwise} \end{array}\right. = \text{Leeward Drift}$$

$$h_{d`} = \left|\begin{array}{ll} h_{dl} & \text{if } Check_{7,7,2} = \text{"Leeward Drift"} \\ h_{dw} & \text{otherwise} \end{array}\right. = 6.82\ \text{ft}$$ Calculated theoretical drift height

$$w` = \left|\begin{array}{ll} w_l & \text{if } Check_{7,7,2} = \text{"Leeward Drift"} \\ w_w & \text{otherwise} \end{array}\right. = 40.90\ \text{ft}$$ Calculated theoretical drift width

$$w = \left|\begin{array}{ll} 0\ \text{ft} & \text{if } w` \le s \\ w` - s & \text{otherwise} \end{array}\right. = 40.65\ \text{ft}$$ Design drift width from edge of low roof

$$h_d = \left|\begin{array}{ll} 0\ \text{ft} & \text{if } w = 0\ \text{ft} \\ h_{d`} & \text{if } w` \le h_{d`} \\ \left(\dfrac{h_{d`}}{w`}\right) \cdot \left(w` - s\right) & \text{otherwise} \end{array}\right. = 6.78\ \text{ft}$$ Truncated design drift height at edge of low roof

Maximum Intensity of Snow Load:

$$p_{max} = h_d \cdot \gamma + p_f = 132.63\ \text{psf}$$

$$p_{d,MAX} = p_{max} - p_f = 112.47\ \text{psf}$$ Drift surcharge load at peak

$$w_{CHNG} = 1 - \frac{18\ \text{ft}}{w} = 0.557$$

$$p_{d,MIN} = p_{d,MAX} \cdot w_{CHNG} = 62.67\ \text{psf}$$ Drift surcharge load at valley

SEISMIC DESIGN FORCES

Purpose Statement : This calculation is for the determination of Seismic Design Forces for Non-Building Structures Similar to Buildings

Reference Standards: IBC 2021, ASCE 7-16

Input Result Check/Confirm

SITE INFORMATION: REFERENCE: General Notes on S0.00

SEISMIC DESIGN CATEGORY: D (default)

$I_e = 1.5$	Importance Factor
$S_S = 1.257$	0.2s Spectral Response Acceleration
$S_1 = 0.433$	1.0s Spectral Response Acceleration
$S_{DS} = 1.006$	Design Spectral Acceleration
$S_{D1} = S_1 \cdot 1.867 \cdot \left(\frac{2}{3}\right) = 0.539$	Design Spectral Acceleration
$T_L = 6$	Long Period, ASCE Design Hazards Report

SEISMIC FORCE RESISTING SYSTEM:

"STEEL SPECIAL CANTILEVER COLUMN"	TBL. 12.2-1
$R = 2.5$	Seismic Response Factor
$\Omega_0 = 1.25$	Overstrength Factor
$C_d = 2.5$	Deflection Amplification Factor
$h_n = 35\ \text{ft}$	Structural Height Limit

SEISMIC BASE SHEAR:

$h_{canopy} = 12\ ft$ — Height of Canopy

$T_a = 0.028 \cdot \left(\frac{h_{canopy}}{1\ ft}\right)^{0.8} = 0.204$ — Fundamental Period, EQN. 12.8-7

$Wt_{seis} = 30\ kip$ — Seismic Weight

Seismic Response Coefficient (S1<0.6g), ASCE 7-16 EQNS. 12.8-2, 12.8-3, 12.8-4, 12.8-5

$$Cs_{CH12} = \begin{cases} \mathrm{Min}\left(\dfrac{S_{DS}}{\frac{R}{I_e}}, \dfrac{S_{D1}}{T_a \cdot \left(\frac{R}{I_e}\right)}\right) & \text{if } T_a < T_L \\ \mathrm{Min}\left(\dfrac{S_{DS}}{\frac{R}{I_e}}, \dfrac{S_{D1} \cdot T_L}{T_a^{\,2} \cdot \left(\frac{R}{I_e}\right)}\right) & \text{otherwise} \end{cases} = 0.604$$

Seismic Response Coefficient (S1<0.6g), ASCE 7-16 EQNS. 12.8-5

$Cs = \mathrm{Max}\left(Cs_{CH12}, 0.044 \cdot S_{DS} \cdot I_e, 0.01\right) = 0.604$

$V = Cs \cdot Wt_{seis} = 18.11\ kip$

SPECIAL CANTILEVER COLUMN SYSTEMS (SCCS)

AISC SEISMIC DESIGN MANUAL E6 (9.1-57, 9.1-250)

W12X96

DCR_axial ≤ 0.15
2% OK (see RISA 3D output)

Section D1.2a/b (Stability Bracing)
Moderately Ductile Members: Lb = 0.19*ry*E/(Ry*Fy)
ry = 3.09in
E = 29000ksi
Ry = 1.1 (ASTM A992) [AISC 341 TBL. A3.1]
Fy = 50ksi
Lb = 25.8ft > 12ft unbraced height **OK**

Section D1.1 (Highly Ductile)
b/t < 0.32*SQRT(E/Ry/Fy)
12.34 < 0.32*SQRT(29000ksi/1.1/50ksi) = 7.34
OK per AISC 341 TABLE 1-3

Section D1.3 + Section E6.5c + Section I2.1 (Protected Zones)
Length of Protected Zone = 2*d_col
Specify with note on detail
no attachment of the veneer or CFS studs to the column in this region

Section A3.4b + Section I2.3 (Demand Critical Welds)
Specify with note on detail
70ksi filler = 58ksi yield, 70ksi tensile
80ksi filler = 68ksi yield, 80ksi tensile

Section D2.6 (Column Base Connection)
base plates, anchor rods
design with overstrength

MAIN WIND FORCE RESISTING SYSTEM DESIGN FORCES

Purpose Statement: This calculation is for the determination of Wind Design Forces for the MWFRS

USE FOR HORIZONTAL WIND CONTRIBUTION ONLY

Reference Standards: IBC 2021, ASCE 7-16

Input Result Check/Confirm

STRUCTURE:

$h_{max} = 12\ ft$ Structure Height

$w_{bldg} = 36\ ft$ Width of Canopy

$L_{span} = 16\ ft$ Span between Tapered Beams

$d_{bldg} = 18\ ft$ Depth of Canopy

SITE INFORMATION: REFERENCE: General Notes on S0.00

EC = "C" Exposure Category

WIND = 108 Wind Speed (mph)

$K_d = 0.85$ Directionality Factor, ASCE 7-16 26.6

$K_{zt} = 1$ Topographic Factor, ASCE 7-16 26.8

$K_e = 1$ Elevation Factor, ASCE 7-16 26.9

$G = 0.85$ Gust Factor, ASCE 7-16 26.11

$$alpha = \begin{cases} 7 & \text{if } EC = \text{"B"} \\ 9.5 & \text{if } EC = \text{"C"} = 9.5 \\ 11.5 & \text{otherwise} \end{cases} \qquad zg = \begin{cases} 1200 & \text{if } EC = \text{"B"} \\ 900 & \text{if } EC = \text{"C"} = 900 \\ 700 & \text{otherwise} \end{cases}$$

Terrain Exposure Constants

$$K_z = \begin{cases} 2.01 \cdot \left(\frac{15}{zg}\right)^{\left(\frac{2}{alpha}\right)} & \text{if } h_{max} < 15\ ft \\ 2.01 \cdot \left(\frac{h_{max}}{zg}\right)^{\left(\frac{2}{alpha}\right)} & \text{otherwise} \end{cases} = 0.849$$

Velocity Pressure Factor, ASCE 7-16 26.10

$qh = 0.00265\ psf \cdot K_d \cdot K_{zt} \cdot K_e \cdot K_z \cdot WIND^2 = 22.30\ psf$

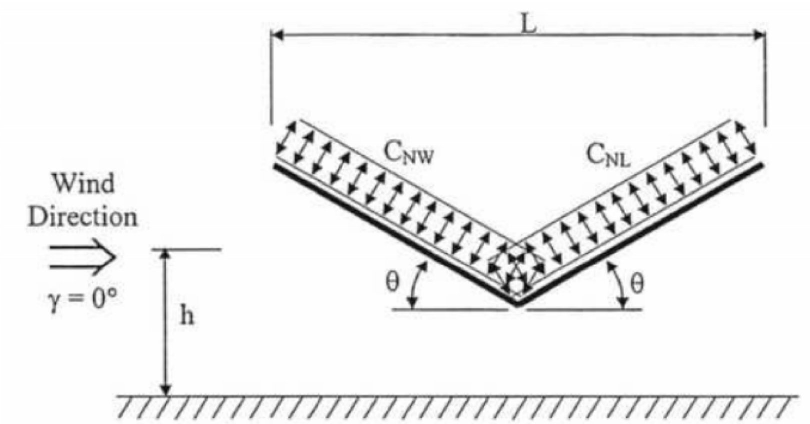
WIND BASE SHEAR (East West):

"OBSTRUCTED WIND FLOW" — ASCE 7-16 FIG. 27.3-6

$A_{rf,WW} = 11\ ft \cdot w_{bldg} = 396\ ft^2$ — Windward Roof Area

$A_{rf,LW} = 7\ ft \cdot w_{bldg} = 252\ ft^2$ — Leeward Roof Area

$ANGL = UnitConvert\left(Asin\left(\frac{2}{12}\right), 1\ deg\right) = 9.59\ deg$ — Canopy Roof Slope Angle



Notation

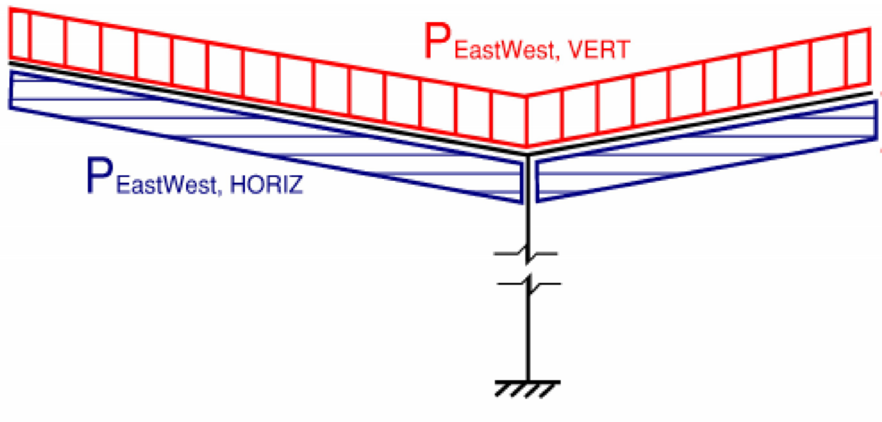
L = Horizontal dimension of roof, measured in the along-wind direction, ft (m).
h = Mean roof height, ft (m).
γ = Direction of wind, degrees.
θ = Angle of plane of roof from horizontal, degrees.

$C_{NW,EastWest,CaseA} = -1.488$ — Windward Wind Force Coefficient

$C_{NW,EastWest,CaseB} = -0.816$ — Interpolated from ASCE 7-16 FIG. 27.3-6

$C_{NL,EastWest,CaseA} = -0.5$ — Leeward Wind Force Coefficient

$C_{NL,EastWest,CaseB} = -0.8$ — ASCE 7-16 FIG. 27.3-6



$P_{NW,EW,CaseA} = qh \cdot G \cdot C_{NW,EastWest,CaseA} = -28.21$ psf

$P_{NL,EW,CaseA} = qh \cdot G \cdot C_{NL,EastWest,CaseA} = -9.48$ psf

$P_{NW,EW,CaseB} = qh \cdot G \cdot C_{NW,EastWest,CaseB} = -15.47$ psf

$P_{NL,EW,CaseB} = qh \cdot G \cdot C_{NL,EastWest,CaseB} = -15.17$ psf

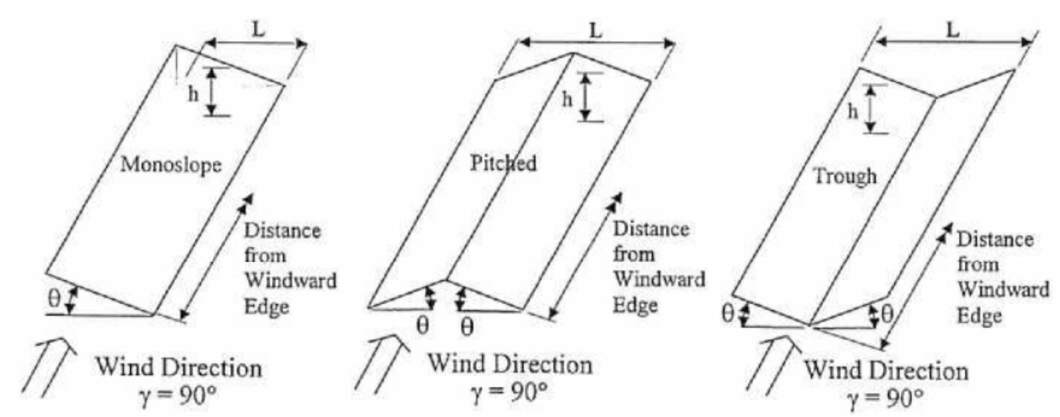
$V_{WIND,EW,VERT} = -9.81$ kip

$V_{WIND,EW,HORIZ} = 2.26$ kip Wind Base Shear

WIND, APPLIED VERTICALLY IN NORTH SOUTH DIRECTION:

"CLEAR WIND FLOW" — ASCE 7-16 FIG. 27.3-6

ANGL = 9.59 deg — Canopy Roof Slope Angle



Notation

L = Horizontal dimension of roof, measured in the along-wind direction, ft (m).
h = Mean roof height, ft (m). See Figs. 27.3-4, 27.3-5, or 27.3-6 for a graphical depiction of this dimension.
γ = Direction of wind, degrees.
θ = Angle of plane of roof from horizontal, degrees.

$C_{0h,NorthSouth,CaseA} = -0.8$ — Wind Force Coefficient

$C_{0h,NorthSouth,CaseB} = 0.8$ — (ASCE 7-16 FIG. 27.3-7)

ONLY applying Wind Force Coefficient within "h" distance from windward edge across roof

$P_{NS,CaseA} = qh \cdot G \cdot C_{0h,NorthSouth,CaseA} = -15.17$ psf

$P_{NS,CaseB} = qh \cdot G \cdot C_{0h,NorthSouth,CaseB} = 15.17$ psf

$V_{WIND,NS,VERT} = Max\left(P_{NS,CaseA}, P_{NS,CaseB}\right) \cdot \left(A_{rf,WW} + A_{rf,LW}\right) = 9.83$ kip — Wind Base Shear

COMPONENTS AND CLADDING WIND DESIGN FORCES

Purpose Statement: This calculation is for the determination of Wind Design Forces for the components and cladding.

USE FOR VERTICAL WIND CONTRIBUTION ONLY

Reference Standards: IBC 2021, ASCE 7-16

Input Result Check/Confirm

STRUCTURE:

$h_{max} = 12\ ft$ Structure Height

$w_{bldg} = 36\ ft$ Width of Canopy

$d_{bldg} = 18\ ft$ Depth of Canopy

$he = 41.5\ ft$ Building Roof Height

$\frac{h_{max}}{he} = 0.289 < 0.5$ Ratio of Canopy to Building Height

SITE INFORMATION:

$qh = MWFRS.qh = 22.30\ psf$

HSS PURLIN WIND PRESSURE:

$w_{trib,HSS,VERT} = 3.5\ ft$ $w_{trib,HSS,HORIZ} = 1.58\ ft$

$L_{HSS} = 16\ ft$

$$A_{eff,VERT} = \begin{cases} \frac{L_{HSS}^2}{3} & \text{if } \frac{L_{HSS}}{3} > w_{trib,HSS,VERT} \\ w_{trib,HSS,VERT} \cdot L_{HSS} & \text{otherwise} \end{cases} = 85.33\ ft^2$$ Effective Area (ASCE 7-16 26.2)

$$A_{eff,HORIZ} = \begin{cases} \frac{L_{HSS}^2}{3} & \text{if } \frac{L_{HSS}}{3} > w_{trib,HSS,HORIZ} \\ w_{trib,HSS,HORIZ} \cdot L_{HSS} & \text{otherwise} \end{cases} = 85.33\ ft^2$$ Area Eff. (ASCE 7-16 26.2)

$GCpi = 0$ Internal Pressure Coefficient

$GC_{PN} = -0.5$ External Pressure Coefficients, ASCE 7-16 FIG. 30.3-11-1B

WT CANTILEVER MIDDLE BEAM WIND PRESSURES:

$w_{trib,WT} = 16\ ft$

$L_{WT} = 11\ ft$ $L_{WT1} = 7\ ft$

$$A_{eff1} = \begin{cases} \dfrac{L_{WT}^2}{3} & \text{if } \dfrac{L_{WT}}{3} > w_{trib,WT} \\ w_{trib,WT} \cdot L_{WT} & \text{otherwise} \end{cases} = 176\ ft^2$$ Effective Area, Long Side, ASCE 7-16 26.2

$$A_{eff2} = \begin{cases} \dfrac{L_{WT1}^2}{3} & \text{if } \dfrac{L_{WT1}}{3} > w_{trib,WT} \\ w_{trib,WT} \cdot L_{WT1} & \text{otherwise} \end{cases} = 112\ ft^2$$ Effective Area, Short Side, ASCE 7-16 26.2

$GC_{PN1} = -0.5$ External Pressure Coefficients, ASCE 7-16 FIG. 30.3-11-1B

$GC_{PN3} = 0.65$ External Pressure Coefficients, ASCE 7-16 FIG. 30.3-11-1B

WT CANTILEVER EDGE BEAM WIND PRESSURES:

$$w_{trib,WT1} = 2\ ft + \frac{16\ ft}{2} = 10\ ft$$

$$A_{eff3} = \begin{cases} \dfrac{L_{WT}^2}{3} & \text{if } \dfrac{L_{WT}}{3} > w_{trib,WT1} \\ w_{trib,WT1} \cdot L_{WT} & \text{otherwise} \end{cases} = 110\ ft^2$$ Effective Area, Long Side, ASCE 7-16 26.2

$$A_{eff4} = \begin{cases} \dfrac{L_{WT1}^2}{3} & \text{if } \dfrac{L_{WT1}}{3} > w_{trib,WT1} \\ w_{trib,WT1} \cdot L_{WT1} & \text{otherwise} \end{cases} = 70\ ft^2$$ Effective Area, Short Side, ASCE 7-16 26.2

$GC_{PN2} = -0.5$ External Pressure Coefficients, ASCE 7-16 FIG. 30.3-11-1B

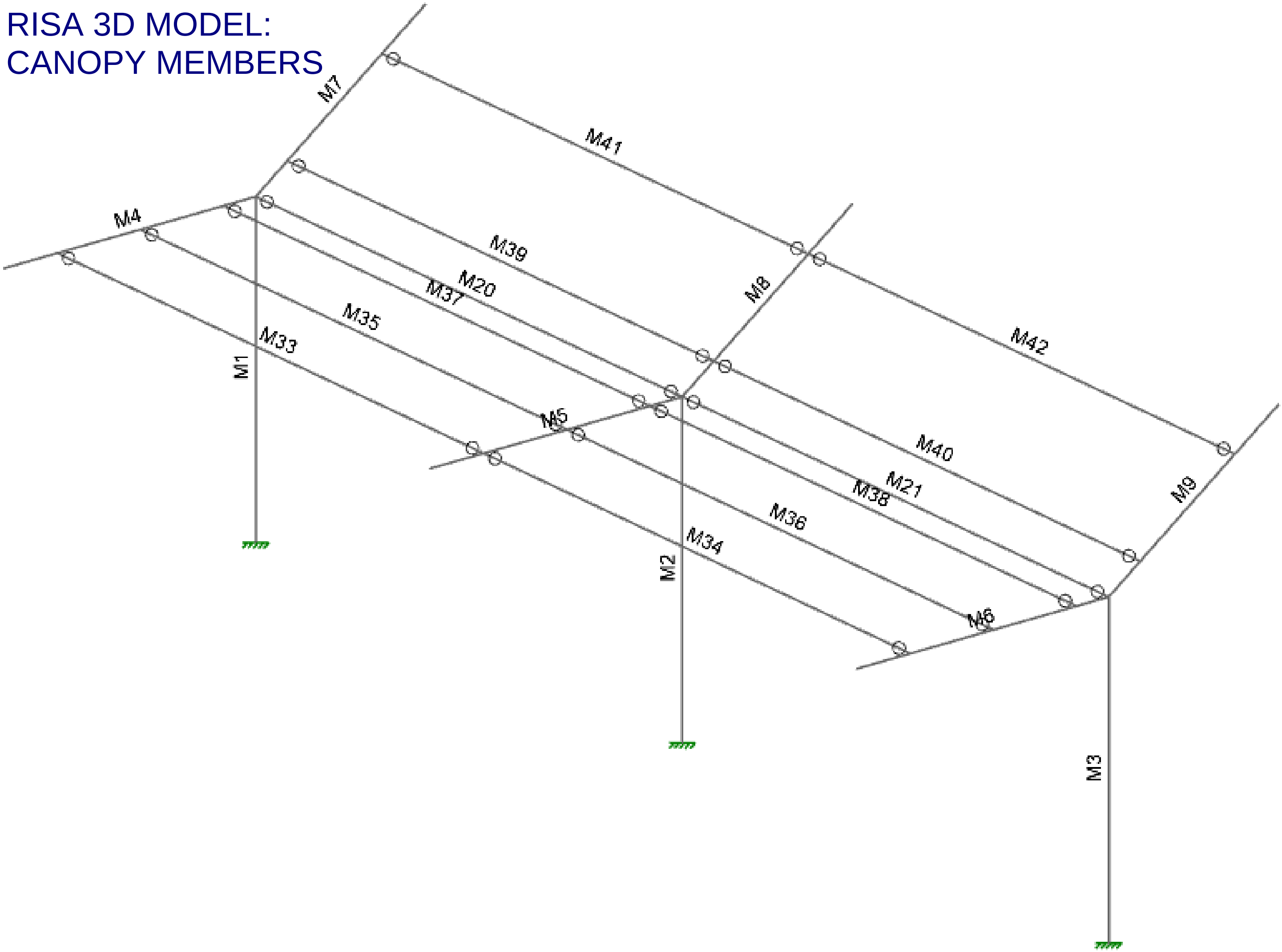
$GC_{PN4} = 0.65$ External Pressure Coefficients, ASCE 7-16 FIG. 30.3-11-1B

$P_{CCWind1} = qh \cdot GC_{PN3} = 14.5\ psf$ ASCE 7-16 EQN. 30.11-1

$P_{CCWind} = qh \cdot GC_{PN} = -11.2\ psf$ ASCE 7-16 EQN. 30.11-1

Wind Pressure (same for all components based on effective areas)

RISA 3D MODEL: CANOPY MEMBERS



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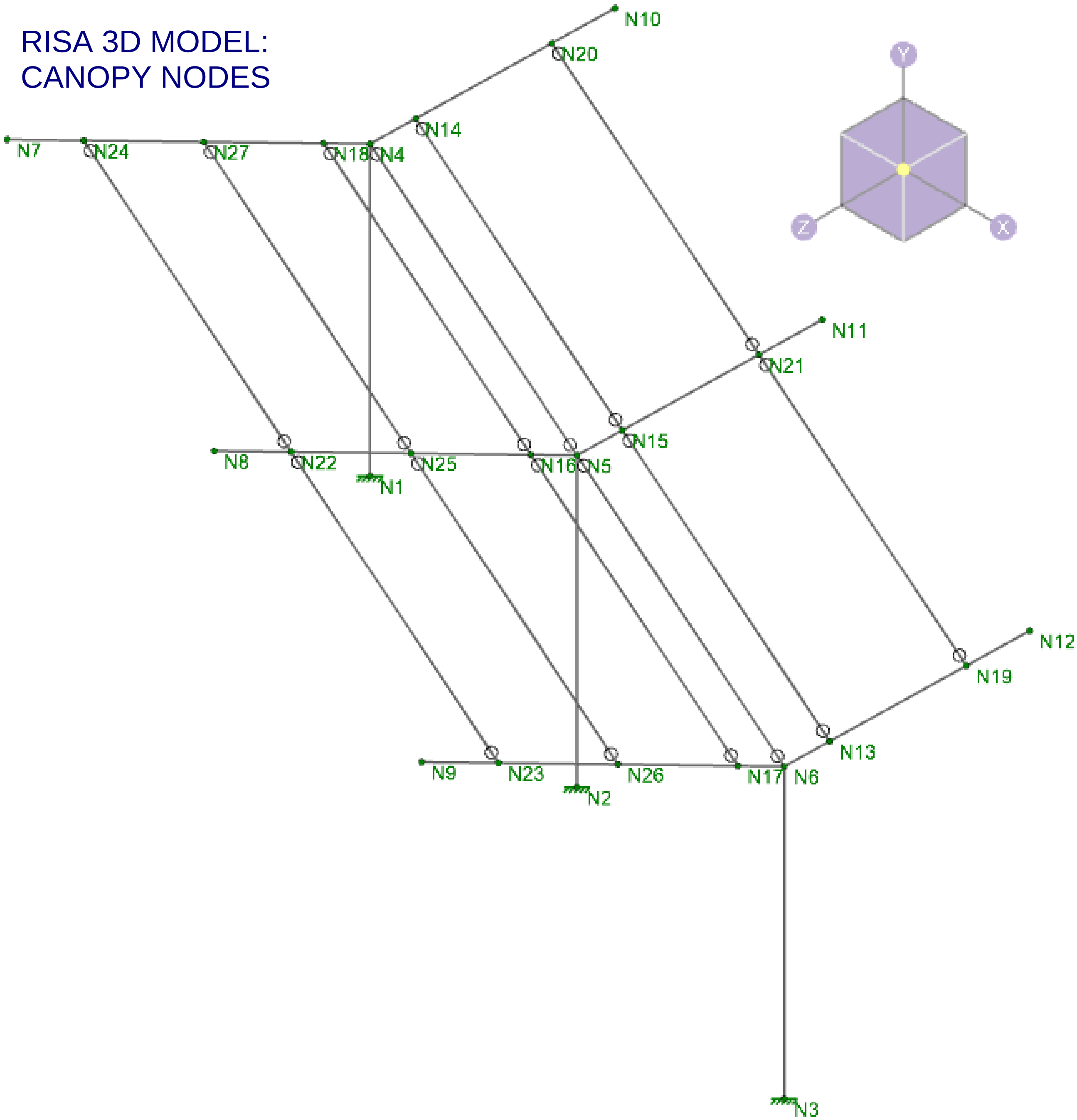
Sht. 01.18 of

RISA 3D MODEL: CANOPY NODES



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Sht. 01.19 of

Hot Rolled Steel Section Sets

	Label	Shape	Type	Design List	Material	Design Rule	Area [in^2]	Iyy [in^4]	Izz [in^4]	J [in^4]
1	COL	W12X96	Column	Wide Flange	A992	Typical	28.2	270	833	6.85
2	TAPER	WT18-WT6	Beam	W_Tee	A992	Typical	11.3	15.388	392.738	0.963
3	PURLIN	HSS6X6X3	Beam	Tube A1085	A500 Gr.C RECT	Typical	3.98	22.3	22.3	35
4	BEAM	W8X24	Beam	Wide Flange	A992	Typical	7.08	18.3	82.7	0.346
5	HR1	WT18-WT6	Beam	None	A992	Typical	11.3	15.388	392.738	0.963

Basic Load Cases

	BLC Description	Category	Y Gravity	Distributed
1	DL	DL	-1	10
2	SL_Drift	SL		10
3	WLz_Case A	WLZ		9
4	WLz_Case B	WLZ		9
5	WLy_down	WLY		6
6	WLy_uplift	WLY		6
7	WLx	WLX		9
8	EQz	ELZ	-0.6	13
9	EQx	ELX	-0.6	13
10	EQy	ELY	0.6	10
11	SL_Sloped	None		10

Load Combinations

	Description	Solve	P-Delta	BLC	Factor	BLC	Factor	BLC	Factor	BLC	Factor
1	1.4D		Y	1	1.4						
2	LC3_Z_case B		Y	1	1.2	2	1.6	3	0.5		
3	LC3_Z_case B		Y	1	1.2	2	1.6	4	0.5		
4	LC3_Y_down		Y	1	1.2	2	1.6	5	0.5		
5	LC3_Y_uplift		Y	1	1.2	2	1.6	6	0.5		
6	LC4_Z_case A		Y	1	1.2	3	1	2	0.5		
7	LC4_Z_case B		Y	1	1.2	4	1	2	0.5		
8	LC4_Y_down		Y	1	1.2	5	1	2	0.5		
9	LC4_Y_uplift		Y	1	1.2	6	1	2	0.5		
10	LC5_Z_case A		Y	1	0.9	3	1				
11	LC5_Z_case B		Y	1	0.9	4	1				
12	LC5_Y_down		Y	1	0.9	5	1				
13	LC5_Y_uplift		Y	1	0.9	6	1				
14	LC6_Z		Y	1	1.2	1	0.201	8	1.25		
15	LC6_X		Y	1	1.2	1	0.201	9	1.25		
16	LC7_Y		Y	1	0.9	1	-0.201	10	1.25		
17	[blank]										
18	ASD_1		Y	1	1	2	1				
19	ASD_3_Z_case a		Y	1	1	2	0.5	3	0.6		
20	ASD_3_Z_case B		Y	1	1	2	0.5	4	0.6		
21	ASD_3_Y_down		Y	1	1	2	0.5	5	0.6		
22	ASD_3_Y_uplift		Y	1	1	2	0.5	6	0.6		
23	ASD_3_X		Y	1	1	2	0.5	7	0.6		
24	ASD_4_Z_case A		Y	1	1	2	1	3	0.3		
25	ASD_4_Z_case B		Y	1	1	2	1	4	0.3		
26	ASD_4_Y_down		Y	1	1	2	1	5	0.3		
27	ASD_4_Y_uplift		Y	1	1	2	1	6	0.3		
28	ASD_4_X		Y	1	1	2	1	7	0.3		
29	ASD_5_Z	Yes	Y	1	1	1	0.201	2	1	8	0.893
30	ASD_5_X	Yes	Y	1	1	1	0.201	2	1	9	0.893

Load Combinations (Continued)

	Description	Solve	P-Delta	BLC	Factor	BLC	Factor	BLC	Factor	BLC	Factor
31	ASD_6_Y	Yes	Y	1	0.6	1	-0.201	10	0.893		
32	[blank]										
33	SERV_E_Z		Y	1	1	1	1.006	7	1		
34	SERV_E_X		Y	1	1	1	1.006	8	1		
35	SERV_D+0.6W_case A		Y	1	1	3	0.6				
36	SERV_D+0.6W_case B		Y	1	1	4	0.6				
37	SERV_D+0.6W_X		Y	1	1	7	0.6				
38	SERV_D+0.5S		Y	1	1	11	0.5				

Load Combination Design

	Description	Service	Hot Rolled	Cold Formed	Wood	Concrete	Masonry	Aluminum	Stainless	Connection
1	1.4D		Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
2	LC3_Z_case B		Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
3	LC3_Z_case B		Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
4	LC3_Y_down		Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
5	LC3_Y_uplift		Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
6	LC4_Z_case A		Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
7	LC4_Z_case B		Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
8	LC4_Y_down		Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
9	LC4_Y_uplift		Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
10	LC5_Z_case A		Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
11	LC5_Z_case B		Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
12	LC5_Y_down		Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
13	LC5_Y_uplift		Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
14	LC6_Z		Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
15	LC6_X		Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
16	LC7_Y		Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
17	[blank]		Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
18	ASD_1		Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
19	ASD_3_Z_case a		Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
20	ASD_3_Z_case B		Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
21	ASD_3_Y_down		Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
22	ASD_3_Y_uplift		Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
23	ASD_3_X		Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
24	ASD_4_Z_case A		Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
25	ASD_4_Z_case B		Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
26	ASD_4_Y_down		Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
27	ASD_4_Y_uplift		Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
28	ASD_4_X		Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
29	ASD_5_Z		Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
30	ASD_5_X		Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
31	ASD_6_Y		Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
32	[blank]		Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
33	SERV_E_Z	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
34	SERV_E_X	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
35	SERV_D+0.6W_case A	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
36	SERV_D+0.6W_case B	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
37	SERV_D+0.6W_X	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
38	SERV_D+0.5S	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes

CANOPY STRENGTH DESIGN

Envelope Maximum Member Section Forces

	Member		Axial[k]	Loc[ft]	LC	y Shear[k]	Loc[ft]	LC	z Shear[k]	Loc[ft]	LC	Torque[k-ft]	Loc[ft]	LC	y-y Moment[k-ft]	Loc[ft]	LC	z-z Moment[k-ft]	Loc[ft]	LC
1	M1	max	29.675	0	4	4.757	0	14	5.865	0	15	7.114	12	15	0	12	16	36.068	0	14
2		min	-0.549	0	16	-0.035	0	4	0	0	1	0	0	1	-63.258	0	15	-30.117	0	4
3	M2	max	56.529	0	4	7.422	0	14	5.861	0	15	7.114	12	15	0	12	16	60.896	0	14
4		min	-1.003	0	16	-0.108	0	4	0	0	1	0	0	1	-63.243	0	15	-48.952	0	4
5	M3	max	29.675	0	4	4.757	0	14	5.866	0	15	7.112	12	15	0	12	16	36.068	0	14
6		min	-0.352	0	16	-0.035	0	4	0	0	1	0	0	1	-63.257	0	15	-30.117	0	4
7	M4	max	4.241	0	2	14.284	0	4	0	11.732	16	0	11.732	16	12.758	0	15	79.892	0	4
8		min	-0.105	0	16	-0.353	0	16	-2.34	0	15	0	0	1	0	0	1	-2.021	0	16
9	M5	max	8.262	0	2	27.854	0	4	0	11.732	16	0	11.732	16	12.823	0	15	145.112	0	4
10		min	-0.204	0	16	-0.689	0	16	-2.401	0	15	0	0	1	0	0	1	-3.571	0	16
11	M6	max	4.241	0	2	14.284	0	4	0	11.732	16	0	11.732	16	12.74	0	15	79.892	0	4
12		min	-0.063	9.288	13	-0.213	9.288	13	-2.309	0	15	0	0	1	0	0	1	-1.066	0	16
13	M7	max	4.464	0	4	12.33	0	4	1.367	1.431	15	0	8.082	16	0.001	6.062	15	50.191	0	4
14		min	-0.067	5.977	14	-0.172	6.062	13	-0.001	6.062	15	0	0	1	-5.444	0	15	-0.388	0	16
15	M8	max	8.749	0	4	24.157	0	4	1.424	1.431	15	0	8.082	16	0.001	6.062	15	97.454	0	4
16		min	-0.213	1.431	14	-0.299	6.062	13	-0.001	6.062	15	0	0	1	-5.51	0	15	-0.843	0	16
17	M9	max	4.464	0	4	12.33	0	4	1.359	1.431	15	0	8.082	16	0.001	6.062	15	50.191	0	4
18		min	-0.067	5.977	14	-0.172	6.062	13	-0.001	6.062	15	0	0	1	-5.428	0	15	-0.364	0	16
19	M20	max	0.033	16	15	0.26	0	15	0	16	16	0	16	14	0	16	16	0.025	8	16
20		min	0	0	1	-0.26	16	14	0	0	1	-0.001	0	4	0	0	1	-1.04	8	14
21	M21	max	0	16	16	0.26	0	15	0	16	16	0.001	16	4	0	16	16	0.025	8	16
22		min	-0.072	0	15	-0.26	16	14	0	0	1	0	0	14	0	0	1	-1.04	8	14
23	M33	max	0.808	0	15	4.488	0	5	0.6	0	14	0.029	16	16	2.4	8	14	0.23	8	16
24		min	-0.392	16	15	-4.488	16	2	-0.6	16	14	-1.19	0	4	0	0	1	-17.954	8	2
25	M34	max	0.395	0	15	4.488	0	5	0.6	0	14	1.19	16	4	2.4	8	14	0.23	8	16
26		min	-0.805	16	15	-4.488	16	2	-0.6	16	14	-0.05	0	16	0	0	1	-17.954	8	2
27	M35	max	0.698	0	15	4.62	0	5	0.52	0	14	0.03	16	16	2.08	8	14	1.408	9	16
28		min	-0.342	16	15	-4.62	16	2	-0.52	16	14	-1.009	0	4	0	0	1	-18.478	8	2
29	M36	max	0.329	0	15	4.62	0	5	0.52	0	14	1.009	16	4	2.08	8	14	0.201	8	16
30		min	-0.711	16	15	-4.62	16	2	-0.52	16	14	-0.052	0	16	0	0	1	-18.478	8	2
31	M37	max	0.78	0	15	4.498	0	5	0.46	0	14	0.056	16	14	1.84	8	14	0.185	8	16
32		min	-0.66	16	15	-4.498	16	2	-0.46	16	14	-0.497	0	4	0	0	1	-17.992	8	2
33	M38	max	0.181	0	15	4.498	0	5	0.46	0	14	0.497	16	4	1.84	8	14	0.185	8	16
34		min	-0.739	16	15	-4.498	16	2	-0.46	16	14	-0.056	0	14	0	0	1	-17.992	8	2
35	M39	max	0.63	0	15	5.285	0	5	0.49	0	14	0.154	16	14	1.96	8	14	0.17	8	16
36		min	-0.35	16	15	-5.285	16	2	-0.49	16	14	-0.077	0	8	0	0	1	-21.141	8	2
37	M40	max	0.357	0	15	5.285	0	5	0.49	0	14	0.077	16	8	1.96	8	14	0.17	8	16
38		min	-0.623	16	15	-5.285	16	2	-0.49	16	14	-0.154	0	14	0	0	1	-21.141	8	2
39	M41	max	0.771	0	15	6.956	0	5	0.58	0	14	0.342	16	2	2.32	8	14	0.217	8	16
40		min	-0.389	16	15	-6.956	16	2	-0.58	16	14	0.001	0	13	0	0	1	-27.822	8	2
41	M42	max	0.39	0	15	6.956	0	5	0.58	0	14	-0.001	16	13	2.32	8	14	0.217	8	16
42		min	-0.77	16	15	-6.956	16	2	-0.58	16	14	-0.342	0	2	0	0	1	-27.822	8	2

Envelope AISC 15TH (360-16): LRFD Member Steel Code Checks

	Member	Shape	Code Check	Loc[ft]	LC	Shear Check	Loc[ft]	Dir	LC	phi*Pnc [k]	phi*Pnt [k]	phi*Mn y-y [k-ft]	phi*Mn z-z [k-ft]	Cb	Eqn
1	M1	W12X106	0.628	0	15	0.269	6	z	15	902.73	1010.88	202.77	442.8	1.002	H1-1b
2	M2	W12X106	0.65	0	15	0.269	6	z	15	902.73	1010.88	202.77	442.8	1.003	H1-1b
3	M3	W12X106	0.628	0	15	0.269	6	z	15	902.73	1010.88	202.77	442.8	1.002	H1-1b
4	M4	WT18-WT6	- The web thickness is greater than the flange thickness -												
5	M5	WT18-WT6	- The web thickness is greater than the flange thickness -												
6	M6	WT18-WT6	- The web thickness is greater than the flange thickness -												
7	M7	WT18-WT6	- The web thickness is greater than the flange thickness -												
8	M8	WT18-WT6	- The web thickness is greater than the flange thickness -												
9	M9	WT18-WT6	- The web thickness is greater than the flange thickness -												

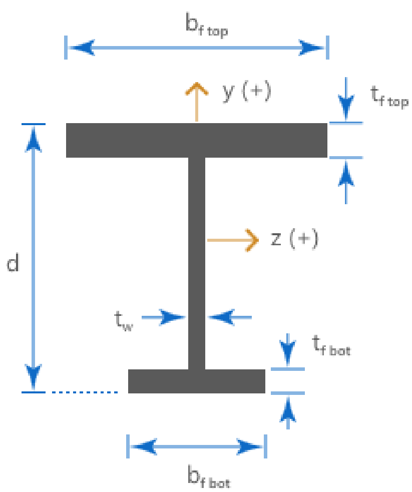
WT members checked separately

TAPERED WT DEMAND

Shape Name: WT18-WT6

Geometric Properties

	Taper WF Start			Taper WF End	
d	18	in	d	6	in
t_w	0.405	in	t_w	0.405	in
$b_{f\,top}$	6.56	in	$b_{f\,top}$	6.56	in
$t_{f\,top}$	0.65	in	$t_{f\,top}$	0.65	in
$b_{f\,bot}$	0.5	in	$b_{f\,bot}$	0.5	in
$t_{f\,bot}$	0.1	in	$t_{f\,bot}$	0.1	in



Section Properties

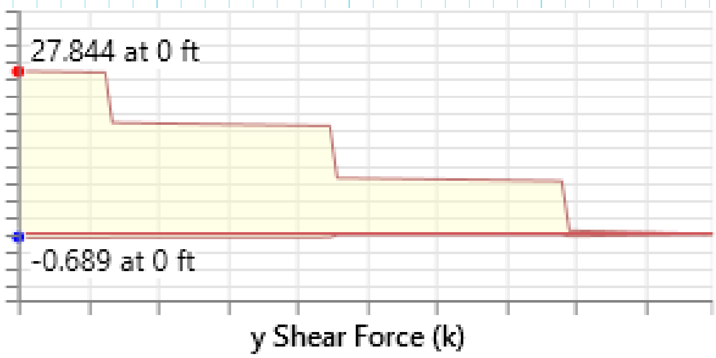
	Taper WF Start			Taper WF End	
I_{yy}	15.388	in^4	I_{yy}	15.321	in^4
I_{zz}	392.738	in^4	I_{zz}	18.45	in^4
Area	11.3	in^2	Area	6.44	in^2
Z_{yy}	7.707	in^3	Z_{yy}	7.214	in^3
Z_{zz}	57.763	in^3	Z_{zz}	6.91	in^3
J	0.963	in^4	J	0.697	in^4

ΦM = Φ*Fy*Z
Fy = 36ksi
Φ_f = 0.9

ΦV = Φ*Fy*b*t*Cv2
Φ_f = 0.6

b/t_start = 49.38 b/t_end = 17.28
1.1*SQRT(1.2*29000ksi/36ksi) = 34.2
Cv2_start = 1.51*1.2*29000ksi/(49.38)^2/36ksi = 0.598
Cv2_end = 1.0

TAPERED WT CAPACITY

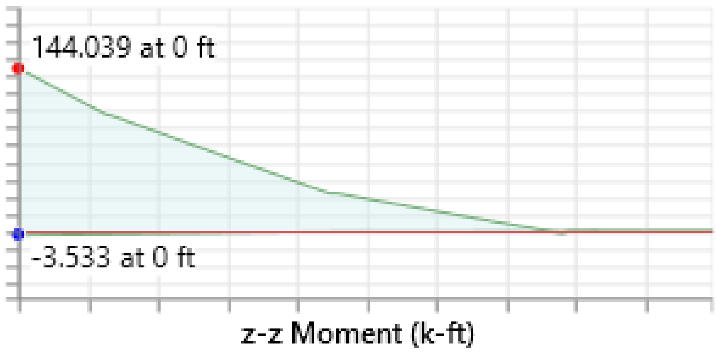


ΦV = 0.6*36ksi*16in*0.405in*0.598
= 83.7K

ΦV = 0.6*36ksi*6in*0.405in
= 52.5K

27.8k < 83.7k
DCR_v: 0.290

0k < 52.5k
DCR_v: 0

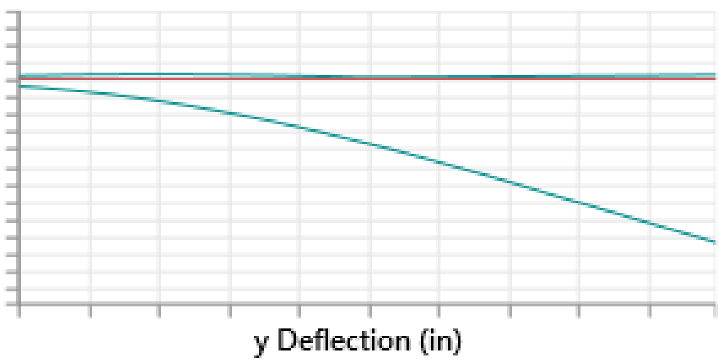


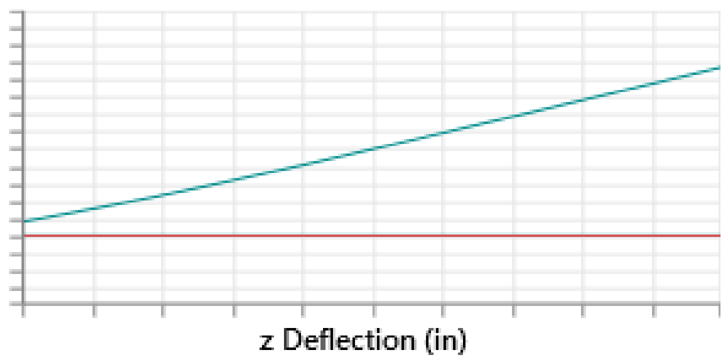
ΦM = 0.9*36ksi*57.7in3
= 1869.4k in = 155.7k ft

ΦM = 0.9*36ksi*6.9in3
= 223.5k in = 18.6k ft

144.1k ft < 155.7k ft
DCR_f: 0.926

0k ft < 18.6k ft
DCR_f: 0.0





STRONG AXIS

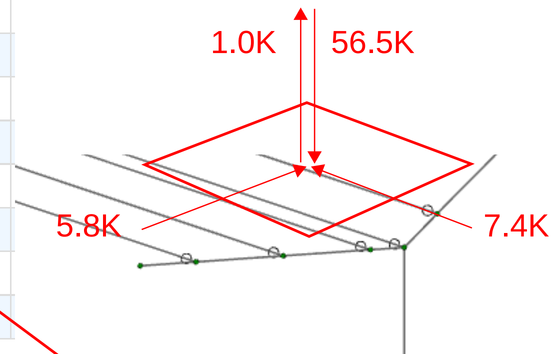
WEAK AXIS

δ = 0.213, -0.528in

δ = 1.693in

LRFD CANOPY BASE REACTIONS

Node Label		X [k]	LC	Y [k]	LC	Z [k]	LC
N1	max	5.819	15	29.675	4	4.75	14
	min	0	1	-0.549	16	0	16
N3	max	5.82	15	29.675	4	4.75	14
	min	0	1	-0.352	16	0	15
N2	max	5.781	15	56.529	4	7.4	14
	min	0	1	-1.003	16	0	16
Totals:	max	17.42	15	115.879	4	16.9	14
	min	0	1	-1.904	16	0	16



(1.2+0.2SDS)DL+omega*EL

(0.9-0.2SDS)DL+omega*EL

(1.2+0.2SDS)DL+omega*EL

Envelope Node Reactions

	Node Label		X [k]	LC	Y [k]	LC	Z [k]	LC	MX [k-ft]	LC	MY [k-ft]	LC	MZ [k-ft]	LC
1	N1	max	5.821	15	29.663	4	4.75	14	35.974	14	7.242	15	0	16
2		min	0	1	-0.549	16	0	16	-30.392	4	0	1	-63.362	15
3	N3	max	5.822	15	29.663	4	4.75	14	35.974	14	7.241	15	0	16
4		min	0	1	-0.352	16	0	15	-30.392	4	0	1	-63.365	15
5	N2	max	5.777	15	56.652	4	7.4	14	60.556	14	7.242	15	0	16
6		min	0	1	-1.008	16	0	16	-49.071	4	0	1	-63.343	15
7	Totals:	max	17.42	15	115.979	4	16.9	14						
8		min	0	1	-1.908	16	0	16						

66	14	N1	0	10.295	4.75	35.974	0	0
67	14	N3	0	10.295	4.75	35.974	0	0
68	14	N2	0	16.827	7.4	60.556	0	0
69	14	Totals:	0	37.418	16.9			
70	14	COG (ft):	X: 16	Y: 11.924	Z: 1.309			
71	15	N1	5.821	10.295	0	-13.81	7.242	-63.362
72	15	N3	5.822	10.296	0	-13.823	7.241	-63.365
73	15	N2	5.777	16.827	0	-21.755	7.242	-63.343

CANOPY STRENGTH DESIGN

Envelope AISC 15TH (360-16): LRFD Member Steel Code Checks (Continued)

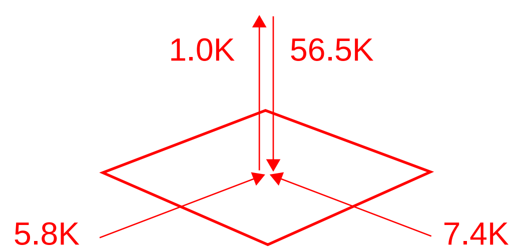
	Member	Shape	Code Check	Loc[ft]	LC	Shear Check	Loc[ft]	Dir	LC	phi*Pnc [k]	phi*Pnt [k]	phi*Mn y-y [k-ft]	phi*Mn z-z [k-ft]	Cb	Eqn
10	M20	W8X15	0.055	8	15	0.007	16	y	14	20.897	143.856	7.209	19.273	1.136	H1-1b
11	M21	W8X15	0.054	8	15	0.007	16	y	14	20.897	143.856	7.209	19.273	1.136	H1-1b
12	M33	HSS6X6X3	0.611	8	5	0.132	16	y	4	110.707	179.1	29.37	29.37	1.136	H1-1b
13	M34	HSS6X6X3	0.611	8	5	0.132	16	y	4	110.707	179.1	29.37	29.37	1.136	H1-1b
14	M35	HSS6X6X3	0.629	8	5	0.128	16	y	4	110.707	179.1	29.37	29.37	1.136	H1-1b
15	M36	HSS6X6X3	0.629	8	5	0.128	16	y	4	110.707	179.1	29.37	29.37	1.136	H1-1b
16	M37	HSS6X6X3	0.613	8	5	0.106	16	y	4	110.707	179.1	29.37	29.37	1.136	H1-1b
17	M38	HSS6X6X3	0.613	8	5	0.106	16	y	4	110.707	179.1	29.37	29.37	1.136	H1-1b
18	M39	HSS6X6X3	0.72	8	5	0.105	16	y	4	110.707	179.1	29.37	29.37	1.136	H1-1b
19	M40	HSS6X6X3	0.72	8	5	0.105	16	y	4	110.707	179.1	29.37	29.37	1.136	H1-1b
20	M41	HSS6X6X3	0.947	8	5	0.148	16	y	2	110.707	179.1	29.37	29.37	1.136	H1-1b
21	M42	HSS6X6X3	0.947	8	5	0.148	16	y	2	110.707	179.1	29.37	29.37	1.136	H1-1b

LRFD CANOPY BASE REACTIONS

APPLIED TO ANCHORAGE AND CONCRETE FOOTING

Envelope Node Reactions

Node Label		X [k]	LC	Y [k]	LC	Z [k]	LC
N1	max	5.819	15	29.675	4	4.75	14
	min	0	1	-0.549	16	0	16
N3	max	5.82	15	29.675	4	4.75	14
	min	0	1	-0.352	16	0	15
N2	max	5.781	15	56.529	4	7.4	14
	min	0	1	-1.003	16	0	16
Totals:	max	17.42	15	115.879	4	16.9	14
	min	0	1	-1.904	16	0	16



(1.2+0.2SDS)DL+omega*EL

(0.9-0.2SDS)DL+omega*EL

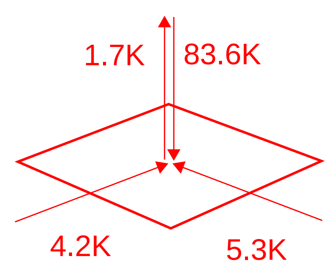
(1.2+0.2SDS)DL+omega*EL

ASD CANOPY BASE REACTIONS

APPLIED TO SOIL THROUGH FOOTING

Envelope Node Reactions

Node Label		X [k]	LC	Y [k]	LC	Z [k]	LC
N1	max	4.172	30	21.716	29	3.393	29
	min	0	18	-0.995	31	0	31
N3	max	4.173	30	21.717	30	3.393	29
	min	0	18	-0.855	31	0	30
N2	max	4.1	30	40.181	29	5.287	29
	min	0	18	-1.734	31	0	31
Totals:	max	12.445	30	83.613	30	12.073	29
	min	0	18	-3.584	31	0	31



(1+0.2*SDS)DL+SL+(0.714*omega)EL

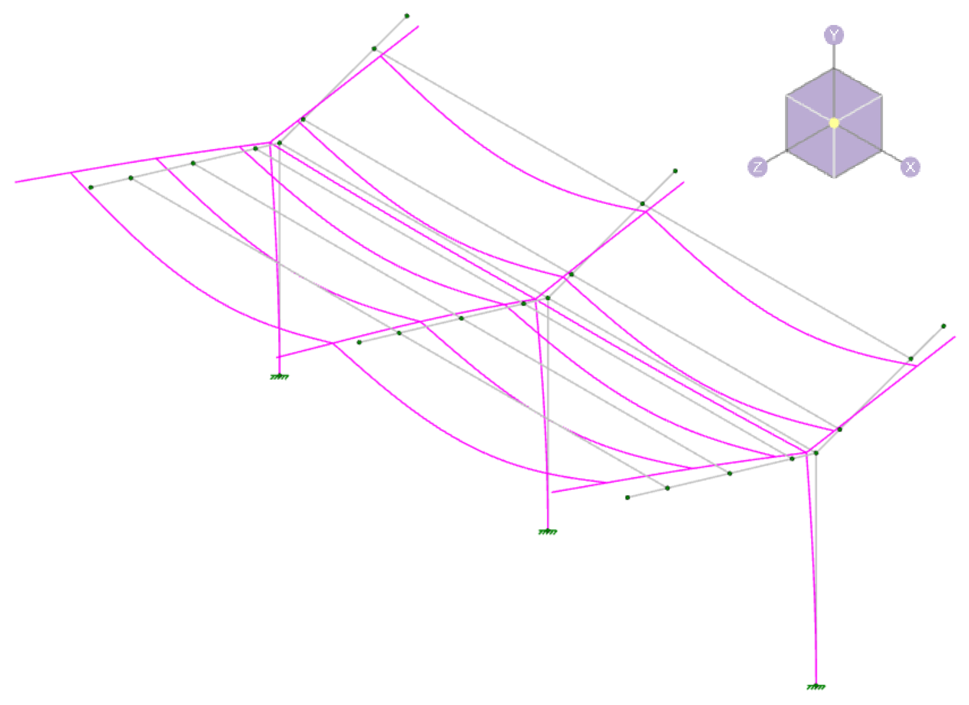
Envelope Node Displacements

	Node Label		X [in]	LC	Y [in]	LC	Z [in]	LC	X Rotation [rad]	LC	Y Rotation [rad]	LC	Z Rotation [rad]	LC
1	N1	max	0	38	0	37	0	38	0	38	0	38	0	38
2		min	0	33	0	34	0	34	0	33	0	33	0	33
3	N2	max	0	38	0	35	0	37	0	38	0	38	0	38
4		min	0	33	0	34	0	34	0	33	0	33	0	33
5	N3	max	0	38	0	35	0	33	0	38	0	38	0	38
6		min	0	33	0	34	0	34	0	33	0	33	0	33
7	N4	max	0	38	-0.001	37	0.1	33	1.384e-3	33	0	38	1.279e-3	33
8		min	-0.119	33	-0.002	34	-0.05	34	-1.384e-4	34	-6.822e-3	33	0	34
9	N5	max	0	38	-0.002	35	0.161	33	2.231e-3	33	0	38	1.279e-3	33
10		min	-0.119	33	-0.004	34	-0.092	34	-3.299e-4	34	-6.822e-3	33	0	34
11	N6	max	0	38	-0.001	35	0.1	33	1.384e-3	33	0	38	1.279e-3	33
12		min	-0.119	33	-0.002	34	-0.05	34	[illegible]	34	-6.822e-3	33	0	34
13	N7	max	0	38	-0.205	35	0.248	33	[illegible]	33	0	38	2.251e-3	33
14		min	-1.454	33	-0.504	33	0.035	34	2.041e-3	35	-1.01e-2	33	0	34
15	N8	max	0	38	-0.346	35	0.402	33	7.651e-3	33	0	38	2.251e-3	33
16		min	-1.454	33	-0.82	33	0.046	34	3.318e-3	35	-1.01e-2	33	0	34
17	N9	max	0	38	-0.205	35	0.248	33	[illegible]	33	0	38	2.251e-3	33
18		min	-1.454	33	-0.504	33	0.035	34	[illegible]	35	-1.01e-2	33	0	34
19	N10	max	0.385	33	0.059	33	0.122	33	3.663e-4	33	0	38	1.63e-3	33
20		min	0	34	-0.093	34	-0.083	34	-1.362e-3	34	-5.853e-3	33	0	34
21	N11	max	0.385	33	0.076	33	0.19	33	3.231e-4	33	0	38	1.63e-3	33
22		min	0	34	-0.171	34	-0.152	34	-2.419e-3	34	-5.853e-3	33	0	34
23	N12	max	0.385	33	0.059	33	0.122	33	3.663e-4	33	0	38	1.63e-3	33
24		min	0	34	-0.093	34	-0.083	34	-1.362e-3	34	-5.853e-3	33	0	34
25	N13	max	0	38	0.018	33	0.107	33	1.077e-3	33	0	38	1.441e-3	33
26		min	-0.016	33	-0.009	34	-0.052	34	-5.027e-4	34	-6.377e-3	33	0	34
27	N14	max	0	38	0.018	33	0.107	33	1.077e-3	33	0	38	1.441e-3	33
28		min	-0.016	33	-0.009	34	-0.052	34	-5.027e-4	34	-6.377e-3	33	0	34
29	N15	max	0	38	0.027	33	0.172	33	1.646e-3	33	0	38	1.44e-3	33
30		min	-0.016	33	-0.017	34	-0.097	34	-9.792e-4	34	-6.377e-3	33	0	34
31	N16	max	0	38	-0.012	34	0.176	33	3.461e-3	33	0	38	1.606e-3	33
32		min	-0.255	33	-0.055	33	-0.09	34	9.02e-4	34	-7.926e-3	33	0	34
33	N17	max	0	38	-0.007	34	0.109	33	2.1e-3	33	0	38	1.606e-3	33
34		min	-0.255	33	-0.034	33	-0.049	34	5.656e-4	34	-7.926e-3	33	0	34
35	N18	max	0	38	-0.007	34	0.109	33	2.1e-3	33	0	38	1.606e-3	33
36		min	-0.255	33	-0.034	33	-0.049	34	5.656e-4	34	-7.926e-3	33	0	34
37	N19	max	0.262	33	0.05	33	0.119	33	3.752e-4	33	0	38	1.624e-3	33
38		min	0	34	-0.061	34	-0.071	34	-1.351e-3	34	-5.87e-3	33	0	34
39	N20	max	0.262	33	0.05	33	0.119	33	3.752e-4	33	0	38	1.624e-3	33
40		min	0	34	-0.061	34	-0.071	34	-1.351e-3	34	-5.87e-3	33	0	34
41	N21	max	0.262	33	0.068	33	0.188	33	3.32e-4	33	0	38	1.624e-3	33
42		min	0	34	-0.114	34	-0.132	34	-2.408e-3	34	-5.87e-3	33	0	34
43	N22	max	0	38	-0.251	35	0.337	33	7.634e-3	33	0	38	2.242e-3	33
44		min	-1.146	33	-0.602	33	0.003	34	3.315e-3	35	-1.007e-2	33	0	34
45	N23	max	0	38	-0.147	35	0.207	33	4.813e-3	33	0	38	2.242e-3	33
46		min	-1.146	33	-0.366	33	0.008	34	2.035e-3	35	-1.007e-2	33	0	34
47	N24	max	0	38	-0.147	35	0.207	33	4.813e-3	33	0	38	2.242e-3	33
48		min	-1.146	33	-0.366	33	0.008	34	2.035e-3	35	-1.007e-2	33	0	34
49	N25	max	0	38	-0.112	35	0.242	33	6.223e-3	33	0	38	2.097e-3	33
50		min	-0.676	33	-0.28	33	-0.058	34	2.632e-3	35	-9.584e-3	33	0	34
51	N26	max	0	38	-0.065	35	0.149	33	3.778e-3	33	0	38	2.097e-3	33
52		min	-0.676	33	-0.169	33	-0.029	34	1.545e-3	35	-9.584e-3	33	0	34
53	N27	max	0	38	-0.065	35	0.149	33	3.778e-3	33	0	38	2.097e-3	33
54		min	-0.676	33	-0.169	33	-0.029	34	1.545e-3	35	-9.584e-3	33	0	34

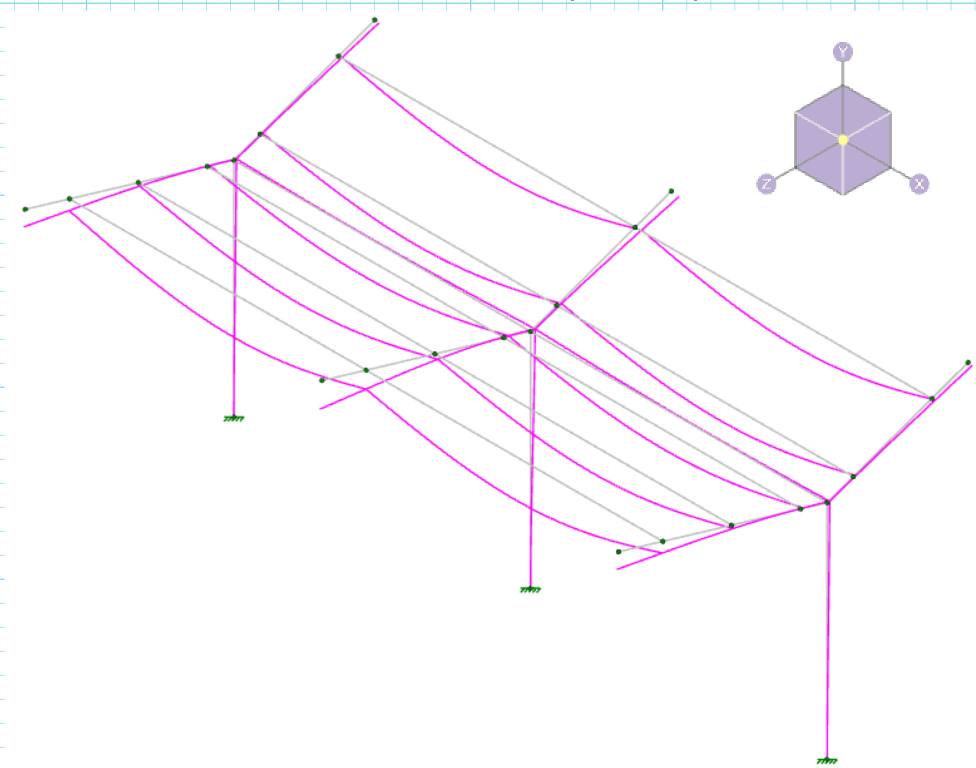
EL_X

EL_Z

RISA 3D MODEL:
CANOPY DEFLECTED SHAPE: (1+SDS)DL + EL_X



RISA 3D MODEL:
CANOPY DEFLECTED SHAPE: (1+SDS)DL + EL_Z



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By RMS

Date 02/26/25

Job# 2170269.07

Sht. 01.28 of

Table 12.12-1 Allowable Story Drift, Δ_a[a,b]

Structure	Risk Category I or II	III	IV
Structures, other than masonry shear wall structures, four stories or less above the base as defined in Section 11.2, with interior walls, partitions, ceilings, and exterior wall systems that have been designed to accommodate the story drifts	$0.025h_{sx}$[c]	$0.020h_{sx}$	$0.015h_{sx}$
Masonry cantilever shear wall structures[d]	$0.010h_{sx}$	$0.010h_{sx}$	$0.010h_{sx}$
Other masonry shear wall structures	$0.007h_{sx}$	$0.007h_{sx}$	$0.007h_{sx}$
All other structures	$0.020h_{sx}$	$0.015h_{sx}$	$0.010h_{sx}$

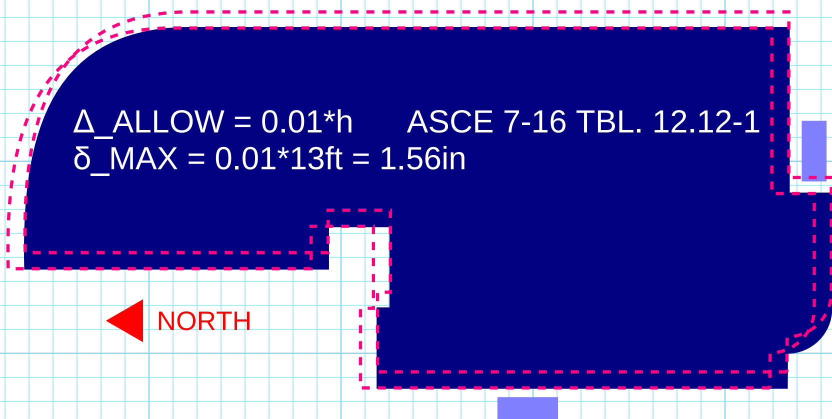
[a] h_{sx} is the story height below level x.
[b] For seismic force-resisting systems solely comprising moment frames in Seismic Design Categories D, E, and F, the allowable story drift shall comply with the requirements of Section 12.12.1.1.
[c] There shall be no drift limit for single-story structures with interior walls, partitions, ceilings, and exterior wall systems that have been designed to accommodate the story drifts. The structure separation requirement of Section 12.12.3 is not waived.
[d] Structures in which the basic structural system consists of masonry shear walls designed as vertical elements cantilevered from their base or foundation support that are so constructed that moment transfer between shear walls (coupling) is negligible.

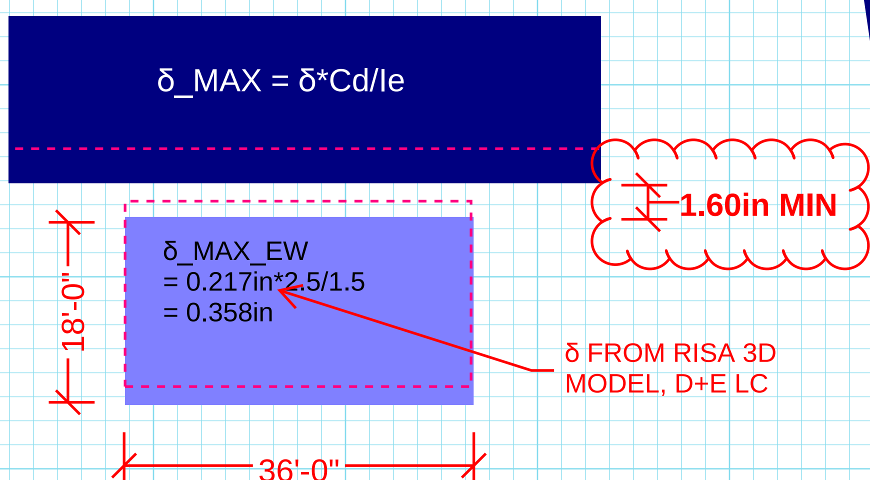
12.12.3 Structural Separation. All portions of the structure shall be designed and constructed to act as an integral unit in resisting seismic forces unless separated structurally by a distance sufficient to avoid damaging contact as set forth in this section.

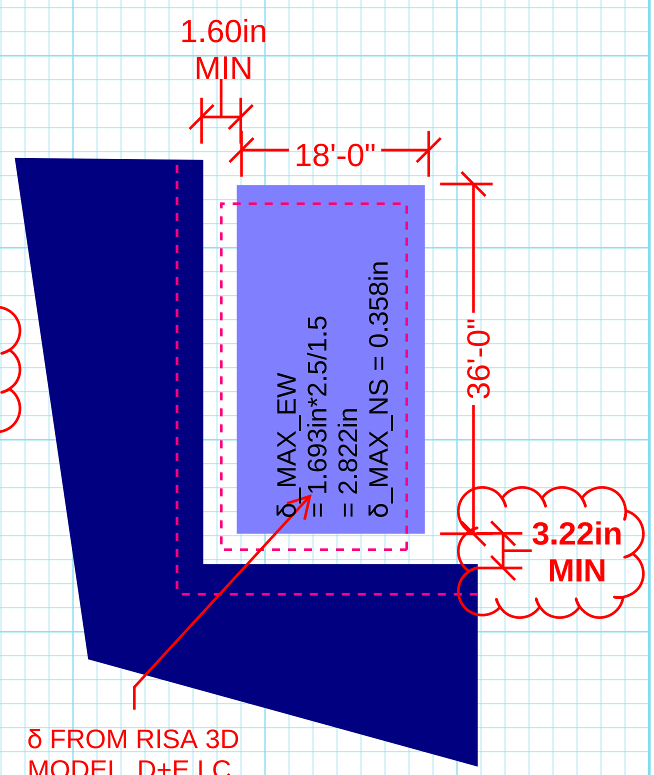
Separations shall allow for the maximum inelastic response displacement (δ_M). δ_M shall be determined at critical locations with consideration for translational and torsional displacements of the structure including torsional amplifications, where applicable, using the following equation:

$$\delta_M = \frac{C_d \delta_{max}}{I_e} \quad (12.12\text{-}1)$$

where δ_{max} = maximum elastic displacement at the critical location.







δ_MT = SQRT(δ_Canopy^2 + δ_Bldg^2)
δ_MT_EW = SQRT(0.36^2+1.56^2) = **1.60in MIN SEPARATE**
δ_MT_NS = SQRT(2.82^2+1.56^2) = **3.22in MIN SEPARATE**

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By RMS
Date 02/26/25
Job# 2170269.07
Sht. 01.29 of

SIGN MOUNTED TO CANOPY

Hot Rolled Steel Section Sets

	Label	Shape	Type	Design List	Material	Design Rules	A [in2]	I (90,270) [i...	I (0,180) [in4]
1	HR1A	W10X33	Beam	None	A992	Typical	9.71	36.6	171

Member Primary Data

	Label	I Joint	J Joint	Rotate(deg)	Section/Shape	Type	Design List	Material	Design Rules
1	M1	N1	N2		HSS4X4X2	Beam	Wide Flange	A500 Gr.B ...	Typical

Basic Load Cases

	BLC Description	Category	X Gravity	Y Gravity	Joint	Point	Distributed
1		None				1	

Load Combinations

	Description	Sol...	PD...	SR...	BLC	Fact...	BLC	Fact...	BLC	Fact...	BLC	Fact...	BLC	Fact...	BLC	Fact...	BLC	Fact...	BLC	Fact...	BLC	Fact...	BLC	Fact...
1	sign load	Yes	Y		1	1																		

Member Point Loads (BLC 1 :)

	Member Label	Direction	Magnitude[k,k-ft]	Location[ft,%]	Inactive
1	M1	y	-0.2	%50	

Member Distributed Loads

Member Label	Direction	Start Magnitud...	End Magnitude...	Start Location[ft,%]	End Location[ft...	Inactive
No Data to Print ...						

Maximum Member Section Forces

	LC	Member Label		Axial[k]	Loc[ft]	Shear[k]	Loc[ft]	Moment[k-ft]	Loc[ft]
1	1	M1	max	0	16	0.1	7.833	0	16
2			min	0	0	-0.1	8	-0.8	8

Member Section Deflections Strength

	LC	Member Label	Sec	x [in]	y [in]	(n) L/y' Ratio
1	1	M1	1	0	0	NC
2			2	0	-0.199	966
3			3	0	-0.289	664
4			4	0	-0.199	966
5			5	0	0	NC

Member AISC 15th (360-16): LRFD Steel Code Checks

	LC	Member	Shape	UC Max	Loc[ft]	Shear UC	Loc[ft]	phi*Pnc[k]	phi*Pnt[k]	phi*Mn[k-ft]	Cb	Eqn
1	1	M1	HSS4X4X2	0.097	8	0.005	16	26.964	73.278	8.24	1.316	H1-1b

MAIN ENTRANCE CANOPY FOOTING:

Purpose Statement: The purpose of this calculation is to check an eccentrically gravity loaded footing for overturning, soil bearing, column punching shear, beam shear, and flexure.

Reference Standards: IBC 2021, ASCE 7-16, ACI 318-19

Input Result Check/Confirm

PROPERTIES:

$q_{all} = 5000$ psf	Soil Bearing
$\gamma_{soil} = 120$ pcf	Density of Soil
$\mu = 0.35$	
$F_y = 60$ ksi	Steel Yield Strength
$f_c = 3$ ksi	Conc. Comp. Strength
$\gamma_{conc} = 150$ pcf	Density of Concrete
$h_{max} = 12$ ft	Canopy Height
$h_{plth} = 1$ ft	Plinth Height
$A_{roof} = 18.5\ \text{ft} \cdot 36\ \text{ft} = 666\ \text{ft}^2$	Roof Area TRIBUTARY TO MIDDLE COL
$W = 32\ \text{ft} + 2\ \text{ft} \cdot 2 = 36\ \text{ft}$	Footing Width
$L = 7$ ft	Footing Length
$t = 1.5$ ft	Footing Thickness
$A_{ftg} = W \cdot L = 252\ \text{ft}^2$	Footing Area
$A_g = W \cdot t = 54\ \text{ft}^2$	Ftg. X-Sect. Area
$d_c = 34\ \text{ft} = 408\ \text{in}$	Column Depth
$b_f = 24$ in	Column Width
$b = b_f = 24$ in $\quad c = d_c = 408$ in	See ACI 318-19 13.2.7.2

Loading:

$P_{canopy,frmg} = 29.7\ kip$ — Canopy + Framing Dead Load to Base Plate

$P_{plth} = \gamma_{conc} \cdot b \cdot c \cdot h_{plth} = 10.2\ kip$ — Plinth Load

$P_{ftg} = \gamma_{conc} \cdot A_{ftg} \cdot t = 56.7\ kip$ — Footing Load

$P_{soil} = \gamma_{soil} \cdot A_{ftg} \cdot h_{plth} - \gamma_{soil} \cdot b \cdot c \cdot h_{plth} = 22.08\ kip$ — Overburden Soil Load

$P_{dl} = P_{canopy,frmg} + P_{plth} + P_{ftg} + P_{soil} = 118.68\ kip$ — Dead Load

$P_{comp,LRFD} = 115.88\ kip$ — LRFD Compress, 1.2D+1.6S+0.5W, RISA3D

$P_{comp,ASD} = 83.62\ kip$ — ASD Compress, D+S+(omega/1.4)E, RISA3D

$P_{tens,LRFD} = -1.91\ kip$ — LRFD Tension, (0.9-0.2SDS)D+(omega)E, RISA3D

$P_{tens,ASD} = -3.58\ kip$ — ASD Tension, (0.9-0.2SDS)D+(omega)E, RISA3D

$V_{WIND} = 2.1\ kip$ — Wind Base Shear

$V_{SEISMIC} = 17.42\ kip$ — Seismic Base Shear

Eccentricity

$$M_{ecc} = \text{Max}\left(\frac{3 \cdot V_{WIND} \cdot h_{max}}{4}, \frac{3 \cdot V_{SEISMIC} \cdot h_{max}}{4}\right) = 1881.36\ kip \cdot in$$

$ecc = \dfrac{M_{ecc}}{P_{dl}} = 15.85\ in$ — Loading Eccentricity

$ecc_{allow,short} = \dfrac{L}{6} = 14\ in$ $\quad ecc_{allow,long} = \dfrac{W}{6} = 72\ in$ — Allowable Eccentricity, Both Directions

Service Load - ASCE 7-16 2.4.1:

$P_{ser,net} = P_{comp,ASD} = 83.62\ kip$ (NOTE: Soil contribution was not included from RISA 3D output load)

$M_{ser,net} = P_{ser,net} \cdot ecc = 110.46\ kip \cdot ft$

Ultimate Loads - ASCE 7-16 2.3.1/2.3.6:

$P_{u,load} = P_{comp,LRFD} = 115.88\ kip$

$M_{u,load} = P_{u,load} \cdot ecc = 153.08\ kip \cdot ft$

Global Stability:

$$FS_{OT} = \frac{\frac{P_{comp,ASD} \cdot L}{2}}{Max\left(0.6 \cdot V_{WIND}, V_{SEISMIC}\right) \cdot \left(\frac{3 \cdot h_{max}}{4}\right)} = 1.87 > 1.5$$

OK

$$FS_{Sliding} = \frac{\mu \cdot P_{dl}}{Max\left(0.6 \cdot V_{WIND}, V_{SEISMIC}\right)} = 2.38 > 1.5$$

OK

Soil Bearing:

$$q_{net} = \begin{cases} \left(\frac{P_{ser,net}}{A_{ftg}}\right) + \left(\frac{6 \cdot M_{ser,net}}{W \cdot L^2}\right) & \text{if } ecc < ecc_{allow,short} \\ \frac{2 \cdot P_{ser,net}}{3 \cdot (0.5 \cdot L - ecc) \cdot W} & \text{if } ecc > ecc_{allow,short} \\ \frac{2 \cdot P_{ser,net}}{A_{ftg}} & \text{otherwise} \end{cases} = 710.67 \text{ psf}$$

NET Bearing Pressure (per geotech report)

$$DCR = \frac{q_{net}}{q_{all}} = 0.142$$

$$Flag = \begin{cases} \text{"OK"} & \text{if } q_{net} \le q_{all} \\ \text{"NG!!"} & \text{otherwise} \end{cases} = OK$$

Beam Flexure Per 1'-0" Width: ACI 318-19 13.2.6.4, 13.2.7.1

ASSUMPTION : q_max is uniformly distributed from the critical section to the edge of footing

$$q_u = \begin{cases} \left(\frac{P_{u,load}}{A_{ftg}}\right) + \left(\frac{6 \cdot M_{u,load}}{W \cdot L^2}\right) & \text{if } ecc < ecc_{allow,short} \\ \frac{2 \cdot P_{u,load}}{3 \cdot (0.5 \cdot L - ecc) \cdot W} & \text{if } ecc > ecc_{allow,short} \\ \frac{2 \cdot P_{u,load}}{A_{ftg}} & \text{otherwise} \end{cases} = 984.84 \text{ psf}$$

Ultimate Soil Bearing Pressure

$M_{u,soil} = q_u \cdot \left(\frac{W - c}{2}\right) \cdot \left(\frac{L - b}{2}\right) \cdot 0.5 \text{ ft} = 14.77 \text{ kip} \cdot \text{in}$ Moment Demand from Soil Bearing

Reinf Size = #6 — Longitudinal Reinf Size

$A_s = 0.44 \text{ in}^2$ — Area of Reinf

$d_s = 0.75 \text{ in}$ — Reinf Diameter

$d = t - 3 \text{ in} - d_s \cdot 1.5 = 13.88 \text{ in}$ — Steel Depth

$A_{s,flex} \cdot F_y \cdot \left(d - \frac{A_{s,flex} \cdot F_y}{0.85 \cdot f_c \cdot 12 \text{ in} \cdot 2}\right) \cdot .9 = M_{u,soil}$ — Required area of steel for footing flexure

$A_{s,flex} = 0.0197 \text{ in}^2$

$A_{smin} = \frac{0.0018 \cdot A_g \cdot (12 \text{ in})}{W} = 0.389 \text{ in}^2$ — Min flexural reinf. per ACI 318-19 8.6.1.1

$sp = 12 \text{ in}$ — Spacing of reinforcement OC

$A_{s,ft} = \frac{A_s \cdot 12 \text{ in}}{sp} = 0.44 \text{ in}^2$ — Area of reinforcement provided per foot

$A_{sreq} = \text{Max}(A_{s,flex}, A_{smin}) = 0.389 \text{ in}^2$

$DCR_s = \frac{A_{sreq}}{A_{s,ft}} = 0.884$

$Flag_2 = \begin{cases} \text{"OK"} & \text{if } A_{s,ft} \geq A_{sreq} \\ \text{"NG!!"} & \text{otherwise} \end{cases} = OK$

Punching Shear Per 1'-0" Width:

ASSUMPTION : q_max is uniformly distributed from the critical section to the edge of footing

$\rho_{\omega} = \dfrac{A_{s,ft}}{12\ in \cdot d} = 0.00264$ Ratio of longitudinal bars defined in R22.5.5

$\lambda_s = 1$ Size effect modification factor, eq 22.5.5.1.3 (Neglected per ACI 318-19 13.2.6.2)

$V_u = P_{u,load} - (c+d)\cdot(b+d)\cdot q_u = 6.60\ kip$ Punching shear demand at critical section per 22.6.4.2

$b_0 = 2\cdot(c+d) + 2\cdot(b+d) = 919.5\ in$ Critical section per 22.6.4.2

$\varphi_v = 0.75$ Strength reduction factor

$\varphi V_c = \varphi_v \cdot \left(4\cdot\lambda_s\cdot\sqrt{f_c\cdot 1\ psi}\cdot b_0\cdot d\right) = 2096.36\ kip$ Table 22.6.5.2 (a)

$DCR_3 = \dfrac{V_u}{\varphi V_c} = 0.00315$

$Flag_3 = \begin{cases} \text{"OK"} & \text{if } \varphi V_c \geq V_u \\ \text{"NG!!"} & \text{otherwise} \end{cases} = OK$

Beam One-Way Shear Per 1'-0" Width:

ASSUMPTION : q_max is uniformly distributed from the critical section to the edge of footing

$V_{u2} = \left(\dfrac{W-c}{2} - d\right)\cdot q_u \cdot 12\ in = -0.154\ kip$ Shear Demand from Soil Bearing "d" Away

$\varphi V_{c,2} = \varphi_v \cdot \left(8\cdot\lambda_s\cdot\rho_{\omega}^{\left(\frac{1}{3}\right)}\cdot\sqrt{f_c\cdot 1\ psi}\cdot d\right)\cdot 12\ in = 7.56\ kip$ One-way Shear Strength, eq 22.5.5.1

$\varphi V_{c,max} = \varphi_v \cdot \left(5\cdot\sqrt{f_c\cdot 1\ psi}\cdot d\cdot 12\ in\right) = 34.20\ kip$ Max One-way Shear Strength, eq 22.5.5.1.1

$\varphi V_{c,3} = \begin{cases} \varphi V_{c,max} & \text{if } \varphi V_{c,2} > \varphi V_{c,max} \\ \varphi V_{c,2} & \text{otherwise} \end{cases} = 7.56\ kip$

$DCR_4 = \dfrac{V_{u2}}{\varphi V_{c,3}} = -0.0203$

$Flag_4 = \begin{cases} \text{"OK"} & \text{if } \varphi V_{c,3} \geq V_{u2} \\ \text{"NG!!"} & \text{otherwise} \end{cases} = OK$

Footing Shrinkage and Temperature Requirements:

$A_{g1} = t \cdot 12\ in = 216\ in^2$	Gross Area of Footing
$A_{s1} = 0.44\ in^2$	Area of Reinf
$A_{sreq1} = 0.0018 \cdot A_{g1} = 0.389\ in^2$	Min S&T reinf. per ACI 318-19 24.4.3.2
$bar_{REQD1} = \dfrac{A_{s1} \cdot 12\ in}{A_{sreq1}} = 13.58\ in$	Bars Spacing Required

1.Project information

Customer company:
Customer contact name:
Customer e-mail:
Comment:

Project description:
Location:
Fastening description:

2. Input Data & Anchor Parameters

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

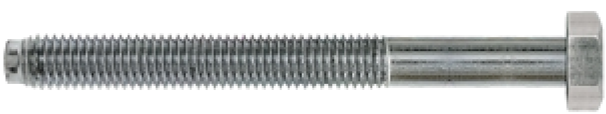
Anchor Information:
Anchor type: Cast-in-place
Material: F1554 Grade 36
Diameter (inch): 1.000
Effective Embedment depth, h_{ef} (inch): 13.000
Anchor category: -
Anchor ductility: Yes
h_{min} (inch): 14.75
C_{min} (inch): 6.00
S_{min} (inch): 6.00

Base Material
Concrete: Normal-weight
Concrete thickness, h (inch): 18.00
State: Uncracked
Compressive strength, f'_c (psi): 3000
$\Psi_{c,V}$: 1.4
Reinforcement condition: Supplementary reinforcement not present
Supplemental edge reinforcement: No
Reinforcement provided at corners: No
Ignore concrete breakout in tension: No
Ignore concrete breakout in shear: No
Ignore 6do requirement: No
Build-up grout pad: Yes

Base Plate
Length x Width x Thickness (inch): 20.00 x 20.00 x 1.00
Yield stress: 36000 psi

Profile type/size: W12X96

Recommended Anchor
Anchor Name: Heavy Hex Bolt - 1"Ø Heavy Hex Bolt, F1554 Gr. 36

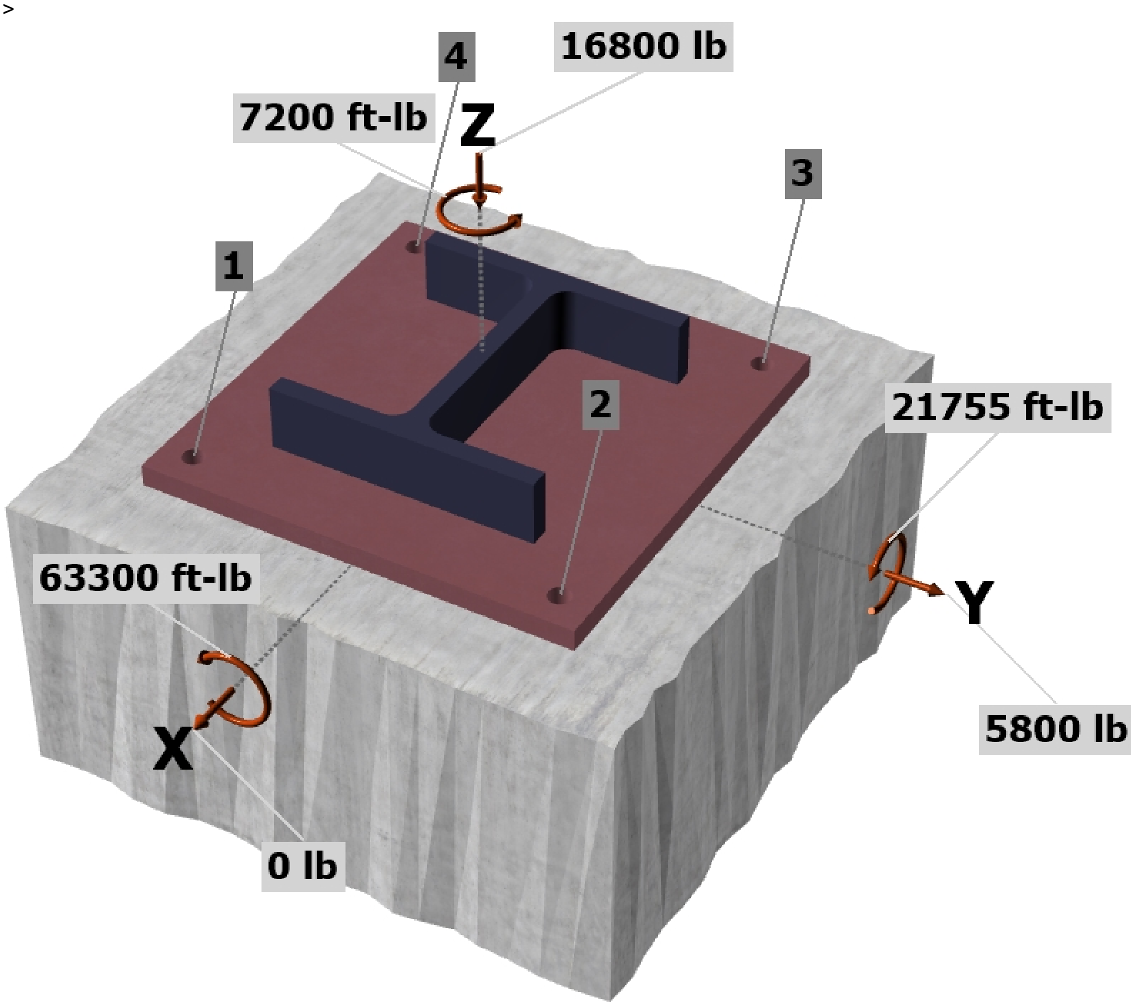


Load and Geometry
Load factor source: ACI 318 Section 5.3
Load combination: not set
Seismic design: Yes
Anchors subjected to sustained tension: Not applicable
Ductility section for tension: 17.10.5.3 (d) is satisfied
Ductility section for shear: 17.10.6.3 (c) is satisfied
Ω_0 factor: not set
Apply entire shear load at front row: No
Anchors only resisting wind and/or seismic loads: Yes

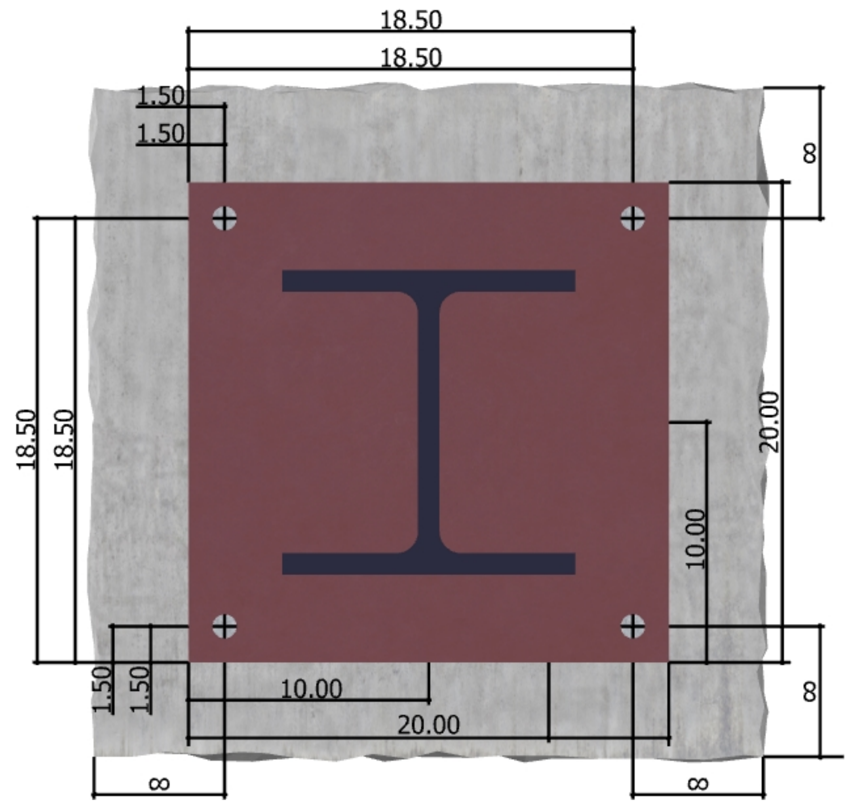
Strength level loads:

N_{ua} [lb]: -16800
V_{uax} [lb]: 0
V_{uay} [lb]: 5800
M_{ux} [ft-lb]: 63300
M_{uy} [ft-lb]: 21755
M_{uz} [ft-lb]: 7200

<Figure 1>



<Figure 2>

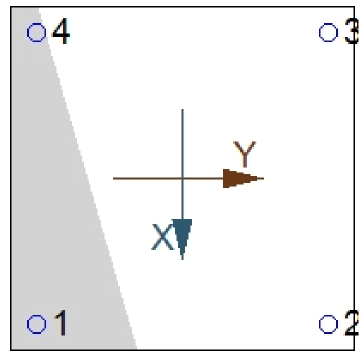


3. Resulting Anchor Forces

Anchor	Tension load, N_{ua} (lb)	Shear load x, V_{uax} (lb)	Shear load y, V_{uay} (lb)	Shear load combined, $\sqrt{(V_{uax})^2+(V_{uay})^2}$ (lb)
1	0.0	1270.5	2720.5	3002.6
2	15426.0	-1270.5	2720.5	3002.6
3	21999.7	-1270.5	179.5	1283.2
4	0.0	1270.5	179.5	1283.2
Sum	37425.7	0.0	5800.0	8571.5

Maximum concrete compression strain (‰): 0.40
Maximum concrete compression stress (psi): 1749
Resultant tension force (lb): 37426
Resultant compression force (lb): 54226
Eccentricity of resultant tension forces in x-axis, e'_{Nx} (inch): 0.00
Eccentricity of resultant tension forces in y-axis, e'_{Ny} (inch): 1.49
Eccentricity of resultant shear forces in x-axis, e'_{Vx} (inch): 0.00
Eccentricity of resultant shear forces in y-axis, e'_{Vy} (inch): 0.00

<Figure 3>



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

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

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

$N_b = 16\lambda_a\sqrt{f'_c}h_{ef}^{5/3}$ (Eq. 17.6.2.2.1)

λ_a	f'_c (psi)	h_{ef} (in)	N_b (lb)
1.00	3000	13.000	62987

$0.75\phi N_{cbg} = 0.75\phi\,(A_{Nc}/A_{Nco})\,\Psi_{ec,N}\Psi_{ed,N}\Psi_{c,N}\Psi_{cp,N}N_b$ (Sec. 17.5.1.2 & Eq. 17.6.2.1a)

A_{Nc} (in^2)	A_{Nco} (in^2)	$c_{a,min}$ (in)	$\Psi_{ec,N}$	$\Psi_{ed,N}$	$\Psi_{c,N}$	$\Psi_{cp,N}$	N_b (lb)	ϕ	$0.75\phi N_{cbg}$ (lb)
2184.00	1521.00	-	0.929	1.000	1.25	1.000	62987	0.70	55132

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

$0.75\phi N_{pn} = 0.75\phi\Psi_{c,P}N_p = 0.75\phi\Psi_{c,P}8A_{brg}f'_c$ (Sec. 17.5.1.2, Eq. 17.6.3.1 & 17.6.3.2.2a)

$\Psi_{c,P}$	A_{brg} (in^2)	f'_c (psi)	ϕ	$0.75\phi N_{pn}$ (lb)
1.4	1.50	3000	0.70	26478

8. Steel Strength of Anchor in Shear (Sec. 17.7.1)

V_{sa} (lb)	ϕ_{grout}	ϕ	$\phi_{grout}\phi V_{sa}$ (lb)
21090	0.8	0.65	10967

10. Concrete Pryout Strength of Anchor in Shear (Sec. 17.7.3)

$\phi V_{cp} = \phi k_{cp} N_{cb} = \phi k_{cp}(A_{Nc} / A_{Nco}) \Psi_{ed,N} \Psi_{c,N} \Psi_{cp,N} N_b$ (Sec. 17.5.1.2 & Eq. 17.7.3.1a)

k_{cp}	A_{Nc} (in^2)	A_{Nco} (in^2)	$\Psi_{ed,N}$	$\Psi_{c,N}$	$\Psi_{cp,N}$	N_b (lb)	ϕ	ϕV_{cp} (lb)
2.0	784.00	1521.00	1.000	1.250	1.000	62987	0.70	56817

11. Results

Interaction of Tensile and Shear Forces (Sec. R17.8)

Tension	Factored Load, N_{ua} (lb)	Design Strength, øN_n (lb)	Ratio	Status
Steel	**22000**	**26363**	**0.83**	**Pass (Governs)**
Concrete breakout	37426	55132	0.68	Pass
Pullout	22000	26478	0.83	Pass

Shear	Factored Load, V_{ua} (lb)	Design Strength, øV_n (lb)	Ratio	Status
Steel	**3003**	**10967**	**0.27**	**Pass (Governs)**
Pryout	3003	56817	0.05	Pass

Interaction check	$(N_{ua}/\phi N_{ua})^{5/3}$	$(V_{ua}/\phi V_{ua})^{5/3}$	Combined Ratio	Permissible	Status
Sec. R17.8	0.74	0.12	85.5%	1.0	Pass

1"Ø Heavy Hex Bolt, F1554 Gr. 36 with hef = 13.000 inch meets the selected design criteria.

12. Warnings

- Calculated concrete compression stress exceeds the permissible bearing stress of Φ 0.85f'c per ACI 318 Section 22.8.3.

- Per designer input, ductility requirements for tension have been determined to be satisfied – designer to verify.

- Per designer input, ductility requirements for shear have been determined to be satisfied – designer to verify.

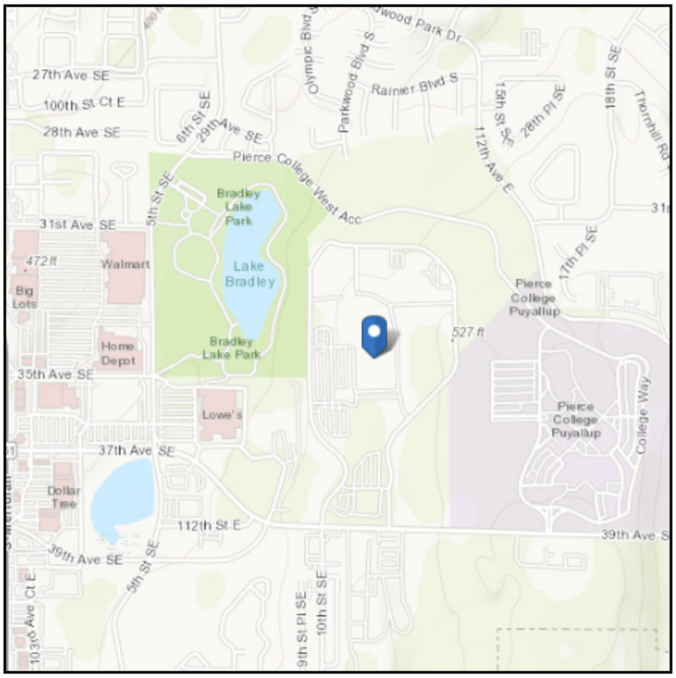
- Designer must exercise own judgement to determine if this design is suitable.

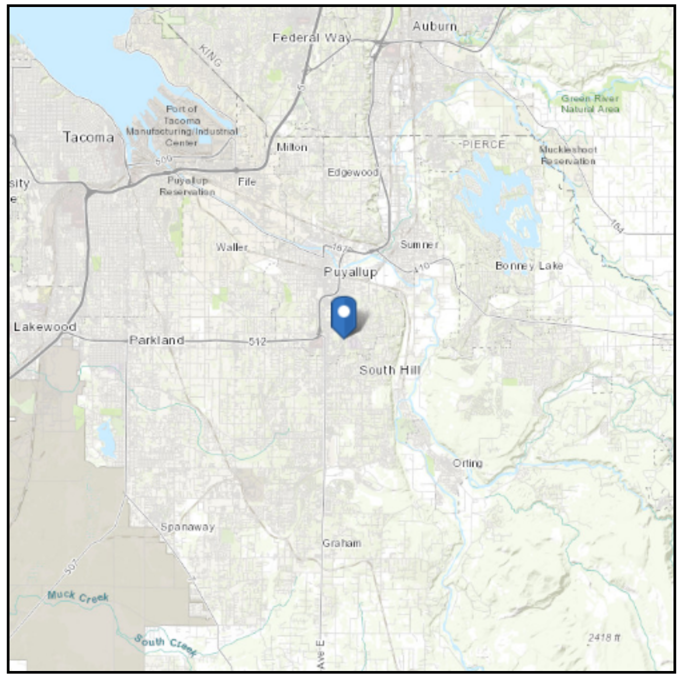
AMERICAN SOCIETY OF CIVIL ENGINEERS

ASCE Hazards Report

Address:
1015 39th Ave SE
Puyallup, Washington
98374

Standard:	ASCE/SEI 7-16	**Latitude:**	47.158856
Risk Category:	IV	**Longitude:**	-122.279864
Soil Class:	D - Default (see Section 11.4.3)	**Elevation:**	487.9235751673597 ft (NAVD 88)





Wind

Results:

Wind Speed	108 Vmph
10-year MRI	67 Vmph
25-year MRI	73 Vmph
50-year MRI	78 Vmph
100-year MRI	83 Vmph

GOVERNING WIND LOADING

Data Source:	ASCE/SEI 7-16, Fig. 26.5-1D and Figs. CC.2-1–CC.2-4, and Section 26.5.2
Date Accessed:	Tue Nov 12 2024

Value provided is 3-second gust wind speeds at 33 ft above ground for Exposure C Category, based on linear interpolation between contours. Wind speeds are interpolated in accordance with the 7-16 Standard. Wind speeds correspond to approximately a 1.6% probability of exceedance in 50 years (annual exceedance probability = 0.00033, MRI = 3,000 years).

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

ASCE
AMERICAN SOCIETY OF CIVIL ENGINEERS

Seismic

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

Results:

S_S :	1.257	S_{D1} :	N/A
S_1 :	0.433	T_L :	6
F_a :	1.2	PGA :	0.5
F_v :	N/A	PGA_M :	0.6
S_{MS} :	1.508	F_{PGA} :	1.2
S_{M1} :	N/A	I_e :	1.5
S_{DS} :	1.006	C_v :	1.351

Ground motion hazard analysis may be required. See ASCE/SEI 7-16 Section 11.4.8.

Data Accessed: Tue Nov 12 2024

Date Source: USGS Seismic Design Maps

01.43

ASCE
AMERICAN SOCIETY OF CIVIL ENGINEERS

Snow

Results:

Mapped Elevation:

Data Source:

Date Accessed: Tue Nov 12 2024

In "Case Study" areas, site-specific case studies are required to establish ground snow loads. Extreme local variations in ground snow loads in these areas preclude mapping at this scale.

Ground snow load determination for such sites shall be based on an extreme value statistical analysis of data available in the vicinity of the site using a value with a 2 percent annual probability of being exceeded (50-year mean recurrence interval).

Statutory requirements of the Authority Having Jurisdiction are not included.

Site is outside ASCE/SEI 7-16, Table 7.2-5 boundaries. For ground snow loads in this area, see SEAW Snow Load Analysis for Washington, 2nd Ed. (1995). Structural Engineers Association of Washington, Seattle, WA.

Snow load values are mapped to a 0.5 mile resolution. This resolution can create a mismatch between the mapped elevation and the site-specific elevation in topographically complex areas. Engineers should consult the local authority having jurisdiction in locations where the reported 'elevation' and 'mapped elevation' differ significantly from each other.

Puyallup Municipal Code

Title 17 BUILDINGS AND CONSTRUCTION
Ch. 17.04 BUILDING CODES

(5) Table R301.2(1), Climatic and Geographical Design Criteria:
(per Ord. 3043 § 6, 2013; Ord. 2962 § 6, 2010)

Table R301.2(1)
Climatic and Geographical Design Criteria

Ground Snow Load	Wind Design		Seismic Design Category[f]	Subject to Damage from			Winter Design Temp[e]	Ice Shield Underlay[h]	Flood Hazards[g]	Air Freeze Index[i]	Mean Annual Temp[j]
	Speed[d] (mph)	Topographical effects[k]		Weath-ering[a]	Frost Line Depth[b]	Termites[c]					
20 lbs/ft	85	No	D-1	Moderate	12 inches	Slight to Moderate	22°	No	Puyallup Municipal Code 21.07	160	51°

USE SL=20psf

WABO - SEAW WHITE PAPER
SNOW LOAD REGULATIONS AND ENGINEERING PRACTICES
WASHINGTON STATE
August 2000

	Elevation[1] (FT)	Recommended Ground Snow Load[2] (PSF)
27. PIERCE		
Ashford	1770	150
Buckley	726	18
Carbonado	1180	60
Chinook Pass	5432 [3]	760
Crystal Mountain Ski Area	4380	438
DuPont	245	15
Eatonville	810	15
Elbe	1211	99
Greenwater	1720	118
Kapowsin	629	35
McMillin Reservoir	580 [3]	18
Longmire	2757	193
Orting	215	18
Paradise	5440 [3]	600
Puyallup	40	18

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ASCE does not intend, nor should anyone interpret, the results provided by this Tool to replace the sound judgment of a competent professional, having knowledge and experience in the appropriate field(s) of practice, nor to substitute for the standard of care required of such professionals in interpreting and applying the contents of this Tool or the ASCE standard.

In using this Tool, you expressly assume all risks associated with your use. Under no circumstances shall ASCE or its officers, directors, employees, members, affiliates, or agents be liable to you or any other person for any direct, indirect, special, incidental, or consequential damages arising from or related to your use of, or reliance on, the Tool or any information obtained therein. To the fullest extent permitted by law, you agree to release and hold harmless ASCE from any and all liability of any nature arising out of or resulting from any use of data provided by the ASCE Hazard Tool.

Title of Calculation

Purpose Statement: The purpose of this calculation is to calculate the out-of-plane capacity of masonry walls with or without openings.

Referenced Standards:
IBC 2018
ASCE 7-16
ACI 318-19
AISC 360-16
TMS 402/602-16

GENERAL

Dimensions

$ht := 20\ ft$ Wall Ht. (Span)

$ht_p := 0\ ft$ Parapet Ht.

$t := 7.625\ in$ Block Thickness

Material Properties

$f_y := 60\ ksi$ Reinf. Yield Strength

$E_s := 29000\ ksi$ Reinf. Elastic Modulus

$\varepsilon_y := \frac{f_y}{E_s} = 0.00207$ Reinf. Yield Strain

$Masonry := "CMU"$ Masonry Type (CMU or Clay)

$f_m := 1500\ psi$ Specified Compressive Strength of Masonry

$E_m = 1350\ ksi$ Elastic Moduli per TMS 402/602-16 4.2.2

$f_r := 200\ psi$ Modulus of Rupture per TMS 402/602-16 9.1.9.2

$\varepsilon_{cu} = 0.0025$ Masonry strain per TMS 402/602-16 9.3.2 (c)

$Wt_p := 81\ psf$ Self weight of wall

Gravity Loads

$DL_{rf} := 0\ psf$ Roof DL

$LL_{rf} := 0\ psf$ Roof Live Load

$SL_{rf} := 0\ psf$ Roof Snow Load

$Drift := 0\ psf$ Drifted Snow

$l_d := 6\ ft$ Drifted Snow Width

$trib_{rf} := 0.1\ ft$ Tributary Width of Roof

$DL_{flr} := 0\ psf$ Floor DL

$LL_{flr} := 0\ psf$ Floor LL

$trib_{flr} := 0.1\ ft$ Tributary Width of Floor

Lateral Loads

Wind Interior 5psf

$p_{net} := 5\ psf = 5\ psf$

Seismic per ASCE 7-16 12.11

$I_e := 1.5$ Importance Factor - Seismic (ASCE 7-16, Table 1.5-2)

$S_{DS} := 1.006$ Design Short Term Spectral Response Acceleration Coefficient

$F_p := \max\left(0.4 \cdot S_{DS} \cdot I_e, 0.1\right) = 0.6$ Out-of-Plane Seismic Design Coefficient (ASCE 7-16, Section 12.11.1)

$q_p := F_p \cdot Wt_p = 48.9\ psf$ Seismic OOP force

Solid Wall

GENERAL

$w := 1\ ft$ Wall Width

$e := \frac{t}{2} + 3\ in = 6.8\ in$ Roof Load Eccentricity

Reinforcement

$d := \frac{t}{2} = 3.8\ in$ Depth of Steel

$s_{vert} := 16\ in$ Spacing of Vertical Reinf.

$nb := 1$ Bars per Cell @ svert

$bar\_size := \#5$ Bar Size

$A_{bar} := A_{bar\_size} = 0.31\ in^2$

$A_s := \frac{nb \cdot A_{bar}}{s_{vert}} = 0.233\ \frac{in^2}{ft}$ Area of Reinforcement per Foot

$\rho := \frac{A_s}{d} = 0.0051$ Reinforcement Ratio per Foot

DEMAND

$P_{dl} := \left(DL_{flr} \cdot trib_{flr}\right) + \left(DL_{rf} \cdot trib_{rf}\right) = 0\ plf$ Axial (Roof + Floor) Dead Load per Foot

$P_{ll\_flr} := LL_{flr} \cdot trib_{flr} = 0\ plf$ Axial (Floor) Live Load per Foot

$P_{ll\_rf} := LL_{rf} \cdot trib_{rf} = 0\ plf$ Axial (Roof) Live Load per Foot

$P_{drift} := Drift \cdot l_d \cdot 0.5 \cdot \frac{trib_{rf} - \left(\frac{l_d}{3}\right)}{trib_{rf}} = 0\ plf$ Snow Drift at Perimeter per Foot

$P_{sl} := \left(SL_{rf} \cdot trib_{rf}\right) + P_{drift} = 0\ plf$ Axial Snow Load (w/Drift) per Foot

$P_{wall} := Wt_p \cdot \left(\frac{ht}{2} + ht_p\right) = 810\ plf$ Weight of Wall per Foot

$P_E := 0.2\ S_{DS} \cdot \left(P_{dl} + P_{wall}\right) = 163\ plf$ Axial Seismic Load per Foot

$$M_w := \frac{p_{net} \cdot ht^2}{8} = 0.3 \; \frac{k \cdot ft}{ft}$$

Ultimate Wind Moment per Foot

$$M_E := \frac{q_p \cdot ht^2}{8} = 2.4 \; \frac{k \cdot ft}{ft}$$

Ultimate Seismic Moment per Foot

Maximum Flexural Steel Reinforcement Check per TMS 402/602-16 9.3.3.2

$$\alpha := 1.5$$

$$\rho_\varepsilon := \frac{0.64 \; f_m \left(\frac{\varepsilon_{cu}}{\varepsilon_{cu} + \alpha \cdot \varepsilon_y}\right) - \frac{\left(P_{dl} + P_{wall}\right) + 0.75 \; \left(P_{ll\_flr} + \left(\max\left(P_{ll\_rf}, P_{sl}\right)\right)\right) + 0.525 \; P_E}{d}}{f_y} = 0.0068$$

$$Ratio := \text{if}\left(\frac{\rho}{\rho_\varepsilon} < 1.0, \text{“OK”}, \text{“NG!!”}\right) = \text{“OK”}$$

CAPACITY

Axial Compression

$$A_n := w \cdot t = 91.5 \; in^2$$

Total Net Area of Wall

$$I_{weak} := \frac{w \cdot t^3}{12} = 443.3 \; in^4$$

Moment of Inertia - Weak Axis

$$r := \sqrt{\frac{I_{weak}}{A_n}} = 2.2 \; in$$

Radius of Gyration

$$\frac{ht}{r} = 109$$

$$P_n := 0.80 \cdot \left(0.80 \; f_m \cdot \left(A_n - A_s \cdot w\right) + f_y \cdot A_s \cdot w\right) = 98.8 \; k$$

$$P_{nlim1} := 0.80 \; \left(0.80 \; f_m \cdot \left(A_n - A_s \cdot w\right) + f_y \cdot A_s \cdot w\right) \cdot \left(1 - \left(\frac{ht}{140 \; r}\right)^2\right) = 38.9 \; k$$

$$P_{nlim2} := 0.80\left(0.80\, f_m \cdot (A_n - A_s \cdot w) + f_y \cdot A_s \cdot w\right) \cdot \left(\frac{70\ r}{ht}\right)^2 = 40.7\ \boldsymbol{k}$$

$$P_n := \text{if}\left(\frac{ht}{r} < 99, min(P_n, P_{nlim1}), min(P_n, P_{nlim2})\right)$$

Nominal Axial Compressive Strength with Slenderness-Dependent Reduction Factors per TMS 402/602-16 9.3.4.1.1

$$K := 1$$

Effective Length Factor

$$\frac{K \cdot ht}{t} = 31.5$$

Slenderness Ratio

$$P_a := \begin{Vmatrix} \text{if } \dfrac{K \cdot ht}{t} \le 30 \\ \quad \Vert (0.20\, f_m \cdot t) \\ \text{else} \\ \quad \Vert 0.05\, f_m \cdot t \end{Vmatrix}$$

Strength Level Axial Load Limit per TMS 402/602-16 9.3.5.4

$$P_n := \max(P_a \cdot w, P_n) = 40.7\ \boldsymbol{kip}$$

$$\phi := 0.9$$

Flexural and Axial Strength Reduction Factor per TMS402/602-16 9.1.4.4

$$\phi P_n := \phi \cdot P_n$$

First Order Analysis

$$x := \begin{bmatrix} 1.2 \cdot P_{dl} \cdot \frac{1}{klf} & 1.2 \cdot P_{wall} \cdot \frac{1}{klf} & 1.6\ \max(P_{ll\_rf}, P_{sl}) \cdot \frac{1}{klf} & 0.5 \cdot M_w \cdot \frac{1}{k} & 0 \\ 1.2 \cdot P_{dl} \cdot \frac{1}{klf} & 1.2 \cdot P_{wall} \cdot \frac{1}{klf} & (P_{ll\_flr} + 0.5 \cdot \max(P_{ll\_rf}, P_{sl})) \cdot \frac{1}{klf} & 1.0 \cdot M_w \cdot \frac{1}{k} & 0 \\ (1.2 + 0.2\ S_{DS}) \cdot P_{dl} \cdot \frac{1}{klf} & (1.2 + 0.2\ S_{DS}) \cdot P_{wall} \cdot \frac{1}{klf} & (0.2 \cdot P_{sl} + P_{ll\_flr}) \cdot \frac{1}{klf} & 1.0 \cdot M_E \cdot \frac{1}{k} & 0 \\ 0.9 \cdot P_{dl} \cdot \frac{1}{klf} & 0.9 \cdot P_{wall} \cdot \frac{1}{klf} & 0 & 1.0 \cdot M_w \cdot \frac{1}{k} & 0 \\ (0.9 - 0.2\ S_{DS}) \cdot P_{dl} \cdot \frac{1}{klf} & (0.9 - 0.2\ S_{DS}) \cdot P_{wall} \cdot \frac{1}{klf} & 0 & 1.0 \cdot M_E \cdot \frac{1}{k} & 0 \end{bmatrix}$$

$$i := 0..4$$

$$A_g := t \cdot 12\ \boldsymbol{in}$$

Gross Area per Foot

$$P_{u_i} := \left(x_{i,0} + x_{i,1} + x_{i,2}\right)\ \boldsymbol{k}$$

$$a_i := \frac{\frac{P_{u_i}}{\phi} + A_s \cdot ft \cdot f_y}{0.8 \cdot f_m \cdot 12 \cdot in}$$

$$M_{n_i} := \left(\frac{P_{u_i}}{\phi} + A_s \cdot ft \cdot f_y\right) \cdot \left(d - \frac{a_i}{2}\right)$$

$$\phi M_{n_i} := \phi \cdot M_{n_i}$$

Second Order Analysis (Alternative Basic Load Combinations)

$$\omega := 1$$

$$y := \begin{bmatrix} 1 \cdot P_{dl} \cdot \frac{1}{klf} & 1 \cdot P_{wall} \cdot \frac{1}{klf} & 1.0 \cdot \left(P_{ll\_flr} + \max\left(P_{sl}, P_{ll\_rf}\right)\right) \cdot \frac{1}{klf} & 0.5 \cdot \left(\omega \cdot 0.6 \cdot M_w\right) \cdot \frac{1}{k} & 0 \\ 1 \cdot P_{dl} \cdot \frac{1}{klf} & 1 \cdot P_{wall} \cdot \frac{1}{klf} & \left(0.5 \cdot P_{sl} + P_{ll\_flr}\right) \cdot \frac{1}{klf} & 1.0 \cdot \left(\omega \cdot 0.6 \cdot M_w\right) \cdot \frac{1}{k} & 0 \\ \left(1 + \frac{0.2\ S_{DS}}{1.4}\right) \cdot P_{dl} \cdot \frac{1}{klf} & \left(1 + \frac{0.2\ S_{DS}}{1.4}\right) \cdot P_{wall} \cdot \frac{1}{klf} & 0 & \left(\frac{1}{1.4}\right) \cdot M_E \cdot \frac{1}{k} & 0 \\ 0.67 \cdot P_{dl} \cdot \frac{1}{klf} & 0.67 \cdot P_{wall} \cdot \frac{1}{klf} & 0 & 1.0 \cdot \left(\omega \cdot 0.6 \cdot M_w\right) \cdot \frac{1}{k} & 0 \\ \left(0.9 - \frac{0.2\ S_{DS}}{1.4}\right) \cdot P_{dl} \cdot \frac{1}{klf} & \left(0.9 - \frac{0.2\ S_{DS}}{1.4}\right) \cdot P_{wall} \cdot \frac{1}{klf} & 0 & \left(\frac{1}{1.4}\right) \cdot M_E \cdot \frac{1}{k} & 0 \end{bmatrix}$$

$$I_g := \frac{12\ in \cdot t^3}{12}$$

$$M_{cr} := \frac{f_r \cdot I_g}{\left(\frac{t}{2}\right)}$$

$$n := \frac{E_s}{E_m} = 21.5$$

$$c_i := \frac{P_{u_i} + A_s \cdot ft \cdot f_y}{0.64 \cdot f_m \cdot 12 \cdot in}$$

Neutral Axis Depth per TMS402/602-16 9.3.5.4.5

$$A_{se_i} := \frac{P_{u_i} \cdot t}{f_y \cdot 2\ d} + A_s \cdot ft$$

$$12 \cdot in \cdot (c)^3$$

$$I_{cr_i} := \frac{(\ \ i)}{3} + n \cdot A_{se_i} \cdot \left(d - c_i\right)^2$$

Cracked moment of inertia per TMS402/602-16 9.3.5.4.5

$$\Delta_{n_i} := \frac{5 \cdot M_{n_i} \cdot ht^2}{48 \cdot E_m \cdot I_{cr_i}}$$

$$M_{u_i} := x_{i,3}\ \boldsymbol{lk} + \left(x_{i,0} + x_{i,2}\right) \boldsymbol{k} \cdot \frac{e}{2} + \left(x_{i,0} + x_{i,1} + x_{i,2}\right) \boldsymbol{k} \cdot \Delta_{n_i}$$

Strength Level Moment at Midheight per TMS402/602-16 Equation 9-23

$$M_{s_i} := y_{i,3}\ \boldsymbol{lk} + \left(y_{i,0} + y_{i,2}\right) \boldsymbol{k} \cdot \frac{e}{2} + \left(y_{i,0} + y_{i,1} + y_{i,2}\right) \boldsymbol{k} \cdot 0.007\ ht$$

Serviceable Level Moment at Midheight per TMS402/602-16 Equation 9-23

$$\Delta_{cr} := \frac{5 \cdot M_{cr} \cdot ht^2}{48 \cdot E_m \cdot I_g}$$

Deflection at cracking moment per TMS 402/602-16 Eq. 9.25

$$\Delta_{s_i} := \Delta_{cr} + \frac{5\left(M_{s_i} - M_{cr}\right) ht^2}{48 \cdot E_m \cdot I_{cr_i}}$$

Deflection due to allowable stress level loads beyond cracking per TMS 402/602-16 Eq. 9-26, 9.3.5.4.2 and 9.3.5.5.1

$$\Delta_{max} := 0.007\ ht$$

Deflection Limit per TMS 402/602-16 9.3.5.5

$$\Delta_{s_i} := \text{if}\left(M_{s_i} < M_{cr}, \frac{5 \cdot M_{s_i} \cdot ht^2}{48 \cdot E_m \cdot I_g}, \Delta_{s_i}\right)$$

SUMMARY

Dimensions

$ht = 20\ ft$ — Wall Ht. (Span)

$w = 1\ ft$ — Wall Width

Masonry

$t = 7.625\ in$ — Block Thickness

$f_m = 1500\ psi$ — Specified Compressive Strength of Masonry

Reinforcement

$d = 3.813\ in$ — Depth of Steel

$s_{vert} = 16\ in$ — Spacing of Vertical Reinf.

$nb = 1$ — Bars per Cell

$rebar_{bar\_size} = \text{“\#5”}$ — Bar Size

Title of Calculation

Strength Capacity (LRFD)

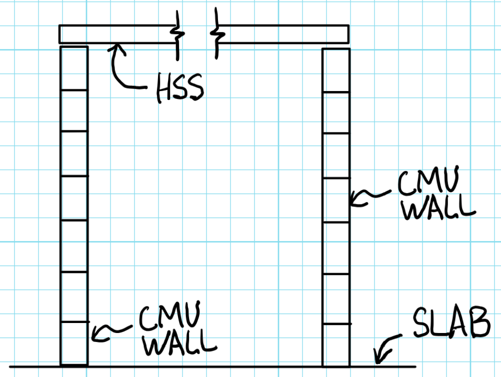
Load Combination	Axial Check	Moment Demand (M_u) (k-in)	Moment Capacity (ΦM_n) (k-in)	DCR	Capacity Check
1.2DL+1.6L+0.5W	OK	6.52	44.51	0.15	OK
1.2D+1.0W+0.5S	OK	8.02	44.51	0.18	OK
1.2D+1.0E+0.5S	OK	35.22	44.96	0.78	OK
0.9D+1.0W	OK	6.75	43.84	0.15	OK
0.9D+1.0E	OK	32.23	43.38	0.74	OK

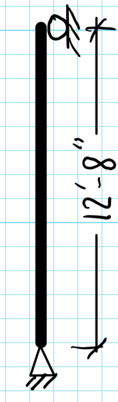
Service Deflection (ASD)

Load Combination	Deflection Defl. (Δ_s) (in)	Deflection Defl. Limit (Δ_{max}) (in)	DCR	Deflection Check
1.0D+1.0L+0.5W	0.02	1.68	0.01	OK
1.0D+1.0W+0.5S	0.03	1.68	0.02	OK
1.0D+1.0S+E/1.4	0.23	1.68	0.13	OK
0.67D+1.0W	0.03	1.68	0.02	OK
0.9D+E/1.4	0.22	1.68	0.13	OK

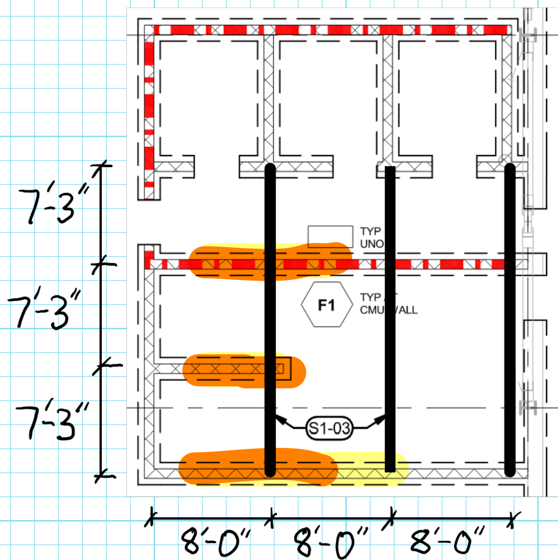
Wall with Opening

HSS WALL TIE





WALL



$S_{DS} = 1.006$
$I_e = 1.5$ (RC IV)

CMU: 8" BLOCK
FULLY GROUTED
125 PCF
$q_{CMU} = 81$ PSF

STL: TRY HSS5X5X1/4
L = 21.75 FT
TRIB WALL
LENGTH: ~~12~~+4+8 = ~~24~~ FT 20FT

· DEMAND

$P_{CMU} = 0.4 \times S_{DS} \times I_e \times q_{CMU} = 0.4(1.006)(1.5)(81 psf) = 49 PSF$

$F_{HSS} = P_{CMU} \times h_{WALL}/2 \times L_{WALL\ TRIB} = 49 PSF\left(\frac{12.7FT}{2}\right)(\cancel{24FT}) = \cancel{7.5K}$ 6.23K ← TOTAL AXIAL DEMAND

~~(2.5K) ← ANCHOR DEMAND~~

· CAPACITY

$1.4\sqrt{E/F_y} = 1.4\sqrt{\frac{29000 ksi}{36 ksi}} = 39.7$ AISC 360 TBL. B4.1a

$\frac{b}{t} = 18.5 < 20.0$ → NON-SLENDER

$4.71\sqrt{E/F_y} = 4.71\sqrt{\frac{29000 ksi}{36 ksi}} = 133.7$ AISC 360 E3

$KL/r = (1.0 \times 21.75 ft)/(1.55 in/12 in) = 168.3 > 133.7$

↳ $F_e = (\pi^2 E)/(KL/r)^2 = (\pi^2 29000 ksi)/(168.3)^2 = 10.1 ksi$ AISC 360 EQN. E3-4

$F_{cr} = 0.877 F_e = 0.877(10.1 ksi) = 8.85 ksi$

$\phi P_n = F_{cr} A_g = 0.9(13.68 ksi)(2.58 in^2) = 20.5K$

$\boxed{\text{DCR: 0.366 \quad HSS4x4x3/16}}$

PPSB

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By RMS
Date 01/30/25
Job# 2170269.07
Sht. ______ of

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Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	HSS TO CMU ANCHOR	Date:	2/7/2025
Fastening point:			

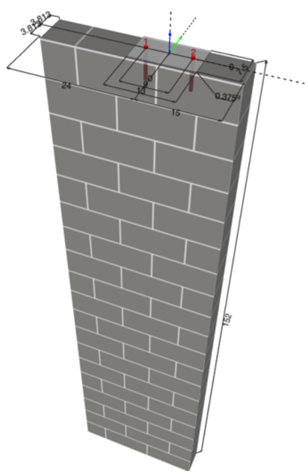
Specifier's comments:

1 Input data

Anchor type and diameter:	**HY 200-A V3 + threaded rod 5.8 3/4**
Item number:	2081383 HAS 5.8 3/4"x8" (element) / 2334276 HIT-HY 200-R V3 (adhesive)
Specification text:	Hilti ∅ 3/4 HY 200-A V3 + threaded rod 5.8 with 6 in nominal embedment depth per ICC-ES ESR-4878 , Hammer drilled installation per MPII
Effective embedment depth:	h_{ef} = 6.000 in.
Material:	5.8
Evaluation Service Report:	ESR-4878
Issued I Valid:	11/1/2023 | 11/1/2024
Proof:	Design Method LRFD (AC58) Masonry + ACI 318-19
Stand-off installation:	e_b = 0.000 in. (no stand-off); t = 0.375 in.
Anchor plate^R :	l_x x l_y x t = 13.000 in. x 7.500 in. x 0.375 in.; (Recommended plate thickness: not calculated)
Profile:	no profile
Base material:	cracked Grout-filled CMU, f=1500, f = 1,500 psi, L x W x H: 16.000 in. x 8.000 in. x 8.000 in.;
	Solid Head Joint: no; open ended unit: no
Temp. short/long:	32 °F / 32 °F
	Joints: vertical: 0.375 in.; horizontal: 0.375 in.
Installation:	Top installation, Drill hole: Hammer drilled, Installation condition: Dry

^{R} - The anchor calculation is based on a rigid anchor plate assumption.

Geometry [in.]



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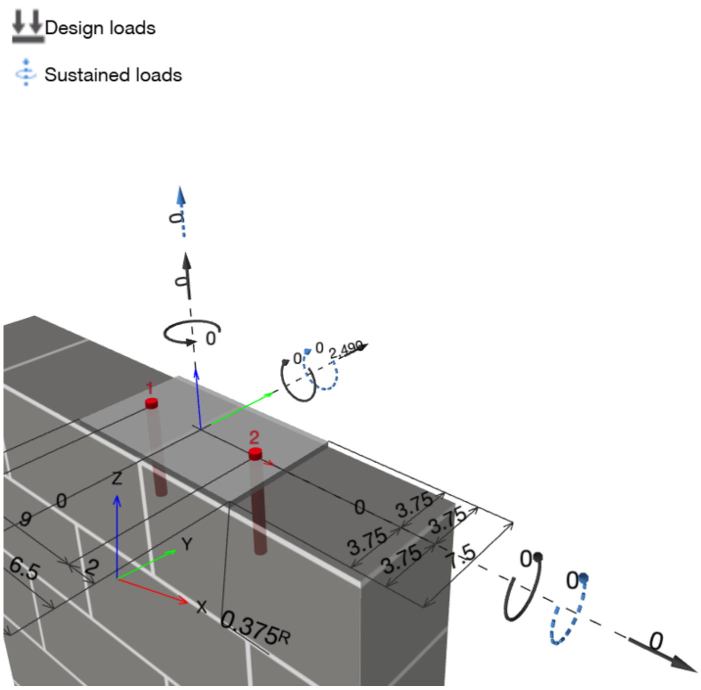
Geometry [in.] & Loading [lb, in.lb]



1.1 Design results

Case	Description	Forces [lb] / Moments [in.lb]	Seismic	Max. Util. Anchor [%]
1	Combination 1	$N = 0; V_x = 0; V_y = 2{,}490;$ $M_x = 0; M_y = 0; M_z = 0;$ $N_{sus} = 0; M_{x,sus} = 0; M_{y,sus} = 0;$	no	95

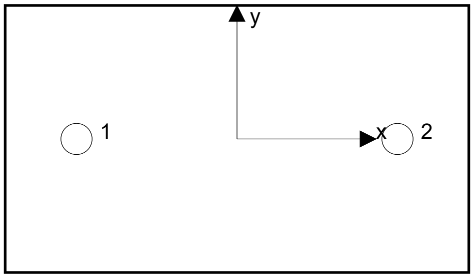
2 Load case/Resulting anchor forces

Anchor reactions [lb]

Tension force: (+Tension, -Compression)

Anchor	Tension force	Shear force	Shear force x	Shear force y
1	0	1,245	0	1,245
2	0	1,245	0	1,245

Max. concrete compressive strain: 0.00 [‰]
Max. concrete compressive stress: 0 [psi]
Resulting tension force in (x/y)=(-/-): 0 [lb]
Resulting compression force in (x/y)=(-/-): 0 [lb]



Anchor forces are calculated based on the assumption of a rigid anchor plate.

3 Shear load

	Load [lb]	Capacity [lb]	Utilization β_V [%]	Status
Steel	1,245	8,730	15	OK
Pryout bond	1,245	1,312	95	OK
Masonry crushing strength	1,245	4,141	31	OK
Masonry breakout in direction y+	1,245	1,334	94	OK

3.1 Steel strength

V_{sa} = ESR value refer to ICC-ES ESR-4878

$\phi\ V_{sa} \geq V_{ua}$ AC58 Table 3.2 + ACI 318-19 Table 17.5.2

V_{sa} [lb]	ϕ	$\phi\ V_{sa}$ [lb]	V_{ua} [lb]
14,549	0.600	8,730	1,245

3.2 Pryout strength (Bond strength controls)

V_{mpg} $=\min(k_{cp} \cdot N_{mag}, k_{cp} \cdot N_{mbg})$

$\phi\ V_{mpg} \geq V_{ua}$ AC58 Table 3.2 + ACI 318-19 Table 17.5.2

A_{Na} see ACI 318-19, Section 17.6.5.1, Fig. 17.6.5.1(b)

$A_{Na0} = (2\ c_{Na})^2$ ACI 318-19 Eq. (17.6.5.1.2a)

$c_{Na} = 10\ d_a \sqrt{\frac{\tau_{uncr,m}}{1100}}$ ACI 318-19 Eq. (17.6.5.1.2b)

$\psi_{ec,Na} = \left(\frac{1}{1 + \frac{e'_N}{c_{Na}}}\right) \leq 1.0$ ACI 318-19 Eq. (17.6.5.3.1)

$\psi_{ed,Na} = 0.7 + 0.3 \left(\frac{c_{a,min}}{c_{ac}}\right) \leq 1.0$ ACI 318-19 Eq. (17.6.5.4.1b)

$N_{ba,m} = \lambda_a \cdot \tau_{cr,m} \cdot \pi \cdot d_a \cdot h_{ef}$ ACI 318-19 Eq. (17.6.5.2.1)

Variables

k_{cp}	d_a [in.]	h_{ef} [in.]	$c_{a,min}$ [in.]	$\tau_{uncr,m}$ [psi]	$\tau_{cr,m}$ [psi]
2	0.750	6.000	3.688	883	173

α_{sat}	α_{top}	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	λ_a
1.000	0.330	0.000	0.000	1.000

Calculations

c_{Na} [in.]	A_{Na} [in.^{2}]	A_{Na0} [in.^{2}]	$\psi_{ed,Na}$	$\psi_{ec1,Na}$	$\psi_{ec2,Na}$
6.689	79.12	178.99	0.865	1.000	1.000

$\psi_{cp,Na}$	$N_{ba,m}$ [lb]
1.000	2,449

Results

V_{mpg} [lb]	ϕ	$\phi\ V_{mpg}$ [lb]	V_{ua} [lb]
1,874	0.700	1,312	1,245

3.3 Masonry crushing strength

$\phi\ V_{mc} \geq V_{ua}$ AC58 Table 3.2

$V_{mc} = 1750 \cdot \left(f'_m \cdot A_{se,V}\right)^{\frac{1}{4}}$ AC58 Eq. 3-1

Variables

f'_m [psi]	$A_{se,V}$ [in.^{2}]
1,500	0.33

Results

V_{mc} [lb]	ϕ	$\phi\ V_{mc}$ [lb]	V_{ua} [lb]
8,282	0.500	4,141	1,245

3.4 Masonry breakout strength y+

$V_{mbg} = \left(\frac{A_{Vm}}{A_{Vm0}}\right) \psi_{ec,V,m}\ \psi_{ed,V,m}\ \psi_{m,V}\ \psi_{h,V,m}\ \psi_{parallel,V}\ V_{b,m}$ ACI 318-19 Eq. (17.7.2.1b)

$\phi\ V_{mbg} \geq V_{ua}$ AC58 Table 3.2 + ACI 318-19 Table 17.5.2

A_{Vm} see ACI 318-19 , Section 17.7.2.1, Fig. R 17.7.2.1(b)

$A_{Vm0} = 4.5\ c_{a1}^2$ ACI 318-19 Eq. (17.7.2.1.3)

$\psi_{ec,V,m} = \left(\frac{1}{1 + \frac{2e'_v}{3c_{a1}}}\right) \leq 1.0$ ACI 318-19 Eq. (17.7.2.3.1)

$\psi_{ed,V,m} = 0.7 + 0.3\left(\frac{c_{a2}}{1.5c_{a1}}\right) \leq 1.0$ ACI 318-19 Eq. (17.7.2.4.1b)

$\psi_{h,V,m} = \sqrt{\frac{1.5c_{a1}}{h_a}} \geq 1.0$ ACI 318-19 Eq. (17.7.2.6.1)

$V_{b,m} = \left(7 \left(\frac{l_e}{d_a}\right)^{0.2} \sqrt{d_a}\right) \lambda_a \sqrt{f'_m}\ c_{a1}^{1.5}$ ACI 318-19 Eq. (17.7.2.2.1a)

Variables

l_e [in.]	d_a [in.]	c_{a1} [in.]	c_{a2} [in.]	A_{Vm} [in.^{2}]	A_{Vm0} [in.^{2}]	f'_m [psi]
6.000	0.750	3.812	3.688	53.79	65.41	1,500

Calculations

$\psi_{ed,V,m}$	$\psi_{parallel,V}$	$e_{c,V}$ [in.]	$\psi_{ec,V,m}$	$\psi_{m,V}$	$\psi_{h,V,m}$
0.893	1.000	0.000	1.000	1.000	1.000

Results

$V_{b,m}$ [lb]	ϕ	$\phi\ V_{mbg}$ [lb]	V_{ua} [lb]
2,595	0.700	1,334	1,245

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5 Installation data

Anchor plate, steel: ASTM A36; E = 29,000,001 psi; f_{yk} = 36,000 psi

Profile: no profile

Hole diameter in the fixture: d_f = 0.875 in.

Plate thickness (input): 0.375 in.

Recommended plate thickness: not calculated

Drilling method: Drilled in hammer mode

Cleaning: No cleaning of the drilled hole is required

Anchor type and diameter: HY 200-A V3 + threaded rod 5.8 3/4

Item number: 2081383 HAS 5.8 3/4"x8" (element) / 2334276 HIT-HY 200-R V3 (adhesive)

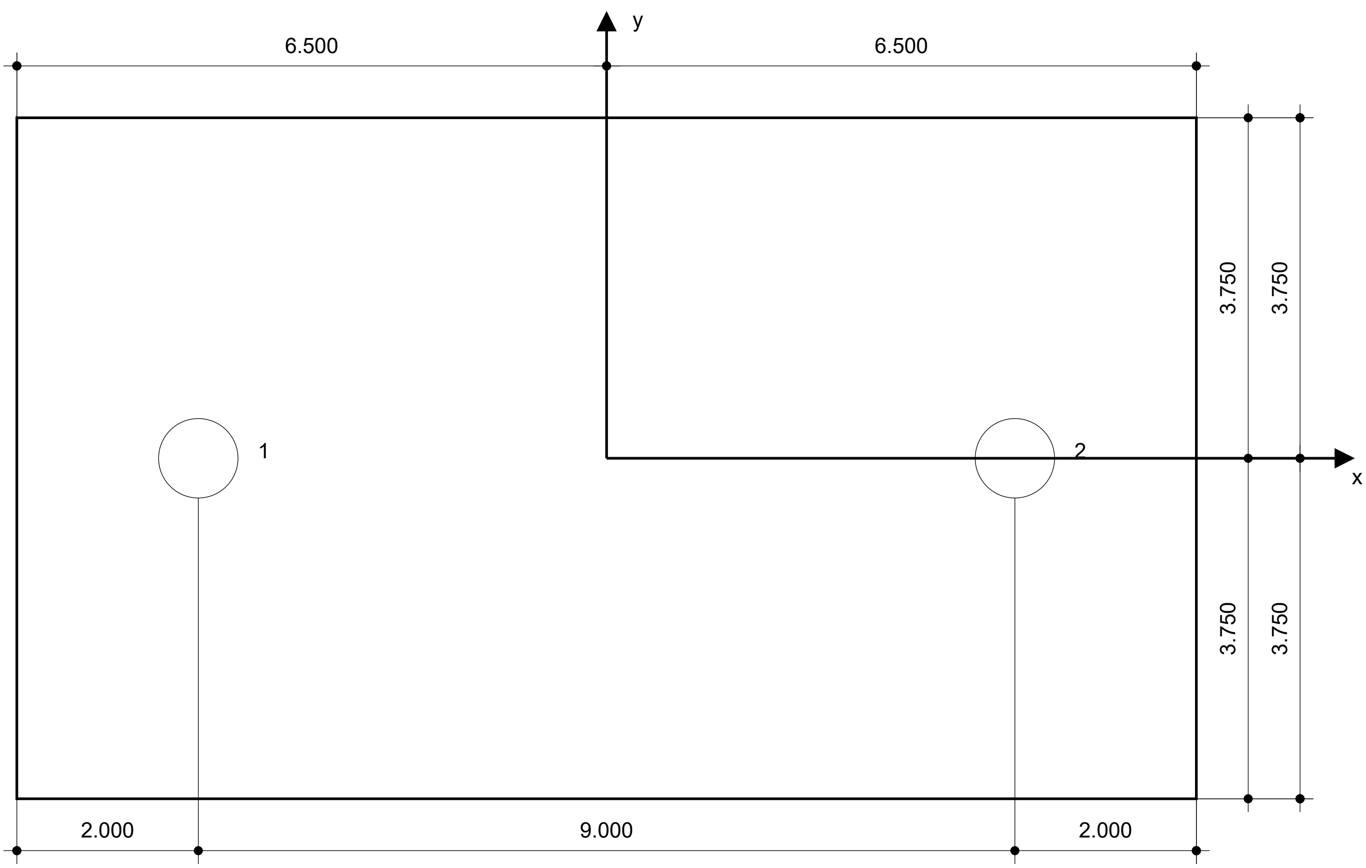
Maximum installation torque: 1,200 in.lb

Hole diameter in the base material: 0.875 in.

Hole depth in the base material: 6.000 in.

Minimum thickness of the base material: 152.000 in.

Hilti ∅ 3/4 HY 200-A V3 + threaded rod 5.8 with 6 in nominal embedment depth per ICC-ES ESR-4878 , Hammer drilled installation per MPII



Coordinates Anchor [in.]

Anchor	x	y	c_{-x}	c_{+x}	c_{-y}	c_{+y}
1	-4.500	0.000	19.500	19.500	3.812	3.812
2	4.500	0.000	28.500	10.500	3.812	3.812

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Fastening point:			

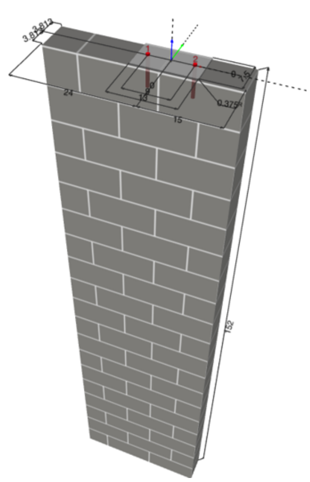
Specifier's comments:

1 Input data

Anchor type and diameter:	**HY 200-A V3 + threaded rod 5.8 3/4**
Item number:	2081383 HAS 5.8 3/4"x8" (element) / 2334276 HIT-HY 200-R V3 (adhesive)
Specification text:	Hilti ∅ 3/4 HY 200-A V3 + threaded rod 5.8 with 6 in nominal embedment depth per ICC-ES ESR-4878 , Hammer drilled installation per MPII
Effective embedment depth:	h_{ef} = 6.000 in.
Material:	5.8
Evaluation Service Report:	ESR-4878
Issued I Valid:	11/1/2023 | 11/1/2024
Proof:	Design Method LRFD (AC58) Masonry + ACI 318-19
Stand-off installation:	e_b = 0.000 in. (no stand-off); t = 0.375 in.
Anchor plate^R :	l_x x l_y x t = 13.000 in. x 7.500 in. x 0.375 in.; (Recommended plate thickness: not calculated)
Profile:	no profile
Base material:	cracked Grout-filled CMU, f=1500, f = 1,500 psi, L x W x H: 16.000 in. x 8.000 in. x 8.000 in.;
	Solid Head Joint: no; open ended unit: no
Temp. short/long:	32 °F / 32 °F
	Joints: vertical: 0.375 in.; horizontal: 0.375 in.
Installation:	Top installation, Drill hole: Hammer drilled, Installation condition: Dry

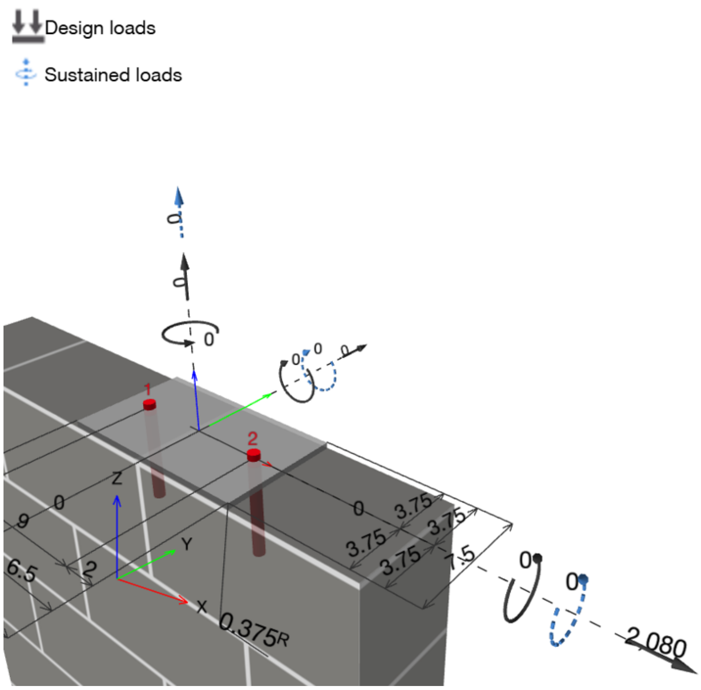
^{R} - The anchor calculation is based on a rigid anchor plate assumption.

Geometry [in.]



Company:		Page:	2
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	HSS TO CMU ANCHOR	Date:	2/7/2025
Fastening point:			

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



1.1 Design results

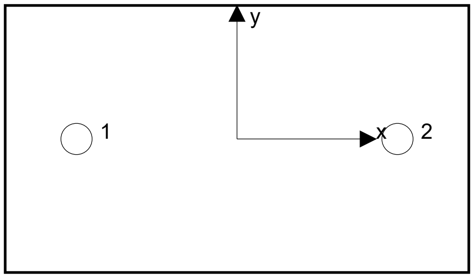
Case	Description	Forces [lb] / Moments [in.lb]	Seismic	Max. Util. Anchor [%]
1	Combination 1	$N = 0$; $V_x = 2{,}080$; $V_y = 0$; $M_x = 0$; $M_y = 0$; $M_z = 0$; $N_{sus} = 0$; $M_{x,sus} = 0$; $M_{y,sus} = 0$;	no	97

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	0	1,040	1,040	0
2	0	1,040	1,040	0



Max. concrete compressive strain: 0.00 [‰]
Max. concrete compressive stress: 0 [psi]
Resulting tension force in (x/y)=(-/-): 0 [lb]
Resulting compression force in (x/y)=(-/-): 0 [lb]

Anchor forces are calculated based on the assumption of a rigid anchor plate.

Company:		Page:	4
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	HSS TO CMU ANCHOR	Date:	2/7/2025
Fastening point:			

3 Shear load

	Load [lb]	Capacity [lb]	Utilization β_V [%]	Status
Steel	1,040	8,730	12	OK
Pryout bond	1,040	1,312	80	OK
Masonry crushing strength	1,040	4,141	26	OK
Masonry breakout in direction x+	1,040	1,080	97	OK

3.1 Steel strength

V_{sa} = ESR value refer to ICC-ES ESR-4878

$\phi\ V_{sa} \geq V_{ua}$ AC58 Table 3.2 + ACI 318-19 Table 17.5.2

V_{sa} [lb]	ϕ	$\phi\ V_{sa}$ [lb]	V_{ua} [lb]
14,549	0.600	8,730	1,040

3.2 Pryout strength (Bond strength controls)

$V_{mpg} = \min(k_{cp} \cdot N_{mag}, k_{cp} \cdot N_{mbg})$

$\phi\ V_{mpg} \geq V_{ua}$ AC58 Table 3.2 + ACI 318-19 Table 17.5.2

A_{Na} see ACI 318-19, Section 17.6.5.1, Fig. 17.6.5.1(b)

$A_{Na0} = (2\ c_{Na})^2$ ACI 318-19 Eq. (17.6.5.1.2a)

$c_{Na} = 10\ d_a \sqrt{\frac{\tau_{uncr,m}}{1100}}$ ACI 318-19 Eq. (17.6.5.1.2b)

$\psi_{ec,Na} = \left(\frac{1}{1 + \frac{e_N}{c_{Na}}}\right) \leq 1.0$ ACI 318-19 Eq. (17.6.5.3.1)

$\psi_{ed,Na} = 0.7 + 0.3 \left(\frac{c_{a,min}}{c_{ac}}\right) \leq 1.0$ ACI 318-19 Eq. (17.6.5.4.1b)

$N_{ba,m} = \lambda_a \cdot \tau_{cr,m} \cdot \pi \cdot d_a \cdot h_{ef}$ ACI 318-19 Eq. (17.6.5.2.1)

Variables

k_{cp}	d_a [in.]	h_{ef} [in.]	$c_{a,min}$ [in.]	$\tau_{uncr,m}$ [psi]	$\tau_{cr,m}$ [psi]
2	0.750	6.000	3.688	883	173

α_{sat}	α_{top}	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	λ_a
1.000	0.330	0.000	0.000	1.000

Calculations

c_{Na} [in.]	A_{Na} [in.^{2}]	A_{Na0} [in.^{2}]	$\psi_{ed,Na}$	$\psi_{ec1,Na}$	$\psi_{ec2,Na}$
6.689	79.12	178.99	0.865	1.000	1.000

$\psi_{cp,Na}$	$N_{ba,m}$ [lb]
1.000	2,449

Results

V_{mpg} [lb]	ϕ	$\phi\ V_{mpg}$ [lb]	V_{ua} [lb]
1,874	0.700	1,312	1,040

3.3 Masonry crushing strength

$\phi\ V_{mc} \geq V_{ua}$ AC58 Table 3.2

$V_{mc} = 1750 \cdot (f'_m \cdot A_{se,V})^{\frac{1}{4}}$ AC58 Eq. 3-1

Variables

f'_m [psi]	$A_{se,V}$ [in.^{2}]
1,500	0.33

Results

V_{mc} [lb]	ϕ	$\phi\ V_{mc}$ [lb]	V_{ua} [lb]
8,282	0.500	4,141	1,040

3.4 Masonry breakout strength x+

$V_{mbg} = \left(\frac{A_{Vm}}{A_{Vm0}}\right) \psi_{ec,V,m}\ \psi_{ed,V,m}\ \psi_{m,V}\ \psi_{h,V,m}\ \psi_{parallel,V}\ V_{b,m}$ ACI 318-19 Eq. (17.7.2.1b)

$\phi\ V_{mbg} \geq V_{ua}$ AC58 Table 3.2 + ACI 318-19 Table 17.5.2

A_{Vm} see ACI 318-19 , Section 17.7.2.1, Fig. R 17.7.2.1(b)

$A_{Vm0} = 4.5\ c_{a1}^2$ ACI 318-19 Eq. (17.7.2.1.3)

$\psi_{ec,V,m} = \left(\frac{1}{1 + \frac{2e'_v}{3c_{a1}}}\right) \leq 1.0$ ACI 318-19 Eq. (17.7.2.3.1)

$\psi_{ed,V,m} = 0.7 + 0.3\left(\frac{c_{a2}}{1.5c_{a1}}\right) \leq 1.0$ ACI 318-19 Eq. (17.7.2.4.1b)

$\psi_{h,V,m} = \sqrt{\frac{1.5c_{a1}}{h_a}} \geq 1.0$ ACI 318-19 Eq. (17.7.2.6.1)

$V_{b,m} = \left(7\left(\frac{l_e}{d_a}\right)^{0.2} \sqrt{d_a}\right) \lambda_a \sqrt{f'_m}\ c_{a1}^{1.5}$ ACI 318-19 Eq. (17.7.2.2.1a)

Variables

l_e [in.]	d_a [in.]	c_{a1} [in.]	c_{a2} [in.]	A_{Vm} [in.^{2}]	A_{Vm0} [in.^{2}]	f'_m [psi]
6.000	0.750	3.688	3.812	42.18	61.19	1,500

Calculations

$\psi_{ed,V,m}$	$\psi_{parallel,V}$	$e_{c,V}$ [in.]	$\psi_{ec,V,m}$	$\psi_{m,V}$	$\psi_{h,V,m}$
0.907	1.000	0.000	1.000	1.000	1.000

Results

$V_{b,m}$ [lb]	ϕ	$\phi\ V_{mbg}$ [lb]	V_{ua} [lb]
2,468	0.700	1,080	1,040

4 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!
- The equations presented in this report are based on imperial units. When inputs are displayed in metric units, the user should be aware that the equations remain in their imperial format.
- Refer to the manufacturer's product literature for cleaning and installation instructions.
- For additional information about ACI 318 strength design provisions, please go to https://submittals.us.hilti.com/PROFISAnchorDesignGuide/
- The min. sizes of the bricks, the masonry compressive strength, the type / strength of the mortar and the grout (in case of fully grouted CMU walls) has to fulfill the requirements given in the relevant ESR-approval or in the PTG.
- Only the local load transfer from the anchor(s) to the wall is considered, a further load transfer in the wall is not covered by PROFIS!
- Wall is assumed as being perfectly aligned vertically – checking required(!): Noncompliance can lead to significantly different distribution of forces and higher tension loads than those calculated by PROFIS. Masonry wall must not have any damages (neither visible nor not visible)! While installation, the positioning of the anchors needs to be maintained as in the design phase i.e. either relative to the brick or relative to the mortar joints.
- The effect of the joints on the compressive stress distribution on the plate / bricks was not taken into consideration.
- If no significant resistance is felt over the entire depth of the hole when drilling (e.g. in unfilled butt joints), the anchor should not be set at this position or the area should be assessed and reinforced. Hilti recommends the anchoring in masonry always with sieve sleeve. Anchors can only be installed without sieve sleeves in solid bricks when it is guaranteed that it has not any hole or void.
- The accessories and installation remarks listed on this report are for the information of the user only. In any case, the instructions for use provided with the product have to be followed to ensure a proper installation.
- The compliance with current standards (e.g. 2018, 2015, 2012, 2009 and 2006 IBC) is the responsibility of the user.
- Drilling method (hammer, rotary) to be in accordance with the approval!
- Masonry needs to be built in a regular way in accordance with state-of the art guidelines!

Fastening meets the design criteria!

5 Installation data

Anchor plate, steel: ASTM A36; E = 29,000,001 psi; f_{yk} = 36,000 psi

Profile: no profile

Hole diameter in the fixture: d_f = 0.875 in.

Plate thickness (input): 0.375 in.

Recommended plate thickness: not calculated

Drilling method: Drilled in hammer mode

Cleaning: No cleaning of the drilled hole is required

Anchor type and diameter: HY 200-A V3 + threaded rod 5.8 3/4

Item number: 2081383 HAS 5.8 3/4"x8" (element) / 2334276 HIT-HY 200-R V3 (adhesive)

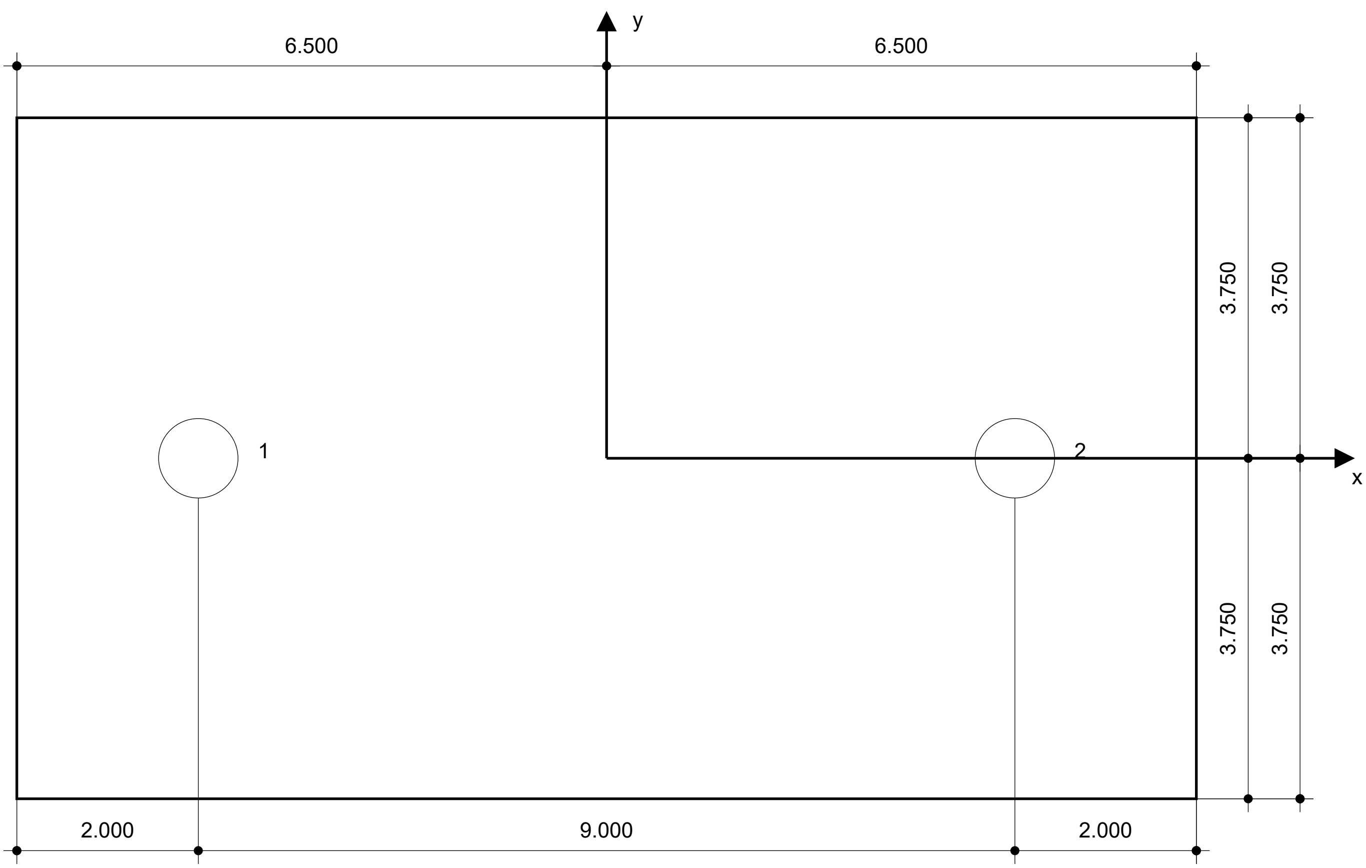
Maximum installation torque: 1,200 in.lb

Hole diameter in the base material: 0.875 in.

Hole depth in the base material: 6.000 in.

Minimum thickness of the base material: 152.000 in.

Hilti ∅ 3/4 HY 200-A V3 + threaded rod 5.8 with 6 in nominal embedment depth per ICC-ES ESR-4878 , Hammer drilled installation per MPII



Coordinates Anchor [in.]

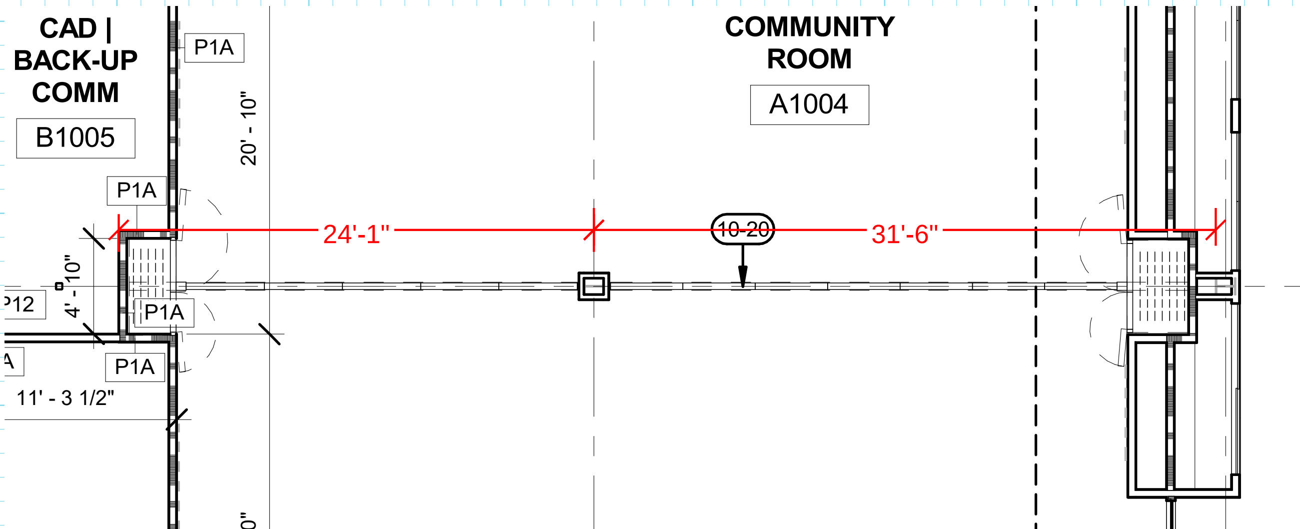
Anchor	x	y	c_{-x}	c_{+x}	c_{-y}	c_{+y}
1	-4.500	0.000	19.500	19.500	3.812	3.812
2	4.500	0.000	28.500	10.500	3.812	3.812

6 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.

Operable partition

Panel height = 11ft (for weight purposes)
Panel weight = 9PSF

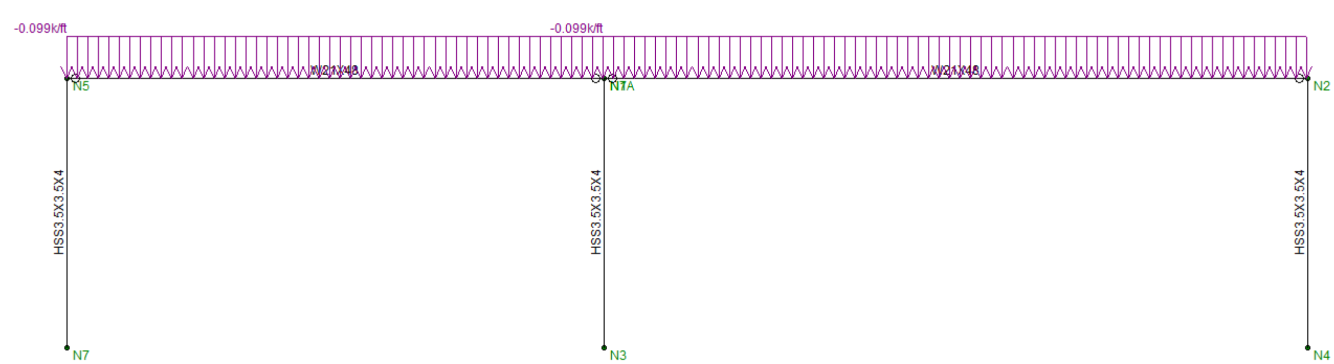


ASTM E557 deflection requirements:

5.5 Structural Support for Operable Partitions — The weight of the operable partition, in addition to all dead loads, should be taken into consideration when designing the supporting member. Deflection under maximum anticipated load should be no more than $^1/_8$ in. (3.2 mm) per 12 ft (3.658 m) of opening width. If greater deflection is anticipated, either a structural member independent of the roof structure should be installed to support the operable partition, or an operable partition with bottom seals designed to accommodate the larger deflection should be specified.

Max deflection 2-span long: (1/8"/12)*31.5ft = 0.328in
Max deflection 2-span short: (1/8"/12)*24ft = 0.25in

$F_p = 0.4 a_p S_{DS} W_p / (R_p/I_p) * (1 + 2z/h)$
Interior nonstructural walls and partitions: All other walls and partitions (footnote c does not apply)
$a_p = 1$, $R_p = 2.5$, $\Omega_O = 2$, $I_p = 1.5$ (RC IV), $S_{DS} = 1.006$
z = 12'-0", h = 41'-0", $z/h \approx 0.29$
$F_p = 0.38 W_p < 0.3 S_{DS} I_p W_p$ lower limit
Fp = 0.453Wp



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By
Date
Job#
Sht. 03.1 of

Hot Rolled Steel Section Sets

	Label	Shape	Type	Design List	Material	Design Rules	A [in2]	I (90,270) [i...	I (0,180) [in4]
1	HR1A	W10X33	Beam	None	A992	Typical	9.71	36.6	171

Hot Rolled Steel Design Parameters

	Label	Shape	Length...	Lb-out[ft]	Lb-in[ft]	Lcomp top[ft]	Lcomp bo...	L-tor...	K-out	K-in	Cb	Chan...	a[ft]	Funct...
1	M1	W21X48	31.5			Lb out						N/A	N/A	Lateral
2	M2	HSS3.5X...	12									N/A	N/A	Lateral
3	M3	HSS3.5X...	12									N/A	N/A	Lateral
4	M4	W21X48	24			Lb out						N/A	N/A	Lateral
5	M5	HSS3.5X...	12									N/A	N/A	Lateral

Member Distributed Loads (BLC 2 : L (dist))

	Member Label	Direction	Start Magnitude[k/ft,F,ksf]	End Magnitu...	Start Location[ft,%]	End Location[ft,%]	Inactive
1	M1	Y	-0.099	-0.099	0	0	
2	M4	Y	-0.099	-0.099	0	0	

Member Point Loads (BLC 3 : L (point - mid))

	Member Label	Direction	Magnitude[k,k-ft]	Location[ft,%]	Inactive
1	M1	Y	-5	%50	
2	M4	Y	-4.2	%50	

Member Point Loads (BLC 4 : L (point - end))

	Member Label	Direction	Magnitude[k,k-ft]	Location[ft,%]	Inactive
1	M1	Y	-2.92	31	
2	M4	Y	-2.27	1	

Envelope Node Reactions

	Node Label		X [k]	LC	Y [k]	LC	Moment [k-ft]	LC
1	N3	max	0	7	9.111	7	0	8
2		min	0	8	1.459	1	0	1
3	N2	max	0	8	0	8	0	8
4		min	0	7	0	1	0	1
5	N4	max	0	7	5.658	8	0	8
6		min	0	8	0.883	1	0	1
7	N7	max	0	7	4.325	8	0	8
8		min	0	8	0.704	1	0	1
9	Totals:	max	0	6	18.375	7		
10		min	0	8	3.046	1		

Node Displacements

	LC	Node Label	X [in]	Y [in]	Rotation [rad]
1	1	N1	0	-0.003	0
2	1	N2	0	-0.002	0
3	1	N3	0	0	0

Node Displacements (Continued)

	LC	Node Label	X [in]	Y [in]	Rotation [rad]
4	1	N4	0	0	0
5	1	N5	0	-0.001	0
6	1	N7	0	0	0
7	1	N7A	0	0	0
8	2	N1	0	-0.009	0
9	2	N2	0	-0.005	0
10	2	N3	0	0	0
11	2	N4	0	0	0
12	2	N5	0	-0.004	0
13	2	N7	0	0	0
14	2	N7A	0	0	0
15	3	N1	0	-0.013	0
16	3	N2	0	-0.007	0
17	3	N3	0	0	0
18	3	N4	0	0	0
19	3	N5	0	-0.006	0
20	3	N7	0	0	0
21	3	N7A	0	0	0
22	4	N1	0	-0.003	0
23	4	N2	0	-0.008	0
24	4	N3	0	0	0
25	4	N4	0	0	0
26	4	N5	0	-0.006	0
27	4	N7	0	0	0
28	4	N7A	0	0	0
29	5	N1	0	-0.004	0
30	5	N2	0	-0.002	0
31	5	N3	0	0	0
32	5	N4	0	0	0
33	5	N5	0	-0.002	0
34	5	N7	0	0	0
35	5	N7A	0	0	0
36	6	N1	0	-0.013	0
37	6	N2	0	-0.007	0
38	6	N3	0	0	0
39	6	N4	0	0	0
40	6	N5	0	-0.006	0
41	6	N7	0	0	0
42	6	N7A	0	0	0
43	7	N1	0	-0.019	0
44	7	N2	0	-0.011	0
45	7	N3	0	0	0
46	7	N4	0	0	0
47	7	N5	0	-0.009	0
48	7	N7	0	0	0
49	7	N7A	0	0	0
50	8	N1	0	-0.004	0
51	8	N2	0	-0.012	0
52	8	N3	0	0	0
53	8	N4	0	0	0
54	8	N5	0	-0.009	0
55	8	N7	0	0	0

Node Displacements (Continued)

	LC	Node Label	X [in]	Y [in]	Rotation [rad]
56	8	N7A	0	0	0

EMBEDDED POST/POLE FOOTING

Purpose Statement: The purpose of this calculation is to design an embedded post/pole footing per IBC 1807.3 that will resist both axial and lateral loads.

Reference: 2021 IBC

Input Result Check/Confirm

Required Depth of Embedment - Per. IBC 1807.3 :

$d = 5\ ft$

Depth of embedment in earth in feet but not over 12 ft for purpose of computing lateral pressure

$b = 2\ ft$

Diameter of round post or footing or diagonal dimension of square post or footing, feet

$h = 30\ ft$

Distance in feet from ground surface to point of application of load "P"

$P_L = 340\ lb$ Applied lateral force in pounds (Includes 5'x8' flag drag force and 30' tall pole)

$P_v = 500\ lb$ Applied downward vertical force in pounds

$S = 150\ \frac{psf}{ft}$

Allowable lateral soil-bearing pressure as set forth in IBC Table 1806.2

Lateral bearing pressure values may be doubled for structures which can tolerate 0.5" of lateral movement at the ground surface for short-term loads per 1806.3.4

Yes $f_1 = 2 = 2$

Per IBC 1806.1, lateral and vertical bearing pressure values may be increased by 1/3 when used with alternative basic load combinations of IBC 1605.3.2 that include wind or earthquake loads.

Yes $f_2 = 1.33 = 1.33$

Allowable vertical soil-bearing pressure as set forth in IBC Table 1806.2:

$B = 5000\ psf$

Allowable lateral soil-bearing pressure as set forth in IBC Table 1806.2, based on a depth of one-third the depth of embedment in pounds per square foot

$$S_1 = S \cdot \left(\frac{d}{3}\right) \cdot f_1 \cdot f_2 = 665 \text{ psf}$$

Allowable lateral soil-bearing pressure as set forth in IBC Table 1806.2, based on a depth equal to the depth of embedment in pounds per square foot:

$$S_3 = S \cdot d \cdot f_1 \cdot f_2 = 1995 \text{ psf}$$

$$B' = B \cdot f_2 = 6650 \text{ psf}$$

Adjusted allowable vertical soil-bearing pressure

Nonconstrained - IBC 1807.3.2.1

$$A1 = \frac{2.34 \cdot P_L}{S_1 \cdot b} = 0.598 \text{ ft}$$

$$d_{res1} = 0.5 \cdot A1 \cdot \left(1 + \left(1 + \frac{4.36 \cdot h}{A1}\right)^{\left(\frac{1}{2}\right)}\right) = 4.73 \text{ ft}$$

Depth of Embedment (IBC Equation 18-1)

$$\begin{cases} \text{"OK"} & \text{if } d_{res1} \le d \\ \text{"Not Good"} & \text{otherwise} \end{cases} = \text{OK}$$

Constrained - IBC 1807.3.2.2

$$d_{res2} = \sqrt{\frac{4.25 \cdot P_L \cdot h}{S_3 \cdot b}} = 3.30 \text{ ft}$$

Depth of Embedment (IBC Equation 18-2)

$$\begin{cases} \text{"OK"} & \text{if } d_{res2} \le d \\ \text{"Not Good"} & \text{otherwise} \end{cases} = \text{OK}$$

Vertical Load Capacity Check

End Bearing

$$W = P_v + 150\ \text{pcf} \cdot \left(\frac{\pi \cdot b^2}{4}\right) \cdot d = 2856.19\ \text{lb}$$

Total weight

$$B_{cap} = B' \cdot \left(\frac{\pi \cdot b^2}{4}\right) = 20891.59\ \text{lb}$$

End bearing capacity

$$\left| \begin{array}{ll} \text{"OK"} & \text{if } W \le B_{cap} \\ \text{"Not Good"} & \text{otherwise} \end{array} \right. = \text{OK}$$