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41987
STRUCTURAL ENGINEER
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6/28/23

STRUCTURAL CALCULATIONS

To
City of Puyallup

Project
Fortress Puyallup
240 15th Street SE
Puyallup, WA 98372

Submitted
June 28 , 2023

Project Number
2220290.00

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STRUCTURAL CALCULATIONS NARRATIVE

The following calculations are for Fortress Puyallup project located in Puyallup, WA. The building is comprised of a one-story concrete tilt up panel structure with wood structural panels and steel open web joists. The lateral system consists of special reinforced concrete shear walls and a flexible hybrid panelized wood roof diaphragm. There is a provision for a future light frame mezzanine in the southeast and southwest corners of the building.

The geotechnical engineer noted that the site would be prone to liquefaction from a seismic event, necessitating the need for foundation ties connecting the spread footings together. However, the differential settlement values stated in the report do not exceed the threshold per ASCE 7-16 requiring the explicit design and detailing of the foundations and connections to accommodate the effects of liquefaction.

The building includes a skewed wall in the northeast corner of the building. These panels are not considered as part of the seismic force resisting system (SFRS) to avoid provisions due to horizontal irregularities per ASCE 7-16. The building also includes large canopies on the east and west side of the building and a blade wall architectural feature on the northeast corner of the building.

The building was designed according to the 2018 International Building Code (IBC) with Washington State Amendments and analyzed with the equivalent lateral force procedure per ASCE 7-16. However, with the anticipated adoption of the 2021 IBC, the tilt panels and foundations were designed according to ACI 318-19 ahead of the building code change.

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6/23/23

01 LOADING

Hazards by Location

Search Information

Address:	240 15th St SE, Puyallup, WA 98372, USA
Coordinates:	47.1898424, -122.2760513
Elevation:	54 ft
Timestamp:	2023-05-16T00:01:30.713Z
Hazard Type:	Seismic
Reference Document:	ASCE7-16
Risk Category:	II
Site Class:	D

Basic Parameters

Name	Value	Description
S_S	1.264	MCE_R ground motion (period=0.2s)
S_1	0.435	MCE_R ground motion (period=1.0s)
S_{MS}	1.264	Site-modified spectral acceleration value
S_{M1}	* null	Site-modified spectral acceleration value
S_{DS}	0.843	Numeric seismic design value at 0.2s SA
S_{D1}	* null	Numeric seismic design value at 1.0s SA

* See Section 11.4.8

▾Additional Information

Name	Value	Description
SDC	* null	Seismic design category
F_a	1	Site amplification factor at 0.2
F_v	* null	Site amplification factor at 1.0
CR_S	0.914	Coefficient of risk (0.2s)
CR_1	0.898	Coefficient of risk (1.0s)
PGA	0.5	MCE_G peak ground acceleration
F_{PGA}	1.1	Site amplification factor at PGA
PGA_M	0.55	Site modified peak ground acceleration
T_L	6	Long-period transition period (s)
SsRT	1.264	Probabilistic risk-targeted ground motion (0.2s)
SsUH	1.383	Factored uniform-hazard spectral acceleration (2% probability of exceedance in 50 years)
SsD	1.5	Factored deterministic acceleration value (0.2s)
S1RT	0.435	Probabilistic risk-targeted ground motion (1.0s)
S1UH	0.484	Factored uniform-hazard spectral acceleration (2% probability of exceedance in 50 years)
S1D	0.6	Factored deterministic acceleration value (1.0s)
PGAd	0.5	Factored deterministic acceleration value (PGA)

* See Section 11.4.8

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

Please note that the ATC Hazards by Location website will not be updated to support ASCE 7-22. Find out why.

Disclaimer

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

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This is a beta release of the new ATC Hazards by Location website. Please contact us with feedback.

The ATC Hazards by Location website will not be updated to support ASCE 7-22. Find out why.

ATC Hazards by Location

Search Information

Address:	240 15th St SE, Puyallup, WA 98372, USA
Coordinates:	47.1898424, -122.2760513
Elevation:	54 ft
Timestamp:	2023-04-28T16:38:55.571Z
Hazard Type:	Wind

ASCE 7-16

MRI 10-Year	67 mph
MRI 25-Year	73 mph
MRI 50-Year	78 mph
MRI 100-Year	82 mph
Risk Category I	92 mph
Risk Category II	97 mph
Risk Category III	104 mph
Risk Category IV	108 mph

ASCE 7-10

MRI 10-Year	72 mph
MRI 25-Year	79 mph
MRI 50-Year	85 mph
MRI 100-Year	91 mph
Risk Category I	100 mph
Risk Category II	110 mph
Risk Category III-IV	115 mph

ASCE 7-05

ASCE 7-05 Wind Speed	85 mph

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Please note that the ATC Hazards by Location website will not be updated to support ASCE 7-22. Find out why.

Disclaimer

Hazard loads are interpolated from data provided in ASCE 7 and rounded up to the nearest whole integer. Per ASCE 7, islands and coastal areas outside the last contour should use the last wind speed contour of the coastal area – in some cases, this website will extrapolate past the last wind speed contour and therefore, provide a wind speed that is slightly higher. NOTE: For queries near wind-borne debris region boundaries, the resulting determination is sensitive to rounding which may affect whether or not it is considered to be within a wind-borne debris region.

Mountainous terrain, gorges, ocean promontories, and special wind regions shall be examined for unusual wind conditions.

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1.0 PROJECT DESCRIPTION

The proposed project is an industrial development consisting of a warehouse-style building and associated paved access, parking, and utility improvements. A conceptual site plan by Mackenzie, dated September 27, 2021, shows a 131,250 square-foot building in the central portion of the site. Truck and trailer parking is shown on the northern and western sides of the building, respectively. Passenger vehicle parking is shown on the eastern side of the building. Building plans are currently not available; however, we expect the building will be constructed using precast concrete tilt-up perimeter wall panels with interior columns spaced at 30 to 50 feet. Building floors will be constructed at grade with dock high access on the northern side of the building. Structural loading is expected to be light to moderate, with isolated columns carrying loads of 50 to 100 kips, and bearing walls carrying 4 to 8 kips per foot.

The recommendations in this report are based on our understanding of the design features outlined above. We should review design drawings as they become available to verify that our recommendations have been properly interpreted and to supplement them, if required.

Geotechnical Report, Page 1

The results of our analysis indicate the site is a seismic hazard area with respect to soil liquefaction. Soil liquefaction could occur during the design earthquake event resulting in total settlements ranging between about 4.5 and 7 inches with about one-half of this settlement likely being differential in nature. In our opinion, this amount of settlement has the potential to structurally impair the building. The structural engineer should review the estimated settlement to determine if additional mitigation measures are necessary. Additionally, cosmetic damage to the structure in the form of misaligned doors and windows, cracking, and floor settlement could occur. Some utility connections may also be impacted. If the owner is not willing to accept the risk of building damage requiring repair should liquefaction-induced settlements occur, foundations should be supported on ground improved using stone columns designed to mitigate soil liquefaction settlements below the building foundations.

Geotechnical Report, Page 4

We recommend designing foundations for a net allowable bearing capacity of 2,500 pounds per square foot (psf). For short-term loads, such as wind and seismic, a one-third increase in this allowable capacity can be used. With the expected building loads and this bearing stress applied, in general, total and differential settlements should not exceed 0.5 inches for perimeter foundations and 1 inch for interior column supports.

Geotechnical Report, Page 9

EXCEPTION: Where the geotechnical investigation report indicates that the differential settlement over a defined length, *L*, does not exceed the differential settlement threshold specified in Table 12.13-3, explicit design beyond the requirements of Section 12.13.9.2.1 to accommodate differential settlements is not required.

Table 12.13-3 Differential Settlement Threshold

	Risk Category		
Structure Type	**I or II**	**III**	**IV**
Single-story structures with concrete or masonry wall systems	0.0075*L*	0.005*L*	0.002*L*
Other single-story structures	0.015*L*	0.010*L*	0.002*L*
Multistory structures with concrete or masonry wall systems	0.005*L*	0.003*L*	0.002*L*
Other multistory structures	0.010*L*	0.006*L*	0.002*L*

per Geotechnical Report:

3.5" differential settlement + 1" from gravity over 50':

(3.5" + 1")/(12 IN/FT)(50') = 0.0075

Limit in ASCE 7-16 is not exceeded, therefore design to accomodate differential settlements is not required.

Gravity and Lateral Loads for Tilt Construction

Purpose Statement: The purpose of this calculation is to cover the gravity and lateral loads for a full concrete tilt building. Calculations include: mass take-off, wind load, seismic load, diaphragm check, chord reinforcement, wall connections, and diaphragm nailing.

Referenced Standards: IBC 2021
ASCE 7-16

LOADING

Roof Live Load

$L_r := 20 \cdot psf$

Snow Load

$SL := 19 \cdot psf$

Roof Dead Loads - Wood Roof

ROOFING	$R := 0.7 \cdot psf$
INSUL	$I := 1.5 \cdot psf$
1/2" PLY	$PL := 1.6 \cdot psf$
2X6 SUBPURLIN	$SP := 1.1 \cdot psf$
JOISTS	$J := 2.5 \cdot psf$
GIRDER	$G := 1.5 \cdot psf$
MECH/ELEC	$M := 2.5 \cdot psf$
SPRINK	$S := 1.5\ psf$
SOLAR	$PV := 4.0 \cdot psf$
MISC	$MISC := 2.1 \cdot psf$

$DL := R + I + PL + SP + J + G + M + S + MISC = 15\ psf$ Total Roof Dead Load

$DL_{PV} := R + I + PL + SP + J + G + M + S + PV + MISC = 19\ psf$ Total Roof Dead Load w/ Solar

$$DL_{sei} := R + I + PL + SP + J + G + \frac{M}{2} + \frac{S}{2} + PV = 14.9\ psf$$ Roof Seismic Dead Load

$DL_{up\_joist} := R + I + PL + SP + J = 7.4\ psf$ DL considered for Uplift @ Joists

$DL_{up\_girder} := R + I + PL + SP + J + G = 8.9\ psf$ DL considered for Uplift @ Girders

Mezzanine Loads:

FLOORING	$F := 1.5\ psf$	Wood Structural Panels
TOPPING	$I := 13\ psf$	Gypcrete Topping 1 1/2" max
PARTITION WALLS	$PW := 1.5\ psf$	5/8" GWP
FRAMING	$FR := 1.8\ psf$	Light Gauge Metal Framing
CARPET	$C := 1.5 \cdot psf$	Carpet
CEILING	$CE := 2 \cdot psf$	Suspended Acoustical Ceiling
MECH/ELEC	$M := 1.5 \cdot psf$	
SPRINK	$S := 2 \cdot psf$	
MISC	$MISC := 1 \cdot psf$	

$DL := F + I + PW + FR + C + CE + M + MISC + S = 25.8\ psf$ Total Mezzanine Dead Load

$DL_{sei\_mezz} := F + I + PW + FR + C + CE + \frac{M}{2} + MISC + \frac{S}{2} = 24.1\ psf$ Total Mezzanine Seismic Dead Load

$LL := 50\ psf + 15\ psf = 65\ psf$ (Office and Partition)

$A_{mezz} := 53\ ft \cdot 59\ ft = 3127\ ft^2$ Area of Mezzanine

$W_{mezz} := A_{mezz} \cdot \left(DL_{sei\_mezz} + 10\ psf\right) = 106.5\ k$ Seismic Weight of Mezzanine

Snow Drift Geometry and Loading

Purpose Statement: Snow drift calculations for Lower Roofs, Adjacent Structures, Rooftop Projections and Parapets. Calculates snow drift only. Ground snow load and minimum roof snow load to be determined outside of this program.

Referenced Standards: 2018 IBC
ASCE 7-16

Basic Input

Basic ground snow load for use with snow drift calculations

$p_g := 20\ psf$

Note that this is not the same as the minimum roof snow load. The minimum roof snow load is calculated outside of this program

Snow exposure factor as defined in *ASCE 7-16, Table 7.3-1*

$Surface\_Roughness\_Category :=$ B — Terrain (exposure) category as defined in *ASCE 7-16, Ch. 26.7*

$Exposure\_of\_Roof :=$ Partially Exposed

Protected Area

$C_e = 1$

Thermal factor as defined in *ASCE 7-16, Table 7.3-2*

$C_t :=$ All structures except as indicated below — $C_t = 1$

Importance factor *ASCE 7-16, Table 1.5-2*

$I_s :=$ Occupancy category II — $I_s = 1$

Sloped roof factor (If applicable) *ASCE 7-16, Figure 7.4-1*

$C_s := 1.0$

Calculated flat roof snow load *(ASCE 7-16, Eq. 7.3-1)*

$p_f := 0.7 \cdot C_e \cdot C_t \cdot I_s \cdot p_g$ — $p_f = 14\ psf$

Balanced roof snow load *(ASCE 7-16, Eq. 7.4-1)*

$p_s := C_s \cdot p_f$ — $p_s = 14\ psf$

Density of snow *(ASCE 7-16, Eq. 7.7-1)*

$\gamma := min\left(30 \cdot pcf, \frac{0.13}{ft} \cdot p_g + 14.0 \cdot pcf\right)$ — $\gamma = 16.6\ pcf$

Height of calculated minimum snow load *ASCE 7-16, 7.7.1*

$h_b := \frac{p_s}{\gamma}$ — $h_b = 0.84\ ft$

Snow Drift Geometry and Loading

DRIFT LOAD FOR PARAPET WALLS - WEST SIDE

(ASCE 7-16 Section 7.8)

$l_u := 550 \cdot ft$ — Horizontal length of roof upwind of projection or parapet wall

$h_p := 2.5 \cdot ft$ — Height of projection or parapet wall 40.5' - (39.25'+36.75')/2

Minimum length of roof upwind

$l_u := \text{if}\left(l_u \geq 20 \cdot ft, l_u, 20\ ft\right)$ $\qquad l_u = 550\ ft$

Unadjusted drift height

$$h'_d := \frac{3}{4} \cdot \sqrt{I_s} \cdot \left(0.43 \cdot \sqrt[3]{\frac{l_u}{ft}} \cdot \sqrt[4]{\frac{p_g}{psf} + 10} - 1.5\right) \cdot ft \qquad h'_d = 5.06\ ft$$

Adjusted height of projection or parapet

$h_c := h_p - h_b$ $\qquad h_c = 1.66\ ft$

Drift height

$h_d := \text{if}\left(h'_d \leq h_c, h'_d, h_c\right)$ $\qquad h_d = 1.66\ ft$

Calculate drift width

$$w := \text{if}\left(h'_d \leq h_c, 4 \cdot h'_d, min\left(4 \cdot \frac{h'^{\,2}_d}{h_c}, 8 \cdot h_c\right)\right)$$

Drift width $\qquad w = 13.25\ ft$

Maximum intensity of snow load

$p_m := \left(h_d\right) \cdot \gamma + p_f$ $\qquad p_m = 41.50\ psf$

$p_d := p_m - p_f$ — **Maximum drift surcharge load** $\qquad p_d = 27.50\ psf$

GEOMETRY/DRIFT LOAD THE SAME FOR EAST SIDE

DRIFT LOAD FOR PARAPET WALLS - WEST SIDE, SW CORNER

(ASCE 7-16 Section 7.8)

$l_u := 550 \cdot ft$ — Horizontal length of roof upwind of projection or parapet wall

$h_p := 6.2 \cdot ft$ — Height of projection or parapet wall 43.5' - (38' + 36.75')/2

Minimum length of roof upwind

$l_u := \text{if}\left(l_u \geq 20 \cdot ft, l_u, 20\ ft\right)$ $\qquad l_u = 550\ ft$

Snow Drift Geometry and Loading

Unadjusted drift height

$$h'_d := \frac{3}{4} \cdot \sqrt{I_s} \cdot \left(0.43 \cdot \sqrt[3]{\frac{l_u}{ft}} \cdot \sqrt[4]{\frac{p_g}{psf} + 10} - 1.5\right) \cdot ft \qquad h'_d = 5.06\ ft$$

Adjusted height of projection or parapet

$$h_c := h_p - h_b \qquad h_c = 5.36\ ft$$

Drift height

$$h_d := \text{if}\left(h'_d \le h_c, h'_d, h_c\right) \qquad h_d = 5.06\ ft$$

Calculate drift width

$$w := \text{if}\left(h'_d \le h_c, 4 \cdot h'_d, min\left(4 \cdot \frac{{h'_d}^2}{h_c}, 8 \cdot h_c\right)\right)$$

Drift width $w = 20.24\ ft$

Maximum intensity of snow load

$$p_m := \left(h_d\right) \cdot \gamma + p_f \qquad p_m = 97.98\ psf$$

$$p_d := p_m - p_f$$

Maximum drift surcharge load $p_d = 83.98\ psf$

GEOMETRY/DRIFT LOAD THE SAME FOR SOUTHEAST SIDE

DRIFT LOAD FOR PARAPET WALLS - NORTH SIDE, NW CORNER

(ASCE 7-16 Section 7.8)

$l_u := 238 \cdot ft$ — Horizontal length of roof upwind of projection or parapet wall

$h_p := 4.6 \cdot ft$ — Height of projection or parapet wall 40.5' - (36.75' + 35.17')/2

Minimum length of roof upwind

$$l_u := \text{if}\left(l_u \ge 20 \cdot ft, l_u, 20\ ft\right) \qquad l_u = 238\ ft$$

Unadjusted drift height

$$h'_d := \frac{3}{4} \cdot \sqrt{I_s} \cdot \left(0.43 \cdot \sqrt[3]{\frac{l_u}{ft}} \cdot \sqrt[4]{\frac{p_g}{psf} + 10} - 1.5\right) \cdot ft \qquad h'_d = 3.55\ ft$$

Snow Drift Geometry and Loading

Adjusted height of projection or parapet

$h_c := h_p - h_b$ $\qquad h_c = 3.76\ ft$

Drift height

$h_d := \text{if}\left(h'_d \le h_c, h'_d, h_c\right)$ $\qquad h_d = 3.55\ ft$

Calculate drift width

$w := \text{if}\left(h'_d \le h_c, 4 \cdot h'_d, min\left(4 \cdot \frac{h'^{\,2}_d}{h_c}, 8 \cdot h_c\right)\right)$ **Drift width** $\qquad w = 14.21\ ft$

Maximum intensity of snow load

$p_m := \left(h_d\right) \cdot \gamma + p_f$ $\qquad p_m = 72.97\ psf$

$p_d := p_m - p_f$ **Maximum drift surcharge load** $\qquad p_d = 58.97\ psf$

DRIFT LOAD FOR PARAPET WALLS - ALONG SOUTH SIDE

(ASCE 7-16 Section 7.8)

$l_u := 238 \cdot ft$ Horizontal length of roof upwind of projection or parapet wall

$h_p := 4.3 \cdot ft$ Height of projection or parapet wall 40.5' - (35.67' + 36.75')/2

Minimum length of roof upwind

$l_u := \text{if}\left(l_u \ge 20 \cdot ft, l_u, 20\ ft\right)$ $\qquad l_u = 238\ ft$

Unadjusted drift height

$h'_d := \frac{3}{4} \cdot \sqrt{I_s} \cdot \left(0.43 \cdot \sqrt[3]{\frac{l_u}{ft}} \cdot \sqrt[4]{\frac{p_g}{psf} + 10} - 1.5\right) \cdot ft$ $\qquad h'_d = 3.55\ ft$

Adjusted height of projection or parapet

$h_c := h_p - h_b$ $\qquad h_c = 3.46\ ft$

Drift height

$h_d := \text{if}\left(h'_d \le h_c, h'_d, h_c\right)$ $h_d = 3.46\ ft$

Calculate drift width

$w := \text{if}\left(h'_d \le h_c, 4 \cdot h'_d, min\left(4 \cdot \frac{h'^2_d}{h_c}, 8 \cdot h_c\right)\right)$ **Drift width** $w = 14.60\ ft$

Maximum intensity of snow load

$p_m := \left(h_d\right) \cdot \gamma + p_f$ $p_m = 71.38\ psf$

$p_d := p_m - p_f$ **Maximum drift surcharge load** $p_d = 57.38\ psf$

DRIFT LOAD FOR PARAPET WALLS - ALONG SOUTH SIDE *(ASCE 7-16 Section 7.8)*

$l_u := 238 \cdot ft$ Horizontal length of roof upwind of projection or parapet wall

$h_p := 7.3 \cdot ft$ Height of projection or parapet wall 43.5' - (35.67' + 36.75')/2

Minimum length of roof upwind

$l_u := \text{if}\left(l_u \ge 20 \cdot ft, l_u, 20\ ft\right)$ $l_u = 238\ ft$

Unadjusted drift height

$h'_d := \frac{3}{4} \cdot \sqrt{I_s} \cdot \left(0.43 \cdot \sqrt[3]{\frac{l_u}{ft}} \cdot \sqrt[4]{\frac{p_g}{psf} + 10} - 1.5\right) \cdot ft$ $h'_d = 3.55\ ft$

Adjusted height of projection or parapet

$h_c := h_p - h_b$ $h_c = 6.46\ ft$

Drift height

$h_d := \text{if}\left(h'_d \le h_c, h'_d, h_c\right)$ $h_d = 3.55\ ft$

Calculate drift width

$w := \text{if}\left(h'_d \le h_c, 4 \cdot h'_d, min\left(4 \cdot \frac{h'^2_d}{h_c}, 8 \cdot h_c\right)\right)$ **Drift width** $w = 14.21\ ft$

Maximum intensity of snow load

$p_m := (h_d) \cdot \gamma + p_f$ $\quad$ $p_m = 72.97\ psf$

$p_d := p_m - p_f$ $\quad$ **Maximum drift surcharge load** $\quad$ $p_d = 58.97\ psf$

DRIFT LOAD FOR PARAPET WALLS - NE CORNER

(ASCE 7-16 Section 7.8)

$l_u := 360 \cdot ft$ $\quad$ Horizontal length of roof upwind of projection or parapet wall

$h_p := 6.7 \cdot ft$ $\quad$ Height of projection or parapet wall 43.5' - (35.75' + 38')/2

Minimum length of roof upwind

$l_u := \text{if}\left(l_u \geq 20 \cdot ft, l_u, 20\ ft\right)$ $\quad$ $l_u = 360\ ft$

Unadjusted drift height

$h'_d := \frac{3}{4} \cdot \sqrt{I_s} \cdot \left(0.43 \cdot \sqrt[3]{\frac{l_u}{ft}} \cdot \sqrt[4]{\frac{p_g}{psf} + 10} - 1.5\right) \cdot ft$ $\quad$ $h'_d = 4.24\ ft$

Adjusted height of projection or parapet

$h_c := h_p - h_b$ $\quad$ $h_c = 5.86\ ft$

Drift height

$h_d := \text{if}\left(h'_d \leq h_c, h'_d, h_c\right)$ $\quad$ $h_d = 4.24\ ft$

Calculate drift width

$w := \text{if}\left(h'_d \leq h_c, 4 \cdot h'_d, min\left(4 \cdot \frac{{h'_d}^2}{h_c}, 8 \cdot h_c\right)\right)$ $\quad$ **Drift width** $\quad$ $w = 16.98\ ft$

Maximum intensity of snow load

$p_m := (h_d) \cdot \gamma + p_f$ $\quad$ $p_m = 84.45\ psf$

$p_d := p_m - p_f$ $\quad$ **Maximum drift surcharge load** $\quad$ $p_d = 70.45\ psf$

02 FRAMING

Joist and Girder Wall OOP Anchorage

Purpose Statement: The purpose of this calculation is to find the joist and girder axial force

Referenced Standards: ASCE 7-16

Joist

Parapet Condition - Northeast Corner

General

$H := 35.5\ ft$ SOG to B/Deck

$H_p := 8\ ft$ Parapet Height

$W := 26\ ft$ Width of Panel

$t_p := 9.5\ in$ Panel Thickness

$s_j := 10\ ft$ Joist Spacing

$h_{mean} := 38\ ft$ Mean Roof Height

Loading

Seismic

$S_{DS} := 0.843$ Design Spectral Acceleration

$I_e := 1.0$ Seismic Importance Factor

$k_a := 2.0$

$C_{Fp} := \max\left(0.4\ S_{DS} \cdot I_e \cdot k_a, 0.2\ I_e \cdot k_a\right) = 0.7$ Wall Anchorage Coefficient

$W_p := 150\ pcf \cdot t_p = 118.8\ psf$ Weight of Panel OOP

$F_p := C_{Fp} \cdot W_p = 80.1\ psf$ Wall Anchorage Force

$$P_{seis} := F_p \cdot \left(\frac{H}{2} + H_p\right) \cdot s_j = 20.6\ kip$$

Wind

$V := 97$ Basic Wind Speed per ASCE 7-16 26.5.1. From ATC Hazard Maps

$Exposure :=$ B Exposure Category per ASCE 7-16 26.7.3

Joist and Girder Wall OOP Anchorage

$K_z := 0.74$ Velocity Pressure Exposure Coefficient per ASCE 7-16 Table 26.10-1.

$K_{zparapet} := 0.79$ Kz for Top of Parapet Elevation

$K_{zt} := 1.0$ Topographic Factor per ASCE 7-16 26.8.2

$K_d := 0.85$ Wind Directionality Factor per ASCE 7-16 Table 26.6-1

$K_e := 1.0$ Ground Elevation factor per ASCE 7-16 Table 26.9-1

$q_h := 0.00256\ K_{zt}\ K_z\ K_d\ K_e\ (V)^2 \cdot 1\ psf = 15.15\ psf$ Velocity Pressure per ASCE 7-16 26.10.2

$q_p := q_h \cdot \left(\frac{K_{zparapet}}{K_z}\right) = 16.17\ psf$ Velocity Pressure for Top of Parapet Elevation

$GCpi := 0.18$ Internal Pressure Coefficient per ASCE 7-16 Table 26.13-1

$A_{eff\_p} := H_p \cdot s_j = 80\ ft^2$ Effective Parapet Wind Area

$A_{eff} := \max\left(\frac{H \cdot s_j}{2}, \frac{1}{3}\left(\frac{H}{2}\right)^2\right) = 177.5\ ft^2$ Effective Wall Wind Area

$Slope :=$ Y If roof slope is <10 degrees, then "Y" (GCp can be reduced by 10%)

Windward

Parapet

$$GCp_{pos} := \begin{array}{l} \text{if } A_{eff\_p} > 500\ ft^2 \\ \quad \| \ 0.7\ (1 - Slope) \\ \text{else if } A_{eff\_p} < 10\ ft^2 \\ \quad \| \ 1.0\ (1 - Slope) \\ \text{else} \\ \quad \| \left(1.1766 - 0.1766\ \log\left(\frac{A_{eff\_p}}{sqft}\right)\right)(1 - Slope) \end{array} = 0.76$$

Positive pressure coefficient (zones 5)

$p_1 := q_p \cdot GCp_{pos} = 12.2\ psf$

Joist and Girder Wall OOP Anchorage

$$GCp_{neg} := \text{if } A_{eff\_p} > 500\ ft^2 \;\; \| -1.4\,(1 - Slope) \;\; \text{else if } A_{eff\_p} < 10\ ft^2 \;\; \| -3.2\,(1 - Slope) \;\; \text{else} \;\; \left\| \left(-4.2595 + 1.0595\,\log\left(\frac{A_{eff\_p}}{sqft}\right)\right)\right. = -2.24$$

Negative pressure coefficient (zone 3)

$$p_2 := q_p \cdot GCp_{neg} = -36.3\ psf$$

$$p_p := p_1 - p_2 = 48.5\ psf$$

Wall

$$GCp_{pos} := \text{if } A_{eff} > 500\ ft^2 \;\; \| 0.7\,(1 - Slope) \;\; \text{else if } A_{eff} < 10\ ft^2 \;\; \| 1.0\,(1 - Slope) \;\; \text{else} \;\; \left\| \left(1.1766 - 0.1766\,\log\left(\frac{A_{eff}}{sqft}\right)\right)(1 - Slope)\right. = 0.7$$

Positive pressure coefficient (zones 5)

$$p_5 := q_h \cdot (GCp_{pos} + GCpi) = 13.4\ psf$$

Pressure at Parapet

$$P_{w\_wind} := \left(p_p \cdot (H_p) + p_5 \cdot \left(\frac{H}{2}\right)\right) s_j = 6.3\ kip$$

Force from windward wind pressure

Leeward

Parapet

$$GCp_{pos} := \text{if } A_{eff\_p} > 500\ ft^2 \;\; \| 0.7\,(1 - Slope) \;\; \text{else if } A_{eff\_p} < 10\ ft^2 \;\; \| 1.0\,(1 - Slope) \;\; \text{else} \;\; \left\| \left(1.1766 - 0.1766\,\log\left(\frac{A_{eff\_p}}{sqft}\right)\right)(1 - Slope)\right. = 0.76$$

Positive pressure coefficient (zones 5)

$$p_3 := q_p \cdot GCp_{pos} = 12.2\ psf$$

Joist and Girder Wall OOP Anchorage

$$GCp_{neg} := \text{if } A_{eff\_p} > 500\ ft^2 \ \| -0.8\,(1-Slope) \ \text{else if } A_{eff\_p} < 10\ ft^2 \ \| -1.4\,(1-Slope) \ \text{else} \ \left\| \left(-1.7532 + 0.3532\,\log\left(\frac{A_{eff\_p}}{sqft}\right)\right)(1-Slope) \right. = -0.97$$

Negative pressure coefficient (zone 5)

$$p_4 := q_p \cdot GCp_{neg} = -15.7\ psf$$

$$p_p := p_4 - p_3 = -28\ psf$$

Pressure at Parapet

Wall

$$GCp_{neg} := \text{if } A_{eff} > 500\ ft^2 \ \| -0.8\,(1-Slope) \ \text{else if } A_{eff} < 10\ ft^2 \ \| -1.4\,(1-Slope) \ \text{else} \ \left\| \left(-1.7532 + 0.3532\,\log\left(\frac{A_{eff}}{sqft}\right)\right)(1-Slope) \right. = -0.86$$

Negative pressure coefficient (zone 5)

$$p_6 := q_h \cdot \left(GCp_{neg} - GCpi\right) = -15.8\ psf$$

$$P_{w\_lee} := \left(p_p \cdot \left(H_p\right) + p_6 \cdot \left(\frac{H}{2}\right)\right) s_j = -5\ kip$$

Force from leeward wind pressure

Demand

$$P_w := \max\left(P_{w\_wind}, \left|P_{w\_lee}\right|\right) = 6.3\ kip$$

$$P_a := \max\left(0.7\ P_{seis}, 0.6\ P_w\right) = 14.4\ kip$$

Controlling Allowable Wall Anchorage Force

Joist and Girder Wall OOP Anchorage

Worst Case Top Plate Condition

General

$H := 35.5\ ft$ SOG to B/Deck

$H_p := 0\ ft$ Parapet Height

$W := 26\ ft$ Width of Panel

$t_p := 9.5\ in$ Panel Thickness

$s_j := 10\ ft$ Joist Spacing

$h_{mean} := 38\ ft$ Mean Roof Height

Loading

Seismic

$S_{DS} := 0.843$ Design Spectral Acceleration

$I_e := 1.0$ Seismic Importance Factor

$k_a := 2.0$

$C_{Fp} := \max\left(0.4\ S_{DS} \cdot I_e \cdot k_a , 0.2\ I_e \cdot k_a\right) = 0.7$ Wall Anchorage Coefficient

$W_p := 150\ pcf \cdot t_p = 118.8\ psf$ Weight of Panel OOP

$F_p := C_{Fp} \cdot W_p = 80.1\ psf$ Wall Anchorage Force

$$P_{seis} := F_p \cdot \left(\frac{H}{2} + H_p\right) \cdot s_j = 14.2\ kip$$

Wind

$V := 94$ Basic Wind Speed per ASCE 7-16 26.5.1. From ATC Hazard Maps

$Exposure :=$ B Exposure Category per ASCE 7-16 26.7.3

$K_z := 0.74$ Velocity Pressure Exposure Coefficient per ASCE 7-16 Table 26.10-1.

$K_{zt} := 1.0$ Topographic Factor per ASCE 7-16 26.8.2

$K_d := 0.85$ Wind Directionality Factor per ASCE 7-16 Table 26.6-1

Joist and Girder Wall OOP Anchorage

$K_e := 1.0$ — Ground Elevation factor per ASCE 7-16 Table 26.9-1

$q_h := 0.00256\ K_{zt}\ K_z\ K_d\ K_e\ (V)^2 \cdot 1\ psf = 14.23\ psf$ — Velocity Pressure per ASCE 7-16 26.10.2

$GCpi := 0.18$ — Internal Pressure Coefficient per ASCE 7-16 Table 26.13-1

$A_{eff\_p} := H_p \cdot s_j = 0\ ft^2$ — Effective Parapet Wind Area

$A_{eff} := \max\left(\frac{H \cdot s_j}{2}, \frac{1}{3}\left(\frac{H}{2}\right)^2\right) = 177.5\ ft^2$ — Effective Wall Wind Area

$Slope := Y$ — If roof slope is <10 degrees, then "Y" (GCp can be reduced by 10%)

Windward

Parapet

$$GCp_{pos} := \begin{cases} 0.7\ (1 - Slope) & \text{if } A_{eff\_p} > 500\ ft^2 \\ 1.0\ (1 - Slope) & \text{else if } A_{eff\_p} < 10\ ft^2 \\ \left(1.1766 - 0.1766\ \log\left(\frac{A_{eff\_p}}{sqft}\right)\right)(1 - Slope) & \text{else} \end{cases} = 0.9$$

Positive pressure coefficient (zones 5)

$p_1 := q_h \cdot GCp_{pos} = 12.8\ psf$

$$GCp_{neg} := \begin{cases} -1.4\ (1 - Slope) & \text{if } A_{eff\_p} > 500\ ft^2 \\ -3.2\ (1 - Slope) & \text{else if } A_{eff\_p} < 10\ ft^2 \\ \left(-4.2595 + 1.0595\ \log\left(\frac{A_{eff\_p}}{sqft}\right)\right)(1 - Slope) & \text{else} \end{cases} = -2.88$$

Negative pressure coefficient (zone 3)

$p_2 := q_h \cdot GCp_{neg} = -41\ psf$

$p_p := p_1 - p_2 = 53.8\ psf$

Joist and Girder Wall OOP Anchorage

Wall

$$GCp_{pos} := \text{if } A_{eff} > 500\ ft^2 \quad \| 0.7\ (1 - Slope) \quad \text{else if } A_{eff} < 10\ ft^2 \quad \| 1.0\ (1 - Slope) \quad \text{else} \quad \| \left(1.1766 - 0.1766 \log\left(\frac{A_{eff}}{sqft}\right)\right)(1 - Slope) = 0.7$$

Positive pressure coefficient (zones 5)

$$p_5 := q_h \cdot (GCp_{pos} + GCpi) = 12.5\ psf$$

Pressure at Parapet

$$P_{w\_wind} := \left(p_p \cdot (H_p) + p_5 \cdot \left(\frac{H}{2}\right)\right) s_j = 2.2\ kip$$

Force from windward wind pressure

Leeward

Parapet

$$GCp_{pos} := \text{if } A_{eff\_p} > 500\ ft^2 \quad \| 0.7\ (1 - Slope) \quad \text{else if } A_{eff\_p} < 10\ ft^2 \quad \| 1.0\ (1 - Slope) \quad \text{else} \quad \| \left(1.1766 - 0.1766 \log\left(\frac{A_{eff\_p}}{sqft}\right)\right)(1 - Slope) = 0.9$$

Positive pressure coefficient (zones 5)

$$p_3 := q_h \cdot GCp_{pos} = 12.8\ psf$$

$$GCp_{neg} := \text{if } A_{eff\_p} > 500\ ft^2 \quad \| -0.8\ (1 - Slope) \quad \text{else if } A_{eff\_p} < 10\ ft^2 \quad \| -1.4\ (1 - Slope) \quad \text{else} \quad \| \left(-1.7532 + 0.3532 \log\left(\frac{A_{eff\_p}}{sqft}\right)\right)(1 - Slope) = -1.26$$

Negative pressure coefficient (zone 5)

$$p_4 := q_h \cdot GCp_{neg} = -17.9\ psf$$

$$p_p := p_4 - p_3 = -30.7\ psf$$

Pressure at Parapet

Joist and Girder Wall OOP Anchorage

Wall

$$GCp_{neg} := \text{if } A_{eff} > 500\ ft^2 \;\; \| -0.8\,(1-Slope) \;\; \text{else if } A_{eff} < 10\ ft^2 \;\; \| -1.4\,(1-Slope) \;\; \text{else} \;\; \| \left(-1.7532 + 0.3532 \log\left(\frac{A_{eff}}{sqft}\right)\right)(1-Slope) = -0.86$$

Negative pressure coefficient (zone 5)

$$p_6 := q_h \cdot (GCp_{neg} - GCpi) = -14.8\ psf$$

$$P_{w\_lee} := \left(p_p \cdot (H_p) + p_6 \cdot \left(\frac{H}{2}\right)\right) s_j = -2.6\ kip$$

Force from leeward wind pressure

Demand

$$P_w := \max\left(P_{w\_wind}, \left|P_{w\_lee}\right|\right) = 2.6\ kip$$

$$P_a := \max\left(0.7\ P_{seis}, 0.6\ P_w\right) = 10\ kip$$

Controlling Allowable Wall Anchorage Force

Joist and Girder Wall OOP Anchorage

Girder

Worst Case Parapet Condition

General

$H := 38\ ft$ — SOG to B/Deck

$H_p := 5.5\ ft$ — Parapet Height

$L_g := 59.33\ ft$ — Joist Span (Girder OC Spacing)

$t_p := 8.75\ in$ — Panel Thickness

$W_p := 150\ pcf \cdot 9\ in = 112.5\ psf$

$F_p := C_{Fp} \cdot W_p = 75.9\ psf$

Loading

Seismic

$$P_{seis} := F_p \cdot \left(\frac{H}{2} + H_p\right) \cdot L_g = 110.3\ kip$$

Wind

$$A_{eff} := \max\left(\left(\frac{H}{2} + H_p\right) L_g, \frac{1}{3}\left(\frac{H}{2}\right)^2\right) = 1453.6\ ft^2$$

Effective Wind Area (MWFRS Procedure Allowed)

$C_p := 0.8$ — Windward Wall Pressure Coefficient per ASCE 7-16 Fig 27.3-1

$G := 0.85$ — Gust Factor per ASCE 7-16 26.11

$GC_{pn} := 1.5$ — Combined Net Pressure Coefficient at Parapet per ASCE 7-16 27.3.4

$p_p := q_h \cdot GC_{pn} = 21.3\ psf$ — Combined Net Pressure on the Parapet

$p := q_h \cdot (G \cdot C_p + GCpi) = 12.2\ psf$ — Combined Net Pressure on the Wall

$$P_{wind} := \left(p \cdot \left(\frac{H}{2}\right) + p_p \cdot (H_p)\right) L_g = 20.8\ kip$$

Wind Axial Load on Girder

Demand

$$P_{ag} := \max(0.7\ P_{seis}, 0.6\ P_{wind}) = 77.2\ kip$$

FORTRESS PUYALLUP - SUBPURLINS FOR SNOW LOAD

USE DFL#2 FOR SUBPURLINS, Fb = 900 PSI

C_D = 1.15 (SNOW)
C_r = 1.15
C_F = 1.3
C_L = 1.0

F'b = (900 PSI)(1.15)(1.15)(1.3)(1.0) = 1547 PSI

USING 2x6:

Sx = (1.5")(5.5")^2/6 = 7.56 IN^3

Mn = (7.56 IN3)(1547 PSI) = 11695 LB-IN = 974.6 LB-FT

USING 3x6:

Sx = (2.5")(5.5")2/6 = 12.6 IN3

Mn = (12.6 IN3)(1547 PSI) = 19492 LB-IN = 1624.3 LB-FT

DL = 0.7 PSF + 1.5 PSF + 1.6 PSF + 1.1 PSF = 4.9 PSF
(SNOW DRIFT NOT IN SOLAR AREA)

MAXIMUM DRIFT FOR 3x6 @ 2'-0" OC:

1624.3 LB-FT = (w*2')(10')^2/8

w = 64.9 PSF

64.9 PSF - 4.9 PSF - 14 PSF = **46 PSF (3x6 @ 2'-0" OC)**

(64.9 PSF)(2) - 4.9 PSF - 14 PSF = **110 PSF (3x6 @ 1'-0" OC)**

MAXIMUM DRIFT FOR 2x6 @ 2'-0" OC:

974.6 LB-FT = (w*2')(10')^2/8

w = 39 PSF

39 PSF - 4.9 PSF - 14 PSF = **20.1 PSF (2x6 @ 2'-0" OC)**

(39 PSF)(2) - 4.9 PSF - 14 PSF = **(59.1 PSF (2x6 @ 1'-0" OC)**

20 PSF(0.7) = 14 PSF FLAT ROOF SNOW + 5 PSF RAIN SURCHARGE

DO NOT NEED TO CONSIDER SURCHARGE FOR DRIFT LOADING

Snow Load

$SL := 19 \cdot psf$

Roof Dead Loads - Wood Roof

ROOFING	$R := 0.7 \cdot psf$
INSUL	$I := 1.5 \cdot psf$
1/2" PLY	$PL := 1.6 \cdot psf$
2X6 SUBPURLIN	$SP := 1.1 \cdot psf$
JOISTS	$J := 2.5 \cdot psf$
GIRDER	$G := 1.5 \cdot psf$
MECH/ELEC	$M := 2.5 \cdot psf$
SPRINK	$S := 1.5\ psf$
SOLAR	$PV := 4.0 \cdot psf$
MISC	$MISC := 2.1 \cdot psf$

15/32 APA STRUCT-1 OSB (32/16 SPAN RATING, STRONG AXIS PARALLEL TO SUBPURLINS), CONSIDER BENDING BETWEEN SUBPURLINS, SO PERPENDICULAR

FbS = 165 LB-IN/FT

Fb'S = (165 LB-IN/FT)(1.15) = 189.75 LB-IN/FT

MULTIPLY BY 1.5 FOR STRUCTURAL I

Fb'S = (189.75 LB-IN/FT)(1.5) = 284.6 LB-IN/FT

Table M9.2-1 Wood Structural Panel Bending Stiffness and Strength Capacities

Span Rating	Stress Parallel to Strength Axis[1] Plywood 3-ply	Plywood 4-ply	Plywood 5-ply	OSB	Stress Perpendicular to Strength Axis[1] Plywood 3-ply	Plywood 4-ply	Plywood 5-ply	OSB
PANEL BENDING STIFFNESS, EI (lbf-in.²/ft of panel width)								
24/0	66,000	66,000	66,000	60,000	3,600	7,900	11,000	11,000
24/16	86,000	86,000	86,000	78,000	5,200	11,500	16,000	16,000
32/16	125,000	125,000	125,000	115,000	8,100	18,000	25,000	25,000
40/20	250,000	250,000	250,000	225,000	18,000	39,500	56,000	56,000
48/24	NA	440,000	440,000	400,000	NA	65,000	91,500	91,500
16oc	165,000	165,000	165,000	150,000	11,000	24,000	34,000	34,000
20oc	230,000	230,000	230,000	210,000	13,000	28,500	40,500	40,500
24oc	NA	330,000	330,000	300,000	NA	57,000	80,500	80,500
32oc	NA	NA	715,000	650,000	NA	NA	235,000	235,000
48oc	NA	NA	1,265,000	1,150,000	NA	NA	495,000	495,000
Multiplier for Structural I Panels	1.0	1.0	1.0	1.0	1.5	1.5	1.6	1.6
PANEL BENDING STRENGTH, F_bS (lbf-in./ft of panel width)								
24/0	250	275	300	300	54	65	97	97
24/16	320	350	385	385	64	77	115	115
32/16	370	405	445	445	92	110	165	165
40/20	625	690	750	750	150	180	270	270
48/24	NA	930	1,000	1,000	NA	270	405	405
16oc	415	455	500	500	100	120	180	180
20oc	480	530	575	575	140	170	250	250
24oc	NA	705	770	770	NA	260	385	385
32oc	NA	NA	1,050	1,050	NA	NA	685	685
48oc	NA	NA	1,900	1,900	NA	NA	1,200	1,200
Multiplier for Structural I Panels	1.0	1.0	1.0	1.0	1.3	1.4	1.5	1.5

1. Strength axis is defined as the axis parallel to the face and back orientation of the flakes or the grain (veneer), which is generally the long panel direction, unless otherwise marked.

NA - Not applicable. Atypical panel construction.

FOR SUPPORTING SUBPURLINS @ 2'-0" OC, MAXIMUM AREA LOAD:

284.6 LB-IN = (w)(2')^2(12 IN/FT)/8

w = 47.4 PSF

DL = 0.7 PSF + 1.5 PSF + 1.6 PSF = 3.8 PSF (SNOW DRIFT NOT IN SOLAR AREA)

47.4 PSF - 3.8 PSF - 14 PSF = **29.6 PSF (MAXIMUM DRIFT FOR 2'-0" OC SUBPURLINS)**

FOR SUPPORTING SUBPURLINS @ 1'-0" OC, MAXIMUM AREA LOAD:

284.6 LB-IN = (w)(1')^2(12 IN/FT)/8

w = 189.7 PSF

189.7 PSF - 3.8 PSF - 14 PSF = **171.9 PSF (MAXIMUM DRIFT, NOT REACHED, OK)**

ON NORTH/SOUTH SIDES:

MAXIMUM DRIFT = 59 PSF @ 14'-8"

MAXIMUM FOR 2x6 @ 2' OC = 20.1 PSF

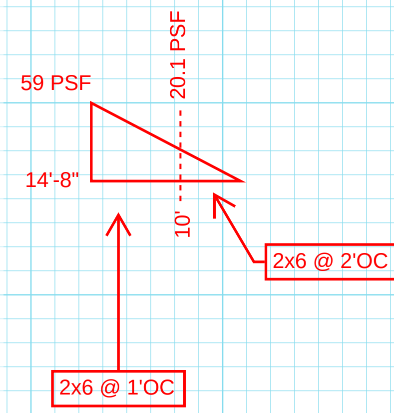
59 PSF/(14.67') = 20.1 PSF/x

x = 5', 14.67' - 5' = 9.67'

USE 2x6 @ 1'-0" OC UP TO 10' FROM WALL (59.1 PSF > 59 PSF)

USE 2x6 @ 2'-0" OC BEYOND

OSB IS ADEQUATE



ON EAST/WEST SIDES:

MAXIMUM DRIFT = 28 PSF @ 13'-3"

MAXIMUM FOR 2x6 @ 2' OC = 20.1 PSF

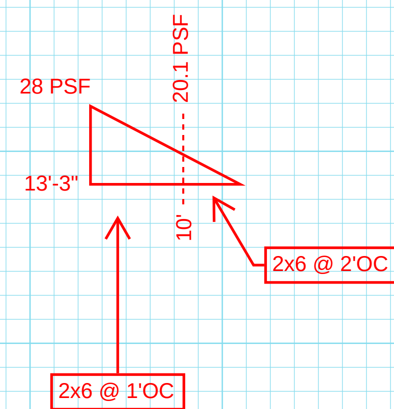
28 PSF/(13.25') = 20.1 PSF/x

x = 9.5', 13.25' - 9.5' = 3.7'

USE 3x6 @ 1'-0" OC UP TO 10' FROM WALL (UP TO 1ST JOIST) 46 PSF > 28 PSF

USE 2x6 @ 2'-0" OC BEYOND

OSB IS ADEQUATE

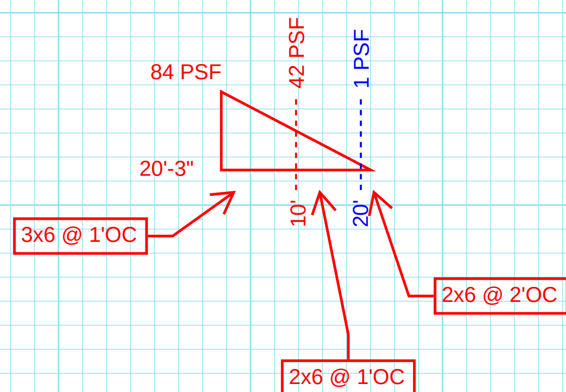


MAXIMUM DRIFT = 84 PSF @ 20'-3"

@ 1ST JOIST: 63 PSF AVG
USE 3X6 @ 1'-0" OC (110 PSF > 63 PSF)

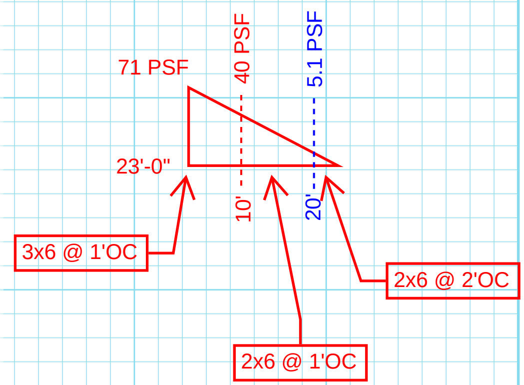
@ 2ND JOIST: 22 PSF AVG
USE 2X6 @ 1'-0" OC (59.1 PSF > 22 PSF)

OSB IS ADEQUATE



AT CORNER:

DRIFT = 71 PSF @ 23'-0" (DUE TO SKEWED FRAMING)



@ FIRST JOIST:
71 PSF/23' = x/(23' - 10')
x = 41 PSF

AVG = 55.5 PSF (DUE TO OSB, USE SUBPURLINS @ 1'-0" OC; 71 PSF MAX)

USE 2X6 @ 1'-0" OC (59.1 PSF > 55.5 PSF)

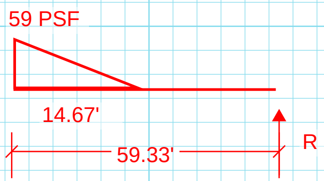
@ 2ND JOIST:
71 PSF/23' = x/(23' - 20')
x = 9.3 PSF

AVG = 25 PSF (DUE TO OSB, USE SUBPURLINS @ 1'-0" OC; 40 PSF MAX)

USE 2X6 @ 1'-0" OC

SNOW DRIFT REACTIONS

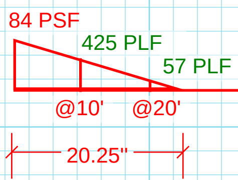
SOUTH SIDE:



R = (0.5)(59 PSF)(14.67')(14.67'/3)/59.33'

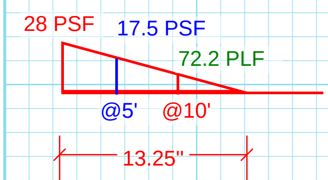
R = 35.7 PLF(10') = 357 LB @ EACH JOIST

EAST/WEST SIDE JOIST DIST LOADS ON GIRDERS



@10', (20.25' - 10'/20.25')(84 PSF) = 42.5 PSF AVG
(10')(42.5 PSF) = 425 PLF

@15', (20.25' - 15'/20.25')(84 PSF) = 21.8 PSF
(0.5)(21.8 PSF)(5.25') = 57.2 PLF



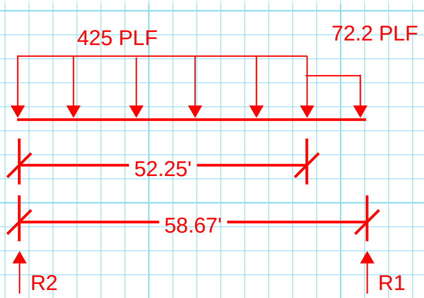
@5', (28 + 6.9)/2 = 17.5 PSF

(0.5)(17.5 PSF)(13.25' - 5') = 72.2 PLF

WEST SIDE GRID B GIRDER:

AT FIRST JOIST

(59.33'/2)(72.2 PLF) = 2.2k ONE SIDE

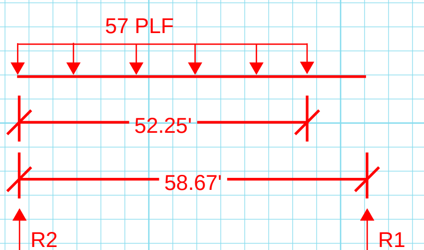


R1 = [(425 PLF)(52.25')(52.25'/2) + (72.2 PLF)(6.25')(6.25'/2 + 52.25')]/58.67'
R1 = [580.1 K-FT + 25 K-FT]/58.67'
R1 = 10.3k

10.3k + 2.2k = 12.5k @ FIRST JOIST FROM WALL

AT SECOND JOIST:

LOADING ON ONE SIDE ONLY



R1 = [(57 PLF)(52.25')(52.25'/2)]/58.67'
R1 = [77.8 K-FT]/58.67'
R1 = 1.9k

1.9k @ SECOND JOIST FROM WALL

WEST SIDE GRID C GIRDER:

LOADING AT FIRST JOIST ONLY

(72.2 PLF)(59.33') = 4.3k

WEST SIDE GRID D GIRDER:

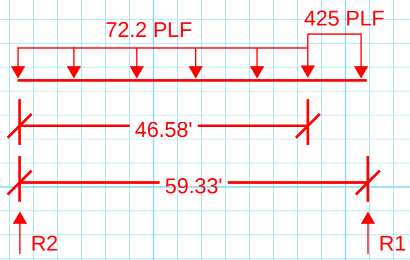
LOADING AT FIRST JOIST ONLY

(72.2 PLF)(59.33' + 60')/2 = 4.4k

EAST SIDE GRID B GIRDER:

AT FIRST JOIST

FROM PREVIOUS CALC, JOIST REACTION = 10.3k FROM SOUTH SIDE



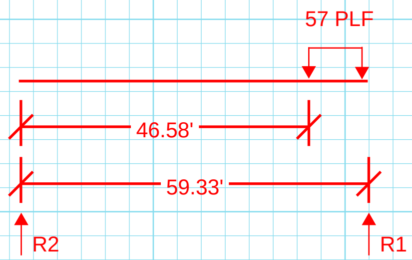
R1 = [(72.2 PLF)(46.58')(46.58'/2) + (425 PLF)(12.75')(12.75'/2 + 46.58')]/59.33'
R1 = [73.4 K-FT + 287 K-FT]/59.33'
R1 = 6.1k

R2 = (72.2 PLF)(46.58') + (425 PLF)(12.75') - 6.1k
R2 = 2.7k

2.7k + 10.3k = 13.0k @ FIRST JOIST FROM WALL

AT SECOND JOIST

FROM PREVIOUS CALCS, 1.9k FROM SOUTH JOIST



R1 = [(57 PLF)(12.75')(12.75'/2 + 46.58')]/59.33'
R1 = [38.5 K-FT]/59.33'
R1 = 0.7k

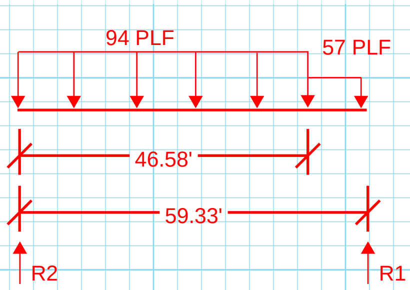
0.7k + 1.9k = 2.6k @ SECOND JOIST FROM WALL

AT SECOND JOIST

FROM PREVIOUS CALCS, 6.1k FROM HIGHER DRIFT

AT SECOND JOIST

FROM PREVIOUS CALC, JOIST REACTION = 1.9k



R1 = [(94 PLF)(46.58')(46.58'/2) + (57 PLF)(12.75')(12.75'/2 + 46.58')]/59.33'
R1 = [102 K-FT + 38.5 K-FT]/59.33'
R1 = 2.4k

R2 = (94 PLF)(46.58') + (57 PLF)(12.75') - 2.4k
R2 = 2.7k

2.7k + 1.9k = 4.6k @ SECOND JOIST FROM WALL

EAST SIDE GRID C GIRDER:

AT FIRST JOIST

(10.4k)(2) = 20.8k

AT SECOND JOIST

(2.4k)(2) = 4.8k

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JOIST WIND UPLIFT

V = 97 MPH

EXPOSURE B

MEAN ROOF HEIGHT:
(39.25' + 35.17')/2 = 37.2' = h

SMALLEST JOIST TRIB AREA:

(59.33')(10') = 593 FT^2 < 700 FT^2, USE C&C WIND LOAD PER ASCE 7-16 30.2.3

30.3.2 Design Wind Pressures. Design wind pressures on C&C elements of low-rise buildings and buildings with $h \leq 60$ ft ($h \leq 18.3$ m) shall be determined from the following equation:

$$p = q_h[(GC_p) - (GC_{pi})](\text{lb/ft}^2) \qquad (30.3\text{-}1)$$

where

q_h = velocity pressure evaluated at mean roof height h as defined in Section 26.10;

(GC_p) = external pressure coefficients given in:

- Fig. 30.3-1 (walls),
- Figs. 30.3-2A–I (flat roofs, gable roofs and hip roofs),
- Fig. 30.3-3 (stepped roofs),
- Fig. 30.3-4 (multispan gable roofs),
- Figs. 30.3-5A–B (monoslope roofs),
- Fig. 30.3-6 (sawtooth roofs),
- Fig. 30.3-7 (domed roofs),
- Fig. 27.3-3, Note 4 (arched roofs);

(GC_{pi}) = internal pressure coefficient given in Table 26.13-1.

26.10.2 Velocity Pressure. Velocity pressure, q_z, evaluated at height z above ground shall be calculated by the following equation:

$$q_z = 0.00256 K_z K_{zt} K_d K_e V^2 \ (\text{lb/ft}^2);\ V \text{ in mi/h} \qquad (26.10\text{-}1)$$

where

K_z = velocity pressure exposure coefficient, see Section 26.10.1.
K_{zt} = topographic factor, see Section 26.8.2.
K_d = wind directionality factor, see Section 26.6.
K_e = ground elevation factor, see Section 26.9.
V = basic wind speed, see Section 26.5.
q_z = velocity pressure at height z.

q_h = (0.00256)(Kz)(Kzt)(Kd)(Ke)(V^2)

q_h = (0.00256)(0.75)(1.0)(0.85)(1.0)$(97)^2$ = 15.4 PSF

Roof Dead Loads - Wood Roof

ROOFING	$R := 0.7 \cdot psf$
INSUL	$I := 1.5 \cdot psf$
1/2" PLY	$PL := 1.6 \cdot psf$
2X6 SUBPURLIN	$SP := 1.1 \cdot psf$
JOISTS	$J := 2.5 \cdot psf$
GIRDER	$G := 1.5 \cdot psf$
MECH/ELEC	$M := 2.5 \cdot psf$
SPRINK	$S := 1.5\ psf$
SOLAR	$PV := 4.0 \cdot psf$
MISC	$MISC := 2.1 \cdot psf$

$DL_{up\_joist} := R + I + PL + SP + J = 7.4\ psf$

$DL_{up\_girder} := R + I + PL + SP + J + G = 8.9\ psf$

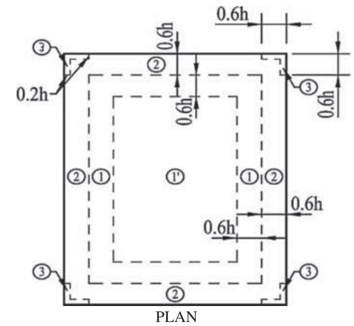
USING h = 37.2'

ZONES 2 AND 1 = (1.2)(37.2') = 44.6', SAY 45' FROM PERIMETER

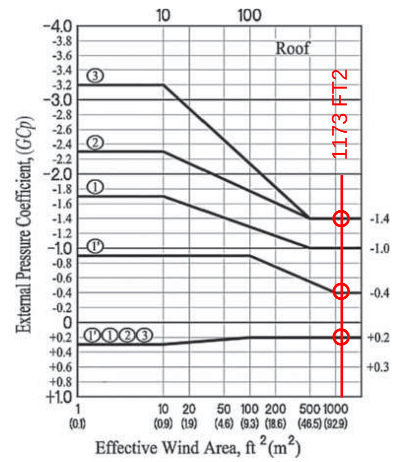
ZONE 1' = INTERIOR

FIGURE 30.3-2A

Diagrams



External Pressure Coefficients



EFFECTIVE WIND AREA:

(59.33')(59.33'/3) = 1173 FT2

GCp:
(2) = -1.4/+0.2
(1') = -0.4/+0.2

GCpi = +/- 0.18

EFFECTIVE WIND AREA, *A*: The area used to determine the external pressure coefficient, (GC_p) and (GC_{rn}). For C&C elements, the effective wind area in Figs. 30.3-1 through 30.3-7, 30.4-1, 30.5-1, and 30.7-1 through 30.7-3 is the span length multiplied by an effective width that need not be less than one-third the span length. For rooftop solar arrays, the effective wind area in Fig. 29.4-7 is equal to the tributary area for the structural element being considered, except that the width of the effective wind area need not be less than one-third its length. For cladding fasteners, the effective wind area shall not be greater than the area that is tributary to an individual fastener.

Table 26.13-1 Main Wind Force Resisting System and Components and Cladding (All Heights): Internal Pressure Coefficient, (GC_{pi}), for Enclosed, Partially Enclosed, Partially Open, and Open Buildings (Walls and Roof)

Enclosure Classification	Criteria for Enclosure Classification	Internal Pressure	Internal Pressure Coefficient, (GC_{pi})
Enclosed buildings	A_o is less than the smaller of $0.01A_g$ or 4 sq ft (0.37 m) and $A_{oi}/A_{gi} \le 0.2$	Moderate	+0.18 −0.18
Partially enclosed buildings	$A_o > 1.1A_{oi}$ and $A_o >$ the lesser of $0.01A_g$ or 4 sq ft (0.37 m) and $A_{oi}/A_{gi} \le 0.2$	High	+0.55 −0.55
Partially open buildings	A building that does not comply with Enclosed, Partially Enclosed, or Open classifications	Moderate	+0.18 −0.18
Open buildings	Each wall is at least 80% open	Negligible	0.00

Notes
1. Plus and minus signs signify pressures acting toward and away from the internal surfaces, respectively.
2. Values of (GC_{pi}) shall be used with q_z or q_h as specified.
3. Two cases shall be considered to determine the critical load requirements for the appropriate condition:
   a. A positive value of (GC_{pi}) applied to all internal surfaces, or
   b. A negative value of (GC_{pi}) applied to all internal surfaces.

p = (qh)[(GCp - GCpi)] = (15.4 PSF)[(-1.4 - 0.18)] = 24.4 PSF UPLIFT - ZONE(2)

p = (15.4 PSF)[(-0.4 - 0.18)] = 9.0 PSF UPLIFT - ZONE (1')

NET UPLIFT: 0.6 DL + 0.6W
USE ACTUAL MINIMUM ROOF DL = 7.4 PSF (SEE LOAD TAKEOFF)

(0.6)(7.4 PSF) - (0.6)(24.4 PSF) = 10.2 PSF ZONE (2)
(0.6)(7.4 PSF) - (0.6)(9.0 PSF) = 1 PSF, NO UPLIFT ZONE (1')

GIRDER WIND UPLIFT

SMALLEST GIRDER TRIB AREA:

(52')(59.33' + 59.33')/2 = 2625 FT² > 700 FT², USE MWFRS LOADS FOR UPL

30.2.3 Tributary Areas Greater than 700 ft² (65 m²). C&C elements with tributary areas greater than 700 ft² (65 m²) shall be permitted to be designed using the provisions for main wind force resisting systems (MWFRS).

USE ASCE 7-16 CHAPTER 27

27.3.1 Enclosed and Partially Enclosed Rigid and Flexible Buildings. Design wind pressures for the MWFRS of buildings of all heights in lb/ft² (N/m²), shall be determined by the following equation:

$$p = qGC_p - q_i(GC_{pi}) \quad (27.3\text{-}1)$$

where

$q = q_z$ for windward walls evaluated at height z above the ground.
$q = q_h$ for leeward walls, sidewalls, and roofs evaluated at height h.
$q_i = q_h$ for windward walls, sidewalls, leeward walls, and roofs of enclosed buildings, and for negative internal pressure evaluation in partially enclosed buildings.
$q_i = q_z$ for positive internal pressure evaluation in partially enclosed buildings where height z is defined as the level of the highest opening in the building that could affect the positive internal pressure. For buildings sited in wind-borne debris regions, glazing that is not impact-resistant or protected with an impact-resistant covering shall be treated as an opening in accordance with Section 26.12.3. For positive internal pressure evaluation, q_i may conservatively be evaluated at height $h(q_i = q_h)$.
G = gust-effect factor; see Section 26.11. For flexible buildings, G_f determined in accordance with Section 26.11.5 shall be substituted for G.
C_p = external pressure coefficient from Figs. 27.3-1, 27.3-2, and 27.3-3.
(GC_{pi}) = internal pressure coefficient from Table 26.13-1.

G = 0.85
GCpi = +/- 0.18

Roof Pressure Coefficients, C_p, for use with q_h

Wind Direction	h/L	Horizontal Distance from Windward Edge	C_p
Normal to Ridge for θ < 10° and Parallel to Ridge for All θ	≤0.5	0 to $h/2$	−0.9, −0.18
		$h/2$ to h	−0.9, −0.18
		h to $2h$	−0.5, −0.18
		$>2h$	−0.3, −0.18
	≥1.0	0 to $h/2$	−1.3[b], −0.18
		$>h/2$	−0.7, −0.18

h/L = 37.2' / 238' = 0.156 < 0.5

FROM 0 TO h, Cp = -0.9, -0.18
FROM h TO INTERIOR, Cp = -0.5, -0.18

p = (q)(G)(Cp) - (q)(GCpi)

p = (15.4 PSF)(0.85)(-0.9) - (15.4 PSF)(+0.18) = 14.6 PSF FROM 0 TO h

p = (15.4 PSF)(0.85)(-0.5) - (15.4 PSF)(+0.18) = 9.4 PSF FROM h TO INTERIOR

NET UPLIFT: 0.6 DL + 0.6W
USE ACTUAL MINIMUM ROOF DL = 7.4 PSF (SEE LOAD TAKEOFF)

(0.6)(8.9 PSF) - (0.6)(14.6 PSF) = 3.5 PSF FROM 0 TO h
(0.6)(8.9 PSF) - (0.6)(9.4 PSF) = 0.3 PSF UPLIFT FROM h TO INTERIOR

03 COLUMNS

Base Plate Design

Purpose Statement: Base Plate Design from AISC Design Guide 1, Second Edition (LRFD)

Referenced Standards: AISC 360-16
ACI 318-19

Element ID: BP 1

Base Plate Properties

$F_y := \text{A36} = 36\ ksi$ — Base Plate Steel Yield Strength

Concrete Properties

$f'_c := 3000\ psi$ — Concrete Compressive Strength

Column Loading & Properties

$P_u := 175 \cdot k$ — Column Axial Load

$d := 10 \cdot in$ — Column Depth

$b_f := 10 \cdot in$ — Column Width

Required BP Thickness

1) Find "A_1"

AISC 360-16 15th Ed. Specification Section J8 and Section 14-4
ACI 318-14 R22.8.3.2
Steel Design Guide 1, Second Edition Base Plate and Anchor Rod Design

$$A_1 := \begin{bmatrix} \dfrac{P_u}{0.65 \cdot 0.85 \cdot f'_c} \\ b_f \cdot d \end{bmatrix} = \begin{bmatrix} 105.58 \\ 100 \end{bmatrix} in^2$$

$$A_1 := \max(A_1) = 105.58\ in^2$$

2) Determine base plate dimensions "N" and "B"

$$\Delta := \frac{0.95 \cdot d - 0.8 \cdot b_f}{2} = 0.75\ in$$

	Calculated Values	User Defined Values
$N := \begin{bmatrix} \sqrt{A_1} + \Delta \\ d \end{bmatrix} = \begin{bmatrix} 11.03 \\ 10 \end{bmatrix} in$	$N := \max(N) = 11.03\ in$	$N := 16 \cdot in$
$B := \begin{bmatrix} \dfrac{A_1}{N} \\ b_f \end{bmatrix} = \begin{bmatrix} 6.6 \\ 10 \end{bmatrix} in$	$B := \max(B) = 10\ in$	$B := 16 \cdot in$

Base Plate Design

3) Determine Allowable Load on Concrete

$$\phi P_p := (0.65) \cdot (0.85) \cdot f'_c \cdot N \cdot B = 424.32 \; k$$

$$\frac{P_u}{\phi P_p} = 0.41$$

$$Check\left(P_u, \phi P_p\right) = \text{“OK”}$$

4) Determine "m" and "n"

$$m := \frac{N - 0.95 \cdot d}{2} = 3.25 \; in$$

$$n := \frac{B - 0.8 \cdot b_f}{2} = 4 \; in$$

5) Compute "X", "λ" and "n'"

$$X := \begin{bmatrix} \dfrac{4 \cdot d \cdot b_f}{\left(d + b_f\right)^2} \cdot \dfrac{P_u}{\phi P_p} \\ 1.0 \end{bmatrix} = \begin{bmatrix} 0.41 \\ 1 \end{bmatrix}$$

$$X := min(X) = 0.41$$

$$\lambda := \begin{bmatrix} \dfrac{2 \cdot \sqrt{X}}{\sqrt{1 - X} + 1} \\ 1 \end{bmatrix} = \begin{bmatrix} 0.73 \\ 1 \end{bmatrix}$$

$$\lambda := min(\lambda) = 0.73$$

$$n' := \frac{\sqrt{d \cdot b_f}}{4} = 2.5 \; in$$

$$n' = 2.5 \; in$$

6) Determine Plate Thickness

$$l := \begin{bmatrix} m \\ n \\ \lambda \cdot n' \end{bmatrix} = \begin{bmatrix} 3.25 \\ 4 \\ 1.82 \end{bmatrix} in$$

$$l := \max(l) = 4 \; in$$

$$t_p := l \cdot \sqrt{\frac{2 \cdot P_u}{.9 \cdot F_y \cdot B \cdot N}}$$

Min Base Plate Thickness $t_p = 0.82 \; in$

HSS Column Design

Purpose Statement: The purpose of this calculation is to calculate the capacity of HSS columns.

Referenced Standards:
IBC 2018
ASCE 7-16
AISC 360-16
AISC Steel Construction Manual, 15th Edition

LOADING

$DL := 19 \cdot psf$ Dead load

$LL := 20 \cdot psf$ Live load

$L_{col} := 52\ ft$ Girder span/column spacing along strong axis

$L_1 := 59.33\ ft$

$L_2 := 60\ ft$ Joist span

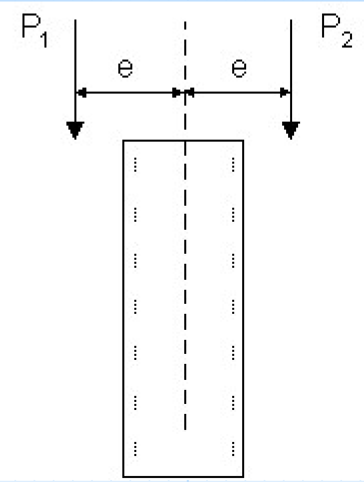
$$P_{u1} := (1.2 \cdot DL + 1.6 \cdot LL) \cdot \frac{L_{col}}{2} \cdot \left(\frac{L_1 + L_2}{2}\right) = 85\ k$$

$$P_{u2} := (1.2 \cdot DL + 1.6 \cdot LL) \cdot \frac{L_{col}}{2} \cdot \left(\frac{L_1 + L_2}{2}\right) = 85\ k$$

$$P_u := P_{u1} + P_{u2} = 170\ k$$

$$P_{2unb} := (0.9\ DL) \cdot \frac{L_{col}}{2} \cdot \left(\frac{L_1 + L_2}{2}\right) = 26.5\ k$$

$$P_{unb} := P_{u1} + P_{2unb} = 111.5\ k$$



GEOMETRY

$ht := 39.25\ ft$ Column height

$K_x := 1$

$K_y := 1$ Effective length factors

$e_1 := 7\ in$

$e_2 := 7\ in$ Eccentricity from the connection

HSS Column Design

Trial HSS-Section and Section Properties

$C := \text{"HSS10X10X3/8"}$

$F_y := 50 \cdot ksi$

$E := 29000 \cdot ksi$

$A := A(C)$	$A = 13.2\ in^2$	$S_x := S_x(C)$	$S_x = 40.4\ in^3$	$Z_x := Z_x(C)$	$Z_x = 47.2\ in^3$
$t := t(C)$	$t = 0.3\ in$	$r_x := r_x(C)$	$r_x = 3.9\ in$	$Z_y := Z_y(C)$	$Z_y = 47.2\ in^3$
$b := b(C)$	$b = 10\ in$	$S_y := S_y(C)$	$S_y = 40.4\ in^3$	$r_y := r_y(C)$	$r_y = 3.9\ in$
$d := d(C)$	$d = 10\ in$	$\lambda := b/t(C)$	$\lambda = 25.7$	$\lambda_w := h/t(C)$	$\lambda_w = 25.7$

Check for Compactness & Slenderness (Table B4.1b)

$\lambda_{pf} := 1.12 \cdot \sqrt{\dfrac{E}{F_y}}$ $\quad$ $\lambda_{rf} := 1.4 \cdot \sqrt{\dfrac{E}{F_y}}$

$\lambda_{pw} := 2.42 \cdot \sqrt{\dfrac{E}{F_y}}$ $\quad$ $\lambda_{rw} := 5.70 \cdot \sqrt{\dfrac{E}{F_y}}$

$$FlangeCompactness := \begin{array}{l} \text{if } \lambda \le \lambda_{pf} \\ \quad \| \text{"Compact"} \\ \text{else if } \lambda_{pf} < \lambda \le \lambda_{rf} \\ \quad \| \text{"Noncompact"} \\ \text{else} \\ \quad \| \text{"Slender"} \end{array}$$

$FlangeCompactness = \text{"Compact"}$

$$WebCompactness := \begin{array}{l} \text{if } \lambda_w \le \lambda_{pw} \\ \quad \| \text{"Compact"} \\ \text{else if } \lambda_{pw} < \lambda_w \le \lambda_{rw} \\ \quad \| \text{"Noncompact"} \\ \text{else} \\ \quad \| \text{"Slender"} \end{array}$$

$WebCompactness = \text{"Compact"}$

HSS Column Design

BALANCED LOADING

Axial

$$L_c/r := \text{if}\left(\frac{K_x \cdot ht}{r_x} < \frac{K_y \cdot ht}{r_y}, \frac{K_y \cdot ht}{r_y}, \frac{K_x \cdot ht}{r_x}\right) = 120.2$$ Effective slenderness ratio

$$F_e := \frac{\pi^2 \cdot E}{L_c/r^2} = 19.8\ ksi$$ Elastic buckling stress (Eq. E3-4)

$$F_{cr1} := 0.658^{\frac{F_y}{F_e}} \cdot F_y = 17.4\ ksi$$

$$F_{cr2} := 0.877 \cdot F_e = 17.4\ ksi$$

$$F_{cr} := \text{if}\left(L_c/r < 4.71\sqrt{\frac{E}{F_y}}, F_{cr1}, F_{cr2}\right) = 17.4\ ksi$$ Critical buckling stress (Eq. E3-2)

$$\phi_c := 0.9$$

$$\phi P_n := \phi_c \cdot A \cdot F_{cr} = 206.6\ k$$ Design axial capacity (Eq. E3-3)

Bending

$$M_{ux} := \left\|P_{u1} \cdot e_1 - P_{u2} \cdot e_2\right\| = 0\ k \cdot ft$$ Moment demand about the x-axis

$$M_{uy} := 0\ k \cdot ft$$ Moment demand about the y-axis

Compact

$$M_{px} := F_y \cdot Z_x = 196.7\ k \cdot ft$$

$$\phi_b := 0.9$$ Strength reduction factor

$$\phi M_{nx\_c} := \phi_b \cdot M_{px} = 177\ k \cdot ft$$ Design moment capacity about the x-axis for a compact section

$$M_{py} := F_y \cdot Z_y = 196.7\ k \cdot ft$$

$$\phi M_{ny\_c} := \phi_b \cdot M_{py} = 177\ k \cdot ft$$ Design moment capacity about the y-axis for a compact section

Noncompact (Flange Local Buckling)

$$M_{nx\_n} := \left(M_{px} - \left(M_{px} - F_y \cdot S_x\right) \cdot \left(3.57\ \lambda \cdot \sqrt{\frac{F_y}{E}} - 4\right)\right) = 202.1\ k \cdot ft$$

$$\phi M_{nx\_n} := min\left(\phi_b \cdot M_{nx\_n}, \phi M_{nx\_c}\right) = 177\ k \cdot ft$$ Design moment capacity about the x-axis for a noncompact section

$$M_{ny\_n} := \left(M_{py} - \left(M_{py} - F_y \cdot S_y\right) \cdot \left(3.57\ \lambda_w \cdot \sqrt{\frac{F_y}{E}} - 4\right)\right) = 202.1\ k \cdot ft$$

$$\phi M_{ny\_n} := min\left(\phi_b \cdot M_{ny\_n}, \phi M_{ny\_c}\right) = 177\ k \cdot ft$$

Design moment capacity about the y-axis for a noncompact section

Slender (Flange Local Buckling)

$$b_e := min\left(1.92\ t \cdot \sqrt{\frac{E}{F_y}} \cdot \left(1 - \frac{0.38}{\lambda} \cdot \sqrt{\frac{E}{F_y}}\right), b\right) = 10\ in$$

Effective width of the compression flange per F7.2

$$I_{eff} := 2\ b_e \cdot t \cdot \left(\frac{b_e - t}{2}\right)^2 + \frac{t \cdot b_e^{\ 3}}{6} = 220.7\ in^4$$

$$S_{eff\_x} := \frac{I_{eff}}{\left(\frac{b_e}{2}\right)} = 44.1\ in^3$$

Effective section modulus per F7.2

$$d_e := min\left(1.92\ t \cdot \sqrt{\frac{E}{F_y}} \cdot \left(1 - \frac{0.38}{\lambda_w} \cdot \sqrt{\frac{E}{F_y}}\right), d\right) = 10\ in$$

Effective width of the compression flange per F7.2

$$I_{eff\_y} := 2\ d_e \cdot t \cdot \left(\frac{d_e - t}{2}\right)^2 + \frac{t \cdot d_e^{\ 3}}{6} = 220.7\ in^4$$

$$S_{eff\_y} := \frac{I_{eff}}{\left(\frac{d_e}{2}\right)} = 44.1\ in^3$$

Effective section modulus per F7.2

$$\phi M_{nx\_s} := \phi_b \cdot S_{eff\_x} \cdot F_y = 165.5\ k \cdot ft$$

Design moment capacity about the x-axis for a slender section

$$\phi M_{ny\_s} := \phi_b \cdot S_{eff\_y} \cdot F_y = 165.5\ k \cdot ft$$

Design moment capacity about the y-axis for a slender section

$$\phi M_{nx} := \text{if } FlangeCompactness = \text{“Compact”} \; \| \; \phi M_{nx\_c} \; \text{else if } FlangeCompactness = \text{“Noncompact”} \; \| \; \phi M_{nx\_n} \; \text{else} \; \| \; \phi M_{nx\_s} \quad = 177 \; k \cdot ft$$

$$\phi M_{ny} := \text{if } WebCompactness = \text{“Compact”} \; \| \; \phi M_{ny\_c} \; \text{else if } WebCompactness = \text{“Noncompact”} \; \| \; \phi M_{ny\_n} \; \text{else} \; \| \; \phi M_{ny\_s} \quad = 177 \; k \cdot ft$$

Axial for Slender Members

$c_1 := 0.2$ — Effective width imperfection adjustment factor from Table E7.1

$c_2 := 1.38$ — Factor from Table E7.1

$F_{el} := \left(c_2 \cdot \frac{\lambda_{rf}}{\lambda}\right)^2 F_y = 163.9 \; ksi$ — Elastic local buckling stress (Eq. E7-5)

$$b_e := \text{if } \lambda \le \lambda_{rf} \cdot \sqrt{\frac{F_y}{F_{cr}}} \; \| \; b \; \text{else} \; \| \; b \cdot \left(1 - c_1 \sqrt{\frac{F_{el}}{F_{cr}}}\right) \cdot \sqrt{\frac{F_{el}}{F_{cr}}} \quad = 10 \; in$$

Effective width per E7.1

$A_{eff} := 2 \; b_e \cdot t + 2 \; d \cdot t = 14 \; in^2$ — Effective area

$\phi P_{n\_s} := \phi_c \cdot A_{eff} \cdot F_{cr} = 218.5 \; k$

$$\phi P_n := \text{if } FlangeCompactness = \text{“Slender”} \; \| \; \phi P_{n\_s} \; \text{else} \; \| \; \phi P_n \quad = 206.6 \; k$$

HSS Column Design

$$Unity := \text{if}\left(\frac{P_u}{\phi P_n} < 0.2, \frac{P_u}{2 \cdot \phi P_n} + \left(\frac{M_{ux}}{\phi M_{nx}} + \frac{M_{uy}}{\phi M_{ny}}\right), \frac{P_u}{\phi P_n} + \frac{8}{9} \cdot \left(\frac{M_{ux}}{\phi M_{nx}} + \frac{M_{uy}}{\phi M_{ny}}\right)\right) = 0.82$$

$$Check_1 := \text{if}\left(Unity \le 1, \text{"OK"}, \text{"NG"}\right) = \text{"OK"}$$

UNBALANCED LOADING

$P_u := P_{unb} = 111.5\ k$ Unbalanced axial demand

$M_{ux} := P_{u1} \cdot e_1 - P_{2unb} \cdot e_2 = 34.1\ k \cdot ft$ Unbalanced loading moment demand about the x-axis

$M_{uy} := 0 \cdot k \cdot ft$ Moment demand about the y-axis

$$Unity := \text{if}\left(\frac{P_u}{\phi P_n} < 0.2, \frac{P_u}{2 \cdot \phi P_n} + \left(\frac{M_{ux}}{\phi M_{nx}} + \frac{M_{uy}}{\phi M_{ny}}\right), \frac{P_u}{\phi P_n} + \frac{8}{9} \cdot \left(\frac{M_{ux}}{\phi M_{nx}} + \frac{M_{uy}}{\phi M_{ny}}\right)\right) = 0.7$$

$$Check_2 := \text{if}\left(Unity \le 1, \text{"OK"}, \text{"NG"}\right) = \text{"OK"}$$

04 FOUNDATIONS

Interior Footings

Purpose Statement: The purpose of this calculation is the design of isolated spread footings for gravity-only columns and loading conditions.

Referenced Standards: 2021 IBC
ASCE 7-16
ACI 318-19

FOOTING TYPE: F1 - Typical Interior Footing

$A_{trib} := 52\ ft \cdot 60\ ft$

$q_{all} := 2500\ psf$ Soil Bearing

$f_y := 60\ ksi$ Steel Yield Strength

$f_c := 3\ ksi$ Conc. Comp. Strength

$W := 7 \cdot ft$ Footing Length & Width

$t := 1.5\ ft$ Footing Thickness

$A_g := W \cdot t$ Ftg. X-Sect. Area

$d_c := 13 \cdot in$ Column Depth*

$b_f := 13 \cdot in$ Column Width*

*Half distance between column face and edge of base plate (16"x16")

$b := b_f \qquad c := d_c$ See ACI 318-19 13.2.7.2

Loading:

$P_{dl} := 19\ psf \cdot A_{trib} = 59.3\ k$ Column Dead Load

$P_{ll} := 20\ psf \cdot A_{trib} = 62.4\ k$ Column Live Load

$P_{sl\_min} := 19\ psf \cdot A_{trib} = 59.3\ k$ Minimum roof snow load

$P_{sl\_drift} := 14\ psf \cdot A_{trib} + 6.3\ k = 50\ k$ Balanced snow load + drift

$P_{sl} := \max\left(P_{sl\_min}, P_{sl\_drift}\right) = 59.3\ k$ Column Snow Load

$P_{uplift} := 3.5\ psf \cdot A_{trib} = 10.9\ k$ Column Wind Uplift Load

Interior Footings

Service Load - ASCE 7-16 2.4.1

$P_{ser\_2} := P_{dl} + P_{sl}$ ASCE 7-16 2.4.1, LC 2

$P_{ser\_7} := 0.6\ P_{dl} - 0.6\ P_{uplift}$ ASCE 7-16 2.4.1, LC 7

Ultimate Loads - ASCE 7-16 2.3.1/2.3.6

$P_{u\_1} := 1.4 \cdot P_{dl}$ ASCE 7-16 2.3.1, LC 1

$P_{u\_2} := 1.2 \cdot P_{dl} + 1.6 \cdot P_{sl}$ ASCE 7-16 2.3.1, LC 2

$P_{u\_5} := 0.9 \cdot P_{dl} - P_{uplift}$ ASCE 7-16 2.3.1, LC 5 (Uplift case)

$P_u := \max\left(P_{u\_1}, P_{u\_2}\right)$

$\text{if}\left(P_{ser\_7} < 0\ k \lor P_{u\_5} < 0\ k, \text{“UPLIFT, REVISE FTG SIZE”}, \text{“Okay”}\right) = \text{“Okay”}$

Soil Bearing:

$P_{ftg} := 150\ pcf \cdot W^2 \cdot t$

$q := \dfrac{P_{ser\_2}}{W^2}$ Net bearing pressure $\dfrac{q}{q_{all}} = 0.968$

$Flag_b := \text{if}\left(q \le q_{all}, \text{“OK”}, \text{“NG!!”}\right)$ $Flag_b = \text{“OK”}$

Interior Footings

Beam Flexure: (per 1' width) ACI 318-19 13.2.6.4, 13.2.7.1

$$q_u := \frac{P_u + P_{ftg} \cdot 1.2}{W^2}$$ Ultimate bearing pressure

$$M_u := q_u \cdot \left(\frac{W - c}{2}\right)^2 \cdot \frac{1}{2} \cdot ft$$ Moment Demand

$\#_{reinf} :=$ Reinforcement size: #5 — Size of reinforcement

$A_s := \#_{reinf_0}$ $\quad d_s := \#_{reinf_1}$

$d := t - 3\ in - d_s \cdot 1.5 = 14.1\ in$ — Steel Depth

Solver — Values

$A_{sreq} := 1\ in^2$

Solver — Constraints

$$A_{sreq} \cdot f_y \cdot \left(d - \frac{A_{sreq} \cdot f_y}{0.85 \cdot f_c \cdot ft \cdot 2}\right) \cdot .9 - M_u = 0$$ Required area of steel for footing flexure

$A_{sreq} := \text{Find}\left(A_{sreq}\right) = 0.26\ in^2$

$$A_{smin} := 0.0018 \cdot A_g \cdot \frac{ft}{W} = 0.39\ in^2$$ Min flexural reinforcement per ACI 318-19 8.6.1.1

$sp := 9\ in$ — Spacing of reinforcement OC

$$A_{s\_ft} := A_s \cdot \frac{12\ in}{sp} = 0.41\ in \cdot in$$ Area of reinforcement provided per foot

$A_{sreq} := \max\left(A_{sreq}, A_{smin}\right)$ $\qquad \frac{A_{sreq}}{A_{s\_ft}} = 0.941$

$Flag_f := \text{if}\left(A_{s\_ft} \ge A_{sreq}, \text{"OK"}, \text{"NG!!"}\right)$ $\qquad Flag_f = \text{"OK"}$

Interior Footings

Punching Shear:

$\rho_w := \frac{A_s}{1\ ft \cdot d} = 0.0018$

Ratio of longitudinal bars defined in R22.5.5

$\lambda_s := 1.0$

Size effect modification factor, eq 22.5.5.1.3 (Neglected per ACI 318-19 13.2.6.2)

$V_u := P_u - (c+d)\,(b+d) \cdot q_u$

Punching shear demand at critical section per 22.6.4.2

$b_o := 2 \cdot (c+d) + 2 \cdot (b+d)$

Critical section per 22.6.4.2

$\phi_v := 0.75$

Strength reduction factor

$\phi V_c := \phi_v \cdot \left(4 \cdot \lambda_s \cdot \sqrt{f_c \cdot psi} \cdot b_o \cdot d\right)$

Table 22.6.5.2 (a)

$\frac{V_u}{\phi V_c} = 0.589$

$Flag_{pv} := \text{if}\left(\phi V_c \geq V_u, \text{"OK"}, \text{"NG!!"}\right)$

$Flag_{pv} = \text{"OK"}$

Beam Shear: (per 1' width) ACI 318-19 8.4.3, 22.5.5.1

$V_u := \left(\frac{W-c}{2} - d\right) \cdot q_u \cdot ft = 6.5\ k$

at "d" from column face

$\phi V_c := \phi_v \cdot \left(8 \cdot \lambda_s \cdot \rho_w^{\frac{1}{3}} \cdot \sqrt{f_c \cdot psi} \cdot d\right) \cdot ft$

Table 22.5.5.1 (c) - assumes Av < Av,min

$\phi V_{c\_max} := \phi_v \cdot \left(5 \cdot \sqrt{f_c \cdot psi} \cdot d\right) \cdot ft$

$\phi V_c := \text{if}\left(\phi V_c > \phi V_{c\_max}, \phi V_{c\_max}, \phi V_c\right)$

$\phi V_c = 6.8\ k$

$d = 14.1\ in$

$\frac{V_u}{\phi V_c} = 0.962$

$Flag_v := \text{if}\left(\phi V_c \geq V_u, \text{"OK"}, \text{"NG!!"}\right)$

$Flag_v = \text{"OK"}$

Panel Footing

Purpose Statement:

The purpose of this calculation is the design of pad/continuous foundations supporting concrete tilt panels serving as shear walls.

Referenced Standards:

2021 IBC
ASCE 7-16
ACI 318-19

PROJECT PARAMETERS

$q_{all} := 2500 \cdot psf$	Allowable soil bearing pressure
$\gamma_{temp} := \frac{4}{3}$	Short-term loading increase
$f_c := 3 \cdot ksi$	Concrete strength
$f_y := 60 \cdot ksi$	Steel strength
$\gamma_{conc} := 150\ pcf$	Density of Concrete
$S_{DS} := 0.843$	Design spectral response acceleration parameter at short periods

Pad Footing - Panel 51 and Similar (Footing Type F4):

Panel Dimensions

$t_p := 10\ in$ — Panel Thickness

$A_o := 229\ ft^2$ — Storefront Area

Loading

$P_{dl} := 19\ psf \cdot 10\ ft \cdot 13\ ft = 2.47\ k$ — Dead Load per Panel Leg

$P_{sl} := 78\ psf \cdot 10\ ft \cdot 13\ ft = 10.14\ k$ — Snow Load per Panel Leg

$$P_{pan} := \gamma_{conc} \cdot t_p \cdot \frac{(45\ ft \cdot 26\ ft - A_o)}{2} + \frac{A_o \cdot 15\ psf}{2} = 60.53\ k$$ Panel(s) Load per Leg

$P_{sei} := 25\ k$ — Seismic Load (including ρ, do not include vertical seismic effects)

Footing Dimensions

$W := 6 \cdot ft$ — Footing width

$L := 6 \cdot ft$ — Footing Length

$t := 16 \cdot in$ — Footing Thickness

$P_{ftg} := \gamma_{conc} \cdot L \cdot W \cdot t$ — Footing Weight

Service Loads - ASCE 7-16 2.4.1/2.4.5

$P_{ser} := P_{dl} + P_{pan} + P_{sl}$ — ASCE 7-16 2.4.1, LC 2

$P_{ser\_sei} := P_{pan} + P_{dl} + 0.75 \cdot P_{sl} + 0.525 \cdot P_{sei}$ — ASCE 7-16 2.4.5, LC 9

$P_{ser\_sei2} := P_{pan} + P_{dl} + 0.7\ P_{sei}$ — ASCE 7-16 2.4.5, LC 8

Ultimate Loads - ASCE 7-16 2.3.1/2.3.6

$P_u := 1.2 \cdot (P_{dl} + P_{pan}) + 1.6 \cdot P_{sl}$ — ASCE 7-16 2.3.1, LC 2

$P_{u\_sei} := (1.2 + 0.2\ S_{DS}) \cdot (P_{dl} + P_{pan}) + 0.2 \cdot P_{sl} + P_{sei}$ — ASCE 7-16 2.3.6, LC 6

Panel Footing

Soil Bearing - ACI 318-19 13.3.1.1

$$q := \max\left(\frac{P_{ser\_sei}}{\gamma_{temp} \cdot W \cdot L}, \frac{P_{ser\_sei2}}{\gamma_{temp} \cdot W \cdot L}, \frac{P_{ser}}{W \cdot L}\right)$$ Net bearing pressure $\frac{q}{q_{all}} = 0.813$

$$Flag_b := \text{if}\left(q \le q_{all}, \text{"OK"}, \text{"NG!!"}\right)$$ $Flag_b = \text{"OK"}$

Flexure (per 1'-0" width) - ACI 318-19 13.2.6.4, 13.2.7.1

$$q_u := \max\left(\frac{P_{u\_sei}}{W \cdot L}, \frac{P_u}{W \cdot L}\right)$$ Ultimate bearing pressure

$$M_u := q_u \cdot \left(\frac{W - t_p}{2}\right)^2 \cdot \frac{1}{2} \cdot ft$$ Moment demand

$\#_{reinf} :=$ Reinforcement size: #5 — Size of reinforcement

$A_s := \#_{reinf_0}$ $\quad d_s := \#_{reinf_1}$

$d := t - 3\ in - d_s \cdot 1.5 = 12.06\ in$ — Steel Depth

Solve Block — Guess Values / Constraints / Solver

$A_{sreq} := 1\ in^2$

$$0.9 \cdot A_{sreq} \cdot f_y \cdot \left(d - \frac{A_{sreq} \cdot f_y}{0.85 \cdot f_c \cdot 1\ ft \cdot 2}\right) - M_u = 0$$

$A_{sreq} := \text{Find}\left(A_{sreq}\right)$ — Required area of steel for footing flexure

$A_{smin} := 0.0018 \cdot t \cdot (1\ ft) = 0.35\ in^2$ — Min flexural reinforcement per ACI 318-19 8.6.1.1

$sp := 9\ in$ — OC spacing of reinforcement

$$A_{s\_ft} := \frac{A_s \cdot (1\ ft)}{sp}$$ Area of reinforcement provided per foot of width

$A_{sreq} := \max\left(A_{sreq}, A_{smin}\right)$ $\quad \frac{A_{sreq}}{A_{s\_ft}} = 0.84$

$$Flag_f := \text{if}\left(A_{s\_ft} \ge A_{sreq}, \text{"OK"}, \text{"NG!!"}\right)$$ $Flag_f = \text{"OK"}$

Beam Shear (per 1' width) - ACI 318-19 8.4.3, 22.5.5.1

$\rho_w := \dfrac{A_{s\_ft}}{1\ ft \cdot d} = 0.0029$ Ratio of longitudinal bars defined in R22.5.5

$\lambda_s := 1.0$ Size effect modification factor, eq 22.5.5.1.3 (Neglected per ACI 318-19 13.2.6.2)

$V_u := \left(\dfrac{W - t_p}{2} - d\right) \cdot q_u \cdot ft$ at "d" from panel face

$\phi_v := 0.6$

Strength reduction factor, 21.2.4.3

$\phi V_c := \phi_v \cdot \left(8 \cdot \lambda_s \cdot \rho_w^{\frac{1}{3}} \cdot \sqrt{f_c \cdot psi} \cdot d\right) \cdot ft$ Table 22.5.5.1 (c) - assumes Av < Av,min

$\phi V_{c\_max} := \phi_v \cdot \left(5 \cdot \sqrt{f_c \cdot psi} \cdot d\right) \cdot ft$ Maximum concrete shear strength, 22.5.5.1.1

$\phi V_c := \text{if}\left(\phi V_c > \phi V_{c\_max}, \phi V_{c\_max}, \phi V_c\right)$ $\phi V_c = 5.4\ k$

$\dfrac{V_u}{\phi V_c} = 0.92$

$Flag_v := \text{if}\left(\phi V_c \geq V_u, \text{“OK”}, \text{“NG!!”}\right)$ $Flag_v = \text{“OK”}$

Beam Shear (Beyond Leg Edge) - ACI 318-19 8.4.3, 22.5.5.1

$\rho_w = 0.0029$ Ratio of longitudinal bars defined in R22.5.5
Assumes same reinforcement each way

$\phi_v := 0.6$ Strength reduction factor, 21.2.4.3

$leg := 4.75\ ft$ Length of panel leg

$V_u := (L - leg - d) \cdot q_u \cdot ft$ Ultimate Shear at "d" from panel face per unit width

$\phi V_c := \phi_v \cdot \left(8 \cdot \lambda_s \cdot \rho_w^{\frac{1}{3}} \cdot \sqrt{f_c \cdot psi} \cdot d\right) \cdot ft$ Table 22.5.5.1 (c) - assumes Av < Av,min

Panel Footing

$\phi V_{c\_max} := \phi_v \cdot \left(5 \cdot \sqrt{f_c \cdot psi} \cdot d\right) \cdot ft$ Maximum concrete shear strength, 22.5.5.1.1

$\phi V_c := \text{if}\left(\phi V_c > \phi V_{c\_max}, \phi V_{c\_max}, \phi V_c\right)$

$\phi V_c = 5.4\ k$

$\dfrac{V_u}{\phi V_c} = 0.14$

$Flag_v := \text{if}\left(\phi V_c \geq V_u, \text{“OK”}, \text{“NG!!”}\right)$

$Flag_v = \text{“OK”}$

Pad Footing - Panel 4, 18 Drive in Door (Footing Type F4):

Panel Dimensions

$t_p := 9.5\ in$ — Panel Thickness

$A_o := 256\ ft^2$ — Door Area

Loading

$P_{dl} := 19\ psf \cdot 30\ ft \cdot 13\ ft = 7.41\ k$ — Dead Load per Panel Leg

$P_{sl} := 19\ psf \cdot 30\ ft \cdot 13\ ft = 7.41\ k$ — Snow Load per Panel Leg

$P_{pan} := \gamma_{conc} \cdot t_p \cdot \dfrac{(42\ ft \cdot 26\ ft - A_o)}{2} + \dfrac{A_o \cdot 15\ psf}{2} = 51.56\ k$ — Panel(s) Load per Leg

$P_{sei} := 25\ k$ — Seismic Load (including ρ, do not include vertical seismic effects)

Footing Dimensions

$W := 6 \cdot ft$ — Footing width

$L := 6 \cdot ft$ — Footing Length

$t := 18 \cdot in$ — Footing Thickness

$P_{ftg} := \gamma_{conc} \cdot L \cdot W \cdot t$ — Footing Weight

Service Loads - ASCE 7-16 2.4.1/2.4.5

$P_{ser} := P_{dl} + P_{pan} + P_{sl}$ — ASCE 7-16 2.4.1, LC 2

$P_{ser\_sei} := P_{pan} + P_{dl} + 0.75 \cdot P_{sl} + 0.525 \cdot P_{sei}$ — ASCE 7-16 2.4.5, LC 9

$P_{ser\_sei2} := P_{pan} + P_{dl} + 0.7\ P_{sei}$ — ASCE 7-16 2.4.5, LC 8

Ultimate Loads - ASCE 7-16 2.3.1/2.3.6

$P_u := 1.2 \cdot (P_{dl} + P_{pan}) + 1.6 \cdot P_{sl}$ — ASCE 7-16 2.3.1, LC 2

$P_{u\_sei} := (1.2 + 0.2\ S_{DS}) \cdot (P_{dl} + P_{pan}) + 0.2 \cdot P_{sl} + P_{sei}$ — ASCE 7-16 2.3.6, LC 6

Panel Footing

Soil Bearing - ACI 318-19 13.3.1.1

$$q := \max\left(\frac{P_{ser\_sei}}{\gamma_{temp} \cdot W \cdot L}, \frac{P_{ser\_sei2}}{\gamma_{temp} \cdot W \cdot L}, \frac{P_{ser}}{W \cdot L}\right)$$

Net bearing pressure

$$\frac{q}{q_{all}} = 0.738$$

$$Flag_b := \text{if}\left(q \le q_{all}, \text{"OK"}, \text{"NG!!"}\right)$$

$$Flag_b = \text{"OK"}$$

Flexure (per 1'-0" width) - ACI 318-19 13.2.6.4, 13.2.7.1

$$q_u := \max\left(\frac{P_{u\_sei}}{W \cdot L}, \frac{P_u}{W \cdot L}\right)$$

Ultimate bearing pressure

$$M_u := q_u \cdot \left(\frac{W - t_p}{2}\right)^2 \cdot \frac{1}{2} \cdot ft$$

Moment demand

$$\#_{reinf} := \text{Reinforcement size: \#5}$$

Size of reinforcement

$$A_s := \#_{reinf_0} \qquad d_s := \#_{reinf_1}$$

$$d := t - 3\ in - d_s \cdot 1.5 = 14.06\ in$$

Steel Depth

Guess Values

$$A_{sreq} := 1\ in^2$$

Constraints

$$0.9 \cdot A_{sreq} \cdot f_y \cdot \left(d - \frac{A_{sreq} \cdot f_y}{0.85 \cdot f_c \cdot 1\ ft \cdot 2}\right) - M_u = 0$$

Solver

$$A_{sreq} := \text{Find}\left(A_{sreq}\right)$$

Required area of steel for footing flexure

$$A_{smin} := 0.0018 \cdot t \cdot (1\ ft) = 0.39\ in^2$$

Min flexural reinforcement per ACI 318-19 8.6.1.1

$$sp := 9\ in$$

OC spacing of reinforcement

$$A_{s\_ft} := \frac{A_s \cdot (1\ ft)}{sp}$$

Area of reinforcement provided per foot of width

$$A_{sreq} := \max\left(A_{sreq}, A_{smin}\right)$$

$$\frac{A_{sreq}}{A_{s\_ft}} = 0.94$$

$$Flag_f := \text{if}\left(A_{s\_ft} \ge A_{sreq}, \text{"OK"}, \text{"NG!!"}\right)$$

$$Flag_f = \text{"OK"}$$

Beam Shear (per 1' width) - ACI 318-19 8.4.3, 22.5.5.1

$$\rho_w := \frac{A_{s\_ft}}{1\ ft \cdot d} = 0.0024$$

Ratio of longitudinal bars defined in R22.5.5

$$\lambda_s := 1.0$$

Size effect modification factor, eq 22.5.5.1.3
(Neglected per ACI 318-19 13.2.6.2)

$$V_u := \left(\frac{W - t_p}{2} - d\right) \cdot q_u \cdot ft$$

at "d" from panel face

$$\phi_v := 0.6$$

Strength reduction factor, 21.2.4.3

$$\phi V_c := \phi_v \cdot \left(8 \cdot \lambda_s \cdot \rho_w^{\frac{1}{3}} \cdot \sqrt{f_c \cdot psi} \cdot d\right) \cdot ft$$

Table 22.5.5.1 (c) - assumes Av < Av,min

$$\phi V_{c\_max} := \phi_v \cdot \left(5 \cdot \sqrt{f_c \cdot psi} \cdot d\right) \cdot ft$$

Maximum concrete shear strength, 22.5.5.1.1

$$\phi V_c := \text{if}\left(\phi V_c > \phi V_{c\_max}, \phi V_{c\_max}, \phi V_c\right)$$

$$\phi V_c = 6\ k$$

$$\frac{V_u}{\phi V_c} = 0.71$$

$$Flag_v := \text{if}\left(\phi V_c \geq V_u, \text{“OK”}, \text{“NG!!”}\right)$$

$$Flag_v = \text{“OK”}$$

Beam Shear (Beyond Leg Edge) - ACI 318-19 8.4.3, 22.5.5.1

$$\rho_w = 0.0024$$

Ratio of longitudinal bars defined in R22.5.5
Assumes same reinforcement each way

$$\phi_v := 0.6$$

Strength reduction factor, 21.2.4.3

$$leg := 5\ ft$$

Length of panel leg

$$V_u := (L - leg - d) \cdot q_u \cdot ft$$

Ultimate Shear at "d" from panel face
per unit width

$$\phi V_c := \phi_v \cdot \left(8 \cdot \lambda_s \cdot \rho_w^{\frac{1}{3}} \cdot \sqrt{f_c \cdot psi} \cdot d\right) \cdot ft$$

Table 22.5.5.1 (c) - assumes Av < Av,min

$\phi V_{c\_max} := \phi_v \cdot \left(5 \cdot \sqrt{f_c \cdot psi} \cdot d\right) \cdot ft$ — Maximum concrete shear strength, 22.5.5.1.1

$\phi V_c := \text{if}\left(\phi V_c > \phi V_{c\_max}, \phi V_{c\_max}, \phi V_c\right)$ — $\phi V_c = 6\ k$

$\dfrac{V_u}{\phi V_c} = -0.09$

$Flag_v := \text{if}\left(\phi V_c \geq V_u, \text{“OK”}, \text{“NG!!”}\right)$ — $Flag_v = \text{“OK”}$

Pad Footing - Panel 1, 2, 3 Entrances(Footing Type F5):

Panel Dimensions

$t_p := 8\ in$ — Panel Thickness

$A_o := 121\ ft^2$ — Door Area

Loading

$P_{dl} := 19\ psf \cdot 30\ ft \cdot 13\ ft = 7.41\ k$ — Dead Load per Leg

$P_{sl} := \left(14\ psf \cdot 30\ ft + 509\ plf\right) \cdot 13\ ft = 12.08\ k$ — Snow Load per Leg

$P_{pan} := \gamma_{conc} \cdot t_p \cdot \dfrac{\left(42\ ft \cdot 26\ ft - A_o\right)}{2} + \dfrac{A_o \cdot 15\ psf}{2} = 49.46\ k$ — Panel(s) Load per Leg

$P_{sei} := 25\ k$ — Seismic Load (including ρ, do not include vertical seismic effects)

Footing Dimensions

$W := 4 \cdot ft$ — Footing width

$L := 8.5 \cdot ft$ — Footing Length

$t := 12 \cdot in$ — Footing Thickness

$P_{ftg} := \gamma_{conc} \cdot L \cdot W \cdot t$ — Footing Weight

Service Loads - ASCE 7-16 2.4.1/2.4.5

$P_{ser} := P_{dl} + P_{pan} + P_{sl}$ — ASCE 7-16 2.4.1, LC 2

$P_{ser\_sei} := P_{pan} + P_{dl} + 0.75 \cdot P_{sl} + 0.525 \cdot P_{sei}$ — ASCE 7-16 2.4.5, LC 9

$P_{ser\_sei2} := P_{pan} + P_{dl} + 0.7\ P_{sei}$ — ASCE 7-16 2.4.5, LC 8

Panel Footing

Ultimate Loads - ASCE 7-16 2.3.1/2.3.6

$P_u := 1.2 \cdot (P_{dl} + P_{pan}) + 1.6 \cdot P_{sl}$ ASCE 7-16 2.3.1, LC 2

$P_{u\_sei} := (1.2 + 0.2\ S_{DS}) \cdot (P_{dl} + P_{pan}) + 0.2 \cdot P_{sl} + P_{sei}$ ASCE 7-16 2.3.6, LC 6

Soil Bearing - ACI 318-19 13.3.1.1

$q := \max\left(\frac{P_{ser\_sei}}{\gamma_{temp} \cdot W \cdot L}, \frac{P_{ser\_sei2}}{\gamma_{temp} \cdot W \cdot L}, \frac{P_{ser}}{W \cdot L}\right)$ Net bearing pressure $\frac{q}{q_{all}} = 0.811$

$Flag_b := \text{if}\left(q \le q_{all}, \text{“OK”}, \text{“NG!!”}\right)$ $Flag_b = \text{“OK”}$

Flexure (per 1'-0" width) - ACI 318-19 13.2.6.4, 13.2.7.1

$q_u := \max\left(\frac{P_{u\_sei}}{W \cdot L}, \frac{P_u}{W \cdot L}\right)$ Ultimate bearing pressure

$M_u := q_u \cdot \left(\frac{W - t_p}{2}\right)^2 \cdot \frac{1}{2} \cdot ft$ Moment demand

$\#_{reinf} :=$ Reinforcement size: #5 Size of reinforcement

$A_s := \#_{reinf_0}$ $d_s := \#_{reinf_1}$

$d := t - 3\ in - d_s \cdot 1.5 = 8.06\ in$ Steel Depth

Solver: Guess Values / Constraints

$A_{sreq} := 1\ in^2$

$0.9 \cdot A_{sreq} \cdot f_y \cdot \left(d - \frac{A_{sreq} \cdot f_y}{0.85 \cdot f_c \cdot 1\ ft \cdot 2}\right) - M_u = 0$ Required area of steel for footing flexure

$A_{sreq} := \text{Find}(A_{sreq})$

$A_{smin} := 0.0018 \cdot t \cdot (1\ ft) = 0.26\ in^2$ Min flexural reinforcement per ACI 318-19 8.6.1.1

$sp := 12\ in$ OC spacing of reinforcement

$A_{s\_ft} := \frac{A_s \cdot (1\ ft)}{sp}$ Area of reinforcement provided per foot of width

$A_{sreq} := \max(A_{sreq}, A_{smin})$ $\quad \frac{A_{sreq}}{A_{s\_ft}} = 0.84$

$Flag_f := \text{if}(A_{s\_ft} \geq A_{sreq}, \text{“OK”}, \text{“NG!!”})$ $\quad Flag_f = \text{“OK”}$

Beam Shear (per 1' width) - ACI 318-19 8.4.3, 22.5.5.1

$\rho_w := \frac{A_{s\_ft}}{1\ ft \cdot d} = 0.0032$ Ratio of longitudinal bars defined in R22.5.5

$\lambda_s := 1.0$ Size effect modification factor, eq 22.5.5.1.3 (Neglected per ACI 318-19 13.2.6.2)

$V_u := \left(\frac{W - t_p}{2} - d\right) \cdot q_u \cdot ft$ at "d" from panel face

$\phi_v := 0.6$ Strength reduction factor, 21.2.4.3

$\phi V_c := \phi_v \cdot \left(8 \cdot \lambda_s \cdot \rho_w^{\frac{1}{3}} \cdot \sqrt{f_c \cdot psi} \cdot d\right) \cdot ft$ Table 22.5.5.1 (c) - assumes Av < Av,min

$\phi V_{c\_max} := \phi_v \cdot \left(5 \cdot \sqrt{f_c \cdot psi} \cdot d\right) \cdot ft$ Maximum concrete shear strength, 22.5.5.1.1

$\phi V_c := \text{if}(\phi V_c > \phi V_{c\_max}, \phi V_{c\_max}, \phi V_c)$ $\quad \phi V_c = 3.7\ k$

$\frac{V_u}{\phi V_c} = 0.82$

$Flag_v := \text{if}(\phi V_c \geq V_u, \text{“OK”}, \text{“NG!!”})$ $\quad Flag_v = \text{“OK”}$

Beam Shear (Beyond Leg Edge) - ACI 318-19 8.4.3, 22.5.5.1

$\rho_w = 0.0032$ Ratio of longitudinal bars defined in R22.5.5
Assumes same reinforcement each way

$\phi_v := 0.6$ Strength reduction factor, 21.2.4.3

$leg := 8.5\ ft$ — Length of panel leg

$V_u := (L - leg - d) \cdot q_u \cdot ft = -2.1\ k$ — Ultimate Shear at "d" from panel face per unit width

$\phi V_c := \phi_v \cdot \left(8 \cdot \lambda_s \cdot \rho_w^{\frac{1}{3}} \cdot \sqrt{f_c \cdot psi} \cdot d\right) \cdot ft$ — Table 22.5.5.1 (c) - assumes Av < Av,min

$\phi V_{c\_max} := \phi_v \cdot \left(5 \cdot \sqrt{f_c \cdot psi} \cdot d\right) \cdot ft$ — Maximum concrete shear strength, 22.5.5.1.1

$\phi V_c := \text{if}\left(\phi V_c > \phi V_{c\_max}, \phi V_{c\_max}, \phi V_c\right)$ — $\phi V_c = 3.7\ k$

$\frac{V_u}{\phi V_c} = -0.55$

$Flag_v := \text{if}\left(\phi V_c \ge V_u, \text{“OK”}, \text{“NG!!”}\right)$ — $Flag_v = \text{“OK”}$

Continuous Footing - Dock Door Panels (Footing Type F2)

Panel Dimensions

$w_{leg} := 2\ ft$ — Width of governing panel leg

$t_p := 9.5 \cdot in$ — Panel Thickness

Loading

$P_{dl} := 19\ psf \cdot 30\ ft \cdot 7\ ft = 3.99\ k$ — Dead Load

$P_{sl} := 20\ psf \cdot 30\ ft \cdot 7\ ft = 4.2\ k$ — Snow Load

$P_{pan} := \gamma_{conc} \cdot t_p \cdot 40.75\ ft \cdot 7\ ft = 33.87\ k$ — Panel(s) Load

$P_{sei} := 22\ k$ — Seismic Load (including ρ, do not include vertical seismic effects)

Footing Dimensions

$W := 2.5\ ft$ — Footing Width

$t := 12 \cdot in$ — Footing Thickness

$L := w_{leg} + 5\ ft = 7\ ft$ — Footing Length

$P_{ftg} := \gamma_{conc} \cdot L \cdot W \cdot t$ Footing Weight

Service Load - ASCE 7-16 2.4.1/2.4.5

Per Geotech Report, Net bearing, Ignore Self Weight

$P_{ser} := P_{dl} + P_{pan} + P_{sl}$ ASCE 7-16 2.4.1, LC 2

$P_{ser\_sei} := P_{pan} + P_{dl} + 0.75 \cdot P_{sl} + 0.525 \cdot P_{sei}$ ASCE 7-16 2.4.5, LC 9

$P_{ser\_sei2} := P_{pan} + P_{dl} + 0.7\ P_{sei}$ ASCE 7-16 2.4.5, LC 8

Ultimate Loads - ASCE 7-16 2.3.1/2.3.6

$P_u := 1.2 \cdot (P_{dl} + P_{pan}) + 1.6 \cdot P_{sl} = 52.2\ k$ ASCE 7-16 2.3.1, LC 2

$P_{u\_sei} := (1.2 + 0.2\ S_{DS}) \cdot (P_{dl} + P_{pan}) + 0.2 \cdot P_{sl} + P_{sei}$ ASCE 7-16 2.3.6, LC 6

Soil Bearing - ACI 318-19 13.3.1.1

$$q := \max\left(\frac{P_{ser\_sei}}{\gamma_{temp} \cdot W \cdot L}, \frac{P_{ser\_sei2}}{\gamma_{temp} \cdot W \cdot L}, \frac{P_{ser}}{W \cdot L}\right)$$

Net bearing pressure

$$\frac{q}{q_{all}} = 0.961$$

$Flag_b := \text{if}\ (q \le q_{all}, \text{“OK”}, \text{“NG!!”})$

$Flag_b = \text{“OK”}$

Flexure (per 1' width) - ACI 318-19 13.2.6.4, 13.2.7.1

$$q_u := \max\left(\frac{P_{u\_sei}}{W \cdot L}, \frac{P_u}{W \cdot L}\right) = 4.3\ ksf$$

Ultimate bearing pressure

$$M_u := q_u \cdot \left(\frac{W - t_p}{2}\right)^2 \cdot \frac{1}{2} \cdot ft$$

Moment demand

$\#_{reinf} :=$ Reinforcement size: #5 Size of reinforcement

$A_s := \#_{reinf_0}$ $d_s := \#_{reinf_1}$

$d := t - 3\ in - d_s \cdot 1.5 = 8.1\ in$ Steel Depth

Guess Values

$A_{sreq} := 1\ in^2$

Constraints

$$A_{sreq} \cdot f_y \cdot \left(d - \frac{A_{sreq} \cdot f_y}{.85 \cdot f_c \cdot 1\ ft \cdot 2}\right) \cdot .9 - M_u = 0$$

Required area of steel for footing flexure

Solver

$A_{sreq} := \text{Find}\ (A_{sreq}) = 0.04\ in^2$

Panel Footing

$A_{smin} := 0.0018 \cdot t \cdot (1\ ft) = 0.26\ in^2$

Min flexural reinforcement per ACI 318-19 8.6.1.1

$sp := 12\ in$

OC spacing of reinforcement

$No_{long} := \frac{W}{sp} + 1 = 3.5$

Number of longitudinal bars

$A_{s\_ft} := \frac{A_s \cdot (1\ ft)}{sp} = 0.31\ in^2$

Area of reinforcement provided per foot of width

$A_{sreq} := \max\left(A_{sreq}, A_{smin}\right) = 0.26\ in^2$

$\frac{A_{sreq}}{A_{s\_ft}} = 0.8$

$Flag_f := \text{if}\left(A_{s\_ft} \ge A_{sreq}, \text{“OK”}, \text{“NG!!”}\right)$

$Flag_f = \text{“OK”}$

Beam Shear (per 1' width) - ACI 318-19 8.4.3, 22.5.5.1

$\rho_w := \frac{A_{s\_ft}}{1\ ft \cdot d} = 0.0032$

Ratio of longitudinal bars defined in R22.5.5

$\lambda_s := 1.0$

Size effect modification factor, eq 22.5.5.1.3 (Neglected per ACI 318-19 13.2.6.2)

$V_u := \left(\frac{W - t_p}{2} - d\right) \cdot q_u \cdot ft = 0.8\ k$

at "d" from column face

$\phi_v := 0.6$

Strength reduction factor, 21.2.4.3

$\phi V_c := \phi_v \cdot \left(8 \cdot \lambda_s \cdot \rho_w^{\frac{1}{3}} \cdot \sqrt{f_c \cdot psi} \cdot d\right) \cdot ft = 3.7\ k$

Table 22.5.5.1 (c) - assumes Av < Av,min

$\phi V_{c\_max} := \phi_v \left(5 \sqrt{f_c \cdot psi} \cdot d\right) \cdot ft = 15.9\ k$

Maximum concrete shear strength, 22.5.5.1.1

$\phi V_c := \text{if}\left(\phi V_c > \phi V_{c\_max}, \phi V_{c\_max}, \phi V_c\right)$

$\phi V_c = 3.7\ k$

$\frac{V_u}{\phi V_c} = 0.207$

$Flag_v := \text{if}\left(\phi V_c \ge V_u, \text{“OK”}, \text{“NG!!”}\right)$

$Flag_v = \text{“OK”}$

Continuous Footing - Solid Panels w/ Drift Reaction (Footing Type F2)

Panel Dimensions

$w_{leg} := 26\ ft$ — Width of governing panel leg

$t_p := 8 \cdot in$ — Panel Thickness

Loading

$P_{dl} := 19\ psf \cdot 30\ ft \cdot 26\ ft = 14.82\ k$ — Dead Load

$P_{sl} := 14\ psf \cdot 30\ ft \cdot 26\ ft + (425\ plf + 57\ plf)\ 30\ ft = 25.38\ k$ — Snow Load

$P_{pan} := \gamma_{conc} \cdot t_p \cdot 45\ ft \cdot 26\ ft = 117\ k$ — Panel(s) Load

$P_{sei} := 33\ k$ — Seismic Load (including ρ, do not include vertical seismic effects)

Footing Dimensions

$W := 2.5\ ft$ — Footing Width

$t := 12 \cdot in$ — Footing Thickness

$L := w_{leg} + 1\ ft = 27\ ft$ — Footing Length

$P_{ftg} := \gamma_{conc} \cdot L \cdot W \cdot t$ — Footing Weight

Service Load - ASCE 7-16 2.4.1/2.4.5

Per Geotech Report, Net bearing, Ignore Self Weight

$P_{ser} := P_{dl} + P_{pan} + P_{sl}$ — ASCE 7-16 2.4.1, LC 2

$P_{ser\_sei} := P_{pan} + P_{dl} + 0.75 \cdot P_{sl} + 0.525 \cdot P_{sei}$ — ASCE 7-16 2.4.5, LC 9

$P_{ser\_sei2} := P_{pan} + P_{dl} + 0.7\ P_{sei}$ — ASCE 7-16 2.4.5, LC 8

Ultimate Loads - ASCE 7-16 2.3.1/2.3.6

$P_u := 1.2 \cdot (P_{dl} + P_{pan}) + 1.6 \cdot P_{sl}$ — ASCE 7-16 2.3.1, LC 2

$P_{u\_sei} := (1.2 + 0.2\ S_{DS}) \cdot (P_{dl} + P_{pan}) + 0.2 \cdot P_{sl} + P_{sei}$ — ASCE 7-16 2.3.6, LC 6

Soil Bearing - ACI 318-19 13.3.1.1

$q := \max\left(\frac{P_{ser\_sei}}{\ldots}, \frac{P_{ser\_sei2}}{\ldots}, \frac{P_{ser}}{\ldots}\right)$ q

$\left(\gamma_{temp} \cdot W \cdot L , \gamma_{temp} \cdot W \cdot L , W \cdot L\right)$ Net bearing pressure $\frac{q}{q_{all}} = 0.932$

$Flag_b := \text{if}\left(q \le q_{all}, \text{"OK"}, \text{"NG!!"}\right)$ $Flag_b = \text{"OK"}$

Flexure (per 1' width) - ACI 318-19 13.2.6.4, 13.2.7.1

$q_u := \max\left(\frac{P_{u\_sei}}{W \cdot L}, \frac{P_u}{W \cdot L}\right) = 3.2\ ksf$ Ultimate bearing pressure

$M_u := q_u \cdot \left(\frac{W - t_p}{2}\right)^2 \cdot \frac{1}{2} \cdot ft$ Moment demand

$\#_{reinf}$:= Reinforcement size: #5 Size of reinforcement

$A_s := \#_{reinf_0}$ $d_s := \#_{reinf_1}$

$d := t - 3\ in - d_s \cdot 1.5 = 8.1\ in$ Steel Depth

Solve Block — Guess Values / Constraints / Solver

$A_{sreq} := 1\ in^2$

$A_{sreq} \cdot f_y \cdot \left(d - \frac{A_{sreq} \cdot f_y}{.85 \cdot f_c \cdot 1\ ft \cdot 2}\right) \cdot .9 - M_u = 0$ Required area of steel for footing flexure

$A_{sreq} := \text{Find}\left(A_{sreq}\right) = 0.04\ in^2$

$A_{smin} := 0.0018 \cdot t \cdot (1\ ft) = 0.26\ in^2$ Min flexural reinforcement per ACI 318-19 8.6.1.1

$sp := 12\ in$ OC spacing of reinforcement

$No_{long} := \frac{W}{sp} + 1 = 3.5$ Number of longitudinal bars

$A_{s\_ft} := \frac{A_s \cdot (1\ ft)}{sp} = 0.31\ in^2$ Area of reinforcement provided per foot of width

$A_{sreq} := \max\left(A_{sreq}, A_{smin}\right) = 0.26\ in^2$ $\frac{A_{sreq}}{A_{s\_ft}} = 0.8$

$Flag_f := \text{if}\left(A_{s\_ft} \ge A_{sreq}, \text{"OK"}, \text{"NG!!"}\right)$ $Flag_f = \text{"OK"}$

Beam Shear (per 1' width) - ACI 318-19 8.4.3, 22.5.5.1

A_{s_ft}

$\rho_w := \frac{\ldots}{1\ ft \cdot d} = 0.0032$ — Ratio of longitudinal bars defined in R22.5.5

$\lambda_s := 1.0$ — Size effect modification factor, eq 22.5.5.1.3 (Neglected per ACI 318-19 13.2.6.2)

$V_u := \left(\frac{W - t_p}{2} - d\right) \cdot q_u \cdot ft = 0.8\ k$ — at "d" from column face

$\phi_v := 0.6$ — Strength reduction factor, 21.2.4.3

$\phi V_c := \phi_v \cdot \left(8 \cdot \lambda_s \cdot \rho_w^{\frac{1}{3}} \cdot \sqrt{f_c \cdot psi} \cdot d\right) \cdot ft = 3.7\ k$ — Table 22.5.5.1 (c) - assumes Av < Av,min

$\phi V_{c\_max} := \phi_v \left(5\ \sqrt{f_c \cdot psi} \cdot d\right) \cdot ft = 15.9\ k$ — Maximum concrete shear strength, 22.5.5.1.1

$\phi V_c := \text{if}\left(\phi V_c > \phi V_{c\_max}, \phi V_{c\_max}, \phi V_c\right)$

$\phi V_c = 3.7\ k$

$\frac{V_u}{\phi V_c} = 0.211$

$Flag_v := \text{if}\left(\phi V_c \geq V_u, \text{"OK"}, \text{"NG!!"}\right)$

$Flag_v = \text{"OK"}$

Continuous Footing - Blade Wall Panels (Footing Type F3)

Panel Dimensions

$w_{leg} := 1\ ft$ — Width of governing panel leg

$t_p := 12.1 \cdot in$ — Panel Thickness (avg thickness)

Loading

$P_{dl} := 19\ psf \cdot 30\ ft \cdot 1\ ft = 0.57\ k$ — Dead Load

$P_{sl} := 14\ psf \cdot 30\ ft \cdot 1\ ft + .76\ k = 1.18\ k$ — Snow Load

$P_{pan} := \gamma_{conc} \cdot t_p \cdot 40.75\ ft \cdot 1\ ft = 6.16\ k$ — Panel(s) Load

$P_{sei} := 5\ k$ — Seismic Load (including ρ, do not include vertical seismic effects)

Footing Dimensions

Panel Footing

$W := 4.5\ ft$ — Footing Width

$t := 12 \cdot in$ — Footing Thickness

$L := w_{leg} = 1\ ft$ — Footing Length

$P_{ftg} := \gamma_{conc} \cdot L \cdot W \cdot t$ — Footing Weight

Service Load - ASCE 7-16 2.4.1/2.4.5

Per Geotech Report, Net bearing, Ignore Self Weight

$P_{ser} := P_{dl} + P_{pan} + P_{sl}$ — ASCE 7-16 2.4.1, LC 2

$P_{ser\_sei} := P_{pan} + P_{dl} + 0.75 \cdot P_{sl} + 0.525 \cdot P_{sei}$ — ASCE 7-16 2.4.5, LC 9

$P_{ser\_sei2} := P_{pan} + P_{dl} + 0.7\ P_{sei}$ — ASCE 7-16 2.4.5, LC 8

Ultimate Loads - ASCE 7-16 2.3.1/2.3.6

$P_u := 1.2 \cdot (P_{dl} + P_{pan}) + 1.6 \cdot P_{sl} = 10\ k$ — ASCE 7-16 2.3.1, LC 2

$P_{u\_sei} := (1.2 + 0.2\ S_{DS}) \cdot (P_{dl} + P_{pan}) + 0.2 \cdot P_{sl} + P_{sei}$ — ASCE 7-16 2.3.6, LC 6

Soil Bearing - ACI 318-19 13.3.1.1

$$q := \max\left(\frac{P_{ser\_sei}}{\gamma_{temp} \cdot W \cdot L}, \frac{P_{ser\_sei2}}{\gamma_{temp} \cdot W \cdot L}, \frac{P_{ser}}{W \cdot L}\right)$$

Net bearing pressure

$$\frac{q}{q_{all}} = 0.703$$

$$Flag_b := \text{if}\left(q \le q_{all}, \text{"OK"}, \text{"NG!!"}\right)$$

$$Flag_b = \text{"OK"}$$

Flexure (per 1' width) - ACI 318-19 13.2.6.4, 13.2.7.1

$$q_u := \max\left(\frac{P_{u\_sei}}{W \cdot L}, \frac{P_u}{W \cdot L}\right) = 3.2\ ksf$$

Ultimate bearing pressure

$$M_u := q_u \cdot \left(\frac{W - t_p}{2}\right)^2 \cdot \frac{1}{2} \cdot ft$$

Moment demand

$\#_{reinf} :=$ Reinforcement size: #5 — Size of reinforcement

$A_s := \#_{reinf_0}$ $\quad d_s := \#_{reinf_1}$

$d := t - 3\ in - d_s \cdot 1.5 = 8.1\ in$ — Steel Depth

Solve Constraints Guess Values

$A_{sreq} := 1\ in^2$

$A_{sreq} \cdot f_y \cdot \left(d - \frac{A_{sreq} \cdot f_y}{.85 \cdot f_c \cdot 1\ ft \cdot 2}\right) \cdot .9 - M_u = 0$ Required area of steel for footing flexure

$A_{sreq} := \text{Find}\left(A_{sreq}\right) = 0.14\ in^2$

$A_{smin} := 0.0018 \cdot t \cdot (1\ ft) = 0.26\ in^2$ Min flexural reinforcement per ACI 318-19 8.6.1.1

$sp := 12\ in$ OC spacing of reinforcement

$No_{long} := \frac{W}{sp} + 1 = 5.5$ Number of longitudinal bars

$A_{s\_ft} := \frac{A_s \cdot (1\ ft)}{sp} = 0.31\ in^2$ Area of reinforcement provided per foot of width

$A_{sreq} := \max\left(A_{sreq}, A_{smin}\right) = 0.26\ in^2$

$\frac{A_{sreq}}{A_{s\_ft}} = 0.8$

$Flag_f := \text{if}\left(A_{s\_ft} \ge A_{sreq}, \text{“OK”}, \text{“NG!!”}\right)$

$Flag_f = \text{“OK”}$

Beam Shear (per 1' width) - ACI 318-19 8.4.3, 22.5.5.1

$\rho_w := \frac{A_{s\_ft}}{1\ ft \cdot d} = 0.0032$ Ratio of longitudinal bars defined in R22.5.5

$\lambda_s := 1.0$ Size effect modification factor, eq 22.5.5.1.3 (Neglected per ACI 318-19 13.2.6.2)

$V_u := \left(\frac{W - t_p}{2} - d\right) \cdot q_u \cdot ft = 3.4\ k$ at "d" from column face

$\phi_v := 0.6$ Strength reduction factor, 21.2.4.3

$\phi V_c := \phi_v \cdot \left(8 \cdot \lambda_s \cdot \rho_w^{\frac{1}{3}} \cdot \sqrt{f_c \cdot psi} \cdot d\right) \cdot ft = 3.7\ k$ Table 22.5.5.1 (c) - assumes Av < Av,min

$\phi V_{c\_max} := \phi_v \left(5 \sqrt{f_c \cdot psi} \cdot d\right) \cdot ft = 15.9\ k$ Maximum concrete shear strength, 22.5.5.1.1

$\phi V_c := \text{if}\left(\phi V_c > \phi V_{c\_max}, \phi V_{c\_max}, \phi V_c\right)$

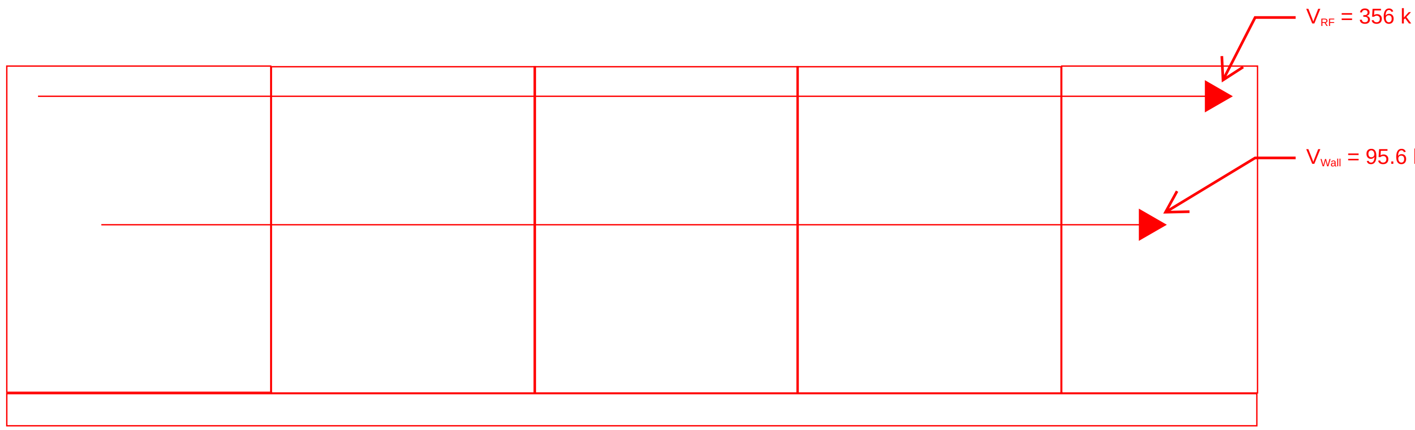
$\phi V_c = 3.7\ k$

$$\frac{V_u}{\phi V_c} = 0.92$$

$$Flag_v := \text{if}\left(\phi V_c \ge V_u, \text{"OK"}, \text{"NG!!"}\right)$$

$$Flag_v = \text{"OK"}$$

Global Soil Bearing Check



M_{OT} = 356 k * 39 ft + 95.6 k * 40.5 ft / 2 = 15,820 k-ft

P_{Wall} = 578 k
$P_{RF,DL}$ = (10.9 psf * 10 ft / 2) x 118.67 ft = 6.5 k
$P_{RF,SL}$ = (14.9 psf * 5 ft) x 118.67 ft = 8.84 k

M_{Resist} = 593 k X 126 ft / 2 = 37,400 k-ft

M_{OT} / 1.4 = 11,300 k-ft

M_{resist} > M_{OT} / 1.4 = NOT OVERTURNING

c_k = L/6 = 126 ft / 6 = 21 ft

in the kern

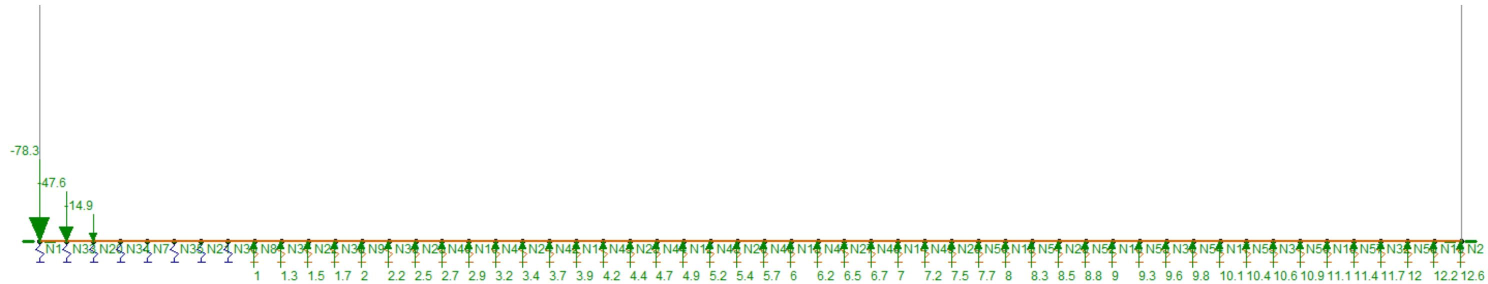
e = (M_{resist} + M_{OT} / 1.4) / P_{tot} - L / 2 = (37,400 k-ft + 11,300 k-ft) / (593 k) - (126 ft)/2 = 19.1 ft

q = (P_{tot} / (W_{ftg} x L_{ftg}))(1+6e/L_{ftg}) = (593 k / (4.5 ft x 126 ft))(1 + 6(19.1ft)/(126ft)) = 2.00 ksf < 3.33 ksf $\longrightarrow$ OKAY

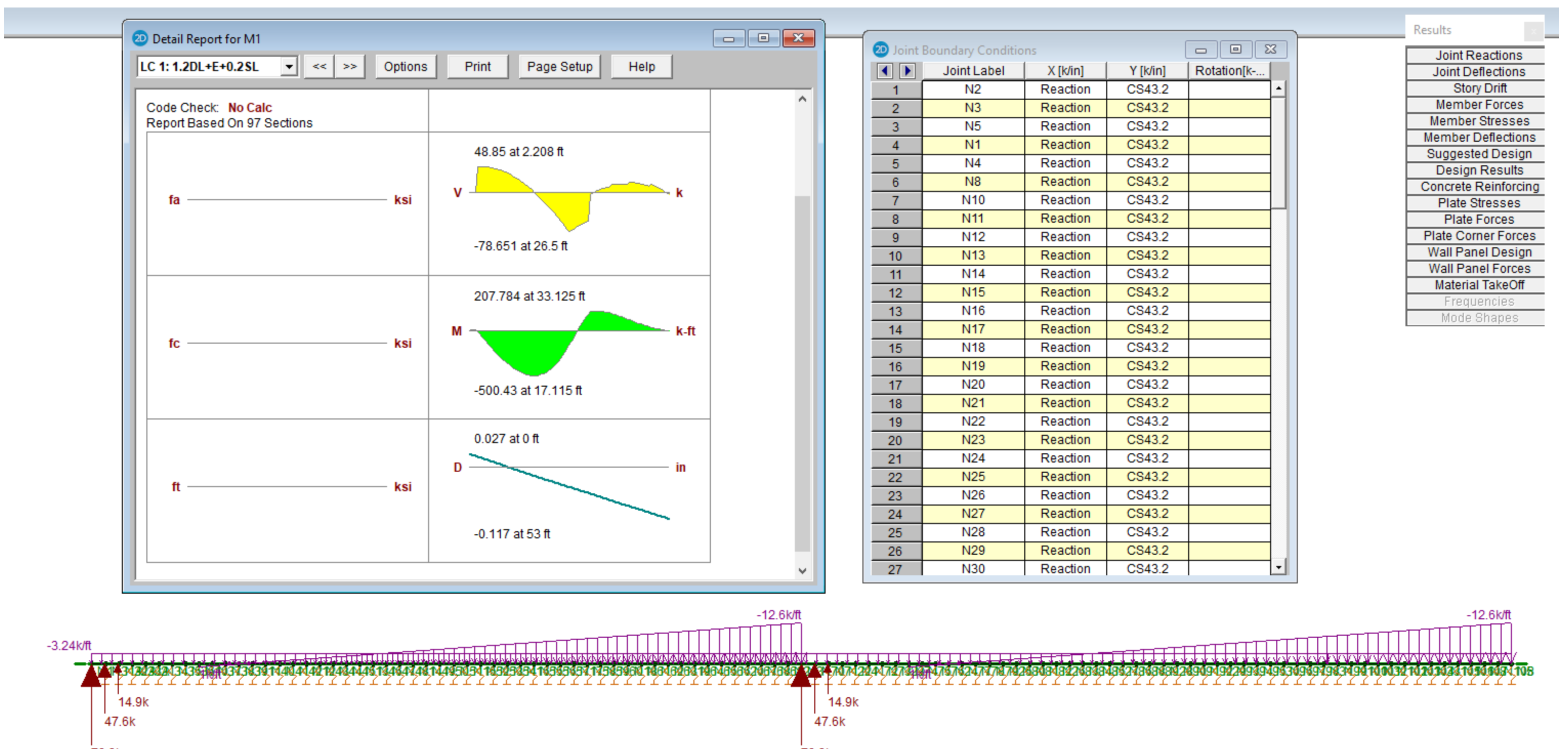
Local Footing Check @ Holdowns

FACTORED REACTIONS FROM PANEL MODEL (SEE RISA REPORT BELOW):

43.2 k/in Compression Spring



FOUNDATION MODEL WITH LOAD FROM PANEL:



	Joint Label	X [k/in]	Y [k/in]	Rotation[k-...
1	N2	Reaction	CS43.2	
2	N3	Reaction	CS43.2	
3	N5	Reaction	CS43.2	
4	N1	Reaction	CS43.2	
5	N4	Reaction	CS43.2	
6	N8	Reaction	CS43.2	
7	N10	Reaction	CS43.2	
8	N11	Reaction	CS43.2	
9	N12	Reaction	CS43.2	
10	N13	Reaction	CS43.2	
11	N14	Reaction	CS43.2	
12	N15	Reaction	CS43.2	
13	N16	Reaction	CS43.2	
14	N17	Reaction	CS43.2	
15	N18	Reaction	CS43.2	
16	N19	Reaction	CS43.2	
17	N20	Reaction	CS43.2	
18	N21	Reaction	CS43.2	
19	N22	Reaction	CS43.2	
20	N23	Reaction	CS43.2	
21	N24	Reaction	CS43.2	
22	N25	Reaction	CS43.2	
23	N26	Reaction	CS43.2	
24	N27	Reaction	CS43.2	
25	N28	Reaction	CS43.2	
26	N29	Reaction	CS43.2	
27	N30	Reaction	CS43.2	

Joint Coordinates and Temperatures

	Label	X [ft]	Y [ft]	Temp [F]
1	N1	0	0	0
2	N2	26.5	0	0
3	N3	26.5	40.5	0
4	N4	0	40.5	0
5	N5	0	38.75	0
6	N6	26.5	38.75	0
7	N7	2	0	0
8	N8	4	0	0
9	N9	6	0	0
10	N10	8	0	0
11	N11	10	0	0
12	N12	12	0	0
13	N13	14	0	0
14	N14	16	0	0
15	N15	18	0	0
16	N16	20	0	0
17	N17	22	0	0
18	N18	24	0	0
19	N19	26	0	0
20	N20	1	0	0
21	N21	3	0	0
22	N22	5	0	0
23	N23	7	0	0
24	N24	9	0	0
25	N25	11	0	0
26	N26	13	0	0
27	N27	15	0	0
28	N28	17	0	0
29	N29	19	0	0
30	N30	21	0	0
31	N31	23	0	0
32	N32	25	0	0
33	N33	0.5	0	0
34	N34	1.5	0	0
35	N35	2.5	0	0
36	N36	3.5	0	0
37	N37	4.5	0	0
38	N38	5.5	0	0
39	N39	6.5	0	0
40	N40	7.5	0	0
41	N41	8.5	0	0
42	N42	9.5	0	0
43	N43	10.5	0	0
44	N44	11.5	0	0
45	N45	12.5	0	0
46	N46	13.5	0	0
47	N47	14.5	0	0
48	N48	15.5	0	0
49	N49	16.5	0	0
50	N50	17.5	0	0
51	N51	18.5	0	0

Joint Coordinates and Temperatures (Continued)

	Label	X [ft]	Y [ft]	Temp [F]
52	N52	19.5	0	0
53	N53	20.5	0	0
54	N54	21.5	0	0
55	N55	22.5	0	0
56	N56	23.5	0	0
57	N57	24.5	0	0
58	N58	25.5	0	0

Joint Boundary Conditions

	Joint Label	X [k/in]	Y [k/in]	Rotation[k-ft/rad]
1	N1	Reaction	TS1e+5	
2	N2	Reaction	CS43.2	
3	N7		TS1e+5	
4	N8		CS43.2	
5	N9		CS43.2	
6	N10		CS43.2	
7	N11		CS43.2	
8	N12		CS43.2	
9	N13		CS43.2	
10	N14		CS43.2	
11	N15		CS43.2	
12	N16		CS43.2	
13	N17		CS43.2	
14	N18		CS43.2	
15	N19		CS43.2	
16	N20		TS1e+5	
17	N21		TS1e+5	
18	N22		CS43.2	
19	N23		CS43.2	
20	N24		CS43.2	
21	N25		CS43.2	
22	N26		CS43.2	
23	N27		CS43.2	
24	N28		CS43.2	
25	N29		CS43.2	
26	N30		CS43.2	
27	N31		CS43.2	
28	N32		CS43.2	
29	N33		TS1e+5	
30	N34		TS1e+5	
31	N35		TS1e+5	
32	N36		TS1e+5	
33	N37		CS43.2	
34	N38		CS43.2	
35	N39		CS43.2	
36	N40		CS43.2	
37	N41		CS43.2	
38	N42		CS43.2	
39	N43		CS43.2	
40	N44		CS43.2	

Joint Boundary Conditions (Continued)

	Joint Label	X [k/in]	Y [k/in]	Rotation[k-ft/rad]
41	N45		CS43.2	
42	N46		CS43.2	
43	N47		CS43.2	
44	N48		CS43.2	
45	N49		CS43.2	
46	N50		CS43.2	
47	N51		CS43.2	
48	N52		CS43.2	
49	N53		CS43.2	
50	N54		CS43.2	
51	N55		CS43.2	
52	N56		CS43.2	
53	N57		CS43.2	
54	N58		CS43.2	

Wall Panel Data

	Label	A Joint	B Joint	C Joint	D Joint	Material Type	Material Set	Thicknes...	Design Rule	Panel/Spacing
1	WP1	N4	N3	N2	N1	Concrete	Conc4000NW	8.75	Typical	N/A

Wall Panel Advanced Data

	Label	Seismic Rule	Design Method	SSAF	In-plane Icr Factor	K	Cm
1	WP1	None	N/A	N/A			

Joint Loads and Enforced Displacements (BLC 1 : Stiffness)

	Joint Label	L,D,M	Direction	Magnitude[(k,k-ft), (in,rad), (k*s^2/ft, k*s^2*ft) Inactive	Inactive
1	N5	L	X	50	Active
2	N6	L	X	50	Active

Joint Loads and Enforced Displacements (BLC 6 : E)

	Joint Label	L,D,M	Direction	Magnitude[(k,k-ft), (in,rad), (k*s^2/ft, k*s^2*ft) Inactive	Inactive
1	N5	L	X	39.6	Active
2	N6	L	X	39.6	Active

Wall Panel Distributed Loads (BLC 2 : DL)

	Wall Label	Direction	Start Magnitude[k...	End Magnitude[k/...	Start Location[ft,%]	End Location[ft,%]	Inactive
1	WP1(40.5ft)	Y	-0.05	-0.05	0	26.5	Active

Wall Panel Distributed Loads (BLC 4 : SL)

	Wall Label	Direction	Start Magnitude[k...	End Magnitude[k/...	Start Location[ft,%]	End Location[ft,%]	Inactive
1	WP1(40.5ft)	Y	-0.5	-0.5	0	26.5	Active

Basic Load Cases

	BLC Description	Category	X Gravity	Y Gravity	Joint	Point	Distributed
1	Stiffness	None			2		
2	DL	None		-1			1
3	LL	None					
4	SL	None					1
5	W	None					
6	E	None	0.169	-0.169	2		

Load Combinations

	Description	S...	PDelta	SRSS	B...	Fa...	B...	Fa...	B...	Fa...	B...	Fa...	B...	Fa...	B...	Fa...	B...	Fa...	B...	Fa...	B...	Fa...	B...	Fa...
1	Stiffness	Yes	Y		1	1																		
2	1.2D+1.6L+ 0.5S	Yes	Y		2	1.2	3	1.6	4	0.5														
3	1.2D+1.6S+0.5L	Yes	Y		2	1.2	3	0.5	4	1.6														
4	1.2D+W+0.5L+0.5S	Yes	Y		2	1.2	3	0.5	4	0.5	5	1												
5	0.9D+W	Yes	Y		2	0.9	5	1																
6	1.2D+E+0.5L+0.2S	Yes	Y		2	1.2	3	0.5	4	0.2	6	1												
7	0.9D+E	Yes	Y		2	0.9	6	1																
8	D+L+S	Yes	Y		2	1	3	1	4	1														
9	D+L+0.6W	Yes	Y		2	1	3	1	5	0.6														
10	D+L+0.6W+S/2	Yes	Y		2	1	3	1	4	0.5	5	0.6												
11	D+L+S+0.3W	Yes	Y		2	1	3	1	4	1	5	0.3												
12	D+L+S+E/1.4	Yes	Y		2	1	3	1	4	1	6	0.7...												
13	0.9D+E/1.4	Yes	Y		2	0.9	6	0.7...																
14	DL	Yes	Y		2	1																		
15	Drift: D+E+0.5L+0.2S	Yes	Y		2	1	6	1	3	0.25	4	0.2												

Node Reactions (By Combination)

	LC	Node Label	X [k]	Y [k]	MZ [k-ft]
1	6	N1	-73.391	-78.296	0
2	6	N2	-25.6	12.591	0
3	6	N7	0	0	0
4	6	N8	0	1.049	0
5	6	N9	0	1.972	0
6	6	N10	0	2.94	0
7	6	N11	0	3.931	0
8	6	N12	0	4.937	0
9	6	N13	0	5.953	0
10	6	N14	0	6.977	0
11	6	N15	0	8.008	0
12	6	N16	0	9.045	0
13	6	N17	0	10.089	0
14	6	N18	0	11.143	0
15	6	N19	0	12.237	0
16	6	N20	0	-14.872	0
17	6	N21	0	0	0
18	6	N22	0	1.502	0
19	6	N23	0	2.452	0
20	6	N24	0	3.433	0
21	6	N25	0	4.433	0

Node Reactions (By Combination) (Continued)

	LC	Node Label	X [k]	Y [k]	MZ [k-ft]
22	6	N26	0	5.444	0
23	6	N27	0	6.465	0
24	6	N28	0	7.492	0
25	6	N29	0	8.526	0
26	6	N30	0	9.566	0
27	6	N31	0	10.614	0
28	6	N32	0	11.68	0
29	6	N33	0	-47.599	0
30	6	N34	0	0	0
31	6	N35	0	0	0
32	6	N36	0	0	0
33	6	N37	0	1.273	0
34	6	N38	0	1.735	0
35	6	N39	0	2.211	0
36	6	N40	0	2.695	0
37	6	N41	0	3.186	0
38	6	N42	0	3.682	0
39	6	N43	0	4.182	0
40	6	N44	0	4.684	0
41	6	N45	0	5.19	0
42	6	N46	0	5.698	0
43	6	N47	0	6.209	0
44	6	N48	0	6.72	0
45	6	N49	0	7.235	0
46	6	N50	0	7.75	0
47	6	N51	0	8.267	0
48	6	N52	0	8.786	0
49	6	N53	0	9.306	0
50	6	N54	0	9.827	0
51	6	N55	0	10.352	0
52	6	N56	0	10.878	0
53	6	N57	0	11.411	0
54	6	N58	0	11.954	0
55	6	Totals:	-98.991	164.942	
56	6	COG (ft):	X: 13.25	Y: 20.771	

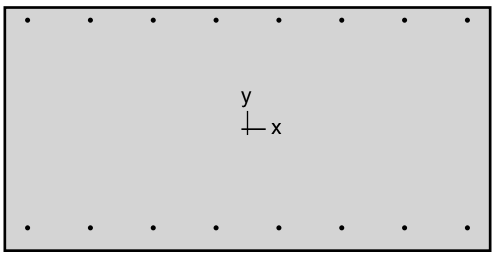
Node Displacements

	LC	Node Label	X [in]	Y [in]	Rotation [rad]
1	6	N1	0	0.001	0
2	6	N2	0	-0.291	0
3	6	N3	0.543	-0.316	0
4	6	N4	0.543	0.041	0
5	6	N5	0.522	0.041	0
6	6	N6	0.522	-0.316	0
7	6	N7	0.01	-0.005	0
8	6	N8	0.01	-0.024	0
9	6	N9	0.009	-0.046	0
10	6	N10	0.009	-0.068	0
11	6	N11	0.009	-0.091	0
12	6	N12	0.009	-0.114	0

Node Displacements (Continued)

	LC	Node Label	X [in]	Y [in]	Rotation [rad]
13	6	N13	0.008	-0.138	0
14	6	N14	0.008	-0.162	0
15	6	N15	0.008	-0.185	0
16	6	N16	0.007	-0.209	0
17	6	N17	0.007	-0.234	0
18	6	N18	0.006	-0.258	0
19	6	N19	0.003	-0.283	0
20	6	N20	0.009	0	0
21	6	N21	0.01	-0.014	0
22	6	N22	0.009	-0.035	0
23	6	N23	0.009	-0.057	0
24	6	N24	0.009	-0.079	0
25	6	N25	0.009	-0.103	0
26	6	N26	0.009	-0.126	0
27	6	N27	0.008	-0.15	0
28	6	N28	0.008	-0.173	0
29	6	N29	0.008	-0.197	0
30	6	N30	0.007	-0.221	0
31	6	N31	0.007	-0.246	0
32	6	N32	0.005	-0.27	0
33	6	N33	0.007	0	0
34	6	N34	0.01	-0.002	0
35	6	N35	0.01	-0.01	0
36	6	N36	0.01	-0.019	0
37	6	N37	0.01	-0.029	0
38	6	N38	0.009	-0.04	0
39	6	N39	0.009	-0.051	0
40	6	N40	0.009	-0.062	0
41	6	N41	0.009	-0.074	0
42	6	N42	0.009	-0.085	0
43	6	N43	0.009	-0.097	0
44	6	N44	0.009	-0.108	0
45	6	N45	0.009	-0.12	0
46	6	N46	0.008	-0.132	0
47	6	N47	0.008	-0.144	0
48	6	N48	0.008	-0.156	0
49	6	N49	0.008	-0.167	0
50	6	N50	0.008	-0.179	0
51	6	N51	0.008	-0.191	0
52	6	N52	0.008	-0.203	0
53	6	N53	0.007	-0.215	0
54	6	N54	0.007	-0.227	0
55	6	N55	0.007	-0.24	0
56	6	N56	0.006	-0.252	0
57	6	N57	0.006	-0.264	0
58	6	N58	0.004	-0.277	0

spColumn v10.00 (TM)
Computer program for the Strength Design of Reinforced Concrete Sections



Contents

List of Figures

1. General Information

File Name	H:\Projects\222...\Panel Footing for Lateral.colx
Project	---
Column	---
Engineer	---
Code	ACI 318-19
Bar Set	ASTM A615
Units	English
Run Option	Investigation
Run Axis	X - axis
Slenderness	Not Considered
Column Type	Structural
Capacity Method	Critical capacity

2. Material Properties

2.1. Concrete

Type	Standard	
f'_c	3	ksi
E_c	3122.02	ksi
f_c	2.55	ksi
ε_u	0.003	in/in
β_1	0.85	

2.2. Steel

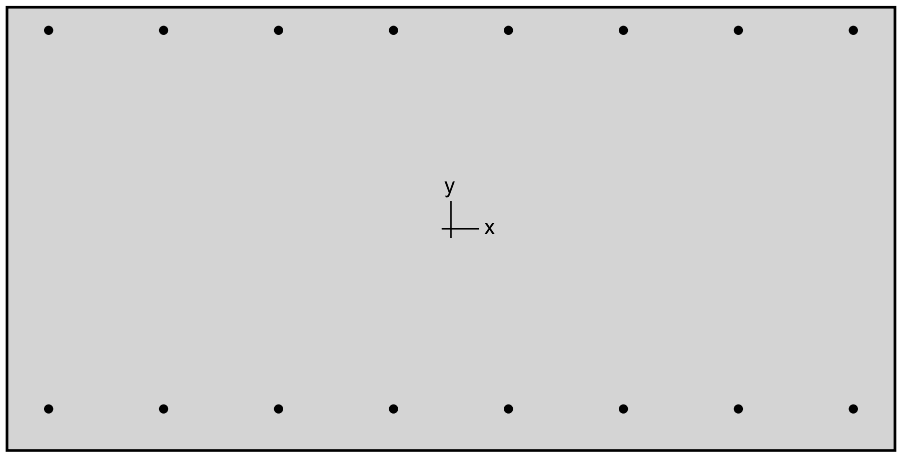
Type	Standard	
f_y	60	ksi
E_s	29000	ksi
ε_{ty}	0.00206897	in/in

3. Section

3.1. Shape and Properties

Type	Rectangular	
Width	72	in
Depth	36	in
A_g	2592	in^2
I_x	279936	in^4
I_y	1.11974e+006	in^4
r_x	10.3923	in
r_y	20.7846	in
X_o	0	in
Y_o	0	in

3.2. Section Figure



Rectangular 72 x 36 in

0.27% reinf.

Figure 1: Column section

4. Reinforcement

4.1. Bar Set: ASTM A615

Bar	Diameter	Area	Bar	Diameter	Area	Bar	Diameter	Area
	in	in²		in	in²		in	in²
#3	0.38	0.11	#4	0.50	0.20	#5	0.63	0.31
#6	0.75	0.44	#7	0.88	0.60	#8	1.00	0.79
#9	1.13	1.00	#10	1.27	1.27	#11	1.41	1.56
#14	1.69	2.25	#18	2.26	4.00			

4.2. Confinement and Factors

Confinement type	Other
For #10 bars or less	#3 ties
For larger bars	#4 ties
Capacity Reduction Factors	
Axial compression, (a)	0.8
Tension controlled ϕ, (b)	0.9
Compression controlled ϕ, (c)	0.65

4.3. Arrangement

Pattern	Sides different
Bar layout	Rectangular
Cover to	Longitudal bars
Clear cover	---
Bars	---
Total steel area, A_s	7.04 in²
Rho	0.27 %
Minimum clear spacing	8.57 in

(Note: Rho < 0.50%)

4.4. Bars Provided

		Bars	Clear cover
			in
Top	8	#6	1.5
Bottom	8	#6	3
Left	0	#6	3
Right	0	#6	3

5. Factored Loads and Moments with Corresponding Capacity Ratios

NOTE: Calculations are based on "Critical Capacity" Method.

No.	Demand		Capacity		Parameters at Capacity			Capacity
	P_u	M_{ux}	ϕP_n	ϕM_{nx}	NA Depth	ε_t	ϕ	Ratio
	kip	k-ft	kip	k-ft	in			
1	0.00	500.00	0.00	551.36	2.29	0.04177	0.900	0.98

05 LATERAL

Panel Types

Panel Type	t (in)	L_{Wall} (ft)	H_{Roof} (ft)	$H_{Parapet}$ (ft)	H_{wall} (ft)	H_{fdn} (ft)	A_o (ft^2)	Wt_{wall} (k)
P55	8.75	26.5	39	1.5	40.5	1.5	0	122
P26	8.75	26.5	38.75	1.75	40.5	1.5	0	122
P25	8.75	26.5	39	4.5	43.5	1.5	0	130
P23	8.75	20.0	38	2.5	40.5	1.5	0	92
P49	8	27.0	36	7.5	43.5	1.5	0	122
P48	8	26.0	36	4.5	40.5	1.5	0	109
P3	8	26.0	36	4.5	40.5	1.5	108	93
P15	9.5	26.0	35.167	0.583	35.75	5	114	109
P17	9.5	26.0	35.167	0.583	35.75	5	180	99
P18	9.5	26.0	35.167	0.583	35.75	5	224	93
P19	9.5	28.0	35.5	8	43.5	1.5	30	145

North
South
East
West

Stiffness

Panel Type	0.7 I_g			0.35 I_g		
	F (k)	Δ (in)	k (k/in)	F (k)	Δ (in)	k (k/in)
P55	100	0.082	1220	100	0.164	610
P26	100	0.078	1282	100	0.156	641
P25	100	0.079	1266	100	0.158	633
P23	100	0.15	667	100	0.3	333
P49	100	0.069	1449	100	0.138	725
P48	100	0.075	1333	100	0.15	667
P3	100	0.093	1075	100	0.186	538
P15	100	0.11	909	100	0.22	455
P17	100	0.195	513	100	0.39	256
P18	100	0.15	667	100	0.3	333
P19	100	0.06	1667	100	0.12	833
0	100	1	100	100	2	50
0	100	1	100	100	2	50
0	100	1	100	100	2	50
0	100	1	100	100	2	50
0	100	1	100	100	2	50
0	100	1	100	100	2	50
0	100	1	100	100	2	50

Input =	
Verify =	
Output =	

Wind Roof Shears	
Direction	V_{roof} (k)
East/West	123
North/South	267

Input =	
Verify =	
Output =	

Seismic Roof Shears			
Direction	V_{roof} (k)	ρ	ρV_{roof} (k)
East/West	487	1.0	487
North/South	713	1.0	713

$Cs=$ 0.169

N/S Wall Line GL F (West Elevation)					SEISMIC			WIND		
					V_{roof} (k) =	356.5		133.50		
Panel Type	Qty	Cracked ? (Y/N)	Stiffness (k/in)	Relative Stiffness	V_{self} (k)	$V_{s,roof}$ (k)	$M_{OT,s}$ (k-ft)	V_w (k)	$M_{OT,w}$ (k-ft)	V_u (k)
P55	7	N	1220	0.143	20.52	50.93	2401.83	19.07	743.79	71.45
					Total V_s (k) =	500.2				

N/S Wall Line GL D (East Elevation)					SEISMIC			WIND		
					V_{roof} (k) =	356.5		133.50		
Panel Type	Qty	Cracked ? (Y/N)	Stiffness (k/in)	Relative Stiffness	V_{self} (k)	$V_{s,roof}$ (k)	$M_{OT,s}$ (k-ft)	V_w (k)	$M_{OT,w}$ (k-ft)	V_u (k)
P23	1	N	667	0.115	15.49	41.13	1876.56	15.40	585.26	56.62
P25	1	N	1266	0.219	21.99	78.09	3523.88	29.24	1140.49	100.08
P26	3	N	1282	0.222	20.52	79.09	3480.48	29.62	1147.71	99.62
					Total V_s (k) =	455.6				

E/W Wall Line GL 1 (South Elevation)					SEISMIC			WIND		
					V_{roof} (k) =	243.5		61.50		
Panel Type	Qty	Cracked ? (Y/N)	Stiffness (k/in)	Relative Stiffness	V_{self} (k)	$V_{s,roof}$ (k)	$M_{OT,s}$ (k-ft)	V_w (k)	$M_{OT,w}$ (k-ft)	V_u (k)
P48	17	N	1333	0.047	18.41	11.41	783.45	2.88	103.71	29.82
P49	4	N	1449	0.051	20.48	12.40	891.88	3.13	112.73	32.88
					Total V_s (k) =	638.4				

E/W Wall Line GL 10 (North Elevation)					SEISMIC			WIND		
					V_{roof} (k) =	243.5		61.50		
Panel Type	Qty	Cracked ? (Y/N)	Stiffness (k/in)	Relative Stiffness	V_{self} (k)	$V_{s,roof}$ (k)	$M_{OT,s}$ (k-ft)	V_w (k)	$M_{OT,w}$ (k-ft)	V_u (k)
P3	3	N	1075	0.076	15.71	18.59	987.45	4.70	169.06	34.30
P15	3	N	909	0.065	18.36	15.72	880.99	3.97	139.63	34.08
P17	10	N	513	0.036	16.71	8.87	610.48	2.24	78.77	25.57
P18	2	N	667	0.047	15.60	11.53	684.34	2.91	102.39	27.13
P19	1	N	1667	0.118	24.48	28.82	1555.48	7.28	258.41	53.30
					Total V_s (k) =	465.5				

RELATIVE STIFFNESS VALUE
DETERMINED FROM RISA-2D
MODEL OF PANELS. SEE
EXAMPLE BELOW (P17 & P18)

PANEL P17 RISA MODEL OUTPUT FOR
DETERMINING RELATIVE STIFFNESS

Joint Coordinates and Temperatures

	Label	X [ft]	Y [ft]	Temp [F]
1	N1	0	0	0
2	N2	26	0	0
3	N3	26	35.75	0
4	N4	0	35.75	0
5	N5	0	35.167	0
6	N6	26	35.167	0
7	N7	2	0	0
8	N8	11	0	0
9	N9	2	10	0
10	N10	11	10	0
11	N11	15	0	0
12	N12	24	0	0
13	N13	15	10	0
14	N14	24	10	0
15	N15	15	24	0
16	N16	24	24	0
17	N17	15	27	0
18	N18	24	27	0

Joint Boundary Conditions

Joint Label	X [k/in]	Y [k/in]	Rotation[k-ft/rad]
No Data to Print ...			

Wall Panel Data

	Label	A Joint	B Joint	C Joint	D Joint	Material Type	Material Set	Thicknes...	Design Rule	Panel/Spacing
1	WP1	N4	N3	N2	N1	Concrete	Conc4000NW	9.5	Typical	N/A

Wall Panel Advanced Data

	Label	Seismic Rule	Design Method	SSAF	In-plane Icr Factor	K	Cm
1	WP1	None	N/A	N/A			

Joint Loads and Enforced Displacements (BLC 1 : Stiffness)

	Joint Label	L,D,M	Direction	Magnitude[(k,k-ft), (in,rad), (k*s^2/ft, k*s^2*ft) Inactive	Inactive
1	N5	L	X	50	Active
2	N6	L	X	50	Active

Wall Panel Distributed Loads

Wall Label	Direction	Start Magnitude[k...	End Magnitude[k/...	Start Location[ft,%]	End Location[ft,%]	Inactive
No Data to Print ...						

Basic Load Cases

	BLC Description	Category	X Gravity	Y Gravity	Joint	Point	Distributed
1	Stiffness	None			2		
2	DL	None		-1			
3	LL	None					
4	SL	None					
5	W	None					
6	E	None					

Load Combinations

	Description	S...	PDelta	SRSS	B...	Fa...	B...	Fa...	B...	Fa...	B...	Fa...	B...	Fa...	B...	Fa...	B...	Fa...	B...	Fa...	B...	Fa...	B...	Fa...
1	Stiffness	Yes	Y		1	1																		
2	1.2D+1.6L+ 0.5S	Yes	Y		2	1.2	3	1.6	4	0.5														
3	1.2D+1.6S+0.5L	Yes	Y		2	1.2	3	0.5	4	1.6														
4	1.2D+W+0.5L+0.5S	Yes	Y		2	1.2	3	0.5	4	0.5	5	1												
5	0.9D+W	Yes	Y		2	0.9	5	1																
6	1.2D+E+0.5L+0.2S	Yes	Y		2	1.2	3	0.5	4	0.2	6	1												
7	0.9D+E	Yes	Y		2	0.9	6	1																
8	D+L+S	Yes	Y		2	1	3	1	4	1														
9	D+L+0.6W	Yes	Y		2	1	3	1	5	0.6														
10	D+L+0.6W+S/2	Yes	Y		2	1	3	1	4	0.5	5	0.6												
11	D+L+S+0.3W	Yes	Y		2	1	3	1	4	1	5	0.3												
12	D+L+S+E/1.4	Yes	Y		2	1	3	1	4	1	6	0.7...												
13	0.9D+E/1.4	Yes	Y		2	0.9	6	0.7...																

Node Reactions (By Combination)

	LC	Node Label	X [k]	Y [k]	MZ [k-ft]
1	1	WP1	-100	0	3516.682
2	1	Totals:	-100	0	
3	1	COG (ft):	NC	NC	

Node Displacements

	LC	Node Label	X [in]	Y [in]	Rotation [rad]
1	1	N1	0	0	0
2	1	N2	0	0	0
3	1	N3	0.195	-0.048	0
4	1	N4	0.196	0.049	0
5	1	N5	0.195	0.05	0
6	1	N6	0.194	-0.048	0
7	1	N7	0	0	0
8	1	N8	0	0	0
9	1	N9	0.097	0.022	0
10	1	N10	0.092	0.011	0
11	1	N11	0	0	0
12	1	N12	0	0	0
13	1	N13	0.092	-0.012	0
14	1	N14	0.098	-0.021	0
15	1	N15	0.14	-0.009	0

Node Displacements (Continued)

	LC	Node Label	X [in]	Y [in]	Rotation [rad]
16	1	N16	0.14	-0.033	0
17	1	N17	0.155	-0.01	0
18	1	N18	0.157	-0.037	0

Joint Coordinates and Temperatures

	Label	X [ft]	Y [ft]	Temp [F]
1	N1	0	0	0
2	N2	26	0	0
3	N3	26	35.75	0
4	N4	0	35.75	0
5	N5	0	35.167	0
6	N6	26	35.167	0
7	N7	5	0	0
8	N8	21	0	0
9	N9	5	14	0
10	N10	21	14	0

Joint Boundary Conditions

Joint Label	X [k/in]	Y [k/in]	Rotation[k-ft/rad]
No Data to Print ...			

Wall Panel Data

	Label	A Joint	B Joint	C Joint	D Joint	Material Type	Material Set	Thicknes...	Design Rule	Panel/Spacing
1	WP1	N4	N3	N2	N1	Concrete	Conc4000NW	9.5	Typical	N/A

Wall Panel Advanced Data

	Label	Seismic Rule	Design Method	SSAF	In-plane Icr Factor	K	Cm
1	WP1	None	N/A	N/A			

Joint Loads and Enforced Displacements (BLC 1 : Stiffness)

	Joint Label	L,D,M	Direction	Magnitude[(k,k-ft), (in,rad), (k*s^2/ft, k*s^2*ft) Inactive	Inactive
1	N5	L	X	50	Active
2	N6	L	X	50	Active

Wall Panel Distributed Loads

Wall Label	Direction	Start Magnitude[k...	End Magnitude[k/...	Start Location[ft,%]	End Location[ft,%]	Inactive
No Data to Print ...						

Basic Load Cases

	BLC Description	Category	X Gravity	Y Gravity	Joint	Point	Distributed
1	Stiffness	None			2		
2	DL	None		-1			
3	LL	None					
4	SL	None					
5	W	None					
6	E	None					

Load Combinations

	Description	S...	PDelta	SRSS	B...	Fa...	B...	Fa...	B...	Fa...	B...	Fa...	B...	Fa...	B...	Fa...	B...	Fa...	B...	Fa...	B...	Fa...	B...	Fa...
1	Stiffness	Yes	Y		1	1																		
2	1.2D+1.6L+ 0.5S	Yes	Y		2	1.2	3	1.6	4	0.5														
3	1.2D+1.6S+0.5L	Yes	Y		2	1.2	3	0.5	4	1.6														
4	1.2D+W+0.5L+0.5S	Yes	Y		2	1.2	3	0.5	4	0.5	5	1												
5	0.9D+W	Yes	Y		2	0.9	5	1																
6	1.2D+E+0.5L+0.2S	Yes	Y		2	1.2	3	0.5	4	0.2	6	1												
7	0.9D+E	Yes	Y		2	0.9	6	1																
8	D+L+S	Yes	Y		2	1	3	1	4	1														
9	D+L+0.6W	Yes	Y		2	1	3	1	5	0.6														
10	D+L+0.6W+S/2	Yes	Y		2	1	3	1	4	0.5	5	0.6												
11	D+L+S+0.3W	Yes	Y		2	1	3	1	4	1	5	0.3												
12	D+L+S+E/1.4	Yes	Y		2	1	3	1	4	1	6	0.7...												
13	0.9D+E/1.4	Yes	Y		2	0.9	6	0.7...																

Node Reactions (By Combination)

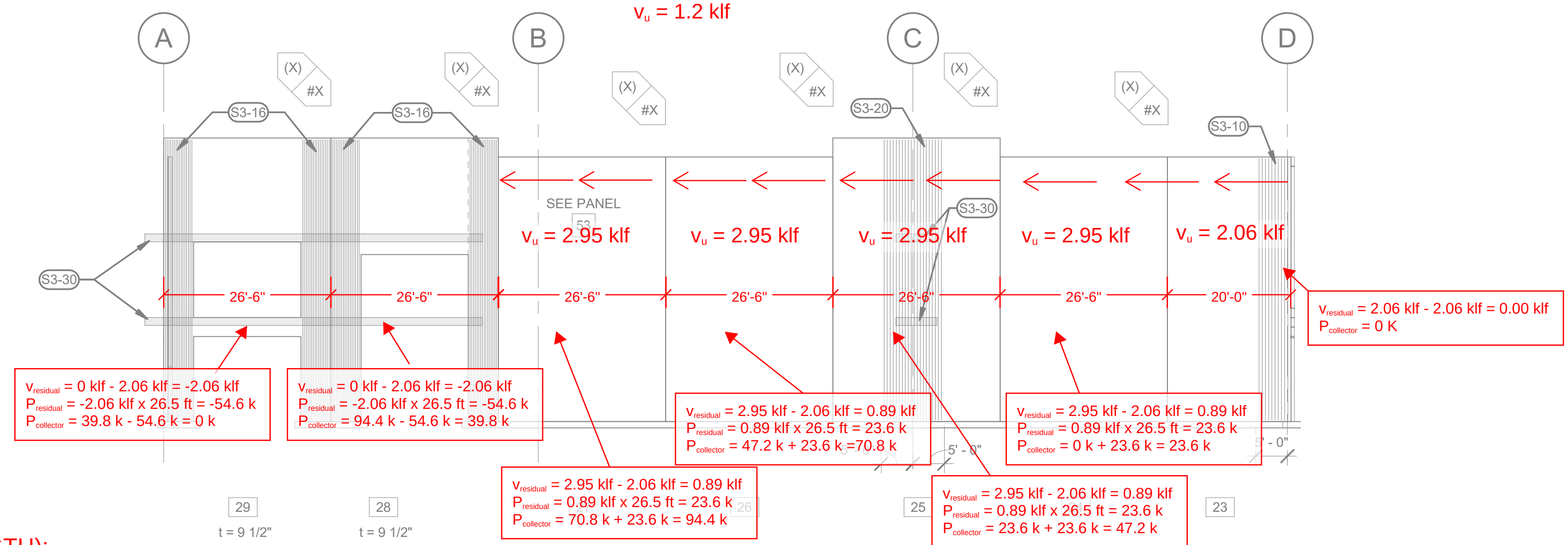
	LC	Node Label	X [k]	Y [k]	MZ [k-ft]
1	1	WP1	-100	0	3516.7
2	1	Totals:	-100	0	
3	1	COG (ft):	NC	NC	

Node Displacements

	LC	Node Label	X [in]	Y [in]	Rotation [rad]
1	1	N1	0	0	0
2	1	N2	0	0	0
3	1	N3	0.149	-0.029	0
4	1	N4	0.149	0.029	0
5	1	N5	0.15	0.029	0
6	1	N6	0.15	-0.029	0
7	1	N7	0	0	0
8	1	N8	0	0	0
9	1	N9	0.092	0.004	0
10	1	N10	0.092	-0.004	0

Collector Calc (EAST ELEV)

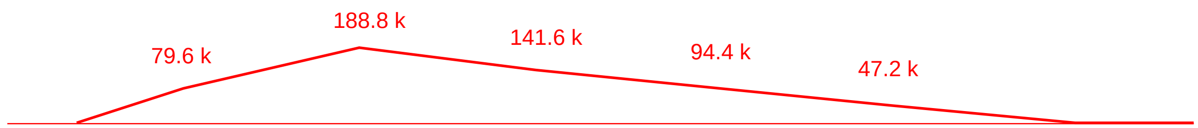
v_u = 2.06 klf
v_u = 1.2 klf



P (NO OVERSTRENGTH):

Works in tension

P (w/ OVERSTRENGTH = 2.0):



79.6 k / (0.9*60 ksi) = 1.47 in^2 -> (4) #6

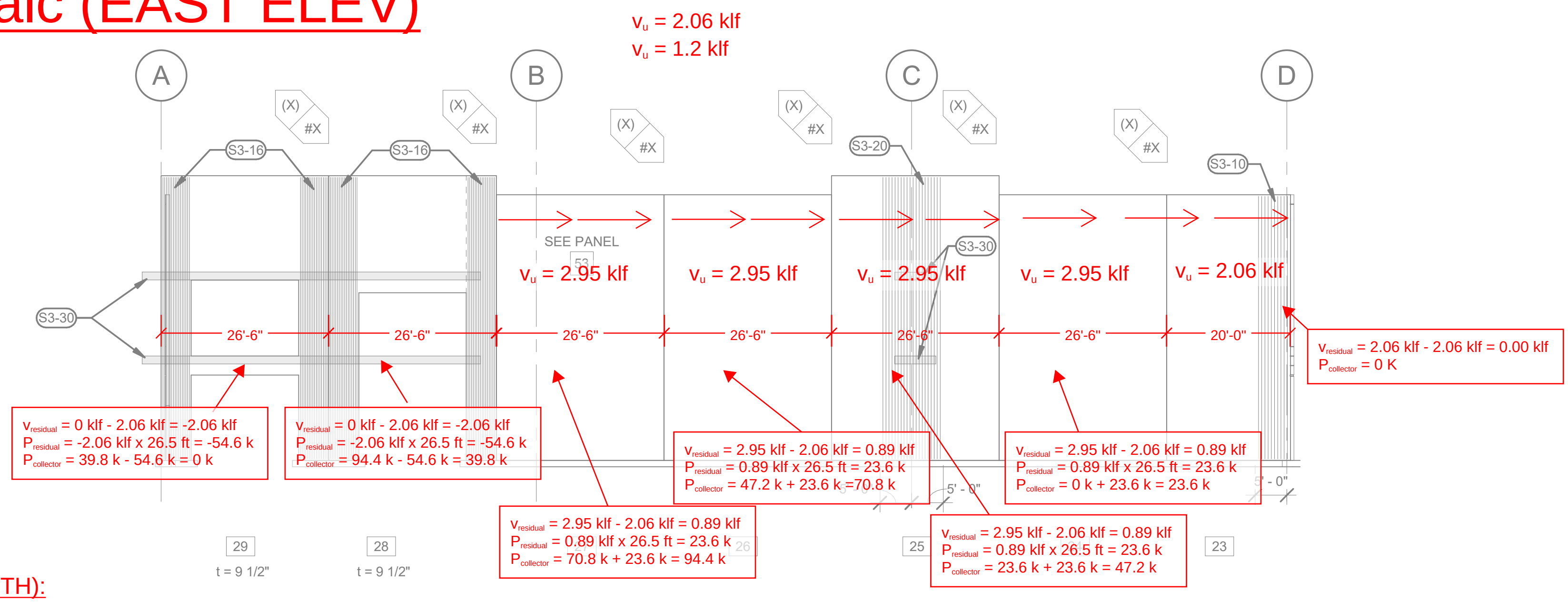
141.6 k / (0.9*60 ksi) = 2.62 in^2 -> (5) #7

47.2 k / (0.9*60 ksi) = 0.87 in^2 -> (2) #7

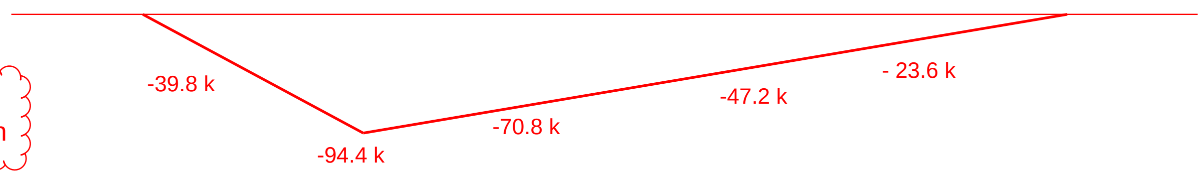
188.8 k / (0.9*60 ksi) = 3.49 in^2 -> (6) #7

94.4 k / (0.9*60 ksi) = 1.75 in^2 -> (3) #7

Collector Calc (EAST ELEV)



P (NO OVERSTRENGTH):



Works in compression

79.6 k / (0.9*60 ksi) = 1.47 in^2 -> (3) #7

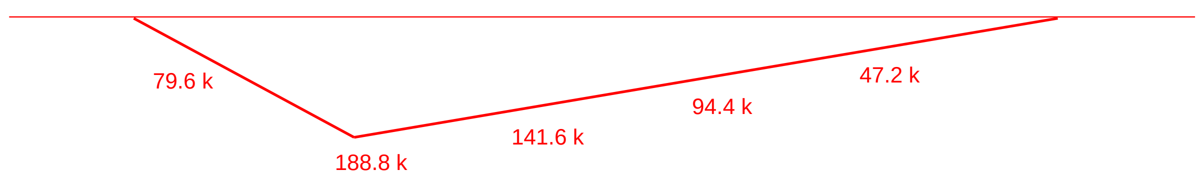
141.6 k / (0.9*60 ksi) = 2.62 in^2 -> (5) #7

47.2 k / (0.9*60 ksi) = 0.87 in^2 -> (2) #7

P (w/ OVERSTRENGTH = 2.0):

188.8 k / (0.9*60 ksi) = 3.49 in^2 -> (6) #7

94.4 k / (0.9*60 ksi) = 1.75 in^2 -> (3) #7

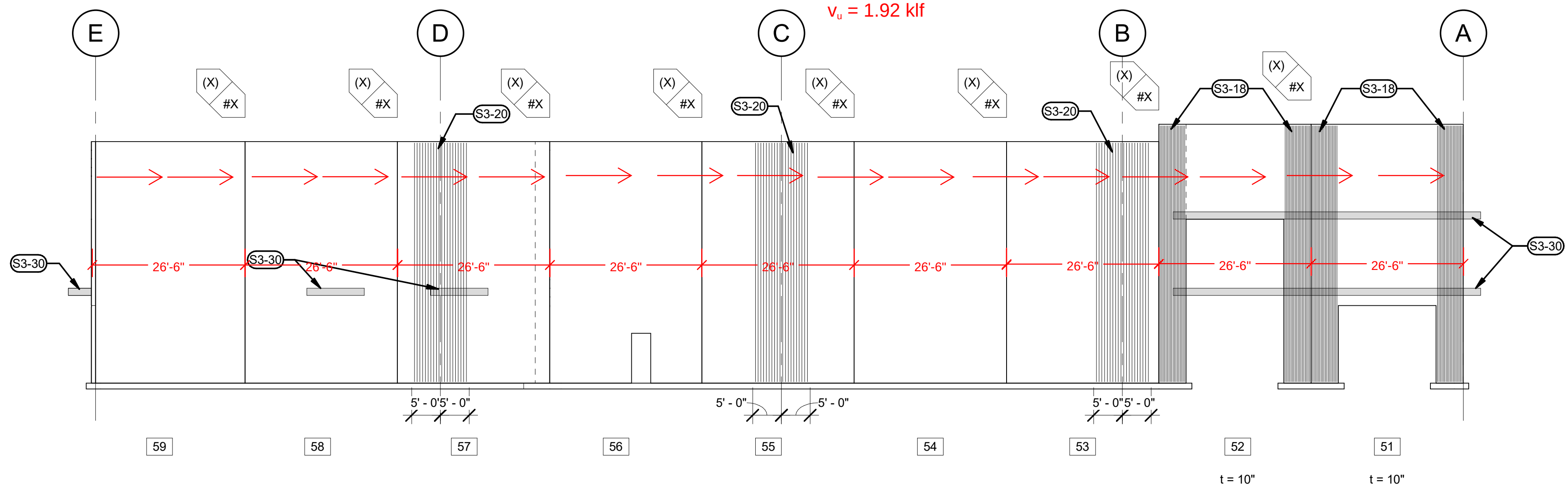


Collector Calc (WEST ELEV)

collector not required

$\phi v_n = 2.06$ klf

$v_u = 1.92$ klf



SEISMIC CALCULATIONS

SOUTH ELEVATION:

(150 PCF)[(8/12)(1333 SF + 9927 SF + 1310 SF) + (16/12)(64 SF + 64 SF)]

(150 PCF)[8380 FT3 + 171 FT3] = 1283k

NORTH ELEVATION:

(150 PCF)[(8/12)(1738 SF) + (17.5/12)(57 SF + 65 SF) + (9.5/12)(7092 SF + 678 SF - (13)(27 SF)]

(150 PCF)[1159 FT3 + 178 FT3 + 5874 SF] = 1082k

WEST ELEVATION:

(150 PCF)[(8.75/12)(1660 SF + 2277 SF) + (17.5/12)(54 SF) + (10/12)(1051 SF) + (18.75/12)(115 SF)]

(150 PCF)[2871 FT3 + 79 FT3 + 876 FT3 + 180 FT3] = 601k

EAST ELEVATION:

(150 PCF)[(8.75/12)(2812 SF) + (9.5/12)(947 SF) + (18.25/12)(118 SF)]

(150 PCF)[2051 FT3 + 750 FT3 + 180 FT3] = 448k

SKEWED CORNER:

(150 PCF)[(8/12)(573 SF + 1271 SF) + (16/12)(63 SF)]

(150 PCF)[1230 FT3 + 84 FT3] = 198k

ROOF:

(129800 SF)(10.9 PSF) = 1415k

(129800 SF)(0.4)(4 PSF) = 208k (SOLAR WEIGHT)

1415k + 208k = 1623k

STEEL COLUMNS, MISC:

(39'/2)(47.9 PLF)(30 COLUMNS) = 26k... SAY 40k FOR STOREFRONT, HIGH CANOPIES, ETC

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Seattle, WA
206.749.9993
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N/S SEISMIC WEIGHT:

1283k + 1082k + 198k + 1623k + 40k = 4226k

Cs = 0.843/5 = 0.1686

V = (4226k)(0.1686) = 713k

E/W SEISMIC WEIGHT:

601k + 448k + 198k + 1623k + 40k = 2910k

V = (2910k)(0.1686) = 487k

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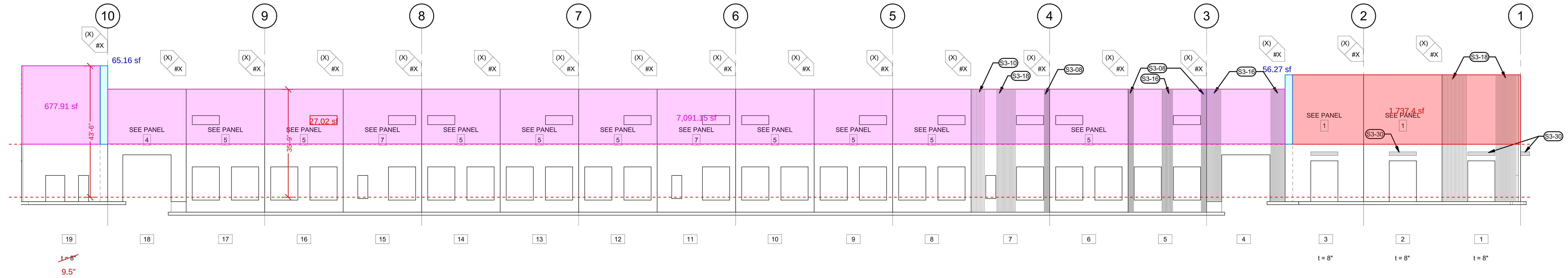
By

Date

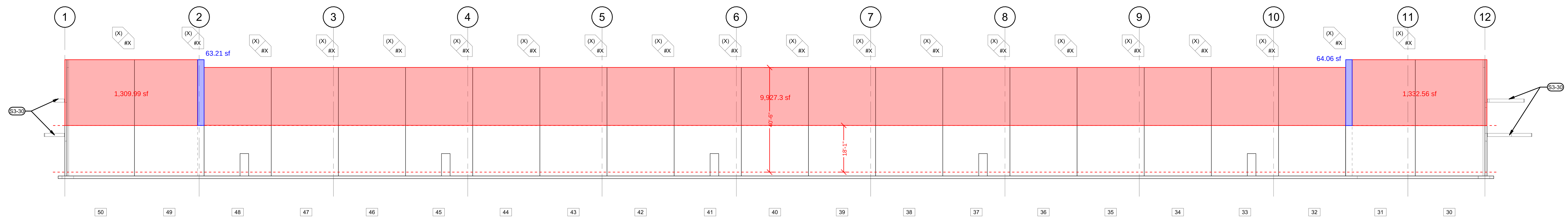
Job#

Sht. of



1 S2.10 NORTH ELEVATION
1/16" = 1'-0"

t=9 1/2", UNO



2 S2.10 SOUTH ELEVATION
1/16" = 1'-0"

t=8", UNO

TYPICAL SHEET NOTES

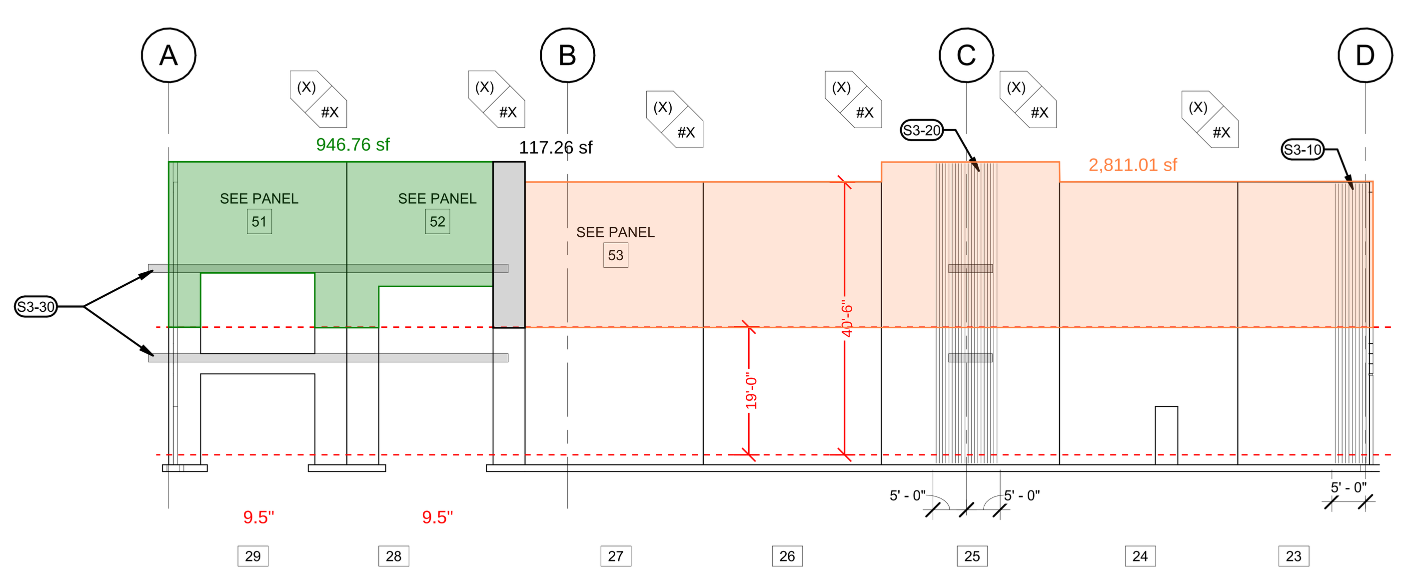
A. FOR GENERAL STRUCTURAL NOTES SEE S0.00 SERIES SHEETS
B. FOR TYPICAL STRUCTURAL DETAILS SEE S0.10 SERIES SHEETS
C. SEE ARCHITECTURAL DRAWINGS FOR CONTROL ELEVATIONS

LEGEND

99	PANEL NUMBER
(x)	# CHORD BARS, REF. 15/S5.81 AND 2/S5.81 WHERE APPLICABLE
(x)	SIZE OF REINF.
●	HOLD DOWN PER 19/S5.80 W/ (X) #X BARS
t =	PANEL THICKNESS

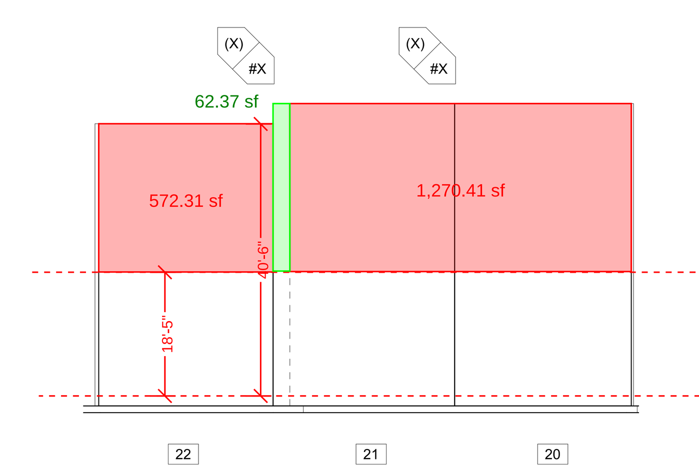
KEYNOTES

S3-08	(8) #5 VERT BARS EF, EQ SPACED IN LEG
S3-10	(10) #5 VERT BARS EF, EQ SPACED IN LEG
S3-16	(16) #5 VERT BARS EF, EQ SPACED IN LEG
S3-18	(18) #5 VERT BARS EF, EQ SPACED IN LEG
S3-20	(20) #5 VERT BARS EF, EQ SPACED IN LEG
S3-30	EXTERIOR ACCENT/CANOPY FRAMING, TBD SEE ARCH.

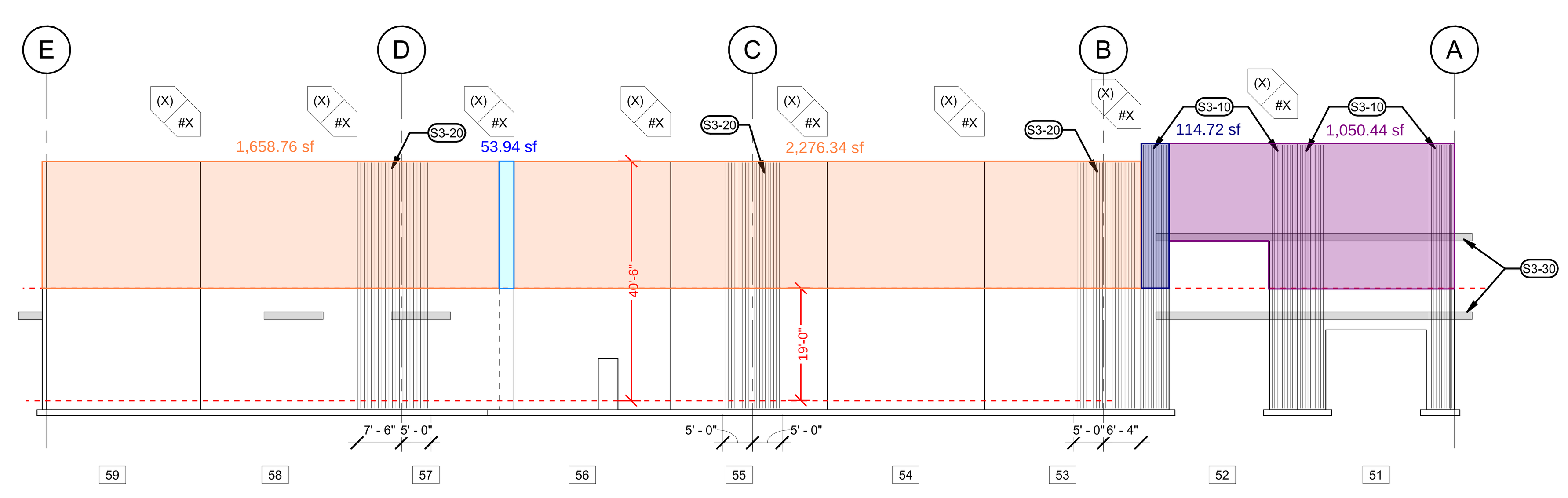


3 S2.10 EAST ELEVATION
1/16" = 1'-0"

t=8 3/4", UNO

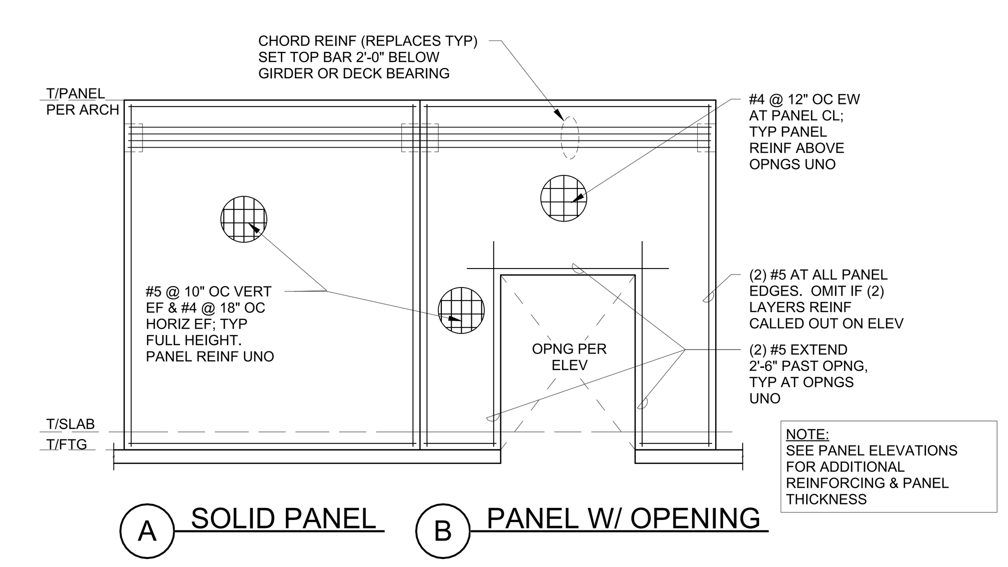


4 S2.10 NORTHEAST ELEVATION
1/16" = 1'-0"



5 S2.10 WEST ELEVATION
1/16" = 1'-0"

t=8 3/4", UNO



7 S2.10 TYPICAL PANEL REINFORCING
1/8" = 1'-0"

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Planning ▪ Engineering

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Client

CREF3 PUYALLUP OWNER LLC
11611 SAN VICENTE BLVD
10TH FLOOR LOS ANGELES
CA 90049

Project

240 15TH ST SE
PUYALLUP, WA, 98372

Mechanical/Electrical

REFERENCE ONLY UNLESS STAMP IS PLACED
(PLACE STAMP HERE)

REVISION SCHEDULE

Delta	Issued As	Issue Date

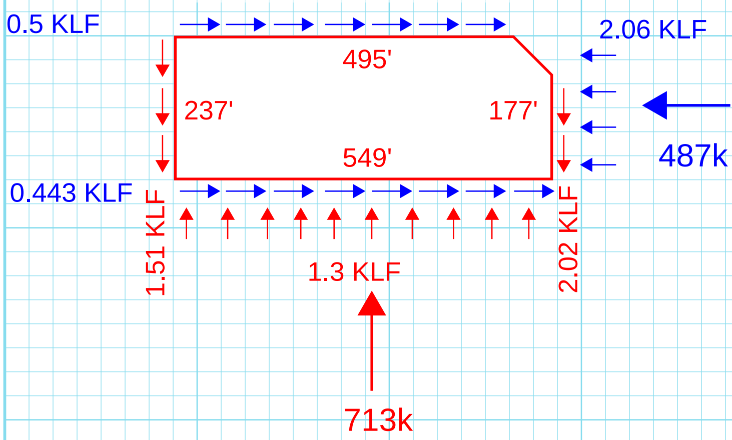
SHEET TITLE:
EXTERIOR WALL ELEVATIONS

SHEET
S2.10

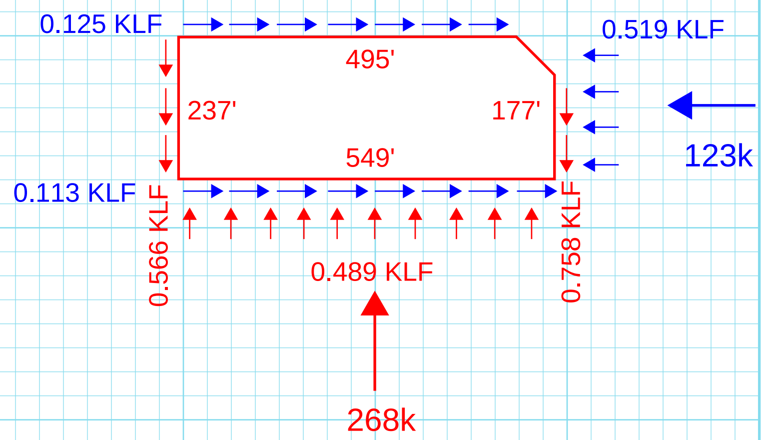
JOB NO. 2220290.00

FORTRESS PUYALLUP - DIAPHRAGM DESIGN

SEISMIC LOADING



WIND LOADING



SEISMIC DIAPHRAGM SHEARS, DIST LOAD

713k/(2)(177') = 2.02 KLF
713k/(2)(237') = 1.51 KLF
713k/549' = 1.3 KLF

487k/(2)(495') = 0.5 KLF
487k/(2)(549') = 0.443 KLF
487k/237' = 2.06 KLF

WIND DIAPHRAGM SHEARS, DIST LOAD

268k/(2)(177') = 0.758 KLF
268k/(2)(237') = 0.566 KLF
268k/549' = 0.489 KLF

123k/(2)(495') = 0.125 KLF
123k/(2)(549') = 0.113 KLF
123k/237' = 0.519 KLF

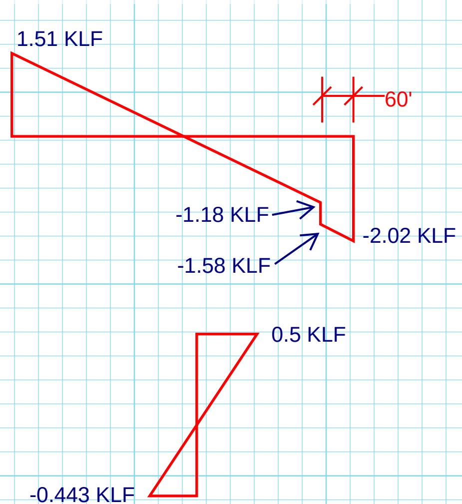
SHEAR AT DIAGONAL WALL

713k/2 - (1.3 KLF)(60') = 278.5k
278.5k/177' = 1.58 KLF
278.5k/237' = 1.18 KLF

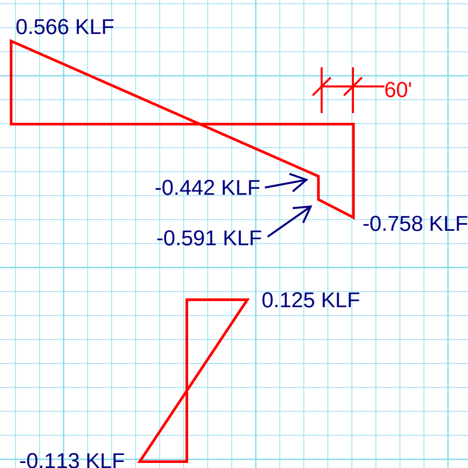
SHEAR AT DIAGONAL WALL

268k/2 - (0.489 KLF)(60') = 104.7k
104.7k/177' = 0.591 KLF
104.7k/237' = 0.442 KLF

SHEAR DIAGRAMS (SEISMIC)



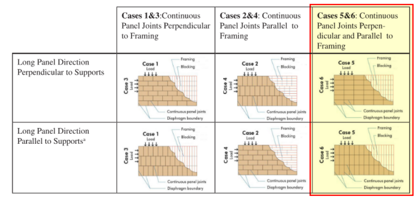
SHEAR DIAGRAMS (WIND)



Sheathing Grade	Common Nail Size	Minimum Fastener Penetration in Framing Member or Blocking (in.)	Minimum Nominal Panel Thickness (in.)	Minimum Nominal Width of Nailed Face at Adjoining Panel Edges and Boundaries (in.)	Lines of Fasteners	A SEISMIC: Nail Spacing (in.) at diaphragm boundaries (all cases), at continuous panel edges parallel to load (Cases 3 & 4), and at all panel edges (Cases 5 & 6): 4			4			2-1/2			2-1/2			B WIND: Nail Spacing (in.) at diaphragm boundaries (all cases), at continuous panel edges parallel to load (Cases 3 & 4), and at all panel edges (Cases 5 & 6): 4	4	2-1/2	2-1/2
						Nail Spacing (in.) at other panel edges (Cases 1, 2, 3, & 4): 6			4			4			3			Nail Spacing (in.) at other panel edges (Cases 1, 2, 3, & 4): 6	4	4	3
						v_s (plf)	G_a (kips/in.) OSB	PLY	v_s (plf)	G_a (kips/in.) OSB	PLY	v_s (plf)	G_a (kips/in.) OSB	PLY	v_s (plf)	G_a (kips/in.) OSB	PLY	v_w (plf)	v_w (plf)	v_w (plf)	v_w (plf)
Structural I	10d	1-1/2	15/32	3	2	1210	40	24	1630	53	28	1750	50	27	2300	56	29	1695	2280	2450	3220
				4	2	1400	33	21	1830	48	27	2010	44	25	2580	51	28	1960	2560	2815	3610
				4	3	1750	50	27	2440	61	30	2570	59	30	2790	70	32	2450	3415	3600	3905
			19/32	3	2	1340	36	23	1760	52	29	1930	47	27	2510	54	29	1875	2465	2700	3515
				4	2	1560	29	20	1980	46	27	2220	40	25	2880	48	27	2185	2770	3110	4030
				4	3	1930	47	27	2640	60	31	2810	57	30	3580	64	32	2700	3695	3935	5010
			23/32	3	2	1460	33	22	1910	50	29	2100	45	27	2730	53	30	2045	2675	2940	3820
				4	2	1710	26	19	2140	43	27	2420	37	24	3130	45	27	2395	2995	3390	4380
				4	3	2100	45	27	2860	59	32	3050	56	31	3600	68	34	2940	4005	4270	5040
Sheathing and Single-Floor	10d	1-1/2	15/32	3	2	1050	43	21	1450	55	23	1530	53	23	2020	58	24	1470	2030	2140	2830
				4	2	1210	36	19	1630	50	22	1750	46	21	2210	55	23	1695	2280	2450	3095
				4	3	1530	53	23	2170	62	24	2260	61	24	2390	72	26	2140	3040	3165	3345
			19/32	3	2	1300	34	19	1720	49	23	1870	45	22	2450	52	23	1820	2410	2620	3430
				4	2	1510	27	16	1930	43	21	2160	37	20	2740	46	22	2115	2700	3025	3835
				4	3	1870	45	22	2580	57	24	2730	55	24	2970	68	26	2620	3610	3820	4160
			23/32	3	2	1420	30	18	1870	46	23	2040	42	22	2670	50	24	1990	2620	2855	3740
				4	2	1650	24	16	2100	40	21	2350	34	20	2890	45	23	2310	2940	3290	4045
				4	3	2040	42	22	2800	56	25	2960	53	25	3130	71	28	2855	3920	4145	4380

1750 PLF(0.8) = 1400 PLF

2570 PLF(0.8) = 2056 PLF



(a) Panel span rating for out-of-plane loads may be lower than the span rating with the long panel direction perpendicular to supports (See Section 3.2.2 and Section 3.2.3)

1. Nominal unit shear capacities shall be adjusted in accordance with 4.2.3 to determine ASD allowable unit shear capacity and LRFD factored unit resistance. For general construction requirements see 4.2.6. For specific requirements, see 4.2.7.1 for wood structural panel diaphragms. See Appendix A for common nail dimensions.
2. For species and grades of framing other than Douglas-Fir-Larch or Southern Pine, reduced nominal unit shear capacities shall be determined by multiplying the tabulated nominal unit shear capacity by the Specific Gravity Adjustment Factor = [1-(0.5-G)], where G = Specific Gravity of the framing lumber from the *NDS* (Table 12.3.3A). The Specific Gravity Adjustment Factor shall not be greater than 1.
3. Apparent shear stiffness values, G_a, are based on nail slip in framing with moisture content less than or equal to 19% at time of fabrication and panel stiffness values for diaphragms constructed with either OSB or 3-ply plywood panels. When 4-ply or 5-ply plywood panels or composite panels are used, G_a values shall be permitted to be multiplied by 1.2.
4. Where moisture content of the framing is greater than 19% at time of fabrication, G_a values shall be multiplied by 0.5.
5. Diaphragm resistance depends on the direction of continuous panel joints with respect to the loading direction and direction of framing members, and is independent of the panel orientation.

Table 4.2A Nominal Unit Shear Capacities for Wood-Frame Diaphragms

Blocked Wood Structural Panel Diaphragms[1,2,3,4,5]

Sheathing Grade	Common Nail Size	Minimum Fastener Penetration in Framing Member or Blocking (in.)	Minimum Nominal Panel Thickness (in.)	Minimum Nominal Width of Nailed Face at Adjoining Panel Edges and Boundaries (in.)	A SEISMIC: Nail Spacing (in.) at diaphragm boundaries (all cases), at continuous panel edges parallel to load (Cases 3 & 4), and at all panel edges (Cases 5 & 6): 6			4			2-1/2			2			B WIND: Nail Spacing (in.) at diaphragm boundaries (all cases), at continuous panel edges parallel to load (Cases 3 & 4), and at all panel edges (Cases 5 & 6): 6	4	2-1/2	2
					Nail Spacing (in.) at other panel edges (Cases 1, 2, 3, & 4): 6			6			4			3			Nail Spacing (in.) at other panel edges (Cases 1, 2, 3, & 4): 6	6	4	3
					v_s (plf)	G_a (kips/in.) OSB	PLY	v_s (plf)	G_a (kips/in.) OSB	PLY	v_s (plf)	G_a (kips/in.) OSB	PLY	v_s (plf)	G_a (kips/in.) OSB	PLY	v_w (plf)	v_w (plf)	v_w (plf)	v_w (plf)
Structural I	6d	1-1/4	5/16	2	370	15	12	500	8.5	7.5	750	12	10	840	20	15	520	700	1050	1175
				3	420	12	9.5	560	7.0	6.0	840	9.5	8.5	950	17	13	590	785	1175	1330
	8d	1-3/8	3/8	2	540	14	11	720	9.0	7.5	1060	13	10	1200	21	15	755	1010	1485	1680
				3	600	12	10	800	7.5	6.5	1200	10	9.0	1350	18	13	840	1120	1680	1890
	10d	1-1/2	15/32	2	640	24	17	850	15	12	1280	20	15	1460	31	21	895	1190	1790	2045
				3	720	20	15	960	12	9.5	1440	16	13	1640	26	18	1010	1345	2015	2295
Sheathing and Single-Floor	6d	1-1/4	5/16	2	340	15	10	450	9.0	7.0	670	13	9.5	760	21	13	475	630	940	1065
				3	380	12	9.0	500	7.0	6.0	760	10	8.0	860	17	12	530	700	1065	1205
			3/8	2	370	13	9.5	500	7.0	6.0	750	10	8.0	840	18	12	520	700	1050	1175
				3	420	10	8.0	560	5.5	5.0	840	8.5	7.0	950	14	10	590	785	1175	1330
	8d	1-3/8	3/8	2	480	15	11	640	9.5	7.5	960	13	9.5	1090	21	13	670	895	1345	1525
				3	540	12	9.5	720	7.5	6.0	1080	11	8.5	1220	18	12	755	1010	1510	1710
			7/16	2	510	14	10	680	8.5	7.0	1010	12	9.5	1150	20	13	715	950	1415	1610
				3	570	11	9.0	760	7.0	6.0	1140	10	8.0	1290	17	12	800	1065	1595	1805
			15/32	2	540	13	9.5	720	7.5	6.5	1060	11	8.5	1200	19	13	755	1010	1485	1680
				3	600	10	8.5	800	6.0	5.5	1200	9.0	7.5	1350	15	11	840	1120	1680	1890
	10d	1-1/2	15/32	2	580	25	15	770	15	11	1150	21	14	1310	33	18	810	1080	1610	1835
				3	650	21	14	860	12	9.5	1300	17	12	1470	28	16	910	1205	1820	2060
			19/32	2	640	21	14	850	13	9.5	1280	18	12	1460	28	17	895	1190	1790	2045
				3	720	17	12	960	10	8.0	1440	14	11	1640	24	15	1010	1345	2015	2295

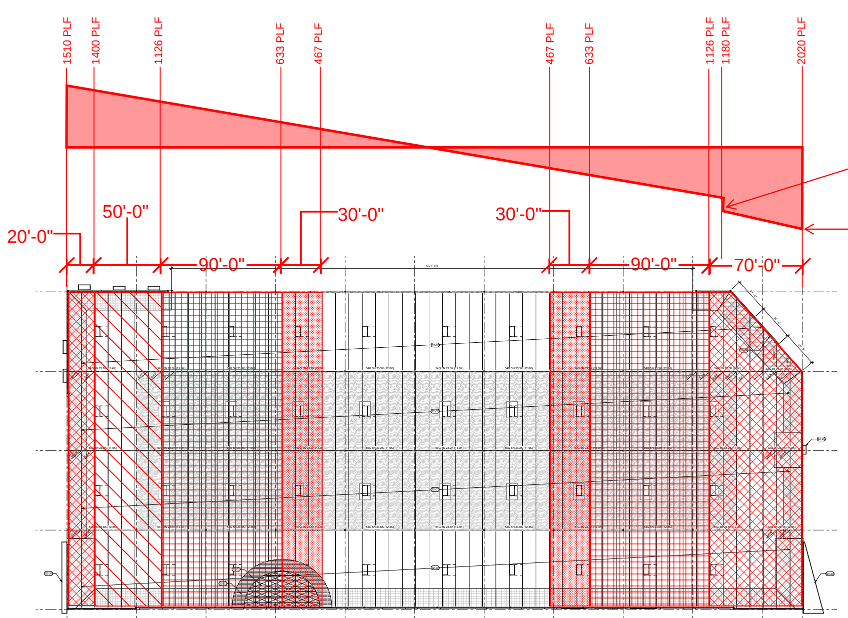
4.2.3 Unit Shear Capacities

Tabulated nominal unit shear capacities for seismic design are provided in Column A of Tables 4.2A, 4.2B, 4.2C, and 4.2D; and for wind design in Column B of Tables 4.2A, 4.2B, 4.2C, and 4.2D. The ASD allowable unit shear capacity shall be determined by dividing the tabulated nominal unit shear capacity, modified by applicable footnotes, by the ASD reduction factor of 2.0. The LRFD factored unit resistance shall be determined by multiplying the tabulated nominal unit shear capacity, modified by applicable footnotes, by a resistance factor, ϕ_D, of 0.80. No further increases shall be permitted.

640 PLF(0.8) = 512 PLF

1440 PLF(0.8) = 1152 PLF

850 PLF(0.8) = 680 PLF



SEISMIC IN E-W DIRECTION IS SATISFIED. WIND DESIGN DOES NOT GOVERN, IS SATISFIED

10d @ 2 1/2" OC STAGGERED NAILING USING 3 ROWS OF FASTENERS PER 19/S0.10 AT PANEL EDGES AND 12" OC AT INTERIOR SUPPORTS. USE 4x SUBPURLINS

10d @ 4" OC STAGGERED NAILING USING 3 ROWS OF FASTENERS PER 19/S0.10 AT PANEL EDGES AND 12" OC AT INTERIOR SUPPORTS. USE 4x SUBPURLINS

10d @ 2 1/2" OC NAILING AT PANEL EDGES AND 12" OC AT INTERIOR SUPPORTS. USE 3x MIN SUBPURLINS UNO

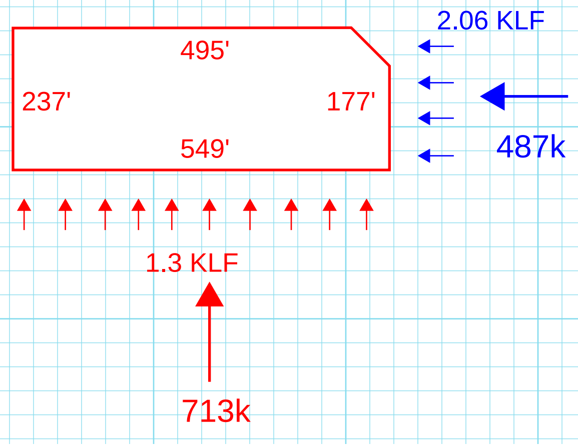
10d @ 4" OC NAILING AT PANEL EDGES AND 12" OC AT INTERIOR SUPPORTS. USE 2x MIN SUBPURLINS UNO

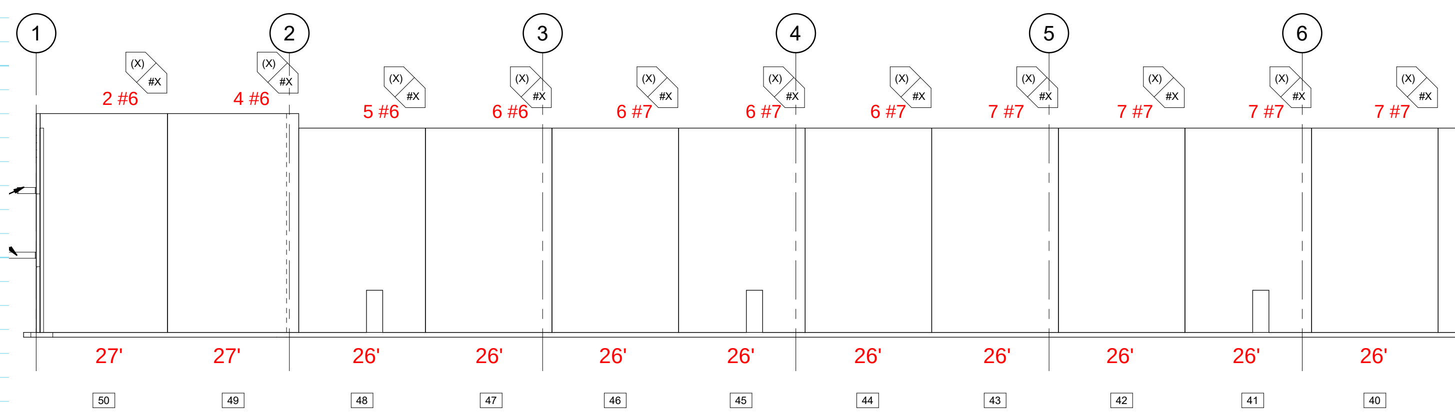
10d @ 6" OC NAILING AT PANEL EDGES AND 12" OC AT INTERIOR SUPPORTS. USE 2x SUBPURLINS UNO

CHORD DESIGN

FROM INSPECTION, SEISMIC LOADING WILL GOVERN CHORD DESIGN

SEISMIC LOADING





GRIDLINE A CHORD BARS

Pu = (wx)(.5)(l-x)/238'

(1.3 KLF)(27')(.5)(550'-27')/238' = 38.6k
As = (38.6k)/((0.9)(60ksi)) = 0.72in2, USE 2#6 (0.88in2)

(1.3 KLF)(54')(.5)(550'-54')/238' = 73.2k
As = (73.2k)/((0.9)(60ksi)) = 1.36in2, USE 4#6 (1.76in2)

(1.3 KLF)(80')(.5)(550'-80')/238' = 102.7k
As = (102.7k)/((0.9)(60ksi)) = 1.91in2, USE 5#6 (2.2in2)

(1.3 KLF)(106')(.5)(550'-106')/238' = 128.6k
As = (128.6k)/((0.9)(60ksi)) = 2.38in2, USE 6#6 (2.64in2)

(1.3 KLF)(132')(.5)(550'-132')/238' = 150.7k
As = (150.7k)/((0.9)(60ksi)) = 2.79in2, USE 6#7 (3.6in2)

(1.3 KLF)(158')(.5)(550'-158')/238' = 169.2k
As = (169.2k)/((0.9)(60ksi)) = 3.13in2, USE 6#7 (3.6in2)

(1.3 KLF)(184')(.5)(550'-184')/238' = 184k
As = (184k)/((0.9)(60ksi)) = 3.4in2, USE 6#7 (3.6in2)

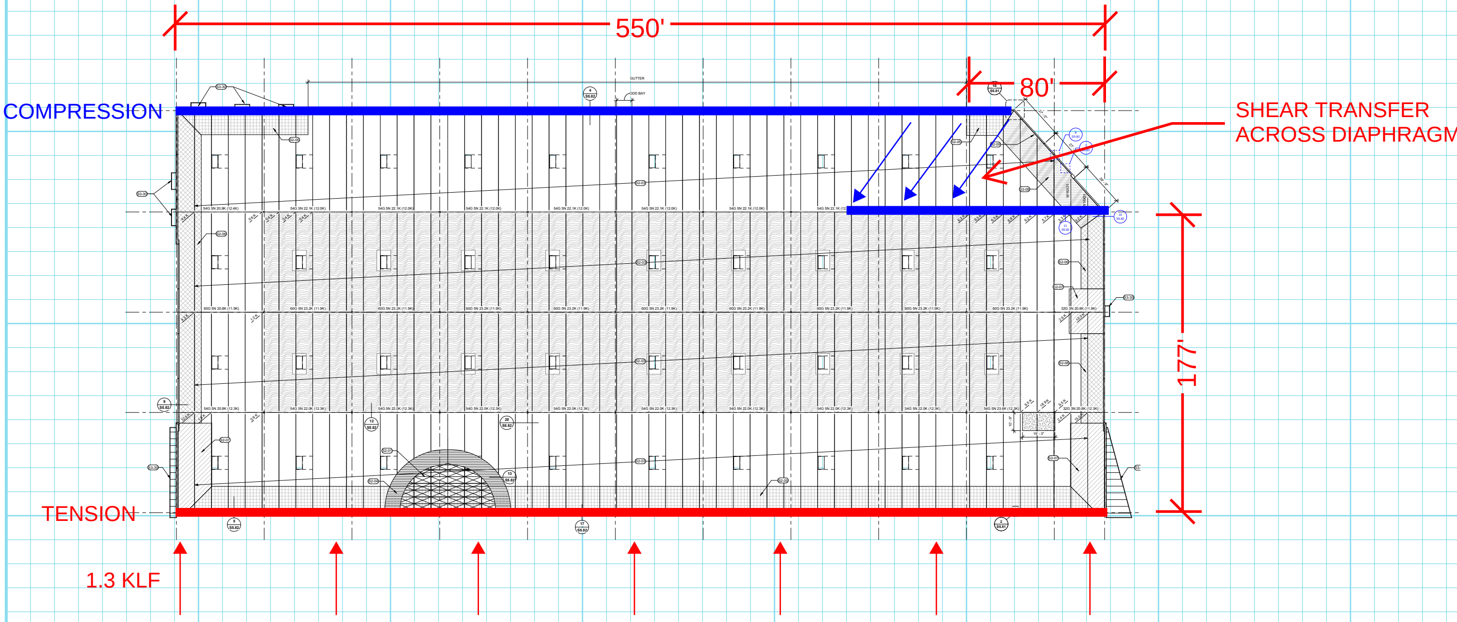
(1.3 KLF)(210')(.5)(550'-210')/238' = 195k
As = (195k)/((0.9)(60ksi)) = 3.61in2, USE 7#7 (4.2in2)

(1.3 KLF)(236')(.5)(550'-236')/238' = 202.4k
As = (202.4k)/((0.9)(60ksi)) = 3.74in2, USE 7#7 (4.2in2)

(1.3 KLF)(262')(.5)(550'-262')/238' = 206.1k
As = (206.1k)/((0.9)(60ksi)) = 3.81in2, USE 7#7 (4.2in2)

(1.3 KLF)(275')(.5)(550'-275')/238' = 206.6k
As = (206.6k)/((0.9)(60ksi)) = 3.83in2, USE 7#7 (4.2in2)

SYMMETRIC, USE SAME REINF ON OTHER SIDE



GRIDLINE E CHORD BARS

UP TO GRIDLINE 10, WILL BE THE SAME AS GRID A

MOMENT FROM 80' FROM EAST END (LAST STRAIGHT PANEL JOINT)

Mu = (1.3 KLF)(80')(.5)(550' - 80') = 24440 K-FT

AXIAL FORCE IN GRID D GIRDER TO ACT AS CHORD

Pu = (24440 K-FT)/207' = 118k

CONVERT TO ASD: (118k)(0.7) = 82.6k (MAX REQ'D GIRDER AXIAL FORCE, OCCURS AT GRID 10)

DIAPHRAGM WILL SHED LOAD WEST OF GRID 10 INTO GIRDER LINE

FORCE AT GRID 10:

(1.3 KLF)(60')(.5)(550'-60')/238' = 80.3k
(80.3k/1.152 KLF) = 69.7'

GIRDER NAILING TO EXTEND 70' + 60' = 130' FROM WALL

GRIDLINE 1 CHORD BARS

Pu = [(2.06 KLF)(237')^2/8]/550' = 26.3k

As = (26.3k)/(0.9)(60 KSI) = 0.486 IN2,

MINIMUM 2#5 WILL BE ADEQUATE AS CHORD REINF

GRID 12 WILL BE GOVERNED BY COLLECTOR DEMANDS

FORTRESS PUYALLUP - ROOF STRAPPING DESIGN

DISTRIBUTION OF Fp OVER SUBPURLINS

FROM INSPECTION, 10" PANELS ON WEST SIDE WILL GOVERN Fp FORCE:

AVG ROOF HEIGHT = (38' + 36.75')/2 = 37.4'
PARAPET ELEVATION = 43.5'

Fp = (0.4)(Sds)(ka)(Ie)(Wp)

Wp = (10/12)(1')[(37.4'/2)+6.1'](150 PCF) = 3.1 KLF

Fp = (0.4)(0.843)(2)(1)(3.1 KLF) = 2.1 KLF

Fp_{min} = (0.2)(ka)(Ie)(Wp)

= (0.2)(2)(1)(3.1 KLF) = 1.24 KLF (DOES NOT GOVERN)

MAXIMUM BAY WIDTH = 60'

60'/2.5 = 24' .... USE 30' AS MINIMUM SUBDIAPHRAGM DEPTH

JOIST AXIAL = 14.5k ASD (FROM MATHCAD)

M = (2.1)(60')^2/8 = 945 K-FT

945 K-FT/(14.5k/0.7)44.6' . . . USE 50' SUBDIAPHRAGM DEPTH

DETERMINE Fp @ 8.75" WALLS:

AVG ROOF HEIGHT = (38' + 39.25')/2 = 38.6'
PARAPET ELEVATION = 43.5'

Fp = (0.4)(Sds)(ka)(Ie)(Wp)

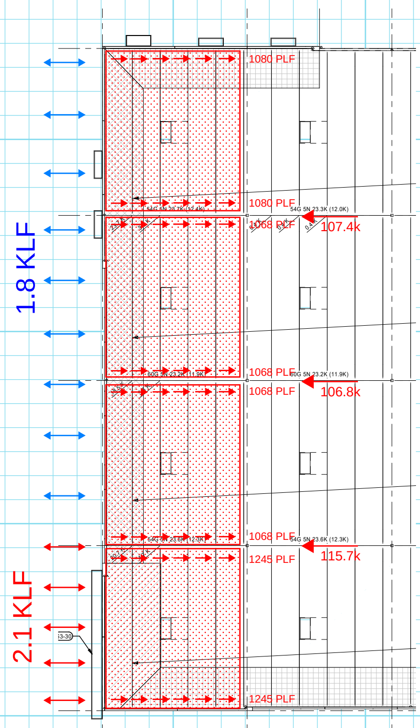
Wp = (8.75/12)(1')[(38.6'/2)+4.9'](150 PCF) = 2.65 KLF

Fp = (0.4)(0.843)(2)(1)(2.65 KLF) = 1.8 KLF

Fp_{min} = (0.2)(ka)(Ie)(Wp)

= (0.2)(2)(1)(2.65 KLF) = 0.72 KLF (DOES NOT GOVERN)

1.8 KLF CLOSE TO 2.1, USE 2.1 KLF FOR DESIGN ALONG ELEVATION



(2.1 KLF)(59.33')/(2)(50') = 1245 PLF AT A-B SUBDIAPHRAGM EDGE (LRFD)
(1.8 KLF)(59.33')/(2)(50') = 1068 PLF AT B-D SUB DIAPHRAGM EDGES (LRFD)
(1.8 KLF)(60')/(2)(50') = 1080 PLF AT D-E SUBDIAPHRAGM EDGE (LRFD)

(1245 PLF + 1068 PLF)(50') = 115.7k
(1068 PLF + 1068 PLF)(50') = 106.8k
(1068 PLF + 1080 PLF)(50') = 107.4k

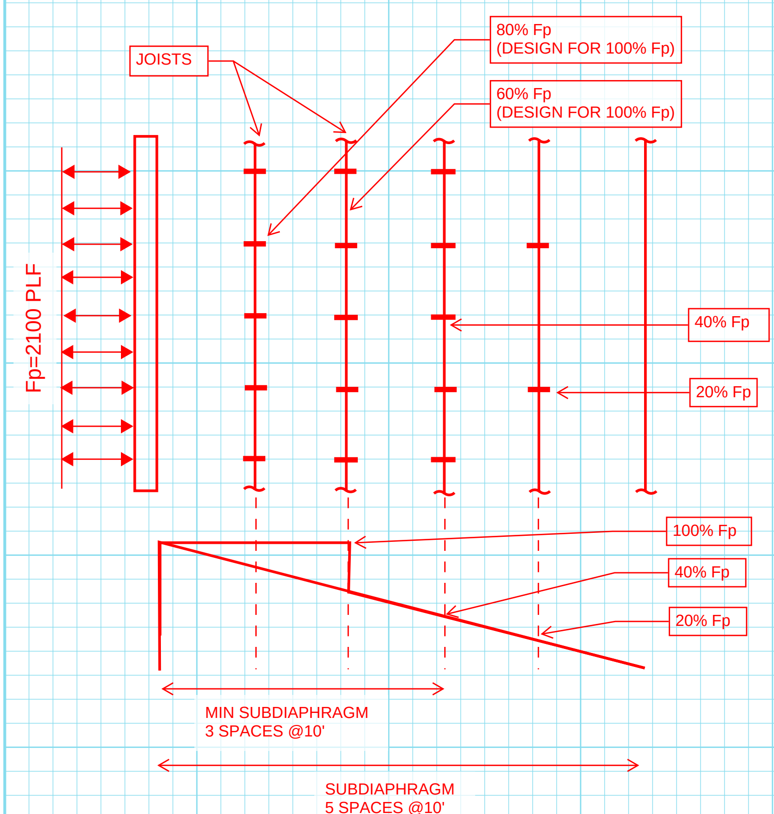
100% Fp, CONVERT TO ASD:

STRAPS @4' OC
(2100 PLF)(1.0)(4')(0.7)(1.4) = 8232# (STRAPS)
CMST12 @ 4' OC

40% Fp, CONVERT TO ASD:
STRAPS @4' OC
(2100 PLF)(0.4)(4')(0.7)(1.4) = 3293# (STRAPS)

20% Fp, CONVERT TO ASD:
STRAPS @8' OC
(2100 PLF)(0.2)(8')(0.7)(1.4) = 3293# (STRAPS)

(INCLUDE 1.4 FACTOR PER ASCE 7-16 12.11.2.2.2)



Model No.	Total L	Ga.	DF/SP		SPF/HF		Allowable Tension Loads (160)
			Fasteners (in.)	End Length (in.)	Fasteners (in.)	End Length (in.)	
CMST12	40'	12	(74) 0.162 x 2½	33	(84) 0.162 x 2½	38	9,215
			(86) 0.148 x 2½	39	(98) 0.148 x 2½	44	9,215

CMST12 STRAPS W/ (86) TOTAL NAILS
9215# > 8232#, OK

MSTI26	12	2 1/16	26	(26) 0.148 x 1½	2,745	2,380
MSTI36		2 1/16	36	(36) 0.148 x 1½	3,800	3,295
MSTI48		2 1/16	48	(48) 0.148 x 1½	5,070	4,390
MSTI60		2 1/16	60	(60) 0.148 x 1½	5,070	5,070
MSTI72		2 1/16	72	(72) 0.148 x 1½	5,070	5,070

MSTI36 STRAPS W/ (36) TOTAL NAILS
3800# > 3293#, OK

CONSERVATIVELY CARRY THRY 100% Fp OVER MIN SUBDIAPHRAGM WIDTH... OVER 1ST, 2ND JOISTS

HOLD DOWN SELECTION

Fp = 2.1 KLF (10" PANELS)

(2.1 KLF)(2')(1.4)(0.7) = 4116#

(HOLDDOWN @ 2' OC, CONVERTED TO ASD, INCLUDE 1.4 FACTOR)

Model No.	Ga.	Dimensions (in.)					Fasteners (in.)		Minimum Wood Member Size (in.)	Allowable Tension Loads (160)		
		W	H	B	CL	SO	Anchor Bolt Dia. (in.)	Wood Fasteners		DF/SP	SPF/HF	Deflection at Allowable Load (in.)
HDU2-SDS2.5	14	3	8 11/16	3 1/4	1 5/16	1 3/8	5/8	(6) 1/4 x 2 1/2 SDS	3 x 3 1/2	3,075	2,215	0.088
HDU4-SDS2.5	14	3	10 15/16	3 1/4	1 5/16	1 3/8	5/8	(10) 1/4 x 2 1/2 SDS	3 x 3 1/2	4,565	3,285	0.114
HDU5-SDS2.5	14	3	13 3/16	3 1/4	1 5/16	1 3/8	5/8	(14) 1/4 x 2 1/2 SDS	3 x 3 1/2	5,645	4,340	0.115

USE HDU4-SDS2.5 @ 2' OC (4116# < 4565#)

CHECK EPOXY BOLT IN SIMPSON SOFTWARE

OVERSTRENGTH = 2.0, ACI318-19 17.10.5.3d SATISFIED

Pu = (2.1 KLF)(2')(2) = 8400 LB

SIMPSON SET 3G, 5/8" DIA, 7.5" EFF EMBED

Fp = 1.8 KLF (8.75" PANELS)

(1.8 KLF)(2')(1.4)(0.7) = 3528#

(HOLDDOWN @ 2' OC, CONVERTED TO ASD, INCLUDE 1.4 FACTOR)

Model No.	Ga.	Dimensions (in.)					Fasteners (in.)		Minimum Wood Member Size (in.)	Allowable Tension Loads (160)		
		W	H	B	CL	SO	Anchor Bolt Dia. (in.)	Wood Fasteners		DF/SP	SPF/HF	Deflection at Allowable Load (in.)
HDU2-SDS2.5	14	3	8 11/16	3 1/4	1 5/16	1 3/8	5/8	(6) 1/4 x 2 1/2 SDS	3 x 3 1/2	3,075	2,215	0.088
HDU4-SDS2.5	14	3	10 15/16	3 1/4	1 5/16	1 3/8	5/8	(10) 1/4 x 2 1/2 SDS	3 x 3 1/2	4,565	3,285	0.114
HDU5-SDS2.5	14	3	13 3/16	3 1/4	1 5/16	1 3/8	5/8	(14) 1/4 x 2 1/2 SDS	3 x 3 1/2	5,645	4,340	0.115

USE HDU4-SDS2.5 @ 2' OC (3528# < 4565#)

CHECK EPOXY BOLT IN SIMPSON SOFTWARE

OVERSTRENGTH = 2.0, ACI318-19 17.10.5.3d SATISFIED

Pu = (1.8 KLF)(2')(2) = 7200 LB

SIMPSON SET 3G, 5/8" DIA, 6.25" EFF EMBED

SUBDIAPHRAGM EDGE NAILING

AT EDGE OF SUBDIAPHRAGM 1245 PLF LRFD

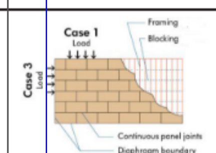
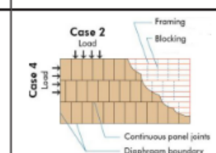
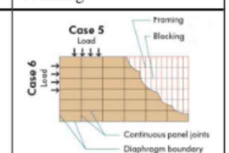
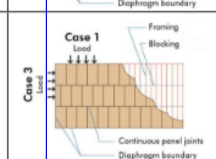
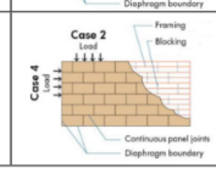
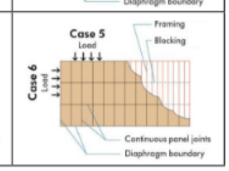
Table 4.2B Nominal Unit Shear Capacities for Wood-Frame Diaphragms

Blocked Wood Structural Panel Diaphragms Utilizing Multiple Rows of Fasteners (High Load Diaphragms)[1,2,3,4,5]

CASE 5 & 6, USE LOWER CAPACITIES, CONSERVATIVE

Sheathing Grade	Common Nail Size	Minimum Fastener Penetration in Framing Member or Blocking (in.)	Minimum Nominal Panel Thickness (in.)	Minimum Nominal Width of Nailed Face at Adjoining Panel Edges and Boundaries (in.)	Lines of Fasteners	A SEISMIC												B WIND			
						Nail Spacing (in.) at diaphragm boundaries (all cases), at continuous panel edges parallel to load (Cases 3 & 4), and at all panel edges (Cases 5 & 6)												Nail Spacing (in.) at diaphragm boundaries (all cases), at continuous panel edges parallel to load (Cases 3 & 4), and at all panel edges (Cases 5 & 6)			
						4			4			2-1/2			2-1/2			4	4	2-1/2	2-1/2
						Nail Spacing (in.) at other panel edges (Cases 1, 2, 3, & 4)												Nail Spacing (in.) at other panel edges (Cases 1, 2, 3, & 4)			
						6			4			4			3			6	4	4	3
						v_s (plf)	G_a (kips/in.) OSB	G_a (kips/in.) PLY	v_s (plf)	G_a (kips/in.) OSB	G_a (kips/in.) PLY	v_s (plf)	G_a (kips/in.) OSB	G_a (kips/in.) PLY	v_s (plf)	G_a (kips/in.) OSB	G_a (kips/in.) PLY	v_w (plf)	v_w (plf)	v_w (plf)	v_w (plf)
Structural I	10d	1-1/2	15/32	3	2	1210	40	24	1630	53	28	1750	50	27	2300	56	29	1695	2280	2450	3220
				4	2	1400	33	21	1830	48	27	2010	44	25	2580	51	28	1960	2560	2815	3610
				4	3	1750	50	27	2440	61	30	2570	59	30	2790	70	32	2450	3415	3600	3905
			19/32	3	2	1340	36	23	1760	52	29	1930	47	27	2510	54	29	1875	2465	2700	[illegible]
				4	2	1560	29	20	1980	46	27	2220	40	25	2880	48	27	2185	2770	3110	[illegible]
				4	3	1930	47	27	2640	60	31	2810	57	30	3580	64	32	2700	3595	3935	5010
			23/32	3	2	1460	33	22	1910	50	29	2100	45	27	2730	53	30	2045	2675	2940	3820
				4	2	1710	26	19	2140	43	27	2420	37	24	3130	45	27	2395	2995	3390	4380
				4	3	2100	45	27	2860	59	32	3050	56	31	3600	68	34	2940	4005	4270	5040
Sheathing and Single-Floor	10d	1-1/2	15/32	3	2	1050	43	21	1450	55	23	1530	53	23	2020	58	24	1470	2030	2140	2830
				4	2	1210	36	19	1630	50	22	1750	46	21	2210	55	23	1695	2280	2450	3095
				4	3	1530	53	23	2170	62	24	2260	61	24	2390	72	26	2140	3040	3165	3345
			19/32	3	2	1300	34	19	1720	49	23	1870	45	22	2450	52	23	1820	2410	2620	3430
				4	2	1510	27	16	1930	43	21	2160	37	20	2740	46	22	2115	2700	3025	3835
				4	3	1870	45	22	2580	57	24	2730	55	24	2970	68	26	2620	3610	3820	4160
			23/32	3	2	1420	30	18	1870	46	23	2040	42	22	2670	50	24	1990	2620	2855	3740
				4	2	1650	24	16	2100	40	21	2350	34	20	2890	45	23	2310	2940	3290	4045
				4	3	2040	42	22	2800	56	25	2960	53	25	3130	71	28	2855	3920	4145	4380

1. Nominal unit shear capacities shall be adjusted in accordance with 4.2.3 to determine ASD allowable unit shear capacity and LRFD factored unit resistance. For general construction requirements see 4.2.6. For specific requirements, see 4.2.7.1 for wood structural panel diaphragms. See Appendix A for common nail dimensions.
2. For species and grades of framing other than Douglas-Fir-Larch or Southern Pine, reduced nominal unit shear capacities shall be determined by multiplying the tabulated nominal unit shear capacity by the Specific Gravity Adjustment Factor = [1-(0.5-G)], where G = Specific Gravity of the framing lumber from the *NDS* (Table 12.3.3A). The Specific Gravity Adjustment Factor shall not be greater than 1.
3. Apparent shear stiffness values, G_a, are based on nail slip in framing with moisture content less than or equal to 19% at time of fabrication and panel stiffness values for diaphragms constructed with either OSB or 3-ply plywood panels. When 4-ply or 5-ply plywood panels or composite panels are used, G_a values shall be permitted to be multiplied by 1.2.
4. Where moisture content of the framing is greater than 19% at time of fabrication, G_a values shall be multiplied by 0.5.
5. Diaphragm resistance depends on the direction of continuous panel joints with respect to the loading direction and direction of framing members, and is independent of the panel orientation.

	Cases 1&3: Continuous Panel Joints Perpendicular to Framing	**Cases 2&4**: Continuous Panel Joints Parallel to Framing	**Cases 5&6**: Continuous Panel Joints Perpendicular and Parallel to Framing
Long Panel Direction Perpendicular to Supports			
Long Panel Direction Parallel to Supports[a]			

(a) Panel span rating for out-of-plane loads may be lower than the span rating with the long panel direction perpendicular to supports (See Section 3.2.2 and Section 3.2.3)

1750 PLF(0.8) = 1400 PLF (SEISMIC CAPACITY) > 1245 PLF

2570PLF(0.8) = 2056 PLF (SEISMIC CAPACITY) > 1245 PLF

SUBDIAPHRAGM EDGE DEMAND SATISFIED BY DIAPHRAGM NAILING

SIMPSON Strong-Tie®

Anchor Designer™
Software
Version 3.1.2303.1

Company:	Mackenzie	Date:	5/25/2023
Engineer:	ATT	Page:	1/5
Project:	Fortress Puyallup		
Address:			
Phone:			
E-mail:			

1.Project information

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

Project description:
Location: Hold Down Anchors at 10" Panels
Fastening description:

2. Input Data & Anchor Parameters

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

Anchor Information:
Anchor type: Bonded anchor
Material: F1554 Grade 36
Diameter (inch): 0.750
Effective Embedment depth, h_{ef} (inch): 7.500
Code report: ICC-ES ESR-4057
Anchor category: -
Anchor ductility: Yes
h_{min} (inch): 9.25
c_{ac} (inch): 19.22
C_{min} (inch): 1.75
S_{min} (inch): 3.00

Base Material
Concrete: Normal-weight
Concrete thickness, h (inch): 10.00
State: Cracked
Compressive strength, f'_c (psi): 4000
$\Psi_{c,V}$: 1.0
Reinforcement condition: Supplementary reinforcement not present
Supplemental edge reinforcement: Not applicable
Reinforcement provided at corners: No
Ignore concrete breakout in tension: No
Ignore concrete breakout in shear: No
Hole condition: Dry concrete
Inspection: Continuous
Temperature range, Short/Long: 150/110°F
Reduced installation torque (for AT-3G): Not applicable
Ignore 6do requirement: Not applicable
Build-up grout pad: No

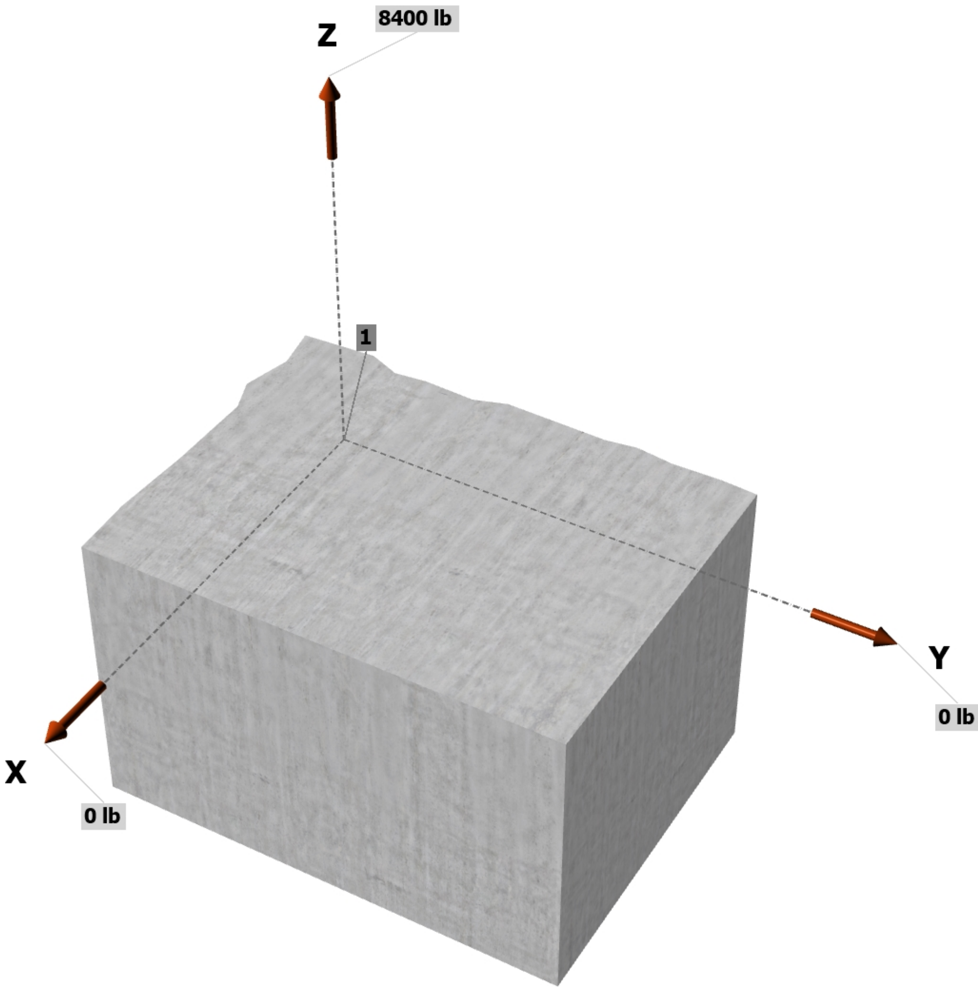
Recommended Anchor
Anchor Name: SET-3G™ - SET-3G w/ 3/4"Ø F1554 Gr. 36
Code Report: ICC-ES ESR-4057

Load and Geometry
Load factor source: ACI 318 Section 5.3
Load combination: not set
Seismic design: Yes
Anchors subjected to sustained tension: No
Ductility section for tension: 17.10.5.3 (d) is satisfied
Ductility section for shear: 17.10.6.2 not applicable
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]: 8400
V_{uax} [lb]: 0
V_{uay} [lb]: 0

<Figure 1>



<Figure 2>

3. Resulting Anchor Forces

Anchor	Tension load, N_{ua} (lb)	Shear load x, V_{uax} (lb)	Shear load y, V_{uay} (lb)	Shear load combined, $\sqrt{(V_{uax})^2+(V_{uay})^2}$ (lb)
1	8400.0	0.0	0.0	0.0
Sum	8400.0	0.0	0.0	0.0

Maximum concrete compression strain (‰): 0.00
Maximum concrete compression stress (psi): 0
Resultant tension force (lb): 8400
Resultant compression force (lb): 0
Eccentricity of resultant tension forces in x-axis, e'_{Nx} (inch): 0.00
Eccentricity of resultant tension forces in y-axis, e'_{Ny} (inch): 0.00

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

N_{sa} (lb)	ϕ	ϕN_{sa} (lb)
19370	0.75	14528

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

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

k_c	λ_a	f'_c (psi)	h_{ef} (in)	N_b (lb)
17.0	1.00	4000	7.500	22084

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

A_{Nc} (in^2)	A_{Nco} (in^2	$c_{a,min}$ (in)	$\Psi_{ed,N}$	$\Psi_{c,N}$	$\Psi_{cp,N}$	N_b (lb)	ϕ	$0.75\phi N_{cb}$ (lb)
433.13	506.25	8.00	0.913	1.00	1.000	22084	0.65	8412

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

$\tau_{k,cr} = \tau_{k,cr} f_{short\text{-}term} K_{sat} (f'_c / 2{,}500)^n \alpha_{N.seis}$

$\tau_{k,cr}$ (psi)	$f_{short\text{-}term}$	K_{sat}	$\alpha_{N.seis}$	f'_c (psi)	n	$\tau_{k,cr}$ (psi)
1310	1.00	1.00	1.00	4000	0.24	1466

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

λ_a	τ_{cr} (psi)	d_a (in)	h_{ef} (in)	N_{ba} (lb)
1.00	1466	0.75	7.500	25914

$0.75\phi N_a = 0.75\phi\,(A_{Na} / A_{Na0})\,\Psi_{ed,Na}\,\Psi_{cp,Na} N_{ba}$ (Sec. 17.5.1.2 & Eq. 17.6.5.1a)

A_{Na} (in^2)	A_{Na0} (in^2)	c_{Na} (in)	$c_{a,min}$ (in)	$\Psi_{ed,Na}$	$\Psi_{p,Na}$	N_{a0} (lb)	ϕ	$0.75\phi N_a$ (lb)
375.47	422.18	10.27	8.00	0.934	1.000	25914	0.65	10489

11. Results

Interaction of Tensile and Shear Forces (Sec. 17.8)

Tension	Factored Load, N_{ua} (lb)	Design Strength, $øN_n$ (lb)	Ratio	Status
Steel	8400	14528	0.58	Pass
Concrete breakout	**8400**	**8412**	**1.00**	**Pass (Governs)**
Adhesive	8400	10489	0.80	Pass

SET-3G w/ 3/4"Ø F1554 Gr. 36 with hef = 7.500 inch meets the selected design criteria.

12. Warnings

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

- Per designer input, the shear component of the strength-level earthquake force applied to anchors does not exceed 20 percent of the total factored anchor shear force associated with the same load combination. Therefore the ductility requirements of ACI 318 17.10.6.2 for shear need not be satisfied – designer to verify.

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

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

SIMPSON Strong-Tie®

Anchor Designer™
Software
Version 3.1.2303.1

Company:	Mackenzie	Date:	5/25/2023
Engineer:	ATT	Page:	1/5
Project:	Fortress Puyallup		
Address:			
Phone:			
E-mail:			

1.Project information

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

Project description:
Location: Hold Down Anchors at 8.75" Panels
Fastening description:

2. Input Data & Anchor Parameters

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

Anchor Information:
Anchor type: Bonded anchor
Material: F1554 Grade 36
Diameter (inch): 0.750
Effective Embedment depth, h_{ef} (inch): 6.250
Code report: ICC-ES ESR-4057
Anchor category: -
Anchor ductility: Yes
h_{min} (inch): 8.00
c_{ac} (inch): 15.11
C_{min} (inch): 1.75
S_{min} (inch): 3.00

Base Material
Concrete: Normal-weight
Concrete thickness, h (inch): 8.75
State: Cracked
Compressive strength, f'_c (psi): 4000
$\Psi_{c,V}$: 1.0
Reinforcement condition: Supplementary reinforcement not present
Supplemental edge reinforcement: Not applicable
Reinforcement provided at corners: No
Ignore concrete breakout in tension: No
Ignore concrete breakout in shear: No
Hole condition: Dry concrete
Inspection: Continuous
Temperature range, Short/Long: 150/110°F
Reduced installation torque (for AT-3G): Not applicable
Ignore 6do requirement: Not applicable
Build-up grout pad: No

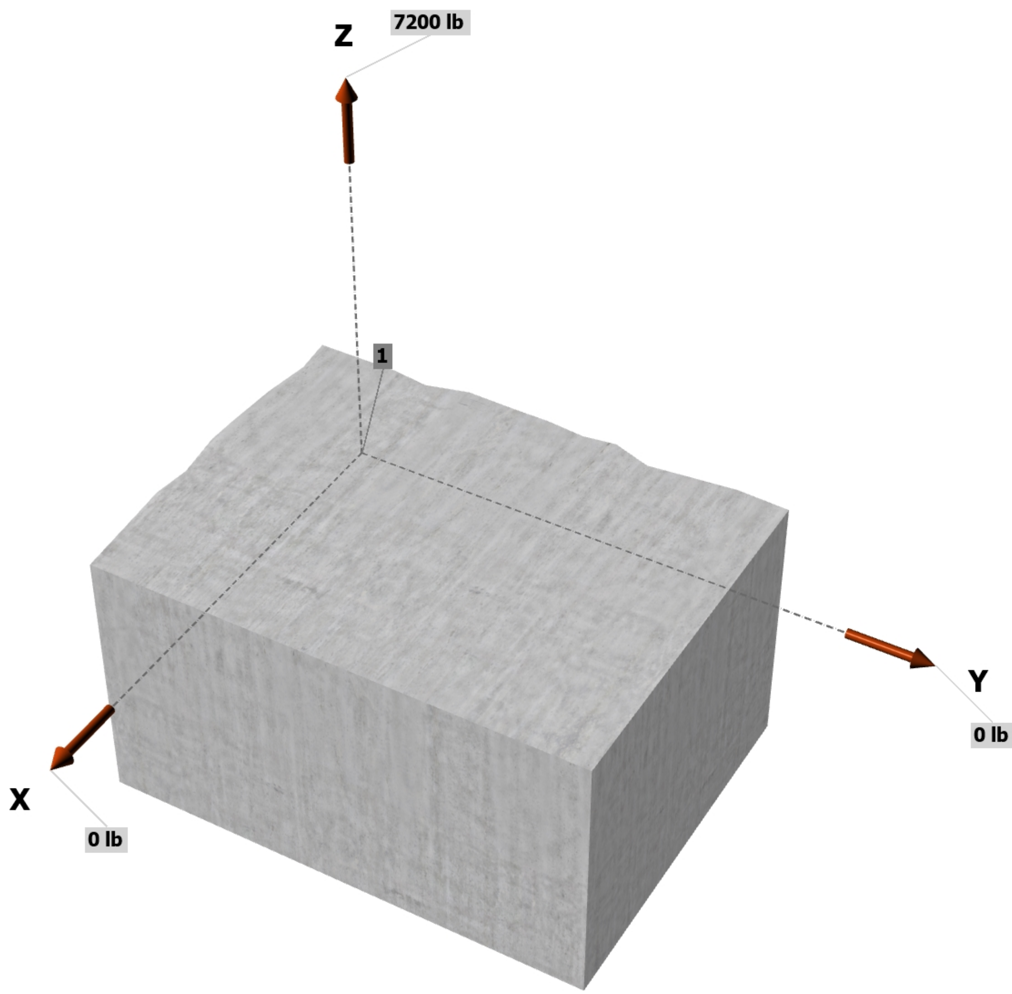
Recommended Anchor
Anchor Name: SET-3G™ - SET-3G w/ 3/4"Ø F1554 Gr. 36
Code Report: ICC-ES ESR-4057

Load and Geometry
Load factor source: ACI 318 Section 5.3
Load combination: not set
Seismic design: Yes
Anchors subjected to sustained tension: No
Ductility section for tension: 17.10.5.3 (d) is satisfied
Ductility section for shear: 17.10.6.2 not applicable
Ω_0 factor: not set
Apply entire shear load at front row: No
Anchors only resisting wind and/or seismic loads: Yes

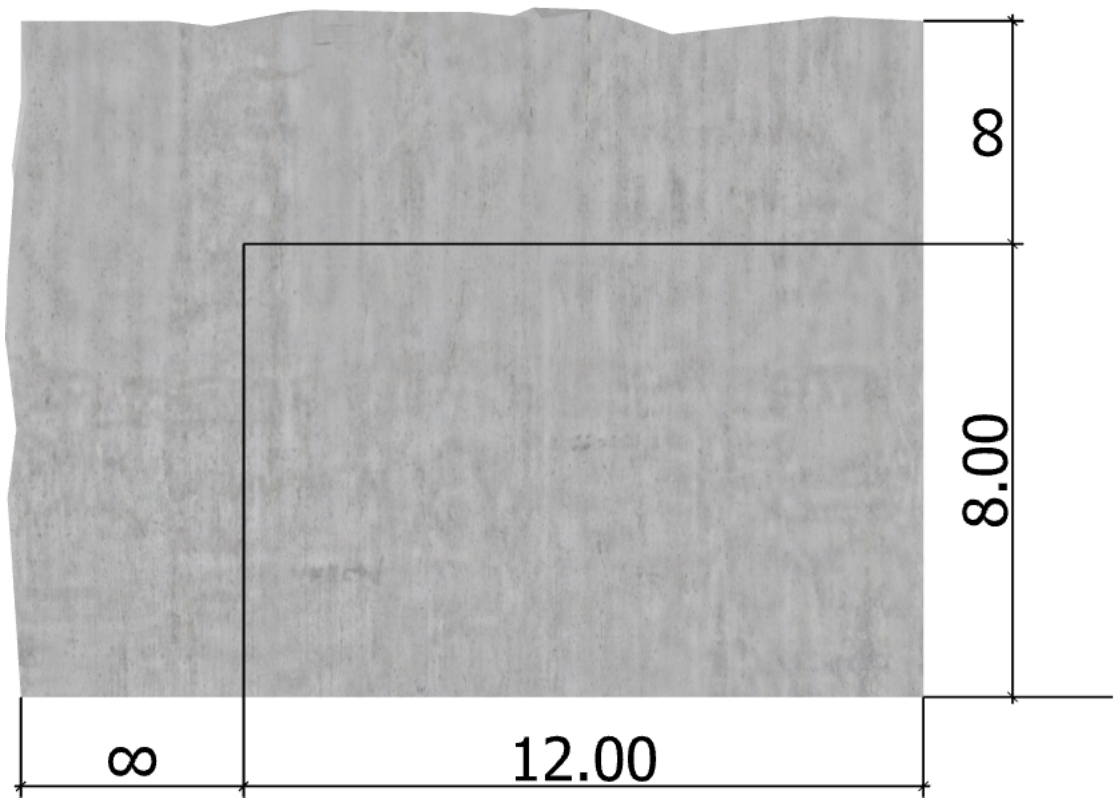
Strength level loads:

N_{ua} [lb]: 7200
V_{uax} [lb]: 0
V_{uay} [lb]: 0

<Figure 1>



<Figure 2>



3. Resulting Anchor Forces

Anchor	Tension load, N_{ua} (lb)	Shear load x, V_{uax} (lb)	Shear load y, V_{uay} (lb)	Shear load combined, $\sqrt{(V_{uax})^2+(V_{uay})^2}$ (lb)
1	7200.0	0.0	0.0	0.0
Sum	7200.0	0.0	0.0	0.0

Maximum concrete compression strain (‰): 0.00
Maximum concrete compression stress (psi): 0
Resultant tension force (lb): 7200
Resultant compression force (lb): 0
Eccentricity of resultant tension forces in x-axis, e'_{Nx} (inch): 0.00
Eccentricity of resultant tension forces in y-axis, e'_{Ny} (inch): 0.00

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

N_{sa} (lb)	ϕ	ϕN_{sa} (lb)
19370	0.75	14528

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

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

k_c	λ_a	f'_c (psi)	h_{ef} (in)	N_b (lb)
17.0	1.00	4000	6.250	16800

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

A_{Nc} (in^2)	A_{Nco} (in^2	$c_{a,min}$ (in)	$\Psi_{ed,N}$	$\Psi_{c,N}$	$\Psi_{cp,N}$	N_b (lb)	ϕ	$0.75\phi N_{cb}$ (lb)
325.78	351.56	8.00	0.956	1.00	1.000	16800	0.65	7255

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

$\tau_{k,cr} = \tau_{k,cr} f_{short\text{-}term} K_{sat} (f'_c / 2{,}500)^n \alpha_{N.seis}$

$\tau_{k,cr}$ (psi)	$f_{short\text{-}term}$	K_{sat}	$\alpha_{N.seis}$	f'_c (psi)	n	$\tau_{k,cr}$ (psi)
1310	1.00	1.00	1.00	4000	0.24	1466

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

λ_a	τ_{cr} (psi)	d_a (in)	h_{ef} (in)	N_{ba} (lb)
1.00	1466	0.75	6.250	21595

$0.75\phi N_a = 0.75\phi\,(A_{Na} / A_{Na0})\,\Psi_{ed,Na}\,\Psi_{cp,Na} N_{ba}$ (Sec. 17.5.1.2 & Eq. 17.6.5.1a)

A_{Na} (in^2)	A_{Na0} (in^2)	c_{Na} (in)	$c_{a,min}$ (in)	$\Psi_{ed,Na}$	$\Psi_{p,Na}$	N_{a0} (lb)	ϕ	$0.75\phi N_a$ (lb)
375.47	422.18	10.27	8.00	0.934	1.000	21595	0.65	8741

11. Results

Interaction of Tensile and Shear Forces (Sec. 17.8)

Tension	Factored Load, N_{ua} (lb)	Design Strength, $øN_n$ (lb)	Ratio	Status
Steel	7200	14528	0.50	Pass
Concrete breakout	**7200**	**7255**	**0.99**	**Pass (Governs)**
Adhesive	7200	8741	0.82	Pass

SET-3G w/ 3/4"Ø F1554 Gr. 36 with hef = 6.250 inch meets the selected design criteria.

12. Warnings

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

- Per designer input, the shear component of the strength-level earthquake force applied to anchors does not exceed 20 percent of the total factored anchor shear force associated with the same load combination. Therefore the ductility requirements of ACI 318 17.10.6.2 for shear need not be satisfied – designer to verify.

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

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

POST INSTALLED LEDGER BOLT

BASED ON DIAPHRAGM CALCS, SEISMIC WILL GOVERN

(2020 PLF)(2') = 4040# (SEISMIC)

PER ACI 318-19 17.10.6.3c, USE OVERSTRENGTH, = 2.0

(4040#)(2) = 8080# SEISMIC W/ OVERSTREGTH

GRAVITY SHEAR FORCE:

(10'/2)(2' OC)(19 PSF) = 190# DL

(10'/2)(2' OC)(20 PSF) = 200# LL

(10'/2)(2' OC)(19 PSF + 63 PSF) = 820# SL (ASSUMES FULL SNOW DRIFT)

Vu = (1.2)(190#) + (1.0)(200#) + (0.2)(820#) = 592#

SEE SIMPSON CALCS

1.Project information

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

Project description:
Location: Ledger Bolt
Fastening description:

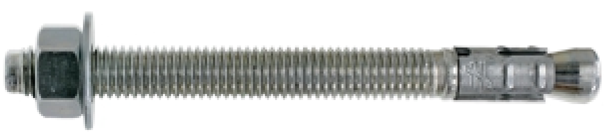
2. Input Data & Anchor Parameters

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

Anchor Information:
Anchor type: Torque controlled expansion anchor
Material: Stainless Steel
Diameter (inch): 0.750
Nominal Embedment depth (inch): 5.750
Effective Embedment depth, h_{ef} (inch): 5.000
Code report: ICC-ES ESR-3037
Anchor category: 1
Anchor ductility: Yes
h_{min} (inch): 8.75
c_{ac} (inch): 8.00
C_{min} (inch): 6.00
S_{min} (inch): 6.50

Base Material
Concrete: Normal-weight
Concrete thickness, h (inch): 8.75
State: Cracked
Compressive strength, f'_c (psi): 4000
$\Psi_{c,V}$: 1.0
Reinforcement condition: Supplementary reinforcement present
Supplemental edge reinforcement: Not applicable
Reinforcement provided at corners: No
Ignore concrete breakout in tension: No
Ignore concrete breakout in shear: No
Ignore 6do requirement: Not applicable
Build-up grout pad: No

Recommended Anchor
Anchor Name: Strong-Bolt® 2 Stainless Steel - 3/4"Ø SS Strong-Bolt 2, hnom:5.75" (146mm)
Code Report: ICC-ES ESR-3037

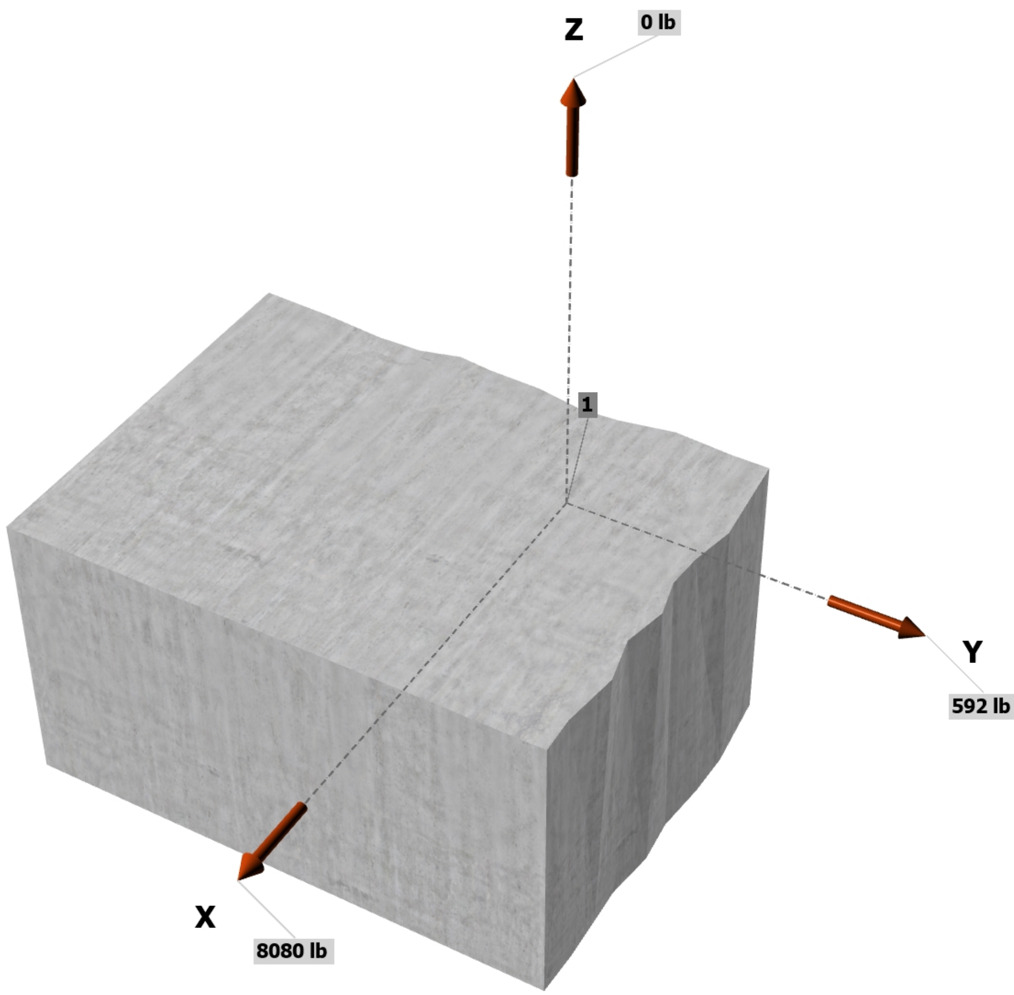


Load and Geometry
Load factor source: ACI 318 Section 5.3
Load combination: U = 1.2D + 1.0E + 1.0L + 0.2S
Seismic design: Yes
Anchors subjected to sustained tension: Not applicable
Ductility section for tension: 17.10.5.2 not applicable
Ductility section for shear: 17.10.6.3 (c) is satisfied
Ω_0 factor: 2.0
Apply entire shear load at front row: No
Anchors only resisting wind and/or seismic loads: Yes

Service level loads:

	D	E	L	S	*Strength level loads*
N_a [lb]:	0	0	0	0	*0*
V_{ax} [lb]:	0	4040	0	0	*8080*
V_{ay} [lb]:	190	0	200	820	*592*

<Figure 1>



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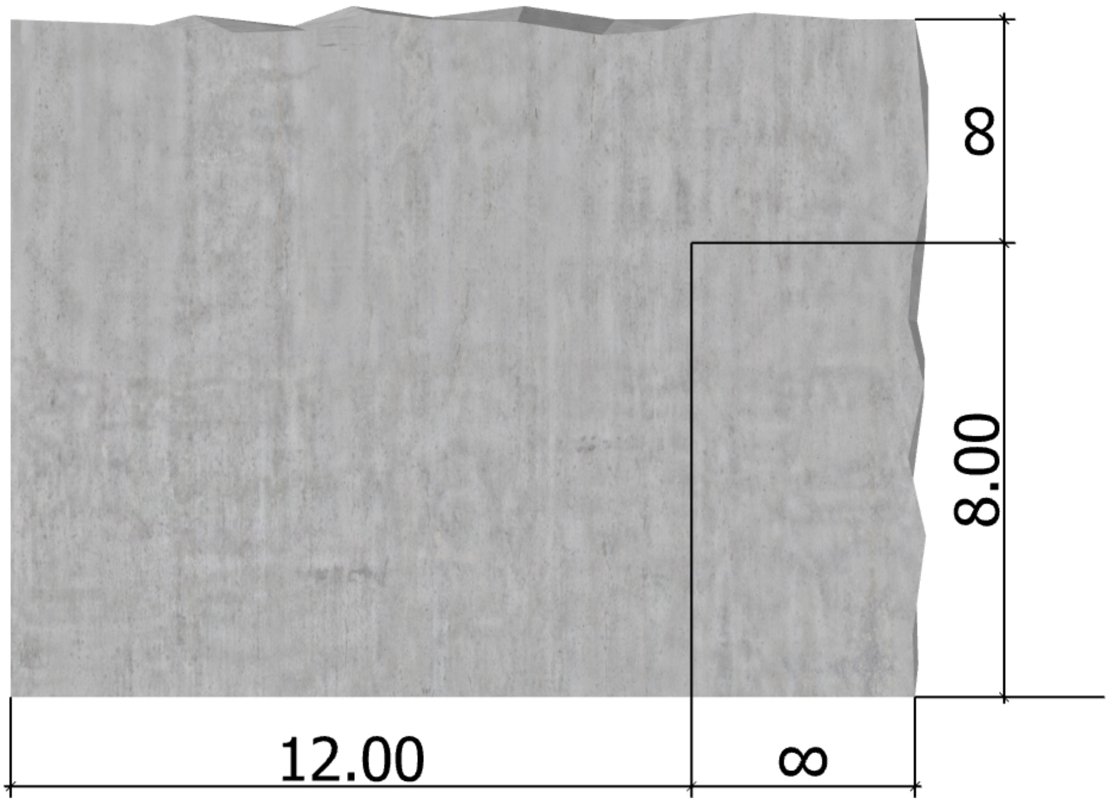


3. Resulting Anchor Forces

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	8080.0	592.0	8101.7
Sum	0.0	8080.0	592.0	8101.7

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

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

V_{sa} (lb)	ϕ_{grout}	ϕ	$\phi_{grout}\phi V_{sa}$ (lb)
13620	1.0	0.65	8853

9. Concrete Breakout Strength of Anchor in Shear (Sec. 17.7.2)

Shear perpendicular to edge in x-direction:

$V_{bx} = \min|7(l_e/d_a)^{0.2}\sqrt{d_a}\lambda_a\sqrt{f'_c}c_{a1}^{1.5}; 9\lambda_a\sqrt{f'_c}c_{a1}^{1.5}|$ (Eq. 17.7.2.2.1a & Eq. 17.7.2.2.1b)

l_e (in)	d_a (in)	λ_a	f'_c (psi)	c_{a1} (in)	V_{bx} (lb)
5.00	0.750	1.00	4000	8.00	12679

$\phi V_{cbx} = \phi\,(A_{Vc}/A_{Vco})\Psi_{ed,V}\Psi_{c,V}\Psi_{h,V}V_{bx}$ (Sec. 17.5.1.2 & Eq. 17.7.2.1a)

A_{Vc} (in^2)	A_{Vco} (in^2)	$\Psi_{ed,V}$	$\Psi_{c,V}$	$\Psi_{h,V}$	V_{bx} (lb)	ϕ	ϕV_{cbx} (lb)
210.00	288.00	1.000	1.000	1.171	12679	0.75	8120

Shear parallel to edge in x-direction:

$V_{by} = \min|7(l_e/d_a)^{0.2}\sqrt{d_a}\lambda_a\sqrt{f'_c}c_{a1}^{1.5}; 9\lambda_a\sqrt{f'_c}c_{a1}^{1.5}|$ (Eq. 17.7.2.2.1a & Eq. 17.7.2.2.1b)

l_e (in)	d_a (in)	λ_a	f'_c (psi)	c_{a1} (in)	V_{by} (lb)
5.00	0.750	1.00	4000	8.00	12679

$\phi V_{cbx} = \phi\,(2)(A_{Vc}/A_{Vco})\Psi_{ed,V}\Psi_{c,V}\Psi_{h,V}V_{by}$ (Sec. 17.5.1.2, 17.7.2.1(c) & Eq. 17.7.2.1a)

A_{Vc} (in^2)	A_{Vco} (in^2)	$\Psi_{ed,V}$	$\Psi_{c,V}$	$\Psi_{h,V}$	V_{by} (lb)	ϕ	ϕV_{cbx} (lb)
210.00	288.00	1.000	1.000	1.171	12679	0.75	16240

Shear parallel to edge in y-direction:

$V_{bx} = \min|7(l_e/d_a)^{0.2}\sqrt{d_a}\lambda_a\sqrt{f'_c}c_{a1}^{1.5}; 9\lambda_a\sqrt{f'_c}c_{a1}^{1.5}|$ (Eq. 17.7.2.2.1a & Eq. 17.7.2.2.1b)

l_e (in)	d_a (in)	λ_a	f'_c (psi)	c_{a1} (in)	V_{bx} (lb)

5.00	0.750	1.00	4000	12.00	23292

$\phi V_{cby} = \phi\ (2)(A_{Vc} / A_{Vco})\,\Psi_{ed,V}\,\Psi_{c,V}\,\Psi_{h,V}V_{bx}$ (Sec. 17.5.1.2, 17.7.2.1(c) & Eq. 17.7.2.1a)

A_{Vc} (in^2)	A_{Vco} (in^2)	$\Psi_{ed,V}$	$\Psi_{c,V}$	$\Psi_{h,V}$	V_{bx} (lb)	ϕ	ϕV_{cby} (lb)
227.50	648.00	1.000	1.000	1.434	23292	0.75	17593

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	225.00	225.00	1.000	1.000	1.000	12021	0.70	16829

11. Results

Interaction of Tensile and Shear Forces (Sec. 17.8)

Shear	Factored Load, V_{ua} (lb)	Design Strength, $øV_n$ (lb)	Ratio	Status
Steel	8102	8853	0.92	Pass
T Concrete breakout x+	**8080**	**8120**	**1.00**	**Pass (Governs)**
|| Concrete breakout x+	**592**	**16240**	**0.04**	**Pass (Governs)**
|| Concrete breakout y-	**8080**	**17593**	**0.46**	**Pass (Governs)**
Concrete breakout, combined	-	-	0.46	Pass
Pryout	8102	16829	0.48	Pass

3/4"Ø SS Strong-Bolt 2, hnom:5.75" (146mm) meets the selected design criteria.

12. Warnings

- Per designer input, the tensile component of the strength-level earthquake force applied to anchors does not exceed 20 percent of the total factored anchor tensile force associated with the same load combination. Therefore the ductility requirements of ACI 318 17.10.5.2 for tension need not 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.

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

ROOF STRAPPING/HOLD DOWNS AT DIAGONAL WALL

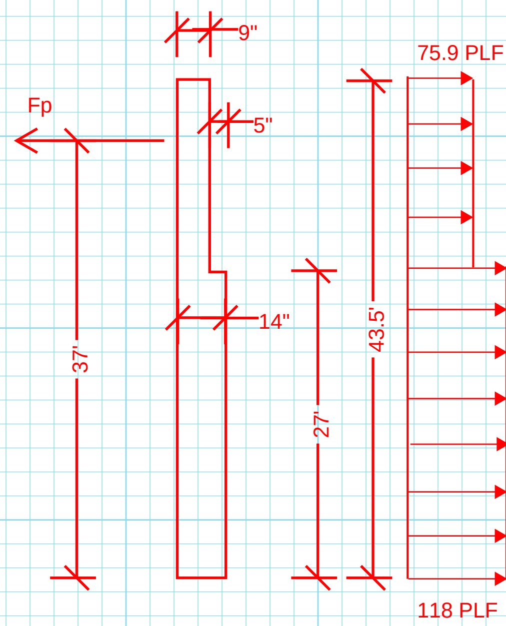
Fp = (0.4)(Sds)(ka)(Ie)(Wp)

@ 9" = (9/12)(.15 KCF) = 112.5 PSF
@ 14" = (14/12)(.15 KCF) = 175 PSF

Fp = (0.4)(0.843)(2)(1)(112.5 PSF) = 75.9 PSF @ 9"

Fp = 118 PSF @ 14"

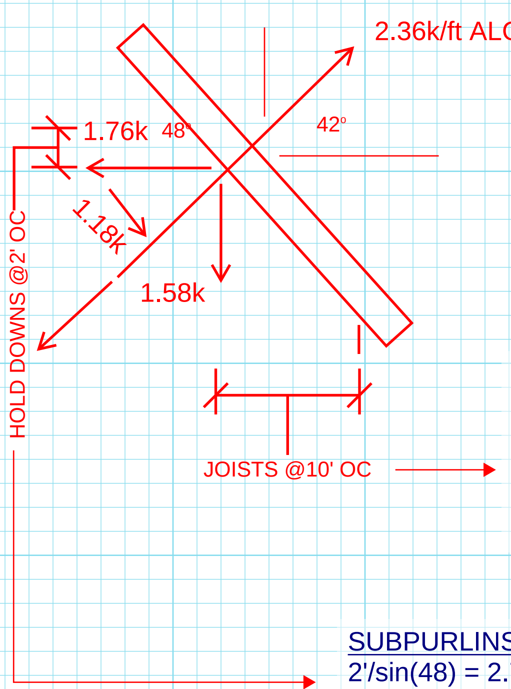
(MIN Fp DOES NOT GOVERN)



Fp = [(75.9 PLF)(16.5')(16.5'/2 + 27') + (118 PLF)(27')(27'/2)]/37'

Fp = [44415 LB-FT/FT + 43011 LB-FT/FT]/37'

Fp = 2.36 KLF



sin(42)(2.36k) = 1.58k
cos(42)(2.36k) = 1.76k

cos(48)(1.76k) = 1.18k

JOISTS TO TAKE N/S COMPONENT ONLY:
10'/cos(48) = 14.94' TRIB LENGTH

(14.94')(1.58k/ft) = 23.6k

(0.7)(23.6k) = 16.6k ASD ANCHORAGE AT SKEWED WALL JOISTS

SUBPURLINS TO TAKE E/W COMPONENT ONLY:
2'/sin(48) = 2.7' TRIB LENGTH

(2.7')(1.76k/ft) = 4.75k
(2.7')(1.18k/ft) = 3.19k

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: Bonded anchor
Material: F1554 Grade 55
Diameter (inch): 0.875
Effective Embedment depth, h_{ef} (inch): 6.000
Code report: ICC-ES ESR-4057
Anchor category: -
Anchor ductility: Yes
h_{min} (inch): 8.00
c_{ac} (inch): 13.08
C_{min} (inch): 1.75
S_{min} (inch): 3.00

Base Material
Concrete: Normal-weight
Concrete thickness, h (inch): 9.00
State: Cracked
Compressive strength, f'_c (psi): 4000
$\Psi_{c,V}$: 1.0
Reinforcement condition: Supplementary reinforcement not present
Supplemental edge reinforcement: Not applicable
Reinforcement provided at corners: No
Ignore concrete breakout in tension: No
Ignore concrete breakout in shear: No
Hole condition: Dry concrete
Inspection: Continuous
Temperature range, Short/Long: 150/110°F
Reduced installation torque (for AT-3G): Not applicable
Ignore 6do requirement: Not applicable
Build-up grout pad: No

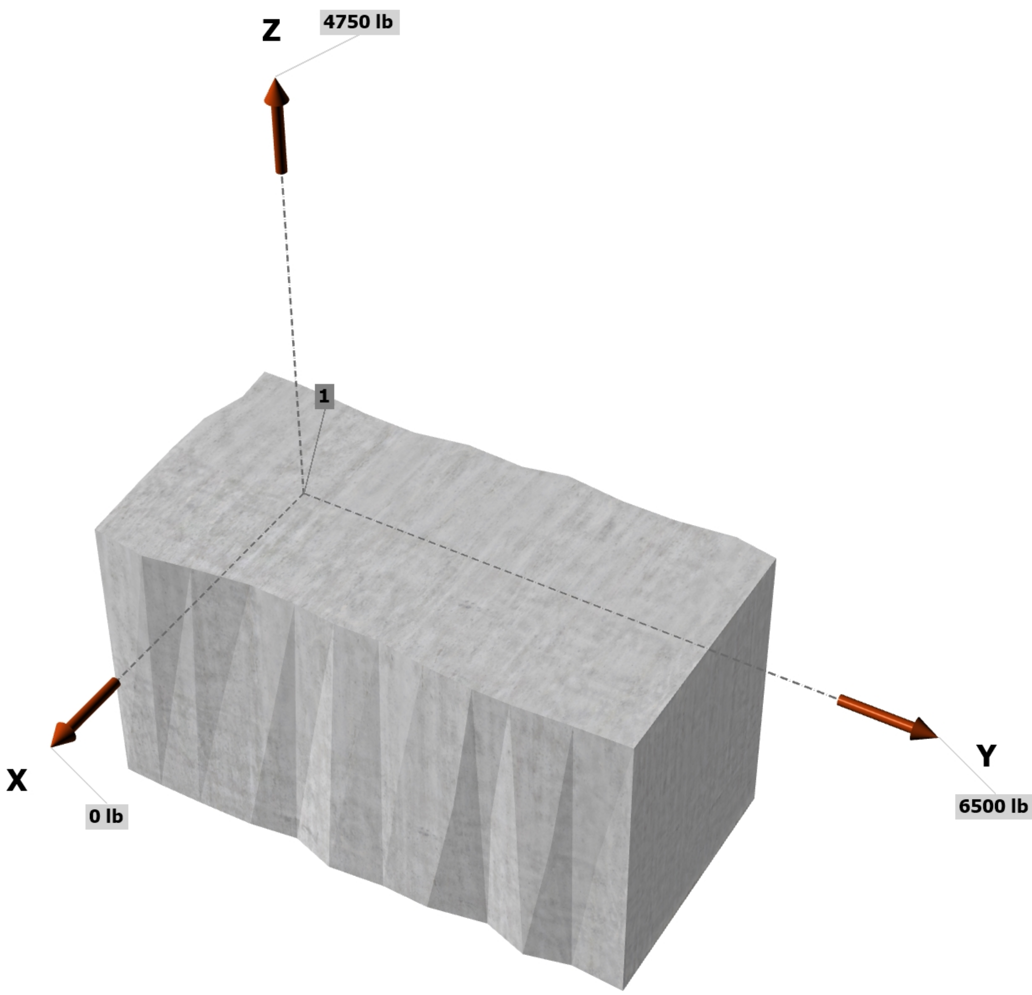
Recommended Anchor
Anchor Name: SET-3G™ - SET-3G w/ 7/8"Ø F1554 Gr. 55
Code Report: ICC-ES ESR-4057

Load and Geometry
Load factor source: ACI 318 Section 5.3
Load combination: not set
Seismic design: Yes
Anchors subjected to sustained tension: No
Ductility section for tension: 17.10.5.3 (d) is satisfied
Ductility section for shear: 17.10.6.2 not applicable
Ω_0 factor: not set
Apply entire shear load at front row: No
Anchors only resisting wind and/or seismic loads: Yes

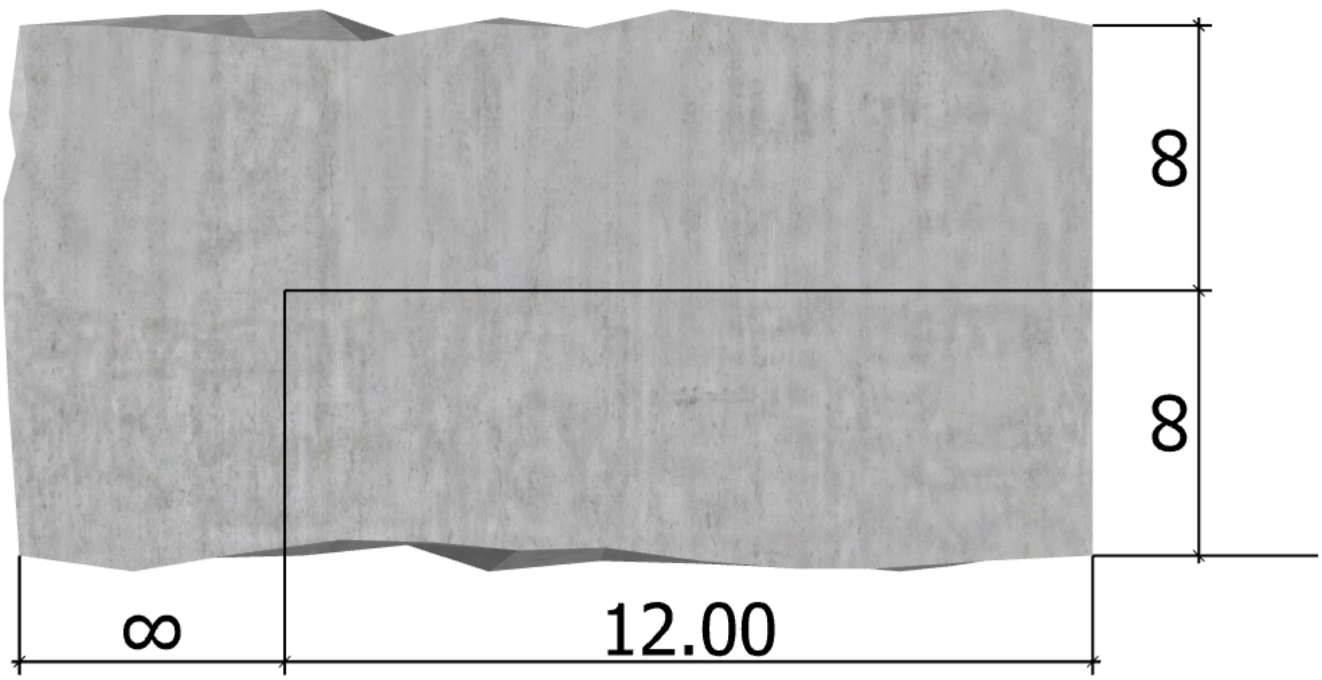
Strength level loads:

N_{ua} [lb]: 4750
V_{uax} [lb]: 0
V_{uay} [lb]: 6500

<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	4750.0	0.0	6500.0	6500.0
Sum	4750.0	0.0	6500.0	6500.0

Maximum concrete compression strain (‰): 0.00
Maximum concrete compression stress (psi): 0
Resultant tension force (lb): 4750
Resultant compression force (lb): 0
Eccentricity of resultant tension forces in x-axis, e'_{Nx} (inch): 0.00
Eccentricity of resultant tension forces in y-axis, e'_{Ny} (inch): 0.00
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

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

N_{sa} (lb)	ϕ	ϕN_{sa} (lb)
34650	0.75	25988

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

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

k_c	λ_a	f'_c (psi)	h_{ef} (in)	N_b (lb)
17.0	1.00	4000	6.000	15802

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

A_{Nc} (in^2)	A_{Nco} (in^2	$c_{a,min}$ (in)	$\Psi_{ed,N}$	$\Psi_{c,N}$	$\Psi_{cp,N}$	N_b (lb)	ϕ	$0.75\phi N_{cb}$ (lb)
324.00	324.00	12.00	1.000	1.00	1.000	15802	0.65	7703

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

$\tau_{k,cr} = \tau_{k,cr} f_{short\text{-}term} K_{sat} (f'_c / 2{,}500)^n \alpha_{N.seis}$

$\tau_{k,cr}$ (psi)	$f_{short\text{-}term}$	K_{sat}	$\alpha_{N.seis}$	f'_c (psi)	n	$\tau_{k,cr}$ (psi)
1265	1.00	1.00	1.00	4000	0.24	1416

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

λ_a	τ_{cr} (psi)	d_a (in)	h_{ef} (in)	N_{ba} (lb)
1.00	1416	0.88	6.000	23355

$0.75\phi N_a = 0.75\phi\,(A_{Na} / A_{Na0})\,\Psi_{ed,Na}\Psi_{cp,Na}N_{ba}$ (Sec. 17.5.1.2 & Eq. 17.6.5.1a)

A_{Na} (in^2)	A_{Na0} (in^2)	c_{Na} (in)	$c_{a,min}$ (in)	$\Psi_{ed,Na}$	$\Psi_{p,Na}$	N_{a0} (lb)	ϕ	$0.75\phi N_a$ (lb)
547.63	547.63	11.70	12.00	1.000	1.000	23355	0.65	11386

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

V_{sa} (lb)	ϕ_{grout}	ϕ	$\alpha_{V,seis}$	$\phi_{grout}\alpha_{V,seis}\phi V_{sa}$ (lb)
20790	1.0	0.65	0.75	10135

9. Concrete Breakout Strength of Anchor in Shear (Sec. 17.7.2)

Shear perpendicular to edge in y-direction:

$V_{by} = \min|7(l_e/d_a)^{0.2}\sqrt{d_a}\lambda_a\sqrt{f'_c}c_{a1}^{1.5}; 9\lambda_a\sqrt{f'_c}c_{a1}^{1.5}|$ (Eq. 17.7.2.2.1a & Eq. 17.7.2.2.1b)

l_e (in)	d_a (in)	λ_a	f'_c (psi)	c_{a1} (in)	V_{by} (lb)
6.00	0.875	1.00	4000	12.00	23662

$\phi V_{cby} = \phi\,(A_{Vc}/A_{Vco})\Psi_{ed,V}\Psi_{c,V}\Psi_{h,V}V_{by}$ (Sec. 17.5.1.2 & Eq. 17.7.2.1a)

A_{Vc} (in^2)	A_{Vco} (in^2)	$\Psi_{ed,V}$	$\Psi_{c,V}$	$\Psi_{h,V}$	V_{by} (lb)	ϕ	ϕV_{cby} (lb)
324.00	648.00	1.000	1.000	1.414	23662	0.70	11712

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

$\phi V_{cp} = \phi\min|k_{cp}N_a\,;\,k_{cp}N_{cb}| = \phi\min|k_{cp}(A_{Na}/A_{Na0})\Psi_{ed,Na}\Psi_{cp,Na}N_{ba}\,;\,k_{cp}(A_{Nc}/A_{Nco})\Psi_{ed,N}\Psi_{c,N}\Psi_{cp,N}N_b|$ (Sec. 17.5.1.2 & Eq. 17.7.3.1a)

k_{cp}	A_{Na} (in^2)	A_{Na0} (in^2)	$\Psi_{ed,Na}$	$\Psi_{cp,Na}$	N_{ba} (lb)	N_a (lb)
2.0	547.63	547.63	1.000	1.000	23355	23355

A_{Nc} (in^2)	A_{Nco} (in^2)	$\Psi_{ed,N}$	$\Psi_{c,N}$	$\Psi_{cp,N}$	N_b (lb)	N_{cb} (lb)	ϕ	ϕV_{cp} (lb)
324.00	324.00	1.000	1.000	1.000	15802	15802	0.70	22122

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	4750	25988	0.18	Pass
Concrete breakout	**4750**	**7703**	**0.62**	**Pass (Governs)**
Adhesive	4750	11386	0.42	Pass

Shear	Factored Load, V_{ua} (lb)	Design Strength, $øV_n$ (lb)	Ratio	Status
Steel	**6500**	**10135**	**0.64**	**Pass (Governs)**
T Concrete breakout y+	6500	11712	0.55	Pass
Pryout	6500	22122	0.29	Pass

Interaction check	$(N_{ua}/\phi N_{ua})^{5/3}$	$(V_{ua}/\phi V_{ua})^{5/3}$	Combined Ratio	Permissible	Status
Sec. R17.8	0.45	0.48	92.4%	1.0	Pass

SET-3G w/ 7/8"Ø F1554 Gr. 55 with hef = 6.000 inch meets the selected design criteria.

12. Warnings

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

- Per designer input, the shear component of the strength-level earthquake force applied to anchors does not exceed 20 percent of the total factored anchor shear force associated with the same load combination. Therefore the ductility requirements of ACI 318 17.10.6.2 for shear need not be satisfied – designer to verify.

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

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

4.75k (LRFD)

(4.75k)(1.4)(0.7) = 4.7k ASD W/ 1.4 FACTOR PER ASCE 7-16 12.11.2.2.2

Simpson Strong-Tie® Wood Construction Connectors

CS/CMST/CMSTC/CSHP

Coiled Straps (cont.)

Model No.	Total L	Ga.	DF/SP Fasteners (in.)	DF/SP End Length (in.)	SPF/HF Fasteners (in.)	SPF/HF End Length (in.)	Allowable Tension Loads (160)	Code Ref.
CMST12	40'	12	(74) 0.162 x 2½	33	(84) 0.162 x 2½	38	9,215	
			(86) 0.148 x 2½	39	(98) 0.148 x 2½	44	9,215	

1. See pp. 266–267 for Straps and Ties General Notes.
2. Calculate the connector value for a reduced number of nails as follows:

$$\text{Allowable Load} = \frac{\text{No. of Nails Used}}{\text{No. of Nails in Table}} \times \text{Table Load}$$

Example: CMSTC16 in DF/SP with 40 nails total.
(Half of the nails in each member being connected)

$$\text{Allowable Load} = \frac{\text{40 Nails (Used)}}{\text{50 Nails (Table)}} \times 4{,}690 \text{ lb.} = 3{,}752 \text{ lb.}$$

3. See p. 274 for alternate nailing and lap splice information.
4. **Fasteners:** Nail dimensions are listed diameter by length. See pp. 21–22 for fastener information.

x/86 NAILS = (4700 LB)/(9215 LB)

TABLE CONSIDERS NAILS ON BOTH SIDES, SO USE 44 MIN NAILS TO TRANSFER FORCE ON ONE SIDE

USE 25 10D NAILS ONE SIDE TO DEVELOP 4.7k ASD

Straps and Ties General Notes

These general notes are provided to ensure proper installation of Simpson Strong-Tie straps and ties.

a. The (160) loads have been increased for wind or earthquake loading, with no further increase allowed. Reduce where other loads govern.
b. When installing strap over 5/8" maximum wood structural panel sheathing, use 2 1/2"-long nails minimum.
c. SD screws are Simpson Strong-Tie® Strong-Drive® SD Connector screws. See **Fastener Types and Sizes Specified for Simpson Strong-Tie Connectors**.
d. For straight straps in tension, use half of the fasteners in each member being connected to achieve the listed loads.
e. Tension loads apply for uplift when installed vertically.
f. Field-bending straps is not recommended unless otherwise noted.
g. If wood splitting is a concern, consider spacing the nails at every other location.
h. The cut length of coil strap shall be equal to twice the "end length" noted in the tables plus the clear-span dimension.
i. Straps 16 ga. and heavier can be fillet welded to structural steel members. The designer shall specify the weld size and length. Welding and specification shall be in compliance with the current American Welding Society ANSI/AWS D1.3, Structural Welding Code – Sheet Steel.

DETERMINE WELD TO ANGLE

4.75k = (1.391)(3)(L)

L = 1.14".... WELD ENTIRE STRAP WIDTH 3/16

DISTRIBUTE Fp OVER MULTIPLE JOISTS FROM PANEL

Fp = 5.2k @ WALL @ 2' OC

60'/2.5 = 24' MIN SUB DIAPHRAGM... USE 30'

FOR 100% LOAD, USE CMST 14 COILED STRAPS

(5.2k/6.475k)(66 NAILS) = 54 NAILS TOTAL OVER 1ST AND 2ND JOISTS

CMST14	52½'	14	(56) 0.162 x 2½	26	(66) 0.162 x 2½	30	6,475
			(66) 0.148 x 2½	30	(76) 0.148 x 2½	34	6,475

(5.2k)(2)(0.4) = 4.2k @ 4' OC

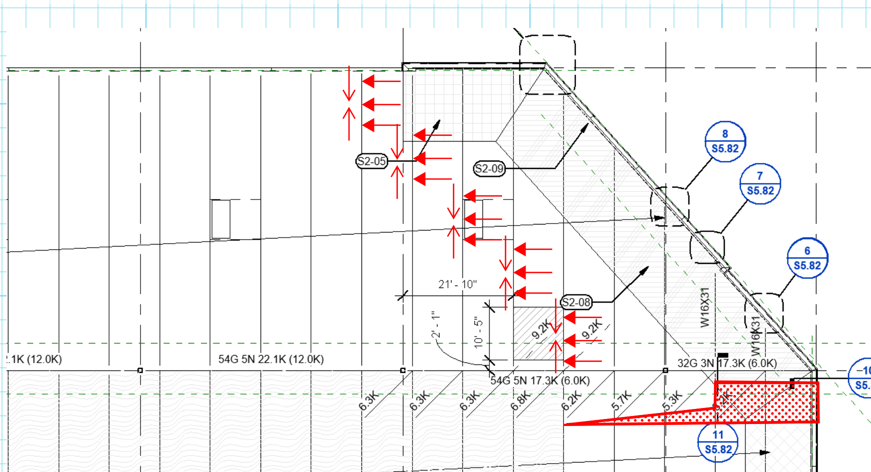
USE MSTI48 W/ (48) NAILS @ 4'-0" OVER 3RD JOIST

MSTI48	12	2 1/16	48	(48) 0.148 x 1½	5,070	4,390

(5.2k)(4)(0.2) = 4.2k @ 8' OC

USE MSTI48 W/ (48) NAILS @ 8'-0" OVER 4TH JOIST

SUBDIAPHRAGM CHORD COMPRESSION INTO JOISTS
TO BE DISTRIBUTED ALONG SKEWED WALL



JOIST CONNECTION TO SKEWED WALL

Fp = 23.6k LRFD PER PREVIOUS CALCS

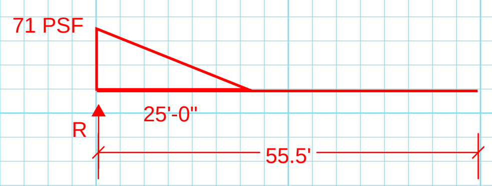
GRAVITY LOADING:

(15 PSF)(10')(60'/2) = 4.5k

(20 PSF)(10')(60'/2) = 6k ROOF LIVE LOAD

(14 PSF)(10')(60'/2) = 4.2k BALANCED SNOW LOAD

DRIFT:



R = (0.5)(71 PSF)(25')(55.5'-(25'/3))/55.5' = 755 PLF

(755 PLF)(10') = 7.6k DRIFT REACTION

GOVERNING SEISMIC COMBINATION FOR GRAVITY COMPONENT:

1.2DL + 0.2SL = (1.2 + (0.2)(0.843))(4.5k) + (0.2)(7.6k + 4.2k) = 8.6k
1.2DL + 1.0LL = (1.2 + (0.2)(0.843))(4.5k) + (1.0)(6k) = 12.2k (GOVERNS)

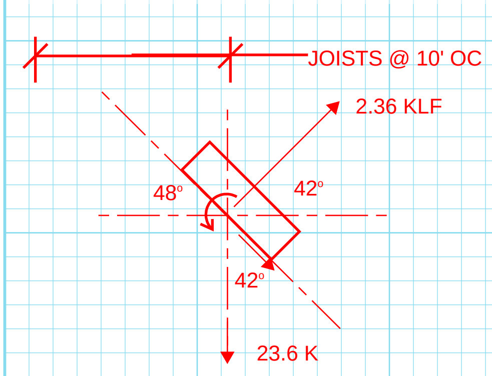
e = 8"

Mu = (12.2k)(8") = 97.3 K-IN

BREAK UP INTO COMPONENTS

JOIST-WALL ANCHORAGE FORCE

(2.36 KLF)(sin(42))[10'/cos(48)] = 23.6k (0.7)(23.6k) = 16.6 ASD AXIAL



(cos(42))(23.6k) = 17.7k SHEAR ALONG PLANE OF PLATE

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: AWS Type A
Diameter (inch): 0.750
Effective Embedment depth, h_{ef} (inch): 7.000
Anchor category: -
Anchor ductility: Yes
h_{min} (inch): 8.50
C_{min} (inch): 1.38
S_{min} (inch): 3.00

Base Material
Concrete: Normal-weight
Concrete thickness, h (inch): 9.00
State: Cracked
Compressive strength, f'_c (psi): 4000
$\Psi_{c,V}$: 1.0
Reinforcement condition: Supplementary reinforcement not present
Supplemental edge reinforcement: Not applicable
Reinforcement provided at corners: No
Ignore concrete breakout in tension: No
Ignore concrete breakout in shear: Yes
Ignore 6do requirement: No
Build-up grout pad: No

Base Plate
Length x Width x Thickness (inch): 16.00 x 16.00 x 0.25

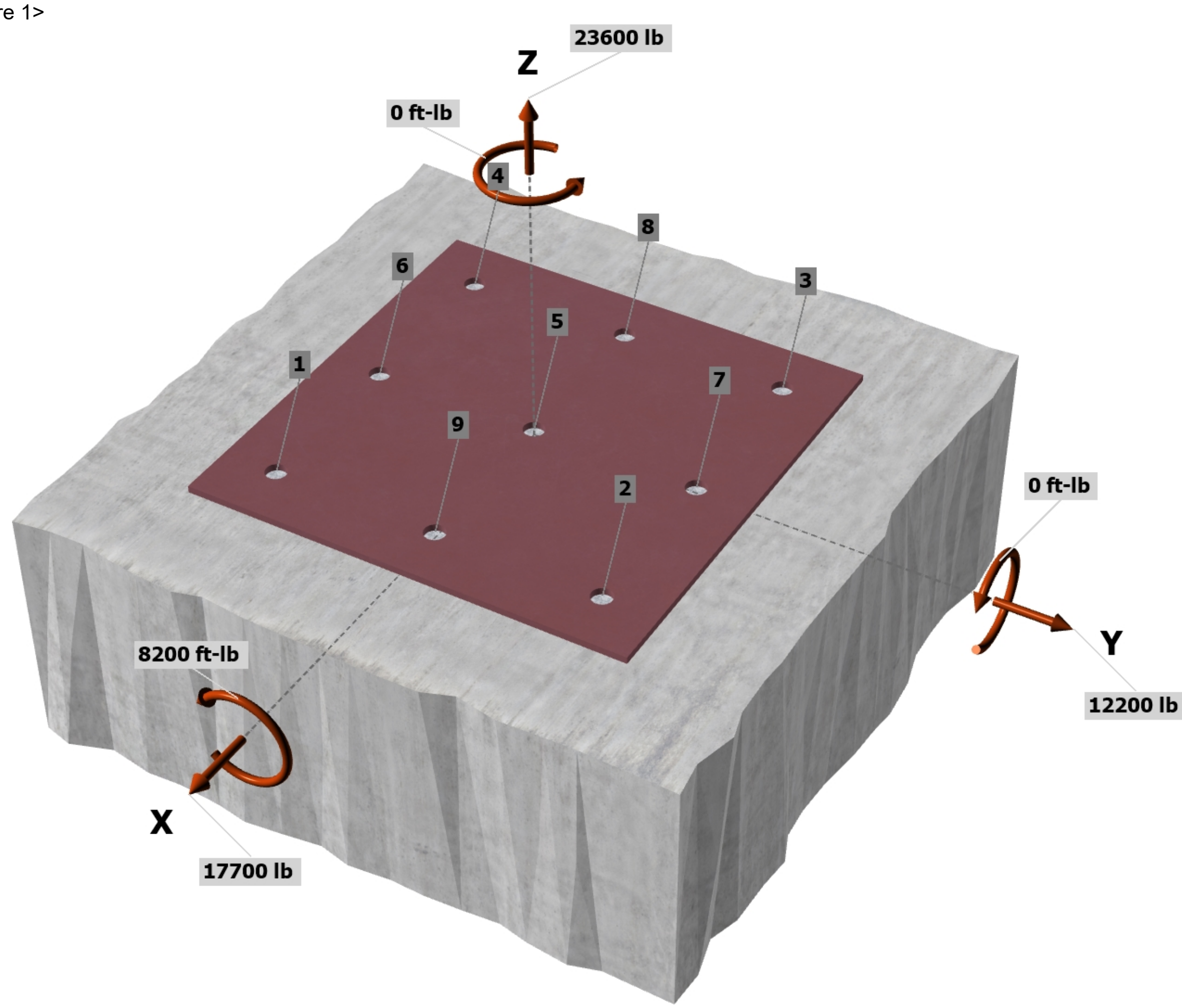
Recommended Anchor
Anchor Name: Headed Stud - 3/4"Ø AWS Type A Headed Stud

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 (c) is satisfied
Ductility section for shear: 17.10.6.2 not applicable
Ω_0 factor: not set
Apply entire shear load at front row: No
Anchors only resisting wind and/or seismic loads: No

Strength level loads:

N_{ua} [lb]: 23600
V_{uax} [lb]: 17700
V_{uay} [lb]: 12200
M_{ux} [ft-lb]: 8200
M_{uy} [ft-lb]: 0
M_{uz} [ft-lb]: 0

<Figure 1>



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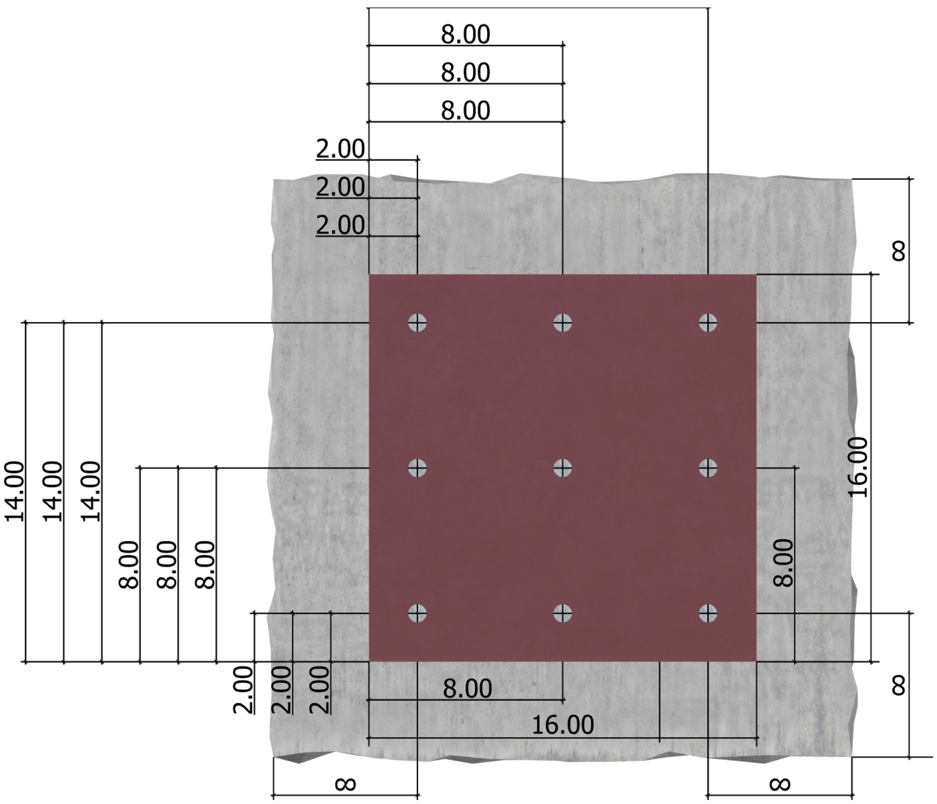
8.00
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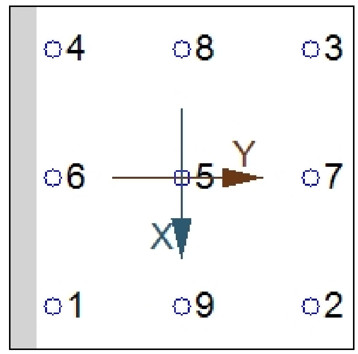
3. Resulting Anchor Forces

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	322.4	1966.7	1355.6	2388.6
2	5221.0	1966.7	1355.6	2388.6
3	5221.0	1966.7	1355.6	2388.6
4	322.4	1966.7	1355.6	2388.6
5	2771.7	1966.7	1355.6	2388.6
6	322.4	1966.7	1355.6	2388.6
7	5221.0	1966.7	1355.6	2388.6
8	2771.7	1966.7	1355.6	2388.6
9	2771.7	1966.7	1355.6	2388.6
Sum	24945.3	17700.0	12200.0	21497.2

Maximum concrete compression strain (‰): 0.03
Maximum concrete compression stress (psi): 139
Resultant tension force (lb): 24945
Resultant compression force (lb): 1345
Eccentricity of resultant tension forces in x-axis, e'_{Nx} (inch): 3.53
Eccentricity of resultant tension forces in y-axis, e'_{Ny} (inch): 0.00
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)
26950	0.75	20213

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

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

k_c	λ_a	f'_c (psi)	h_{ef} (in)	N_b (lb)
24.0	1.00	4000	7.000	28112

$0.75\phi N_{cbg} = 0.75\phi\,(A_{Nc} / A_{Nco})\,\Psi_{ec,N}\,\Psi_{ed,N}\,\Psi_{c,N}\,\Psi_{cp,N} N_b$ (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)
1089.00	441.00	-	0.748	1.000	1.00	1.000	28112	0.70	27266

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

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

$\Psi_{c,P}$	A_{brg} (in^2)	f'_c (psi)	ϕ	$0.75\phi N_{pn}$ (lb)
1.0	0.79	4000	0.70	13188

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

V_{sa} (lb)	ϕ_{grout}	ϕ	$\phi_{grout}\phi V_{sa}$ (lb)
26950	1.0	0.65	17518

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

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

k_{cp}	A_{Nc} (in^2)	A_{Nco} (in^2)	$\Psi_{ec,N}$	$\Psi_{ed,N}$	$\Psi_{c,N}$	$\Psi_{cp,N}$	N_b (lb)	ϕ	ϕV_{cpg} (lb)
2.0	1089.00	441.00	1.000	1.000	1.000	1.000	28112	0.70	97186

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	5221	20213	0.26	Pass
Concrete breakout	**24945**	**27266**	**0.91**	**Pass (Governs)**
Pullout	5221	13188	0.40	Pass

Shear	Factored Load, V_{ua} (lb)	Design Strength, øV_n (lb)	Ratio	Status
Steel	2389	17518	0.14	Pass
Pryout	**21497**	**97186**	**0.22**	**Pass (Governs)**

Interaction check	$(N_{ua}/\phi N_{ua})^{5/3}$	$(V_{ua}/\phi V_{ua})^{5/3}$	Combined Ratio	Permissible	Status
Sec. R17.8	0.86	0.08	94.3%	1.0	Pass

3/4"Ø AWS Type A Headed Stud with hef = 7.000 inch meets the selected design criteria.

12. Warnings

- Concrete breakout strength in shear has not been evaluated against applied shear load(s) per designer option. Refer to ACI 318 Section 17.5.2.1 for conditions where calculations of the concrete breakout strength may not be required.

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

- Per designer input, the shear component of the strength-level earthquake force applied to anchors does not exceed 20 percent of the total factored anchor shear force associated with the same load combination. Therefore the ductility requirements of ACI 318 17.10.6.2 for shear need not be satisfied – designer to verify.

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

MAXIMUM GRAVITY LOAD INTO PLATE:

(1.2)(4.5k) + (1.6)(11.8k) = 24.3k

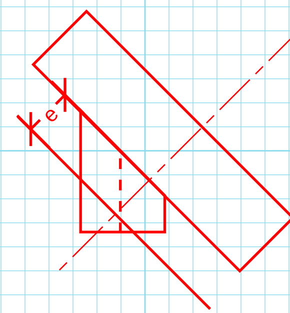
e = sin(42)(8") = 5.35"

Mu = (24.3k)(5.35") = 130 K-IN

Sx_{REQ} = (130 K-IN)/(0.9)(50 KSI) = 2.89 IN^3

WT5x24.5 Sx = 2.39 IN3, NG

USE WT5x50, Sx = 5.56 IN^3 > 2.89 IN3



CHECK WELD TO WT FLANGE

130 K-IN/(0.711 IN) = 183k

(1.392)(5)(16) = 111.3k < 206k

USE WELD TOP AND BOTTOM OF WT FLANGE

(1.392)(5)(16 + 16 - (2)(1.62)) = 200k

WELD TO WT STEM:

(1.392)(4)(5.5" - 1.62")(2) = 43.2k, OK

JOIST CONNECTION TO SKEWED WALL

Fp = 23.6k LRFD PER PREVIOUS CALCS

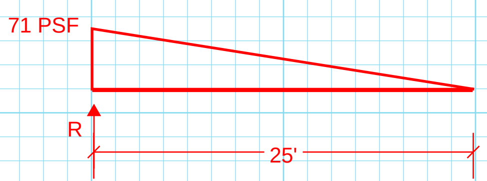
GRAVITY LOADING:

(15 PSF)(10')(25'/2) = 1.9k

(20 PSF)(10')(25'/2) = 2.5k ROOF LIVE LOAD

(14 PSF)(10')(25'/2) = 1.8k BALANCED SNOW LOAD

DRIFT:



R = (0.5)(71 PSF)(25')(25'-(25'/3))/25 = 592 PLF

(592 PLF)(10') = 6k DRIFT REACTION

GOVERNING SEISMIC COMBINATION FOR GRAVITY COMPONENT:

1.2DL + 0.2SL = (1.2 + (0.2)(0.843))(1.9k) + (0.2)(1.8k + 6k) = 4.2k (GOVERNS)
1.2DL + 1.0LL = (1.2 + (0.2)(0.843))(1.9k) + (1.0)(1.2k) = 3.8k

SEE DRAGSTRUT MATHCAD

Vu = 4.2k (SEISMIC CASE)

CHECK COMBINED SHEAR AND AXIAL ON WELD

SAY 12" WEB PLATE

4.2k/12" = 0.35 K/IN SHEAR

23.6k/12" = 2 K/IN AXIAL

sqrt((0.35 K/IN)2 + (2 K/IN)2) = 2.03 K/IN DEMAND

(1.392)(5)(1") = 6.96 K/IN > 2.03 K/IN, OK

CHECK AGAINST 1.2DL + 1.6 SL

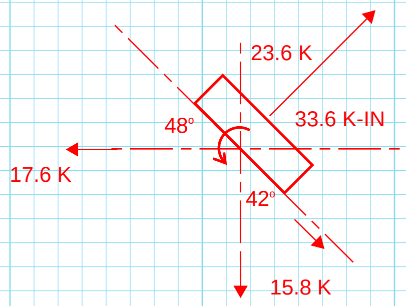
(1.2 + (0.2)(0.843))(1.9k) + (1.6)(7.8k) = 15.1k

15.1k/12" = 1.25 K/IN, OK

CHECK EMBED PLATE

M = (4.2k)(8") = 33.6 K-IN

cos(42)(15.8k) = 11.8k



WELD PER TABLE 8-8

e = 9"

Pu = (1.2)(1.9k) + (1.6)(7.8k) = 14.8k

Mu = (14.8k)(8") = 118.8 K-IN

l = 10"

ex = 9"

a = 0.9; k = 0.3

C = 1.34

Dmin = (14.8k)/((0.75)(1.34)(1.0)(10")) = 1.47

USE 5/16" ON 3 SIDES

5 - 1.34 = 3.66

(1.392)(5)(3.66)(2) = 50.9k > 23.6k FOR Fp WALL ANCHORAGE, OK

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By

Date

Job#

Sht. of

Company:		Date:	6/13/2023
Engineer:	ATT	Page:	1/5
Project:	Fortress Puyallup		
Address:			
Phone:			
E-mail:			

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: AWS Type A
Diameter (inch): 0.750
Effective Embedment depth, h_{ef} (inch): 6.500
Anchor category: -
Anchor ductility: Yes
h_{min} (inch): 8.00
C_{min} (inch): 1.38
S_{min} (inch): 3.00

Base Material
Concrete: Normal-weight
Concrete thickness, h (inch): 9.00
State: Cracked
Compressive strength, f'_c (psi): 4000
$\Psi_{c,V}$: 1.0
Reinforcement condition: Supplementary reinforcement not present
Supplemental edge reinforcement: Not applicable
Reinforcement provided at corners: No
Ignore concrete breakout in tension: No
Ignore concrete breakout in shear: Yes
Ignore 6do requirement: No
Build-up grout pad: No

Base Plate
Length x Width x Thickness (inch): 16.00 x 16.00 x 0.25

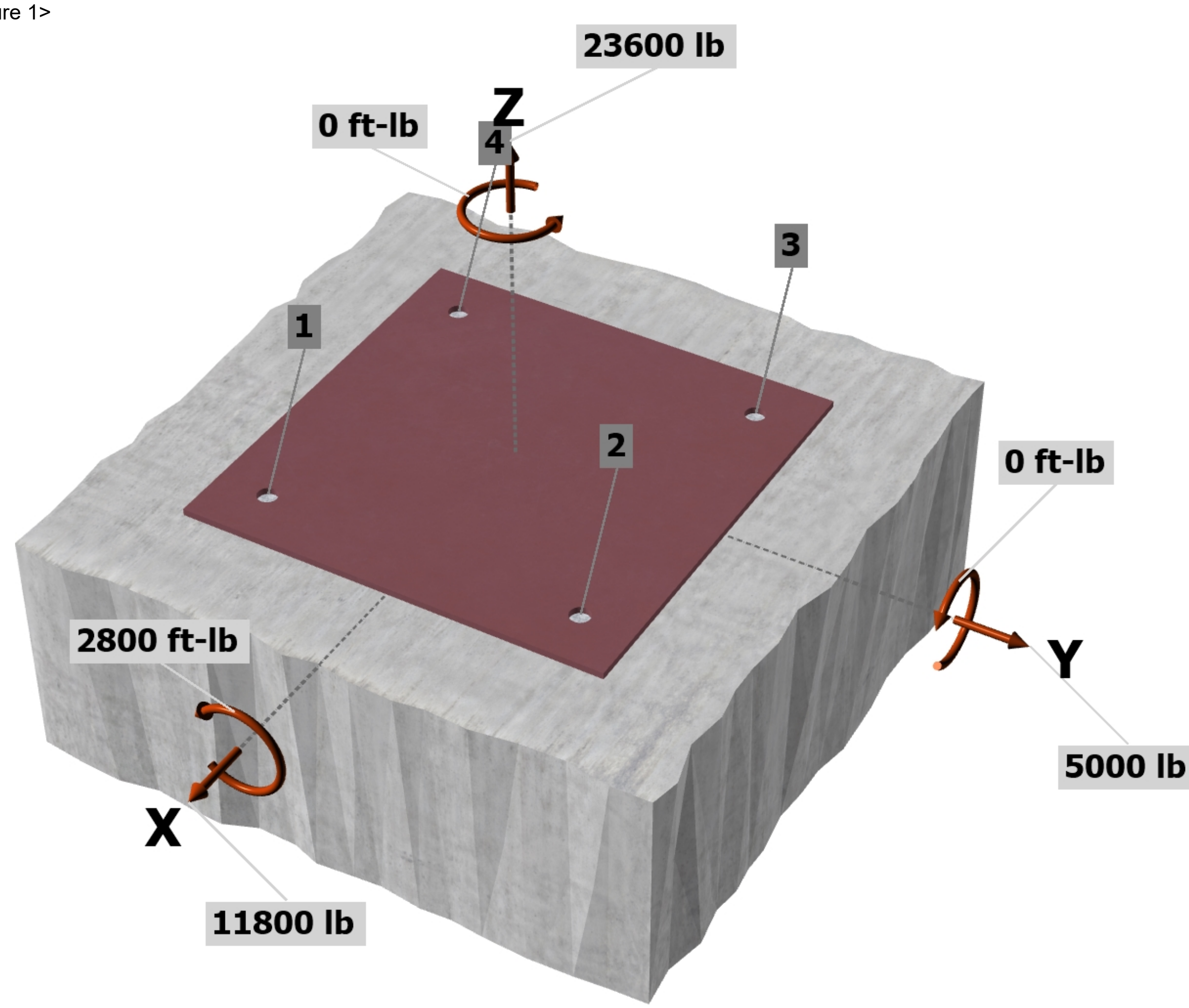
Recommended Anchor
Anchor Name: Headed Stud - 3/4"Ø AWS Type A Headed Stud

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 (c) is satisfied
Ductility section for shear: 17.10.6.2 not applicable
Ω_0 factor: not set
Apply entire shear load at front row: No
Anchors only resisting wind and/or seismic loads: No

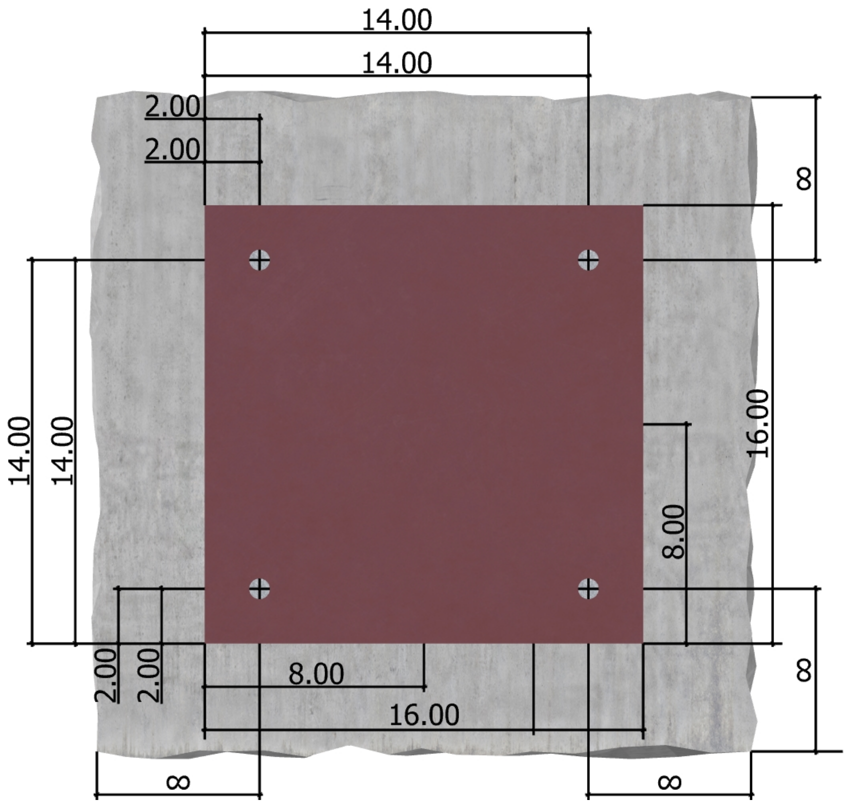
Strength level loads:

N_{ua} [lb]: 23600
V_{uax} [lb]: 11800
V_{uay} [lb]: 5000
M_{ux} [ft-lb]: 2800
M_{uy} [ft-lb]: 0
M_{uz} [ft-lb]: 0

<Figure 1>



<Figure 2>

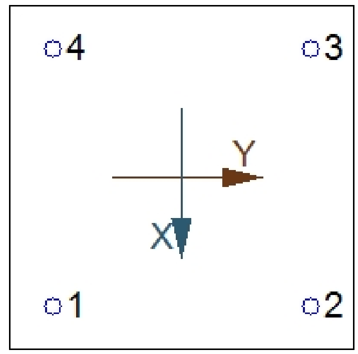


3. Resulting Anchor Forces

Anchor	Tension load, N_{ua} (lb)	Shear load x, V_{uax} (lb)	Shear load y, V_{uay} (lb)	Shear load combined, $\sqrt{(V_{uax})^2+(V_{uay})^2}$ (lb)
1	4500.3	2950.0	1250.0	3203.9
2	7299.7	2950.0	1250.0	3203.9
3	7299.7	2950.0	1250.0	3203.9
4	4500.3	2950.0	1250.0	3203.9
Sum	23600.0	11800.0	5000.0	12815.6

Maximum concrete compression strain (‰): 0.00
Maximum concrete compression stress (psi): 0
Resultant tension force (lb): 23600
Resultant compression force (lb): 0
Eccentricity of resultant tension forces in x-axis, e'_{Nx} (inch): 1.42
Eccentricity of resultant tension forces in y-axis, e'_{Ny} (inch): 0.00
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)
26950	0.75	20213

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

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

k_c	λ_a	f'_c (psi)	h_{ef} (in)	N_b (lb)
24.0	1.00	4000	6.500	25154

$0.75\phi N_{cbg} = 0.75\phi\,(A_{Nc}/A_{Nco})\,\Psi_{ec,N}\Psi_{ed,N}\Psi_{c,N}\Psi_{cp,N}N_b$ (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)
992.25	380.25	-	0.873	1.000	1.00	1.000	25154	0.70	30070

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)
1.0	0.79	4000	0.70	13188

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

V_{sa} (lb)	ϕ_{grout}	ϕ	$\phi_{grout}\phi V_{sa}$ (lb)
26950	1.0	0.65	17518

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

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

k_{cp}	A_{Nc} (in^2)	A_{Nco} (in^2)	$\Psi_{ec,N}$	$\Psi_{ed,N}$	$\Psi_{c,N}$	$\Psi_{cp,N}$	N_b (lb)	ϕ	ϕV_{cpg} (lb)
2.0	992.25	380.25	1.000	1.000	1.000	1.000	25154	0.70	91895

11. Results

Interaction of Tensile and Shear Forces (Sec. 17.8)

Tension	Factored Load, N_{ua} (lb)	Design Strength, øN_n (lb)	Ratio	Status
Steel	7300	20213	0.36	Pass
Concrete breakout	**23600**	**30070**	**0.78**	**Pass (Governs)**
Pullout	7300	13188	0.55	Pass

Shear	Factored Load, V_{ua} (lb)	Design Strength, øV_n (lb)	Ratio	Status
Steel	**3204**	**17518**	**0.18**	**Pass (Governs)**
Pryout	12816	91895	0.14	Pass

Interaction check	$N_{ua}/\phi N_n$	$V_{ua}/\phi V_n$	Combined Ratio	Permissible	Status
Sec. 17.8.1	0.78	0.00	78.5%	1.0	Pass

3/4"Ø AWS Type A Headed Stud with hef = 6.500 inch meets the selected design criteria.

12. Warnings

- Concrete breakout strength in shear has not been evaluated against applied shear load(s) per designer option. Refer to ACI 318 Section 17.5.2.1 for conditions where calculations of the concrete breakout strength may not be required.

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

- Per designer input, the shear component of the strength-level earthquake force applied to anchors does not exceed 20 percent of the total factored anchor shear force associated with the same load combination. Therefore the ductility requirements of ACI 318 17.10.6.2 for shear need not be satisfied – designer to verify.

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

Dragstrut Design

Purpose Statement:
Dragstrut W-Section Beam Design

Referenced Standards:
AISC 360-16
IBC 2018
ASCE 7-16

Element ID: W16

AISC section properties functions are adopted by reference:

Include << O:\Structural\4) Programs\STEEL\AISC Lookup.mcdx

Section Properties

$C := \text{"W16X31"}$	Trial Section
$F_y := 50 \cdot ksi$	Section Strength
$F_u := 58\ ksi$	Section Tensile Strength
$E := 29000 \cdot ksi$	Elastic Modulus
$G := 11200 \cdot ksi$	Shear Modulus of Elasticity of Steel
$K_x := 1.0 \quad K_y := 1.0$	

$A := A(C)$	$A = 9.13\ in^2$	$S_x := S_x(C)$	$S_x = 47.2\ in^3$	$Z_x := Z_x(C)$	$Z_x = 54\ in^3$
$t := t_f(C)$	$t = 0.44\ in$	$r_x := r_x(C)$	$r_x = 6.41\ in$	$Z_y := Z_y(C)$	$Z_y = 7.03\ in^3$
$b_f := b_f(C)$	$b_f = 5.53\ in$	$S_y := S_y(C)$	$S_y = 4.49\ in^3$	$J := J(C)$	$J = 0.46\ in^4$
$d := d(C)$	$d = 15.9\ in$	$r_y := r_y(C)$	$r_y = 1.17\ in$	$I_x := I_x(C)$	$I_x = 375\ in^4$
$t_w := t_w(C)$	$t_w = 0.28\ in$	$C_w := C_w(C)$	$C_w = 739\ in^6$	$I_y := I_y(C)$	$I_y = 12.4\ in^4$
$w_{self} := W(C)$	$w_{self} = 31\ plf$	$k_{des} := k_{des}(C)$	$T_{web} := d - 2\ k_{des} = 14.22\ in$		

$$b := \frac{b_f}{2 \cdot t} = 6.28$$

$$h := \frac{T_{web}}{t_w} = 51.69$$

$L_{xx} := 25 \cdot ft$	Beam Span
$L_{yy} := 1 \cdot ft$	Unbraced Length about Y axis (Compression flange, usually braced by deck)
$L_T := \dfrac{L_{xx}}{2} = 12.5\ ft$	Torsional Unbraced Length (Lxx or the distance b/t bottom flange kickers)

Dragstrut Design

Loading

$f := \text{Roof Member}$ — Member Location;

$f = 0.2$

"Roof Member" assumes Snow loading combinations
"Floor Member" assumes Live loading combinations

Gravity Loading

$DL := 15 \cdot psf$ — Dead Load

$LL := 20 \cdot psf$ — Live or Snow Load

$TW := 10\ ft$ — Beam Tributary Width

Seismic Loading

$S_{ds} := 0.843$ — Seismic Criteria

$\rho := 1.0$

$\Omega := 2$

$P := 25 \cdot k$ — Axial Tension or Compression (Ultimate Load)

$P_{\Omega} := P \cdot \dfrac{\Omega}{\rho} = 50\ k$ — Ultimate, Omegafied Axial Load

$P_u := P_{\Omega} = 50\ k$

$w_{self} = 31\ plf$ — Include Beam Self Weight

$w_{dl} := TW \cdot DL + w_{self} = 181\ plf$

$w_{ll} := TW \cdot LL = 200\ plf$

$l_x := L_{xx} = 25\ ft$ — Effective Unbraced Lengths

$l_y := L_{yy} = 1\ ft$

Gravity Load Checks

Load Combinations

$w_{u1} := 1.4 \cdot w_{dl} = 253.4\ plf$

$w_{u2} := 1.2 \cdot w_{dl} + 1.6 \cdot w_{ll} = 537.2\ plf$

$w_u := \max\left(w_{u1}, w_{u2}\right) = 537.2\ plf$

$V_u := \frac{1}{2} \cdot w_u \cdot L_{xx} = 6.72\ k$

$M_u := \frac{1}{8} \cdot w_u \cdot {L_{xx}}^2 = 41.97\ k \cdot ft$

$M_{u2} := \left(1.2 + \left(0.2 \cdot S_{ds}\right)\right) \frac{DL \cdot TW \cdot {L_{xx}}^2}{8} + 0.2 \cdot 28.4\ k \cdot ft = 21.72\ k \cdot ft$

Check against moment under drift loading, does not govern

FLEXURE CHECK

$L_b := L_{yy} = 1\ ft$

$L_p := 1.76 \cdot r_y \cdot \sqrt{\frac{E}{F_y}} = 4.13\ ft$

$r_{ts} := \sqrt{\frac{\sqrt{I_y \cdot C_w}}{S_x}} = 1.42\ in$

$h_0 := d - t = 15.46\ in$

$L_r := 1.95 \cdot r_{ts} \cdot \frac{E}{0.7 \cdot F_y} \cdot \sqrt{\frac{J}{S_x \cdot h_0} + \sqrt{\left(\frac{J}{S_x \cdot h_0}\right)^2 + 6.76 \cdot \left(0.7 \cdot \frac{F_y}{E}\right)^2}} = 11.87\ ft$

$M_p := F_y \cdot Z_x = 225\ k \cdot ft$ (F2-1)

$M_{n1} := M_p - \left(M_p - 0.7 \cdot F_y \cdot S_x\right) \cdot \left(\frac{L_b - L_p}{L_r - L_p}\right) = 260.36\ k \cdot ft$ (F2-2)

$F_{cr} := \frac{\pi^2 \cdot E}{\left(\frac{L_b}{r_{ts}}\right)^2} \cdot \sqrt{\left(1 + 0.078 \cdot \frac{J}{S_x \cdot h_0} \cdot \left(\frac{L_b}{r_{ts}}\right)^2\right)} = 4038.17\ ksi$ (F2-4)

$M_{n2} := F_{cr} \cdot S_x = 15883.47\ k \cdot ft$ (F2-3)

Dragstrut Design

$$M_n := \begin{Vmatrix} \text{if } L_b \le L_p \\ \quad \| M_p \\ \text{if } L_p < L_b \le L_r \\ \quad \| M_{n1} \\ \text{if } L_b > L_r \\ \quad \| M_{n2} \end{Vmatrix} = 225\ k \cdot ft$$

$\phi M_n := 0.9 \cdot M_n = 202.5\ k \cdot ft$

$DCR_1 := M_u \div \phi M_n = 0.21$

$Result := \text{if}\left(M_u \le \phi M_n, \text{“OK”}, \text{“NG”}\right)$ $Result = \text{“OK”}$

SHEAR CHECK

$\phi_v := \text{if}\left(h \le 2.24 \cdot \sqrt{\frac{E}{F_y}}, 1.0, 0.9\right) = 1$

$A_w := t_w \cdot d = 4.37\ in^2$

$C_v := 1.0$

$V_n := 0.6 \cdot F_y \cdot A_w \cdot C_v$

$\phi V_n := \phi_v \cdot V_n = 131.18\ k$

$V_u = 6.72\ k$

$DCR_2 := V_u \div \phi V_n = 0.05$

$Result := \text{if}\left(V_u \le \phi V_n, \text{“OK”}, \text{“NG”}\right)$ $Result = \text{“OK”}$

Deflection Check

$\Delta_{all\_total} := \frac{L_{xx}}{240} = 1.25\ in$ Allowable Deflections

$\Delta_{all\_live} := \frac{L_{xx}}{360} = 0.83\ in$

$\Delta_{total} := \frac{5 \cdot (w_{dl} + w_{ll}) \cdot L_{xx}^4}{384 \cdot E \cdot I_x} = 0.31\ in$

$$Result_2 := \text{if}\left(\Delta_{total} \le \Delta_{all\_total}, \text{"OK"}, \text{"NG"}\right) \qquad Result_2 = \text{"OK"}$$

$$\Delta_{live} := \frac{5 \cdot w_{ll} \cdot L_{xx}^{\ 4}}{384 \cdot E \cdot I_x} = 0.16\ in$$

$$Result_3 := \text{if}\left(\Delta_{live} \le \Delta_{all\_live}, \text{"OK"}, \text{"NG"}\right) \qquad Result_3 = \text{"OK"}$$

Seismic Load Check

$$M_{dl} := \frac{w_{dl} \cdot L_{xx}^{\ 2}}{8} = 14.14\ k \cdot ft$$ Moments (Working Load)

$$M_{ll} := \frac{w_{ll} \cdot L_{xx}^{\ 2}}{8} = 15.63\ k \cdot ft$$

$$M_{ux} := M_{dl} \cdot \left(1.2 + 0.2 \cdot S_{ds}\right) + M_{ll} \cdot f = 22.48\ k \cdot ft$$ Ultimate Gravity Loads

$$R_\Omega := \left(w_{dl} \cdot \left(1.2 + 0.2 \cdot S_{ds}\right) + w_{ll} \cdot f\right) \cdot \frac{L_{xx}}{2} = 3.6\ k$$

Ultimate Capacities

$$\lambda_r := 0.56 \cdot \sqrt{\frac{E}{F_y}} = 13.49$$ Check Flange Compactness

$$Ratio := b = 6.28$$ Compactness check

$$Check_0 := \text{if}\left(Ratio < \lambda_r, \text{"Non-Slender"}, \text{"Slender"}\right) \qquad Check_0 = \text{"Non-Slender"}$$

$$\lambda_r := 1.49 \cdot \sqrt{\frac{E}{F_y}} = 35.88$$ Check Web Compactness - In axial compression

$$Ratio := h = 51.69$$ Compactness check

$$Check_1 := \text{if}\left(Ratio < \lambda_r, \text{"Non-Slender"}, \text{"Slender"}\right) \qquad Check_1 = \text{"Slender"}$$

COMPRESSION CHECK:

Check Flexural Buckling

Non-Slender Section in Compression (E3)

$\frac{K_x \cdot l_x}{r_x} = 46.8$ $\frac{K_y \cdot l_y}{r_y} = 10.26$

$Klr := \text{if}\left(\frac{K_x \cdot l_x}{r_x} \ge \frac{K_y \cdot l_y}{r_y}, \frac{K_x \cdot l_x}{r_x}, \frac{K_y \cdot l_y}{r_y}\right) = 46.8$

$F_e := \frac{\pi^2 \cdot E}{Klr^2} = 130.67\ ksi$ (E3-4)

$\boxed{F_{cr}} := \text{if}\left(F_e \ge 0.44 \cdot F_y, 0.658^{\frac{F_y}{F_e}} \cdot F_y, 0.877 \cdot F_e\right) = 42.6\ ksi$ (E3-2) (E3-3)

$P_{n\_NS} := A \cdot F_{cr} = 388.94\ k$ (E3-1)

Slender Section in Compression (E7)

$\lambda := h = 51.69$

$\lambda_r = 35.88$

$c_1 := 0.18$ (Table E7.1)

$c_2 := \frac{1 - \sqrt{1 - 4\ c_1}}{2\ c_1} = 1.31$ (E7-4)

$F_{el} := \left(c_2 \cdot \frac{\lambda_r}{\lambda}\right)^2 \cdot F_y = 41.21\ ksi$ (E7-5)

$h_w := d - 2 \cdot t = 15.02\ in$ Height of web

$h_e := \text{if}\left(\lambda \le \lambda_r \cdot \sqrt{\frac{F_y}{F_{cr}}}, (d - 2\ t), (d - 2\ t) \cdot \left(1 - c_1 \cdot \sqrt{\frac{F_{el}}{F_{cr}}}\right) \cdot \sqrt{\frac{F_{el}}{F_{cr}}}\right) = 12.16\ in$ Effective height of web (E7-2), (E7-3)

$A_e := \text{if}\left(Check_0 = \text{“Non−Slender”} \land Check_1 = \text{“Non−Slender”}, A, h_e \cdot t_w + 2 \cdot b_f \cdot t\right)$ Effective Area

$A_e = 8.21\ in^2$

$P_{n\_S} := A_e \cdot F_{cr} = 349.74\ k$ (E7-1)

$P_{nfb} := \text{if}\left(Check_0 = \text{“Non−Slender”} \land Check_1 = \text{“Non−Slender”}, P_{n\_NS}, P_{n\_S}\right)$ $P_{nfb} = 349.74\ k$

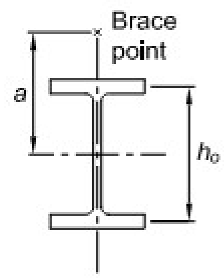
Check Constrained-Axis Torsional Buckling (C-E4)

$t_{nailer} := 5 \cdot in$

$t_{deck} := 0 \cdot in$

$a := \frac{d}{2} + t_{nailer} + \frac{t_{deck}}{2}$

Defines bracing offset along weak-axis to shear centroid. Half of sheathing thickness at wood roof and half of metal deck material thickness (negligible) at metal deck.



(a) Bracing offset along minor axis

$h_o := (d - t) = 15.46\ in$

Distance between flange centroids.

$r_{o.sq} := a^2 + r_x^2 + r_y^2 = 210.16\ in^2$

Polar radius of gyration as defined in commentary.

$$F_{e.ctb} := 0.9 \left(\frac{\pi^2 \cdot E \cdot I_y}{L_T^2} \cdot \left(\frac{h_o^2}{4} + a^2 \right) + G \cdot J \right) \left(\frac{1}{A_e \cdot r_{o.sq}} \right) = 21.41\ ksi$$

(C-E4-1)

For finite bracing stiffness use ω = 0.9

Bracing offset along the minor axis by an amount "a" [see Figure C-E4.2(a)]:

$$F_e = \omega \left[\frac{\pi^2 E I_y}{(L_{cz})^2} \left(\frac{h_o^2}{4} + a^2 \right) + GJ \right] \frac{1}{A r_o^2} \qquad \text{(C-E4-1)}$$

$$F_{cr\_ctb} := \text{if} \left(F_{e.ctb} \geq 0.44 \cdot F_y , 0.658^{\frac{F_y}{F_{e.ctb}}} \cdot F_y , 0.877 \cdot F_{e.ctb} \right) = 18.78\ ksi$$

(E3-2) (E3-3)

$P_{ctb} := F_{cr\_ctb} \cdot A_e = 154.14\ k$

$P_n := min(P_{nfb}, P_{ctb}) = 154.14\ k$

Controlling Compression Capacity

$\phi P_n := 0.9 \cdot P_n$

$\phi P_n = 138.73\ k$

Check combined forces

$$\frac{P_u}{\phi P_n} = 0.36$$

$$DCR_3 := \text{if}\left(\frac{P_u}{\phi P_n} \ge 0.2, \frac{P_u}{\phi P_n} + \frac{8}{9} \cdot \left(\frac{M_{ux}}{\phi M_n}\right), \frac{P_u}{2 \cdot (\phi P_n)} + \left(\frac{M_{ux}}{\phi M_n}\right)\right)$$

$$DCR_3 = 0.46$$

$$Result := \text{if}\left(DCR_3 \le 1.0, \text{“OK”}, \text{“NG”}\right)$$

$$Result = \text{“OK”}$$

06 CONCRETE WALLS

Concrete Panel Out-of-Plane Analysis

Purpose Statement: Out-of-Plane (OOP) Design of Concrete Tilt Up Panels

Referenced Standards:
2021 IBC
ASCE 7-16
ACI 318-19

General Design Criteria

Inputs
Verify
Outputs

$r_v := 0.75\ in$	Reveal Depth
$clr := 0.75\ in$	Concrete Clear Cover
$f_c := 4000 \cdot psi$	Concrete Compressive Strength
$f_y := 60 \cdot ksi$	Reinforcement Yield Strength
$Risk := \text{II}$	Risk Category

General Loading

Gravity

$DL := 14.9 \cdot psf$	Roof Dead Load
$SL := 14 \cdot psf$	Roof Snow Load

Wind (ASCE 7-16 30.4)

$S_w := 97 \cdot mph$	Mean Wind Speed
$Exposure := \text{B}$	Exposure Category
$h_{mean} := 37\ ft$	Mean Roof Height
$K_z := 0.74$	*(Table 26.10-1)*
$K_{zt} := 1.0$	*(Section 26.8)*
$K_d := 0.85$	*(Table 26.6-1)*
$K_e := 1.0$	*(Table 26.9-1)*

$$q_h := 0.00256\ K_{zt}\ K_z\ K_d\ K_e \left(\frac{S_w}{mph}\right)^2 \cdot 1\ psf = 15.15\ psf$$

Concrete Panel Out-of-Plane Analysis

Seismic (ASCE 7-16 12.11.1)

$I_e = 1$ Importance Factor - Seismic (Table 1.5-2)

$S_{DS} := 0.843$ Design Spectral Acceleration

$F_p := \max\left(0.4 \cdot S_{DS} \cdot I_e, 0.1\right) = 0.34$ Seismic OOP coefficient

green shading is done

South Elevation

Solid Panel w/ Mezzanine- 8" THK w/ #5 @ 10 " OC VERT EF

Panel Geometry

$t := 8\ in$ Panel Thickness

$ht := 36\ ft + \frac{5}{24}\ in$ Roof Height (Panel Span)

$h_p := 43.5\ ft - ht = 7.48\ ft$ Parapet Height

$DockHigh :=$ No Dock High Span Reduction

$w := 26\ ft + 2\ in$ Panel Width

$e := \frac{t}{2} + 3 \cdot in = 7\ in$ Roof Load Eccentricity

$h_T := ht + h_p = 43.5\ ft$ Total Panel Height

Panel Load Factors

$L := 1\ ft$ Panel Width Considered (PWC) for girder/pad ftg

$x_g := 1$ Gravity Factor

$x_{OOP} := 1.0$ OOP Factor

Panel Wind Loading Per ASCE 7-16 Chapter 30 - Part 1 Method

$GCpi := 0.18$ Internal Pressure Coefficient per Table 26.13-1

$$A_{eff} := \frac{1}{3}\ ht^2 = 432.4\ ft^2$$

$Slope :=$ Y

If roof slope is <10 degrees, GCp can be reduced by 10%

$$GCp_{poscalc} := \left(1.0 - \frac{\ln\left(\frac{A_{eff}}{ft^2}\right) - \ln(10)}{\ln(500) - \ln(10)} (1 - 0.7)\right) (1 - Slope)$$

Positive pressure coefficient (zones 4 and 5)

$$GCp_{negcalc} := \left(1.4 - \frac{\ln\left(\frac{A_{eff}}{ft^2}\right) - \ln(10)}{\ln(500) - \ln(10)} (1.4 - 0.8)\right) (1 - Slope) (-1)$$

Negative pressure coefficient (zone 5)

$$GCp_{pos} := \begin{cases} 0.7\,(1 - Slope) & \text{if } A_{eff} > 500\ ft^2 \\ 1.0\,(1 - Slope) & \text{else if } A_{eff} < 10\ ft^2 \\ GCp_{poscalc} & \text{else} \end{cases}$$

Check against upper and lower bound values for CGp based on Aeff

$$GCp_{neg} := \begin{cases} -0.8\,(1 - Slope) & \text{if } A_{eff} > 500\ ft^2 \\ -1.4\,(1 - Slope) & \text{else if } A_{eff} < 10\ ft^2 \\ GCp_{negcalc} & \text{else} \end{cases}$$

$$p_1 := q_h \cdot (GCp_{pos} - GCpi)$$

$$p_2 := q_h \cdot (GCp_{neg} - GCpi)$$

$$p_3 := q_h \cdot (GCp_{pos} + GCpi)$$

$$p_4 := q_h \cdot (GCp_{neg} + GCpi)$$

$$p_{net} := \max(|p_1|, |p_2|, |p_3|, |p_4|) = 13.9\ psf$$

Panel Loading

$w_{rf} := 59\ ft + 4\ in$ Joist Span/Spacing Adjacent to Panel

$load_{drift} := 59\ psf$ Drift Load at Panel

$w_{drift} := 14\ ft + 3\ in$ Drift Length

Concrete Panel Out-of-Plane Analysis

$$P_{drift} := \text{if } w_{rf} \geq w_{drift} \;\; \left\| P \leftarrow load_{drift} \cdot w_{drift} \cdot \left(\frac{1}{2} - \frac{w_{drift}}{6\ w_{rf}}\right) \right. \;\; \text{else if } w_{rf} < w_{drift} \;\; \left\| P \leftarrow load_{drift} \cdot w_{rf} \cdot \left(\frac{1}{2} - \frac{w_{rf}}{6 \cdot w_{drift}}\right) \right.$$

Snow Drift Reaction at Panel $P_{drift} = 0.39\ \frac{k}{ft}$

$$P_{dlr} := x_g \cdot DL \cdot \frac{w_{rf}}{2} + x_g \cdot 25\ psf \cdot \frac{20\ ft}{2}$$

Roof Dead Load + Mezzanine Dead Load $P_{dlr} = 0.7\ \frac{k}{ft}$

$$P_{dlp} := x_g \cdot t \cdot 150\ pcf \cdot \left(\frac{ht}{2} + h_p\right)$$

Panel Dead Load $P_{dlp} = 2.5\ \frac{k}{ft}$

$$P_{sl} := x_g \cdot \left(SL \cdot \frac{w_{rf}}{2} + P_{drift}\right)$$

Total Snow Load $P_{sl} = 0.8\ \frac{k}{ft}$

$$P_{ll\_mezz} := x_g \cdot 65\ psf \cdot \frac{20\ ft}{2}$$

Mezzanine Live Load $P_{ll\_mezz} = 0.7\ \frac{k}{ft}$

$$M_w := x_{OOP} \cdot \frac{p_{net} \cdot ht^2}{8}$$

Moment Due To Wind $M_w = 2.3\ \frac{k \cdot ft}{ft}$

$$M_s := x_{OOP} \cdot \frac{F_p \cdot t \cdot 150\ pcf \cdot ht^2}{8}$$

Moment Due to Seismic $M_s = 5.5\ \frac{k \cdot ft}{ft}$

Strength Load Combinations (LRFD)

(ASCE 7-16 2.3)

$$x := \begin{bmatrix} 1.2 \cdot P_{dlr} & 1.2 \cdot P_{dlp} & 1.6 \cdot P_{sl} & 0.5 \cdot M_w \\ 1.2 \cdot P_{dlr} & 1.2 \cdot P_{dlp} & 0.5 \cdot (P_{sl}) + P_{ll\_mezz} & 1.0 \cdot M_w \\ 1.2 \cdot P_{dlr} & 1.2 \cdot P_{dlp} & 0.2 \cdot P_{sl} + 0.5 \cdot P_{ll\_mezz} & 1.0 \cdot M_s \\ 0.9 \cdot P_{dlr} & 0.9 \cdot P_{dlp} & 0 \cdot \frac{k}{ft} & 1.0 \cdot M_w \\ 0.9 \cdot P_{dlr} & 0.9 \cdot P_{dlp} & 0 \cdot \frac{k}{ft} & 1.0 \cdot M_s \end{bmatrix}$$

$$i := 0 \,..\, \text{rows}(x) - 1$$

Service/Deflection Load Combinations (ASD)

(2021 IBC 1605.2)

$$\omega := 1.0$$

$$y := \begin{bmatrix} 1.0 \cdot P_{dlr} & 1.0\ P_{dlp} & P_{sl} + P_{ll\_mezz} & 0.5 \cdot (\omega \cdot 0.6 \cdot M_w) \\ 1.0 \cdot P_{dlr} & 1.0 \cdot P_{dlp} & 0.5 \cdot P_{sl} + P_{ll\_mezz} & 1.0 \cdot (\omega \cdot 0.6 \cdot M_w) \\ 1.0 \cdot P_{dlr} & 1.0 \cdot P_{dlp} & P_{sl} + P_{ll\_mezz} & \left(\frac{1}{1.4}\right) \cdot M_s \\ 0.67 \cdot P_{dlr} & 0.67 \cdot P_{dlp} & 0 \cdot \frac{k}{ft} & 1.0 \cdot (\omega \cdot 0.6 \cdot M_w) \\ 0.9 \cdot P_{dlr} & 0.9 \cdot P_{dlp} & 0 \cdot \frac{k}{ft} & \left(\frac{1}{1.4}\right) \cdot M_s \end{bmatrix}$$

Define Panel Reinforcing

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

$SP := 10 \cdot in$ Vertical Bar Spacing

Concrete Panel Out-of-Plane Analysis

$rebar\_location := EF$ — Steel Layer: Each Face (EF) or Centerline (CL)

$stirrup\_size := NA$ — Stirrup Bar Size

$A_b := A_{bar\_size}$ — Vertical Bar Area

$diam_b := D_{bar\_size}$ — Vertical Bar Diameter

$diam_{str} := D_{stirrup\_size}$ — Stirrup Bar Diameter

$d_{ef} := t - \frac{diam_b}{2} - diam_{str} - clr - r_v$ — Steel Depth If Each Face

$d_{cl} := \frac{t}{2} - r_v$ — Steel Depth If Centerline

$d := \text{if}\left(rebar\_location = 0, d_{ef}, d_{cl}\right)$ — Steel Depth — $d = 6.19\ in$

$A_s := \frac{A_b}{SP}$ — Vertical Area of Steel

Check Axial Capacity

(ACI 318-19 11.8.1.1(d))

$$P_{u_i} := \left\| \begin{array}{l} P_u \leftarrow 0 \\ \left(x_{i,0} + x_{i,1} + x_{i,2}\right) \end{array} \right.$$

$P_a := 0.06 \cdot f_c \cdot t$

$$flag_i := \text{if}\left(\frac{P_{u_i}}{P_a} \le 1, \text{“OK”}, \text{“Revise”}\right)$$

$$\frac{P_u}{P_a} = \begin{bmatrix} 0.2 \\ 0.2 \\ 0.2 \\ 0.1 \\ 0.1 \end{bmatrix} \qquad flag = \begin{bmatrix} \text{“OK”} \\ \text{“OK”} \\ \text{“OK”} \\ \text{“OK”} \\ \text{“OK”} \end{bmatrix}$$

$compression\_check := flag$

Determine Moment Capacity

$$\beta_1 := 0.85 - 0.05 \cdot \left(\frac{f_c - 4000 \cdot psi}{1000 \cdot psi}\right)$$

$P_u := P_u \cdot ft \qquad A_s := A_s \cdot ft$

$$X_i := \frac{P_{u_i} + A_s \cdot f_y}{0.85 \cdot f_c \cdot ft} \qquad M_{n_i} := \left(A_s \cdot f_y + P_{u_i}\right) \cdot \left(d - \frac{X_i}{2}\right) - P_{u_i} \cdot \left(d - \frac{t}{2}\right)$$

$\phi M_{n_i} := 0.9 \cdot M_{n_i}$ — Factored Moment Capacity

$$\phi M_n = \begin{bmatrix} 11.2 \\ 11.2 \\ 11 \\ 10.6 \\ 10.6 \end{bmatrix} k \cdot ft$$

Concrete Panel Out-of-Plane Analysis

Check for Tension-Controlled Section

(ACI 318-19 11.8.1.1(b))

$$C_i := \frac{X_i}{\beta_1}$$

$$flag_i := \text{if}\left(\frac{C_i}{d} < 0.375, \text{“OK”}, \text{“Revise”}\right)$$

$$\frac{C}{d} = \begin{bmatrix} 0.128 \\ 0.127 \\ 0.124 \\ 0.118 \\ 0.118 \end{bmatrix} \qquad flag = \begin{bmatrix} \text{“OK”} \\ \text{“OK”} \\ \text{“OK”} \\ \text{“OK”} \\ \text{“OK”} \end{bmatrix}$$

$$tension\_controlled := flag$$

Determine Deflection Limits

$$\Delta_{max} := \frac{ht}{150}$$ Allowable Panel Deflection

$$ht' := \text{if}(DockHigh = 0, ht, 0.8 \cdot ht) = 36\ ft$$ Adjust Span per Dock High Condition

$$E_c := 57000 \cdot \sqrt{\frac{f_c}{psi}} \cdot psi \qquad n := \frac{29000 \cdot ksi}{E_c} \qquad A_{se_i} := A_s + \frac{P_{u_i} \cdot t}{2 \cdot f_y \cdot d} \qquad C_i := \frac{X_i}{0.85}$$

$$f_r := 7.5 \cdot \sqrt{\frac{f_c}{psi}} \cdot psi \qquad I_g := \frac{(1\ ft) \cdot t^3}{12} \qquad I_{cr_i} := \frac{12 \cdot in \cdot \left(C_i\right)^3}{3} + n \cdot A_{se_i} \cdot \left(d - C_i\right)^2$$

$$M_{cr} := \frac{2 \cdot f_r \cdot I_g}{t} \qquad \Delta_{cr} := \frac{5 \cdot M_{cr} \cdot ht'^2}{48 \cdot E_c \cdot I_g} \qquad \Delta_{n_i} := \frac{5 \cdot M_{n_i} \cdot ht'^2}{48 \cdot E_c \cdot I_{cr_i}}$$

Check Moment Capacity

(ACI 318-19 11.8.3.1)

$$M_{ua_i} := \left(x_{i,3} + \left(x_{i,0} + x_{i,2}\right) \cdot \frac{e}{2}\right) \cdot ft \qquad M_{u_i} := \frac{M_{ua_i}}{1 - \frac{5 \cdot P_{u_i} \cdot ht'^2}{0.75 \cdot 48 \cdot E_c \cdot I_{cr_i}}}$$

$$flag_i := \text{if}\left(\frac{M_{u_i}}{\phi M_{n_i}} \le 1, \text{“OK”}, \text{“Revise”}\right)$$

$$\frac{M_u}{\phi M_n} = \begin{bmatrix} 0.2 \\ 0.4 \\ 0.8 \\ 0.3 \\ 0.7 \end{bmatrix} \qquad flag = \begin{bmatrix} \text{“OK”} \\ \text{“OK”} \\ \text{“OK”} \\ \text{“OK”} \\ \text{“OK”} \end{bmatrix}$$

$$capacity\_check := flag$$

Check Deflection

(ACI 318-19 11.8.4)

$$M_{a_i} := \left(y_{i,3} + \left(y_{i,0} + y_{i,2}\right) \cdot \frac{e}{2}\right) ft \qquad P_{s_i} := \left(y_{i,0} + y_{i,1} + y_{i,2}\right) \cdot ft$$

Iterate Service Deflection

$$\Delta_{s_i} := 0 \cdot in$$ Initial Guess

$$\Delta_s := \Delta f_s\left(M_a, P_s, \Delta_s, M_{cr}, \Delta_{cr}, M_n, \Delta_n\right)$$ Function iterates service deflections per ACI 318-19 *Sec 11.8.4*

$$flag_i := \text{if}\left(\frac{\Delta_{s_i}}{\Delta_{max}} \le 1, \text{“OK”}, \text{“Revise”}\right)$$

$$\frac{\Delta_s}{\Delta_{max}} = \begin{bmatrix} 0.1 \\ 0.1 \\ 0.6 \\ 0.1 \\ 0.4 \end{bmatrix} \qquad flag = \begin{bmatrix} \text{“OK”} \\ \text{“OK”} \\ \text{“OK”} \\ \text{“OK”} \\ \text{“OK”} \end{bmatrix}$$

$$deflection\_check := flag$$

Results Summary

Panel Thickness $t = 8\ in$

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

Bar Spacing $SP = 10\ in$

Bar Location $layer = \text{“EF”}$

Concrete Panel Out-of-Plane Analysis

North Elevation

Panel w/ Drive-In Door (P4) - 9.5" THK w/ (12) #5 VERT EF

Drive In Door Leg

Panel Geometry

$t := 9.5\ in$ — Panel Thickness

$ht := 36\ ft + 9\ in$ — Roof Height (Panel Span)

$h_p := 0\ ft$ — Parapet Height

$DockHigh :=$ Yes — Dock High Span Reduction

$w := 26 \cdot ft$ — Panel Width

$L := 5\ ft$ — Panel Leg Width

$ht_o := 14\ ft$ — Panel Opening Height

$w_o := 16\ ft$ — Panel Opening Width

$e := \frac{t}{2} + 3 \cdot in = 7.75\ in$ — Roof Load Eccentricity

$h_T := ht + h_p = 36.75\ ft$ — Total Panel Height

Panel Load Factors

$x_g := 1 + \frac{w_o}{2 \cdot L} = 2.6$ — Gravity Factor

$x_{OOP} := 1 + \frac{w_o}{2 \cdot L} = 2.6$ — OOP Factor

Panel Wind Loading Per ASCE 7-16 Chapter 30 - Part 1 Method

$GCpi := 0.18$ — Internal Pressure Coefficient per Table 26.13-1

$A_{eff} := \frac{1}{3}\ ht^2 = 450.2\ ft^2$

$Slope :=$ Y — If roof slope is <10 degrees, GCp can be reduced by 10%

$$GCp_{poscalc} := \left(1.0 - \frac{\ln\left(\frac{A_{eff}}{ft^2}\right) - \ln(10)}{\ln(500) - \ln(10)}(1-0.7)\right)(1-Slope)$$

Positive pressure coefficient (zones 4 and 5)

$$GCp_{negcalc} := \left(1.4 - \frac{\ln\left(\frac{A_{eff}}{ft^2}\right) - \ln(10)}{\ln(500) - \ln(10)}(1.4-0.8)\right)(1-Slope)(-1)$$

Negative pressure coefficient (zone 5)

$$GCp_{pos} := \begin{cases} 0.7\,(1-Slope) & \text{if } A_{eff} > 500\ ft^2 \\ 1.0\,(1-Slope) & \text{else if } A_{eff} < 10\ ft^2 \\ GCp_{poscalc} & \text{else} \end{cases}$$

Check against upper and lower bound values for CGp based on Aeff

$$GCp_{neg} := \begin{cases} -0.8\,(1-Slope) & \text{if } A_{eff} > 500\ ft^2 \\ -1.4\,(1-Slope) & \text{else if } A_{eff} < 10\ ft^2 \\ GCp_{negcalc} & \text{else} \end{cases}$$

$$p_1 := q_h \cdot (GCp_{pos} - GCpi)$$

$$p_2 := q_h \cdot (GCp_{neg} - GCpi)$$

$$p_3 := q_h \cdot (GCp_{pos} + GCpi)$$

$$p_4 := q_h \cdot (GCp_{neg} + GCpi)$$

$$p := \max\left(|p_1|, |p_2|, |p_3|, |p_4|\right) = 13.9\ psf$$

Panel Loading

$w_{rf} := 60 \cdot ft$ Joist Span/Spacing Adjacent to Panel

$load_{drift} := 0\ psf$ Drift Load at Panel

$w_{drift} := 0\ ft$ Drift Length

Concrete Panel Out-of-Plane Analysis

$$P_{drift} := \text{if } w_{rf} \geq w_{drift}$$
$$\quad \| P \leftarrow load_{drift} \cdot w_{drift} \cdot \left(\frac{1}{2} - \frac{w_{drift}}{6\ w_{rf}}\right)$$
$$\quad \text{else if } w_{rf} < w_{drift}$$
$$\quad \| P \leftarrow load_{drift} \cdot w_{rf} \cdot \left(\frac{1}{2} - \frac{w_{rf}}{6 \cdot w_{drift}}\right)$$

Snow Drift Reaction at Panel

$$P_{drift} = 0\ \frac{k}{ft}$$

$$P_{dlr} := x_g \cdot DL \cdot \frac{w_{rf}}{2}$$

Roof Dead Load

$$P_{dlr} = 1.2\ \frac{k}{ft}$$

$$P_{dlp} := x_g \cdot t \cdot 150\ pcf \cdot \left(\frac{ht}{2} + h_p\right)$$

Panel Dead Load

$$P_{dlp} = 5.7\ \frac{k}{ft}$$

$$P_{sl} := x_g \cdot \left((20\ psf) \cdot \frac{w_{rf}}{2}\right)$$

Total Snow Load

$$P_{sl} = 1.6\ \frac{k}{ft}$$

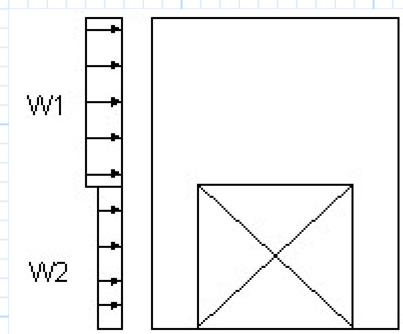
$$M_w := x_{OOP} \cdot \frac{p \cdot ht^2}{8}$$

Moment Due To Wind

$$M_w = 6.1\ \frac{k \cdot ft}{ft}$$

$$w_1 := F_p \cdot 150\ pcf\ t \cdot x_{OOP}$$

$$w_2 := \left(F_p \cdot 10 \cdot psf \cdot w_o \cdot 0.5 + F_p \cdot 150\ pcf \cdot t \cdot L\right) \cdot \frac{1}{L}$$



$$R_{bot} := \frac{w_2 \cdot ht}{2} + \frac{(w_1 - w_2) \cdot (ht - ht_o)^2}{2 \cdot ht}$$

$$R_{top} := \left(w_2 \cdot ht_o + w_1 \cdot (ht - ht_o)\right) - R_{bot}$$

$$x_m := \frac{R_{top}}{w_1}$$

$$M_s := R_{top} \cdot x_m - \frac{w_1 \cdot {x_m}^2}{2}$$

Moment Due to Seismic w/ Opening

$$M_s = 14.8\ \frac{k \cdot ft}{ft}$$

Concrete Panel Out-of-Plane Analysis

Strength Load Combinations (LRFD)

(ASCE 7-16 2.3)

$$x := \begin{bmatrix} 1.2 \cdot P_{dlr} & 1.2 \cdot P_{dlp} & 1.6 \cdot P_{sl} & 0.5 \cdot M_w \\ 1.2 \cdot P_{dlr} & 1.2 \cdot P_{dlp} & 0.5 \cdot P_{sl} & 1.0 \cdot M_w \\ 1.2 \cdot P_{dlr} & 1.2 \cdot P_{dlp} & 0.5 \cdot P_{sl} & 1.0 \cdot M_s \\ 0.9 \cdot P_{dlr} & 0.9 \cdot P_{dlp} & 0 \cdot \frac{k}{ft} & 1.0 \cdot M_w \\ 0.9 \cdot P_{dlr} & 0.9 \cdot P_{dlp} & 0 \cdot \frac{k}{ft} & 1.0 \cdot M_s \end{bmatrix}$$

$$i := 0 .. \text{rows}(x) - 1$$

Service/Deflection Load Combinations (ASD)

(2021 IBC 1605.2)

$$\omega := 1.0$$

$$y := \begin{bmatrix} 1.0 \cdot P_{dlr} & 1.0 \cdot P_{dlp} & 1.0 \cdot P_{sl} & 0.5 \cdot (\omega \cdot 0.6 \cdot M_w) \\ 1.0 \cdot P_{dlr} & 1.0 \cdot P_{dlp} & 0.5 \cdot P_{sl} & 1.0 \cdot (\omega \cdot 0.6 \cdot M_w) \\ 1.0 \cdot P_{dlr} & 1.0 \cdot P_{dlp} & 1.0 \cdot P_{sl} & \left(\frac{1}{1.4}\right) \cdot M_s \\ 0.67 \cdot P_{dlr} & 0.67 \cdot P_{dlp} & 0 \cdot \frac{k}{ft} & 1.0 \cdot (\omega \cdot 0.6 \cdot M_w) \\ 0.9 \cdot P_{dlr} & 0.9 \cdot P_{dlp} & 0 \cdot \frac{k}{ft} & \left(\frac{1}{1.4}\right) \cdot M_s \end{bmatrix}$$

Define Panel Reinforcing

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

$NB := 12$ — Number of Vertical Bars

$rebar\_location := \text{EF}$ — Steel Layer: Each Face (EF) or Centerline (CL)

$stirrup\_size := \text{NA}$ — Stirrup Bar Size

$A_b := A_{bar\_size}$ — Vertical Bar Area

$diam_b := D_{bar\_size}$ — Vertical Bar Diameter

$diam_{str} := D_{stirrup\_size}$ — Stirrup Bar Diameter

$d_{ef} := t - \frac{diam_b}{2} - diam_{str} - clr - r_v$ — Steel Depth If Each Face

$d_{cl} := \frac{t}{2} - r_v$ — Steel Depth If Centerline

$d := \text{if}\left(rebar\_location = 0, d_{ef}, d_{cl}\right)$ — Steel Depth — $d = 7.69\ in$

$S := \frac{L - 1.5\ in}{NB - 1}$ — Vertical Bar Spacing OC

$flag := \text{if}\left(S \geq 3\ in, \text{“OK”}, \text{“Revise”}\right)$ — $S = 5.3\ in$ — $flag = \text{“OK”}$

$A_s := \frac{NB \cdot A_b}{L}$ — Vertical Area of Steel

Concrete Panel Out-of-Plane Analysis

Check Axial Capacity

(ACI 318-19 11.8.1.1(d))

$$P_{u_i} := \left\| \begin{array}{l} P_u \leftarrow 0 \\ \left(x_{i,0} + x_{i,1} + x_{i,2}\right) \end{array} \right.$$

$$P_a := 0.06 \cdot f_c \cdot t$$

$$flag_i := \text{if}\left(\frac{P_{u_i}}{P_a} \le 1, \text{“OK”}, \text{“Revise”}\right)$$

$$compression\_check := flag$$

$$\frac{P_u}{P_a} = \begin{bmatrix} 0.4 \\ 0.3 \\ 0.3 \\ 0.2 \\ 0.2 \end{bmatrix} \qquad flag = \begin{bmatrix} \text{“OK”} \\ \text{“OK”} \\ \text{“OK”} \\ \text{“OK”} \\ \text{“OK”} \end{bmatrix}$$

Determine Moment Capacity

$$\beta_1 := 0.85 - 0.05 \cdot \left(\frac{f_c - 4000 \cdot psi}{1000 \cdot psi}\right)$$

$$P_u := P_u \cdot ft \qquad A_s := A_s \cdot ft$$

$$X_i := \frac{P_{u_i} + A_s \cdot f_y}{0.85 \cdot f_c \cdot ft}$$

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

$$\phi M_{n_i} := 0.9 \cdot M_{n_i}$$

Factored Moment Capacity

$$\phi M_n = \begin{bmatrix} 26.7 \\ 26.3 \\ 26.3 \\ 25.6 \\ 25.6 \end{bmatrix} k \cdot ft$$

Check for Tension-Controlled Section

(ACI 318-19 11.8.1.1(b))

$$C_i := \frac{X_i}{\beta_1}$$

$$flag_i := \text{if}\left(\frac{C_i}{d} < 0.375, \text{“OK”}, \text{“Revise”}\right)$$

$$tension\_controlled := flag$$

$$\frac{C}{d} = \begin{bmatrix} 0.208 \\ 0.201 \\ 0.201 \\ 0.191 \\ 0.191 \end{bmatrix} \qquad flag = \begin{bmatrix} \text{“OK”} \\ \text{“OK”} \\ \text{“OK”} \\ \text{“OK”} \\ \text{“OK”} \end{bmatrix}$$

Determine Deflection Limits

$$\Delta_{max} := \frac{ht}{150}$$

Allowable Panel Deflection

$$ht' := \text{if}\left(DockHigh = 0, ht, 0.8 \cdot ht\right) = 29.4\ ft$$

Adjust Span per Dock High Condition

$$E_c := 57000 \cdot \sqrt{\frac{f_c}{psi}} \cdot psi \qquad n := \frac{29000 \cdot ksi}{E_c} \qquad A_{se_i} := A_s + \frac{P_{u_i} \cdot t}{2 \cdot f_y \cdot d} \qquad C_i := \frac{X_i}{0.85}$$

$$f_r := 7.5 \cdot \sqrt{\frac{f_c}{psi}} \cdot psi \qquad I_g := \frac{(1\ ft) \cdot t^3}{12} \qquad I_{cr_i} := \frac{12 \cdot in \cdot \left(C_i\right)^3}{3} + n \cdot A_{se_i} \cdot \left(d - C_i\right)^2$$

$$M_{cr} := \frac{2 \cdot f_r \cdot I_g}{t} \qquad \Delta_{cr} := \frac{5 \cdot M_{cr} \cdot ht'^2}{48 \cdot E_c \cdot I_g} \qquad \Delta_{n_i} := \frac{5 \cdot M_{n_i} \cdot ht'^2}{48 \cdot E_c \cdot I_{cr_i}}$$

Concrete Panel Out-of-Plane Analysis

Check Moment Capacity

(ACI 318-19 11.8.3.1)

$$M_{ua_i} := \left(x_{i,3} + \left(x_{i,0} + x_{i,2}\right) \cdot \frac{e}{2}\right) \cdot ft$$

$$M_{u_i} := \frac{M_{ua_i}}{1 - \dfrac{5 \cdot P_{u_i} \cdot ht'^2}{0.75 \cdot 48 \cdot E_c \cdot I_{cr_i}}}$$

$$flag_i := \text{if}\left(\frac{M_{u_i}}{\phi M_{n_i}} \le 1, \text{“OK”}, \text{“Revise”}\right)$$

$$capacity\_check := flag$$

$$\frac{M_u}{\phi M_n} = \begin{bmatrix} 0.2 \\ 0.31 \\ 0.7 \\ 0.28 \\ 0.67 \end{bmatrix} \qquad flag = \begin{bmatrix} \text{“OK”} \\ \text{“OK”} \\ \text{“OK”} \\ \text{“OK”} \\ \text{“OK”} \end{bmatrix}$$

Check Deflection

(ACI 318-19 11.8.4)

$$M_{a_i} := \left(y_{i,3} + \left(y_{i,0} + y_{i,2}\right) \cdot \frac{e}{2}\right) ft \qquad P_{s_i} := \left(y_{i,0} + y_{i,1} + y_{i,2}\right) \cdot ft$$

Iterate Service Deflection

$\Delta_{s_i} := 0 \cdot in$ Initial Guess

$\Delta_s := \Delta f_s\left(M_a, P_s, \Delta_s, M_{cr}, \Delta_{cr}, M_n, \Delta_n\right)$ Function iterates service deflections per ACI 318-19 *Sec 11.8.4*

$$flag_i := \text{if}\left(\frac{\Delta_{s_i}}{\Delta_{max}} \le 1, \text{“OK”}, \text{“Revise”}\right)$$

$$deflection\_check := flag$$

$$\frac{\Delta_s}{\Delta_{max}} = \begin{bmatrix} 0.05 \\ 0.07 \\ 0.53 \\ 0.07 \\ 0.5 \end{bmatrix} \qquad flag = \begin{bmatrix} \text{“OK”} \\ \text{“OK”} \\ \text{“OK”} \\ \text{“OK”} \\ \text{“OK”} \end{bmatrix}$$

Results Summary

Panel Thickness	$t = 9.5\ in$	**Bar Size**	$rebar_{bar\_size} = \text{“\#5”}$
Panel Leg Width	$L = 5\ ft$	**Bar Quantity**	$NB = 12$
		Bar Location	$layer = \text{“EF”}$

Concrete Panel Out-of-Plane Analysis

Panel w/ Dock Door (P5) - 2' leg 9.5" THK w/ (8) #5 VERT EF

Panel Geometry

$t := 9.5\ in$ — Panel Thickness

$ht := 36\ ft + 9\ in$ — Roof Height (Panel Span)

$h_p := 0$ — Parapet Height

$DockHigh := \text{Yes}$ — Dock High Span Reduction

$w := 26 \cdot ft$ — Panel Width

$L := 2\ ft$ — Panel Leg Width

$ht_o := 10\ ft$ — Panel Opening Height

$w_o := 9\ ft$ — Panel Opening Width

$e := \frac{t}{2} + 3 \cdot in = 7.75\ in$ — Roof Load Eccentricity

$h_T := ht + h_p = 36.75\ ft$ — Total Panel Height

Panel Load Factors

$x_g := 1 + \frac{w_o}{2 \cdot L} = 3.3$ — Gravity Factor

$x_{OOP} := 1 + \frac{w_o}{2 \cdot L} = 3.3$ — OOP Factor

Panel Wind Loading Per ASCE 7-16 Chapter 30 - Part 1 Method

$GCpi := 0.18$ — Coefficient per Table 26.13-1

$A_{eff} := \frac{1}{3}\ ht^2 = 450.2\ ft^2$

$Slope := \text{Y}$ — If roof slope is <10 degrees, GCp can be reduced by 10%

Concrete Panel Out-of-Plane Analysis

$$GCp_{poscalc} := \left(1.0 - \frac{\ln\left(\frac{A_{eff}}{ft^2}\right) - \ln(10)}{\ln(500) - \ln(10)} (1 - 0.7)\right)(1 - Slope)$$

Positive pressure coefficient (zones 4 and 5)

$$GCp_{negcalc} := \left(1.4 - \frac{\ln\left(\frac{A_{eff}}{ft^2}\right) - \ln(10)}{\ln(500) - \ln(10)} (1.4 - 0.8)\right)(1 - Slope)(-1)$$

Negative pressure coefficient (zone 5)

$$GCp_{pos} := \begin{array}{l} \text{if } A_{eff} > 500\ ft^2 \\ \quad \| 0.7\ (1 - Slope) \\ \text{else if } A_{eff} < 10\ ft^2 \\ \quad \| 1.0\ (1 - Slope) \\ \text{else} \\ \quad \| GCp_{poscalc} \end{array}$$

Check against upper and lower bound values for CGp based on Aeff

$$GCp_{neg} := \begin{array}{l} \text{if } A_{eff} > 500\ ft^2 \\ \quad \| -0.8\ (1 - Slope) \\ \text{else if } A_{eff} < 10\ ft^2 \\ \quad \| -1.4\ (1 - Slope) \\ \text{else} \\ \quad \| GCp_{negcalc} \end{array}$$

$$p_1 := q_h \cdot (GCp_{pos} - GCpi)$$

$$p_2 := q_h \cdot (GCp_{neg} - GCpi)$$

$$p_3 := q_h \cdot (GCp_{pos} + GCpi)$$

$$p_4 := q_h \cdot (GCp_{neg} + GCpi)$$

$$p := \max\left(|p_1|, |p_2|, |p_3|, |p_4|\right) = 13.9\ psf$$

Panel Loading

$w_{rf} := 60\ ft$ — Joist Span/Spacing Adjacent to Panel

$load_{drift} := 0\ psf$ — Drift Load at Panel

$w_{drift} := 0.0\ ft$ — Drift Length

Concrete Panel Out-of-Plane Analysis

$$P_{drift} := \text{if } w_{rf} \ge w_{drift}\ \Big\|\ P \leftarrow load_{drift} \cdot w_{drift} \cdot \left(\frac{1}{2} - \frac{w_{drift}}{6\ w_{rf}}\right)\quad \text{else if } w_{rf} < w_{drift}\ \Big\|\ P \leftarrow load_{drift} \cdot w_{rf} \cdot \left(\frac{1}{2} - \frac{w_{rf}}{6 \cdot w_{drift}}\right)$$

Snow Drift Reaction at Panel $P_{drift} = 0\ \frac{k}{ft}$

$$P_{dlr} := x_g \cdot DL \cdot \frac{w_{rf}}{2}$$

Roof Dead Load $P_{dlr} = 1.2\ \frac{k}{ft}$

$$P_{dlp} := x_g \cdot t \cdot 150\ pcf \cdot \left(\frac{ht}{2} + h_p\right)$$

Panel Dead Load $P_{dlp} = 7.1\ \frac{k}{ft}$

$$P_{sl} := x_g \cdot \left(20\ psf \cdot \frac{w_{rf}}{2}\right)$$

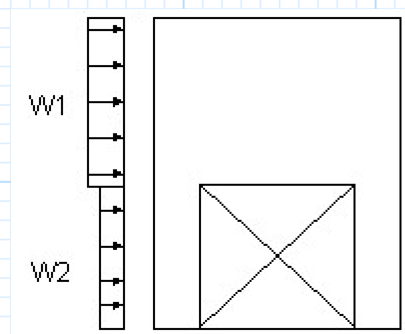
Total Snow Load $P_{sl} = 2\ \frac{k}{ft}$

$$M_w := x_{OOP} \cdot \frac{p \cdot ht^2}{8}$$

Moment Due To Wind $M_w = 7.6\ \frac{k \cdot ft}{ft}$

$$w_1 := F_p \cdot 150\ pcf\ t \cdot x_{OOP}$$

$$w_2 := \left(F_p \cdot 10 \cdot psf \cdot w_o \cdot 0.5 + F_p \cdot 150\ pcf \cdot t \cdot L\right) \cdot \frac{1}{L}$$



$$R_{bot} := \frac{w_2 \cdot ht}{2} + \frac{(w_1 - w_2) \cdot (ht - ht_o)^2}{2 \cdot ht}$$

$$R_{top} := \left(w_2 \cdot ht_o + w_1 \cdot (ht - ht_o)\right) - R_{bot}$$

$$x_m := \frac{R_{top}}{w_1}$$

$$M_s := R_{top} \cdot x_m - \frac{w_1 \cdot {x_m}^2}{2}$$

Moment Due to Seismic w/ Opening $M_s = 20\ \frac{k \cdot ft}{ft}$

Strength Load Combinations (LRFD)

(ASCE 7-16 2.3)

$$x := \begin{bmatrix} 1.2 \cdot P_{dlr} & 1.2 \cdot P_{dlp} & 1.6 \cdot P_{sl} & 0.5 \cdot M_w \\ 1.2 \cdot P_{dlr} & 1.2 \cdot P_{dlp} & 0.5 \cdot P_{sl} & 1.0 \cdot M_w \\ 1.2 \cdot P_{dlr} & 1.2 \cdot P_{dlp} & 0.5 \cdot P_{sl} & 1.0 \cdot M_s \\ 0.9 \cdot P_{dlr} & 0.9 \cdot P_{dlp} & 0 \cdot \frac{k}{ft} & 1.0 \cdot M_w \\ 0.9 \cdot P_{dlr} & 0.9 \cdot P_{dlp} & 0 \cdot \frac{k}{ft} & 1.0 \cdot M_s \end{bmatrix}$$

Service/Deflection Load Combinations (ASD)

(2021 IBC 1605.2)

$$\omega := 1.0$$

$$y := \begin{bmatrix} 1.0 \cdot P_{dlr} & 1.0 \cdot P_{dlp} & 1.0 \cdot P_{sl} & 0.5 \cdot (\omega \cdot 0.6 \cdot M_w) \\ 1.0 \cdot P_{dlr} & 1.0 \cdot P_{dlp} & 0.5 \cdot P_{sl} & 1.0 \cdot (\omega \cdot 0.6 \cdot M_w) \\ 1.0 \cdot P_{dlr} & 1.0 \cdot P_{dlp} & 1.0 \cdot P_{sl} & \left(\frac{1}{1.4}\right) \cdot M_s \\ 0.67 \cdot P_{dlr} & 0.67 \cdot P_{dlp} & 0 \cdot \frac{k}{ft} & 1.0 \cdot (\omega \cdot 0.6 \cdot M_w) \\ 0.9 \cdot P_{dlr} & 0.9 \cdot P_{dlp} & 0 \cdot \frac{k}{ft} & \left(\frac{1}{1.4}\right) \cdot M_s \end{bmatrix}$$

$i := 0 .. \text{rows}(x) - 1$

Define Panel Reinforcing

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

$NB := 8$ — Number of Vertical Bars

$rebar\_location := \text{EF}$ — Steel Layer: Each Face (EF) or Centerline (CL)

$stirrup\_size := \#3$ — Stirrup Bar Size

$A_b := A_{bar\_size}$ — Vertical Bar Area

$diam_b := D_{bar\_size}$ — Vertical Bar Diameter

$diam_{str} := D_{stirrup\_size}$ — Stirrup Bar Diameter

$d_{ef} := t - \dfrac{diam_b}{2} - diam_{str} - clr - r_v$ — Steel Depth If Each Face

$d_{cl} := \dfrac{t}{2} - r_v$ — Steel Depth If Centerline

$d := \text{if}\left(rebar\_location = 0, d_{ef}, d_{cl}\right)$ — Steel Depth — $d = 7.31\ in$

$S := \dfrac{L - 1.5\ in}{NB - 1}$ — Vertical Bar Spacing OC

$flag := \text{if}\left(S \geq 3\ in, \text{"OK"}, \text{"Revise"}\right)$ — $S = 3.2\ in$ — $flag = \text{"OK"}$

$A_s := \dfrac{NB \cdot A_b}{L}$ — Vertical Area of Steel

Check Axial Capacity

(ACI 318-19 11.8.1.1(d))

$$P_{u_i} := \left\| \begin{array}{l} P_u \leftarrow 0 \\ \left(x_{i,0} + x_{i,1} + x_{i,2}\right) \end{array} \right.$$

$P_a := 0.06 \cdot f_c \cdot t$

$$flag_i := \text{if}\left(\frac{P_{u_i}}{P_a} \leq 1, \text{"OK"}, \text{"Revise"}\right)$$

$$\frac{P_u}{P_a} = \begin{bmatrix} 0.5 \\ 0.4 \\ 0.4 \\ 0.3 \\ 0.3 \end{bmatrix} \qquad flag = \begin{bmatrix} \text{"OK"} \\ \text{"OK"} \\ \text{"OK"} \\ \text{"OK"} \\ \text{"OK"} \end{bmatrix}$$

$compression\_check := flag$

Determine Moment Capacity

$$\beta_1 := 0.85 - 0.05 \cdot \left(\frac{f_c - 4000 \cdot psi}{1000 \cdot psi}\right)$$

$P_u := P_u \cdot ft \qquad A_s := A_s \cdot ft$

Concrete Panel Out-of-Plane Analysis

$$X_i := \frac{P_{u_i} + A_s \cdot f_y}{0.85 \cdot f_c \cdot ft} \qquad M_{n_i} := \left(A_s \cdot f_y + P_{u_i}\right) \cdot \left(d - \frac{X_i}{2}\right) - P_{u_i} \cdot \left(d - \frac{t}{2}\right)$$

$$\phi M_{n_i} := 0.9 \cdot M_{n_i}$$ Factored Moment Capacity $$\phi M_n = \begin{bmatrix} 38.5 \\ 38.1 \\ 38.1 \\ 37.3 \\ 37.3 \end{bmatrix} k \cdot ft$$

Check for Tension-Controlled Section

(ACI 318-19 11.8.1.1(b))

$$C_i := \frac{X_i}{\beta_1} \qquad flag_i := \text{if}\left(\frac{C_i}{d} < 0.375, \text{“OK”}, \text{“Revise”}\right)$$

$$tension\_controlled := flag$$

$$\frac{C}{d} = \begin{bmatrix} 0.346 \\ 0.338 \\ 0.338 \\ 0.324 \\ 0.324 \end{bmatrix} \qquad flag = \begin{bmatrix} \text{“OK”} \\ \text{“OK”} \\ \text{“OK”} \\ \text{“OK”} \\ \text{“OK”} \end{bmatrix}$$

Determine Deflection Limits

$$\Delta_{max} := \frac{ht}{150}$$ Allowable Panel Deflection

$$ht' := \text{if}\left(DockHigh = 0, ht, 0.8 \cdot ht\right) = 29.4\ ft$$ Adjust Span per Dock High Condition

$$E_c := 57000 \cdot \sqrt{\frac{f_c}{psi}} \cdot psi \qquad n := \frac{29000 \cdot ksi}{E_c} \qquad A_{se_i} := A_s + \frac{P_{u_i} \cdot t}{2 \cdot f_y \cdot d} \qquad C_i := \frac{X_i}{0.85}$$

$$f_r := 7.5 \cdot \sqrt{\frac{f_c}{psi}} \cdot psi \qquad I_g := \frac{(1\ ft) \cdot t^3}{12} \qquad I_{cr_i} := \frac{12 \cdot in \cdot \left(C_i\right)^3}{3} + n \cdot A_{se_i} \cdot \left(d - C_i\right)^2$$

$$M_{cr} := \frac{2 \cdot f_r \cdot I_g}{t} \qquad \Delta_{cr} := \frac{5 \cdot M_{cr} \cdot ht'^2}{48 \cdot E_c \cdot I_g} \qquad \Delta_{n_i} := \frac{5 \cdot M_{n_i} \cdot ht'^2}{48 \cdot E_c \cdot I_{cr_i}}$$

Check Moment Capacity

(ACI 318-19 11.8.3.1)

$$M_{ua_i} := \left(x_{i,3} + \left(x_{i,0} + x_{i,2}\right) \cdot \frac{e}{2}\right) \cdot ft \qquad M_{u_i} := \frac{M_{ua_i}}{1 - \frac{5 \cdot P_{u_i} \cdot ht'^2}{0.75 \cdot 48 \cdot E_c \cdot I_{cr_i}}}$$

$$flag_i := \text{if}\left(\frac{M_{u_i}}{\phi M_{n_i}} \le 1, \text{“OK”}, \text{“Revise”}\right)$$

$$capacity\_check := flag$$

$$\frac{M_u}{\phi M_n} = \begin{bmatrix} 0.17 \\ 0.27 \\ 0.66 \\ 0.24 \\ 0.62 \end{bmatrix} \qquad flag = \begin{bmatrix} \text{“OK”} \\ \text{“OK”} \\ \text{“OK”} \\ \text{“OK”} \\ \text{“OK”} \end{bmatrix}$$

Check Deflection
(ACI 318-19 11.8.4)

$M_{a_i} := \left(y_{i,3} + \left(y_{i,0} + y_{i,2}\right) \cdot \frac{e}{2}\right) ft$ $\quad P_{s_i} := \left(y_{i,0} + y_{i,1} + y_{i,2}\right) \cdot ft$

Iterate Service Deflection

$\Delta_{s_i} := 0 \cdot in$ Initial Guess

$\Delta_s := \Delta f_s \left(M_a, P_s, \Delta_s, M_{cr}, \Delta_{cr}, M_n, \Delta_n\right)$ Function iterates service deflections per ACI 318-19 *Sec 11.8.4*

$flag_i := \text{if}\left(\frac{\Delta_{s_i}}{\Delta_{max}} \le 1, \text{"OK"}, \text{"Revise"}\right)$

$\frac{\Delta_s}{\Delta_{max}} = \begin{bmatrix} 0.06 \\ 0.12 \\ 0.65 \\ 0.09 \\ 0.61 \end{bmatrix}$ $\quad flag = \begin{bmatrix} \text{"OK"} \\ \text{"OK"} \\ \text{"OK"} \\ \text{"OK"} \\ \text{"OK"} \end{bmatrix}$

$deflection\_check := flag$

Results Summary

Panel Thickness $t = 9.5\ in$ **Bar Size** $rebar_{bar\_size} = \text{"\#5"}$

Panel Leg Width $L = 2\ ft$ **Bar Quantity** $NB = 8$

Bar Location $layer = \text{"EF"}$

Concrete Panel Out-of-Plane Analysis

Panel w/ Dock Door (P5) - 4' Center Leg 9.5" THK w/ (14) #5 VERT EF

Panel Geometry

$t := 9.5\ in$	Panel Thickness
$ht := 36\ ft + 9\ in$	Roof Height (Panel Span)
$h_p := 0$	Parapet Height
$DockHigh :=$ Yes	Dock High Span Reduction
$w := 26 \cdot ft$	Panel Width
$L := 4\ ft$	Panel Leg Width
$ht_o := 10\ ft$	Panel Opening Height
$w_o := 9\ ft$	Panel Opening Width
$e := \frac{t}{2} + 3 \cdot in = 7.75\ in$	Roof Load Eccentricity
$h_T := ht + h_p = 36.75\ ft$	Total Panel Height

Panel Load Factors

$x_g := 1 + \frac{w_o}{L} = 3.3$	Gravity Factor
$x_{OOP} := 1 + \frac{w_o}{L} = 3.3$	OOP Factor

Panel Wind Loading Per ASCE 7-16 Chapter 30 - Part 1 Method

$GCpi := 0.18$ Coefficient per Table 26.13-1

$$A_{eff} := \frac{1}{3}\ ht^2 = 450.2\ ft^2$$

$Slope :=$ Y

If roof slope is <10 degrees,
GCp can be reduced by 10%

Concrete Panel Out-of-Plane Analysis

$$GCp_{poscalc} := \left(1.0 - \frac{\ln\left(\frac{A_{eff}}{ft^2}\right) - \ln(10)}{\ln(500) - \ln(10)}(1 - 0.7)\right)(1 - Slope)$$

Positive pressure coefficient (zones 4 and 5)

$$GCp_{negcalc} := \left(1.4 - \frac{\ln\left(\frac{A_{eff}}{ft^2}\right) - \ln(10)}{\ln(500) - \ln(10)}(1.4 - 0.8)\right)(1 - Slope)(-1)$$

Negative pressure coefficient (zone 5)

$$GCp_{pos} := \begin{array}{l} \text{if } A_{eff} > 500\ ft^2 \\ \quad \| 0.7\ (1 - Slope) \\ \text{else if } A_{eff} < 10\ ft^2 \\ \quad \| 1.0\ (1 - Slope) \\ \text{else} \\ \quad \| GCp_{poscalc} \end{array}$$

Check against upper and lower bound values for CGp based on Aeff

$$GCp_{neg} := \begin{array}{l} \text{if } A_{eff} > 500\ ft^2 \\ \quad \| -0.8\ (1 - Slope) \\ \text{else if } A_{eff} < 10\ ft^2 \\ \quad \| -1.4\ (1 - Slope) \\ \text{else} \\ \quad \| GCp_{negcalc} \end{array}$$

$$p_1 := q_h \cdot (GCp_{pos} - GCpi)$$

$$p_2 := q_h \cdot (GCp_{neg} - GCpi)$$

$$p_3 := q_h \cdot (GCp_{pos} + GCpi)$$

$$p_4 := q_h \cdot (GCp_{neg} + GCpi)$$

$$p := \max(|p_1|, |p_2|, |p_3|, |p_4|) = 13.9\ psf$$

Panel Loading

$w_{rf} := 60\ ft$ — Joist Span/Spacing Adjacent to Panel

$load_{drift} := 0\ psf$ — Drift Load at Panel

$w_{drift} := 0.0\ ft$ — Drift Length

$$P_{drift} := \text{if } w_{rf} \geq w_{drift}$$
$$\quad P \leftarrow load_{drift} \cdot w_{drift} \cdot \left(\frac{1}{2} - \frac{w_{drift}}{6\ w_{rf}}\right)$$
$$\text{else if } w_{rf} < w_{drift}$$
$$\quad P \leftarrow load_{drift} \cdot w_{rf} \cdot \left(\frac{1}{2} - \frac{w_{rf}}{6 \cdot w_{drift}}\right)$$

Snow Drift Reaction at Panel $P_{drift} = 0\ \frac{k}{ft}$

$$P_{dlr} := x_g \cdot DL \cdot \frac{w_{rf}}{2}$$

Roof Dead Load $P_{dlr} = 1.5\ \frac{k}{ft}$

$$P_{dlp} := x_g \cdot t \cdot 150\ pcf \cdot \left(\frac{ht}{2} + h_p\right)$$

Panel Dead Load $P_{dlp} = 7.1\ \frac{k}{ft}$

$$P_{sl} := x_g \cdot \left(20\ psf \cdot \frac{w_{rf}}{2}\right)$$

Total Snow Load $P_{sl} = 2\ \frac{k}{ft}$

$$M_w := x_{OOP} \cdot \frac{p \cdot ht^2}{8}$$

Moment Due To Wind $M_w = 7.6\ \frac{k \cdot ft}{ft}$

$$w_1 := F_p \cdot 150\ pcf\ t \cdot x_{OOP}$$

$$w_2 := \left(F_p \cdot 10 \cdot psf \cdot w_o \cdot 0.5 + F_p \cdot 150\ pcf \cdot t \cdot L\right) \cdot \frac{1}{L}$$

$$R_{bot} := \frac{w_2 \cdot ht}{2} + \frac{(w_1 - w_2) \cdot (ht - ht_o)^2}{2 \cdot ht}$$

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$$R_{top} := \left(w_2 \cdot ht_o + w_1 \cdot (ht - ht_o)\right) - R_{bot}$$

$$x_m := \frac{R_{top}}{w_1}$$

$$M_s := R_{top} \cdot x_m - \frac{w_1 \cdot {x_m}^2}{2}$$

Moment Due to Seismic w/ Opening $M_s = 19.9\ \frac{k \cdot ft}{ft}$

<u>Strength Load Combinations (LRFD)</u>

(ASCE 7-16 2.3)

$$x := \begin{bmatrix} 1.2 \cdot P_{dlr} & 1.2 \cdot P_{dlp} & 1.6 \cdot P_{sl} & 0.5 \cdot M_w \\ 1.2 \cdot P_{dlr} & 1.2 \cdot P_{dlp} & 0.5 \cdot P_{sl} & 1.0 \cdot M_w \\ 1.2 \cdot P_{dlr} & 1.2 \cdot P_{dlp} & 0.5 \cdot P_{sl} & 1.0 \cdot M_s \\ 0.9 \cdot P_{dlr} & 0.9 \cdot P_{dlp} & 0 \cdot \frac{k}{ft} & 1.0 \cdot M_w \\ 0.9 \cdot P_{dlr} & 0.9 \cdot P_{dlp} & 0 \cdot \frac{k}{ft} & 1.0 \cdot M_s \end{bmatrix}$$

<u>Service/Deflection Load Combinations (ASD)</u>

(2021 IBC 1605.2)

$$\omega := 1.0$$

$$y := \begin{bmatrix} 1.0 \cdot P_{dlr} & 1.0 \cdot P_{dlp} & 1.0 \cdot P_{sl} & 0.5 \cdot (\omega \cdot 0.6 \cdot M_w) \\ 1.0 \cdot P_{dlr} & 1.0 \cdot P_{dlp} & 0.5 \cdot P_{sl} & 1.0 \cdot (\omega \cdot 0.6 \cdot M_w) \\ 1.0 \cdot P_{dlr} & 1.0 \cdot P_{dlp} & 1.0 \cdot P_{sl} & \left(\frac{1}{1.4}\right) \cdot M_s \\ 0.67 \cdot P_{dlr} & 0.67 \cdot P_{dlp} & 0 \cdot \frac{k}{ft} & 1.0 \cdot (\omega \cdot 0.6 \cdot M_w) \\ 0.9 \cdot P_{dlr} & 0.9 \cdot P_{dlp} & 0 \cdot \frac{k}{ft} & \left(\frac{1}{1.4}\right) \cdot M_s \end{bmatrix}$$

$i := 0 .. \text{rows}(x) - 1$

Define Panel Reinforcing

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

$NB := 14$ — Number of Vertical Bars

$rebar\_location := \text{EF}$ — Steel Layer: Each Face (EF) or Centerline (CL)

$stirrup\_size := \#3$ — Stirrup Bar Size

$A_b := A_{bar\_size}$ — Vertical Bar Area

$diam_b := D_{bar\_size}$ — Vertical Bar Diameter

$diam_{str} := D_{stirrup\_size}$ — Stirrup Bar Diameter

$d_{ef} := t - \frac{diam_b}{2} - diam_{str} - clr - r_v$ — Steel Depth If Each Face

$d_{cl} := \frac{t}{2} - r_v$ — Steel Depth If Centerline

$d := \text{if}\left(rebar\_location = 0, d_{ef}, d_{cl}\right)$ — Steel Depth — $d = 7.31\ in$

$S := \frac{L - 1.5\ in}{NB - 1}$ — Vertical Bar Spacing OC

$flag := \text{if}\left(S \geq 3\ in, \text{“OK”}, \text{“Revise”}\right)$ — $S = 3.6\ in$ — $flag = \text{“OK”}$

$A_s := \frac{NB \cdot A_b}{L} = 1.09\ \frac{in^2}{ft}$ — Vertical Area of Steel

Check Axial Capacity

(ACI 318-19 11.8.1.1(d))

$$P_{u_i} := \left\| \begin{array}{l} P_u \leftarrow 0 \\ \left(x_{i,0} + x_{i,1} + x_{i,2}\right) \end{array} \right.$$

$P_a := 0.06 \cdot f_c \cdot t$

$$flag_i := \text{if}\left(\frac{P_{u_i}}{P_a} \leq 1, \text{“OK”}, \text{“Revise”}\right)$$

$$\frac{P_u}{P_a} = \begin{bmatrix} 0.5 \\ 0.4 \\ 0.4 \\ 0.3 \\ 0.3 \end{bmatrix} \quad flag = \begin{bmatrix} \text{“OK”} \\ \text{“OK”} \\ \text{“OK”} \\ \text{“OK”} \\ \text{“OK”} \end{bmatrix}$$

$compression\_check := flag$

Determine Moment Capacity

$$\beta_1 := 0.85 - 0.05 \cdot \left(\frac{f_c - 4000 \cdot psi}{1000 \cdot psi}\right)$$

$P_u := P_u \cdot ft \quad A_s := A_s \cdot ft$

Concrete Panel Out-of-Plane Analysis

$$X_i := \frac{P_{u_i} + A_s \cdot f_y}{0.85 \cdot f_c \cdot ft}$$

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

$$\phi M_{n_i} := 0.9 \cdot M_{n_i}$$

Factored Moment Capacity

$$\phi M_n = \begin{bmatrix} 34.8 \\ 34.3 \\ 34.3 \\ 33.6 \\ 33.6 \end{bmatrix} k \cdot ft$$

Check for Tension-Controlled Section

(ACI 318-19 11.8.1.1(b))

$$C_i := \frac{X_i}{\beta_1}$$

$$flag_i := \text{if}\left(\frac{C_i}{d} < 0.375, \text{“OK”}, \text{“Revise”}\right)$$

$$tension\_controlled := flag$$

$$\frac{C}{d} = \begin{bmatrix} 0.309 \\ 0.301 \\ 0.301 \\ 0.287 \\ 0.287 \end{bmatrix} \qquad flag = \begin{bmatrix} \text{“OK”} \\ \text{“OK”} \\ \text{“OK”} \\ \text{“OK”} \\ \text{“OK”} \end{bmatrix}$$

Determine Deflection Limits

$$\Delta_{max} := \frac{ht}{150}$$

Allowable Panel Deflection

$$ht' := \text{if}\left(DockHigh = 0, ht, 0.8 \cdot ht\right) = 29.4\ ft$$

Adjust Span per Dock High Condition

$$E_c := 57000 \cdot \sqrt{\frac{f_c}{psi}} \cdot psi \qquad n := \frac{29000 \cdot ksi}{E_c} \qquad A_{se_i} := A_s + \frac{P_{u_i} \cdot t}{2 \cdot f_y \cdot d} \qquad C_i := \frac{X_i}{0.85}$$

$$f_r := 7.5 \cdot \sqrt{\frac{f_c}{psi}} \cdot psi \qquad I_g := \frac{(1\ ft) \cdot t^3}{12} \qquad I_{cr_i} := \frac{12 \cdot in \cdot \left(C_i\right)^3}{3} + n \cdot A_{se_i} \cdot \left(d - C_i\right)^2$$

$$M_{cr} := \frac{2 \cdot f_r \cdot I_g}{t} \qquad \Delta_{cr} := \frac{5 \cdot M_{cr} \cdot ht'^2}{48 \cdot E_c \cdot I_g} \qquad \Delta_{n_i} := \frac{5 \cdot M_{n_i} \cdot ht'^2}{48 \cdot E_c \cdot I_{cr_i}}$$

Check Moment Capacity

(ACI 318-19 11.8.3.1)

$$M_{ua_i} := \left(x_{i,3} + \left(x_{i,0} + x_{i,2}\right) \cdot \frac{e}{2}\right) \cdot ft$$

$$M_{u_i} := \frac{M_{ua_i}}{1 - \frac{5 \cdot P_{u_i} \cdot ht'^2}{0.75 \cdot 48 \cdot E_c \cdot I_{cr_i}}}$$

$$flag_i := \text{if}\left(\frac{M_{u_i}}{\phi M_{n_i}} \le 1, \text{“OK”}, \text{“Revise”}\right)$$

$$capacity\_check := flag$$

$$\frac{M_u}{\phi M_n} = \begin{bmatrix} 0.2 \\ 0.3 \\ 0.74 \\ 0.27 \\ 0.69 \end{bmatrix} \qquad flag = \begin{bmatrix} \text{“OK”} \\ \text{“OK”} \\ \text{“OK”} \\ \text{“OK”} \\ \text{“OK”} \end{bmatrix}$$

Concrete Panel Out-of-Plane Analysis

Check Deflection

(ACI 318-19 11.8.4)

$M_{a_i} := \left(y_{i,3} + \left(y_{i,0} + y_{i,2}\right) \cdot \frac{e}{2}\right) ft$

$P_{s_i} := \left(y_{i,0} + y_{i,1} + y_{i,2}\right) \cdot ft$

Iterate Service Deflection

$\Delta_{s_i} := 0 \cdot in$

Initial Guess

$\Delta_s := \Delta f_s\left(M_a, P_s, \Delta_s, M_{cr}, \Delta_{cr}, M_n, \Delta_n\right)$

Function iterates service deflections per ACI 318-19 *Sec 11.8.4*

$flag_i := \text{if}\left(\frac{\Delta_{s_i}}{\Delta_{max}} \le 1, \text{“OK”}, \text{“Revise”}\right)$

$\frac{\Delta_s}{\Delta_{max}} = \begin{bmatrix} 0.06 \\ 0.12 \\ 0.7 \\ 0.09 \\ 0.66 \end{bmatrix}$

$flag = \begin{bmatrix} \text{“OK”} \\ \text{“OK”} \\ \text{“OK”} \\ \text{“OK”} \\ \text{“OK”} \end{bmatrix}$

$deflection\_check := flag$

Results Summary

Panel Thickness $t = 9.5\ in$

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

Panel Leg Width $L = 4\ ft$

Bar Quantity $NB = 14$

Bar Location $layer = \text{“EF”}$

Concrete Panel Out-of-Plane Analysis

Panel w/ Dock Door + Personnel Door (P7)- 6'-10" Center Leg 9.5" THK w/ (18) #5 VERT EF

Panel Geometry

$t := 9.5\ in$ Panel Thickness

$ht := 39\ ft + 10.5\ in$ Roof Height (Panel Span)

$h_p := 0$ Parapet Height

$DockHigh :=$ Yes Dock High Span Reduction

$w := 26 \cdot ft$ Panel Width

$L := 6\ ft + 10\ in$ Panel Leg Width

$ht_{o1} := 7\ ft + 8\ in$ Panel Opening Height

$w_{o1} := 3\ ft + 4\ in$ Panel Opening Width

$ht_{o2} := 10\ ft$

$w_{o2} := 9\ ft$

$e := \frac{t}{2} + 3 \cdot in = 7.75\ in$ Roof Load Eccentricity

$h_T := ht + h_p = 39.88\ ft$ Total Panel Height

Panel Load Factors

$x_g := 1 + \frac{w_{o1} + w_{o2}}{2\ L} = 1.9$ Gravity Factor

$x_{OOP} := 1 + \frac{w_o + w_{o2}}{2\ L} = 2.3$ OOP Factor

Panel Wind Loading Per ASCE 7-16 Chapter 30 - Part 1 Method

$GCpi := 0.18$ Coefficient per Table 26.13-1

$A_{eff} := \frac{1}{3}\ ht^2 = 530\ ft^2$

$Slope :=$ Y

If roof slope is <10 degrees,
GCp can be reduced by 10%

Concrete Panel Out-of-Plane Analysis

$$GCp_{poscalc} := \left(1.0 - \frac{\ln\left(\frac{A_{eff}}{ft^2}\right) - \ln(10)}{\ln(500) - \ln(10)}(1-0.7)\right)(1 - Slope)$$

Positive pressure coefficient (zones 4 and 5)

$$GCp_{negcalc} := \left(1.4 - \frac{\ln\left(\frac{A_{eff}}{ft^2}\right) - \ln(10)}{\ln(500) - \ln(10)}(1.4-0.8)\right)(1 - Slope)(-1)$$

Negative pressure coefficient (zone 5)

$$GCp_{pos} := \begin{aligned} &\text{if } A_{eff} > 500\ ft^2 \\ &\quad \| \ 0.7\,(1 - Slope) \\ &\text{else if } A_{eff} < 10\ ft^2 \\ &\quad \| \ 1.0\,(1 - Slope) \\ &\text{else} \\ &\quad \| \ GCp_{poscalc} \end{aligned}$$

Check against upper and lower bound values for CGp based on Aeff

$$GCp_{neg} := \begin{aligned} &\text{if } A_{eff} > 500\ ft^2 \\ &\quad \| \ -0.8\,(1 - Slope) \\ &\text{else if } A_{eff} < 10\ ft^2 \\ &\quad \| \ -1.4\,(1 - Slope) \\ &\text{else} \\ &\quad \| \ GCp_{negcalc} \end{aligned}$$

$$p_1 := q_h \cdot (GCp_{pos} - GCpi)$$

$$p_2 := q_h \cdot (GCp_{neg} - GCpi)$$

$$p_3 := q_h \cdot (GCp_{pos} + GCpi)$$

$$p_4 := q_h \cdot (GCp_{neg} + GCpi)$$

$$p := \max\left(|p_1|, |p_2|, |p_3|, |p_4|\right) = 13.6\ psf$$

Panel Loading

$w_{rf} := 60\ ft$ Joist Span/Spacing Adjacent to Panel

$load_{drift} := 0\ psf$ Drift Load at Panel

$w_{drift} := 0.0\ ft$ Drift Length

$$P_{drift} := \text{if } w_{rf} \geq w_{drift} \;\; \left\| P \leftarrow load_{drift} \cdot w_{drift} \cdot \left(\frac{1}{2} - \frac{w_{drift}}{6\ w_{rf}}\right) \right. \;\; \text{else if } w_{rf} < w_{drift} \;\; \left\| P \leftarrow load_{drift} \cdot w_{rf} \cdot \left(\frac{1}{2} - \frac{w_{rf}}{6 \cdot w_{drift}}\right) \right.$$

Snow Drift Reaction at Panel $P_{drift} = 0\ \frac{k}{ft}$

$$P_{dlr} := x_g \cdot DL \cdot \frac{w_{rf}}{2}$$

Roof Dead Load $P_{dlr} = 1.5\ \frac{k}{ft}$

$$P_{dlp} := x_g \cdot t \cdot 150\ pcf \cdot \left(\frac{ht}{2} + h_p\right)$$

Panel Dead Load $P_{dlp} = 4.5\ \frac{k}{ft}$

$$P_{sl} := x_g \cdot \left(20\ psf \cdot \frac{w_{rf}}{2}\right)$$

Total Snow Load $P_{sl} = 1.1\ \frac{k}{ft}$

$$M_w := x_{OOP} \cdot \frac{p \cdot ht^2}{8}$$

Moment Due To Wind $M_w = 6.3\ \frac{k \cdot ft}{ft}$

$$w_1 := F_p \cdot 150\ pcf\ t \cdot x_{OOP}$$

$$w_2 := \left(F_p \cdot 10 \cdot psf \cdot (w_{o1} + w_{o2}) \cdot 0.5 + F_p \cdot 150\ pcf \cdot t \cdot L\right) \cdot \frac{1}{L} = 43.1\ psf$$

$$R_{bot} := \frac{w_2 \cdot ht}{2} + \frac{(w_1 - w_2) \cdot (ht - ht_{o1})^2}{2 \cdot ht} = 1.5\ \frac{k}{ft}$$

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$$R_{top} := \left(w_2 \cdot ht_{o1} + w_1 \cdot (ht - ht_{o1})\right) - R_{bot} = 1.8\ \frac{k}{ft}$$

$$x_m := \frac{R_{top}}{w_1}$$

$$M_s := R_{top} \cdot x_m - \frac{w_1 \cdot {x_m}^2}{2}$$

Moment Due to Seismic w/ Opening $M_s = 17.7\ \frac{k \cdot ft}{ft}$

<u>Strength Load Combinations (LRFD)</u>

(ASCE 7-16 2.3)

$$x := \begin{bmatrix} 1.2 \cdot P_{dlr} & 1.2 \cdot P_{dlp} & 1.6 \cdot P_{sl} & 0.5 \cdot M_w \\ 1.2 \cdot P_{dlr} & 1.2 \cdot P_{dlp} & 0.5 \cdot P_{sl} & 1.0 \cdot M_w \\ 1.2 \cdot P_{dlr} & 1.2 \cdot P_{dlp} & 0.5\ P_{sl} & 1.0 \cdot M_s \\ 0.9 \cdot P_{dlr} & 0.9 \cdot P_{dlp} & 0 \cdot \frac{k}{ft} & 1.0 \cdot M_w \\ 0.9 \cdot P_{dlr} & 0.9 \cdot P_{dlp} & 0 \cdot \frac{k}{ft} & 1.0 \cdot M_s \end{bmatrix}$$

<u>Service/Deflection Load Combinations (ASD)</u>

(2021 IBC 1605.2)

$$\omega := 1.0$$

$$y := \begin{bmatrix} 1.0 \cdot P_{dlr} & 1.0 \cdot P_{dlp} & 1.0 \cdot P_{sl} & 0.5 \cdot (\omega \cdot 0.6 \cdot M_w) \\ 1.0 \cdot P_{dlr} & 1.0 \cdot P_{dlp} & 0.5 \cdot P_{sl} & 1.0 \cdot (\omega \cdot 0.6 \cdot M_w) \\ 1.0 \cdot P_{dlr} & 1.0 \cdot P_{dlp} & 1.0 \cdot P_{sl} & \left(\frac{1}{1.4}\right) \cdot M_s \\ 0.67 \cdot P_{dlr} & 0.67 \cdot P_{dlp} & 0 \cdot \frac{k}{ft} & 1.0 \cdot (\omega \cdot 0.6 \cdot M_w) \end{bmatrix}$$

$i := 0 .. \text{rows}(x) - 1$ $\quad \left[0.9 \cdot P_{dlr} \quad 0.9 \cdot P_{dlp} \quad 0 \cdot \frac{k}{ft} \quad \left(\frac{1}{1.4}\right) \cdot M_s \right]$

Define Panel Reinforcing

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

$NB := 18$ — Number of Vertical Bars

$rebar\_location := \text{EF}$ — Steel Layer: Each Face (EF) or Centerline (CL)

$stirrup\_size := \text{NA}$ — Stirrup Bar Size

$A_b := A_{bar\_size}$ — Vertical Bar Area

$diam_b := D_{bar\_size}$ — Vertical Bar Diameter

$diam_{str} := D_{stirrup\_size}$ — Stirrup Bar Diameter

$d_{ef} := t - \frac{diam_b}{2} - diam_{str} - clr - r_v$ — Steel Depth If Each Face

$d_{cl} := \frac{t}{2} - r_v$ — Steel Depth If Centerline

$d := \text{if}\left(rebar\_location = 0, d_{ef}, d_{cl}\right)$ — Steel Depth $\quad d = 7.69\ in$

$S := \frac{L - 1.5\ in}{NB - 1}$ — Vertical Bar Spacing OC

$flag := \text{if}\left(S \geq 3\ in, \text{"OK"}, \text{"Revise"}\right)$ $\quad S = 4.7\ in \quad flag = \text{"OK"}$

$A_s := \frac{NB \cdot A_b}{L}$ — Vertical Area of Steel

Check Axial Capacity

(ACI 318-19 11.8.1.1(d))

$$P_{u_i} := \left\| \begin{array}{l} P_u \leftarrow 0 \\ \left(x_{i,0} + x_{i,1} + x_{i,2}\right) \end{array} \right.$$

$P_a := 0.06 \cdot f_c \cdot t$

$$flag_i := \text{if}\left(\frac{P_{u_i}}{P_a} \leq 1, \text{"OK"}, \text{"Revise"}\right) \qquad \frac{P_u}{P_a} = \begin{bmatrix} 0.3 \\ 0.3 \\ 0.3 \\ 0.2 \\ 0.2 \end{bmatrix} \qquad flag = \begin{bmatrix} \text{"OK"} \\ \text{"OK"} \\ \text{"OK"} \\ \text{"OK"} \\ \text{"OK"} \end{bmatrix}$$

$compression\_check := flag$

Determine Moment Capacity

$$\beta_1 := 0.85 - 0.05 \cdot \left(\frac{f_c - 4000 \cdot psi}{1000 \cdot psi}\right) \qquad P_u := P_u \cdot ft \qquad A_s := A_s \cdot ft$$

Concrete Panel Out-of-Plane Analysis

$$X_i := \frac{P_{u_i} + A_s \cdot f_y}{0.85 \cdot f_c \cdot ft} \qquad M_{n_i} := \left(A_s \cdot f_y + P_{u_i}\right) \cdot \left(d - \frac{X_i}{2}\right) - P_{u_i} \cdot \left(d - \frac{t}{2}\right)$$

$$\phi M_{n_i} := 0.9 \cdot M_{n_i}$$ Factored Moment Capacity $$\phi M_n = \begin{bmatrix} 28.2 \\ 27.9 \\ 27.9 \\ 27.3 \\ 27.3 \end{bmatrix} k \cdot ft$$

Check for Tension-Controlled Section

(ACI 318-19 11.8.1.1(b))

$$C_i := \frac{X_i}{\beta_1} \qquad flag_i := \text{if}\left(\frac{C_i}{d} < 0.375, \text{"OK"}, \text{"Revise"}\right)$$

$$tension\_controlled := flag$$

$$\frac{C}{d} = \begin{bmatrix} 0.215 \\ 0.21 \\ 0.21 \\ 0.202 \\ 0.202 \end{bmatrix} \qquad flag = \begin{bmatrix} \text{"OK"} \\ \text{"OK"} \\ \text{"OK"} \\ \text{"OK"} \\ \text{"OK"} \end{bmatrix}$$

Determine Deflection Limits

$$\Delta_{max} := \frac{ht}{150}$$ Allowable Panel Deflection

$$ht' := \text{if}\left(DockHigh = 0, ht, 0.8 \cdot ht\right) = 31.9\ ft$$ Adjust Span per Dock High Condition

$$E_c := 57000 \cdot \sqrt{\frac{f_c}{psi}} \cdot psi \qquad n := \frac{29000 \cdot ksi}{E_c} \qquad A_{se_i} := A_s + \frac{P_{u_i} \cdot t}{2 \cdot f_y \cdot d} \qquad C_i := \frac{X_i}{0.85}$$

$$f_r := 7.5 \cdot \sqrt{\frac{f_c}{psi}} \cdot psi \qquad I_g := \frac{(1\ ft) \cdot t^3}{12} \qquad I_{cr_i} := \frac{12 \cdot in \cdot \left(C_i\right)^3}{3} + n \cdot A_{se_i} \cdot \left(d - C_i\right)^2$$

$$M_{cr} := \frac{2 \cdot f_r \cdot I_g}{t} \qquad \Delta_{cr} := \frac{5 \cdot M_{cr} \cdot ht'^2}{48 \cdot E_c \cdot I_g} \qquad \Delta_{n_i} := \frac{5 \cdot M_{n_i} \cdot ht'^2}{48 \cdot E_c \cdot I_{cr_i}}$$

Check Moment Capacity

(ACI 318-19 11.8.3.1)

$$M_{ua_i} := \left(x_{i,3} + \left(x_{i,0} + x_{i,2}\right) \cdot \frac{e}{2}\right) \cdot ft \qquad M_{u_i} := \frac{M_{ua_i}}{1 - \dfrac{5 \cdot P_{u_i} \cdot ht'^2}{0.75 \cdot 48 \cdot E_c \cdot I_{cr_i}}}$$

$$flag_i := \text{if}\left(\frac{M_{u_i}}{\phi M_{n_i}} \le 1, \text{"OK"}, \text{"Revise"}\right)$$

$$capacity\_check := flag$$

$$\frac{M_u}{\phi M_n} = \begin{bmatrix} 0.17 \\ 0.28 \\ 0.76 \\ 0.27 \\ 0.73 \end{bmatrix} \qquad flag = \begin{bmatrix} \text{"OK"} \\ \text{"OK"} \\ \text{"OK"} \\ \text{"OK"} \\ \text{"OK"} \end{bmatrix}$$

Check Deflection

(ACI 318-19 11.8.4)

$M_{a_i} := \left(y_{i,3} + \left(y_{i,0} + y_{i,2}\right) \cdot \frac{e}{2}\right) ft$ $P_{s_i} := \left(y_{i,0} + y_{i,1} + y_{i,2}\right) \cdot ft$

Iterate Service Deflection

$\Delta_{s_i} := 0 \cdot in$ Initial Guess

$\Delta_s := \Delta f_s \left(M_a, P_s, \Delta_s, M_{cr}, \Delta_{cr}, M_n, \Delta_n\right)$ Function iterates service deflections per ACI 318-19 *Sec 11.8.4*

$flag_i := \text{if}\left(\frac{\Delta_{s_i}}{\Delta_{max}} \le 1, \text{“OK”}, \text{“Revise”}\right)$

$\frac{\Delta_s}{\Delta_{max}} = \begin{bmatrix} 0 \\ 0.1 \\ 0.7 \\ 0.1 \\ 0.6 \end{bmatrix}$ $flag = \begin{bmatrix} \text{“OK”} \\ \text{“OK”} \\ \text{“OK”} \\ \text{“OK”} \\ \text{“OK”} \end{bmatrix}$

$deflection\_check := flag$

Results Summary

Panel Thickness	$t = 9.5\ in$	**Bar Size**	$rebar_{bar\_size} = \text{“\#5”}$
Panel Leg Width	$L = 6.8\ ft$	**Bar Quantity**	$NB = 18$
		Bar Location	$layer = \text{“EF”}$

Concrete Panel Out-of-Plane Analysis

Panel w/ Dock Door + Personnel Door (P7)- 4'-10" Leg 9.5" THK w/ (10) #5 VERT EF

Panel Geometry

$t := 9.5\ in$ Panel Thickness

$ht := 39\ ft + 10.5\ in$ Roof Height (Panel Span)

$h_p := 0$ Parapet Height

$DockHigh := \text{Yes}$ Dock High Span Reduction

$w := 26 \cdot ft$ Panel Width

$L := 4\ ft + 10\ in$ Panel Leg Width

$ht_o := 7\ ft + 8\ in$ Panel Opening Height

$w_o := 3\ ft + 4\ in$ Panel Opening Width

$e := \frac{t}{2} + 3 \cdot in = 7.75\ in$ Roof Load Eccentricity

$h_T := ht + h_p = 39.88\ ft$ Total Panel Height

Panel Load Factors

$x_g := 1 + \frac{w_o}{2 \cdot L} = 1.3$ Gravity Factor

$x_{OOP} := 1 + \frac{w_o}{2 \cdot L} = 1.3$ OOP Factor

Panel Wind Loading Per ASCE 7-16 Chapter 30 - Part 1 Method

$GCpi := 0.18$ Coefficient per Table 26.13-1

$A_{eff} := \frac{1}{3}\ ht^2 = 530\ ft^2$

$Slope := \text{Y}$

If roof slope is <10 degrees, GCp can be reduced by 10%

$$GCp_{poscalc} := \left(1.0 - \frac{\ln\left(\frac{A_{eff}}{ft^2}\right) - \ln(10)}{\ln(500) - \ln(10)} (1 - 0.7)\right) (1 - Slope)$$

Positive pressure coefficient (zones 4 and 5)

$$GCp_{negcalc} := \left(1.4 - \frac{\ln\left(\frac{A_{eff}}{ft^2}\right) - \ln(10)}{\ln(500) - \ln(10)} (1.4 - 0.8)\right) (1 - Slope) (-1)$$

Negative pressure coefficient (zone 5)

$$GCp_{pos} := \begin{cases} 0.7 (1 - Slope) & \text{if } A_{eff} > 500\ ft^2 \\ 1.0 (1 - Slope) & \text{else if } A_{eff} < 10\ ft^2 \\ GCp_{poscalc} & \text{else} \end{cases}$$

$$GCp_{neg} := \begin{cases} -0.8 (1 - Slope) & \text{if } A_{eff} > 500\ ft^2 \\ -1.4 (1 - Slope) & \text{else if } A_{eff} < 10\ ft^2 \\ GCp_{negcalc} & \text{else} \end{cases}$$

Check against upper and lower bound values for CGp based on Aeff

$$p_1 := q_h \cdot (GCp_{pos} - GCpi)$$

$$p_2 := q_h \cdot (GCp_{neg} - GCpi)$$

$$p_3 := q_h \cdot (GCp_{pos} + GCpi)$$

$$p_4 := q_h \cdot (GCp_{neg} + GCpi)$$

$$p := \max(|p_1|, |p_2|, |p_3|, |p_4|) = 13.6\ psf$$

Panel Loading

$w_{rf} := 60\ ft$ Joist Span/Spacing Adjacent to Panel

$load_{drift} := 0\ psf$ Drift Load at Panel

$w_{drift} := 0.0\ ft$ Drift Length

Concrete Panel Out-of-Plane Analysis

$$P_{drift} := \text{if } w_{rf} \geq w_{drift}$$
$$\quad P \leftarrow load_{drift} \cdot w_{drift} \cdot \left(\frac{1}{2} - \frac{w_{drift}}{6\ w_{rf}}\right)$$
$$\text{else if } w_{rf} < w_{drift}$$
$$\quad P \leftarrow load_{drift} \cdot w_{rf} \cdot \left(\frac{1}{2} - \frac{w_{rf}}{6 \cdot w_{drift}}\right)$$

Snow Drift Reaction at Panel $\quad P_{drift} = 0\ \frac{k}{ft}$

$$P_{dlr} := x_g \cdot DL \cdot \frac{w_{rf}}{2}$$

Roof Dead Load $\quad P_{dlr} = 0.9\ \frac{k}{ft}$

$$P_{dlp} := x_g \cdot t \cdot 150\ pcf \cdot \left(\frac{ht}{2} + h_p\right)$$

Panel Dead Load $\quad P_{dlp} = 3.2\ \frac{k}{ft}$

$$P_{sl} := x_g \cdot \left(20\ psf \cdot \frac{w_{rf}}{2}\right)$$

Total Snow Load $\quad P_{sl} = 0.8\ \frac{k}{ft}$

$$M_w := x_{OOP} \cdot \frac{p \cdot ht^2}{8}$$

Moment Due To Wind $\quad M_w = 3.6\ \frac{k \cdot ft}{ft}$

$$w_1 := F_p \cdot 150\ pcf\ t \cdot x_{OOP}$$

$$w_2 := \left(F_p \cdot 10 \cdot psf \cdot w_o \cdot 0.5 + F_p \cdot 150\ pcf \cdot t \cdot L\right) \cdot \frac{1}{L}$$

$$R_{bot} := \frac{w_2 \cdot ht}{2} + \frac{\left(w_1 - w_2\right) \cdot \left(ht - ht_o\right)^2}{2 \cdot ht}$$

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$$R_{top} := \left(w_2 \cdot ht_o + w_1 \cdot \left(ht - ht_o\right)\right) - R_{bot}$$

$$x_m := \frac{R_{top}}{w_1}$$

$$M_s := R_{top} \cdot x_m - \frac{w_1 \cdot {x_m}^2}{2}$$

Moment Due to Seismic w/ Opening $\quad M_s = 10.5\ \frac{k \cdot ft}{ft}$

Strength Load Combinations (LRFD)

(ASCE 7-16 2.3)

$$x := \begin{bmatrix} 1.2 \cdot P_{dlr} & 1.2 \cdot P_{dlp} & 1.6 \cdot P_{sl} & 0.5 \cdot M_w \\ 1.2 \cdot P_{dlr} & 1.2 \cdot P_{dlp} & 0.5 \cdot P_{sl} & 1.0 \cdot M_w \\ 1.2 \cdot P_{dlr} & 1.2 \cdot P_{dlp} & 0.5 \cdot P_{sl} & 1.0 \cdot M_s \\ 0.9 \cdot P_{dlr} & 0.9 \cdot P_{dlp} & 0 \cdot \frac{k}{ft} & 1.0 \cdot M_w \\ 0.9 \cdot P_{dlr} & 0.9 \cdot P_{dlp} & 0 \cdot \frac{k}{ft} & 1.0 \cdot M_s \end{bmatrix}$$

Service/Deflection Load Combinations (ASD)

(2021 IBC 1605.2)

$$\omega := 1.0$$

$$y := \begin{bmatrix} 1.0 \cdot P_{dlr} & 1.0 \cdot P_{dlp} & 1.0 \cdot P_{sl} & 0.5 \cdot \left(\omega \cdot 0.6 \cdot M_w\right) \\ 1.0 \cdot P_{dlr} & 1.0 \cdot P_{dlp} & 0.5 \cdot P_{sl} & 1.0 \cdot \left(\omega \cdot 0.6 \cdot M_w\right) \\ 1.0 \cdot P_{dlr} & 1.0 \cdot P_{dlp} & 1.0 \cdot P_{sl} & \left(\frac{1}{1.4}\right) \cdot M_s \\ 0.67 \cdot P_{dlr} & 0.67 \cdot P_{dlp} & 0 \cdot \frac{k}{ft} & 1.0 \cdot \left(\omega \cdot 0.6 \cdot M_w\right) \\ 0.9 \cdot P_{dlr} & 0.9 \cdot P_{dlp} & 0 \cdot \frac{k}{ft} & \left(\frac{1}{\ldots}\right) \cdot M_{\ldots} \end{bmatrix}$$

$i := 0 .. \text{rows}(x) - 1$

Define Panel Reinforcing

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

$NB := 10$ Number of Vertical Bars

$rebar\_location := \text{EF}$ Steel Layer: Each Face (EF) or Centerline (CL)

$stirrup\_size := \#3$ Stirrup Bar Size

$A_b := A_{bar\_size}$ Vertical Bar Area

$diam_b := D_{bar\_size}$ Vertical Bar Diameter

$diam_{str} := D_{stirrup\_size}$ Stirrup Bar Diameter

$d_{ef} := t - \frac{diam_b}{2} - diam_{str} - clr - r_v$ Steel Depth If Each Face

$d_{cl} := \frac{t}{2} - r_v$ Steel Depth If Centerline

$d := \text{if}\left(rebar\_location = 0, d_{ef}, d_{cl}\right)$ Steel Depth $d = 7.31\ in$

$S := \frac{L - 1.5\ in}{NB - 1}$ Vertical Bar Spacing OC

$flag := \text{if}\left(S \geq 3\ in, \text{“OK”}, \text{“Revise”}\right)$ $S = 6.3\ in$ $flag = \text{“OK”}$

$A_s := \frac{NB \cdot A_b}{L}$ Vertical Area of Steel

Check Axial Capacity

(ACI 318-19 11.8.1.1(d))

$$P_{u_i} := \left\| \begin{array}{l} P_u \leftarrow 0 \\ \left(x_{i,0} + x_{i,1} + x_{i,2}\right) \end{array} \right.$$

$P_a := 0.06 \cdot f_c \cdot t$

$$flag_i := \text{if}\left(\frac{P_{u_i}}{P_a} \leq 1, \text{“OK”}, \text{“Revise”}\right)$$

$$\frac{P_u}{P_a} = \begin{bmatrix} 0.2 \\ 0.2 \\ 0.2 \\ 0.1 \\ 0.1 \end{bmatrix} \qquad flag = \begin{bmatrix} \text{“OK”} \\ \text{“OK”} \\ \text{“OK”} \\ \text{“OK”} \\ \text{“OK”} \end{bmatrix}$$

$compression\_check := flag$

Determine Moment Capacity

$$\beta_1 := 0.85 - 0.05 \cdot \left(\frac{f_c - 4000 \cdot psi}{1000 \cdot psi}\right) = 0.85$$

$P_u := P_u \cdot ft$ $A_s := A_s \cdot ft$

Concrete Panel Out-of-Plane Analysis

$$X_i := \frac{P_{u_i} + A_s \cdot f_y}{0.85 \cdot f_c \cdot ft} \qquad M_{n_i} := \left(A_s \cdot f_y + P_{u_i}\right) \cdot \left(d - \frac{X_i}{2}\right) - P_{u_i} \cdot \left(d - \frac{t}{2}\right)$$

$$\phi M_{n_i} := 0.9 \cdot M_{n_i}$$ Factored Moment Capacity $$\phi M_n = \begin{bmatrix} 21.4 \\ 21.1 \\ 21.1 \\ 20.7 \\ 20.7 \end{bmatrix} k \cdot ft$$

Check for Tension-Controlled Section

(ACI 318-19 11.8.1.1(b))

$$C_i := \frac{X_i}{\beta_1} \qquad flag_i := \text{if}\left(\frac{C_i}{d} < 0.375, \text{"OK"}, \text{"Revise"}\right)$$

$$\frac{C}{d} = \begin{bmatrix} 0.175 \\ 0.171 \\ 0.171 \\ 0.165 \\ 0.165 \end{bmatrix} \qquad flag = \begin{bmatrix} \text{"OK"} \\ \text{"OK"} \\ \text{"OK"} \\ \text{"OK"} \\ \text{"OK"} \end{bmatrix}$$

$$tension\_controlled := flag$$

Determine Deflection Limits

$$\Delta_{max} := \frac{ht}{150}$$ Allowable Panel Deflection

$$ht' := \text{if}\left(DockHigh = 0, ht, 0.8 \cdot ht\right) = 31.9\ ft$$ Adjust Span per Dock High Condition

$$E_c := 57000 \cdot \sqrt{\frac{f_c}{psi}} \cdot psi \qquad n := \frac{29000 \cdot ksi}{E_c} \qquad A_{se_i} := A_s + \frac{P_{u_i} \cdot t}{2 \cdot f_y \cdot d} \qquad C_i := \frac{X_i}{0.85}$$

$$f_r := 7.5 \cdot \sqrt{\frac{f_c}{psi}} \cdot psi \qquad I_g := \frac{(1\ ft) \cdot t^3}{12} \qquad I_{cr_i} := \frac{12 \cdot in \cdot \left(C_i\right)^3}{3} + n \cdot A_{se_i} \cdot \left(d - C_i\right)^2$$

$$M_{cr} := \frac{2 \cdot f_r \cdot I_g}{t} \qquad \Delta_{cr} := \frac{5 \cdot M_{cr} \cdot ht'^2}{48 \cdot E_c \cdot I_g} \qquad \Delta_{n_i} := \frac{5 \cdot M_{n_i} \cdot ht'^2}{48 \cdot E_c \cdot I_{cr_i}}$$

Check Moment Capacity

(ACI 318-19 11.8.3.1)

$$M_{ua_i} := \left(x_{i,3} + \left(x_{i,0} + x_{i,2}\right) \cdot \frac{e}{2}\right) \cdot ft \qquad M_{u_i} := \frac{M_{ua_i}}{1 - \frac{5 \cdot P_{u_i} \cdot ht'^2}{0.75 \cdot 48 \cdot E_c \cdot I_{cr_i}}}$$

$$flag_i := \text{if}\left(\frac{M_{u_i}}{\phi M_{n_i}} \le 1, \text{"OK"}, \text{"Revise"}\right)$$

$$\frac{M_u}{\phi M_n} = \begin{bmatrix} 0.14 \\ 0.22 \\ 0.59 \\ 0.2 \\ 0.57 \end{bmatrix} \qquad flag = \begin{bmatrix} \text{"OK"} \\ \text{"OK"} \\ \text{"OK"} \\ \text{"OK"} \\ \text{"OK"} \end{bmatrix}$$

$$capacity\_check := flag$$

Check Deflection

(ACI 318-19 11.8.4)

$M_{a_i} := \left(y_{i,3} + \left(y_{i,0} + y_{i,2}\right) \cdot \frac{e}{2}\right) ft$

$P_{s_i} := \left(y_{i,0} + y_{i,1} + y_{i,2}\right) \cdot ft$

Iterate Service Deflection

$\Delta_{s_i} := 0 \cdot in$ Initial Guess

$\Delta_s := \Delta f_s \left(M_a, P_s, \Delta_s, M_{cr}, \Delta_{cr}, M_n, \Delta_n\right)$ Function iterates service deflections per ACI 318-19 *Sec 11.8.4*

$flag_i := \text{if}\left(\frac{\Delta_{s_i}}{\Delta_{max}} \le 1, \text{"OK"}, \text{"Revise"}\right)$

$\frac{\Delta_s}{\Delta_{max}} = \begin{bmatrix} 0 \\ 0 \\ 0.4 \\ 0 \\ 0.4 \end{bmatrix}$

$flag = \begin{bmatrix} \text{"OK"} \\ \text{"OK"} \\ \text{"OK"} \\ \text{"OK"} \\ \text{"OK"} \end{bmatrix}$

$deflection\_check := flag$

Results Summary

Panel Thickness $t = 9.5\ in$

Bar Size $rebar_{bar\_size} = \text{"\#5"}$

Panel Leg Width $L = 4.8\ ft$

Bar Quantity $NB = 10$

Bar Location $layer = \text{"EF"}$

Concrete Panel Out-of-Plane Analysis

Panel w/ Dock Door + Personnel Door (P19) 7'-6" Leg 9.5" THK w/ (12) #5 VERT EF

Panel Geometry

$t := 9.5\ in$	Panel Thickness
$ht := 35\ ft + 6\ in$	Roof Height (Panel Span)
$h_p := 8\ ft$	Parapet Height
$DockHigh :=$ No	Dock High Span Reduction
$w := 28 \cdot ft$	Panel Width
$L := 7\ ft + 6\ in$	Panel Leg Width
$ht_o := 7\ ft + 4\ in$	Panel Opening Height
$w_o := 6\ ft + 4\ in$	Panel Opening Width
$e := \frac{t}{2} + 3 \cdot in = 7.75\ in$	Roof Load Eccentricity
$h_T := ht + h_p = 43.5\ ft$	Total Panel Height

Panel Load Factors

$x_g := 1 + \frac{w_o}{2 \cdot L} = 1.4$	Gravity Factor
$x_{OOP} := 1 + \frac{w_o}{2 \cdot L} = 1.4$	OOP Factor

Panel Wind Loading Per ASCE 7-16 Chapter 30 - Part 1 Method

$GCpi := 0.18$ Coefficient per Table 26.13-1

$$A_{eff} := \frac{1}{3}\ ht^2 = 420.1\ ft^2$$

$Slope :=$ Y

If roof slope is <10 degrees, GCp can be reduced by 10%

Concrete Panel Out-of-Plane Analysis

$$GCp_{poscalc} := \left(1.0 - \frac{\ln\left(\frac{A_{eff}}{ft^2}\right) - \ln(10)}{\ln(500) - \ln(10)} (1 - 0.7)\right) (1 - Slope)$$

Positive pressure coefficient (zones 4 and 5)

$$GCp_{negcalc} := \left(1.4 - \frac{\ln\left(\frac{A_{eff}}{ft^2}\right) - \ln(10)}{\ln(500) - \ln(10)} (1.4 - 0.8)\right) (1 - Slope)\ (-1)$$

Negative pressure coefficient (zone 5)

$$GCp_{pos} := \text{if } A_{eff} > 500\ ft^2 \;\|\; 0.7\ (1 - Slope) \;\; \text{else if } A_{eff} < 10\ ft^2 \;\|\; 1.0\ (1 - Slope) \;\; \text{else } \;\|\; GCp_{poscalc}$$

Check against upper and lower bound values for CGp based on Aeff

$$GCp_{neg} := \text{if } A_{eff} > 500\ ft^2 \;\|\; -0.8\ (1 - Slope) \;\; \text{else if } A_{eff} < 10\ ft^2 \;\|\; -1.4\ (1 - Slope) \;\; \text{else } \;\|\; GCp_{negcalc}$$

$$p_1 := q_h \cdot (GCp_{pos} - GCpi)$$

$$p_2 := q_h \cdot (GCp_{neg} - GCpi)$$

$$p_3 := q_h \cdot (GCp_{pos} + GCpi)$$

$$p_4 := q_h \cdot (GCp_{neg} + GCpi)$$

$$p := \max\left(|p_1|, |p_2|, |p_3|, |p_4|\right) = 14\ psf$$

Panel Loading

$w_{rf} := 60\ ft$ — Joist Span/Spacing Adjacent to Panel

$load_{drift} := 59\ psf$ — Drift Load at Panel

$w_{drift} := 14.67\ ft$ — Drift Length

$$P_{drift} := \text{if } w_{rf} \geq w_{drift} \;\; \left\| P \leftarrow load_{drift} \cdot w_{drift} \cdot \left(\frac{1}{2} - \frac{w_{drift}}{6\, w_{rf}}\right) \right. \;\; \text{else if } w_{rf} < w_{drift} \;\; \left\| P \leftarrow load_{drift} \cdot w_{rf} \cdot \left(\frac{1}{2} - \frac{w_{rf}}{6 \cdot w_{drift}}\right) \right.$$

Snow Drift Reaction at Panel $P_{drift} = 0.4\ \frac{k}{ft}$

$$P_{dlr} := x_g \cdot DL \cdot \frac{w_{rf}}{2}$$

Roof Dead Load $P_{dlr} = 0.6\ \frac{k}{ft}$

$$P_{dlp} := x_g \cdot t \cdot 150\ pcf \cdot \left(\frac{ht}{2} + h_p\right)$$

Panel Dead Load $P_{dlp} = 4.3\ \frac{k}{ft}$

$$P_{sl} := x_g \cdot \left(SL \cdot \frac{w_{rf}}{2} + P_{drift}\right)$$

Total Snow Load $P_{sl} = 1.2\ \frac{k}{ft}$

$$M_w := x_{OOP} \cdot \frac{p \cdot ht^2}{8}$$

Moment Due To Wind $M_w = 3.1\ \frac{k \cdot ft}{ft}$

$$w_1 := F_p \cdot 150\ pcf\ t \cdot x_{OOP}$$

$$w_2 := \left(F_p \cdot 10 \cdot psf \cdot w_o \cdot 0.5 + F_p \cdot 150\ pcf \cdot t \cdot L\right) \cdot \frac{1}{L}$$

$$R_{bot} := \frac{w_2 \cdot ht}{2} + \frac{\left(w_1 - w_2\right) \cdot \left(ht - ht_o\right)^2}{2 \cdot ht}$$

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$$R_{top} := \left(w_2 \cdot ht_o + w_1 \cdot \left(ht - ht_o\right)\right) - R_{bot}$$

$$x_m := \frac{R_{top}}{w_1}$$

$$M_s := R_{top} \cdot x_m - \frac{w_1 \cdot {x_m}^2}{2}$$

Moment Due to Seismic w/ Opening $M_s = 8.8\ \frac{k \cdot ft}{ft}$

Strength Load Combinations (LRFD)

(ASCE 7-16 2.3)

$$x := \begin{bmatrix} 1.2 \cdot P_{dlr} & 1.2 \cdot P_{dlp} & 1.6 \cdot P_{sl} & 0.5 \cdot M_w \\ 1.2 \cdot P_{dlr} & 1.2 \cdot P_{dlp} & 0.5 \cdot P_{sl} & 1.0 \cdot M_w \\ 1.2 \cdot P_{dlr} & 1.2 \cdot P_{dlp} & 0.2 \cdot P_{sl} & 1.0 \cdot M_s \\ 0.9 \cdot P_{dlr} & 0.9 \cdot P_{dlp} & 0 \cdot \frac{k}{ft} & 1.0 \cdot M_w \end{bmatrix}$$

Service/Deflection Load Combinations (ASD)

(2021 IBC 1605.2)

$$\omega := 1.0$$

$$y := \begin{bmatrix} 1.0 \cdot P_{dlr} & 1.0 \cdot P_{dlp} & 1.0 \cdot P_{sl} & 0.5 \cdot \left(\omega \cdot 0.6 \cdot M_w\right) \\ 1.0 \cdot P_{dlr} & 1.0 \cdot P_{dlp} & 0.5 \cdot P_{sl} & 1.0 \cdot \left(\omega \cdot 0.6 \cdot M_w\right) \\ 1.0 \cdot P_{dlr} & 1.0 \cdot P_{dlp} & 1.0 \cdot P_{sl} & \left(\frac{1}{1.4}\right) \cdot M_s \\ 0.67 \cdot P_{dlr} & 0.67 \cdot P_{dlp} & 0 \cdot \frac{k}{ft} & 1.0 \cdot \left(\omega \cdot 0.6 \cdot M_w\right) \end{bmatrix}$$

$$\begin{bmatrix} 0.9 \cdot P_{dlr} & 0.9 \cdot P_{dlp} & 0 \cdot \frac{k}{ft} & 1.0 \cdot M_s \end{bmatrix} \qquad \begin{bmatrix} 0.9 \cdot P_{dlr} & 0.9 \cdot P_{dlp} & 0 \cdot \frac{k}{ft} & \left(\frac{1}{1.4}\right) \cdot M_s \end{bmatrix}$$

$i := 0 .. \text{rows}(x) - 1$

Define Panel Reinforcing

$bar\_size := \#5$	Vertical Bar Size	
$NB := 12$	Number of Vertical Bars	
$rebar\_location := \text{EF}$	Steel Layer: Each Face (EF) or Centerline (CL)	
$stirrup\_size := \#3$	Stirrup Bar Size	
$A_b := A_{bar\_size}$	Vertical Bar Area	
$diam_b := D_{bar\_size}$	Vertical Bar Diameter	
$diam_{str} := D_{stirrup\_size}$	Stirrup Bar Diameter	
$d_{ef} := t - \frac{diam_b}{2} - diam_{str} - clr - r_v$	Steel Depth If Each Face	
$d_{cl} := \frac{t}{2} - r_v$	Steel Depth If Centerline	
$d := \text{if}\left(rebar\_location = 0, d_{ef}, d_{cl}\right)$	Steel Depth	$d = 7.31\ in$
$S := \frac{L - 1.5\ in}{NB - 1}$	Vertical Bar Spacing OC	
$flag := \text{if}\left(S \geq 3\ in, \text{“OK”}, \text{“Revise”}\right)$	$S = 8\ in$	$flag = \text{“OK”}$
$A_s := \frac{NB \cdot A_b}{L}$	Vertical Area of Steel	

Check Axial Capacity

(ACI 318-19 11.8.1.1(d))

$$P_{u_i} := \left\| \begin{array}{l} P_u \leftarrow 0 \\ \left(x_{i,0} + x_{i,1} + x_{i,2}\right) \end{array} \right. \qquad P_a := 0.06 \cdot f_c \cdot t$$

$$flag_i := \text{if}\left(\frac{P_{u_i}}{P_a} \leq 1, \text{“OK”}, \text{“Revise”}\right) \qquad \frac{P_u}{P_a} = \begin{bmatrix} 0.3 \\ 0.2 \\ 0.2 \\ 0.2 \\ 0.2 \end{bmatrix} \qquad flag = \begin{bmatrix} \text{“OK”} \\ \text{“OK”} \\ \text{“OK”} \\ \text{“OK”} \\ \text{“OK”} \end{bmatrix}$$

$compression\_check := flag$

Determine Moment Capacity

Concrete Panel Out-of-Plane Analysis

$$\beta_1 := 0.85 - 0.05 \cdot \left(\frac{f_c - 4000 \cdot psi}{1000 \cdot psi}\right) = 0.85 \qquad P_u := P_u \cdot ft \qquad A_s := A_s \cdot ft$$

$$X_i := \frac{P_{u_i} + A_s \cdot f_y}{0.85 \cdot f_c \cdot ft} \qquad M_{n_i} := \left(A_s \cdot f_y + P_{u_i}\right) \cdot \left(d - \frac{X_i}{2}\right) - P_{u_i} \cdot \left(d - \frac{t}{2}\right)$$

$$\phi M_{n_i} := 0.9 \cdot M_{n_i}$$

Factored Moment Capacity

$$\phi M_n = \begin{bmatrix} 17.8 \\ 17.4 \\ 17.3 \\ 16.8 \\ 16.8 \end{bmatrix} k \cdot ft$$

Check for Tension-Controlled Section

(ACI 318-19 11.8.1.1(b))

$$C_i := \frac{X_i}{\beta_1} \qquad flag_i := \text{if}\left(\frac{C_i}{d} < 0.375, \text{“OK”}, \text{“Revise”}\right)$$

$$tension\_controlled := flag$$

$$\frac{C}{d} = \begin{bmatrix} 0.148 \\ 0.143 \\ 0.142 \\ 0.135 \\ 0.135 \end{bmatrix} \qquad flag = \begin{bmatrix} \text{“OK”} \\ \text{“OK”} \\ \text{“OK”} \\ \text{“OK”} \\ \text{“OK”} \end{bmatrix}$$

Determine Deflection Limits

$$\Delta_{max} := \frac{ht}{150}$$

Allowable Panel Deflection

$$ht' := \text{if}\left(DockHigh = 0, ht, 0.8 \cdot ht\right) = 35.5\ ft$$

Adjust Span per Dock High Condition

$$E_c := 57000 \cdot \sqrt{\frac{f_c}{psi}} \cdot psi \qquad n := \frac{29000 \cdot ksi}{E_c} \qquad A_{se_i} := A_s + \frac{P_{u_i} \cdot t}{2 \cdot f_y \cdot d} \qquad C_i := \frac{X_i}{0.85}$$

$$f_r := 7.5 \cdot \sqrt{\frac{f_c}{psi}} \cdot psi \qquad I_g := \frac{(1\ ft) \cdot t^3}{12} \qquad I_{cr_i} := \frac{12 \cdot in \cdot \left(C_i\right)^3}{3} + n \cdot A_{se_i} \cdot \left(d - C_i\right)^2$$

$$M_{cr} := \frac{2 \cdot f_r \cdot I_g}{t} \qquad \Delta_{cr} := \frac{5 \cdot M_{cr} \cdot ht'^2}{48 \cdot E_c \cdot I_g} \qquad \Delta_{n_i} := \frac{5 \cdot M_{n_i} \cdot ht'^2}{48 \cdot E_c \cdot I_{cr_i}}$$

Check Moment Capacity

(ACI 318-19 11.8.3.1)

$$M_{ua_i} := \left(x_{i,3} + \left(x_{i,0} + x_{i,2}\right) \cdot \frac{e}{2}\right) \cdot ft \qquad M_{u_i} := \frac{M_{ua_i}}{1 - \dfrac{5 \cdot P_{u_i} \cdot ht'^2}{0.75 \cdot 48 \cdot E_c \cdot I_{cr_i}}}$$

$$flag_i := \text{if}\left(\frac{M_{u_i}}{\phi M_{n_i}} \le 1, \text{“OK”}, \text{“Revise”}\right)$$

$$capacity\_check := flag$$

$$\frac{M_u}{\phi M_n} = \begin{bmatrix} 0.2 \\ 0.3 \\ 0.7 \\ 0.2 \\ 0.6 \end{bmatrix} \qquad flag = \begin{bmatrix} \text{“OK”} \\ \text{“OK”} \\ \text{“OK”} \\ \text{“OK”} \\ \text{“OK”} \end{bmatrix}$$

Check Deflection (ACI 318-19 11.8.4)

$M_{a_i} := \left(y_{i,3} + \left(y_{i,0} + y_{i,2}\right) \cdot \frac{e}{2}\right) ft$ $\quad P_{s_i} := \left(y_{i,0} + y_{i,1} + y_{i,2}\right) \cdot ft$

Iterate Service Deflection

$\Delta_{s_i} := 0 \cdot in$ Initial Guess

$\Delta_s := \Delta f_s\left(M_a, P_s, \Delta_s, M_{cr}, \Delta_{cr}, M_n, \Delta_n\right)$ Function iterates service deflections per ACI 318-19 *Sec 11.8.4*

$flag_i := \text{if}\left(\frac{\Delta_{s_i}}{\Delta_{max}} \le 1, \text{“OK”}, \text{“Revise”}\right)$

$\frac{\Delta_s}{\Delta_{max}} = \begin{bmatrix} 0 \\ 0.1 \\ 0.5 \\ 0.1 \\ 0.4 \end{bmatrix}$ $\quad flag = \begin{bmatrix} \text{“OK”} \\ \text{“OK”} \\ \text{“OK”} \\ \text{“OK”} \\ \text{“OK”} \end{bmatrix}$

$deflection\_check := flag$

Results Summary

Panel Thickness	$t = 9.5\ in$	**Bar Size**	$rebar_{bar\_size} = \text{“\#5”}$
Panel Leg Width	$L = 7.5\ ft$	**Bar Quantity**	$NB = 12$
		Bar Location	$layer = \text{“EF”}$

Concrete Panel Out-of-Plane Analysis

Panel w/ Double Door + Personnel Door (P19) 6'-4" Leg 9.5" THK w/ (10) #5 VERT EF

Panel Geometry

$t := 9.5\ in$	Panel Thickness
$ht := 35\ ft + 6\ in$	Roof Height (Panel Span)
$h_p := 8\ ft$	Parapet Height
$DockHigh :=$ No	Dock High Span Reduction
$w := 28 \cdot ft$	Panel Width
$L := 6\ ft + 4\ in$	Panel Leg Width
$ht_o := 7\ ft + 4\ in$	Panel Opening Height
$w_o := 3\ ft + 4\ in$	Panel Opening Width
$e := \frac{t}{2} + 3 \cdot in = 7.75\ in$	Roof Load Eccentricity
$h_T := ht + h_p = 43.5\ ft$	Total Panel Height

Panel Load Factors

$x_g := 1 + \frac{w_o}{2 \cdot L} = 1.3$	Gravity Factor
$x_{OOP} := 1 + \frac{w_o}{2 \cdot L} = 1.3$	OOP Factor

Panel Wind Loading Per ASCE 7-16 Chapter 30 - Part 1 Method

$GCpi := 0.18$ Coefficient per Table 26.13-1

$$A_{eff} := \frac{1}{3}\ ht^2 = 420.1\ ft^2$$

$Slope :=$ Y

If roof slope is <10 degrees,
GCp can be reduced by 10%

$$GCp_{poscalc} := \left(1.0 - \frac{\ln\left(\frac{A_{eff}}{ft^2}\right) - \ln(10)}{\ln(500) - \ln(10)}(1 - 0.7)\right)(1 - Slope)$$

Positive pressure coefficient (zones 4 and 5)

$$GCp_{negcalc} := \left(1.4 - \frac{\ln\left(\frac{A_{eff}}{ft^2}\right) - \ln(10)}{\ln(500) - \ln(10)}(1.4 - 0.8)\right)(1 - Slope)(-1)$$

Negative pressure coefficient (zone 5)

$$GCp_{pos} := \begin{array}{l} \text{if } A_{eff} > 500\ ft^2 \\ \quad \| 0.7\ (1 - Slope) \\ \text{else if } A_{eff} < 10\ ft^2 \\ \quad \| 1.0\ (1 - Slope) \\ \text{else} \\ \quad \| GCp_{poscalc} \end{array}$$

Check against upper and lower bound values for CGp based on Aeff

$$GCp_{neg} := \begin{array}{l} \text{if } A_{eff} > 500\ ft^2 \\ \quad \| -0.8\ (1 - Slope) \\ \text{else if } A_{eff} < 10\ ft^2 \\ \quad \| -1.4\ (1 - Slope) \\ \text{else} \\ \quad \| GCp_{negcalc} \end{array}$$

$$p_1 := q_h \cdot (GCp_{pos} - GCpi)$$

$$p_2 := q_h \cdot (GCp_{neg} - GCpi)$$

$$p_3 := q_h \cdot (GCp_{pos} + GCpi)$$

$$p_4 := q_h \cdot (GCp_{neg} + GCpi)$$

$$p := \max(|p_1|, |p_2|, |p_3|, |p_4|) = 14\ psf$$

Panel Loading

$w_{rf} := 60\ ft$ Joist Span/Spacing Adjacent to Panel

$load_{drift} := 59\ psf$ Drift Load at Panel

$w_{drift} := 14.67\ ft$ Drift Length

Concrete Panel Out-of-Plane Analysis

$$P_{drift} := \text{if } w_{rf} \ge w_{drift} \;\; \left\| P \leftarrow load_{drift} \cdot w_{drift} \cdot \left(\frac{1}{2} - \frac{w_{drift}}{6\ w_{rf}}\right) \right. \;\; \text{else if } w_{rf} < w_{drift} \;\; \left\| P \leftarrow load_{drift} \cdot w_{rf} \cdot \left(\frac{1}{2} - \frac{w_{rf}}{6 \cdot w_{drift}}\right) \right.$$

Snow Drift Reaction at Panel $P_{drift} = 0.4\ \frac{k}{ft}$

$$P_{dlr} := x_g \cdot DL \cdot \frac{w_{rf}}{2}$$

Roof Dead Load $P_{dlr} = 0.6\ \frac{k}{ft}$

$$P_{dlp} := x_g \cdot t \cdot 150\ pcf \cdot \left(\frac{ht}{2} + h_p\right)$$

Panel Dead Load $P_{dlp} = 3.9\ \frac{k}{ft}$

$$P_{sl} := x_g \cdot \left(SL \cdot \frac{w_{rf}}{2} + P_{drift}\right)$$

Total Snow Load $P_{sl} = 1.03\ \frac{k}{ft}$

$$M_w := x_{OOP} \cdot \frac{p \cdot ht^2}{8}$$

Moment Due To Wind $M_w = 2.8\ \frac{k \cdot ft}{ft}$

$$w_1 := F_p \cdot 150\ pcf\ t \cdot x_{OOP}$$

$$w_2 := \left(F_p \cdot 10 \cdot psf \cdot w_o \cdot 0.5 + F_p \cdot 150\ pcf \cdot t \cdot L\right) \cdot \frac{1}{L}$$

$$R_{bot} := \frac{w_2 \cdot ht}{2} + \frac{(w_1 - w_2) \cdot (ht - ht_o)^2}{2 \cdot ht}$$

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$$R_{top} := \left(w_2 \cdot ht_o + w_1 \cdot (ht - ht_o)\right) - R_{bot}$$

$$x_m := \frac{R_{top}}{w_1}$$

$$M_s := R_{top} \cdot x_m - \frac{w_1 \cdot x_m^{\ 2}}{2}$$

Moment Due to Seismic w/ Opening $M_s = 7.8\ \frac{k \cdot ft}{ft}$

Strength Load Combinations (LRFD)

(ASCE 7-16 2.3)

$$x := \begin{bmatrix} 1.2 \cdot P_{dlr} & 1.2 \cdot P_{dlp} & 1.6 \cdot P_{sl} & 0.5 \cdot M_w \\ 1.2 \cdot P_{dlr} & 1.2 \cdot P_{dlp} & 0.5 \cdot P_{sl} & 1.0 \cdot M_w \\ 1.2 \cdot P_{dlr} & 1.2 \cdot P_{dlp} & 0.2 \cdot P_{sl} & 1.0 \cdot M_s \\ 0.9 \cdot P_{dlr} & 0.9 \cdot P_{dlp} & 0 \cdot \frac{k}{ft} & 1.0 \cdot M_w \\ 0.9 \cdot P_{dlr} & 0.9 \cdot P_{dlp} & 0 \cdot \frac{k}{ft} & 1.0 \cdot M_s \end{bmatrix}$$

Service/Deflection Load Combinations (ASD)

(2021 IBC 1605.2)

$$\omega := 1.0$$

$$y := \begin{bmatrix} 1.0 \cdot P_{dlr} & 1.0 \cdot P_{dlp} & 1.0 \cdot P_{sl} & 0.5 \cdot (\omega \cdot 0.6 \cdot M_w) \\ 1.0 \cdot P_{dlr} & 1.0 \cdot P_{dlp} & 0.5 \cdot P_{sl} & 1.0 \cdot (\omega \cdot 0.6 \cdot M_w) \\ 1.0 \cdot P_{dlr} & 1.0 \cdot P_{dlp} & 1.0 \cdot P_{sl} & \left(\frac{1}{1.4}\right) \cdot M_s \\ 0.67 \cdot P_{dlr} & 0.67 \cdot P_{dlp} & 0 \cdot \frac{k}{ft} & 1.0 \cdot (\omega \cdot 0.6 \cdot M_w) \end{bmatrix}$$

$i := 0 .. \text{rows}(x) - 1$ $\quad \left[0.9 \cdot P_{dlr} \quad 0.9 \cdot P_{dlp} \quad 0 \cdot \frac{k}{ft} \quad \left(\frac{1}{1.4}\right) \cdot M_s \right]$

Define Panel Reinforcing

$bar\_size := \#5$	Vertical Bar Size	
$NB := 10$	Number of Vertical Bars	
$rebar\_location := \text{EF}$	Steel Layer: Each Face (EF) or Centerline (CL)	
$stirrup\_size := \#3$	Stirrup Bar Size	
$A_b := A_{bar\_size}$	Vertical Bar Area	
$diam_b := D_{bar\_size}$	Vertical Bar Diameter	
$diam_{str} := D_{stirrup\_size}$	Stirrup Bar Diameter	
$d_{ef} := t - \frac{diam_b}{2} - diam_{str} - clr - r_v$	Steel Depth If Each Face	
$d_{cl} := \frac{t}{2} - r_v$	Steel Depth If Centerline	
$d := \text{if}\left(rebar\_location = 0, d_{ef}, d_{cl}\right)$	Steel Depth	$d = 7.31\ in$
$S := \frac{L - 1.5\ in}{NB - 1}$	Vertical Bar Spacing OC	
$flag := \text{if}\left(S \geq 3\ in, \text{"OK"}, \text{"Revise"}\right)$	$S = 8.3\ in$	$flag = \text{"OK"}$
$A_s := \frac{NB \cdot A_b}{L}$	Vertical Area of Steel	

Check Axial Capacity

(ACI 318-19 11.8.1.1(d))

$$P_{u_i} := \left\| \begin{array}{l} P_u \leftarrow 0 \\ \left(x_{i,0} + x_{i,1} + x_{i,2}\right) \end{array} \right.$$

$$P_a := 0.06 \cdot f_c \cdot t$$

$$flag_i := \text{if}\left(\frac{P_{u_i}}{P_a} \leq 1, \text{"OK"}, \text{"Revise"}\right)$$

$$\frac{P_u}{P_a} = \begin{bmatrix} 0.3 \\ 0.2 \\ 0.2 \\ 0.1 \\ 0.1 \end{bmatrix} \qquad flag = \begin{bmatrix} \text{"OK"} \\ \text{"OK"} \\ \text{"OK"} \\ \text{"OK"} \\ \text{"OK"} \end{bmatrix}$$

$compression\_check := flag$

Determine Moment Capacity

$$\beta_1 := 0.85 - 0.05 \cdot \left(\frac{f_c - 4000 \cdot psi}{1000 \cdot psi}\right) = 0.85$$

$P_u := P_u \cdot ft \qquad A_s := A_s \cdot ft$

Concrete Panel Out-of-Plane Analysis

$X_i := \frac{P_{u_i} + A_s \cdot f_y}{0.85 \cdot f_c \cdot ft}$ $M_{n_i} := \left(A_s \cdot f_y + P_{u_i}\right) \cdot \left(d - \frac{X_i}{2}\right) - P_{u_i} \cdot \left(d - \frac{t}{2}\right)$

$\phi M_{n_i} := 0.9 \cdot M_{n_i}$ Factored Moment Capacity $\phi M_n = \begin{bmatrix} 17.4 \\ 17 \\ 17 \\ 16.5 \\ 16.5 \end{bmatrix} k \cdot ft$

Check for Tension-Controlled Section

(ACI 318-19 11.8.1.1(b))

$C_i := \frac{X_i}{\beta_1}$ $flag_i := \text{if}\left(\frac{C_i}{d} < 0.375, \text{"OK"}, \text{"Revise"}\right)$ $\frac{C}{d} = \begin{bmatrix} 0.143 \\ 0.139 \\ 0.138 \\ 0.132 \\ 0.132 \end{bmatrix}$ $flag = \begin{bmatrix} \text{"OK"} \\ \text{"OK"} \\ \text{"OK"} \\ \text{"OK"} \\ \text{"OK"} \end{bmatrix}$

$tension\_controlled := flag$

Determine Deflection Limits

$\Delta_{max} := \frac{ht}{150}$ Allowable Panel Deflection

$ht' := \text{if}\left(DockHigh = 0, ht, 0.8 \cdot ht\right) = 35.5\ ft$ Adjust Span per Dock High Condition

$E_c := 57000 \cdot \sqrt{\frac{f_c}{psi}} \cdot psi$ $n := \frac{29000 \cdot ksi}{E_c}$ $A_{se_i} := A_s + \frac{P_{u_i} \cdot t}{2 \cdot f_y \cdot d}$ $C_i := \frac{X_i}{0.85}$

$f_r := 7.5 \cdot \sqrt{\frac{f_c}{psi}} \cdot psi$ $I_g := \frac{(1\ ft) \cdot t^3}{12}$ $I_{cr_i} := \frac{12 \cdot in \cdot \left(C_i\right)^3}{3} + n \cdot A_{se_i} \cdot \left(d - C_i\right)^2$

$M_{cr} := \frac{2 \cdot f_r \cdot I_g}{t}$ $\Delta_{cr} := \frac{5 \cdot M_{cr} \cdot ht'^2}{48 \cdot E_c \cdot I_g}$ $\Delta_{n_i} := \frac{5 \cdot M_{n_i} \cdot ht'^2}{48 \cdot E_c \cdot I_{cr_i}}$

Check Moment Capacity

(ACI 318-19 11.8.3.1)

$M_{ua_i} := \left(x_{i,3} + \left(x_{i,0} + x_{i,2}\right) \cdot \frac{e}{2}\right) \cdot ft$ $M_{u_i} := \frac{M_{ua_i}}{1 - \frac{5 \cdot P_{u_i} \cdot ht'^2}{0.75 \cdot 48 \cdot E_c \cdot I_{cr_i}}}$

$flag_i := \text{if}\left(\frac{M_{u_i}}{\phi M_{n_i}} \le 1, \text{"OK"}, \text{"Revise"}\right)$ $\frac{M_u}{\phi M_n} = \begin{bmatrix} 0.17 \\ 0.24 \\ 0.61 \\ 0.21 \\ 0.58 \end{bmatrix}$ $flag = \begin{bmatrix} \text{"OK"} \\ \text{"OK"} \\ \text{"OK"} \\ \text{"OK"} \\ \text{"OK"} \end{bmatrix}$

$capacity\_check := flag$

Check Deflection
(ACI 318-19 11.8.4)

$M_{a_i} := \left(y_{i,3} + \left(y_{i,0} + y_{i,2}\right) \cdot \frac{e}{2}\right) ft$ $\qquad P_{s_i} := \left(y_{i,0} + y_{i,1} + y_{i,2}\right) \cdot ft$

Iterate Service Deflection

$\Delta_{s_i} := 0 \cdot in$ Initial Guess

$\Delta_s := \Delta f_s\left(M_a, P_s, \Delta_s, M_{cr}, \Delta_{cr}, M_n, \Delta_n\right)$ Function iterates service deflections per ACI 318-19 *Sec 11.8.4*

$flag_i := \text{if}\left(\frac{\Delta_{s_i}}{\Delta_{max}} \le 1, \text{"OK"}, \text{"Revise"}\right)$

$$\frac{\Delta_s}{\Delta_{max}} = \begin{bmatrix} 0.04 \\ 0.05 \\ 0.37 \\ 0.05 \\ 0.31 \end{bmatrix} \qquad flag = \begin{bmatrix} \text{"OK"} \\ \text{"OK"} \\ \text{"OK"} \\ \text{"OK"} \\ \text{"OK"} \end{bmatrix}$$

$deflection\_check := flag$

Results Summary

Panel Thickness $t = 9.5\ in$ **Bar Size** $rebar_{bar\_size} = \text{"\#5"}$

Panel Leg Width $L = 6.3\ ft$ **Bar Quantity** $NB = 10$

Bar Location $layer = \text{"EF"}$

Concrete Panel Out-of-Plane Analysis

Panel w/ Double Door + Personnel Door (P19) 4'-6" Center Leg 9.5" THK w/ (10) #5 VERT EF

Panel Geometry

$t := 9.5\ in$ Panel Thickness

$ht := 35\ ft + 6\ in$ Roof Height (Panel Span)

$h_p := 8\ ft$ Parapet Height

$DockHigh := \text{No}$ Dock High Span Reduction

$w := 28 \cdot ft$ Panel Width

$L := 4\ ft + 6\ in$ Panel Leg Width

$ht_{o1} := 7\ ft + 4\ in$ Panel Opening Height

$w_{o1} := 3\ ft + 4\ in$ Panel Opening Width

$ht_{o2} := 7\ ft + 4\ in$

$w_{o2} := 6\ ft + 4\ in$

$e := \dfrac{t}{2} + 3 \cdot in = 7.75\ in$ Roof Load Eccentricity

$h_T := ht + h_p = 43.5\ ft$ Total Panel Height

Panel Load Factors

$x_g := 1 + \dfrac{w_{o1} + w_{o2}}{2\ L} = 2.1$ Gravity Factor

$x_{OOP} := 1 + \dfrac{w_o + w_{o2}}{2\ L} = 2.1$ OOP Factor

Panel Wind Loading Per ASCE 7-16 Chapter 30 - Part 1 Method

$GCpi := 0.18$ Coefficient per Table 26.13-1

$A_{eff} := \dfrac{1}{3}\ ht^2 = 420.1\ ft^2$

$Slope := \text{Y}$ If roof slope is <10 degrees, GCp can be reduced by 10%

Concrete Panel Out-of-Plane Analysis

$$GCp_{poscalc} := \left(1.0 - \frac{\ln\left(\frac{A_{eff}}{ft^2}\right) - \ln(10)}{\ln(500) - \ln(10)} (1 - 0.7)\right)(1 - Slope)$$

Positive pressure coefficient (zones 4 and 5)

$$GCp_{negcalc} := \left(1.4 - \frac{\ln\left(\frac{A_{eff}}{ft^2}\right) - \ln(10)}{\ln(500) - \ln(10)} (1.4 - 0.8)\right)(1 - Slope)(-1)$$

Negative pressure coefficient (zone 5)

$$GCp_{pos} := \begin{array}{l} \text{if } A_{eff} > 500\ ft^2 \\ \quad \| 0.7\ (1 - Slope) \\ \text{else if } A_{eff} < 10\ ft^2 \\ \quad \| 1.0\ (1 - Slope) \\ \text{else} \\ \quad \| GCp_{poscalc} \end{array}$$

$$GCp_{neg} := \begin{array}{l} \text{if } A_{eff} > 500\ ft^2 \\ \quad \| -0.8\ (1 - Slope) \\ \text{else if } A_{eff} < 10\ ft^2 \\ \quad \| -1.4\ (1 - Slope) \\ \text{else} \\ \quad \| GCp_{negcalc} \end{array}$$

Check against upper and lower bound values for CGp based on Aeff

$$p_1 := q_h \cdot (GCp_{pos} - GCpi)$$

$$p_2 := q_h \cdot (GCp_{neg} - GCpi)$$

$$p_3 := q_h \cdot (GCp_{pos} + GCpi)$$

$$p_4 := q_h \cdot (GCp_{neg} + GCpi)$$

$$p := \max(|p_1|, |p_2|, |p_3|, |p_4|) = 14\ psf$$

Panel Loading

$w_{rf} := 60\ ft$ Joist Span/Spacing Adjacent to Panel

$load_{drift} := 59\ psf$ Drift Load at Panel

$w_{drift} := 14.67\ ft$ Drift Length

Concrete Panel Out-of-Plane Analysis

$$P_{drift} := \text{if } w_{rf} \ge w_{drift} \quad \left\| P \leftarrow load_{drift} \cdot w_{drift} \cdot \left(\frac{1}{2} - \frac{w_{drift}}{6\ w_{rf}}\right) \right. \quad \text{else if } w_{rf} < w_{drift} \quad \left\| P \leftarrow load_{drift} \cdot w_{rf} \cdot \left(\frac{1}{2} - \frac{w_{rf}}{6 \cdot w_{drift}}\right) \right.$$

Snow Drift Reaction at Panel $P_{drift} = 0.4\ \frac{k}{ft}$

$$P_{dlr} := x_g \cdot DL \cdot \frac{w_{rf}}{2}$$

Roof Dead Load $P_{dlr} = 0.6\ \frac{k}{ft}$

$$P_{dlp} := x_g \cdot t \cdot 150\ pcf \cdot \left(\frac{ht}{2} + h_p\right)$$

Panel Dead Load $P_{dlp} = 6.3\ \frac{k}{ft}$

$$P_{sl} := x_g \cdot \left(SL \cdot \frac{w_{rf}}{2} + P_{drift}\right)$$

Total Snow Load $P_{sl} = 1.7\ \frac{k}{ft}$

$$M_w := x_{OOP} \cdot \frac{p \cdot ht^2}{8}$$

Moment Due To Wind $M_w = 4.6\ \frac{k \cdot ft}{ft}$

$$w_1 := F_p \cdot 150\ pcf\ \ t \cdot x_{OOP}$$

$$w_2 := \left(F_p \cdot 10 \cdot psf \cdot (w_{o1} + w_{o2}) \cdot 0.5 + F_p \cdot 150\ pcf \cdot t \cdot L\right) \cdot \frac{1}{L} = 43.7\ psf$$

$$R_{bot} := \frac{w_2 \cdot ht}{2} + \frac{(w_1 - w_2) \cdot (ht - ht_{o1})^2}{2 \cdot ht} = 1.2\ \frac{k}{ft}$$

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$$R_{top} := \left(w_2 \cdot ht_{o1} + w_1 \cdot (ht - ht_{o1})\right) - R_{bot} = 1.4\ \frac{k}{ft}$$

$$x_m := \frac{R_{top}}{w_1}$$

$$M_s := R_{top} \cdot x_m - \frac{w_1 \cdot {x_m}^2}{2}$$

Moment Due to Seismic w/ Opening $M_s = 12.6\ \frac{k \cdot ft}{ft}$

<u>Strength Load Combinations (LRFD)</u>

(ASCE 7-16 2.3)

$$x := \begin{bmatrix} 1.2 \cdot P_{dlr} & 1.2 \cdot P_{dlp} & 1.6 \cdot P_{sl} & 0.5 \cdot M_w \\ 1.2 \cdot P_{dlr} & 1.2 \cdot P_{dlp} & 0.5 \cdot P_{sl} & 1.0 \cdot M_w \\ 1.2 \cdot P_{dlr} & 1.2 \cdot P_{dlp} & 0.2\ P_{sl} & 1.0 \cdot M_s \\ 0.9 \cdot P_{dlr} & 0.9 \cdot P_{dlp} & 0 \cdot \frac{k}{ft} & 1.0 \cdot M_w \\ 0.9 \cdot P_{dlr} & 0.9 \cdot P_{dlp} & 0 \cdot \frac{k}{ft} & 1.0 \cdot M_s \end{bmatrix}$$

<u>Service/Deflection Load Combinations (ASD)</u>

(2021 IBC 1605.2)

$$\omega := 1.0$$

$$y := \begin{bmatrix} 1.0 \cdot P_{dlr} & 1.0 \cdot P_{dlp} & 1.0 \cdot P_{sl} & 0.5 \cdot (\omega \cdot 0.6 \cdot M_w) \\ 1.0 \cdot P_{dlr} & 1.0 \cdot P_{dlp} & 0.5 \cdot P_{sl} & 1.0 \cdot (\omega \cdot 0.6 \cdot M_w) \\ 1.0 \cdot P_{dlr} & 1.0 \cdot P_{dlp} & 1.0 \cdot P_{sl} & \left(\frac{1}{1.4}\right) \cdot M_s \\ 0.67 \cdot P_{dlr} & 0.67 \cdot P_{dlp} & 0 \cdot \frac{k}{ft} & 1.0 \cdot (\omega \cdot 0.6 \cdot M_w) \end{bmatrix}$$

$$\begin{bmatrix} 0.9 \cdot P_{dlr} & 0.9 \cdot P_{dlp} & 0 \cdot \frac{k}{ft} & 1.0 \cdot M_s \end{bmatrix}$$

$i := 0 .. \text{rows}(x) - 1$

$$\begin{bmatrix} 0.9 \cdot P_{dlr} & 0.9 \cdot P_{dlp} & 0 \cdot \frac{k}{ft} & \left(\frac{1}{1.4}\right) \cdot M_s \end{bmatrix}$$

Define Panel Reinforcing

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

$NB := 10$ — Number of Vertical Bars

$rebar\_location := EF$ — Steel Layer: Each Face (EF) or Centerline (CL)

$stirrup\_size := \#3$ — Stirrup Bar Size

$A_b := A_{bar\_size}$ — Vertical Bar Area

$diam_b := D_{bar\_size}$ — Vertical Bar Diameter

$diam_{str} := D_{stirrup\_size}$ — Stirrup Bar Diameter

$d_{ef} := t - \frac{diam_b}{2} - diam_{str} - clr - r_v$ — Steel Depth If Each Face

$d_{cl} := \frac{t}{2} - r_v$ — Steel Depth If Centerline

$d := \text{if}\left(rebar\_location = 0, d_{ef}, d_{cl}\right)$ — Steel Depth — $d = 7.31\ in$

$S := \frac{L - 1.5\ in}{NB - 1}$ — Vertical Bar Spacing OC

$flag := \text{if}\left(S \geq 3\ in, \text{"OK"}, \text{"Revise"}\right)$ — $S = 5.8\ in$ — $flag = \text{"OK"}$

$A_s := \frac{NB \cdot A_b}{L}$ — Vertical Area of Steel

Check Axial Capacity — *(ACI 318-19 11.8.1.1(d))*

$$P_{u_i} := \left\| \begin{array}{l} P_u \leftarrow 0 \\ \left(x_{i,0} + x_{i,1} + x_{i,2}\right) \end{array} \right.$$

$P_a := 0.06 \cdot f_c \cdot t$

$$flag_i := \text{if}\left(\frac{P_{u_i}}{P_a} \leq 1, \text{"OK"}, \text{"Revise"}\right)$$

$$\frac{P_u}{P_a} = \begin{bmatrix} 0.4 \\ 0.3 \\ 0.3 \\ 0.2 \\ 0.2 \end{bmatrix} \qquad flag = \begin{bmatrix} \text{"OK"} \\ \text{"OK"} \\ \text{"OK"} \\ \text{"OK"} \\ \text{"OK"} \end{bmatrix}$$

$compression\_check := flag$

Determine Moment Capacity

$$\beta_1 := 0.85 - 0.05 \cdot \left(\frac{f_c - 4000 \cdot psi}{1000 \cdot psi}\right)$$

$P_u := P_u \cdot ft \qquad A_s := A_s \cdot ft$

$X_i := \dfrac{P_{u_i} + A_s \cdot f_y}{0.85 \cdot f_c \cdot ft}$ $\quad M_{n_i} := \left(A_s \cdot f_y + P_{u_i}\right) \cdot \left(d - \dfrac{X_i}{2}\right) - P_{u_i} \cdot \left(d - \dfrac{t}{2}\right)$

$\phi M_{n_i} := 0.9 \cdot M_{n_i}$ Factored Moment Capacity $\quad \phi M_n = \begin{bmatrix} 24.2 \\ 23.7 \\ 23.6 \\ 22.9 \\ 22.9 \end{bmatrix} k \cdot ft$

Check for Tension-Controlled Section (ACI 318-19 11.8.1.1(b))

$C_i := \dfrac{X_i}{\beta_1}$ $\quad flag_i := \text{if}\left(\dfrac{C_i}{d} < 0.375, \text{“OK”}, \text{“Revise”}\right)$ $\quad \dfrac{C}{d} = \begin{bmatrix} 0.208 \\ 0.201 \\ 0.199 \\ 0.189 \\ 0.189 \end{bmatrix}$ $\quad flag = \begin{bmatrix} \text{“OK”} \\ \text{“OK”} \\ \text{“OK”} \\ \text{“OK”} \\ \text{“OK”} \end{bmatrix}$

$tension\_controlled := flag$

Determine Deflection Limits

$\Delta_{max} := \dfrac{ht}{150}$ Allowable Panel Deflection

$ht' := \text{if}\left(DockHigh = 0, ht, 0.8 \cdot ht\right) = 35.5\ ft$ Adjust Span per Dock High Condition

$E_c := 57000 \cdot \sqrt{\dfrac{f_c}{psi}} \cdot psi$ $\quad n := \dfrac{29000 \cdot ksi}{E_c}$ $\quad A_{se_i} := A_s + \dfrac{P_{u_i} \cdot t}{2 \cdot f_y \cdot d}$ $\quad C_i := \dfrac{X_i}{0.85}$

$f_r := 7.5 \cdot \sqrt{\dfrac{f_c}{psi}} \cdot psi$ $\quad I_g := \dfrac{(1\ ft) \cdot t^3}{12}$ $\quad I_{cr_i} := \dfrac{12 \cdot in \cdot \left(C_i\right)^3}{3} + n \cdot A_{se_i} \cdot \left(d - C_i\right)^2$

$M_{cr} := \dfrac{2 \cdot f_r \cdot I_g}{t}$ $\quad \Delta_{cr} := \dfrac{5 \cdot M_{cr} \cdot ht'^2}{48 \cdot E_c \cdot I_g}$ $\quad \Delta_{n_i} := \dfrac{5 \cdot M_{n_i} \cdot ht'^2}{48 \cdot E_c \cdot I_{cr_i}}$

Check Moment Capacity (ACI 318-19 11.8.3.1)

$M_{ua_i} := \left(x_{i,3} + \left(x_{i,0} + x_{i,2}\right) \cdot \dfrac{e}{2}\right) \cdot ft$ $\quad M_{u_i} := \dfrac{M_{ua_i}}{1 - \dfrac{5 \cdot P_{u_i} \cdot ht'^2}{0.75 \cdot 48 \cdot E_c \cdot I_{cr_i}}}$

$flag_i := \text{if}\left(\dfrac{M_{u_i}}{\phi M_{n_i}} \le 1, \text{“OK”}, \text{“Revise”}\right)$ $\quad \dfrac{M_u}{\phi M_n} = \begin{bmatrix} 0.2 \\ 0.3 \\ 0.8 \\ 0.3 \\ 0.7 \end{bmatrix}$ $\quad flag = \begin{bmatrix} \text{“OK”} \\ \text{“OK”} \\ \text{“OK”} \\ \text{“OK”} \\ \text{“OK”} \end{bmatrix}$

$capacity\_check := flag$

Check Deflection (ACI 318-19 11.8.4)

$$M_{a_i} := \left(y_{i,3} + \left(y_{i,0} + y_{i,2}\right) \cdot \frac{e}{2}\right) ft \qquad P_{s_i} := \left(y_{i,0} + y_{i,1} + y_{i,2}\right) \cdot ft$$

Iterate Service Deflection

$\Delta_{s_i} := 0 \cdot in$ Initial Guess

$\Delta_s := \Delta f_s\left(M_a, P_s, \Delta_s, M_{cr}, \Delta_{cr}, M_n, \Delta_n\right)$ Function iterates service deflections per ACI 318-19 *Sec 11.8.4*

$$flag_i := \text{if}\left(\frac{\Delta_{s_i}}{\Delta_{max}} \le 1, \text{“OK”}, \text{“Revise”}\right)$$

$$\frac{\Delta_s}{\Delta_{max}} = \begin{bmatrix} 0.1 \\ 0.1 \\ 0.8 \\ 0.1 \\ 0.7 \end{bmatrix} \qquad flag = \begin{bmatrix} \text{“OK”} \\ \text{“OK”} \\ \text{“OK”} \\ \text{“OK”} \\ \text{“OK”} \end{bmatrix}$$

$deflection\_check := flag$

Results Summary

Panel Thickness	$t = 9.5\ in$	**Bar Size**	$rebar_{bar\_size} = \text{“\#5”}$
Panel Leg Width	$L = 4.5\ ft$	**Bar Quantity**	$NB = 10$
		Bar Location	$layer = \text{“EF”}$

Concrete Panel Out-of-Plane Analysis

Panel w/ Double Door Opening w/ Canopy (P1) 8" THK, (16) #5 VERT EF

Panel Geometry

$t := 8\ in$ Panel Thickness

$ht := 36\ ft + 3\ in$ Roof Height (Panel Span)

$h_p := 4\ ft + 3\ in$ Parapet Height

$DockHigh :=$ No Dock High Span Reduction

$w := 25\ ft + 4\ in$ Panel Width

$L := 7\ ft + 8\ in$ Panel Leg Width

$ht_o := 12\ ft + 0\ in$ Panel Opening Height

$w_o := 9\ ft$ Panel Opening Width

$e := \frac{t}{2} + 3 \cdot in = 7\ in$ Roof Load Eccentricity

$h_T := ht + h_p = 40.5\ ft$ Total Panel Height

Panel Load Factors

$x_g := 1 + \frac{w_o}{2 \cdot L} = 1.6$ Gravity Factor

$x_{OOP} := 1 + \frac{w_o}{2 \cdot L} = 1.6$ OOP Factor

Panel Wind Loading Per ASCE 7-16 Chapter 30 - Part 1 Method

$GCpi := 0.18$ Coefficient per Table 26.13-1

$A_{eff} := \frac{1}{3}\ ht^2 = 438\ ft^2$

$Slope :=$ Y

If roof slope is <10 degrees, GCp can be reduced by 10%

Concrete Panel Out-of-Plane Analysis

$$GCp_{poscalc} := \left(1.0 - \frac{\ln\left(\frac{A_{eff}}{ft^2}\right) - \ln(10)}{\ln(500) - \ln(10)} (1 - 0.7)\right)(1 - Slope)$$

Positive pressure coefficient (zones 4 and 5)

$$GCp_{negcalc} := \left(1.4 - \frac{\ln\left(\frac{A_{eff}}{ft^2}\right) - \ln(10)}{\ln(500) - \ln(10)} (1.4 - 0.8)\right)(1 - Slope)(-1)$$

Negative pressure coefficient (zone 5)

$$GCp_{pos} := \text{if } A_{eff} > 500\ ft^2 \quad \| \ 0.7\ (1 - Slope) \quad \text{else if } A_{eff} < 10\ ft^2 \quad \| \ 1.0\ (1 - Slope) \quad \text{else} \quad \| \ GCp_{poscalc}$$

$$GCp_{neg} := \text{if } A_{eff} > 500\ ft^2 \quad \| \ -0.8\ (1 - Slope) \quad \text{else if } A_{eff} < 10\ ft^2 \quad \| \ -1.4\ (1 - Slope) \quad \text{else} \quad \| \ GCp_{negcalc}$$

Check against upper and lower bound values for CGp based on Aeff

$$p_1 := q_h \cdot (GCp_{pos} - GCpi)$$

$$p_2 := q_h \cdot (GCp_{neg} - GCpi)$$

$$p_3 := q_h \cdot (GCp_{pos} + GCpi)$$

$$p_4 := q_h \cdot (GCp_{neg} + GCpi)$$

$$p := \max(|p_1|, |p_2|, |p_3|, |p_4|) = 13.9\ psf$$

Panel Loading

$w_{rf} := 60\ ft$ — Joist Span/Spacing Adjacent to Panel

$load_{drift} := 59\ psf$ — Drift Load at Panel

$w_{drift} := 14.75\ ft$ — Drift Length

$$P_{drift} := \text{if } w_{rf} \ge w_{drift} \quad \left\| P \leftarrow load_{drift} \cdot w_{drift} \cdot \left(\frac{1}{2} - \frac{w_{drift}}{6\ w_{rf}}\right) \right. \quad \text{else if } w_{rf} < w_{drift} \quad \left\| P \leftarrow load_{drift} \cdot w_{rf} \cdot \left(\frac{1}{2} - \frac{w_{rf}}{6 \cdot w_{drift}}\right) \right.$$

Snow Drift Reaction at Panel $P_{drift} = 0.4\ \frac{k}{ft}$

$$P_{dlr} := x_g \cdot DL \cdot \frac{w_{rf}}{2}$$

Roof Dead Load $P_{dlr} = 0.7\ \frac{k}{ft}$

$$P_{dlp} := x_g \cdot 150\ pcf \cdot \left(\frac{ht}{2} + h_p\right) \cdot t$$

Panel Dead Load $P_{dlp} = 3.6\ \frac{k}{ft}$

$$P_{sl} := x_g \cdot \left(SL \cdot \frac{w_{rf}}{2} + P_{drift}\right)$$

Total Snow Load $P_{sl} = 1.3\ \frac{k}{ft}$

$$P_{dl\_mezz} := 0\ \frac{k}{ft}$$

Mezzanine Dead Load $P_{dl\_mezz} = 0\ \frac{k}{ft}$

$$P_{ll\_mezz} := 0\ \frac{k}{ft}$$

Mezzanine Live Load $P_{ll\_mezz} = 0\ \frac{k}{ft}$

$$M_w := x_{OOP} \cdot \frac{p \cdot ht^2}{8}$$

Moment Due To Wind $M_w = 3.6\ \frac{k \cdot ft}{ft}$

$$w_1 := F_p \cdot 150\ pcf \cdot t \cdot x_{OOP}$$

$$w_2 := \left(F_p \cdot 10 \cdot psf \cdot w_o \cdot 0.5 + F_p \cdot 150\ pcf \cdot t \cdot L\right) \cdot \frac{1}{L}$$

$$R_{bot} := \frac{w_2 \cdot ht}{2} + \frac{\left(w_1 - w_2\right) \cdot \left(ht - ht_o\right)^2}{2 \cdot ht}$$

$$R_{top} := \left(w_2 \cdot ht_o + w_1 \cdot \left(ht - ht_o\right)\right) - R_{bot}$$

$$x_m := \frac{R_{top}}{w_1}$$

$$M_s := R_{top} \cdot x_m - \frac{w_1 \cdot {x_m}^2}{2}$$

Moment Due to Seismic w/ Opening $M_s = 8.2\ \frac{k \cdot ft}{ft}$

Canopy Loading

$$P_{dl\_c} := \frac{170\ lbf}{L}$$

Canopy Dead Load $P_{dl\_c} = 0\ \frac{k}{ft}$

$M_{dl\_c} := \frac{316 \cdot lbf \cdot ft}{L}$ **Canopy Dead Load (Moment)** $M_{dl\_c} = 0 \ \frac{k \cdot ft}{ft}$

$P_{sl\_c} := \frac{600 \ lbf}{L}$ **Canopy Snow Load** $P_{sl\_c} = 0.1 \ \frac{k}{ft}$

$M_{sl\_c} := \frac{930 \cdot lbf \cdot ft}{L}$ **Canopy Snow Load (Moment)** $M_{sl\_c} = 0.1 \ \frac{k \cdot ft}{ft}$

$P_{w\_c} := \frac{200 \ lbf}{L}$ **Canopy Wind Load** $P_{w\_c} = 0 \ \frac{k}{ft}$

$M_{w\_c} := \frac{386 \ lbf \cdot ft}{L}$ **Canopy Wind (Moment)** $M_{w\_c} = 0.1 \ \frac{k \cdot ft}{ft}$

Strength Load Combinations (LRFD)

(ASCE 7-16 2.3)

$$x := \begin{bmatrix} 1.2 \cdot (P_{dlr} + P_{dl\_mezz}) & 1.2 \cdot P_{dlp} & 1.6 \cdot P_{sl} + 0.5 \ P_{ll\_mezz} & 0 \ \frac{k \cdot ft}{ft} & 1.2 \cdot P_{dl\_c} & 1.2 \cdot M_{dl\_c} & 1.6 \cdot P_{sl\_c} & 1.6 \cdot M_{sl\_c} & 0 \ \frac{k}{ft} & 0.5 \cdot M_{w\_c} \\ 1.2 \cdot (P_{dlr} + P_{dl\_mezz}) & 1.2 \cdot P_{dlp} & 0.5 \cdot (P_{sl} + P_{ll\_mezz}) & 1.0 \cdot M_w & 1.2 \cdot P_{dl\_c} & 1.2 \cdot M_{dl\_c} & 0.5 \cdot P_{sl\_c} & 0.5 \cdot M_{sl\_c} & 1.0 \cdot P_{w\_c} & 1.0 \cdot M_{w\_c} \\ 1.2 \cdot (P_{dlr} + P_{dl\_mezz}) & 1.2 \cdot P_{dlp} & 0.5 \ P_{ll\_mezz} + 0.2 \cdot P_{sl} & 1.0 \cdot M_s & 1.2 \cdot P_{dl\_c} & 1.2 \cdot M_{dl\_c} & 0.2 \cdot P_{sl\_c} & 0.2 \cdot M_{sl\_c} & 0 \ \frac{k}{ft} & 0 \ \frac{k \cdot ft}{ft} \\ 0.9 \cdot (P_{dlr} + P_{dl\_mezz}) & 0.9 \cdot P_{dlp} & 0 \cdot \frac{k}{ft} & 1.0 \cdot M_w & 0.9 \cdot P_{dl\_c} & 0.9 \cdot M_{dl\_c} & 0 \cdot \frac{k}{ft} & 0 \cdot \frac{k \cdot ft}{ft} & 1.0 \cdot P_{w\_c} & 1.0 \cdot M_{w\_c} \\ 0.9 \cdot (P_{dlr} + P_{dl\_mezz}) & 0.9 \cdot P_{dlp} & 0 \cdot \frac{k}{ft} & 1.0 \cdot M_s & 0.9 \cdot P_{dl\_c} & 0.9 \cdot M_{dl\_c} & 0 \cdot \frac{k}{ft} & 0 \cdot \frac{k \cdot ft}{ft} & 0 \ \frac{k}{ft} & 0 \cdot \frac{k \cdot ft}{ft} \end{bmatrix}$$

Service/Deflection Load Combinations (ASD)

(2021 IBC 1605.2)

$\omega := 1.0$ $i := 0 .. \text{rows}(x) - 1$

$$y := \begin{bmatrix} 1.0 \cdot (P_{dlr} + P_{dl\_mezz}) & 1.0 \cdot P_{dlp} & P_{sl} + P_{ll\_mezz} & \frac{\omega \cdot 0.6 \cdot M_w}{2} & 1.0 \cdot P_{dl\_c} & 1.0 \cdot M_{dl\_c} & 1.0 \cdot P_{sl\_c} & 1.0 \cdot M_{sl\_c} & \frac{\omega \cdot 0.6 \cdot P_{w\_c}}{2} & \frac{\omega \cdot 0.6 \cdot M_{w\_c}}{2} \\ 1.0 \cdot (P_{dlr} + P_{dl\_mezz}) & 1.0 \cdot P_{dlp} & P_{ll\_mezz} + 0.5 \cdot P_{sl} & \omega \cdot 0.6 \cdot M_w & 1.0 \cdot P_{dl\_c} & 1.0 \cdot M_{dl\_c} & 0.5 \cdot P_{sl\_c} & 0.5 \cdot M_{sl\_c} & \omega \cdot 0.6 \cdot P_{w\_c} & \omega \cdot 0.6 \cdot M_{w\_c} \\ 1.0 \cdot (P_{dlr} + P_{dl\_mezz}) & 1.0 \cdot P_{dlp} & P_{sl} + P_{ll\_mezz} & \left(\frac{1}{1.4}\right) \cdot M_s & 1.0 \cdot P_{dl\_c} & 1.0 \cdot M_{dl\_c} & 1.0 \cdot P_{sl\_c} & 1.0 \cdot M_{sl\_c} & 0 \ \frac{k}{ft} & 0 \cdot \frac{k \cdot ft}{ft} \\ 0.67 \cdot (P_{dlr} + P_{dl\_mezz}) & 0.67 \cdot P_{dlp} & 0 \cdot \frac{k}{ft} & \omega \cdot 0.6 \cdot M_w & 0.67 \cdot P_{dl\_c} & 0.67 \cdot M_{dl\_c} & 0 \cdot \frac{k}{ft} & 0 \cdot \frac{k \cdot ft}{ft} & \omega \cdot 0.6 \cdot P_{w\_c} & \omega \cdot 0.6 \cdot M_{w\_c} \\ 0.9 \cdot (P_{dlr} + P_{dl\_mezz}) & 0.9 \cdot P_{dlp} & 0 \cdot \frac{k}{ft} & \left(\frac{1}{1.4}\right) \cdot M_s & 0.9 \cdot P_{dl\_c} & 0.9 \cdot M_{dl\_c} & 0 \cdot \frac{k}{ft} & 0 \cdot \frac{k \cdot ft}{ft} & 0 \ \frac{k}{ft} & 0 \ \frac{k \cdot ft}{ft} \end{bmatrix}$$

Define Panel Reinforcing

Concrete Panel Out-of-Plane Analysis

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

$NB := 14$ Vertical Bar Spacing

$rebar\_location := EF$ Steel Layer: Each Face (EF) or Centerline (CL)

$stirrup\_size := NA$ Stirrup Bar Size

$A_b := A_{bar\_size}$ Vertical Bar Area

$diam_b := D_{bar\_size}$ Vertical Bar Diameter

$diam_{str} := D_{stirrup\_size}$ Stirrup Bar Diameter

$d_{ef} := t - \frac{diam_b}{2} - diam_{str} - clr - r_v$ Steel Depth If Each Face

$d_{cl} := \frac{t}{2} - r_v$ Steel Depth If Centerline

$d := \text{if}\left(rebar\_location = 0, d_{ef}, d_{cl}\right)$ Steel Depth $d = 6.19\ in$

$S := \frac{L - 1.5\ in}{NB - 1} = 7\ in$ Vertical Bar Spacing OC

$flag := \text{if}\left(S \ge 3\ in, \text{“OK”}, \text{“Revise”}\right)$ $S = 7\ in$ $flag = \text{“OK”}$

$A_s := \frac{NB \cdot A_b}{L} = 0.57\ \frac{in^2}{ft}$ Vertical Area of Steel

Check Axial Capacity (ACI 318-19 11.8.1.1(d))

$$P_{u_i} := \left\| \begin{array}{l} P_u \leftarrow 0 \\ \left(x_{i,0} + x_{i,1} + x_{i,2} + x_{i,4} + x_{i,6} + x_{i,8}\right) \end{array} \right. \qquad P_a := 0.06 \cdot f_c \cdot t$$

$flag_i := \text{if}\left(\frac{P_{u_i}}{P_a} \le 1, \text{“OK”}, \text{“Revise”}\right)$ $\frac{P_u}{P_a} = \begin{bmatrix} 0.3 \\ 0.3 \\ 0.2 \\ 0.2 \\ 0.2 \end{bmatrix}$ $flag = \begin{bmatrix} \text{“OK”} \\ \text{“OK”} \\ \text{“OK”} \\ \text{“OK”} \\ \text{“OK”} \end{bmatrix}$

$compression\_check := flag$

Determine Moment Capacity

$\beta_1 := 0.85 - 0.05 \cdot \left(\frac{f_c - 4000 \cdot psi}{1000 \cdot psi}\right)$ $P_u := P_u \cdot ft$ $A_s := A_s \cdot ft$

$$X_i := \frac{P_{u_i} + A_s \cdot f_y}{0.85 \cdot f_c \cdot ft} \qquad M_{n_i} := \left(A_s \cdot f_y + P_{u_i}\right) \cdot \left(d - \frac{X_i}{2}\right) - P_{u_i} \cdot \left(d - \frac{t}{2}\right)$$

$$\phi M_{n_i} := 0.9 \cdot M_{n_i}$$ Factored Moment Capacity $$\phi M_n = \begin{bmatrix} 16.4 \\ 16.1 \\ 16 \\ 15.6 \\ 15.6 \end{bmatrix} k \cdot ft$$

Check for Tension-Controlled Section

(ACI 318-19 11.8.1.1(b))

$$C_i := \frac{X_i}{\beta_1} \qquad flag_i := \text{if}\left(\frac{C_i}{d} < 0.375, \text{“OK”}, \text{“Revise”}\right)$$

$$tension\_controlled := flag$$

$$\frac{C}{d} = \begin{bmatrix} 0.193 \\ 0.186 \\ 0.184 \\ 0.176 \\ 0.176 \end{bmatrix} \qquad flag = \begin{bmatrix} \text{“OK”} \\ \text{“OK”} \\ \text{“OK”} \\ \text{“OK”} \\ \text{“OK”} \end{bmatrix}$$

Determine Deflection Limits

$$\Delta_{max} := \frac{ht}{150}$$ Allowable Panel Deflection

$$ht' := \text{if}\left(DockHigh = 0, ht, 0.8 \cdot ht\right) = 36.3\ ft$$ Adjust Span per Dock High Condition

$$E_c := 57000 \cdot \sqrt{\frac{f_c}{psi}} \cdot psi \qquad n := \frac{29000 \cdot ksi}{E_c} \qquad A_{se_i} := A_s + \frac{P_{u_i} \cdot t}{2 \cdot f_y \cdot d} \qquad C_i := \frac{X_i}{0.85}$$

$$f_r := 7.5 \cdot \sqrt{\frac{f_c}{psi}} \cdot psi \qquad I_g := \frac{(1\ ft) \cdot t^3}{12} \qquad I_{cr_i} := \frac{12 \cdot in \cdot \left(C_i\right)^3}{3} + n \cdot A_{se_i} \cdot \left(d - C_i\right)^2$$

$$M_{cr} := \frac{2 \cdot f_r \cdot I_g}{t} \qquad \Delta_{cr} := \frac{5 \cdot M_{cr} \cdot ht'^2}{48 \cdot E_c \cdot I_g} \qquad \Delta_{n_i} := \frac{5 \cdot M_{n_i} \cdot ht'^2}{48 \cdot E_c \cdot I_{cr_i}}$$

Check Moment Capacity

(ACI 318-19 11.8.3.1)

$$M_{ua_i} := \left(x_{i,3} + \left(x_{i,0} + x_{i,2}\right) \cdot \frac{e}{2} + x_{i,5} + x_{i,7} + x_{i,9}\right) \cdot ft \qquad M_{u_i} := \frac{M_{ua_i}}{1 - \frac{5 \cdot P_{u_i} \cdot ht'^2}{0.75 \cdot 48 \cdot E_c \cdot I_{cr_i}}}$$

$$flag_i := \text{if}\left(\frac{M_{u_i}}{\phi M_{n_i}} \le 1, \text{“OK”}, \text{“Revise”}\right)$$

$$capacity\_check := flag$$

$$\frac{M_u}{\phi M_n} = \begin{bmatrix} 0.1 \\ 0.4 \\ 0.8 \\ 0.3 \\ 0.7 \end{bmatrix} \qquad flag = \begin{bmatrix} \text{“OK”} \\ \text{“OK”} \\ \text{“OK”} \\ \text{“OK”} \\ \text{“OK”} \end{bmatrix}$$

Concrete Panel Out-of-Plane Analysis

Check Deflection

(ACI 318-19 11.8.4)

$$M_{a_i} := \left(y_{i,3} + \left(y_{i,0} + y_{i,2}\right) \cdot \frac{e}{2} + y_{i,5} + y_{i,7} + y_{i,9}\right) ft \qquad P_{s_i} := \left(y_{i,0} + y_{i,1} + y_{i,2} + y_{i,4} + y_{i,6} + y_{i,8}\right) \cdot ft$$

Iterate Service Deflection

$\Delta_{s_i} := 0 \cdot in$ Initial Guess

$\Delta_s := \Delta f_s\left(M_a, P_s, \Delta_s, M_{cr}, \Delta_{cr}, M_n, \Delta_n\right)$ Function iterates service deflections per ACI 318-19 *Sec 11.8.4*

$$flag_i := \text{if}\left(\frac{\Delta_{s_i}}{\Delta_{max}} \le 1, \text{“OK”}, \text{“Revise”}\right)$$

$$\frac{\Delta_s}{\Delta_{max}} = \begin{bmatrix} 0.08 \\ 0.12 \\ 0.91 \\ 0.11 \\ 0.76 \end{bmatrix} \qquad flag = \begin{bmatrix} \text{“OK”} \\ \text{“OK”} \\ \text{“OK”} \\ \text{“OK”} \\ \text{“OK”} \end{bmatrix}$$

$deflection\_check := flag$

Results Summary

Panel Thickness $t = 8\ in$ **Bar Size** $rebar_{bar\_size} = \text{“\#5”}$

Panel Leg Width $L = 7.7\ ft$ **Bar Quantity** $NB = 14$

Bar Location $layer = \text{“EF”}$

Concrete Panel Out-of-Plane Analysis

Purpose Statement: Out-of-Plane (OOP) Design of Concrete Tilt Up Panels

Referenced Standards: 2021 IBC
ASCE 7-16
ACI 318-19

General Design Criteria

$r_v := 0.75\ in$	Reveal Depth	Inputs
		Verify
$clr := 0.75\ in$	Concrete Clear Cover	Outputs
$f_c := 4000 \cdot psi$	Concrete Compressive Strength	
$f_y := 60 \cdot ksi$	Reinforcement Yield Strength	
$Risk := \text{II}$	Risk Category	

General Loading

Gravity

$DL := 14.9 \cdot psf$	Roof Dead Load
$SL := 14 \cdot psf$	Roof Snow Load

Wind (ASCE 7-16 30.4)

$S_w := 97 \cdot mph$	Mean Wind Speed
$Exposure := \text{B}$	Exposure Category
$h_{mean} := 37\ ft$	Mean Roof Height
$K_z := 0.74$	*(Table 26.10-1)*
$K_{zt} := 1.0$	*(Section 26.8)*
$K_d := 0.85$	*(Table 26.6-1)*
$K_e := 1.0$	*(Table 26.9-1)*

$$q_h := 0.00256\ K_{zt}\ K_z\ K_d\ K_e \left(\frac{S_w}{mph}\right)^2 \cdot 1\ psf = 15.15\ psf$$

Concrete Panel Out-of-Plane Analysis

Seismic (ASCE 7-16 12.11.1)

$I_e = 1$ Importance Factor - Seismic (Table 1.5-2)

$S_{DS} := 0.843$ Design Spectral Acceleration

$F_p := \max\left(0.4 \cdot S_{DS} \cdot I_e, 0.1\right) = 0.34$ Seismic OOP coefficient

green shading is done

West Elevation

Solid Panel - Girder Loading (P55 @ GL C) - 8.75" THK 10' Eff Width, (20) #5 VERT EF

Panel Geometry

$t := 8.75\ in$ — Panel Thickness

$ht := 39.25 \cdot ft$ — Roof Height (Panel Span)

$h_p := 1.25 \cdot ft$ — Parapet Height

$DockHigh :=$ No — Dock High Span Reduction

$w := 26.5 \cdot ft$ — Panel Width

$e := \frac{t}{2} + 3 \cdot in = 7.38\ in$ — Roof Load Eccentricity

$h_T := ht + h_p = 40.5\ ft$ — Total Panel Height

Panel Load Factors

$L := 10\ ft$ — Panel Width Considered (PWC) for girder/pad ftg

$x_g := 1.0$ — Gravity Factor

$x_{OOP} := 1.0$ — OOP Factor

Panel Wind Loading Per ASCE 7-16 Chapter 30 - Part 1 Method

$GCpi := 0.18$ — Internal Pressure Coefficient per Table 26.13-1

$A_{eff} := \frac{1}{3}\ ht^2 = 513.5\ ft^2$

$Slope :=$ Y — If roof slope is <10 degrees, GCp can be reduced by 10%

$$GCp_{poscalc} := \left(1.0 - \frac{\ln\left(\frac{A_{eff}}{ft^2}\right) - \ln(10)}{\ln(500) - \ln(10)} (1 - 0.7)\right)(1 - Slope)$$

Positive pressure coefficient (zones 4 and 5)

$$\left(\quad \ln\left(\frac{A_{eff}}{\ }\right) - \ln(10) \quad \right)$$

$$GCp_{negcalc} := \left(1.4 - \frac{\left(ft^2\right)}{\ln(500) - \ln(10)}(1.4 - 0.8)\right)(1 - Slope)(-1)$$

Negative pressure coefficient (zone 5)

$$GCp_{pos} := \begin{array}{l} \text{if } A_{eff} > 500\ ft^2 \\ \quad \| 0.7\ (1 - Slope) \\ \text{else if } A_{eff} < 10\ ft^2 \\ \quad \| 1.0\ (1 - Slope) \\ \text{else} \\ \quad \| GCp_{poscalc} \end{array}$$

Check against upper and lower bound values for CGp based on Aeff

$$GCp_{neg} := \begin{array}{l} \text{if } A_{eff} > 500\ ft^2 \\ \quad \| -0.8\ (1 - Slope) \\ \text{else if } A_{eff} < 10\ ft^2 \\ \quad \| -1.4\ (1 - Slope) \\ \text{else} \\ \quad \| GCp_{negcalc} \end{array}$$

$$p_1 := q_h \cdot (GCp_{pos} - GCpi)$$

$$p_2 := q_h \cdot (GCp_{neg} - GCpi)$$

$$p_3 := q_h \cdot (GCp_{pos} + GCpi)$$

$$p_4 := q_h \cdot (GCp_{neg} + GCpi)$$

$$p := \max(|p_1|, |p_2|, |p_3|, |p_4|) = 13.6\ psf$$

Panel Loading

$w_{grdr} := 52\ ft$ Girder Span

$s_{grdr} := 59.33\ ft$ Girder Tributary Width

$load_{drift} := 51\ psf$ Drift Load at Girder Panel

$w_{drift} := 24.5\ ft$ Drift Length at Girder Panel

$f_{grdr} := 1.0$ Portion of Girder Load on Panel Leg

$x_{grdr} := f_{grdr} \cdot \frac{s_{grdr}}{L} = 5.9$ Girder Factor

Concrete Panel Out-of-Plane Analysis

$s_{jst} := 10\ ft + 0\ in$ Joist Spacing/Tributary Width

$$P_{drift\_1} := \frac{load_{drift} \cdot s_{jst}}{2} \cdot \left(1 - \frac{s_{jst}}{4\ w_{drift}}\right)$$ Drift Load on Panel

$$P_{drift\_2} := \frac{load_{drift} \cdot w_{drift}}{2} - P_{drift\_1}$$ Girder Reaction from Drift

$w_{jst} := 59.33\ ft$ Joist Span

$load_{drift\_jst} := 0\ psf$ Drift Load at Joist Panel

$w_{drift\_jst} := 0\ ft$ Drift Length at Joist Panel

$$P_{drift\_3} := \frac{w_{grdr}}{2\ L}\left(\frac{load_{drift\_jst} \cdot w_{drift\_jst}^{2}}{6 \cdot w_{jst}}\right)$$

$$P_{sl\_1} := x_g \cdot \left(SL \cdot \frac{s_{jst}}{2} + P_{drift\_1}\right)$$

$$P_{sl\_2} := x_{grdr} \cdot \left(SL \cdot \left(\frac{w_{grdr} - s_{jst}}{2}\right) + P_{drift\_2}\right) + f_{grdr} \cdot P_{drift\_3}$$

$P_{sl} := P_{sl\_1} + P_{sl\_2}$ **Total Snow Load** $P_{sl} = 4.4\ \frac{k}{ft}$

$$P_{dlr} := x_g \cdot DL \cdot \frac{s_{jst}}{2} + x_{grdr} \cdot DL \cdot \left(\frac{w_{grdr} - s_{jst}}{2}\right)$$ **Roof Dead Load** $P_{dlr} = 1.9\ \frac{k}{ft}$

$$P_{dlp} := x_g \cdot t \cdot 150\ pcf \cdot \left(\frac{ht}{2} + h_p\right)$$ **Panel Dead Load** $P_{dlp} = 2.3\ \frac{k}{ft}$

$$M_w := x_{OOP} \cdot \frac{p \cdot ht^2}{8}$$ **Moment Due To Wind** $M_w = 2.6\ \frac{k \cdot ft}{ft}$

$$M_s := x_{OOP} \cdot \frac{F_p \cdot t \cdot 150\ pcf \cdot ht^2}{8}$$ **Moment Due to Seismic** $M_s = 7.1\ \frac{k \cdot ft}{ft}$

Strength Load Combinations (LRFD)

(IBC 2018 1605.2)

Service/Deflection Load Combinations (ASD)

(2021 IBC 1605.2)

$\omega := 1.0$

$$x := \begin{bmatrix} 1.2 \cdot P_{dlr} & 1.2 \cdot P_{dlp} & 1.6 \cdot P_{sl} & 0.5 \cdot M_w \\ 1.2 \cdot P_{dlr} & 1.2 \cdot P_{dlp} & 0.5 \cdot P_{sl} & 1.0 \cdot M_w \\ 1.2 \cdot P_{dlr} & 1.2 \cdot P_{dlp} & 0.2 \cdot P_{sl} & 1.0 \cdot M_s \\ 0.9 \cdot P_{dlr} & 0.9 \cdot P_{dlp} & 0 \cdot \frac{k}{ft} & 1.0 \cdot M_w \\ 0.9 \cdot P_{dlr} & 0.9 \cdot P_{dlp} & 0 \cdot \frac{k}{ft} & 1.0 \cdot M_s \end{bmatrix}$$

$$y := \begin{bmatrix} 1.0 \cdot P_{dlr} & 1.0 \cdot P_{dlp} & 1.0 \cdot P_{sl} & 0.5 \cdot (\omega \cdot 0.6 \cdot M_w) \\ 1.0 \cdot P_{dlr} & 1.0 \cdot P_{dlp} & 0.5 \cdot P_{sl} & 1.0 \cdot (\omega \cdot 0.6 \cdot M_w) \\ 1.0 \cdot P_{dlr} & 1.0 \cdot P_{dlp} & 1.0 \cdot P_{sl} & \left(\frac{1}{1.4}\right) \cdot M_s \\ 0.67 \cdot P_{dlr} & 0.67 \cdot P_{dlp} & 0 \cdot \frac{k}{ft} & 1.0 \cdot (\omega \cdot 0.6 \cdot M_w) \\ 0.9 \cdot P_{dlr} & 0.9 \cdot P_{dlp} & 0 \cdot \frac{k}{ft} & \left(\frac{1}{1.4}\right) \cdot M_s \end{bmatrix}$$

$i := 0 .. \text{rows}(x) - 1$

Define Panel Reinforcing

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

$NB := 20$ — Vertical Bar Spacing

$S := \frac{L - 1.5\ in}{NB - 1} = 6.2\ in$

$flag := \text{if}(S > 3\ in, \text{"OK"}, \text{"Revise"}) = \text{"OK"}$

$rebar\_location := \text{EF}$ — Steel Layer: Each Face (EF) or Centerline (CL)

$stirrup\_size := \text{NA}$ — Stirrup Bar Size

$A_b := A_{bar\_size}$ — Vertical Bar Area

$diam_b := D_{bar\_size}$ — Vertical Bar Diameter

$diam_{str} := D_{stirrup\_size}$ — Stirrup Bar Diameter

$d_{ef} := t - \frac{diam_b}{2} - diam_{str} - clr - r_v$ — Steel Depth If Each Face

$d_{cl} := \frac{t}{2} - r_v$ — Steel Depth If Centerline

$d := \text{if}(rebar\_location = 0, d_{ef}, d_{cl})$ — Steel Depth — $d = 6.94\ in$

$A_s := \frac{NB \cdot A_b}{L}$ — Vertical Area of Steel

Check Axial Capacity

(ACI 318-19 11.8.1.1(d))

$$P_{u_i} := \left\| \begin{array}{l} P_u \leftarrow 0 \\ \left(x_{i,0} + x_{i,1} + x_{i,2}\right) \end{array} \right.$$

$P_a := 0.06 \cdot f_c \cdot t$

Concrete Panel Out-of-Plane Analysis

$$flag_i := \text{if}\left(\frac{P_{u_i}}{P_a} \le 1, \text{“OK”}, \text{“Revise”}\right)$$

$$compression\_check := flag$$

$$\frac{P_u}{P_a} = \begin{bmatrix} 0.5 \\ 0.3 \\ 0.2 \\ 0.2 \\ 0.2 \end{bmatrix} \qquad flag = \begin{bmatrix} \text{“OK”} \\ \text{“OK”} \\ \text{“OK”} \\ \text{“OK”} \\ \text{“OK”} \end{bmatrix}$$

Determine Moment Capacity

$$\beta_1 := 0.85 - 0.05 \cdot \left(\frac{f_c - 4000 \cdot psi}{1000 \cdot psi}\right) \qquad P_u := P_u \cdot ft \qquad A_s := A_s \cdot ft$$

$$X_i := \frac{P_{u_i} + A_s \cdot f_y}{0.85 \cdot f_c \cdot ft} \qquad M_{n_i} := \left(A_s \cdot f_y + P_{u_i}\right) \cdot \left(d - \frac{X_i}{2}\right) - P_{u_i} \cdot \left(d - \frac{t}{2}\right)$$

$$\phi M_{n_i} := 0.9 \cdot M_{n_i}$$

Factored Moment Capacity

$$\phi M_n = \begin{bmatrix} 21.1 \\ 19.9 \\ 19.6 \\ 19.1 \\ 19.1 \end{bmatrix} k \cdot ft$$

Check for Tension-Controlled Section

(ACI 318-19 11.8.1.1(b))

$$C_i := \frac{X_i}{\beta_1} \qquad flag_i := \text{if}\left(\frac{C_i}{d} < 0.375, \text{“OK”}, \text{“Revise”}\right)$$

$$tension\_controlled := flag$$

$$\frac{C}{d} = \begin{bmatrix} 0.205 \\ 0.185 \\ 0.179 \\ 0.17 \\ 0.17 \end{bmatrix} \qquad flag = \begin{bmatrix} \text{“OK”} \\ \text{“OK”} \\ \text{“OK”} \\ \text{“OK”} \\ \text{“OK”} \end{bmatrix}$$

Determine Deflection Limits

$$\Delta_{max} := \frac{ht}{150}$$

Allowable Panel Deflection

$$ht' := \text{if}\left(DockHigh = 0, ht, 0.8 \cdot ht\right) = 39.3\ ft$$

Adjust Span per Dock High Condition

$$E_c := 57000 \cdot \sqrt{\frac{f_c}{psi}} \cdot psi \qquad n := \frac{29000 \cdot ksi}{E_c} \qquad A_{se_i} := A_s + \frac{P_{u_i} \cdot t}{2 \cdot f_y \cdot d} \qquad C_i := \frac{X_i}{0.85}$$

$$f_r := 7.5 \cdot \sqrt{\frac{f_c}{psi}} \cdot psi \qquad I_g := \frac{(1\ ft) \cdot t^3}{12} \qquad I_{cr_i} := \frac{12 \cdot in \cdot \left(C_i\right)^3}{3} + n \cdot A_{se_i} \cdot \left(d - C_i\right)^2$$

$$M_{cr} := \frac{2 \cdot f_r \cdot I_g}{t} \qquad \Delta_{cr} := \frac{5 \cdot M_{cr} \cdot ht'^2}{48 \cdot E_c \cdot I_g} \qquad \Delta_{n_i} := \frac{5 \cdot M_{n_i} \cdot ht'^2}{48 \cdot E_c \cdot I_{cr_i}}$$

Check Moment Capacity

(ACI 318-19 11.8.3.1)

$$M_{ua_i} := \left(x_{i,3} + \left(x_{i,0} + x_{i,2}\right) \cdot \frac{e}{2}\right) \cdot ft \qquad M_{u_i} := \frac{M_{ua_i}}{5 \cdot P_{u_i} \cdot ht'^2}$$

$$1-\frac{\quad}{0.75 \cdot 48 \cdot E_c \cdot I_{cr_i}}$$

$$flag_i := \text{if}\left(\frac{M_{u_i}}{\phi M_{n_i}} \le 1, \text{"OK"}, \text{"Revise"}\right)$$

$$\frac{M_u}{\phi M_n} = \begin{bmatrix} 0.4 \\ 0.3 \\ 0.6 \\ 0.2 \\ 0.5 \end{bmatrix} \qquad flag = \begin{bmatrix} \text{"OK"} \\ \text{"OK"} \\ \text{"OK"} \\ \text{"OK"} \\ \text{"OK"} \end{bmatrix}$$

$$capacity\_check := flag$$

Check Deflection (ACI 318-19 11.8.4)

$$M_{a_i} := \left(y_{i,3} + \left(y_{i,0} + y_{i,2}\right) \cdot \frac{e}{2}\right) ft \qquad P_{s_i} := \left(y_{i,0} + y_{i,1} + y_{i,2}\right) \cdot ft$$

Iterate Service Deflection

$\Delta_{s_i} := 0 \cdot in$ Initial Guess

$\Delta_s := \Delta f_s\left(M_a, P_s, \Delta_s, M_{cr}, \Delta_{cr}, M_n, \Delta_n\right)$ Function iterates service deflections per ACI 318-19 *Sec 11.8.4*

$$flag_i := \text{if}\left(\frac{\Delta_{s_i}}{\Delta_{max}} \le 1, \text{"OK"}, \text{"Revise"}\right)$$

$$\frac{\Delta_s}{\Delta_{max}} = \begin{bmatrix} 0.1 \\ 0.1 \\ 0.7 \\ 0.1 \\ 0.4 \end{bmatrix} \qquad flag = \begin{bmatrix} \text{"OK"} \\ \text{"OK"} \\ \text{"OK"} \\ \text{"OK"} \\ \text{"OK"} \end{bmatrix}$$

$$deflection\_check := flag$$

Results Summary

Panel Thickness $t = 8.8\ in$ **Bar Size** $rebar_{bar\_size} = \text{"\#5"}$ **Bar Spacing** $NB = 20$

Bar Location $layer = \text{"EF"}$

Concrete Panel Out-of-Plane Analysis

Solid Panel - Girder Loading (P57 @ GL D) - 8.75" THK 10'-0" Eff Width, (20) #5 VERT EF

Panel Geometry

$t := 8.75\ in$ — Panel Thickness

$ht := 38 \cdot ft$ — Roof Height (Panel Span)

$h_p := 2.5 \cdot ft$ — Parapet Height

$DockHigh := No$ — Dock High Span Reduction

$w := 26.5 \cdot ft$ — Panel Width

$e := \frac{t}{2} + 3 \cdot in = 7.38\ in$ — Roof Load Eccentricity

$h_T := ht + h_p = 40.5\ ft$ — Total Panel Height

Panel Load Factors

$L := 10\ ft + 0\ in$ — Panel Width Considered (PWC) for girder/pad ftg

$x_g := 1.0$ — Gravity Factor

$x_{OOP} := 1.0$ — OOP Factor

Panel Wind Loading Per ASCE 7-16 Chapter 30 - Part 1 Method

$GCpi := 0.18$ — Internal Pressure Coefficient per Table 26.13-1

$$A_{eff} := \frac{1}{3}\ ht^2 = 481.3\ ft^2$$

$Slope := Y$ — If roof slope is <10 degrees, GCp can be reduced by 10%

$$GCp_{poscalc} := \left(1.0 - \frac{\ln\left(\frac{A_{eff}}{ft^2}\right) - \ln(10)}{\ln(500) - \ln(10)} (1 - 0.7)\right)(1 - Slope)$$

Positive pressure coefficient (zones 4 and 5)

$$GCp_{negcalc} := \left(1.4 - \frac{\ln\left(\frac{A_{eff}}{ft^2}\right) - \ln(10)}{\ldots} (1.4 - 0.8)\right)(1 - Slope)(-1)$$

Negative pressure coefficient

$\ln(500) - \ln(10)$ (zone 5)

$$GCp_{pos} := \text{if } A_{eff} > 500\ ft^2 \;\|\; 0.7\,(1 - Slope) \;\text{else if } A_{eff} < 10\ ft^2 \;\|\; 1.0\,(1 - Slope) \;\text{else } \|\; GCp_{poscalc}$$

$$GCp_{neg} := \text{if } A_{eff} > 500\ ft^2 \;\|\; -0.8\,(1 - Slope) \;\text{else if } A_{eff} < 10\ ft^2 \;\|\; -1.4\,(1 - Slope) \;\text{else } \|\; GCp_{negcalc}$$

Check against upper and lower bound values for CGp based on Aeff

$$p_1 := q_h \cdot (GCp_{pos} - GCpi)$$

$$p_2 := q_h \cdot (GCp_{neg} - GCpi)$$

$$p_3 := q_h \cdot (GCp_{pos} + GCpi)$$

$$p_4 := q_h \cdot (GCp_{neg} + GCpi)$$

$$p := \max(|p_1|, |p_2|, |p_3|, |p_4|) = 13.7\ psf$$

Panel Loading

$w_{grdr} := 52\ ft$ Girder Span

$s_{grdr} := 59.33\ ft$ Girder Tributary Width

$load_{drift} := 28\ psf$ Drift Load at Girder Panel

$w_{drift} := 13.25\ ft$ Drift Length at Girder Panel

$f_{grdr} := 1.0$ Portion of Girder Load on Panel Leg

$x_{grdr} := f_{grdr} \cdot \dfrac{s_{grdr}}{L} = 5.9$ Girder Factor

$s_{jst} := 10\ ft + 0\ in$ Joist Spacing/Tributary Width

$P_{drift\_1} := \frac{load_{drift} \cdot s_{jst}}{2} \cdot \left(1 - \frac{s_{jst}}{4\ w_{drift}}\right)$ Drift Load on Panel

$P_{drift\_2} := \frac{load_{drift} \cdot w_{drift}}{2} - P_{drift\_1}$ Girder Reaction from Drift

$w_{jst} := 60\ ft$ Joist Span

$load_{drift\_jst} := 59\ psf$ Drift Load at Joist Panel

$w_{drift\_jst} := 14.25\ ft$ Drift Length at Joist Panel

$P_{drift\_3} := \frac{w_{grdr}}{2\ L} \left(\frac{load_{drift\_jst} \cdot w_{drift\_jst}^{2}}{6 \cdot w_{jst}}\right)$

$P_{sl\_1} := x_g \cdot \left(SL \cdot \frac{s_{jst}}{2} + P_{drift\_1}\right)$

$P_{sl\_2} := x_{grdr} \cdot \left(SL \cdot \left(\frac{w_{grdr} - s_{jst}}{2}\right) + P_{drift\_2}\right) + f_{grdr} \cdot P_{drift\_3}$

$P_{sl} := P_{sl\_1} + P_{sl\_2}$ **Total Snow Load** $P_{sl} = 2.4\ \frac{k}{ft}$

$P_{dlr} := x_g \cdot DL \cdot \frac{s_{jst}}{2} + x_{grdr} \cdot DL \cdot \left(\frac{w_{grdr} - s_{jst}}{2}\right)$ **Roof Dead Load** $P_{dlr} = 1.9\ \frac{k}{ft}$

$P_{dlp} := x_g \cdot t \cdot 150\ pcf \cdot \left(\frac{ht}{2} + h_p\right)$ **Panel Dead Load** $P_{dlp} = 2.4\ \frac{k}{ft}$

$M_w := x_{OOP} \cdot \frac{p \cdot ht^2}{8}$ **Moment Due To Wind** $M_w = 2.5\ \frac{k \cdot ft}{ft}$

$M_s := x_{OOP} \cdot \frac{F_p \cdot t \cdot 150\ pcf \cdot ht^2}{8}$ **Moment Due to Seismic** $M_s = 6.7\ \frac{k \cdot ft}{ft}$

Strength Load Combinations (LRFD)

(ASCE 7-16 2.3)

Service/Deflection Load Combinations (ASD)

(2021 IBC 1605.2)

$\omega := 1.0$

$$x:=\begin{bmatrix}1.2\cdot P_{dlr} & 1.2\cdot P_{dlp} & 1.6\cdot P_{sl} & 0.5\cdot M_w\\ 1.2\cdot P_{dlr} & 1.2\cdot P_{dlp} & 0.5\cdot P_{sl} & 1.0\cdot M_w\\ 1.2\cdot P_{dlr} & 1.2\cdot P_{dlp} & 0.2\cdot P_{sl} & 1.0\cdot M_s\\ 0.9\cdot P_{dlr} & 0.9\cdot P_{dlp} & 0\cdot\frac{k}{ft} & 1.0\cdot M_w\\ 0.9\cdot P_{dlr} & 0.9\cdot P_{dlp} & 0\cdot\frac{k}{ft} & 1.0\cdot M_s\end{bmatrix}$$

$$y:=\begin{bmatrix}1.0\cdot P_{dlr} & 1.0\cdot P_{dlp} & 1.0\cdot P_{sl} & 0.5\cdot(\omega\cdot 0.6\cdot M_w)\\ 1.0\cdot P_{dlr} & 1.0\cdot P_{dlp} & 0.5\cdot P_{sl} & 1.0\cdot(\omega\cdot 0.6\cdot M_w)\\ 1.0\cdot P_{dlr} & 1.0\cdot P_{dlp} & 1.0\cdot P_{sl} & \left(\frac{1}{1.4}\right)\cdot M_s\\ 0.67\cdot P_{dlr} & 0.67\cdot P_{dlp} & 0\cdot\frac{k}{ft} & 1.0\cdot(\omega\cdot 0.6\cdot M_w)\\ 0.9\cdot P_{dlr} & 0.9\cdot P_{dlp} & 0\cdot\frac{k}{ft} & \left(\frac{1}{1.4}\right)\cdot M_s\end{bmatrix}$$

$i:=0..\text{rows}(x)-1$

Define Panel Reinforcing

$bar\_size:=\#5$ — Vertical Bar Size

$NB:=20$ — Vertical Bar Spacing

$$S:=\frac{L-1.5\ in}{NB-1}=6.2\ in$$

$flag:=\text{if}(S>3\ in,\text{“OK”},\text{“Revise”})=\text{“OK”}$

$rebar\_location:=EF$ — Steel Layer: Each Face (EF) or Centerline (CL)

$stirrup\_size:=NA$ — Stirrup Bar Size

$A_b:=A_{bar\_size}$ — Vertical Bar Area

$diam_b:=D_{bar\_size}$ — Vertical Bar Diameter

$diam_{str}:=D_{stirrup\_size}$ — Stirrup Bar Diameter

$$d_{ef}:=t-\frac{diam_b}{2}-diam_{str}-clr-r_v$$ Steel Depth If Each Face

$$d_{cl}:=\frac{t}{2}-r_v$$ Steel Depth If Centerline

$d:=\text{if}(rebar\_location=0,d_{ef},d_{cl})$ — Steel Depth — $d=6.94\ in$

$$A_s:=\frac{NB\cdot A_b}{L}$$ Vertical Area of Steel

Check Axial Capacity

(ACI 318-19 11.8.1.1(d))

$$P_{u_i}:=\left\|\begin{array}{l}P_u\leftarrow 0\\ \left(x_{i,0}+x_{i,1}+x_{i,2}\right)\end{array}\right.$$

$P_a:=0.06\cdot f_c\cdot t$

$flag_i := \text{if}\left(\frac{P_{u_i}}{P_a} \le 1, \text{“OK”}, \text{“Revise”}\right)$

$compression\_check := flag$

$\frac{P_u}{P_a} = \begin{bmatrix} 0.4 \\ 0.3 \\ 0.2 \\ 0.2 \\ 0.2 \end{bmatrix}$ $flag = \begin{bmatrix} \text{“OK”} \\ \text{“OK”} \\ \text{“OK”} \\ \text{“OK”} \\ \text{“OK”} \end{bmatrix}$

Determine Moment Capacity

$\beta_1 := 0.85 - 0.05 \cdot \left(\frac{f_c - 4000 \cdot psi}{1000 \cdot psi}\right)$ $P_u := P_u \cdot ft$ $A_s := A_s \cdot ft$

$X_i := \frac{P_{u_i} + A_s \cdot f_y}{0.85 \cdot f_c \cdot ft}$ $M_{n_i} := \left(A_s \cdot f_y + P_{u_i}\right) \cdot \left(d - \frac{X_i}{2}\right) - P_{u_i} \cdot \left(d - \frac{t}{2}\right)$

$\phi M_{n_i} := 0.9 \cdot M_{n_i}$ Factored Moment Capacity $\phi M_n = \begin{bmatrix} 20.4 \\ 19.7 \\ 19.5 \\ 19.1 \\ 19.1 \end{bmatrix} k \cdot ft$

Check for Tension-Controlled Section

(ACI 318-19 11.8.1.1(b))

$C_i := \frac{X_i}{\beta_1}$ $flag_i := \text{if}\left(\frac{C_i}{d} < 0.375, \text{“OK”}, \text{“Revise”}\right)$

$tension\_controlled := flag$

$\frac{C}{d} = \begin{bmatrix} 0.192 \\ 0.181 \\ 0.178 \\ 0.171 \\ 0.171 \end{bmatrix}$ $flag = \begin{bmatrix} \text{“OK”} \\ \text{“OK”} \\ \text{“OK”} \\ \text{“OK”} \\ \text{“OK”} \end{bmatrix}$

Determine Deflection Limits

$\Delta_{max} := \frac{ht}{150}$ Allowable Panel Deflection

$ht' := \text{if}\left(DockHigh = 0, ht, 0.8 \cdot ht\right) = 38\ ft$ Adjust Span per Dock High Condition

$E_c := 57000 \cdot \sqrt{\frac{f_c}{psi}} \cdot psi$ $n := \frac{29000 \cdot ksi}{E_c}$ $A_{se_i} := A_s + \frac{P_{u_i} \cdot t}{2 \cdot f_y \cdot d}$ $C_i := \frac{X_i}{0.85}$

$f_r := 7.5 \cdot \sqrt{\frac{f_c}{psi}} \cdot psi$ $I_g := \frac{(1\ ft) \cdot t^3}{12}$ $I_{cr_i} := \frac{12 \cdot in \cdot \left(C_i\right)^3}{3} + n \cdot A_{se_i} \cdot \left(d - C_i\right)^2$

$M_{cr} := \frac{2 \cdot f_r \cdot I_g}{t}$ $\Delta_{cr} := \frac{5 \cdot M_{cr} \cdot ht'^2}{48 \cdot E_c \cdot I_g}$ $\Delta_{n_i} := \frac{5 \cdot M_{n_i} \cdot ht'^2}{48 \cdot E_c \cdot I_{cr_i}}$

Check Moment Capacity

(ACI 318-19 11.8.3.1)

Concrete Panel Out-of-Plane Analysis

$$M_{ua_i} := \left(x_{i,3} + \left(x_{i,0} + x_{i,2}\right) \cdot \frac{e}{2}\right) \cdot ft$$

$$M_{u_i} := \frac{M_{ua_i}}{1 - \dfrac{5 \cdot P_{u_i} \cdot ht'^2}{0.75 \cdot 48 \cdot E_c \cdot I_{cr_i}}}$$

$$flag_i := \text{if}\left(\frac{M_{u_i}}{\phi M_{n_i}} \le 1, \text{“OK”}, \text{“Revise”}\right)$$

$$\frac{M_u}{\phi M_n} = \begin{bmatrix} 0.3 \\ 0.2 \\ 0.5 \\ 0.2 \\ 0.5 \end{bmatrix} \qquad flag = \begin{bmatrix} \text{“OK”} \\ \text{“OK”} \\ \text{“OK”} \\ \text{“OK”} \\ \text{“OK”} \end{bmatrix}$$

$capacity\_check := flag$

Check Deflection (ACI 318-19 11.8.4)

$$M_{a_i} := \left(y_{i,3} + \left(y_{i,0} + y_{i,2}\right) \cdot \frac{e}{2}\right) ft \qquad P_{s_i} := \left(y_{i,0} + y_{i,1} + y_{i,2}\right) \cdot ft$$

Iterate Service Deflection

$\Delta_{s_i} := 0 \cdot in$ Initial Guess

$\Delta_s := \Delta f_s\left(M_a, P_s, \Delta_s, M_{cr}, \Delta_{cr}, M_n, \Delta_n\right)$ Function iterates service deflections per ACI 318-19 *Sec 11.8.4*

$$flag_i := \text{if}\left(\frac{\Delta_{s_i}}{\Delta_{max}} \le 1, \text{“OK”}, \text{“Revise”}\right)$$

$$\frac{\Delta_s}{\Delta_{max}} = \begin{bmatrix} 0.1 \\ 0.1 \\ 0.5 \\ 0.1 \\ 0.4 \end{bmatrix} \qquad flag = \begin{bmatrix} \text{“OK”} \\ \text{“OK”} \\ \text{“OK”} \\ \text{“OK”} \\ \text{“OK”} \end{bmatrix}$$

$deflection\_check := flag$

Results Summary

Panel Thickness $t = 8.8\ in$ **Bar Size** $rebar_{bar\_size} = \text{“\#5”}$ **Bar Spacing** $NB = 20$

Bar Location $layer = \text{“EF”}$

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Engineer: ATT/SHA
Job #: 2220290.00
Date: 04/28/2023
Sheet #: 06.78

Concrete Panel Out-of-Plane Analysis

Solid Panel (P56)- 8.75" THK, #5 @ 10" OC VERT EF

Panel Geometry

$t := 8.75\ in$ Panel Thickness

$ht := 38.75 \cdot ft$ Roof Height (Panel Span)

$h_p := 1.75 \cdot ft$ Parapet Height

$DockHigh := \text{No}$ Dock High Span Reduction

$w := 26.5 \cdot ft$ Panel Width

$e := \frac{t}{2} + 3 \cdot in = 7.38\ in$ Roof Load Eccentricity

$h_T := ht + h_p = 40.5\ ft$ Total Panel Height

Panel Load Factors

$L := 1\ ft$ Panel Width Considered (PWC) for girder/pad ftg

$x_g := 1$ Gravity Factor

$x_{OOP} := 1.0$ OOP Factor

Panel Wind Loading Per ASCE 7-16 Chapter 30 - Part 1 Method

$GCpi := 0.18$ Internal Pressure Coefficient per Table 26.13-1

$A_{eff} := \frac{1}{3}\ ht^2 = 500.5\ ft^2$

$Slope := \text{Y}$ If roof slope is <10 degrees, GCp can be reduced by 10%

$$GCp_{poscalc} := \left(1.0 - \frac{\ln\left(\frac{A_{eff}}{ft^2}\right) - \ln(10)}{\ln(500) - \ln(10)}(1 - 0.7)\right)(1 - Slope)$$

Positive pressure coefficient (zones 4 and 5)

$$GCp_{negcalc} := \left(1.4 - \frac{\ln\left(\frac{A_{eff}}{ft^2}\right) - \ln(10)}{\ln(500) - \ln(10)}(1.4 - 0.8)\right)(1 - Slope)(-1)$$

Negative pressure coefficient (zone 5)

$$GCp_{pos} := \text{if } A_{eff} > 500\ ft^2 \;\Big\|\; 0.7\ (1 - Slope) \;\text{ else if } A_{eff} < 10\ ft^2 \;\Big\|\; 1.0\ (1 - Slope) \;\text{ else } \Big\|\; GCp_{poscalc}$$

$$GCp_{neg} := \text{if } A_{eff} > 500\ ft^2 \;\Big\|\; -0.8\ (1 - Slope) \;\text{ else if } A_{eff} < 10\ ft^2 \;\Big\|\; -1.4\ (1 - Slope) \;\text{ else } \Big\|\; GCp_{negcalc}$$

Check against upper and lower bound values for CGp based on Aeff

$$p_1 := q_h \cdot (GCp_{pos} - GCpi)$$

$$p_2 := q_h \cdot (GCp_{neg} - GCpi)$$

$$p_3 := q_h \cdot (GCp_{pos} + GCpi)$$

$$p_4 := q_h \cdot (GCp_{neg} + GCpi)$$

$$p_{net} := \max\left(|p_1|, |p_2|, |p_3|, |p_4|\right) = 13.6\ psf$$

Panel Loading

$w_{rf} := 10 \cdot ft$ Joist Span/Spacing Adjacent to Panel

$load_{drift} := 51\ psf$ Drift Load at Panel

$w_{drift} := 24.5\ ft$ Drift Length

$$P_{drift} := \text{if } w_{rf} \geq w_{drift} \;\Big\|\; P \leftarrow load_{drift} \cdot w_{drift} \cdot \left(\frac{1}{2} - \frac{w_{drift}}{6\ w_{rf}}\right) \;\text{ else if } w_{rf} < w_{drift} \;\Big\|\; P \leftarrow load_{drift} \cdot w_{rf} \cdot \left(\frac{1}{2} - \frac{w_{rf}}{6 \cdot w_{drift}}\right)$$

Snow Drift Reaction at Panel $P_{drift} = 0.2\ \frac{k}{ft}$

Concrete Panel Out-of-Plane Analysis

$P_{dlr} := x_g \cdot DL \cdot \frac{w_{rf}}{2}$ **Roof Dead Load** $P_{dlr} = 0.1\ \frac{k}{ft}$

$P_{dlp} := x_g \cdot t \cdot 150\ pcf \cdot \left(\frac{ht}{2} + h_p\right)$ **Panel Dead Load** $P_{dlp} = 2.3\ \frac{k}{ft}$

$P_{sl} := x_g \cdot \left(SL \cdot \frac{w_{rf}}{2} + P_{drift}\right)$ **Total Snow Load** $P_{sl} = 0.3\ \frac{k}{ft}$

$M_w := x_{OOP} \cdot \frac{p_{net} \cdot ht^2}{8}$ **Moment Due To Wind** $M_w = 2.6\ \frac{k \cdot ft}{ft}$

$M_s := x_{OOP} \cdot \frac{F_p \cdot t \cdot 150\ pcf \cdot ht^2}{8}$ **Moment Due to Seismic** $M_s = 6.9\ \frac{k \cdot ft}{ft}$

Strength Load Combinations (LRFD)

(ASCE 7-16 2.3)

$$x := \begin{bmatrix} 1.2 \cdot P_{dlr} & 1.2 \cdot P_{dlp} & 1.6 \cdot P_{sl} & 0.5 \cdot M_w \\ 1.2 \cdot P_{dlr} & 1.2 \cdot P_{dlp} & 0.5 \cdot P_{sl} & 1.0 \cdot M_w \\ 1.2 \cdot P_{dlr} & 1.2 \cdot P_{dlp} & 0.2 \cdot P_{sl} & 1.0 \cdot M_s \\ 0.9 \cdot P_{dlr} & 0.9 \cdot P_{dlp} & 0 \cdot \frac{k}{ft} & 1.0 \cdot M_w \\ 0.9 \cdot P_{dlr} & 0.9 \cdot P_{dlp} & 0 \cdot \frac{k}{ft} & 1.0 \cdot M_s \end{bmatrix}$$

$i := 0 .. \text{rows}(x) - 1$

Service/Deflection Load Combinations (ASD)

(2021 IBC 1605.2)

$\omega := 1.0$

$$y := \begin{bmatrix} 1.0 \cdot P_{dlr} & 1.0 \cdot P_{dlp} & 1.0 \cdot P_{sl} & 0.5 \cdot (\omega \cdot 0.6 \cdot M_w) \\ 1.0 \cdot P_{dlr} & 1.0 \cdot P_{dlp} & 0.5 \cdot P_{sl} & 1.0 \cdot (\omega \cdot 0.6 \cdot M_w) \\ 1.0 \cdot P_{dlr} & 1.0 \cdot P_{dlp} & 1.0 \cdot P_{sl} & \left(\frac{1}{1.4}\right) \cdot M_s \\ 0.67 \cdot P_{dlr} & 0.67 \cdot P_{dlp} & 0 \cdot \frac{k}{ft} & 1.0 \cdot (\omega \cdot 0.6 \cdot M_w) \\ 0.9 \cdot P_{dlr} & 0.9 \cdot P_{dlp} & 0 \cdot \frac{k}{ft} & \left(\frac{1}{1.4}\right) \cdot M_s \end{bmatrix}$$

Define Panel Reinforcing

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

$SP := 10 \cdot in$ Vertical Bar Spacing

$rebar\_location := \text{EF}$ Steel Layer: Each Face (EF) or Centerline (CL)

$stirrup\_size := \text{NA}$ Stirrup Bar Size

$A_b := A_{bar\_size}$ Vertical Bar Area

$diam_b := D_{bar\_size}$ Vertical Bar Diameter

Concrete Panel Out-of-Plane Analysis

$diam_{str} := D_{stirrup\_size}$ Stirrup Bar Diameter

$d_{ef} := t - \frac{diam_b}{2} - diam_{str} - clr - r_v$ Steel Depth If Each Face

$d_{cl} := \frac{t}{2} - r_v$ Steel Depth If Centerline

$d := \text{if}\left(rebar\_location = 0, d_{ef}, d_{cl}\right)$ Steel Depth $d = 6.94\ in$

$A_s := \frac{A_b}{SP}$ Vertical Area of Steel

Check Axial Capacity

(ACI 318-19 11.8.1.1(d))

$$P_{u_i} := \left\| \begin{array}{l} P_u \leftarrow 0 \\ \left(x_{i,0} + x_{i,1} + x_{i,2}\right) \end{array} \right.$$

$$P_a := 0.06 \cdot f_c \cdot t$$

$$flag_i := \text{if}\left(\frac{P_{u_i}}{P_a} \le 1, \text{“OK”}, \text{“Revise”}\right)$$

$$\frac{P_u}{P_a} = \begin{bmatrix} 0.1 \\ 0.1 \\ 0.1 \\ 0.1 \\ 0.1 \end{bmatrix} \qquad flag = \begin{bmatrix} \text{“OK”} \\ \text{“OK”} \\ \text{“OK”} \\ \text{“OK”} \\ \text{“OK”} \end{bmatrix}$$

$compression\_check := flag$

Determine Moment Capacity

$$\beta_1 := 0.85 - 0.05 \cdot \left(\frac{f_c - 4000 \cdot psi}{1000 \cdot psi}\right) \qquad P_u := P_u \cdot ft \qquad A_s := A_s \cdot ft$$

$$X_i := \frac{P_{u_i} + A_s \cdot f_y}{0.85 \cdot f_c \cdot ft} \qquad M_{n_i} := \left(A_s \cdot f_y + P_{u_i}\right) \cdot \left(d - \frac{X_i}{2}\right) - P_{u_i} \cdot \left(d - \frac{t}{2}\right)$$

$\phi M_{n_i} := 0.9 \cdot M_{n_i}$ Factored Moment Capacity

$$\phi M_n = \begin{bmatrix} 12.1 \\ 12 \\ 12 \\ 11.8 \\ 11.8 \end{bmatrix} k \cdot ft$$

Check for Tension-Controlled Section

(ACI 318-19 11.8.1.1(b))

$$C_i := \frac{X_i}{\beta_1} \qquad flag_i := \text{if}\left(\frac{C_i}{d} < 0.375, \text{“OK”}, \text{“Revise”}\right)$$

$$\frac{C}{d} = \begin{bmatrix} 0.107 \\ 0.105 \\ 0.105 \\ 0.102 \\ 0.102 \end{bmatrix} \qquad flag = \begin{bmatrix} \text{“OK”} \\ \text{“OK”} \\ \text{“OK”} \\ \text{“OK”} \\ \text{“OK”} \end{bmatrix}$$

$tension\_controlled := flag$

Determine Deflection Limits

$$\Delta_{max} := \frac{ht}{150}$$ Allowable Panel Deflection

$$ht' := \text{if}(DockHigh = 0, ht, 0.8 \cdot ht) = 38.8\ ft$$ Adjust Span per Dock High Condition

$$E_c := 57000 \cdot \sqrt{\frac{f_c}{psi}} \cdot psi \qquad n := \frac{29000 \cdot ksi}{E_c} \qquad A_{se_i} := A_s + \frac{P_{u_i} \cdot t}{2 \cdot f_y \cdot d} \qquad C_i := \frac{X_i}{0.85}$$

$$f_r := 7.5 \cdot \sqrt{\frac{f_c}{psi}} \cdot psi \qquad I_g := \frac{(1\ ft) \cdot t^3}{12} \qquad I_{cr_i} := \frac{12 \cdot in \cdot (C_i)^3}{3} + n \cdot A_{se_i} \cdot (d - C_i)^2$$

$$M_{cr} := \frac{2 \cdot f_r \cdot I_g}{t} \qquad \Delta_{cr} := \frac{5 \cdot M_{cr} \cdot ht'^2}{48 \cdot E_c \cdot I_g} \qquad \Delta_{n_i} := \frac{5 \cdot M_{n_i} \cdot ht'^2}{48 \cdot E_c \cdot I_{cr_i}}$$

Check Moment Capacity (ACI 318-19 11.8.3.1)

$$M_{ua_i} := \left(x_{i,3} + \left(x_{i,0} + x_{i,2}\right) \cdot \frac{e}{2}\right) \cdot ft \qquad M_{u_i} := \frac{M_{ua_i}}{1 - \frac{5 \cdot P_{u_i} \cdot ht'^2}{0.75 \cdot 48 \cdot E_c \cdot I_{cr_i}}}$$

$$flag_i := \text{if}\left(\frac{M_{u_i}}{\phi M_{n_i}} \le 1, \text{"OK"}, \text{"Revise"}\right)$$

$$\frac{M_u}{\phi M_n} = \begin{bmatrix} 0.2 \\ 0.3 \\ 0.7 \\ 0.3 \\ 0.7 \end{bmatrix} \qquad flag = \begin{bmatrix} \text{"OK"} \\ \text{"OK"} \\ \text{"OK"} \\ \text{"OK"} \\ \text{"OK"} \end{bmatrix}$$

$$capacity\_check := flag$$

Check Deflection (ACI 318-19 11.8.4)

$$M_{a_i} := \left(y_{i,3} + \left(y_{i,0} + y_{i,2}\right) \cdot \frac{e}{2}\right)\ ft \qquad P_{s_i} := \left(y_{i,0} + y_{i,1} + y_{i,2}\right) \cdot ft$$

Iterate Service Deflection

$$\Delta_{s_i} := 0 \cdot in$$ Initial Guess

$$\Delta_s := \Delta f_s\left(M_a, P_s, \Delta_s, M_{cr}, \Delta_{cr}, M_n, \Delta_n\right)$$ Function iterates service deflections per ACI 318-19 *Sec 11.8.4*

$$flag_i := \text{if}\left(\frac{\Delta_{s_i}}{\Delta_{max}} \le 1, \text{"OK"}, \text{"Revise"}\right)$$

$$\frac{\Delta_s}{\Delta_{max}} = \begin{bmatrix} 0 \\ 0.1 \\ 0.4 \\ 0.1 \\ 0.4 \end{bmatrix} \qquad flag = \begin{bmatrix} \text{"OK"} \\ \text{"OK"} \\ \text{"OK"} \\ \text{"OK"} \\ \text{"OK"} \end{bmatrix}$$

$$deflection\_check := flag$$

Results Summary

Concrete Panel Out-of-Plane Analysis

Panel Thickness $t=8.8\ in$ **Bar Size** $rebar_{bar\_size}=\text{“\#5”}$ **Bar Spacing** $SP=10\ in$

Bar Location $layer=\text{“EF”}$

Concrete Panel Out-of-Plane Analysis

Solid Panel (P58)- 8.75" THK, #5 @ 10" OC VERT EF

Panel Geometry

$t := 8.75\ in$ — Panel Thickness

$ht := 38 \cdot ft$ — Roof Height (Panel Span)

$h_p := 5.5 \cdot ft$ — Parapet Height

$DockHigh := \text{No}$ — Dock High Span Reduction

$w := 26.5 \cdot ft$ — Panel Width

$e := \frac{t}{2} + 3 \cdot in = 7.38\ in$ — Roof Load Eccentricity

$h_T := ht + h_p = 43.5\ ft$ — Total Panel Height

Panel Load Factors

$L := 1\ ft$ — Panel Width Considered (PWC) for girder/pad ftg

$x_g := 1$ — Gravity Factor

$x_{OOP} := 1.0$ — OOP Factor

Panel Wind Loading Per ASCE 7-16 Chapter 30 - Part 1 Method

$GCpi := 0.18$ — Internal Pressure Coefficient per Table 26.13-1

$$A_{eff} := \frac{1}{3}\ ht^2 = 481.3\ ft^2$$

$Slope := \text{Y}$ — If roof slope is <10 degrees, GCp can be reduced by 10%

$$GCp_{poscalc} := \left(1.0 - \frac{\ln\left(\frac{A_{eff}}{ft^2}\right) - \ln(10)}{\ln(500) - \ln(10)} (1 - 0.7)\right)(1 - Slope)$$

Positive pressure coefficient (zones 4 and 5)

$$GCp_{negcalc} := \left(1.4 - \frac{\ln\left(\frac{A_{eff}}{ft^2}\right) - \ln(10)}{\ln(500) - \ln(10)} (1.4 - 0.8)\right)(1 - Slope)(-1)$$

Negative pressure coefficient (zone 5)

$$GCp_{pos} := \text{if } A_{eff} > 500\ ft^2 \;\Big\|\; 0.7\ (1 - Slope) \;\; \text{else if } A_{eff} < 10\ ft^2 \;\Big\|\; 1.0\ (1 - Slope) \;\; \text{else} \;\Big\|\; GCp_{poscalc}$$

Check against upper and lower bound values for CGp based on Aeff

$$GCp_{neg} := \text{if } A_{eff} > 500\ ft^2 \;\Big\|\; -0.8\ (1 - Slope) \;\; \text{else if } A_{eff} < 10\ ft^2 \;\Big\|\; -1.4\ (1 - Slope) \;\; \text{else} \;\Big\|\; GCp_{negcalc}$$

$$p_1 := q_h \cdot (GCp_{pos} - GCpi)$$

$$p_2 := q_h \cdot (GCp_{neg} - GCpi)$$

$$p_3 := q_h \cdot (GCp_{pos} + GCpi)$$

$$p_4 := q_h \cdot (GCp_{neg} + GCpi)$$

$$p_{net} := \max\left(|p_1|, |p_2|, |p_3|, |p_4|\right) = 13.7\ psf$$

Panel Loading

$w_{rf} := 10 \cdot ft$ — Joist Span/Spacing Adjacent to Panel

$load_{drift} := 84\ psf$ — Drift Load at Panel

$w_{drift} := 20.25\ ft$ — Drift Length

Snow Drift Reaction at Panel

$$P_{drift} := \text{if } w_{rf} \ge w_{drift} \;\Big\|\; P \leftarrow load_{drift} \cdot w_{drift} \cdot \left(\frac{1}{2} - \frac{w_{drift}}{6\ w_{rf}}\right) \;\; \text{else if } w_{rf} < w_{drift} \;\Big\|\; P \leftarrow load_{drift} \cdot w_{rf} \cdot \left(\frac{1}{2} - \frac{w_{rf}}{6 \cdot w_{drift}}\right)$$

$$P_{drift} = 0.4\ \frac{k}{ft}$$

Concrete Panel Out-of-Plane Analysis

$P_{dlr} := x_g \cdot DL \cdot \frac{w_{rf}}{2}$ **Roof Dead Load** $P_{dlr} = 0.1\ \frac{k}{ft}$

$P_{dlp} := x_g \cdot t \cdot 150\ pcf \cdot \left(\frac{ht}{2} + h_p\right)$ **Panel Dead Load** $P_{dlp} = 2.7\ \frac{k}{ft}$

$P_{sl} := x_g \cdot \left(SL \cdot \frac{w_{rf}}{2} + P_{drift}\right)$ **Total Snow Load** $P_{sl} = 0.4\ \frac{k}{ft}$

$M_w := x_{OOP} \cdot \frac{p_{net} \cdot ht^2}{8}$ **Moment Due To Wind** $M_w = 2.5\ \frac{k \cdot ft}{ft}$

$M_s := x_{OOP} \cdot \frac{F_p \cdot t \cdot 150\ pcf \cdot ht^2}{8}$ **Moment Due to Seismic** $M_s = 6.7\ \frac{k \cdot ft}{ft}$

Strength Load Combinations (LRFD)

(ASCE 7-16 2.3)

$$x := \begin{bmatrix} 1.2 \cdot P_{dlr} & 1.2 \cdot P_{dlp} & 1.6 \cdot P_{sl} & 0.5 \cdot M_w \\ 1.2 \cdot P_{dlr} & 1.2 \cdot P_{dlp} & 0.5 \cdot P_{sl} & 1.0 \cdot M_w \\ 1.2 \cdot P_{dlr} & 1.2 \cdot P_{dlp} & 0.2 \cdot P_{sl} & 1.0 \cdot M_s \\ 0.9 \cdot P_{dlr} & 0.9 \cdot P_{dlp} & 0 \cdot \frac{k}{ft} & 1.0 \cdot M_w \\ 0.9 \cdot P_{dlr} & 0.9 \cdot P_{dlp} & 0 \cdot \frac{k}{ft} & 1.0 \cdot M_s \end{bmatrix}$$

$i := 0 .. \text{rows}(x) - 1$

Service/Deflection Load Combinations (ASD)

(2021 IBC 1605.2)

$\omega := 1.0$

$$y := \begin{bmatrix} 1.0 \cdot P_{dlr} & 1.0 \cdot P_{dlp} & 1.0 \cdot P_{sl} & 0.5 \cdot (\omega \cdot 0.6 \cdot M_w) \\ 1.0 \cdot P_{dlr} & 1.0 \cdot P_{dlp} & 0.5 \cdot P_{sl} & 1.0 \cdot (\omega \cdot 0.6 \cdot M_w) \\ 1.0 \cdot P_{dlr} & 1.0 \cdot P_{dlp} & 1.0 \cdot P_{sl} & \left(\frac{1}{1.4}\right) \cdot M_s \\ 0.67 \cdot P_{dlr} & 0.67 \cdot P_{dlp} & 0 \cdot \frac{k}{ft} & 1.0 \cdot (\omega \cdot 0.6 \cdot M_w) \\ 0.9 \cdot P_{dlr} & 0.9 \cdot P_{dlp} & 0 \cdot \frac{k}{ft} & \left(\frac{1}{1.4}\right) \cdot M_s \end{bmatrix}$$

Define Panel Reinforcing

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

$SP := 10 \cdot in$ Vertical Bar Spacing

$rebar\_location := \text{EF}$ Steel Layer: Each Face (EF) or Centerline (CL)

$stirrup\_size := \text{NA}$ Stirrup Bar Size

$A_b := A_{bar\_size}$ Vertical Bar Area

$diam_b := D_{bar\_size}$ Vertical Bar Diameter

$diam_{str} := D_{stirrup\_size}$ Stirrup Bar Diameter

Concrete Panel Out-of-Plane Analysis

$d_{ef} := t - \frac{diam_b}{2} - diam_{str} - clr - r_v$ Steel Depth If Each Face

$d_{cl} := \frac{t}{2} - r_v$ Steel Depth If Centerline

$d := \text{if}\left(rebar\_location = 0, d_{ef}, d_{cl}\right)$ Steel Depth $d = 6.94\ in$

$A_s := \frac{A_b}{SP}$ Vertical Area of Steel

Check Axial Capacity

(ACI 318-19 11.8.1.1(d))

$$P_{u_i} := \left\| \begin{array}{l} P_u \leftarrow 0 \\ \left(x_{i,0} + x_{i,1} + x_{i,2}\right) \end{array} \right.$$

$P_a := 0.06 \cdot f_c \cdot t$

$flag_i := \text{if}\left(\frac{P_{u_i}}{P_a} \le 1, \text{"OK"}, \text{"Revise"}\right)$

$\frac{P_u}{P_a} = \begin{bmatrix} 0.2 \\ 0.1 \\ 0.1 \\ 0.1 \\ 0.1 \end{bmatrix}$ $flag = \begin{bmatrix} \text{"OK"} \\ \text{"OK"} \\ \text{"OK"} \\ \text{"OK"} \\ \text{"OK"} \end{bmatrix}$

$compression\_check := flag$

Determine Moment Capacity

$\beta_1 := 0.85 - 0.05 \cdot \left(\frac{f_c - 4000 \cdot psi}{1000 \cdot psi}\right)$ $P_u := P_u \cdot ft$ $A_s := A_s \cdot ft$

$X_i := \frac{P_{u_i} + A_s \cdot f_y}{0.85 \cdot f_c \cdot ft}$ $M_{n_i} := \left(A_s \cdot f_y + P_{u_i}\right) \cdot \left(d - \frac{X_i}{2}\right) - P_{u_i} \cdot \left(d - \frac{t}{2}\right)$

$\phi M_{n_i} := 0.9 \cdot M_{n_i}$ Factored Moment Capacity $\phi M_n = \begin{bmatrix} 12.3 \\ 12.2 \\ 12.1 \\ 11.9 \\ 11.9 \end{bmatrix} k \cdot ft$

Check for Tension-Controlled Section

(ACI 318-19 11.8.1.1(b))

$C_i := \frac{X_i}{\beta_1}$ $flag_i := \text{if}\left(\frac{C_i}{d} < 0.375, \text{"OK"}, \text{"Revise"}\right)$ $\frac{C}{d} = \begin{bmatrix} 0.109 \\ 0.107 \\ 0.107 \\ 0.103 \\ 0.103 \end{bmatrix}$ $flag = \begin{bmatrix} \text{"OK"} \\ \text{"OK"} \\ \text{"OK"} \\ \text{"OK"} \\ \text{"OK"} \end{bmatrix}$

$tension\_controlled := flag$

Determine Deflection Limits

$\Delta_{max} := \frac{ht}{150}$ Allowable Panel Deflection

Concrete Panel Out-of-Plane Analysis

$ht' := \text{if}(DockHigh = 0, ht, 0.8 \cdot ht) = 38\ ft$ Adjust Span per Dock High Condition

$E_c := 57000 \cdot \sqrt{\frac{f_c}{psi}} \cdot psi$ $n := \frac{29000 \cdot ksi}{E_c}$ $A_{se_i} := A_s + \frac{P_{u_i} \cdot t}{2 \cdot f_y \cdot d}$ $C_i := \frac{X_i}{0.85}$

$f_r := 7.5 \cdot \sqrt{\frac{f_c}{psi}} \cdot psi$ $I_g := \frac{(1\ ft) \cdot t^3}{12}$ $I_{cr_i} := \frac{12 \cdot in \cdot (C_i)^3}{3} + n \cdot A_{se_i} \cdot (d - C_i)^2$

$M_{cr} := \frac{2 \cdot f_r \cdot I_g}{t}$ $\Delta_{cr} := \frac{5 \cdot M_{cr} \cdot ht'^2}{48 \cdot E_c \cdot I_g}$ $\Delta_{n_i} := \frac{5 \cdot M_{n_i} \cdot ht'^2}{48 \cdot E_c \cdot I_{cr_i}}$

Check Moment Capacity (ACI 318-19 11.8.3.1)

$M_{ua_i} := \left(x_{i,3} + \left(x_{i,0} + x_{i,2}\right) \cdot \frac{e}{2}\right) \cdot ft$ $M_{u_i} := \frac{M_{ua_i}}{1 - \frac{5 \cdot P_{u_i} \cdot ht'^2}{0.75 \cdot 48 \cdot E_c \cdot I_{cr_i}}}$

$flag_i := \text{if}\left(\frac{M_{u_i}}{\phi M_{n_i}} \le 1, \text{“OK”}, \text{“Revise”}\right)$

$\frac{M_u}{\phi M_n} = \begin{bmatrix} 0.2 \\ 0.3 \\ 0.7 \\ 0.3 \\ 0.7 \end{bmatrix}$ $flag = \begin{bmatrix} \text{“OK”} \\ \text{“OK”} \\ \text{“OK”} \\ \text{“OK”} \\ \text{“OK”} \end{bmatrix}$

$capacity\_check := flag$

Check Deflection (ACI 318-19 11.8.4)

$M_{a_i} := \left(y_{i,3} + \left(y_{i,0} + y_{i,2}\right) \cdot \frac{e}{2}\right) ft$ $P_{s_i} := \left(y_{i,0} + y_{i,1} + y_{i,2}\right) \cdot ft$

Iterate Service Deflection

$\Delta_{s_i} := 0 \cdot in$ Initial Guess

$\Delta_s := \Delta f_s(M_a, P_s, \Delta_s, M_{cr}, \Delta_{cr}, M_n, \Delta_n)$ Function iterates service deflections per ACI 318-19 *Sec 11.8.4*

$flag_i := \text{if}\left(\frac{\Delta_{s_i}}{\Delta_{max}} \le 1, \text{“OK”}, \text{“Revise”}\right)$

$\frac{\Delta_s}{\Delta_{max}} = \begin{bmatrix} 0 \\ 0.1 \\ 0.4 \\ 0.1 \\ 0.4 \end{bmatrix}$ $flag = \begin{bmatrix} \text{“OK”} \\ \text{“OK”} \\ \text{“OK”} \\ \text{“OK”} \\ \text{“OK”} \end{bmatrix}$

$deflection\_check := flag$

Results Summary

Panel Thickness $t = 8.8\ in$ **Bar Size** $rebar_{bar\_size} = \text{“\#5”}$ **Bar Spacing** $SP = 10\ in$

Bar Location $layer = \text{“EF”}$

Panel w/ Storefront Opening w/ Canopy (P29) 9.25" THK, (18) #5 VERT EF

RE

Panel Geometry

$t := 9.25\ in$ — Panel Thickness

$ht := 37\ ft$ — Roof Height (Panel Span)

$h_p := 6.5\ ft$ — Parapet Height

$DockHigh := \text{No}$ — Dock High Span Reduction

$w := 26.5\ ft$ — Panel Width

$L := 4.75\ ft$ — Panel Leg Width

$ht_o := 28\ ft + 4\ in$ — Panel Opening Height

$w_o := 17\ ft$ — Panel Opening Width

$e := \frac{t}{2} + 3 \cdot in = 7.63\ in$ — Roof Load Eccentricity

$h_T := ht + h_p = 43.5\ ft$ — Total Panel Height

Panel Load Factors

$x_g := 1 + \frac{w_o}{2 \cdot L} = 2.8$ — Gravity Factor

$x_{OOP} := 1 + \frac{w_o}{2 \cdot L} = 2.8$ — OOP Factor

Panel Wind Loading Per ASCE 7-16 Chapter 30 - Part 1 Method

$GCpi := 0.18$ — Coefficient per Table 26.13-1

$$A_{eff} := \frac{1}{3}\ ht^2 = 456.3\ ft^2$$

$Slope := \text{Y}$ — If roof slope is <10 degrees, GCp can be reduced by 10%

$$\ln\left(\frac{A_{eff}}{ft^2}\right) - \ln(10)$$

Positive pressure coefficient

$$GCp_{poscalc} := \left(1.0 - \frac{\ldots}{\ln(500) - \ln(10)} (1 - 0.7)\right) (1 - Slope)$$

(zones 4 and 5)

$$GCp_{negcalc} := \left(1.4 - \frac{\ln\left(\frac{A_{eff}}{ft^2}\right) - \ln(10)}{\ln(500) - \ln(10)} (1.4 - 0.8)\right) (1 - Slope)\,(-1)$$

Negative pressure coefficient (zone 5)

$$GCp_{pos} := \begin{array}{l} \text{if } A_{eff} > 500\ ft^2 \\ \quad \| 0.7\ (1 - Slope) \\ \text{else if } A_{eff} < 10\ ft^2 \\ \quad \| 1.0\ (1 - Slope) \\ \text{else} \\ \quad \| GCp_{poscalc} \end{array}$$

Check against upper and lower bound values for CGp based on Aeff

$$GCp_{neg} := \begin{array}{l} \text{if } A_{eff} > 500\ ft^2 \\ \quad \| -0.8\ (1 - Slope) \\ \text{else if } A_{eff} < 10\ ft^2 \\ \quad \| -1.4\ (1 - Slope) \\ \text{else} \\ \quad \| GCp_{negcalc} \end{array}$$

$$p_1 := q_h \cdot (GCp_{pos} - GCpi)$$

$$p_2 := q_h \cdot (GCp_{neg} - GCpi)$$

$$p_3 := q_h \cdot (GCp_{pos} + GCpi)$$

$$p_4 := q_h \cdot (GCp_{neg} + GCpi)$$

$$p := \max(|p_1|, |p_2|, |p_3|, |p_4|) = 13.8\ psf$$

Panel Loading

$w_{rf} := 10\ ft$ Joist Span/Spacing Adjacent to Panel

$load_{drift} := 84\ psf$ Drift Load at Panel

$w_{drift} := 20.25\ ft$ Drift Length

$$P_{drift} := \text{if } w_{rf} \geq w_{drift}$$

Snow Drift Reaction at Panel

$$P_{drift} = 0.4\ \frac{k}{ft}$$

$$\| P \ldots \left(1 \quad w_{drift}\right)$$

$$P \leftarrow load_{drift} \cdot w_{drift} \cdot \left(\frac{}{2} - \frac{}{6\ w_{rf}}\right)$$

else if $w_{rf} < w_{drift}$

$$P \leftarrow load_{drift} \cdot w_{rf} \cdot \left(\frac{1}{2} - \frac{w_{rf}}{6 \cdot w_{drift}}\right)$$

$$P_{dlr} := x_g \cdot DL \cdot \frac{w_{rf}}{2}$$ **Roof Dead Load** $$P_{dlr} = 0.2\ \frac{k}{ft}$$

$$P_{dlp} := x_g \cdot 150\ pcf \cdot \left(\frac{ht}{2} + h_p\right) \cdot t$$ **Panel Dead Load** $$P_{dlp} = 8.1\ \frac{k}{ft}$$

$$P_{sl} := x_g \cdot \left(SL \cdot \frac{w_{rf}}{2} + P_{drift}\right)$$ **Total Snow Load** $$P_{sl} = 1.2\ \frac{k}{ft}$$

$$P_{dl\_mezz} := x_g \cdot \left(25\ psf \cdot \frac{20\ ft}{2}\right)$$ $$P_{dl\_mezz} = 0.7\ \frac{k}{ft}$$

$$P_{ll\_mezz} := x_g \cdot \left(65\ psf \cdot \frac{20\ ft}{2}\right)$$ $$P_{ll\_mezz} = 1.8\ \frac{k}{ft}$$

$$M_w := x_{OOP} \cdot \frac{p \cdot ht^2}{8}$$ **Moment Due To Wind** $$M_w = 6.6\ \frac{k \cdot ft}{ft}$$

$$w_1 := F_p \cdot 150\ pcf \cdot t \cdot x_{OOP}$$

$$w_2 := \left(F_p \cdot 10 \cdot psf \cdot w_o \cdot 0.5 + F_p \cdot 150\ pcf \cdot t \cdot L + F_p \cdot 25\ psf \cdot w_o\right) \cdot \frac{1}{L}$$

$$R_{bot} := \frac{w_2 \cdot ht}{2} + \frac{\left(w_1 - w_2\right) \cdot \left(ht - ht_o\right)^2}{2 \cdot ht}$$

$$R_{top} := \left(w_2 \cdot ht_o + w_1 \cdot \left(ht - ht_o\right)\right) - R_{bot}$$

$$x_m := \frac{R_{top}}{w_1}$$

$$M_s := R_{top} \cdot x_m - \frac{w_1 \cdot {x_m}^2}{2}$$ **Moment Due to Seismic w/ Opening** $$M_s = 12.5\ \frac{k \cdot ft}{ft}$$

Canopy Loading

$$P_{dl\_c} := \frac{1540\ lbf}{L}$$ **Canopy Dead Load** $$P_{dl\_c} = 0.3\ \frac{k}{ft}$$

$$M_{dl\_c} := \frac{8.12\ k \cdot ft}{L}$$ **Canopy Dead Load (Moment)** $$M_{dl\_c} = 1.7\ \frac{k \cdot ft}{ft}$$

$P_{sl\_c} := \frac{520\ lbf}{L}$ Canopy Snow Load $P_{sl\_c} = 0.1\ \frac{k}{ft}$

$M_{sl\_c} := \frac{2.06 \cdot k \cdot ft}{L}$ Canopy Snow Load (Moment) $M_{sl\_c} = 0.4\ \frac{k \cdot ft}{ft}$

$P_{w\_c} := \frac{270\ lbf}{L}$ Canopy Wind Load $P_{w\_c} = 0.1\ \frac{k}{ft}$

$M_{w\_c} := \frac{1.28\ k \cdot ft}{L}$ Canopy Wind (Moment) $M_{w\_c} = 0.3\ \frac{k \cdot ft}{ft}$

Strength Load Combinations (LRFD)

(ASCE 7-16 2.3)

$$x := \begin{bmatrix} 1.2 \cdot (P_{dlr} + P_{dl\_mezz}) & 1.2 \cdot P_{dlp} & 1.6 \cdot P_{sl} + 0.5\ P_{ll\_mezz} & 0\ \frac{k \cdot ft}{ft} & 1.2 \cdot P_{dl\_c} & 1.2 \cdot M_{dl\_c} & 1.6 \cdot P_{sl\_c} & 1.6 \cdot M_{sl\_c} & 0\ \frac{k}{ft} & 0.5 \cdot M_{w\_c} \\ 1.2 \cdot (P_{dlr} + P_{dl\_mezz}) & 1.2 \cdot P_{dlp} & 0.5 \cdot (P_{sl} + P_{ll\_mezz}) & 1.0 \cdot M_w & 1.2 \cdot P_{dl\_c} & 1.2 \cdot M_{dl\_c} & 0.5 \cdot P_{sl\_c} & 0.5 \cdot M_{sl\_c} & 1.0 \cdot P_{w\_c} & 1.0 \cdot M_{w\_c} \\ 1.2 \cdot (P_{dlr} + P_{dl\_mezz}) & 1.2 \cdot P_{dlp} & 0.5\ P_{ll\_mezz} + 0.2 \cdot P_{sl} & 1.0 \cdot M_s & 1.2 \cdot P_{dl\_c} & 1.2 \cdot M_{dl\_c} & 0.2 \cdot P_{sl\_c} & 0.2 \cdot M_{sl\_c} & 0\ \frac{k}{ft} & 0\ \frac{k \cdot ft}{ft} \\ 0.9 \cdot (P_{dlr} + P_{dl\_mezz}) & 0.9 \cdot P_{dlp} & 0 \cdot \frac{k}{ft} & 1.0 \cdot M_w & 0.9 \cdot P_{dl\_c} & 0.9 \cdot M_{dl\_c} & 0 \cdot \frac{k}{ft} & 0 \cdot \frac{k \cdot ft}{ft} & 1.0 \cdot P_{w\_c} & 1.0 \cdot M_{w\_c} \\ 0.9 \cdot (P_{dlr} + P_{dl\_mezz}) & 0.9 \cdot P_{dlp} & 0 \cdot \frac{k}{ft} & 1.0 \cdot M_s & 0.9 \cdot P_{dl\_c} & 0.9 \cdot M_{dl\_c} & 0 \cdot \frac{k}{ft} & 0 \cdot \frac{k \cdot ft}{ft} & 0\ \frac{k}{ft} & 0 \cdot \frac{k \cdot ft}{ft} \end{bmatrix}$$

Service/Deflection Load Combinations (ASD)

(2021 IBC 1605.2)

$\omega := 1.0$ $i := 0 .. \text{rows}(x) - 1$

$$y := \begin{bmatrix} 1.0 \cdot (P_{dlr} + P_{dl\_mezz}) & 1.0 \cdot P_{dlp} & P_{sl} + P_{ll\_mezz} & \frac{\omega \cdot 0.6 \cdot M_w}{2} & 1.0 \cdot P_{dl\_c} & 1.0 \cdot M_{dl\_c} & 1.0 \cdot P_{sl\_c} & 1.0 \cdot M_{sl\_c} & \frac{\omega \cdot 0.6 \cdot P_{w\_c}}{2} & \frac{\omega \cdot 0.6 \cdot M_{w\_c}}{2} \\ 1.0 \cdot (P_{dlr} + P_{dl\_mezz}) & 1.0 \cdot P_{dlp} & P_{ll\_mezz} + 0.5 \cdot P_{sl} & \omega \cdot 0.6 \cdot M_w & 1.0 \cdot P_{dl\_c} & 1.0 \cdot M_{dl\_c} & 0.5 \cdot P_{sl\_c} & 0.5 \cdot M_{sl\_c} & \omega \cdot 0.6 \cdot P_{w\_c} & \omega \cdot 0.6 \cdot M_{w\_c} \\ 1.0 \cdot (P_{dlr} + P_{dl\_mezz}) & 1.0 \cdot P_{dlp} & P_{sl} + P_{ll\_mezz} & \left(\frac{1}{1.4}\right) \cdot M_s & 1.0 \cdot P_{dl\_c} & 1.0 \cdot M_{dl\_c} & 1.0 \cdot P_{sl\_c} & 1.0 \cdot M_{sl\_c} & 0\ \frac{k}{ft} & 0 \cdot \frac{k \cdot ft}{ft} \\ 0.67 \cdot (P_{dlr} + P_{dl\_mezz}) & 0.67 \cdot P_{dlp} & 0 \cdot \frac{k}{ft} & \omega \cdot 0.6 \cdot M_w & 0.67 \cdot P_{dl\_c} & 0.67 \cdot M_{dl\_c} & 0 \cdot \frac{k}{ft} & 0 \cdot \frac{k \cdot ft}{ft} & \omega \cdot 0.6 \cdot P_{w\_c} & \omega \cdot 0.6 \cdot M_{w\_c} \\ 0.9 \cdot (P_{dlr} + P_{dl\_mezz}) & 0.9 \cdot P_{dlp} & 0 \cdot \frac{k}{ft} & \left(\frac{1}{1.4}\right) \cdot M_s & 0.9 \cdot P_{dl\_c} & 0.9 \cdot M_{dl\_c} & 0 \cdot \frac{k}{ft} & 0 \cdot \frac{k \cdot ft}{ft} & 0\ \frac{k}{ft} & 0\ \frac{k \cdot ft}{ft} \end{bmatrix}$$

Define Panel Reinforcing

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

Concrete Panel Out-of-Plane Analysis

$NB := 18$ — Vertical Bar Spacing

$rebar\_location := \text{EF}$ — Steel Layer: Each Face (EF) or Centerline (CL)

$stirrup\_size := \#3$ — Stirrup Bar Size

$A_b := A_{bar\_size}$ — Vertical Bar Area

$diam_b := D_{bar\_size}$ — Vertical Bar Diameter

$diam_{str} := D_{stirrup\_size}$ — Stirrup Bar Diameter

$d_{ef} := t - \dfrac{diam_b}{2} - diam_{str} - clr - r_v$ — Steel Depth If Each Face

$d_{cl} := \dfrac{t}{2} - r_v$ — Steel Depth If Centerline

$d := \text{if}\left(rebar\_location = 0, d_{ef}, d_{cl}\right)$ — Steel Depth — $d = 7.06\ in$

$S := \dfrac{L - 1.5\ in}{NB - 1} = 3.3\ in$ — Vertical Bar Spacing OC

$flag := \text{if}\left(S \geq 3\ in, \text{“OK”}, \text{“Revise”}\right)$ — $S = 3.3\ in$ — $flag = \text{“OK”}$

$A_s := \dfrac{NB \cdot A_b}{L} = 1.17\ \dfrac{in^2}{ft}$ — Vertical Area of Steel

Check Axial Capacity

(ACI 318-19 11.8.1.1(d))

$$P_{u_i} := \left\| \begin{array}{l} P_u \leftarrow 0 \\ \left(x_{i,0} + x_{i,1} + x_{i,2} + x_{i,4} + x_{i,6} + x_{i,8}\right) \end{array} \right.$$

$P_a := 0.06 \cdot f_c \cdot t$

$$flag_i := \text{if}\left(\frac{P_{u_i}}{P_a} \leq 1, \text{“OK”}, \text{“Revise”}\right)$$

$$\frac{P_u}{P_a} = \begin{bmatrix} 0.5 \\ 0.5 \\ 0.5 \\ 0.3 \\ 0.3 \end{bmatrix} \qquad flag = \begin{bmatrix} \text{“OK”} \\ \text{“OK”} \\ \text{“OK”} \\ \text{“OK”} \\ \text{“OK”} \end{bmatrix}$$

$compression\_check := flag$

Determine Moment Capacity

$$\beta_1 := 0.85 - 0.05 \cdot \left(\frac{f_c - 4000 \cdot psi}{1000 \cdot psi}\right)$$

$P_u := P_u \cdot ft \qquad A_s := A_s \cdot ft$

$$X_i := \frac{P_{u_i} + A_s \cdot f_y}{0.85 \cdot f_c \cdot ft}$$

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

$\phi M_{n_i} := 0.9 \cdot M_{n_i}$ Factored Moment Capacity $\phi M_n = \begin{bmatrix} 35.7 \\ 35.4 \\ 35.3 \\ 34.5 \\ 34.5 \end{bmatrix} k \cdot ft$

Check for Tension-Controlled Section

(ACI 318-19 11.8.1.1(b))

$C_i := \dfrac{X_i}{\beta_1}$

$flag_i := \text{if}\left(\dfrac{C_i}{d} < 0.375, \text{“OK”}, \text{“Revise”}\right)$

$tension\_controlled := flag$

$\dfrac{C}{d} = \begin{bmatrix} 0.345 \\ 0.34 \\ 0.338 \\ 0.322 \\ 0.322 \end{bmatrix}$ $flag = \begin{bmatrix} \text{“OK”} \\ \text{“OK”} \\ \text{“OK”} \\ \text{“OK”} \\ \text{“OK”} \end{bmatrix}$

Determine Deflection Limits

$\Delta_{max} := \dfrac{ht}{150}$ Allowable Panel Deflection

$ht' := \text{if}(DockHigh = 0, ht, 0.8 \cdot ht) = 37\ ft$ Adjust Span per Dock High Condition

$E_c := 57000 \cdot \sqrt{\dfrac{f_c}{psi}} \cdot psi$ $n := \dfrac{29000 \cdot ksi}{E_c}$ $A_{se_i} := A_s + \dfrac{P_{u_i} \cdot t}{2 \cdot f_y \cdot d}$ $C_i := \dfrac{X_i}{0.85}$

$f_r := 7.5 \cdot \sqrt{\dfrac{f_c}{psi}} \cdot psi$ $I_g := \dfrac{(1\ ft) \cdot t^3}{12}$ $I_{cr_i} := \dfrac{12 \cdot in \cdot \left(C_i\right)^3}{3} + n \cdot A_{se_i} \cdot \left(d - C_i\right)^2$

$M_{cr} := \dfrac{2 \cdot f_r \cdot I_g}{t}$ $\Delta_{cr} := \dfrac{5 \cdot M_{cr} \cdot ht'^2}{48 \cdot E_c \cdot I_g}$ $\Delta_{n_i} := \dfrac{5 \cdot M_{n_i} \cdot ht'^2}{48 \cdot E_c \cdot I_{cr_i}}$

Check Moment Capacity

(ACI 318-19 11.8.3.1)

$M_{ua_i} := \left(x_{i,3} + \left(x_{i,0} + x_{i,2}\right) \cdot \dfrac{e}{2} + x_{i,5} + x_{i,7} + x_{i,9}\right) \cdot ft$ $M_{u_i} := \dfrac{M_{ua_i}}{1 - \dfrac{5 \cdot P_{u_i} \cdot ht'^2}{0.75 \cdot 48 \cdot E_c \cdot I_{cr_i}}}$

$flag_i := \text{if}\left(\dfrac{M_{u_i}}{\phi M_{n_i}} \le 1, \text{“OK”}, \text{“Revise”}\right)$

$capacity\_check := flag$

$\dfrac{M_u}{\phi M_n} = \begin{bmatrix} 0.2 \\ 0.4 \\ 0.6 \\ 0.3 \\ 0.5 \end{bmatrix}$ $flag = \begin{bmatrix} \text{“OK”} \\ \text{“OK”} \\ \text{“OK”} \\ \text{“OK”} \\ \text{“OK”} \end{bmatrix}$

Check Deflection

(ACI 318-19 11.8.4)

$M_{a_i} := \left(y_{i,3} + \left(y_{i,0} + y_{i,2}\right) \cdot \dfrac{e}{2} + y_{i,5} + y_{i,7} + y_{i,9}\right) ft$ $P_{s_i} := \left(y_{i,0} + y_{i,1} + y_{i,2} + y_{i,4} + y_{i,6} + y_{i,8}\right) \cdot ft$

Concrete Panel Out-of-Plane Analysis

Iterate Service Deflection

$\Delta_{s_i} := 0 \cdot in$ Initial Guess

$\Delta_s := \Delta f_s(M_a, P_s, \Delta_s, M_{cr}, \Delta_{cr}, M_n, \Delta_n)$ Function iterates service deflections per ACI 318-19 *Sec 11.8.4*

$flag_i := \text{if}\left(\frac{\Delta_{s_i}}{\Delta_{max}} \le 1, \text{"OK"}, \text{"Revise"}\right)$

$\frac{\Delta_s}{\Delta_{max}} = \begin{bmatrix} 0.2 \\ 0.4 \\ 0.9 \\ 0.2 \\ 0.8 \end{bmatrix}$ $flag = \begin{bmatrix} \text{"OK"} \\ \text{"OK"} \\ \text{"OK"} \\ \text{"OK"} \\ \text{"OK"} \end{bmatrix}$

$deflection\_check := flag$

Results Summary

Panel Thickness	$t = 9.3\ in$	**Bar Size**	$rebar_{bar\_size} = \text{"\#5"}$
Panel Leg Width	$L = 4.8\ ft$	**Bar Quantity**	$NB = 18$
		Bar Location	$layer = \text{"EF"}$

East Elevation

Solid Panel - Girder Loading (P25 @ GL C) - 8.75" THK 10' Eff Width, (20) #5 VERT EF

Panel Geometry

$t := 8.75\ in$ Panel Thickness

$ht := 39.25 \cdot ft$ Roof Height (Panel Span)

$h_p := 4.25 \cdot ft$ Parapet Height

$DockHigh := \text{No}$ Dock High Span Reduction

$w := 26.5 \cdot ft$ Panel Width

$e := \frac{t}{2} + 3 \cdot in = 7.38\ in$ Roof Load Eccentricity

$h_T := ht + h_p = 43.5\ ft$ Total Panel Height

Panel Load Factors

$L := 10\ ft$ Panel Width Considered (PWC) for girder/pad ftg

$x_g := 1.0$ Gravity Factor

$x_{OOP} := 1.0$ OOP Factor

Panel Wind Loading Per ASCE 7-16 Chapter 30 - Part 1 Method

$GCpi := 0.18$ Internal Pressure Coefficient per Table 26.13-1

$A_{eff} := \frac{1}{3}\ ht^2 = 513.5\ ft^2$

$Slope := \text{Y}$ If roof slope is <10 degrees, GCp can be reduced by 10%

$$GCp_{poscalc} := \left(1.0 - \frac{\ln\left(\frac{A_{eff}}{ft^2}\right) - \ln(10)}{\ln(500) - \ln(10)}(1 - 0.7)\right)(1 - Slope)$$

Positive pressure coefficient (zones 4 and 5)

$$\left(\quad \frac{\ln\left(\frac{A_{eff}}{ft^2}\right) - \ln(10)}{\ldots} \right)$$

Negative pressure coefficient

$$GCp_{negcalc} := \left(1.4 - \frac{\ldots}{\ln(500) - \ln(10)} (1.4 - 0.8)\right) (1 - Slope)(-1)$$

(zone 5)

$$GCp_{pos} := \text{if } A_{eff} > 500\ ft^2 \;\; \| \; 0.7\ (1 - Slope) \;\; \text{else if } A_{eff} < 10\ ft^2 \;\; \| \; 1.0\ (1 - Slope) \;\; \text{else} \;\; \| \; GCp_{poscalc}$$

$$GCp_{neg} := \text{if } A_{eff} > 500\ ft^2 \;\; \| \; -0.8\ (1 - Slope) \;\; \text{else if } A_{eff} < 10\ ft^2 \;\; \| \; -1.4\ (1 - Slope) \;\; \text{else} \;\; \| \; GCp_{negcalc}$$

Check against upper and lower bound values for CGp based on Aeff

$$p_1 := q_h \cdot (GCp_{pos} - GCpi)$$

$$p_2 := q_h \cdot (GCp_{neg} - GCpi)$$

$$p_3 := q_h \cdot (GCp_{pos} + GCpi)$$

$$p_4 := q_h \cdot (GCp_{neg} + GCpi)$$

$$p := \max(|p_1|, |p_2|, |p_3|, |p_4|) = 13.6\ psf$$

Panel Loading

$w_{grdr} := 29\ ft + 10\ in$ — Girder Span

$s_{grdr} := 59.33\ ft$ — Girder Tributary Width

$load_{drift} := 84\ psf$ — Drift Load at Girder Panel

$w_{drift} := 20.25\ ft$ — Drift Length at Girder Panel

$f_{grdr} := 1.0$ — Portion of Girder Load on Panel Leg

$x_{grdr} := f_{grdr} \cdot \frac{s_{grdr}}{L} = 5.9$ — Girder Factor

Concrete Panel Out-of-Plane Analysis

$s_{jst} := 10\ ft + 0\ in$ Joist Spacing/Tributary Width

$P_{drift\_1} := \frac{load_{drift} \cdot s_{jst}}{2} \cdot \left(1 - \frac{s_{jst}}{4\ w_{drift}}\right)$ Drift Load on Panel

$P_{drift\_2} := \frac{load_{drift} \cdot w_{drift}}{2} - P_{drift\_1}$ Girder Reaction from Drift

$w_{jst} := 59.33\ ft$ Joist Span

$load_{drift\_jst} := 0\ psf$ Drift Load at Joist Panel

$w_{drift\_jst} := 0\ ft$ Drift Length at Joist Panel

$$P_{drift\_3} := \frac{w_{grdr}}{2\ L} \left(\frac{load_{drift\_jst} \cdot {w_{drift\_jst}}^2}{6 \cdot w_{jst}}\right)$$

$$P_{sl\_1} := x_g \cdot \left(SL \cdot \frac{s_{jst}}{2} + P_{drift\_1}\right)$$

$$P_{sl\_2} := x_{grdr} \cdot \left(SL \cdot \left(\frac{w_{grdr} - s_{jst}}{2}\right) + P_{drift\_2}\right) + f_{grdr} \cdot P_{drift\_3}$$

$P_{sl} := P_{sl\_1} + P_{sl\_2}$ **Total Snow Load** $P_{sl} = 4.1\ \frac{k}{ft}$

$P_{dlr} := x_g \cdot DL \cdot \frac{s_{jst}}{2} + x_{grdr} \cdot DL \cdot \left(\frac{w_{grdr} - s_{jst}}{2}\right)$ **Roof Dead Load** $P_{dlr} = 1\ \frac{k}{ft}$

$P_{dlp} := x_g \cdot t \cdot 150\ pcf \cdot \left(\frac{ht}{2} + h_p\right)$ **Panel Dead Load** $P_{dlp} = 2.6\ \frac{k}{ft}$

$M_w := x_{OOP} \cdot \frac{p \cdot ht^2}{8}$ **Moment Due To Wind** $M_w = 2.6\ \frac{k \cdot ft}{ft}$

$M_s := x_{OOP} \cdot \frac{F_p \cdot t \cdot 150\ pcf \cdot ht^2}{8}$ **Moment Due to Seismic** $M_s = 7.1\ \frac{k \cdot ft}{ft}$

Strength Load Combinations (LRFD)

(ASCE 7-16 2.3)

Service/Deflection Load Combinations (ASD)

(2021 IBC 1605.2)

$\omega := 1.0$

$$x := \begin{bmatrix} 1.2 \cdot P_{dlr} & 1.2 \cdot P_{dlp} & 1.6 \cdot P_{sl} & 0.5 \cdot M_w \\ 1.2 \cdot P_{dlr} & 1.2 \cdot P_{dlp} & 0.5 \cdot P_{sl} & 1.0 \cdot M_w \\ 1.2 \cdot P_{dlr} & 1.2 \cdot P_{dlp} & 0.2 \cdot P_{sl} & 1.0 \cdot M_s \\ 0.9 \cdot P_{dlr} & 0.9 \cdot P_{dlp} & 0 \cdot \frac{k}{ft} & 1.0 \cdot M_w \\ 0.9 \cdot P_{dlr} & 0.9 \cdot P_{dlp} & 0 \cdot \frac{k}{ft} & 1.0 \cdot M_s \end{bmatrix} \qquad y := \begin{bmatrix} 1.0 \cdot P_{dlr} & 1.0 \cdot P_{dlp} & 1.0 \cdot P_{sl} & 0.5 \cdot (\omega \cdot 0.6 \cdot M_w) \\ 1.0 \cdot P_{dlr} & 1.0 \cdot P_{dlp} & 0.5 \cdot P_{sl} & 1.0 \cdot (\omega \cdot 0.6 \cdot M_w) \\ 1.0 \cdot P_{dlr} & 1.0 \cdot P_{dlp} & 1.0 \cdot P_{sl} & \left(\frac{1}{1.4}\right) \cdot M_s \\ 0.67 \cdot P_{dlr} & 0.67 \cdot P_{dlp} & 0 \cdot \frac{k}{ft} & 1.0 \cdot (\omega \cdot 0.6 \cdot M_w) \\ 0.9 \cdot P_{dlr} & 0.9 \cdot P_{dlp} & 0 \cdot \frac{k}{ft} & \left(\frac{1}{1.4}\right) \cdot M_s \end{bmatrix}$$

$i := 0 \,..\, \text{rows}(x) - 1$

Define Panel Reinforcing

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

$NB := 20$ — Vertical Bar Spacing

$$S := \frac{L - 1.5\ in}{NB - 1} = 6.2\ in$$

$flag := \text{if}(S > 3\ in, \text{“OK”}, \text{“Revise”}) = \text{“OK”}$

$rebar\_location := \text{EF}$ — Steel Layer: Each Face (EF) or Centerline (CL)

$stirrup\_size := \text{NA}$ — Stirrup Bar Size

$A_b := A_{bar\_size}$ — Vertical Bar Area

$diam_b := D_{bar\_size}$ — Vertical Bar Diameter

$diam_{str} := D_{stirrup\_size}$ — Stirrup Bar Diameter

$$d_{ef} := t - \frac{diam_b}{2} - diam_{str} - clr - r_v$$ Steel Depth If Each Face

$$d_{cl} := \frac{t}{2} - r_v$$ Steel Depth If Centerline

$d := \text{if}(rebar\_location = 0, d_{ef}, d_{cl})$ — Steel Depth — $d = 6.94\ in$

$$A_s := \frac{NB \cdot A_b}{L}$$ Vertical Area of Steel

Check Axial Capacity

(ACI 318-19 11.8.1.1(d))

$$P_{u_i} := \left\| \begin{array}{l} P_u \leftarrow 0 \\ \left(x_{i,0} + x_{i,1} + x_{i,2}\right) \end{array} \right.$$

$P_a := 0.06 \cdot f_c \cdot t$

$flag_i := \text{if}\left(\frac{P_{u_i}}{P_a} \le 1, \text{"OK"}, \text{"Revise"}\right)$

$\frac{P_u}{P_a} = \begin{bmatrix} 0.4 \\ 0.3 \\ 0.2 \\ 0.1 \\ 0.1 \end{bmatrix}$ $flag = \begin{bmatrix} \text{"OK"} \\ \text{"OK"} \\ \text{"OK"} \\ \text{"OK"} \\ \text{"OK"} \end{bmatrix}$

$compression\_check := flag$

Determine Moment Capacity

$\beta_1 := 0.85 - 0.05 \cdot \left(\frac{f_c - 4000 \cdot psi}{1000 \cdot psi}\right)$ $P_u := P_u \cdot ft$ $A_s := A_s \cdot ft$

$X_i := \frac{P_{u_i} + A_s \cdot f_y}{0.85 \cdot f_c \cdot ft}$ $M_{n_i} := \left(A_s \cdot f_y + P_{u_i}\right) \cdot \left(d - \frac{X_i}{2}\right) - P_{u_i} \cdot \left(d - \frac{t}{2}\right)$

$\phi M_{n_i} := 0.9 \cdot M_{n_i}$ Factored Moment Capacity $\phi M_n = \begin{bmatrix} 20.8 \\ 19.7 \\ 19.4 \\ 18.9 \\ 18.9 \end{bmatrix} k \cdot ft$

Check for Tension-Controlled Section

(ACI 318-19 11.8.1.1(b))

$C_i := \frac{X_i}{\beta_1}$ $flag_i := \text{if}\left(\frac{C_i}{d} < 0.375, \text{"OK"}, \text{"Revise"}\right)$

$\frac{C}{d} = \begin{bmatrix} 0.2 \\ 0.181 \\ 0.176 \\ 0.168 \\ 0.168 \end{bmatrix}$ $flag = \begin{bmatrix} \text{"OK"} \\ \text{"OK"} \\ \text{"OK"} \\ \text{"OK"} \\ \text{"OK"} \end{bmatrix}$

$tension\_controlled := flag$

Determine Deflection Limits

$\Delta_{max} := \frac{ht}{150}$ Allowable Panel Deflection

$ht' := \text{if}\left(DockHigh = 0, ht, 0.8 \cdot ht\right) = 39.3\ ft$ Adjust Span per Dock High Condition

$E_c := 57000 \cdot \sqrt{\frac{f_c}{psi}} \cdot psi$ $n := \frac{29000 \cdot ksi}{E_c}$ $A_{se_i} := A_s + \frac{P_{u_i} \cdot t}{2 \cdot f_y \cdot d}$ $C_i := \frac{X_i}{0.85}$

$f_r := 7.5 \cdot \sqrt{\frac{f_c}{psi}} \cdot psi$ $I_g := \frac{(1\ ft) \cdot t^3}{12}$ $I_{cr_i} := \frac{12 \cdot in \cdot \left(C_i\right)^3}{3} + n \cdot A_{se_i} \cdot \left(d - C_i\right)^2$

$M_{cr} := \frac{2 \cdot f_r \cdot I_g}{t}$ $\Delta_{cr} := \frac{5 \cdot M_{cr} \cdot ht'^2}{48 \cdot E_c \cdot I_g}$ $\Delta_{n_i} := \frac{5 \cdot M_{n_i} \cdot ht'^2}{48 \cdot E_c \cdot I_{cr_i}}$

Check Moment Capacity

(ACI 318-19 11.8.3.1)

$M_{ua_i} := \left(x_{i,3} + \left(x_{i,0} + x_{i,2}\right) \cdot \frac{e}{2}\right) \cdot ft$ $M_{u_i} := \frac{M_{ua_i}}{1 - \frac{5 \cdot P_{u_i} \cdot ht'^2}{}}$

$0.75 \cdot 48 \cdot E_c \cdot I_{cr_i}$

$flag_i := \text{if}\left(\dfrac{M_{u_i}}{\phi M_{n_i}} \le 1, \text{“OK”}, \text{“Revise”}\right)$

$\dfrac{M_u}{\phi M_n} = \begin{bmatrix} 0.3 \\ 0.3 \\ 0.5 \\ 0.2 \\ 0.5 \end{bmatrix}$ $flag = \begin{bmatrix} \text{“OK”} \\ \text{“OK”} \\ \text{“OK”} \\ \text{“OK”} \\ \text{“OK”} \end{bmatrix}$

$capacity\_check := flag$

Check Deflection *(ACI 318-19 11.8.4)*

$M_{a_i} := \left(y_{i,3} + \left(y_{i,0} + y_{i,2}\right) \cdot \dfrac{e}{2}\right) ft$ $P_{s_i} := \left(y_{i,0} + y_{i,1} + y_{i,2}\right) \cdot ft$

Iterate Service Deflection

$\Delta_{s_i} := 0 \cdot in$ Initial Guess

$\Delta_s := \Delta f_s\left(M_a, P_s, \Delta_s, M_{cr}, \Delta_{cr}, M_n, \Delta_n\right)$ Function iterates service deflections per ACI 318-19 *Sec 11.8.4*

$flag_i := \text{if}\left(\dfrac{\Delta_{s_i}}{\Delta_{max}} \le 1, \text{“OK”}, \text{“Revise”}\right)$

$\dfrac{\Delta_s}{\Delta_{max}} = \begin{bmatrix} 0.1 \\ 0.1 \\ 0.7 \\ 0.1 \\ 0.4 \end{bmatrix}$ $flag = \begin{bmatrix} \text{“OK”} \\ \text{“OK”} \\ \text{“OK”} \\ \text{“OK”} \\ \text{“OK”} \end{bmatrix}$

$deflection\_check := flag$

Results Summary

Panel Thickness $t = 8.8\ in$ **Bar Size** $rebar_{bar\_size} = \text{“\#5”}$ **Bar Spacing** $NB = 20$

Bar Location $layer = \text{“EF”}$

Solid Panel - Girder Loading (P23 @ GL D) - 8.75" THK 5'-0" Eff Width, (10) #5 VERT EF

Panel Geometry

$t := 8.75\ in$ Panel Thickness

$ht := 38 \cdot ft$ Roof Height (Panel Span)

$h_p := 2.5 \cdot ft$ Parapet Height

$DockHigh := \text{No}$ Dock High Span Reduction

$w := 20 \cdot ft$ Panel Width

$e := \frac{t}{2} + 3 \cdot in = 7.38\ in$ Roof Load Eccentricity

$h_T := ht + h_p = 40.5\ ft$ Total Panel Height

Panel Load Factors

$L := 5\ ft + 0\ in$ Panel Width Considered (PWC) for girder/pad ftg

$x_g := 1.0$ Gravity Factor

$x_{OOP} := 1.0$ OOP Factor

Panel Wind Loading Per ASCE 7-16 Chapter 30 - Part 1 Method

$GCpi := 0.18$ Internal Pressure Coefficient per Table 26.13-1

$A_{eff} := \frac{1}{3}\ ht^2 = 481.3\ ft^2$

$Slope := \text{Y}$ If roof slope is <10 degrees, GCp can be reduced by 10%

$$GCp_{poscalc} := \left(1.0 - \frac{\ln\left(\frac{A_{eff}}{ft^2}\right) - \ln(10)}{\ln(500) - \ln(10)}(1 - 0.7)\right)(1 - Slope)$$

Positive pressure coefficient (zones 4 and 5)

$$GCp_{negcalc} := \left(1.4 - \frac{\ln\left(\frac{A_{eff}}{ft^2}\right) - \ln(10)}{\ln(500) - \ln(10)}(1.4 - 0.8)\right)(1 - Slope)(-1)$$

Negative pressure coefficient (zone 5)

Concrete Panel Out-of-Plane Analysis

$$GCp_{pos} := \text{if } A_{eff} > 500\ ft^2 \;\; \| 0.7\ (1 - Slope) \;\; \text{else if } A_{eff} < 10\ ft^2 \;\; \| 1.0\ (1 - Slope) \;\; \text{else} \;\; \| GCp_{poscalc}$$

Check against upper and lower bound values for CGp based on Aeff

$$GCp_{neg} := \text{if } A_{eff} > 500\ ft^2 \;\; \| -0.8\ (1 - Slope) \;\; \text{else if } A_{eff} < 10\ ft^2 \;\; \| -1.4\ (1 - Slope) \;\; \text{else} \;\; \| GCp_{negcalc}$$

$$p_1 := q_h \cdot (GCp_{pos} - GCpi)$$

$$p_2 := q_h \cdot (GCp_{neg} - GCpi)$$

$$p_3 := q_h \cdot (GCp_{pos} + GCpi)$$

$$p_4 := q_h \cdot (GCp_{neg} + GCpi)$$

$$p := \max(|p_1|, |p_2|, |p_3|, |p_4|) = 13.7\ psf$$

Panel Loading

$w_{grdr} := 29\ ft + 10\ in$ Girder Span

$s_{grdr} := \dfrac{59.33\ ft}{1.25}$ Girder Tributary Width

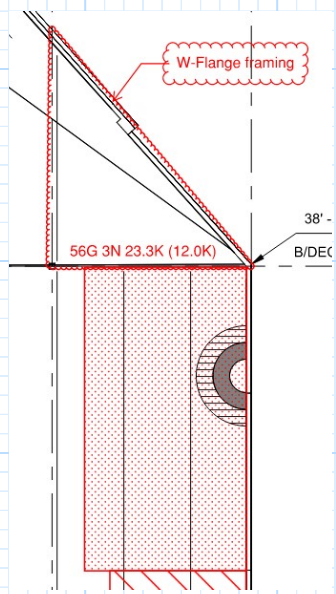
$load_{drift} := 51\ psf$ Drift Load at Girder Panel

$w_{drift} := 24.5\ ft$ Drift Length at Girder Panel

$f_{grdr} := 1$ Portion of Girder Load on Panel Leg

$x_{grdr} := f_{grdr} \cdot \dfrac{s_{grdr}}{L} = 9.5$ Girder Factor

$s_{jst} := 10\ ft + 0\ in$ Joist Spacing/Tributary Width



Concrete Panel Out-of-Plane Analysis

$P_{drift\_1} := \frac{load_{drift} \cdot s_{jst}}{2} \cdot \left(1 - \frac{s_{jst}}{4\ w_{drift}}\right)$ Drift Load on Panel

$P_{drift\_2} := \frac{load_{drift} \cdot w_{drift}}{2} - P_{drift\_1}$ Girder Reaction from Drift

$w_{jst} := 60\ ft$ Joist Span

$load_{drift\_jst} := 0\ psf$ Drift Load at Joist Panel

$w_{drift\_jst} := 0\ ft$ Drift Length at Joist Panel

$P_{drift\_3} := \frac{w_{grdr}}{2\ L} \left(\frac{load_{drift\_jst} \cdot w_{drift\_jst}^{2}}{6 \cdot w_{jst}}\right)$

$P_{sl\_1} := x_g \cdot \left(SL \cdot \frac{s_{jst}}{2} + P_{drift\_1}\right)$

$P_{sl\_2} := x_{grdr} \cdot \left(SL \cdot \left(\frac{w_{grdr} - s_{jst}}{2}\right) + P_{drift\_2}\right) + f_{grdr} \cdot P_{drift\_3}$

$P_{sl} := P_{sl\_1} + P_{sl\_2}$ **Total Snow Load** $P_{sl} = 5.4\ \frac{k}{ft}$

$P_{dlr} := x_g \cdot DL \cdot \frac{s_{jst}}{2} + x_{grdr} \cdot DL \cdot \left(\frac{w_{grdr} - s_{jst}}{2}\right)$ **Roof Dead Load** $P_{dlr} = 1.5\ \frac{k}{ft}$

$P_{dlp} := x_g \cdot t \cdot 150\ pcf \cdot \left(\frac{ht}{2} + h_p\right)$ **Panel Dead Load** $P_{dlp} = 2.4\ \frac{k}{ft}$

$M_w := x_{OOP} \cdot \frac{p \cdot ht^2}{8}$ **Moment Due To Wind** $M_w = 2.5\ \frac{k \cdot ft}{ft}$

$M_s := x_{OOP} \cdot \frac{F_p \cdot t \cdot 150\ pcf \cdot ht^2}{8}$ **Moment Due to Seismic** $M_s = 6.7\ \frac{k \cdot ft}{ft}$

Strength Load Combinations (LRFD)

(ASCE 7-16 2.3)

Service/Deflection Load Combinations (ASD)

(2021 IBC 1605.2)

$\omega := 1.0$

$\begin{bmatrix} 1.2 \cdot P_{dlr} & 1.2 \cdot P_{dlp} & 1.6 \cdot P_{sl} & 0.5 \cdot M_w \\ 1.2 \cdot P_{dlr} & 1.2 \cdot P_{dlp} & 0.5 \cdot P_{sl} & 1.0 \cdot M_w \end{bmatrix}$

$\begin{bmatrix} 1.0 \cdot P_{dlr} & 1.0 \cdot P_{dlp} & 1.0 \cdot P_{sl} & 0.5 \cdot (\omega \cdot 0.6 \cdot M_w) \\ 1.0 \cdot P_{dlr} & 1.0 \cdot P_{dlp} & 0.5 \cdot P_{sl} & 1.0 \cdot (\omega \cdot 0.6 \cdot M_w) \end{bmatrix}$

$$x := \begin{bmatrix} 1.2 \cdot P_{dlr} & 1.2 \cdot P_{dlp} & 0.2 \cdot P_{sl} & 1.0 \cdot M_s \\ 0.9 \cdot P_{dlr} & 0.9 \cdot P_{dlp} & 0 \cdot \frac{k}{ft} & 1.0 \cdot M_w \\ 0.9 \cdot P_{dlr} & 0.9 \cdot P_{dlp} & 0 \cdot \frac{k}{ft} & 1.0 \cdot M_s \end{bmatrix} \qquad y := \begin{bmatrix} 1.0 \cdot P_{dlr} & 1.0 \cdot P_{dlp} & 1.0 \cdot P_{sl} & \left(\frac{1}{1.4}\right) \cdot M_s \\ 0.67 \cdot P_{dlr} & 0.67 \cdot P_{dlp} & 0 \cdot \frac{k}{ft} & 1.0 \cdot (\omega \cdot 0.6 \cdot M_w) \\ 0.9 \cdot P_{dlr} & 0.9 \cdot P_{dlp} & 0 \cdot \frac{k}{ft} & \left(\frac{1}{1.4}\right) \cdot M_s \end{bmatrix}$$

$i := 0 .. \text{rows}(x) - 1$

Define Panel Reinforcing

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

$NB := 10$ — Vertical Bar Spacing

$$S := \frac{L - 1.5\ in}{NB - 1} = 6.5\ in$$

$flag := \text{if}(S > 3\ in, \text{"OK"}, \text{"Revise"}) = \text{"OK"}$

$rebar\_location := EF$ — Steel Layer: Each Face (EF) or Centerline (CL)

$stirrup\_size := NA$ — Stirrup Bar Size

$A_b := A_{bar\_size}$ — Vertical Bar Area

$diam_b := D_{bar\_size}$ — Vertical Bar Diameter

$diam_{str} := D_{stirrup\_size}$ — Stirrup Bar Diameter

$$d_{ef} := t - \frac{diam_b}{2} - diam_{str} - clr - r_v$$ Steel Depth If Each Face

$$d_{cl} := \frac{t}{2} - r_v$$ Steel Depth If Centerline

$d := \text{if}(rebar\_location = 0, d_{ef}, d_{cl})$ — Steel Depth — $d = 6.94\ in$

$$A_s := \frac{NB \cdot A_b}{L}$$ Vertical Area of Steel

Check Axial Capacity

(ACI 318-19 11.8.1.1(d))

$$P_{u_i} := \left\| \begin{array}{l} P_u \leftarrow 0 \\ \left(x_{i,0} + x_{i,1} + x_{i,2}\right) \end{array} \right.$$

$P_a := 0.06 \cdot f_c \cdot t$

Concrete Panel Out-of-Plane Analysis

$$flag_i := \text{if}\left(\frac{P_{u_i}}{P_a} \le 1, \text{"OK"}, \text{"Revise"}\right)$$

$$\frac{P_u}{P_a} = \begin{bmatrix} 0.5 \\ 0.3 \\ 0.2 \\ 0.1 \\ 0.1 \end{bmatrix} \qquad flag = \begin{bmatrix} \text{"OK"} \\ \text{"OK"} \\ \text{"OK"} \\ \text{"OK"} \\ \text{"OK"} \end{bmatrix}$$

$$compression\_check := flag$$

Determine Moment Capacity

$$\beta_1 := 0.85 - 0.05 \cdot \left(\frac{f_c - 4000 \cdot psi}{1000 \cdot psi}\right) \qquad P_u := P_u \cdot ft \qquad A_s := A_s \cdot ft$$

$$X_i := \frac{P_{u_i} + A_s \cdot f_y}{0.85 \cdot f_c \cdot ft} \qquad M_{n_i} := \left(A_s \cdot f_y + P_{u_i}\right) \cdot \left(d - \frac{X_i}{2}\right) - P_{u_i} \cdot \left(d - \frac{t}{2}\right)$$

$$\phi M_{n_i} := 0.9 \cdot M_{n_i}$$

Factored Moment Capacity

$$\phi M_n = \begin{bmatrix} 21.4 \\ 19.9 \\ 19.5 \\ 19 \\ 19 \end{bmatrix} k \cdot ft$$

Check for Tension-Controlled Section

(ACI 318-19 11.8.1.1(b))

$$C_i := \frac{X_i}{\beta_1} \qquad flag_i := \text{if}\left(\frac{C_i}{d} < 0.375, \text{"OK"}, \text{"Revise"}\right)$$

$$\frac{C}{d} = \begin{bmatrix} 0.209 \\ 0.185 \\ 0.178 \\ 0.169 \\ 0.169 \end{bmatrix} \qquad flag = \begin{bmatrix} \text{"OK"} \\ \text{"OK"} \\ \text{"OK"} \\ \text{"OK"} \\ \text{"OK"} \end{bmatrix}$$

$$tension\_controlled := flag$$

Determine Deflection Limits

$$\Delta_{max} := \frac{ht}{150}$$

Allowable Panel Deflection

$$ht' := \text{if}\left(DockHigh = 0, ht, 0.8 \cdot ht\right) = 38\ ft$$

Adjust Span per Dock High Condition

$$E_c := 57000 \cdot \sqrt{\frac{f_c}{psi}} \cdot psi \qquad n := \frac{29000 \cdot ksi}{E_c} \qquad A_{se_i} := A_s + \frac{P_{u_i} \cdot t}{2 \cdot f_y \cdot d} \qquad C_i := \frac{X_i}{0.85}$$

$$f_r := 7.5 \cdot \sqrt{\frac{f_c}{psi}} \cdot psi \qquad I_g := \frac{(1\ ft) \cdot t^3}{12} \qquad I_{cr_i} := \frac{12 \cdot in \cdot \left(C_i\right)^3}{3} + n \cdot A_{se_i} \cdot \left(d - C_i\right)^2$$

$$M_{cr} := \frac{2 \cdot f_r \cdot I_g}{t} \qquad \Delta_{cr} := \frac{5 \cdot M_{cr} \cdot ht'^2}{48 \cdot E_c \cdot I_g} \qquad \Delta_{n_i} := \frac{5 \cdot M_{n_i} \cdot ht'^2}{48 \cdot E_c \cdot I_{cr_i}}$$

Check Moment Capacity

(ACI 318-19 11.8.3.1)

$$M_{ua_i} := \left(x_{i,3} + \left(x_{i,0} + x_{i,2}\right) \cdot \frac{e}{2}\right) \cdot ft \qquad M_{u_i} := \frac{M_{ua_i}}{5 \cdot P_{u_i} \cdot ht'^2}$$

$$1-\frac{\quad}{0.75\cdot 48\cdot E_c\cdot I_{cr_i}}$$

$$flag_i := \text{if}\left(\frac{M_{u_i}}{\phi M_{n_i}} \le 1, \text{"OK"}, \text{"Revise"}\right)$$

$$\frac{M_u}{\phi M_n} = \begin{bmatrix} 0.5 \\ 0.3 \\ 0.5 \\ 0.2 \\ 0.4 \end{bmatrix} \qquad flag = \begin{bmatrix} \text{"OK"} \\ \text{"OK"} \\ \text{"OK"} \\ \text{"OK"} \\ \text{"OK"} \end{bmatrix}$$

$$capacity\_check := flag$$

Check Deflection

(ACI 318-19 11.8.4)

$$M_{a_i} := \left(y_{i,3} + \left(y_{i,0} + y_{i,2}\right)\cdot\frac{e}{2}\right) ft \qquad P_{s_i} := \left(y_{i,0} + y_{i,1} + y_{i,2}\right)\cdot ft$$

Iterate Service Deflection

$$\Delta_{s_i} := 0\cdot in$$ Initial Guess

$$\Delta_s := \Delta f_s\left(M_a, P_s, \Delta_s, M_{cr}, \Delta_{cr}, M_n, \Delta_n\right)$$ Function iterates service deflections per ACI 318-19 *Sec 11.8.4*

$$flag_i := \text{if}\left(\frac{\Delta_{s_i}}{\Delta_{max}} \le 1, \text{"OK"}, \text{"Revise"}\right)$$

$$\frac{\Delta_s}{\Delta_{max}} = \begin{bmatrix} 0.1 \\ 0.1 \\ 0.7 \\ 0.1 \\ 0.3 \end{bmatrix} \qquad flag = \begin{bmatrix} \text{"OK"} \\ \text{"OK"} \\ \text{"OK"} \\ \text{"OK"} \\ \text{"OK"} \end{bmatrix}$$

$$deflection\_check := flag$$

Results Summary

Panel Thickness $t = 8.8\ in$ **Bar Size** $rebar_{bar\_size} = \text{"\#5"}$ **Bar Spacing** $NB = 10$

Bar Location $layer = \text{"EF"}$

Concrete Panel Out-of-Plane Analysis

Panel w/ Storefront Opening w/ Canopy (P52) 10" THK, (18) #5 VERT EF

Panel Geometry

$t := 10\ in$	Panel Thickness
$ht := 37\ ft$	Roof Height (Panel Span)
$h_p := 6.5\ ft$	Parapet Height
$DockHigh := \text{No}$	Dock High Span Reduction
$w := 26.5\ ft$	Panel Width
$L := 4\ ft + 9\ in$	Panel Leg Width
$ht_o := 12\ ft + 0\ in$	Panel Opening Height
$w_o := 17\ ft$	Panel Opening Width
$e := \frac{t}{2} + 3 \cdot in = 8\ in$	Roof Load Eccentricity
$h_T := ht + h_p = 43.5\ ft$	Total Panel Height

Panel Load Factors

$x_g := 1 + \frac{w_o}{2 \cdot L} = 2.8$	Gravity Factor
$x_{OOP} := 1 + \frac{w_o}{2 \cdot L} = 2.8$	OOP Factor

Panel Wind Loading Per ASCE 7-16 Chapter 30 - Part 1 Method

$GCpi := 0.18$ Coefficient per Table 26.13-1

$$A_{eff} := \frac{1}{3}\ ht^2 = 456.3\ ft^2$$

$Slope := \text{Y}$ If roof slope is <10 degrees, GCp can be reduced by 10%

$$\ln\left(\frac{A_{eff}}{ft^2}\right) - \ln(10)$$

Positive pressure coefficient

Concrete Panel Out-of-Plane Analysis

$$GCp_{poscalc} := \left(1.0 - \frac{\ldots}{\ln(500) - \ln(10)}(1-0.7)\right)(1-Slope)$$

(zones 4 and 5)

$$GCp_{negcalc} := \left(1.4 - \frac{\ln\left(\frac{A_{eff}}{ft^2}\right) - \ln(10)}{\ln(500) - \ln(10)}(1.4-0.8)\right)(1-Slope)(-1)$$

Negative pressure coefficient (zone 5)

$$\begin{aligned} GCp_{pos} := &\text{ if } A_{eff} > 500\ ft^2 \\ &\quad \| 0.7\ (1-Slope) \\ &\text{else if } A_{eff} < 10\ ft^2 \\ &\quad \| 1.0\ (1-Slope) \\ &\text{else} \\ &\quad \| GCp_{poscalc} \end{aligned}$$

Check against upper and lower bound values for CGp based on Aeff

$$\begin{aligned} GCp_{neg} := &\text{ if } A_{eff} > 500\ ft^2 \\ &\quad \| -0.8\ (1-Slope) \\ &\text{else if } A_{eff} < 10\ ft^2 \\ &\quad \| -1.4\ (1-Slope) \\ &\text{else} \\ &\quad \| GCp_{negcalc} \end{aligned}$$

$$p_1 := q_h \cdot (GCp_{pos} - GCpi)$$

$$p_2 := q_h \cdot (GCp_{neg} - GCpi)$$

$$p_3 := q_h \cdot (GCp_{pos} + GCpi)$$

$$p_4 := q_h \cdot (GCp_{neg} + GCpi)$$

$$p := \max(|p_1|, |p_2|, |p_3|, |p_4|) = 13.8\ psf$$

Panel Loading

$w_{rf} := 10\ ft$ Joist Span/Spacing Adjacent to Panel

$load_{drift} := 84\ psf$ Drift Load at Panel

$w_{drift} := 20.25\ ft$ Drift Length

Concrete Panel Out-of-Plane Analysis

$$P_{drift} := \text{if } w_{rf} \geq w_{drift}$$
$$\quad \left\| P \leftarrow load_{drift} \cdot w_{drift} \cdot \left(\frac{1}{2} - \frac{w_{drift}}{6\ w_{rf}}\right)\right.$$
$$\text{else if } w_{rf} < w_{drift}$$
$$\quad \left\| P \leftarrow load_{drift} \cdot w_{rf} \cdot \left(\frac{1}{2} - \frac{w_{rf}}{6 \cdot w_{drift}}\right)\right.$$

Snow Drift Reaction at Panel $P_{drift} = 0.4\ \frac{k}{ft}$

$$P_{dlr} := x_g \cdot DL \cdot \frac{w_{rf}}{2}$$

Roof Dead Load $P_{dlr} = 0.2\ \frac{k}{ft}$

$$P_{dlp} := x_g \cdot 150\ pcf \cdot \left(\frac{ht}{2} + h_p\right) \cdot t$$

Panel Dead Load $P_{dlp} = 8.7\ \frac{k}{ft}$

$$P_{sl} := x_g \cdot \left(SL \cdot \frac{w_{rf}}{2} + P_{drift}\right)$$

Total Snow Load $P_{sl} = 1.2\ \frac{k}{ft}$

$$P_{dl\_mezz} := x_g \cdot \left(25\ psf \cdot \frac{20\ ft}{2}\right)$$

$P_{dl\_mezz} = 0.7\ \frac{k}{ft}$

$$P_{ll\_mezz} := x_g \cdot \left(65\ psf \cdot \frac{20\ ft}{2}\right)$$

$P_{ll\_mezz} = 1.8\ \frac{k}{ft}$

$$M_w := x_{OOP} \cdot \frac{p \cdot ht^2}{8}$$

Moment Due To Wind $M_w = 6.6\ \frac{k \cdot ft}{ft}$

$$w_1 := F_p \cdot 150\ pcf \cdot t \cdot x_{OOP}$$

$$w_2 := \left(F_p \cdot 10 \cdot psf \cdot w_o \cdot 0.5 + F_p \cdot 150\ pcf \cdot t \cdot L\right) \cdot \frac{1}{L}$$

$$R_{bot} := \frac{w_2 \cdot ht}{2} + \frac{\left(w_1 - w_2\right) \cdot \left(ht - ht_o\right)^2}{2 \cdot ht}$$

$$R_{top} := \left(w_2 \cdot ht_o + w_1 \cdot \left(ht - ht_o\right)\right) - R_{bot}$$

$$x_m := \frac{R_{top}}{w_1}$$

$$M_s := R_{top} \cdot x_m - \frac{w_1 \cdot {x_m}^2}{2}$$

Moment Due to Seismic w/ Opening $M_s = 17.7\ \frac{k \cdot ft}{ft}$

Canopy Loading

$$P_{dl\_c} := \frac{433\ lbf}{L}$$

Canopy Dead Load $P_{dl\_c} = 0.1\ \frac{k}{ft}$

$M_{dl\_c} := 0.0 \cdot \frac{k \cdot ft}{ft}$ **Canopy Dead Load (Moment)** $M_{dl\_c} = 0 \ \frac{k \cdot ft}{ft}$

$P_{sl\_c} := \frac{3.03\ k}{L}$ **Canopy Snow Load** $P_{sl\_c} = 0.6 \ \frac{k}{ft}$

$M_{sl\_c} := 0.0 \cdot \frac{k \cdot ft}{ft}$ **Canopy Snow Load (Moment)** $M_{sl\_c} = 0 \ \frac{k \cdot ft}{ft}$

$P_{w\_c} := \frac{529\ lbf}{L}$ **Canopy Wind Load** $P_{w\_c} = 0.1 \ \frac{k}{ft}$

$M_{w\_c} := 0 \cdot \frac{k \cdot ft}{ft}$ **Canopy Wind (Moment)** $M_{w\_c} = 0 \ \frac{k \cdot ft}{ft}$

Strength Load Combinations (LRFD) *(ASCE 7-16 2.3)*

$$x := \begin{bmatrix} 1.2 \cdot (P_{dlr} + P_{dl\_mezz}) & 1.2 \cdot P_{dlp} & 1.6 \cdot P_{sl} + 0.5\ P_{ll\_mezz} & 0\ \frac{k \cdot ft}{ft} & 1.2 \cdot P_{dl\_c} & 1.2 \cdot M_{dl\_c} & 1.6 \cdot P_{sl\_c} & 1.6 \cdot M_{sl\_c} & 0\ \frac{k}{ft} & 0.5 \cdot M_{w\_c} \\ 1.2 \cdot (P_{dlr} + P_{dl\_mezz}) & 1.2 \cdot P_{dlp} & 0.5 \cdot (P_{sl} + P_{ll\_mezz}) & 1.0 \cdot M_w & 1.2 \cdot P_{dl\_c} & 1.2 \cdot M_{dl\_c} & 0.5 \cdot P_{sl\_c} & 0.5 \cdot M_{sl\_c} & 1.0 \cdot P_{w\_c} & 1.0 \cdot M_{w\_c} \\ 1.2 \cdot (P_{dlr} + P_{dl\_mezz}) & 1.2 \cdot P_{dlp} & 0.5\ P_{ll\_mezz} + 0.2 \cdot P_{sl} & 1.0 \cdot M_s & 1.2 \cdot P_{dl\_c} & 1.2 \cdot M_{dl\_c} & 0.2 \cdot P_{sl\_c} & 0.2 \cdot M_{sl\_c} & 0\ \frac{k}{ft} & 0\ \frac{k \cdot ft}{ft} \\ 0.9 \cdot (P_{dlr} + P_{dl\_mezz}) & 0.9 \cdot P_{dlp} & 0 \cdot \frac{k}{ft} & 1.0 \cdot M_w & 0.9 \cdot P_{dl\_c} & 0.9 \cdot M_{dl\_c} & 0 \cdot \frac{k}{ft} & 0 \cdot \frac{k \cdot ft}{ft} & 1.0 \cdot P_{w\_c} & 1.0 \cdot M_{w\_c} \\ 0.9 \cdot (P_{dlr} + P_{dl\_mezz}) & 0.9 \cdot P_{dlp} & 0 \cdot \frac{k}{ft} & 1.0 \cdot M_s & 0.9 \cdot P_{dl\_c} & 0.9 \cdot M_{dl\_c} & 0 \cdot \frac{k}{ft} & 0 \cdot \frac{k \cdot ft}{ft} & 0\ \frac{k}{ft} & 0 \cdot \frac{k \cdot ft}{ft} \end{bmatrix}$$

Service/Deflection Load Combinations (ASD) *(2021 IBC 1605.2)*

$\omega := 1.0$ $i := 0 .. \mathrm{rows}(x) - 1$

$$y := \begin{bmatrix} 1.0 \cdot (P_{dlr} + P_{dl\_mezz}) & 1.0 \cdot P_{dlp} & P_{sl} + P_{ll\_mezz} & \frac{\omega \cdot 0.6 \cdot M_w}{2} & 1.0 \cdot P_{dl\_c} & 1.0 \cdot M_{dl\_c} & 1.0 \cdot P_{sl\_c} & 1.0 \cdot M_{sl\_c} & \frac{\omega \cdot 0.6 \cdot P_{w\_c}}{2} & \frac{\omega \cdot 0.6 \cdot M_{w\_c}}{2} \\ 1.0 \cdot (P_{dlr} + P_{dl\_mezz}) & 1.0 \cdot P_{dlp} & P_{ll\_mezz} + 0.5 \cdot P_{sl} & \omega \cdot 0.6 \cdot M_w & 1.0 \cdot P_{dl\_c} & 1.0 \cdot M_{dl\_c} & 0.5 \cdot P_{sl\_c} & 0.5 \cdot M_{sl\_c} & \omega \cdot 0.6 \cdot P_{w\_c} & \omega \cdot 0.6 \cdot M_{w\_c} \\ 1.0 \cdot (P_{dlr} + P_{dl\_mezz}) & 1.0 \cdot P_{dlp} & P_{sl} + P_{ll\_mezz} & \left(\frac{1}{1.4}\right) \cdot M_s & 1.0 \cdot P_{dl\_c} & 1.0 \cdot M_{dl\_c} & 1.0 \cdot P_{sl\_c} & 1.0 \cdot M_{sl\_c} & 0\ \frac{k}{ft} & 0 \cdot \frac{k \cdot ft}{ft} \\ 0.67 \cdot (P_{dlr} + P_{dl\_mezz}) & 0.67 \cdot P_{dlp} & 0 \cdot \frac{k}{ft} & \omega \cdot 0.6 \cdot M_w & 0.67 \cdot P_{dl\_c} & 0.67 \cdot M_{dl\_c} & 0 \cdot \frac{k}{ft} & 0 \cdot \frac{k \cdot ft}{ft} & \omega \cdot 0.6 \cdot P_{w\_c} & \omega \cdot 0.6 \cdot M_{w\_c} \\ 0.9 \cdot (P_{dlr} + P_{dl\_mezz}) & 0.9 \cdot P_{dlp} & 0 \cdot \frac{k}{ft} & \left(\frac{1}{1.4}\right) \cdot M_s & 0.9 \cdot P_{dl\_c} & 0.9 \cdot M_{dl\_c} & 0 \cdot \frac{k}{ft} & 0 \cdot \frac{k \cdot ft}{ft} & 0\ \frac{k}{ft} & 0\ \frac{k \cdot ft}{ft} \end{bmatrix}$$

Concrete Panel Out-of-Plane Analysis

Define Panel Reinforcing

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

$NB := 18$ — Vertical Bar Spacing

$rebar\_location := \text{EF}$ — Steel Layer: Each Face (EF) or Centerline (CL)

$stirrup\_size := \#3$ — Stirrup Bar Size

$A_b := A_{bar\_size}$ — Vertical Bar Area

$diam_b := D_{bar\_size}$ — Vertical Bar Diameter

$diam_{str} := D_{stirrup\_size}$ — Stirrup Bar Diameter

$$d_{ef} := t - \frac{diam_b}{2} - diam_{str} - clr - r_v$$ Steel Depth If Each Face

$$d_{cl} := \frac{t}{2} - r_v$$ Steel Depth If Centerline

$d := \text{if}\left(rebar\_location = 0, d_{ef}, d_{cl}\right)$ — Steel Depth — $d = 7.81\ in$

$S := \dfrac{L - 1.5\ in}{NB - 1} = 3.3\ in$ — Vertical Bar Spacing OC

$flag := \text{if}\left(S \geq 3\ in, \text{"OK"}, \text{"Revise"}\right)$ — $S = 3.3\ in$ — $flag = \text{"OK"}$

$A_s := \dfrac{NB \cdot A_b}{L} = 1.17\ \dfrac{in^2}{ft}$ — Vertical Area of Steel

Check Axial Capacity

(ACI 318-19 11.8.1.1(d))

$$P_{u_i} := \left\| \begin{array}{l} P_u \leftarrow 0 \\ \left(x_{i,0} + x_{i,1} + x_{i,2} + x_{i,4} + x_{i,6} + x_{i,8}\right) \end{array} \right.$$

$$P_a := 0.06 \cdot f_c \cdot t$$

$$flag_i := \text{if}\left(\frac{P_{u_i}}{P_a} \leq 1, \text{"OK"}, \text{"Revise"}\right)$$

$$\frac{P_u}{P_a} = \begin{bmatrix} 0.5 \\ 0.5 \\ 0.4 \\ 0.3 \\ 0.3 \end{bmatrix} \qquad flag = \begin{bmatrix} \text{"OK"} \\ \text{"OK"} \\ \text{"OK"} \\ \text{"OK"} \\ \text{"OK"} \end{bmatrix}$$

$compression\_check := flag$

Determine Moment Capacity

$$\beta_1 := 0.85 - 0.05 \cdot \left(\frac{f_c - 4000 \cdot psi}{1000 \cdot psi}\right)$$

$P_u := P_u \cdot ft \qquad A_s := A_s \cdot ft$

$X_i := \frac{\ldots}{0.85 \cdot f_c \cdot ft}$ $\quad M_{n_i} := \left(A_s \cdot f_y + P_{u_i}\right) \cdot \left(d - \frac{X_i}{2}\right) - P_{u_i} \cdot \left(d - \frac{t}{2}\right)$

$\phi M_{n_i} := 0.9 \cdot M_{n_i}$ Factored Moment Capacity $\phi M_n = \begin{bmatrix} 40.3 \\ 39.9 \\ 39.8 \\ 38.8 \\ 38.8 \end{bmatrix} k \cdot ft$

Check for Tension-Controlled Section

(ACI 318-19 11.8.1.1(b))

$C_i := \frac{X_i}{\beta_1}$

$flag_i := \text{if}\left(\frac{C_i}{d} < 0.375, \text{“OK”}, \text{“Revise”}\right)$

$tension\_controlled := flag$

$\frac{C}{d} = \begin{bmatrix} 0.317 \\ 0.31 \\ 0.308 \\ 0.293 \\ 0.292 \end{bmatrix}$ $\quad flag = \begin{bmatrix} \text{“OK”} \\ \text{“OK”} \\ \text{“OK”} \\ \text{“OK”} \\ \text{“OK”} \end{bmatrix}$

Determine Deflection Limits

$\Delta_{max} := \frac{ht}{150}$ Allowable Panel Deflection

$ht' := \text{if}\left(DockHigh = 0, ht, 0.8 \cdot ht\right) = 37\ ft$ Adjust Span per Dock High Condition

$E_c := 57000 \cdot \sqrt{\frac{f_c}{psi}} \cdot psi$ $\quad n := \frac{29000 \cdot ksi}{E_c}$ $\quad A_{se_i} := A_s + \frac{P_{u_i} \cdot t}{2 \cdot f_y \cdot d}$ $\quad C_i := \frac{X_i}{0.85}$

$f_r := 7.5 \cdot \sqrt{\frac{f_c}{psi}} \cdot psi$ $\quad I_g := \frac{(1\ ft) \cdot t^3}{12}$ $\quad I_{cr_i} := \frac{12 \cdot in \cdot \left(C_i\right)^3}{3} + n \cdot A_{se_i} \cdot \left(d - C_i\right)^2$

$M_{cr} := \frac{2 \cdot f_r \cdot I_g}{t}$ $\quad \Delta_{cr} := \frac{5 \cdot M_{cr} \cdot ht'^2}{48 \cdot E_c \cdot I_g}$ $\quad \Delta_{n_i} := \frac{5 \cdot M_{n_i} \cdot ht'^2}{48 \cdot E_c \cdot I_{cr_i}}$

Check Moment Capacity

(ACI 318-19 11.8.3.1)

$M_{ua_i} := \left(x_{i,3} + \left(x_{i,0} + x_{i,2}\right) \cdot \frac{e}{2} + x_{i,5} + x_{i,7} + x_{i,9}\right) \cdot ft$ $\quad M_{u_i} := \frac{M_{ua_i}}{1 - \frac{5 \cdot P_{u_i} \cdot ht'^2}{0.75 \cdot 48 \cdot E_c \cdot I_{cr_i}}}$

$flag_i := \text{if}\left(\frac{M_{u_i}}{\phi M_{n_i}} \le 1, \text{“OK”}, \text{“Revise”}\right)$

$capacity\_check := flag$

$\frac{M_u}{\phi M_n} = \begin{bmatrix} 0 \\ 0.3 \\ 0.6 \\ 0.2 \\ 0.6 \end{bmatrix}$ $\quad flag = \begin{bmatrix} \text{“OK”} \\ \text{“OK”} \\ \text{“OK”} \\ \text{“OK”} \\ \text{“OK”} \end{bmatrix}$

Check Deflection

(ACI 318-19 11.8.4)

$$M_{a_i} := \left(y_{i,3} + \left(y_{i,0} + y_{i,2}\right) \cdot \frac{e}{2} + y_{i,5} + y_{i,7} + y_{i,9}\right) ft \qquad P_{s_i} := \left(y_{i,0} + y_{i,1} + y_{i,2} + y_{i,4} + y_{i,6} + y_{i,8}\right) \cdot ft$$

Iterate Service Deflection

$\Delta_{s_i} := 0 \cdot in$ Initial Guess

$\Delta_s := \Delta f_s\left(M_a, P_s, \Delta_s, M_{cr}, \Delta_{cr}, M_n, \Delta_n\right)$ Function iterates service deflections per ACI 318-19 *Sec 11.8.4*

$$flag_i := \text{if}\left(\frac{\Delta_{s_i}}{\Delta_{max}} \le 1, \text{“OK”}, \text{“Revise”}\right)$$

$$\frac{\Delta_s}{\Delta_{max}} = \begin{bmatrix} 0.1 \\ 0.1 \\ 0.8 \\ 0.1 \\ 0.7 \end{bmatrix} \qquad flag = \begin{bmatrix} \text{“OK”} \\ \text{“OK”} \\ \text{“OK”} \\ \text{“OK”} \\ \text{“OK”} \end{bmatrix}$$

$deflection\_check := flag$

Results Summary

Panel Thickness	$t = 10\ in$	**Bar Size**	$rebar_{bar\_size} = \text{“\#5”}$
Panel Leg Width	$L = 4.8\ ft$	**Bar Quantity**	$NB = 18$
		Bar Location	$layer = \text{“EF”}$

Concrete Panel Out-of-Plane Analysis

Purpose Statement:

Out-of-Plane (OOP) Design of Concrete Tilt Up Panels

Referenced Standards:

2021 IBC
ASCE 7-16
ACI 318-19

General Design Criteria

Inputs
Verify
Outputs

$r_v := 0.75\ in$ — Reveal Depth

$clr := 0.75\ in$ — Concrete Clear Cover

$f_c := 4000 \cdot psi$ — Concrete Compressive Strength

$f_y := 60 \cdot ksi$ — Reinforcement Yield Strength

$Risk := \text{II}$ — Risk Category

General Loading

Gravity

$DL := 14.9 \cdot psf$ — Roof Dead Load

$SL := 14 \cdot psf$ — Roof Snow Load

Wind *(ASCE 7-16 30.4)*

$S_w := 97 \cdot mph$ — Mean Wind Speed

$Exposure := \text{B}$ — Exposure Category

$h_{mean} := 37\ ft$ — Mean Roof Height

$K_z := 0.74$ — *(Table 26.10-1)*

$K_{zt} := 1.0$ — *(Section 26.8)*

$K_d := 0.85$ — *(Table 26.6-1)*

$K_e := 1.0$ — *(Table 26.9-1)*

$$q_h := 0.00256\ K_{zt}\ K_z\ K_d\ K_e \left(\frac{S_w}{mph}\right)^2 \cdot 1\ psf = 15.15\ psf$$

Concrete Panel Out-of-Plane Analysis

Seismic (ASCE 7-16 12.11.1)

$I_e = 1$ Importance Factor - Seismic (Table 1.5-2)

$S_{DS} := 0.843$ Design Spectral Acceleration

$F_p := \max\left(0.4 \cdot S_{DS} \cdot I_e, 0.1\right) = 0.34$ Seismic OOP coefficient

green shading is done

Concrete Panel Out-of-Plane Analysis

Blade Wall

Blade Wall 5" THK w/ #5 @ 9" OC CL

Panel Geometry

$t := 5\ in$	Panel Thickness
$ht := 27\ ft + 0\ in$	Roof Height (Panel Span)
$h_p := 0$	Parapet Height
$DockHigh :=$ No	Dock High Span Reduction
$w := 18\ ft + 6\ in$	Panel Width
$e := \frac{t}{2} + 3 \cdot in = 5.5\ in$	Roof Load Eccentricity
$h_T := ht + h_p = 27\ ft$	Total Panel Height

Panel Load Factors

$L := 1\ ft$	Panel Width Considered (PWC) for girder/pad ftg
$x_g := 1$	Gravity Factor
$x_{OOP} := 1.0$	OOP Factor

Panel Wind Loading Per ASCE 7-16 Chapter 30 - Part 1 Method

$GCpi := 0.18$ Internal Pressure Coefficient per Table 26.13-1

$$A_{eff} := \frac{1}{3}\ ht^2 = 243\ ft^2$$

$Slope :=$ Y

If roof slope is <10 degrees, GCp can be reduced by 10%

$$GCp_{poscalc} := \left(1.0 - \frac{\ln\left(\frac{A_{eff}}{ft^2}\right) - \ln(10)}{\ln(500) - \ln(10)} (1 - 0.7)\right) (1 - Slope)$$

Positive pressure coefficient (zones 4 and 5)

$$GCp_{negcalc} := \left(1.4 - \frac{\ln\left(\frac{A_{eff}}{ft^2}\right) - \ln(10)}{\ln(500) - \ln(10)} (1.4 - 0.8)\right)(1 - Slope)(-1)$$

Negative pressure coefficient (zone 5)

$$GCp_{pos} := \begin{cases} 0.7\,(1 - Slope) & \text{if } A_{eff} > 500\ ft^2 \\ 1.0\,(1 - Slope) & \text{else if } A_{eff} < 10\ ft^2 \\ GCp_{poscalc} & \text{else} \end{cases}$$

Check against upper and lower bound values for CGp based on Aeff

$$GCp_{neg} := \begin{cases} -0.8\,(1 - Slope) & \text{if } A_{eff} > 500\ ft^2 \\ -1.4\,(1 - Slope) & \text{else if } A_{eff} < 10\ ft^2 \\ GCp_{negcalc} & \text{else} \end{cases}$$

$$p_1 := q_h \cdot (GCp_{pos} - GCpi)$$

$$p_2 := q_h \cdot (GCp_{neg} - GCpi)$$

$$p_3 := q_h \cdot (GCp_{pos} + GCpi)$$

$$p_4 := q_h \cdot (GCp_{neg} + GCpi)$$

$$p_{net} := \max\left(|p_1|, |p_2|, |p_3|, |p_4|\right) = 15.1\ psf$$

Panel Loading

$w_{rf} := 0\ ft + 0\ in$ — Joist Span/Spacing Adjacent to Panel

$load_{drift} := 0\ psf$ — Drift Load at Panel

$w_{drift} := 17\ ft + 0\ in$ — Drift Length

Concrete Panel Out-of-Plane Analysis

$$P_{drift} := \text{if } w_{rf} \geq w_{drift} \;\; \left\| P \leftarrow load_{drift} \cdot w_{drift} \cdot \left(\frac{1}{2} - \frac{w_{drift}}{6\ w_{rf}}\right) \right. \;\; \text{else if } w_{rf} < w_{drift} \;\; \left\| P \leftarrow load_{drift} \cdot w_{rf} \cdot \left(\frac{1}{2} - \frac{w_{rf}}{6 \cdot w_{drift}}\right) \right.$$

Snow Drift Reaction at Panel $P_{drift} = 0\ \frac{k}{ft}$

$$P_{dlr} := x_g \cdot DL \cdot \frac{w_{rf}}{2}$$

Roof Dead Load + Mezzanine Dead Load $P_{dlr} = 0\ \frac{k}{ft}$

$$P_{dlp} := x_g \cdot t \cdot 150\ pcf \cdot \left(\frac{ht}{2} + h_p\right)$$

Panel Dead Load $P_{dlp} = 0.8\ \frac{k}{ft}$

$$P_{sl} := x_g \cdot \left(SL \cdot \frac{w_{rf}}{2} + P_{drift}\right)$$

Total Snow Load $P_{sl} = 0\ \frac{k}{ft}$

$$P_{ll\_mezz} := 0\ plf$$

Mezzanine Live Load $P_{ll\_mezz} = 0\ \frac{k}{ft}$

$$M_w := x_{OOP} \cdot \frac{p_{net} \cdot ht^2}{8}$$

Moment Due To Wind $M_w = 1.4\ \frac{k \cdot ft}{ft}$

$$M_s := x_{OOP} \cdot \frac{F_p \cdot t \cdot 150\ pcf \cdot ht^2}{8}$$

Moment Due to Seismic $M_s = 1.9\ \frac{k \cdot ft}{ft}$

Strength Load Combinations (LRFD)

(ASCE 7-16 2.3)

$$x := \begin{bmatrix} 1.2 \cdot P_{dlr} & 1.2 \cdot P_{dlp} & 1.6 \cdot P_{sl} & 0.5 \cdot M_w \\ 1.2 \cdot P_{dlr} & 1.2 \cdot P_{dlp} & 0.5 \cdot (P_{sl}) + P_{ll\_mezz} & 1.0 \cdot M_w \\ 1.2 \cdot P_{dlr} & 1.2 \cdot P_{dlp} & 0.2 \cdot P_{sl} + 0.5 \cdot P_{ll\_mezz} & 1.0 \cdot M_s \\ 0.9 \cdot P_{dlr} & 0.9 \cdot P_{dlp} & 0 \cdot \frac{k}{ft} & 1.0 \cdot M_w \\ 0.9 \cdot P_{dlr} & 0.9 \cdot P_{dlp} & 0 \cdot \frac{k}{ft} & 1.0 \cdot M_s \end{bmatrix}$$

$$i := 0 .. \text{rows}(x) - 1$$

Service/Deflection Load Combinations (ASD)

(2021 IBC 1605.2)

$$\omega := 1.0$$

$$y := \begin{bmatrix} 1.0 \cdot P_{dlr} & 1.0\ P_{dlp} & P_{sl} + P_{ll\_mezz} & 0.5 \cdot (\omega \cdot 0.6 \cdot M_w) \\ 1.0 \cdot P_{dlr} & 1.0 \cdot P_{dlp} & 0.5 \cdot P_{sl} + P_{ll\_mezz} & 1.0 \cdot (\omega \cdot 0.6 \cdot M_w) \\ 1.0 \cdot P_{dlr} & 1.0 \cdot P_{dlp} & P_{sl} + P_{ll\_mezz} & \left(\frac{1}{1.4}\right) \cdot M_s \\ 0.67 \cdot P_{dlr} & 0.67 \cdot P_{dlp} & 0 \cdot \frac{k}{ft} & 1.0 \cdot (\omega \cdot 0.6 \cdot M_w) \\ 0.9 \cdot P_{dlr} & 0.9 \cdot P_{dlp} & 0 \cdot \frac{k}{ft} & \left(\frac{1}{1.4}\right) \cdot M_s \end{bmatrix}$$

Define Panel Reinforcing

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

$SP := 9 \cdot in$ Vertical Bar Spacing

Concrete Panel Out-of-Plane Analysis

$rebar\_location := \text{CL}$ Steel Layer: Each Face (EF) or Centerline (CL)

$stirrup\_size := \text{NA}$ Stirrup Bar Size

$A_b := A_{bar\_size}$ Vertical Bar Area

$diam_b := D_{bar\_size}$ Vertical Bar Diameter

$diam_{str} := D_{stirrup\_size}$ Stirrup Bar Diameter

$d_{ef} := t - \frac{diam_b}{2} - diam_{str} - 2 \cdot 1.5\ in$ Steel Depth If Each Face

$d_{cl} := \frac{t}{2}$ Steel Depth If Centerline

$d := \text{if}\left(rebar\_location = 0, d_{ef}, d_{cl}\right)$ Steel Depth $d = 2.5\ in$

$A_s := \frac{A_b}{SP}$ Vertical Area of Steel

Check Axial Capacity

(ACI 318-19 11.8.1.1(d))

$$P_{u_i} := \left\| \begin{array}{l} P_u \leftarrow 0 \\ \left(x_{i,0} + x_{i,1} + x_{i,2}\right) \end{array} \right.$$

$P_a := 0.06 \cdot f_c \cdot t$

$$flag_i := \text{if}\left(\frac{P_{u_i}}{P_a} \le 1, \text{“OK”}, \text{“Revise”}\right)$$

$$\frac{P_u}{P_a} = \begin{bmatrix} 0.1 \\ 0.1 \\ 0.1 \\ 0.1 \\ 0.1 \end{bmatrix} \qquad flag = \begin{bmatrix} \text{“OK”} \\ \text{“OK”} \\ \text{“OK”} \\ \text{“OK”} \\ \text{“OK”} \end{bmatrix}$$

$compression\_check := flag$

Determine Moment Capacity

$$\beta_1 := 0.85 - 0.05 \cdot \left(\frac{f_c - 4000 \cdot psi}{1000 \cdot psi}\right) \qquad P_u := P_u \cdot ft \qquad A_s := A_s \cdot ft$$

$$X_i := \frac{P_{u_i} + A_s \cdot f_y}{0.85 \cdot f_c \cdot ft} \qquad M_{n_i} := \left(A_s \cdot f_y + P_{u_i}\right) \cdot \left(d - \frac{X_i}{2}\right) - P_{u_i} \cdot \left(d - \frac{t}{2}\right)$$

$\phi M_{n_i} := 0.9 \cdot M_{n_i}$ Factored Moment Capacity

$$\phi M_n = \begin{bmatrix} 4.2 \\ 4.2 \\ 4.2 \\ 4.2 \\ 4.2 \end{bmatrix} k \cdot ft$$

Concrete Panel Out-of-Plane Analysis

Check for Tension-Controlled Section

(ACI 318-19 11.8.1.1(b))

$$C_i := \frac{X_i}{\beta_1}$$

$$flag_i := \text{if}\left(\frac{C_i}{d} < 0.375, \text{“OK”}, \text{“Revise”}\right)$$

$$tension\_controlled := flag$$

$$\frac{C}{d} = \begin{bmatrix} 0.298 \\ 0.298 \\ 0.298 \\ 0.295 \\ 0.295 \end{bmatrix} \qquad flag = \begin{bmatrix} \text{“OK”} \\ \text{“OK”} \\ \text{“OK”} \\ \text{“OK”} \\ \text{“OK”} \end{bmatrix}$$

Determine Deflection Limits

$$\Delta_{max} := \frac{ht}{150}$$

Allowable Panel Deflection

$$ht' := \text{if}\left(DockHigh = 0, ht, 0.8 \cdot ht\right) = 27\ ft$$

Adjust Span per Dock High Condition

$$E_c := 57000 \cdot \sqrt{\frac{f_c}{psi}} \cdot psi \qquad n := \frac{29000 \cdot ksi}{E_c} \qquad A_{se_i} := A_s + \frac{P_{u_i} \cdot t}{2 \cdot f_y \cdot d} \qquad C_i := \frac{X_i}{0.85}$$

$$f_r := 7.5 \cdot \sqrt{\frac{f_c}{psi}} \cdot psi \qquad I_g := \frac{(1\ ft) \cdot t^3}{12} \qquad I_{cr_i} := \frac{12 \cdot in \cdot \left(C_i\right)^3}{3} + n \cdot A_{se_i} \cdot \left(d - C_i\right)^2$$

$$M_{cr} := \frac{2 \cdot f_r \cdot I_g}{t} \qquad \Delta_{cr} := \frac{5 \cdot M_{cr} \cdot ht'^2}{48 \cdot E_c \cdot I_g} \qquad \Delta_{n_i} := \frac{5 \cdot M_{n_i} \cdot ht'^2}{48 \cdot E_c \cdot I_{cr_i}}$$

Check Moment Capacity

(ACI 318-19 11.8.3.1)

$$M_{ua_i} := \left(x_{i,3} + \left(x_{i,0} + x_{i,2}\right) \cdot \frac{e}{2}\right) \cdot ft \qquad M_{u_i} := \frac{M_{ua_i}}{1 - \dfrac{5 \cdot P_{u_i} \cdot ht'^2}{0.75 \cdot 48 \cdot E_c \cdot I_{cr_i}}}$$

$$flag_i := \text{if}\left(\frac{M_{u_i}}{\phi M_{n_i}} \le 1, \text{“OK”}, \text{“Revise”}\right)$$

$$capacity\_check := flag$$

$$\frac{M_u}{\phi M_n} = \begin{bmatrix} 0.24 \\ 0.49 \\ 0.68 \\ 0.44 \\ 0.61 \end{bmatrix} \qquad flag = \begin{bmatrix} \text{“OK”} \\ \text{“OK”} \\ \text{“OK”} \\ \text{“OK”} \\ \text{“OK”} \end{bmatrix}$$

Check Deflection

(ACI 318-19 11.8.4)

$$M_{a_i} := \left(y_{i,3} + \left(y_{i,0} + y_{i,2}\right) \cdot \frac{e}{2}\right) ft \qquad P_{s_i} := \left(y_{i,0} + y_{i,1} + y_{i,2}\right) \cdot ft$$

Iterate Service Deflection

$$\Delta_{s_i} := 0 \cdot in$$

Initial Guess

$$\Delta_s := \Delta f_s\left(M_a, P_s, \Delta_s, M_{cr}, \Delta_{cr}, M_n, \Delta_n\right)$$

Function iterates service deflections per ACI 318-19 *Sec 11.8.4*

Concrete Panel Out-of-Plane Analysis

$flag_i := \text{if}\left(\frac{\Delta_{s_i}}{\Delta_{max}} \le 1, \text{"OK"}, \text{"Revise"}\right)$

$deflection\_check := flag$

$\frac{\Delta_s}{\Delta_{max}} = \begin{bmatrix} 0.06 \\ 0.11 \\ 0.32 \\ 0.11 \\ 0.31 \end{bmatrix}$ $flag = \begin{bmatrix} \text{"OK"} \\ \text{"OK"} \\ \text{"OK"} \\ \text{"OK"} \\ \text{"OK"} \end{bmatrix}$

Results Summary

Panel Thickness $t = 5\ in$ **Bar Size** $rebar_{bar\_size} = \text{"\#5"}$ **Bar Spacing** $SP = 9\ in$

Bar Location $layer = \text{"CL"}$

Solid Panel with Step 9" THK w/ #5 @ 6" OC VERT EF

Panel Geometry

$t := 9\ in$ Panel Thickness

$ht := 38\ ft + 6\ in$ Roof Height (Panel Span)

$h_p := 43.5\ ft - ht = 5\ ft$ Parapet Height

$DockHigh := \text{No}$ Dock High Span Reduction

$w := 18\ ft$ Panel Width

$e := \frac{t}{2} + 3 \cdot in = 7.5\ in$ Roof Load Eccentricity

$h_T := ht + h_p = 43.5\ ft$ Total Panel Height

Panel Load Factors

$L := 1\ ft$ Panel Width Considered (PWC) for girder/pad ftg

$x_g := 1$ Gravity Factor

$x_{OOP} := 1.0$ OOP Factor

Panel Wind Loading Per ASCE 7-16 Chapter 30 - Part 1 Method

$GCpi := 0.18$ Internal Pressure Coefficient per Table 26.13-1

$A_{eff} := \frac{1}{3}\ ht^2 = 494.1\ ft^2$

$Slope := Y$

If roof slope is <10 degrees, GCp can be reduced by 10%

$$GCp_{poscalc} := \left(1.0 - \frac{\ln\left(\frac{A_{eff}}{ft^2}\right) - \ln(10)}{\ln(500) - \ln(10)}(1-0.7)\right)(1 - Slope)$$

Positive pressure coefficient (zones 4 and 5)

$$GCp_{negcalc} := \left(1.4 - \frac{\ln\left(\frac{A_{eff}}{ft^2}\right) - \ln(10)}{\ln(500) - \ln(10)}(1.4-0.8)\right)(1 - Slope)(-1)$$

Negative pressure coefficient (zone 5)

$$GCp_{pos} := \begin{cases} 0.7\,(1 - Slope) & \text{if } A_{eff} > 500\ ft^2 \\ 1.0\,(1 - Slope) & \text{else if } A_{eff} < 10\ ft^2 \\ GCp_{poscalc} & \text{else} \end{cases}$$

Check against upper and lower bound values for CGp based on Aeff

$$GCp_{neg} := \begin{cases} -0.8\,(1 - Slope) & \text{if } A_{eff} > 500\ ft^2 \\ -1.4\,(1 - Slope) & \text{else if } A_{eff} < 10\ ft^2 \\ GCp_{negcalc} & \text{else} \end{cases}$$

$$p_1 := q_h \cdot (GCp_{pos} - GCpi)$$

$$p_2 := q_h \cdot (GCp_{neg} - GCpi)$$

$$p_3 := q_h \cdot (GCp_{pos} + GCpi)$$

$$p_4 := q_h \cdot (GCp_{neg} + GCpi)$$

$$p_{net} := \max(|p_1|, |p_2|, |p_3|, |p_4|) = 13.7\ psf$$

Panel Loading

$w_{rf} := 60\ ft + 0\ in$

Joist Span/Spacing Adjacent to Panel

Concrete Panel Out-of-Plane Analysis

$load_{drift} := 71\ psf$ Drift Load at Panel

$w_{drift} := 17\ ft + 0\ in$ Drift Length

$$P_{drift} := \begin{cases} \text{if } w_{rf} \ge w_{drift} \\ \quad P \leftarrow load_{drift} \cdot w_{drift} \cdot \left(\dfrac{1}{2} - \dfrac{w_{drift}}{6\ w_{rf}}\right) \\ \text{else if } w_{rf} < w_{drift} \\ \quad P \leftarrow load_{drift} \cdot w_{rf} \cdot \left(\dfrac{1}{2} - \dfrac{w_{rf}}{6 \cdot w_{drift}}\right) \end{cases}$$

Snow Drift Reaction at Panel $P_{drift} = 0.55\ \dfrac{k}{ft}$

$P_{dlr} := x_g \cdot DL \cdot \dfrac{w_{rf}}{2}$ **Roof Dead Load + Mezzanine Dead Load** $P_{dlr} = 0.4\ \dfrac{k}{ft}$

$P_{dlp} := x_g \cdot (t + 5\ in) \cdot 150\ pcf \cdot \left(\dfrac{ht}{2} + h_p\right)$ **Panel Dead Load** $P_{dlp} = 4.2\ \dfrac{k}{ft}$

$P_{sl} := x_g \cdot \left(SL \cdot \dfrac{w_{rf}}{2} + P_{drift}\right)$ **Total Snow Load** $P_{sl} = 1\ \dfrac{k}{ft}$

$P_{ll\_mezz} := 0\ plf$ **Mezzanine Live Load** $P_{ll\_mezz} = 0\ \dfrac{k}{ft}$

$M_w := x_{OOP} \cdot \dfrac{p_{net} \cdot ht^2}{8}$ **Moment Due To Wind** $M_w = 2.5\ \dfrac{k \cdot ft}{ft}$

$M_s := x_{OOP} \cdot \dfrac{F_p \cdot (t + 5\ in) \cdot 150\ pcf \cdot ht^2}{8} + \dfrac{2.6\ k \cdot ht}{4\ w}$ **Moment Due to Seismic** $M_s = 12.3\ \dfrac{k \cdot ft}{ft}$

Strength Load Combinations (LRFD)

(ASCE 7-16 2.3)

$$x := \begin{bmatrix} 1.2 \cdot P_{dlr} & 1.2 \cdot P_{dlp} & 1.6 \cdot P_{sl} & 0.5 \cdot M_w \\ 1.2 \cdot P_{dlr} & 1.2 \cdot P_{dlp} & 0.5 \cdot (P_{sl}) + P_{ll\_mezz} & 1.0 \cdot M_w \\ 1.2 \cdot P_{dlr} & 1.2 \cdot P_{dlp} & 0.2 \cdot P_{sl} + 0.5 \cdot P_{ll\_mezz} & 1.0 \cdot M_s \\ 0.9 \cdot P_{dlr} & 0.9 \cdot P_{dlp} & 0 \cdot \dfrac{k}{ft} & 1.0 \cdot M_w \end{bmatrix}$$

Service/Deflection Load Combinations (ASD)

(2021 IBC 1605.2)

$\omega := 1.0$

$$y := \begin{bmatrix} 1.0 \cdot P_{dlr} & 1.0\ P_{dlp} & P_{sl} + P_{ll\_mezz} & 0.5 \cdot (\omega \cdot 0.6 \cdot M_w) \\ 1.0 \cdot P_{dlr} & 1.0 \cdot P_{dlp} & 0.5 \cdot P_{sl} + P_{ll\_mezz} & 1.0 \cdot (\omega \cdot 0.6 \cdot M_w) \\ 1.0 \cdot P_{dlr} & 1.0 \cdot P_{dlp} & P_{sl} + P_{ll\_mezz} & \left(\dfrac{1}{1.4}\right) \cdot M_s \end{bmatrix}$$

$$\begin{bmatrix} 0.9 \cdot P_{dlr} & 0.9 \cdot P_{dlp} & 0 \cdot \frac{k}{ft} & 1.0 \cdot M_s \end{bmatrix} \qquad \begin{bmatrix} 0.67 \cdot P_{dlr} & 0.67 \cdot P_{dlp} & 0 \cdot \frac{k}{ft} & 1.0 \cdot (\omega \cdot 0.6 \cdot M_w) \\ 0.9 \cdot P_{dlr} & 0.9 \cdot P_{dlp} & 0 \cdot \frac{k}{ft} & \left(\frac{1}{1.4}\right) \cdot M_s \end{bmatrix}$$

$i := 0 .. \mathrm{rows}(x) - 1$

Define Panel Reinforcing

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

$SP := 6 \cdot in$ — Vertical Bar Spacing

$rebar\_location := \mathrm{EF}$ — Steel Layer: Each Face (EF) or Centerline (CL)

$stirrup\_size := \mathrm{NA}$ — Stirrup Bar Size

$A_b := A_{bar\_size}$ — Vertical Bar Area

$diam_b := D_{bar\_size}$ — Vertical Bar Diameter

$diam_{str} := D_{stirrup\_size}$ — Stirrup Bar Diameter

$d_{ef} := t - \frac{diam_b}{2} - diam_{str} - clr - r_v$ — Steel Depth If Each Face

$d_{cl} := \frac{t}{2} - r_v$ — Steel Depth If Centerline

$d := \mathrm{if}\left(rebar\_location = 0, d_{ef}, d_{cl}\right)$ — Steel Depth — $d = 7.19\ in$

$A_s := \frac{A_b}{SP}$ — Vertical Area of Steel

Check Axial Capacity

(ACI 318-19 11.8.1.1(d))

$$P_{u_i} := \left\| \begin{array}{l} P_u \leftarrow 0 \\ \left(x_{i,0} + x_{i,1} + x_{i,2}\right) \end{array} \right.$$

$P_a := 0.06 \cdot f_c \cdot t$

$$flag_i := \mathrm{if}\left(\frac{P_{u_i}}{P_a} \le 1, \text{“OK”}, \text{“Revise”}\right)$$

$$\frac{P_u}{P_a} = \begin{bmatrix} 0.3 \\ 0.2 \\ 0.2 \\ 0.2 \\ 0.2 \end{bmatrix} \qquad flag = \begin{bmatrix} \text{“OK”} \\ \text{“OK”} \\ \text{“OK”} \\ \text{“OK”} \\ \text{“OK”} \end{bmatrix}$$

$compression\_check := flag$

Determine Moment Capacity

$$\beta_1 := 0.85 - 0.05 \cdot \left(\frac{f_c - 4000 \cdot psi}{1000 \cdot psi}\right)$$

$P_u := P_u \cdot ft \qquad A_s := A_s \cdot ft$

$P_u + A_s \cdot f_y$

$$X_i := \frac{\quad}{0.85 \cdot f_c \cdot ft} \qquad M_{n_i} := \left(A_s \cdot f_y + P_{u_i}\right) \cdot \left(d - \frac{i}{2}\right) - P_{u_i} \cdot \left(d - \frac{t}{2}\right)$$

$\phi M_{n_i} := 0.9 \cdot M_{n_i}$ Factored Moment Capacity $\phi M_n = \begin{bmatrix} 20.7 \\ 20.4 \\ 20.3 \\ 19.9 \\ 19.9 \end{bmatrix} k \cdot ft$

Check for Tension-Controlled Section (ACI 318-19 11.8.1.1(b))

$$C_i := \frac{X_i}{\beta_1} \qquad flag_i := \text{if}\left(\frac{C_i}{d} < 0.375, \text{"OK"}, \text{"Revise"}\right)$$

$tension\_controlled := flag$

$\frac{C}{d} = \begin{bmatrix} 0.178 \\ 0.174 \\ 0.173 \\ 0.166 \\ 0.166 \end{bmatrix}$ $flag = \begin{bmatrix} \text{"OK"} \\ \text{"OK"} \\ \text{"OK"} \\ \text{"OK"} \\ \text{"OK"} \end{bmatrix}$

Determine Deflection Limits

$\Delta_{max} := \frac{ht}{150}$ Allowable Panel Deflection

$ht' := \text{if}\left(DockHigh = 0, ht, 0.8 \cdot ht\right) = 38.5\ ft$ Adjust Span per Dock High Condition

$$E_c := 57000 \cdot \sqrt{\frac{f_c}{psi}} \cdot psi \qquad n := \frac{29000 \cdot ksi}{E_c} \qquad A_{se_i} := A_s + \frac{P_{u_i} \cdot t}{2 \cdot f_y \cdot d} \qquad C_i := \frac{X_i}{0.85}$$

$$f_r := 7.5 \cdot \sqrt{\frac{f_c}{psi}} \cdot psi \qquad I_g := \frac{(1\ ft) \cdot t^3}{12} \qquad I_{cr_i} := \frac{12 \cdot in \cdot \left(C_i\right)^3}{3} + n \cdot A_{se_i} \cdot \left(d - C_i\right)^2$$

$$M_{cr} := \frac{2 \cdot f_r \cdot I_g}{t} \qquad \Delta_{cr} := \frac{5 \cdot M_{cr} \cdot ht'^2}{48 \cdot E_c \cdot I_g} \qquad \Delta_{n_i} := \frac{5 \cdot M_{n_i} \cdot ht'^2}{48 \cdot E_c \cdot I_{cr_i}}$$

Check Moment Capacity (ACI 318-19 11.8.3.1)

$$M_{ua_i} := \left(x_{i,3} + \left(x_{i,0} + x_{i,2}\right) \cdot \frac{e}{2}\right) \cdot ft \qquad M_{u_i} := \frac{M_{ua_i}}{1 - \frac{5 \cdot P_{u_i} \cdot ht'^2}{0.75 \cdot 48 \cdot E_c \cdot I_{cr_i}}}$$

$$flag_i := \text{if}\left(\frac{M_{u_i}}{\phi M_{n_i}} \le 1, \text{"OK"}, \text{"Revise"}\right)$$

$capacity\_check := flag$

$\frac{M_u}{\phi M_n} = \begin{bmatrix} 0.13 \\ 0.19 \\ 0.81 \\ 0.16 \\ 0.76 \end{bmatrix}$ $flag = \begin{bmatrix} \text{"OK"} \\ \text{"OK"} \\ \text{"OK"} \\ \text{"OK"} \\ \text{"OK"} \end{bmatrix}$

Check Deflection

(ACI 318-19 11.8.4)

$$M_{a_i} := \left(y_{i,3} + \left(y_{i,0} + y_{i,2}\right) \cdot \frac{e}{2}\right) ft \qquad P_{s_i} := \left(y_{i,0} + y_{i,1} + y_{i,2}\right) \cdot ft$$

Iterate Service Deflection

$\Delta_{s_i} := 0 \cdot in$ Initial Guess

$\Delta_s := \Delta f_s\left(M_a, P_s, \Delta_s, M_{cr}, \Delta_{cr}, M_n, \Delta_n\right)$ Function iterates service deflections per ACI 318-19 *Sec 11.8.4*

$$flag_i := \text{if}\left(\frac{\Delta_{s_i}}{\Delta_{max}} \le 1, \text{"OK"}, \text{"Revise"}\right)$$

$$\frac{\Delta_s}{\Delta_{max}} = \begin{bmatrix} 0.04 \\ 0.06 \\ 0.93 \\ 0.05 \\ 0.87 \end{bmatrix} \qquad flag = \begin{bmatrix} \text{"OK"} \\ \text{"OK"} \\ \text{"OK"} \\ \text{"OK"} \\ \text{"OK"} \end{bmatrix}$$

$deflection\_check := flag$

Results Summary

Panel Thickness $t = 9\ in$ **Bar Size** $rebar_{bar\_size} = \text{"\#5"}$ **Bar Spacing** $SP = 6\ in$

Bar Location $layer = \text{"EF"}$

$$w_{OOP\_panel} := x_{OOP} \cdot F_p \cdot t \cdot 150\ pcf \cdot \frac{ht}{2} = 0.28\ \frac{k}{ft}$$

$$V_{OOP} := w_{OOP\_panel} \cdot \frac{w}{2} = 2.6\ k$$

$$M_{OOP} := w_{OOP\_panel} \cdot \frac{w^2}{8} = 12.2\ k \cdot ft$$

Loads from Blade Wall OOP design

Company:		Date:	6/9/2023
Engineer:		Page:	1/6
Project:			
Address:			
Phone:			
E-mail:			

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 318-14
Units: Imperial units

Anchor Information:
Anchor type: Cast-in-place
Material: AWS Type A
Diameter (inch): 0.500
Effective Embedment depth, h_{ef} (inch): 3.000
Anchor category: -
Anchor ductility: Yes
h_{min} (inch): 4.25
C_{min} (inch): 1.25
S_{min} (inch): 2.00

Base Material
Concrete: Normal-weight
Concrete thickness, h (inch): 12.00
State: Cracked
Compressive strength, f'_c (psi): 4000
$\Psi_{c,V}$: 1.0
Reinforcement condition: B tension, B shear
Supplemental reinforcement: Not applicable
Reinforcement provided at corners: No
Ignore concrete breakout in tension: No
Ignore concrete breakout in shear: No
Ignore 6do requirement: No
Build-up grout pad: No

Base Plate
Length x Width x Thickness (inch): 10.00 x 10.00 x 0.25

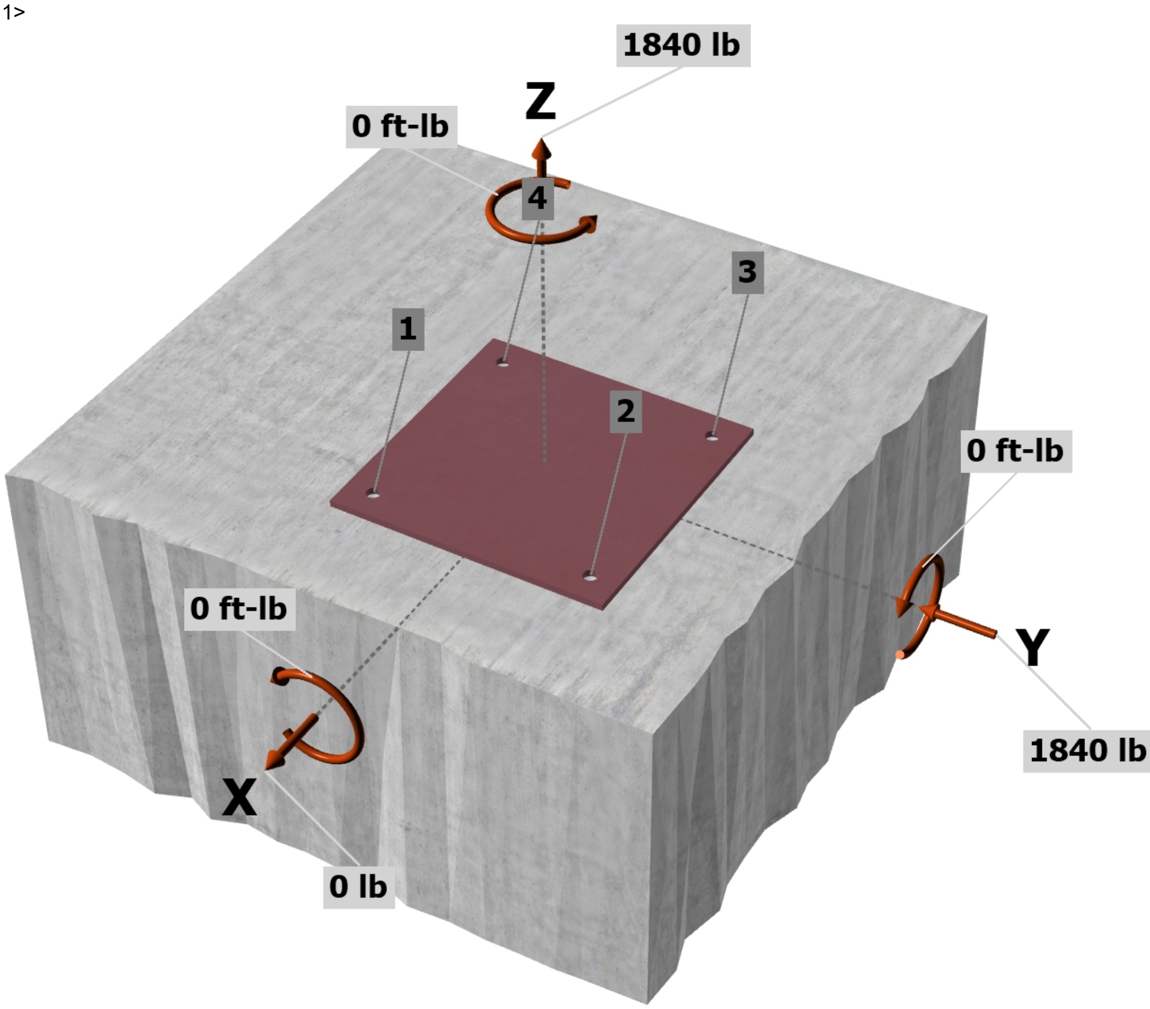
Recommended Anchor
Anchor Name: Headed Stud - 1/2"Ø AWS Type A Headed Stud

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: 17.2.3.4.2 not applicable
Ductility section for shear: 17.2.3.5.2 not applicable
Ω_0 factor: not set
Apply entire shear load at front row: No
Anchors only resisting wind and/or seismic loads: No

Strength level loads:

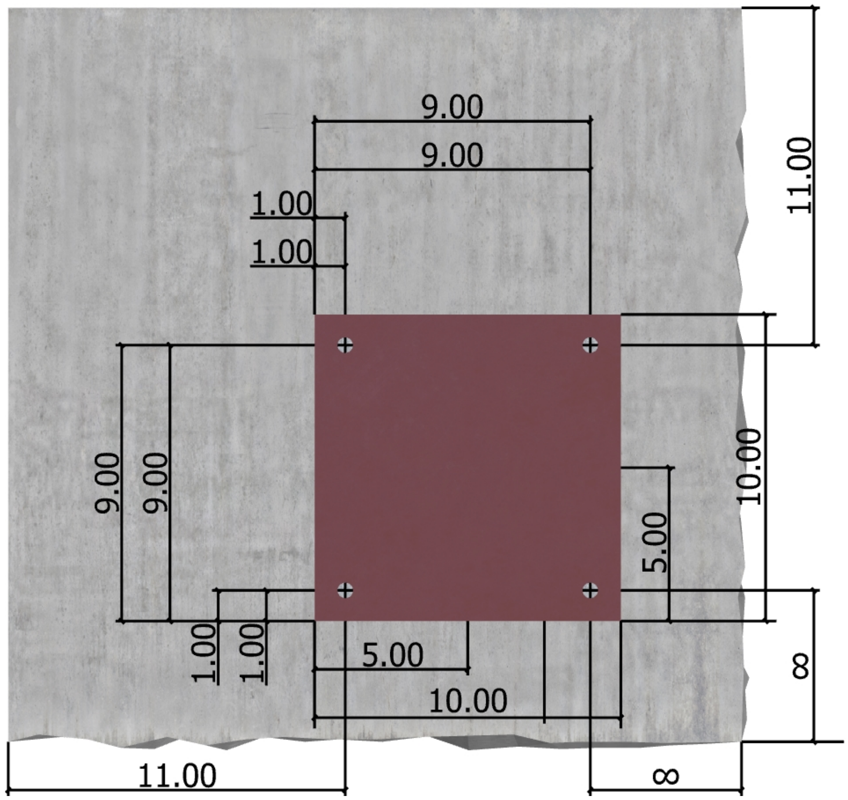
N_{ua} [lb]: 1840
V_{uax} [lb]: 0
V_{uay} [lb]: -1840
M_{ux} [ft-lb]: 0
M_{uy} [ft-lb]: 0
M_{uz} [ft-lb]: 0

<Figure 1>



Company:		Date:	6/9/2023
Engineer:		Page:	3/6
Project:			
Address:			
Phone:			
E-mail:			

<Figure 2>

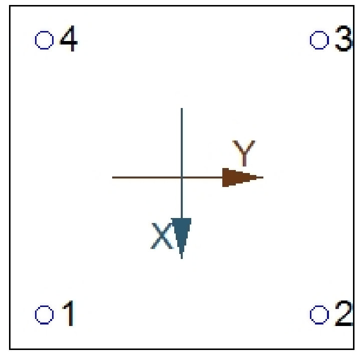


3. Resulting Anchor Forces

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	460.0	0.0	-460.0	460.0
2	460.0	0.0	-460.0	460.0
3	460.0	0.0	-460.0	460.0
4	460.0	0.0	-460.0	460.0
Sum	1840.0	0.0	-1840.0	1840.0

Maximum concrete compression strain (‰): 0.00
Maximum concrete compression stress (psi): 0
Resultant tension force (lb): 1840
Resultant compression force (lb): 0
Eccentricity of resultant tension forces in x-axis, e'_{Nx} (inch): 0.00
Eccentricity of resultant tension forces in y-axis, e'_{Ny} (inch): 0.00
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. 17.4.1)

N_{sa} (lb)	ϕ	ϕN_{sa} (lb)
11975	0.75	8981

5. Concrete Breakout Strength of Anchor in Tension (Sec. 17.4.2)

$N_b = k_c \lambda_a \sqrt{f'_c} h_{ef}^{1.5}$ (Eq. 17.4.2.2a)

k_c	λ_a	f'_c (psi)	h_{ef} (in)	N_b (lb)
24.0	1.00	4000	3.000	7887

$0.75\phi N_{cbg} = 0.75\phi\,(A_{Nc}/A_{Nco})\,\Psi_{ec,N}\Psi_{ed,N}\Psi_{c,N}\Psi_{cp,N}N_b$ (Sec. 17.3.1 & Eq. 17.4.2.1b)

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)
289.00	81.00	11.00	1.000	1.000	1.00	1.000	7887	0.70	14774

6. Pullout Strength of Anchor in Tension (Sec. 17.4.3)

$0.75\phi N_{pn} = 0.75\phi\Psi_{c,P}N_p = 0.75\phi\Psi_{c,P}8A_{brg}f'_c$ (Sec. 17.3.1, Eq. 17.4.3.1 & 17.4.3.4)

$\Psi_{c,P}$	A_{brg} (in^2)	f'_c (psi)	ϕ	$0.75\phi N_{pn}$ (lb)
1.0	0.59	4000	0.70	9895

8. Steel Strength of Anchor in Shear (Sec. 17.5.1)

V_{sa} (lb)	ϕ_{grout}	ϕ	$\phi_{grout}\phi V_{sa}$ (lb)
11975	1.0	0.65	7784

9. Concrete Breakout Strength of Anchor in Shear (Sec. 17.5.2)

Shear perpendicular to edge in y-direction:

$V_{by} = \min|7(l_e / d_a)^{0.2}\sqrt{d_a}\lambda_a\sqrt{f'_c}c_{a1}^{1.5}; 9\lambda_a\sqrt{f'_c}c_{a1}^{1.5}|$ (Eq. 17.5.2.2a & Eq. 17.5.2.2b)

l_e (in)	d_a (in)	λ_a	f'_c (psi)	c_{a1} (in)	V_{by} (lb)
3.00	0.500	1.00	4000	19.00	37100

$\phi V_{cbgy} = \phi\ (A_{Vc} / A_{Vco})\,\Psi_{ec,V}\,\Psi_{ed,V}\,\Psi_{c,V}\,\Psi_{h,V}V_{by}$ (Sec. 17.3.1 & Eq. 17.5.2.1b)

A_{Vc} (in^2)	A_{Vco} (in^2)	$\Psi_{ec,V}$	$\Psi_{ed,V}$	$\Psi_{c,V}$	$\Psi_{h,V}$	V_{by} (lb)	ϕ	ϕV_{cbgy} (lb)
570.00	1624.50	1.000	0.816	1.000	1.541	37100	0.70	11456

Shear parallel to edge in x-direction:

$V_{by} = \min|7(l_e / d_a)^{0.2}\sqrt{d_a}\lambda_a\sqrt{f'_c}c_{a1}^{1.5}; 9\lambda_a\sqrt{f'_c}c_{a1}^{1.5}|$ (Eq. 17.5.2.2a & Eq. 17.5.2.2b)

l_e (in)	d_a (in)	λ_a	f'_c (psi)	c_{a1} (in)	V_{by} (lb)
3.00	0.500	1.00	4000	11.00	16343

$\phi V_{cbgx} = \phi\ (2)(A_{Vc} / A_{Vco})\,\Psi_{ec,V}\,\Psi_{ed,V}\,\Psi_{c,V}\,\Psi_{h,V}V_{by}$ (Sec. 17.3.1, 17.5.2.1(c) & Eq. 17.5.2.1b)

A_{Vc} (in^2)	A_{Vco} (in^2)	$\Psi_{ec,V}$	$\Psi_{ed,V}$	$\Psi_{c,V}$	$\Psi_{h,V}$	V_{by} (lb)	ϕ	ϕV_{cbgx} (lb)
426.00	544.50	1.000	1.000	1.000	1.173	16343	0.70	20991

10. Concrete Pryout Strength of Anchor in Shear (Sec. 17.5.3)

$\phi V_{cpg} = \phi k_{cp}N_{cbg} = \phi k_{cp}(A_{Nc} / A_{Nco})\,\Psi_{ec,N}\,\Psi_{ed,N}\,\Psi_{c,N}\,\Psi_{cp,N}N_b$ (Sec. 17.3.1 & Eq. 17.5.3.1b)

k_{cp}	A_{Nc} (in^2)	A_{Nco} (in^2)	$\Psi_{ec,N}$	$\Psi_{ed,N}$	$\Psi_{c,N}$	$\Psi_{cp,N}$	N_b (lb)	ϕ	ϕV_{cpg} (lb)
2.0	289.00	81.00	1.000	1.000	1.000	1.000	7887	0.70	39397

11. Results

Interaction of Tensile and Shear Forces (Sec. 17.6.)

Tension	Factored Load, N_{ua} (lb)	Design Strength, øN_n (lb)	Ratio	Status
Steel	460	8981	0.05	Pass
Concrete breakout	**1840**	**14774**	**0.12**	**Pass (Governs)**
Pullout	460	9895	0.05	Pass

Shear	Factored Load, V_{ua} (lb)	Design Strength, øV_n (lb)	Ratio	Status
Steel	460	7784	0.06	Pass
T Concrete breakout y-	**1840**	**11456**	**0.16**	**Pass (Governs)**
|| Concrete breakout x-	**920**	**20991**	**0.04**	**Pass (Governs)**
Pryout	1840	39397	0.05	Pass

Interaction check	$N_{ua}/\phi N_n$	$V_{ua}/\phi V_n$	Combined Ratio	Permissible	Status
Sec. 17.6..2	0.00	0.16	16.1%	1.0	Pass

1/2"Ø AWS Type A Headed Stud with hef = 3.000 inch meets the selected design criteria.

12. Warnings

- Per designer input, the tensile component of the strength-level earthquake force applied to anchors does not exceed 20 percent of the total factored anchor tensile force associated with the same load combination. Therefore the ductility requirements of ACI 318 17.2.3.4.2 for tension need not be satisfied – designer to verify.

- Per designer input, the shear component of the strength-level earthquake force applied to anchors does not exceed 20 percent of the total factored anchor shear force associated with the same load combination. Therefore the ductility requirements of ACI 318 17.2.3.5.2 for shear need not be satisfied – designer to verify.

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

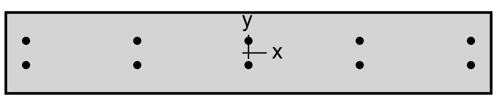
$$w_{OOP\_panel} := x_{OOP} \cdot F_p \cdot t \cdot 150\ pcf \cdot \frac{ht}{2} = 0.28\ \frac{k}{ft}$$

$$V_{OOP} := w_{OOP\_panel} \cdot \frac{w}{2} = 2.6\ k$$

$$M_{OOP} := w_{OOP\_panel} \cdot \frac{w^2}{8} = 12.2\ k \cdot ft$$

Loads from Balde Wall OOP design

spColumn v10.00 (TM)
Computer program for the Strength Design of Reinforced Concrete Sections



Contents

List of Figures

1. General Information

File Name	H:\Projects\222029000\Pro...\Horizontal Beam.colx
Project	---
Column	---
Engineer	---
Code	ACI 318-19
Bar Set	ASTM A615
Units	English
Run Option	Investigation
Run Axis	X - axis
Slenderness	Not Considered
Column Type	Structural
Capacity Method	Moment capacity

2. Material Properties

2.1. Concrete

Type	Standard	
f'_c	4	ksi
E_c	3605	ksi
f_c	3.4	ksi
ε_u	0.003	in/in
β_1	0.85	

2.2. Steel

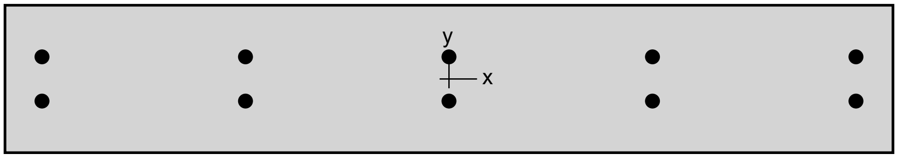
Type	Standard	
f_y	60	ksi
E_s	29000	ksi
ε_{ty}	0.00206897	in/in

3. Section

3.1. Shape and Properties

Type	Rectangular	
Width	30	in
Depth	5	in
A_g	150	in^2
I_x	312.5	in^4
I_y	11250	in^4
r_x	1.44338	in
r_y	8.66025	in
X_o	0	in
Y_o	0	in

3.2. Section Figure



Rectangular 30 x 5 in   1.33% reinf.

Figure 1: Column section

4. Reinforcement

4.1. Bar Set: ASTM A615

Bar	Diameter	Area	Bar	Diameter	Area	Bar	Diameter	Area
	in	in^2		in	in^2		in	in^2
#3	0.38	0.11	#4	0.50	0.20	#5	0.63	0.31
#6	0.75	0.44	#7	0.88	0.60	#8	1.00	0.79
#9	1.13	1.00	#10	1.27	1.27	#11	1.41	1.56
#14	1.69	2.25	#18	2.26	4.00			

4.2. Confinement and Factors

Confinement type	Other
For #10 bars or less	#3 ties
For larger bars	#4 ties
Capacity Reduction Factors	
Axial compression, (a)	0.8
Tension controlled ϕ, (b)	0.9
Compression controlled ϕ, (c)	0.65

4.3. Arrangement

Pattern	Sides different
Bar layout	Rectangular
Cover to	Longitudal bars
Clear cover	---
Bars	---
Total steel area, A_s	2.00 in^2
Rho	1.33 %
Minimum clear spacing	1.00 in

4.4. Bars Provided

		Bars	Clear cover
			in
Top	5	#4	1.5
Bottom	5	#4	1.5
Left	0	#6	1
Right	0	#6	1

5. Factored Loads and Moments with Corresponding Capacity Ratios

NOTE: Calculations are based on "Moment Capacity" Method.

No.	Demand		Capacity		Parameters at Capacity			Capacity
	P_u	M_{ux}	ϕP_n	ϕM_{nx}	NA Depth	ε_t	ϕ	Ratio
	kip	k-ft	kip	k-ft	in			
1	0.00	12.20	0.00	16.32	1.18	0.00527	0.900	0.75

BLADE WALL HSS

$P_E = 2.6^k$

$\omega_{DL} = (15\,psf)(5\,ft) = 75\,plf$

$V_{DL} = \dfrac{(75\,plf + 16\,plf)(20\,ft)}{2} = 910\,lbs$

$M_{DL} = \dfrac{(91\,plf)(20\,ft)^2}{8} = 4.55\,k\text{-}ft$

C&C

$A_{EFF} = (20\,ft)\left(\dfrac{20\,ft}{3}\right) = 133\,ft^2$

$\omega_w = (18.7\,psf + 15.3\,psf)(5\,ft) = 170\,plf$

$M_w = \dfrac{(170\,plf)(20\,ft)^2}{8} = 8.5\,k\text{-}ft$

$\phi P_n = 62.8k$

$\phi M_n = 28.5\,k\text{-}ft$

$\dfrac{2.6^k}{62.8^k} + \left(\dfrac{1.2(4.55\,k\text{-}ft)}{28.5\,k\text{-}ft}\right) + \left(\dfrac{8.5\,k\text{-}ft}{28.5\,k\text{-}ft}\right) = 0.31 < 1.0$

OKAY

$\dfrac{5(0.091\,klf)(20\,ft)^4(12)^3}{384(29000\,ksi)(28.6\,in^4)} = 0.395\,in \Rightarrow$ OKAY

By
Date
Job#
Sht. of

Shear Wall Design

Protected Area

Purpose Statement: The purpose of this calculation is in-plane design of concrete tilt panels as special reinforced shear walls.

Referenced Standards:
IBC 2021
ASCE 7-16
ACI 318-19
Amrhein/Yellow Book method
NEHRP Tech Brief 6

Inputs
Verify
Outputs

PROJECT PARAMETERS

$f_c := 4000 \cdot psi$	Concrete Strength
$q_{all} := 2500 \cdot psf$	Allowable Soil Pressure
$\gamma_{temp} := \frac{4}{3}$	Short-term Loading Increase for Soil
$q_{sei} := \gamma_{temp} \cdot q_{all}$	Short-term Allowable Soil Pressure
$DL_{sei} := 10.9 \cdot psf$	Roof DL
$DL_{flr} := 24 \cdot psf$	Intermediate Floor DL
$t_{SOG} := 7 \cdot in$	Slab-on-Grade Thickness
$\phi_{vE} := 0.6$	Seismic shear strength reduction factor - ACI 318-19 21.2.4.1 (assumes shear-controlled design)
$\phi_f := 0.9$	Flexural Strength Reduction Factor
$\phi_t := 0.9$	Tensile Strength Reduction Factor
$EL_{asd\_factor} := 0.7$	ASD Factor
$f_y := 60 \cdot ksi$	Reinforcing Yield Strength
$E_s := 29000 \cdot ksi$	Steel Modulus of Elasticity
$w_c := 150 \cdot pcf$	Unit Weight of Concrete
$\lambda :=$ Normal Weight	Concrete weight modification factor - ACI 318-19 Table 19.2.4.1 (b)

Shear Wall Design

$S_{ds} := 0.843$ Seismic Coefficient

$\rho := 1.0$ Redundancy Factor - ASCE 7-16 12.3.4

$I_e := 1.0$ Seismic Importance Factor - ASCE 7-16 Table 1.5-2

Design Coefficients and Factors for Special Reinforced Concrete Shear Walls - ASCE 7-16 12.2

$R := 5$ Response Modification Factor

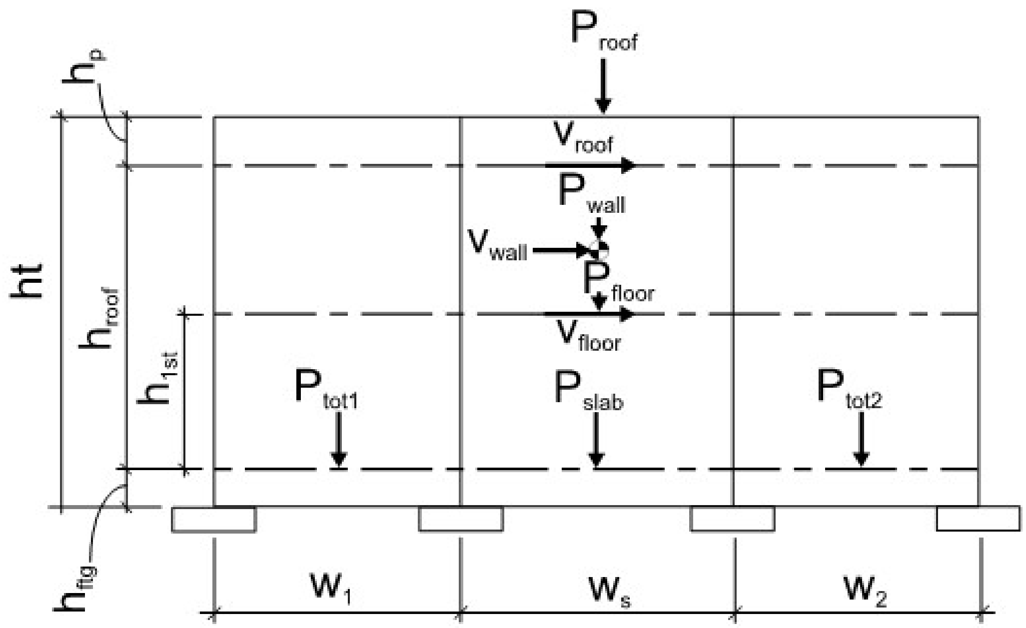
$C_d := 5$ Deflection Amplification Factor

$\Omega_o := 2.0$ Special Concrete Shear Wall Overstrength Factor

$C_s := \frac{S_{ds}}{\left(\frac{R}{I_e}\right)} = 0.1686$ Seismic Coefficient (C_s) - ASCE 7-16 12.8.1

ACI 318-19 Solid Panel - Panel 26 - Grid 12

In-Plane Shear Solid Panel:



Shear Panel (w_s)

$w_s := 26.5\ ft$	Panel Width
$h_p := 1.75 \cdot ft$	Parapet Height
$h_{roof} := 38.75\ ft$	Height of Roof Above FF
$h_{1st} := 0 \cdot ft$	Height of Intermediate Floor Above FF
$h_{ftg} := 1.5 \cdot ft$	FF to T/FTG Height
$ht := h_p + h_{roof} + h_{ftg} = 42\ ft$	Total Panel Height (T/FTG to T/PARAPET)
$t_p := 8.75 \cdot in$	Panel Thickness
$r_v := 0.75 \cdot in$	Reveal Depth
$t := t_p - r_v = 8\ in$	Effective Panel Thickness
$Wt_p := w_c \cdot t_p$	Panel Weight (psf)

Shear Wall Design

Adjacent Panel (w_1)

$w_1 := w_s$

$h_{p1} := h_p$

$h_{roof1} := h_{roof}$

$h_{ftg1} := h_{ftg}$

$ht_1 := h_{p1} + h_{roof1} + h_{ftg1} = 42\ ft$

$t_{p1} := t_p$

$Ao_1 := 0 \cdot ft \cdot 0 \cdot ft$

Adjacent Panel (w_2)

$w_2 := w_s$ — Adjacent Panel Width

$h_{p2} := h_p$ — Parapet Height

$h_{roof2} := h_{roof}$ — Height of Roof Above FF

$h_{ftg2} := h_{ftg}$ — FF to T/FTG Height

$ht_2 := h_{p2} + h_{roof2} + h_{ftg2} = 42\ ft$ — Total Panel Ht. (T/FTG to T/PARAPET)

$t_{p2} := t_p$ — Panel Thickness

$Ao_2 := 0 \cdot ft \cdot 0 \cdot ft$ — Area of Openings

Loading:

Seismic (1.0 Q_e)

$V_{roof} := 79.1\ k$ — Seismic Shear @ Roof- ρ not included

$V_{floor} := 0 \cdot k$ — Seismic Shear @ Floor

$V_{wall} := \left(ht - h_{ftg}\right) \cdot w_s \cdot Wt_p \cdot C_s$ — Seismic Shear due to Wall Panel Mass

$V_{tot} := V_{roof} + V_{wall} + V_{floor} = 98.9\ k$ — Total Seismic Shear

$M_{ot\_roof} := V_{roof} \cdot h_{roof}$ — Overturning Moment due to Roof Shear

$M_{ot\_floor} := V_{floor} \cdot h_{1st}$ — Overturning Moment due to Floor Shear

$M_{ot\_wall} := V_{wall} \cdot \dfrac{\left(ht - h_{ftg}\right)}{2}$ — Overturning Moment due to Panel Mass

$M_{ot} := M_{ot\_roof} + M_{ot\_floor} + M_{ot\_wall} = 3466\ k \cdot ft$ — Total Overturning Moment @ Wall Panel

Gravity:

Shear Panel (w_s):

$l_{roof\_trib} := \dfrac{10}{2} \cdot ft$ — Length of Roof Tributary to Panel

$l_{floor\_trib} := \dfrac{0}{2} \cdot ft$ — Length of Floor Tributary to Panel

Shear Wall Design

$SL := 0.421\ klf$ Roof snow load tributary to panel

$LL := 0.0\ klf$ Floor live load tributary to panel

$P_{roof} := DL_{sei} \cdot l_{roof\_trib} \cdot w_s$ Roof DL

$P_{floor} := DL_{flr} \cdot l_{floor\_trib} \cdot w_s$ Floor DL

$P_{wall} := w_s \cdot ht \cdot t_p \cdot w_c$ Wall Panel DL

$P_{slab} := w_c \cdot t_{SOG} \cdot 8 \cdot ft \cdot w_s$ Slab-on-Grade DL (assumes 8' width of slab over width of panel available to resist OT)

$P_{tot} := P_{roof} + P_{floor} + P_{wall} = 123.2\ k$ Total DL (without slab @ wall considered)

Panel (w_1):

$P_{roof1} := P_{roof} \cdot \left(\frac{w_1}{w_s}\right)$ Roof DL

$P_{floor1} := P_{floor} \cdot \left(\frac{w_1}{w_s}\right)$ Floor DL

$P_{wall1} := \left(w_1 \cdot ht_1 - Ao_1\right) \cdot t_{p1} \cdot w_c$ Wall Panel DL

$P_{slab1} := P_{slab} \cdot \left(\frac{w_1}{w_s}\right)$ Slab-on-Grade DL

$P_{tot1} := P_{roof1} + P_{floor1} + P_{wall1} + P_{slab1} = 141.7\ k$ Total DL (including slab weight)

Panel (w_2):

$P_{roof2} := P_{roof} \cdot \left(\frac{w_2}{w_s}\right)$ Roof DL

$P_{floor2} := P_{floor} \cdot \frac{w_2}{w_s}$ Floor DL

$P_{wall2} := \left(w_2 \cdot ht_2 - Ao_2\right) \cdot t_{p2} \cdot w_c$ Wall Panel DL

$P_{slab2} := P_{slab} \cdot \left(\frac{w_2}{w_s}\right)$ Slab-on-Grade DL

$P_{tot2} := P_{roof2} + P_{floor2} + P_{wall2} + P_{slab2} = 141.7\ k$ Total DL (including slab weight)

Resisting Moment:

$M_{resist} := \left(P_{tot} + P_{slab}\right) \cdot \frac{w_s}{2} = 1878\ k \cdot ft$ Resisting Moment at Shear Panel

Shear Wall Design

Load Combinations:

ASCE 7-16 2.3.6

LRFD 6: (1.2+0.2Sds)D+1.0ρ Q_e + L + 0.2S

LRFD 7: (0.9 - 0.2Sds)D + 1.0ρ Q_e (Resisting Moment of Slab-on-Grade)

IBC 2021 1605.2 - Foundations

ASD 16-5: D + L + S + E/1.4 (Foundation Bearing)

ASD 16-6: 0.9D + E/1.4 (Foundation Uplift/Weight. Ev = 0 for fdn size, Ev included for hold downs)

Panel Hold Down - IBC 2021 1605.2 Alternative Load Combination ASD 16-6

$M_{ot\_IBC} := \rho \cdot \dfrac{M_{ot}}{1.4}$ ASD Overturning Moment

$M_{res\_IBC} := \left(0.9 - \dfrac{0.2 \cdot S_{ds}}{1.4}\right) \cdot M_{resist}$ ASD Resisting Moment

$No_{hd} :=$ #6 Rebar Size No.

$N_{hd} := 4$ Number of Bars in Hold Down

$L_{hd} := \text{if}\left(No_{hd_0} = 0.31\ in^2, L_{HD5_{N_{hd}}}, L_{HD6_{N_{hd}}}\right) = 24\ in$

$A_{shd} := No_{hd_0} \cdot N_{hd}$ Area of Steel in Hold Down

$hd_{check} := M_{ot\_IBC} - M_{res\_IBC} = 1011.68\ k \cdot ft$ Holdown check

$T_{hd} := \dfrac{M_{ot} - (0.9 - 0.2\ S_{ds}) \cdot M_{resist}}{w_s - (1 \cdot ft + 0.5 \cdot L_{hd})} = 85.4\ k$ LRFD Tension at Hold Down

$A_{shdreq} := \dfrac{T_{hd}}{\phi_t \cdot f_y} = 1.58\ in^2$ Required Area of Steel

$DCR_{hd} := \dfrac{A_{shdreq}}{A_{shd}}$ $DCR_{hd} = 0.9$

$hd\_Results =$ “USE (4) #6 BARS AT HOLDOWNS”

Pad Footing Uplift and Seismic Bearing Checks

$l_f := 13.25\ ft$ — Length of Footing

$w_f := 6\ ft$ — Width of Footing

$d_f := 1.5\ ft$ — Depth of Footing (Use 2'-0" MIN @ hold down, see footing calcs)

$P_{ftg} := w_c \cdot l_f \cdot w_f \cdot d_f$ — Footing Weight

Required Footing Weight - Load Combination ASD 16-6

$$M_{res\_uplift} := M_{res\_IBC} + \left(0.9 \cdot \left(min\left(\frac{P_{tot1}}{2}, \frac{P_{tot2}}{2}\right) \cdot w_s\right)\right)$$

$M_{res\_uplift} = 3154\ k \cdot ft$ — Factored Resisting Moment (Including contribution of slab & adjacent panel)

$$Wt_{ftg\_uplift} := \frac{\left(M_{ot\_IBC} - M_{res\_uplift}\right)}{w_s - \left(1\ ft + 0.5 \cdot L_{hd}\right)}$$

$Wt_{ftg\_uplift} = -27.7\ k$ — Required Footing Weight

$flag_w := \text{if}\left(Wt_{ftg\_uplift} \le P_{ftg}, \text{"OK"}, \text{"NG!!!"}\right)$ $\qquad flag_w = \text{"OK"}$

Wall Shear Design

Factored Wall Axial Forces for M_{pr} Calculation

$P_{u1} := \left(1.2 + 0.2 \cdot S_{ds}\right) \cdot P_{tot} + \left(0.2\ SL + LL\right)\ w_s = 171\ \boldsymbol{k}$ ASCE 7-16 12.2.3.6 Eq. 6

$P_{u2} := \left(0.9 - 0.2\ S_{ds}\right)\ P_{tot} = 90.09\ \boldsymbol{k}$ ASCE 7-16 12.2.3.6 Eq. 7

Probable Moment and Moment Demand

$M_{pr} := 18661\ \boldsymbol{k} \cdot \boldsymbol{ft}$ Largest probable flexural strength of the section from SP Column assuming tensile stress of 1.25*Fy and Φ = 1.0

$M_u := \rho \cdot M_{ot}$

$V_u := \rho \cdot V_{tot} = 98.9\ \boldsymbol{k}$ Ultimate Panel Shear (ρ Q_e)

Pier Check - ACI 318-19 Table R18.10.1

If wall section is a pier, requirements of ACI 318-19 18.10.8 or 18.14 must be met.

$$Check_1 := \text{if}\left(\frac{w_s}{t_p} \le 6, \text{“Pier”}, \text{“Segment”}\right)$$

$$PierCheck := \text{if}\left(\frac{h_{roof}}{w_s} \ge 2, Check_1, \text{“Segment”}\right)$$

$PierCheck = \text{“Segment”}$

Design Shear Force - ACI 318-19 18.10.3

$h_{wcs} := h_{roof} + h_p$ Height of Wall above Critical Section

$stories_{wcs} := 1$ Number of Stories above Critical Section

$$n_s := \max\left(stories_{wcs}, 0.007 \cdot \frac{h_{wcs}}{\boldsymbol{in}}\right)$$

$$\omega_v := \begin{array}{l} \text{if } \dfrac{h_{wcs}}{w_s} < 2 \\ \quad 1.0 \\ \text{else if } n_s \le 6.0 \\ \quad 0.9 + \dfrac{n_s}{10} \\ \text{else} \\ \quad min\left(1.3 + \dfrac{n_s}{30}, 1.8\right) \end{array} = 1$$

Dynamic shear amplification factor, ACI 318-19 18.10.3.1.3

$$M_{prratio} := \frac{M_{pr}}{M_u}$$

$$\Omega_v := \text{if}\left(\frac{h_{wcs}}{w_s} \le 1.5, 1.0, \max\left(M_{prratio}, 1.5\right)\right)$$

$$\Omega_v = 5.38$$

Shear Overstrength factor, ACI 318-19 18.10.3.1.2

$$V_{ratio} := min\left(\omega_v \cdot \Omega_v, 3\right) = 3$$

$$V_e := V_{ratio} \cdot V_u$$

$$V_{epiera} := \text{if}\left(\frac{w_s}{t} > 2.5, V_u \cdot \Omega_o, \text{“N/A”}\right)$$

Alternate Ve if pier aspect ratio met
ACI 318-19 18.10.8.1a

$$V_e := \text{if}\left(PierCheck = \text{“Pier”}, \text{if}\left(V_{epiera} = \text{“N/A”}, \frac{M_{pr}}{h_{roof}}, min\left(\frac{M_{pr}}{h_{roof}}, V_{epiera}\right)\right), V_{ratio} \cdot V_u\right)$$

$$V_e = 296.7\ k$$

Design Shear Force,
For Segments: ACI 318-19 18.10.3.1
For Piers: ACI 318-19 18.7.6.1

Shear Steel Requirements - ACI 318-19 18.10.2.1/11.6.1

$$A_{cv} := t \cdot w_s = 2544\ in^2$$

Gross Area of Concrete Section

$$V_{min} := \lambda \cdot \sqrt{\frac{f_c}{psi}} \cdot psi \cdot A_{cv}$$

Shear Strength to determine ρ_{min}

$$\rho_{min} := \text{if}\left(V_u < V_{min}, 0.002, 0.0025\right) = 0.002$$

$$A_{v\_min} := (t) \cdot (12 \cdot in) \cdot (\rho_{min})$$

Minimum area of Shear Steel per Foot

Determine number of curtains of shear steel - ACI 318-19 18.10.2.2

$$V_{reinf} := 2 \cdot \lambda \cdot \sqrt{\frac{f_c}{psi}} \cdot psi \cdot A_{cv}$$

Shear Strength to Determine Required
Number of Layers

$$Reinforcing := \text{if}\left(V_u > V_{reinf} \lor \frac{ht}{w_s} \ge 2, \text{“(2) CURTAINS REQD”}, \text{“(1) CURTAIN REQD”}\right)$$

$$Reinforcing = \text{“(1) CURTAIN REQD”}$$

Define Shear Steel:

$No_v := \#4$ — Shear Reinforcement Bar Size

$A_{sNov} := No_{v_0}$ $\quad d_{sv} := No_{v_1}$ — Area of Shear Reinforcement Bar

$S := 12 \cdot in$ — Shear Reinforcement Spacing (12" max, typical)

$N_L := 1$ — Number of Layers **(CL = 1, EF = 2)**

$$A_{vs} := N_L \cdot \frac{12 \cdot in}{S} \cdot A_{sNov}$$ Area of Shear Reinforcement per Foot

$$\rho_t := \frac{A_{vs}}{(12 \cdot in) \cdot (t)}$$

$A_{v\_min\_check} := \text{if}\left(A_{v\_min} \le A_{vs}, \text{"OK"}, \text{"REVISE!"}\right)$ $\quad A_{v\_min\_check} = \text{"OK"}$

$Check := \text{if}\left(V_u > V_{reinf} \wedge N_L = 1, \text{"REVISE REINF"}, \text{"OK"}\right)$ $\quad Check = \text{"OK"}$

Shear Wall Strength - ACI 318-19 Eq. 18.10.4.1

$$\alpha_c := \begin{array}{l} \text{if } \frac{ht}{w_s} \le 1.5 \\ \quad 3.0 \\ \text{else if } \frac{ht}{w_s} \ge 2.0 \\ \quad 2.0 \\ \text{else} \\ \quad 2 + \frac{2.0 - \frac{ht}{w_s}}{0.5} \end{array} = 2.83$$

$$\phi V_n := \phi_{vE} \cdot \left(A_{cv} \cdot \left(\alpha_c \cdot \lambda \cdot \left(\sqrt{\frac{f_c}{psi}} \cdot psi\right) + \rho_t \cdot f_y\right)\right)$$ ACI 318-19 Eq. 18.10.4.1

Define Shear Strength Max - ACI 318-19 18.10.4.4

$$\phi V_{nmax} := \phi_{vE} \cdot \left(8 \cdot \sqrt{\frac{f_c}{psi}} \cdot psi \cdot A_{cv}\right)$$ ACI 318-19 Eq. 18.10.4.4

$check := \text{if}\left(V_e \le \phi V_{nmax}, \text{"Proceed with Design"}, \text{"Revise!"}\right)$ $\quad check = \text{"Proceed with Design"}$

$check := \text{if}\left(\phi V_n \le \phi V_{nmax}, \text{“OK”}, \text{“Vn MAX exceeded, reduce reinforcement”}\right)$ $check = \text{“OK”}$

$DCR_v := \dfrac{V_e}{min\left(\phi V_n, \phi V_{nmax}\right)}$ $DCR_v = 0.64$

$Flag_v = \text{“USE (1) LAYER /S OF \#4 BARS AT 12''OC HORIZ MIN”}$

Flexural Steel - ACI 318-19 18.10.6

$No_f := \#5$ Longitudinal Reinforcement Bar Size

$A_{sNof} := No_{f_0}$ $d_{Nof} := No_{f_1}$

$N_{Lf} := 2$ Number of Layers of Longitudinal Reinforcement (CL = 1, EF = 2) at Each End

$N_f := 26$ Number of Longitudinal Reinforcement Bars per Layer at Each End

$d_{solve} := 0.8 \cdot w_s = 21.2\ ft$ Depth of Lever Arm

Solve Constraints Guess Values

$A_{sreq} := 1\ in^2$

$\phi_f \cdot A_{sreq} \cdot f_y \cdot \left(d_{solve} - \dfrac{A_{sreq} \cdot f_y}{0.85 \cdot f_c \cdot t \cdot 2}\right) - M_u = 0$

$A_{sreq\_flex} := \text{Find}\left(\mathbf{A_{sreq}}\right)$

$A_{sreq\_flex} = 3.07\ in^2$ Required Area of Flexural Steel

$A_{s\_long} := N_f \cdot N_{Lf} \cdot A_{sNof} = 16.12\ in^2$ Area of Flexural Steel

$DCR_f := \dfrac{A_{sreq\_flex}}{A_{s\_long}}$ $DCR_f = 0.19$

$Flag_f = \text{“USE (52) \#5 BARS EACH END MIN”}$

Transverse Reinforcement Requirements - ACI 318-19 18.10.6.5

$$A_{s\_check} := \frac{400}{60000} \cdot \left(0.2 \cdot w_s\right) \cdot t = 3.392\ in^2$$

Assumes 0.2*Panel Width for N.A. depth. Input leg into SP Column if req'd to refine results.

$$V_{check} := \lambda \cdot \sqrt{\frac{f_c}{psi}} \cdot psi \cdot A_{cv} = 160.9\ k$$

Check shear against 18.10.6.5a

$$Check_{18.10.6.5a} := \text{if}\left(V_u > V_{check}, \text{“PROVIDE HOOKED HORIZ BARS OR HOOPED ENDS”}, \text{“OK”}\right)$$

$$Check_{18.10.6.5a} = \text{“OK”}$$

$$Check_{18.10.6.5b} := \text{if}\left(A_{sreq\_flex} > A_{s\_check}, \text{“PROVIDE CONFINING HOOPS @ 5'' O.C. MAX per 18.10.6.5”}, \text{“OK”}\right)$$

$$Check_{18.10.6.5b} = \text{“OK”}$$

Basic Boundary Element Check w/ Gross Section Properties - ACI 318-19 18.10.6.3

$$I_g := \frac{\left(w_s\right)^3 \cdot t}{12} \qquad A_g := \frac{w_s \cdot t}{2} \qquad S_g := \frac{t \cdot {w_s}^2}{6}$$

$$P_{u1} := \frac{P_{u1}}{2}$$

Factored Axial Load - ASCE 7-16 2.3.6
Load Combo 6

$$\sigma_u := \frac{M_u}{S_g} + \frac{P_{u1}}{A_g}$$

Combined Stress @ Extreme Fiber

$$F_c := 0.2 \cdot f_c$$

$$DCR_\sigma := \frac{\sigma_u}{F_c} \qquad DCR_\sigma = 0.47$$

$$BasicCheck := \text{if}\left(DCR_\sigma > 1, \text{“Perform Alternate Analysis”}, \text{“Okay”}\right) \qquad BasicCheck = \text{“Okay”}$$

If Wall does not meet basic boundary element check, provide boundary elements or perform Alternate check per ACI 318-19 18.10.6.2 ONLY if hwcs/ w_s ≥ 2**.**

$$AlternateCheck := \begin{array}{l} \text{if } \dfrac{h_{wcs}}{w_s} \ge 2 \wedge BasicCheck = \text{“Perform Alternate Analysis”} \\ \quad \text{“Proceed”} \\ \text{also if } BasicCheck = \text{“Okay”} \\ \quad \text{“Not Applicable”} \\ \text{else} \\ \quad \text{“Boundary Element Necessary – Alternate Check Not Allowed”} \end{array}$$

$AlternateCheck = \text{“Not Applicable”}$

Verify Section Uncracked

$y_t := \dfrac{w_s}{2}$ — Neutral Axis of Gross Section

$M_{cr} := \dfrac{7.5 \cdot \lambda \cdot \sqrt{\dfrac{f_c}{psi}} \cdot psi \cdot I_g}{y_t}$ — Cracking Moment - ACI 318-19 19.2.3.1 Eq 19.2.3.1, 24.2.3.5 Eq 24.2.3.5

$M_a := M_u \cdot EL_{asd\_factor}$ — Service Moment

$M_{crCheck} := \text{if}\left(M_{cr} < M_a, \text{“Cracked; If necessary adjust relative stiffness and evaluate load”}, \text{“Uncracked”}\right)$

$M_{crCheck} = \text{“Uncracked”}$

Slab Shear Dowel Bars - Shear Friction per ACI 318-19 22.9.4

$\mu := 0.6$ — Coefficient of Friction - ACI 318-19 22.9.4.2

$No_{dowel} := \#4$ — Dowel Reinforcement Bar Size No.

$A_{s\_Nodowel} := No_{dowel}$

$S_{dowel} := 12 \cdot in$ — Dowel Bar Spacing

$A_{dowel} := \dfrac{A_{s\_Nodowel}}{S_{dowel}}$ — Dowel Bar Area per Foot

$A_{vf} := A_{dowel} \cdot w_s = 5.3\ in^2$ — Area of Shear Friction Steel Across Panel

$\phi V_{vf\_n} := \phi_{vE} \cdot \left(\mu \cdot A_{vf} \cdot f_y\right)$ — Design Shear Capacity - ACI 318-19 22.9.4.2

$V_u := \rho \cdot V_{tot} = 98.9\ k$ — Ultimate Panel Shear ($\rho\, Q_e$)

$$DCR_{vf} := \frac{V_u}{\phi V_{vf\_n}}$$

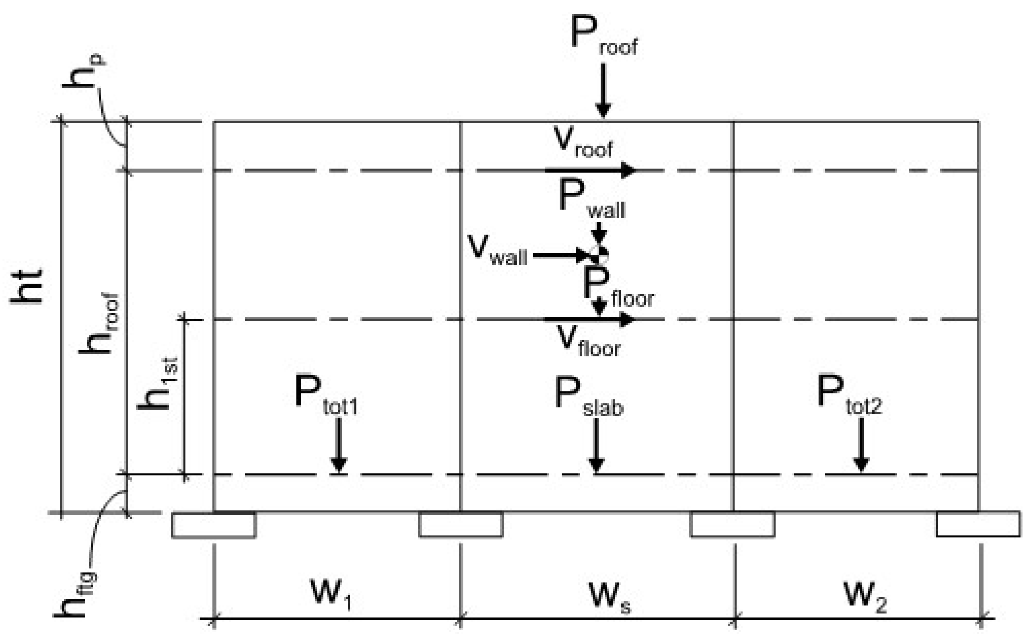
$$DCR_{vf} = 0.86$$

Shear Wall Design

ACI 318-19 Solid Panel - Panel 25 - Grid 12

Go

In-Plane Shear Solid Panel:



Shear Panel (w_s)

$w_s := 26.5\ ft$	Panel Width
$h_p := 4.5 \cdot ft$	Parapet Height
$h_{roof} := 39\ ft$	Height of Roof Above FF
$h_{1st} := 0 \cdot ft$	Height of Intermediate Floor Above FF
$h_{ftg} := 1.5 \cdot ft$	FF to T/FTG Height
$ht := h_p + h_{roof} + h_{ftg} = 45\ ft$	Total Panel Height (T/FTG to T/PARAPET)
$t_p := 8.75 \cdot in$	Panel Thickness
$r_v := 0.75 \cdot in$	Reveal Depth
$t := t_p - r_v = 8\ in$	Effective Panel Thickness
$Wt_p := w_c \cdot t_p$	Panel Weight (psf)

Shear Wall Design

Adjacent Panel (w_1)

$w_1 := w_s$

$h_{p1} := h_p$

$h_{roof1} := h_{roof}$

$h_{ftg1} := h_{ftg}$

$ht_1 := h_{p1} + h_{roof1} + h_{ftg1} = 45\ ft$

$t_{p1} := t_p$

$Ao_1 := 0 \cdot ft \cdot 0 \cdot ft$

Adjacent Panel (w_2)

$w_2 := w_s$ — Adjacent Panel Width

$h_{p2} := h_p$ — Parapet Height

$h_{roof2} := h_{roof}$ — Height of Roof Above FF

$h_{ftg2} := h_{ftg}$ — FF to T/FTG Height

$ht_2 := h_{p2} + h_{roof2} + h_{ftg2} = 45\ ft$ — Total Panel Ht. (T/FTG to T/PARAPET)

$t_{p2} := t_p$ — Panel Thickness

$Ao_2 := 0 \cdot ft \cdot 0 \cdot ft$ — Area of Openings

Loading:

Seismic (1.0 Q_e)

$V_{roof} := 78.1\ k$ — Seismic Shear @ Roof- ρ not included

$V_{floor} := 0 \cdot k$ — Seismic Shear @ Floor

$V_{wall} := \left(ht - h_{ftg}\right) \cdot w_s \cdot Wt_p \cdot C_s$ — Seismic Shear due to Wall Panel Mass

$V_{tot} := V_{roof} + V_{wall} + V_{floor} = 99.4\ k$ — Total Seismic Shear

$M_{ot\_roof} := V_{roof} \cdot h_{roof}$ — Overturning Moment due to Roof Shear

$M_{ot\_floor} := V_{floor} \cdot h_{1st}$ — Overturning Moment due to Floor Shear

$M_{ot\_wall} := V_{wall} \cdot \dfrac{\left(ht - h_{ftg}\right)}{2}$ — Overturning Moment due to Panel Mass

$M_{ot} := M_{ot\_roof} + M_{ot\_floor} + M_{ot\_wall} = 3508\ k \cdot ft$ — Total Overturning Moment @ Wall Panel

Gravity:

Shear Panel (w_s):

$l_{roof\_trib} := \dfrac{10}{2} \cdot ft$ — Length of Roof Tributary to Panel

$l_{floor\_trib} := \dfrac{0}{2} \cdot ft$ — Length of Floor Tributary to Panel

Shear Wall Design

$SL := 4.1\ klf$ — Roof snow load tributary to panel + girder from OOP calc)

$LL := 0.0\ klf$ — Floor live load tributary to panel

$$P_{roof} := DL_{sei} \cdot l_{roof\_trib} \cdot w_s + 23.2\ k \cdot \left(\frac{DL_{sei}}{14.9\ psf}\right) = 18.4\ k$$ Roof DL + girder load from OOP Calc

$$P_{floor} := DL_{flr} \cdot l_{floor\_trib} \cdot w_s$$ Floor DL

$$P_{wall} := w_s \cdot ht \cdot t_p \cdot w_c$$ Wall Panel DL

$$P_{slab} := w_c \cdot t_{SOG} \cdot 8 \cdot ft \cdot w_s$$ Slab-on-Grade DL (assumes 8' width of slab over width of panel available to resist OT)

$$P_{tot} := P_{roof} + P_{floor} + P_{wall} = 148.8\ k$$ Total DL (without slab @ wall considered)

Panel (w_1):

$$P_{roof1} := P_{roof} \cdot \left(\frac{w_1}{w_s}\right)$$ Roof DL

$$P_{floor1} := P_{floor} \cdot \left(\frac{w_1}{w_s}\right)$$ Floor DL

$$P_{wall1} := \left(w_1 \cdot ht_1 - Ao_1\right) \cdot t_{p1} \cdot w_c$$ Wall Panel DL

$$P_{slab1} := P_{slab} \cdot \left(\frac{w_1}{w_s}\right)$$ Slab-on-Grade DL

$$P_{tot1} := P_{roof1} + P_{floor1} + P_{wall1} + P_{slab1} = 167.4\ k$$ Total DL (including slab weight)

Panel (w_2):

$$P_{roof2} := P_{roof} \cdot \left(\frac{w_2}{w_s}\right)$$ Roof DL

$$P_{floor2} := P_{floor} \cdot \frac{w_2}{w_s}$$ Floor DL

$$P_{wall2} := \left(w_2 \cdot ht_2 - Ao_2\right) \cdot t_{p2} \cdot w_c$$ Wall Panel DL

$$P_{slab2} := P_{slab} \cdot \left(\frac{w_2}{w_s}\right)$$ Slab-on-Grade DL

$$P_{tot2} := P_{roof2} + P_{floor2} + P_{wall2} + P_{slab2} = 167.4\ k$$ Total DL (including slab weight)

Resisting Moment:

$$M_{resist} := \left(P_{tot} + P_{slab}\right) \cdot \frac{w_s}{2} = 2218\ k \cdot ft$$ Resisting Moment at Shear Panel

Load Combinations:

ASCE 7-16 2.3.6

LRFD 6: (1.2+0.2Sds)D+1.0ρ Q_e + L + 0.2S

LRFD 7: (0.9 - 0.2Sds)D + 1.0ρ Q_e (Resisting Moment of Slab-on-Grade)

IBC 2021 1605.2 - Foundations

ASD 16-5: D + L + S + E/1.4 (Foundation Bearing)

ASD 16-6: 0.9D + E/1.4 (Foundation Uplift/Weight. Ev = 0 for fdn size, Ev included for hold downs)

Panel Hold Down - IBC 2021 1605.2 Alternative Load Combination ASD 16-6

$$M_{ot\_IBC} := \rho \cdot \frac{M_{ot}}{1.4}$$ ASD Overturning Moment

$$M_{res\_IBC} := \left(0.9 - \frac{0.2 \cdot S_{ds}}{1.4}\right) \cdot M_{resist}$$ ASD Resisting Moment

$$No_{hd} := \#6$$ Rebar Size No.

$$N_{hd} := 4$$ Number of Bars in Hold Down

$$L_{hd} := \text{if}\left(No_{hd_0} = 0.31\ in^2, L_{HD5_{N_{hd}}}, L_{HD6_{N_{hd}}}\right) = 24\ in$$

$$A_{shd} := No_{hd_0} \cdot N_{hd}$$ Area of Steel in Hold Down

$$hd_{check} := M_{ot\_IBC} - M_{res\_IBC}$$ Holdown check

$$T_{hd} := \frac{M_{ot} - \left(0.9 - 0.2\ S_{ds}\right) \cdot M_{resist}}{w_s - \left(1 \cdot ft + 0.5 \cdot L_{hd}\right)} = 77\ k$$ LRFD Tension at Hold Down

$$A_{shdreq} := \frac{T_{hd}}{\phi_t \cdot f_y} = 1.43\ in^2$$ Required Area of Steel

$$DCR_{hd} := \frac{A_{shdreq}}{A_{shd}}$$ $$DCR_{hd} = 0.81$$

$$hd\_Results = \text{“USE (4) \#6 BARS AT HOLDOWNS”}$$

Pad Footing Uplift and Seismic Bearing Checks

$l_f := w_s = 26.5\ ft$ — Length of Footing

$w_f := 4.5\ ft$ — Width of Footing

$d_f := 1\ ft$ — Depth of Footing (Use 2'-0" MIN @ hold down, see footing calcs)

$P_{ftg} := w_c \cdot l_f \cdot w_f \cdot d_f$ — Footing Weight

Required Footing Weight - Load Combination ASD 16-6

$$M_{res\_uplift} := M_{res\_IBC} + \left(0.9 \cdot \left(min\left(\frac{P_{tot1}}{2}, \frac{P_{tot2}}{2}\right) \cdot w_s\right)\right)$$

$M_{res\_uplift} = 3725\ k \cdot ft$ — Factored Resisting Moment (Including contribution of slab & adjacent panel)

$$Wt_{ftg\_uplift} := \frac{\left(M_{ot\_IBC} - M_{res\_uplift}\right)}{w_s - \left(1\ ft + 0.5 \cdot L_{hd}\right)}$$

$Wt_{ftg\_uplift} = -49.8\ k$ — Required Footing Weight

$flag_w := \text{if}\left(Wt_{ftg\_uplift} \le P_{ftg}, \text{“OK”}, \text{“NG!!!”}\right)$ — $flag_w = \text{“OK”}$

Wall Shear Design

Factored Wall Axial Forces for M_{pr} Calculation

$P_{u1} := (1.2 + 0.2 \cdot S_{ds}) \cdot P_{tot} + (0.2\ SL + LL)\ w_s = 225\ k$ ASCE 7-16 12.2.3.6 Eq. 6

$P_{u2} := (0.9 - 0.2\ S_{ds})\ P_{tot} = 108.87\ k$ ASCE 7-16 12.2.3.6 Eq. 7

Probable Moment and Moment Demand

$M_{pr} := 18661\ k \cdot ft$ Largest probable flexural strength of the section from SP Column assuming tensile stress of 1.25*Fy and Φ = 1.0

$M_u := \rho \cdot M_{ot}$

$V_u := \rho \cdot V_{tot} = 99.4\ k$ Ultimate Panel Shear ($\rho\, Q_e$)

Pier Check - ACI 318-19 Table R18.10.1

If wall section is a pier, requirements of ACI 318-19 18.10.8 or 18.14 must be met.

$$Check_1 := \text{if}\left(\frac{w_s}{t_p} \le 6, \text{“Pier”}, \text{“Segment”}\right)$$

$$PierCheck := \text{if}\left(\frac{h_{roof}}{w_s} \ge 2, Check_1, \text{“Segment”}\right)$$

$PierCheck = \text{“Segment”}$

Design Shear Force - ACI 318-19 18.10.3

$h_{wcs} := h_{roof} + h_p$ Height of Wall above Critical Section

$stories_{wcs} := 1$ Number of Stories above Critical Section

$$n_s := \max\left(stories_{wcs}, 0.007 \cdot \frac{h_{wcs}}{in}\right)$$

$$\omega_v := \begin{array}{l} \text{if } \dfrac{h_{wcs}}{w_s} < 2 \\ \quad 1.0 \\ \text{else if } n_s \le 6.0 \\ \quad 0.9 + \dfrac{n_s}{10} \\ \text{else} \\ \quad min\left(1.3 + \dfrac{n_s}{30}, 1.8\right) \end{array} = 1$$

Dynamic shear amplification factor, ACI 318-19 18.10.3.1.3

$$M_{prratio} := \frac{M_{pr}}{M_u}$$

$$\Omega_v := \text{if}\left(\frac{h_{wcs}}{w_s} \le 1.5, 1.0, \max\left(M_{prratio}, 1.5\right)\right)$$

$$\Omega_v = 5.32$$

Shear Overstrength factor, ACI 318-19 18.10.3.1.2

$$V_{ratio} := min\left(\omega_v \cdot \Omega_v, 3\right) = 3$$

$$V_e := V_{ratio} \cdot V_u$$

$$V_{epiera} := \text{if}\left(\frac{w_s}{t} > 2.5, V_u \cdot \Omega_o, \text{"N/A"}\right)$$

Alternate Ve if pier aspect ratio met
ACI 318-19 18.10.8.1a

$$V_e := \text{if}\left(PierCheck = \text{"Pier"}, \text{if}\left(V_{epiera} = \text{"N/A"}, \frac{M_{pr}}{h_{roof}}, min\left(\frac{M_{pr}}{h_{roof}}, V_{epiera}\right)\right), V_{ratio} \cdot V_u\right)$$

$$V_e = 298.1\ k$$

Design Shear Force,
For Segments: ACI 318-19 18.10.3.1
For Piers: ACI 318-19 18.7.6.1

Shear Steel Requirements - ACI 318-19 18.10.2.1/11.6.1

$$A_{cv} := t \cdot w_s = 2544\ in^2$$

Gross Area of Concrete Section

$$V_{min} := \lambda \cdot \sqrt{\frac{f_c}{psi}} \cdot psi \cdot A_{cv}$$

Shear Strength to determine ρ_{min}

$$\rho_{min} := \text{if}\left(V_u < V_{min}, 0.002, 0.0025\right) = 0.002$$

$$A_{v\_min} := (t) \cdot (12 \cdot in) \cdot \left(\rho_{min}\right)$$

Minimum area of Shear Steel per Foot

Determine number of curtains of shear steel - ACI 318-19 18.10.2.2

$$V_{reinf} := 2 \cdot \lambda \cdot \sqrt{\frac{f_c}{psi}} \cdot psi \cdot A_{cv}$$

Shear Strength to Determine Required Number of Layers

$$Reinforcing := \text{if}\left(V_u > V_{reinf} \vee \frac{ht}{w_s} \ge 2, \text{"(2) CURTAINS REQD"}, \text{"(1) CURTAIN REQD"}\right)$$

$$Reinforcing = \text{"(1) CURTAIN REQD"}$$

Define Shear Steel:

$No_v := \#4$ — Shear Reinforcement Bar Size

$A_{sNov} := No_{v_0}$ $\quad d_{sv} := No_{v_1}$ — Area of Shear Reinforcement Bar

$S := 12 \cdot in$ — Shear Reinforcement Spacing (12" max, typical)

$N_L := 1$ — Number of Layers **(CL = 1, EF = 2)**

$$A_{vs} := N_L \cdot \frac{12 \cdot in}{S} \cdot A_{sNov}$$

Area of Shear Reinforcement per Foot

$$\rho_t := \frac{A_{vs}}{(12 \cdot in) \cdot (t)}$$

$$A_{v\_min\_check} := \text{if}\left(A_{v\_min} \le A_{vs}, \text{“OK”}, \text{“REVISE!”}\right)$$

$A_{v\_min\_check} = \text{“OK”}$

$$Check := \text{if}\left(V_u > V_{reinf} \wedge N_L = 1, \text{“REVISE REINF”}, \text{“OK”}\right)$$

$Check = \text{“OK”}$

Shear Wall Strength - ACI 318-19 Eq. 18.10.4.1

$$\alpha_c := \begin{cases} 3.0 & \text{if } \dfrac{ht}{w_s} \le 1.5 \\ 2.0 & \text{else if } \dfrac{ht}{w_s} \ge 2.0 \\ 2 + \dfrac{2.0 - \dfrac{ht}{w_s}}{0.5} & \text{else} \end{cases} = 2.6$$

$$\phi V_n := \phi_{vE} \cdot \left(A_{cv} \cdot \left(\alpha_c \cdot \lambda \cdot \left(\sqrt{\frac{f_c}{psi}} \cdot psi\right) + \rho_t \cdot f_y\right)\right)$$

ACI 318-19 Eq. 18.10.4.1

Define Shear Strength Max - ACI 318-19 18.10.4.4

$$\phi V_{nmax} := \phi_{vE} \cdot \left(8 \cdot \sqrt{\frac{f_c}{psi}} \cdot psi \cdot A_{cv}\right)$$

ACI 318-19 Eq. 18.10.4.4

$$check := \text{if}\left(V_e \le \phi V_{nmax}, \text{“Proceed with Design”}, \text{“Revise!”}\right)$$

$check = \text{“Proceed with Design”}$

$check := \text{if}\left(\phi V_n \le \phi V_{nmax}, \text{“OK”}, \text{“Vn MAX exceeded, reduce reinforcement”}\right)$ $check = \text{“OK”}$

$DCR_v := \dfrac{V_e}{min\left(\phi V_n, \phi V_{nmax}\right)}$ $DCR_v = 0.67$

$Flag_v = \text{“USE (1) LAYER /S OF \#4 BARS AT 12''OC HORIZ MIN”}$

Flexural Steel - ACI 318-19 18.10.6

$No_f := \#5$ Longitudinal Reinforcement Bar Size

$A_{sNof} := No_{f_0}$ $d_{Nof} := No_{f_1}$

$N_{Lf} := 2$ Number of Layers of Longitudinal Reinforcement (CL = 1, EF = 2) at Each End

$N_f := 26$ Number of Longitudinal Reinforcement Bars per Layer at Each End

$d_{solve} := 0.8 \cdot w_s = 21.2\ ft$ Depth of Lever Arm

Solve Block — Guess Values / Constraints / Solver

$A_{sreq} := 1\ in^2$

$\phi_f \cdot A_{sreq} \cdot f_y \cdot \left(d_{solve} - \dfrac{A_{sreq} \cdot f_y}{0.85 \cdot f_c \cdot t \cdot 2}\right) - M_u = 0$

$A_{sreq\_flex} := \text{Find}\left(A_{sreq}\right)$

$A_{sreq\_flex} = 3.11\ in^2$ Required Area of Flexural Steel

$A_{s\_long} := N_f \cdot N_{Lf} \cdot A_{sNof} = 16.12\ in^2$ Area of Flexural Steel

$DCR_f := \dfrac{A_{sreq\_flex}}{A_{s\_long}}$ $DCR_f = 0.19$

$Flag_f = \text{“USE (52) \#5 BARS EACH END MIN”}$

Transverse Reinforcement Requirements - ACI 318-19 18.10.6.5

$$A_{s\_check} := \frac{400}{60000} \cdot (0.2 \cdot w_s) \cdot t = 3.392\ in^2$$

Assumes 0.2*Panel Width for N.A. depth. Input leg into SP Column if req'd to refine results.

$$V_{check} := \lambda \cdot \sqrt{\frac{f_c}{psi}} \cdot psi \cdot A_{cv} = 160.9\ k$$

Check shear against 18.10.6.5a

$$Check_{18.10.6.5a} := \text{if}\left(V_u > V_{check}, \text{“PROVIDE HOOKED HORIZ BARS OR HOOPED ENDS”}, \text{“OK”}\right)$$

$$Check_{18.10.6.5a} = \text{“OK”}$$

$$Check_{18.10.6.5b} := \text{if}\left(A_{sreq\_flex} > A_{s\_check}, \text{“PROVIDE CONFINING HOOPS @ 5'' O.C. MAX per 18.10.6.5”}, \text{“OK”}\right)$$

$$Check_{18.10.6.5b} = \text{“OK”}$$

Basic Boundary Element Check w/ Gross Section Properties - ACI 318-19 18.10.6.3

$$I_g := \frac{(w_s)^3 \cdot t}{12} \qquad A_g := \frac{w_s \cdot t}{2} \qquad S_g := \frac{t \cdot w_s^{\ 2}}{6}$$

$$P_{u1} := \frac{P_{u1}}{2}$$

Factored Axial Load - ASCE 7-16 2.3.6 Load Combo 6

$$\sigma_u := \frac{M_u}{S_g} + \frac{P_{u1}}{A_g}$$

Combined Stress @ Extreme Fiber

$$F_c := 0.2 \cdot f_c$$

$$DCR_\sigma := \frac{\sigma_u}{F_c} \qquad DCR_\sigma = 0.5$$

$$BasicCheck := \text{if}\left(DCR_\sigma > 1, \text{“Perform Alternate Analysis”}, \text{“Okay”}\right) \qquad BasicCheck = \text{“Okay”}$$

If Wall does not meet basic boundary element check, provide boundary elements or perform Alternate check per ACI 318-19 18.10.6.2 ONLY if hwcs/ w_s ≥ 2**.**

$$AlternateCheck := \begin{array}{l} \text{if } \frac{h_{wcs}}{w_s} \geq 2 \wedge BasicCheck = \text{“Perform Alternate Analysis”} \\ \quad \text{“Proceed”} \\ \text{also if } BasicCheck = \text{“Okay”} \\ \quad \text{“Not Applicable”} \\ \text{else} \\ \quad \text{“Boundary Element Necessary – Alternate Check Not Allowed”} \end{array}$$

$AlternateCheck = \text{“Not Applicable”}$

Verify Section Uncracked

$y_t := \frac{w_s}{2}$ — Neutral Axis of Gross Section

$M_{cr} := \frac{7.5 \cdot \lambda \cdot \sqrt{\frac{f_c}{psi}} \cdot psi \cdot I_g}{y_t}$ — Cracking Moment - ACI 318-19 19.2.3.1 Eq 19.2.3.1, 24.2.3.5 Eq 24.2.3.5

$M_a := M_u \cdot EL_{asd\_factor}$ — Service Moment

$M_{crCheck} := \text{if}\left(M_{cr} < M_a, \text{“Cracked; If necessary adjust relative stiffness and evaluate load”}, \text{“Uncracked”}\right)$

$M_{crCheck} = \text{“Uncracked”}$

Slab Shear Dowel Bars - Shear Friction per ACI 318-19 22.9.4

$\mu := 0.6$ — Coefficient of Friction - ACI 318-19 22.9.4.2

$No_{dowel} := \#4$ — Dowel Reinforcement Bar Size No.

$A_{s\_Nodowel} := No_{dowel}$

$S_{dowel} := 12 \cdot in$ — Dowel Bar Spacing

$A_{dowel} := \frac{A_{s\_Nodowel}}{S_{dowel}}$ — Dowel Bar Area per Foot

$A_{vf} := A_{dowel} \cdot w_s = 5.3\ in^2$ — Area of Shear Friction Steel Across Panel

$\phi V_{vf\_n} := \phi_{vE} \cdot \left(\mu \cdot A_{vf} \cdot f_y\right)$ — Design Shear Capacity - ACI 318-19 22.9.4.2

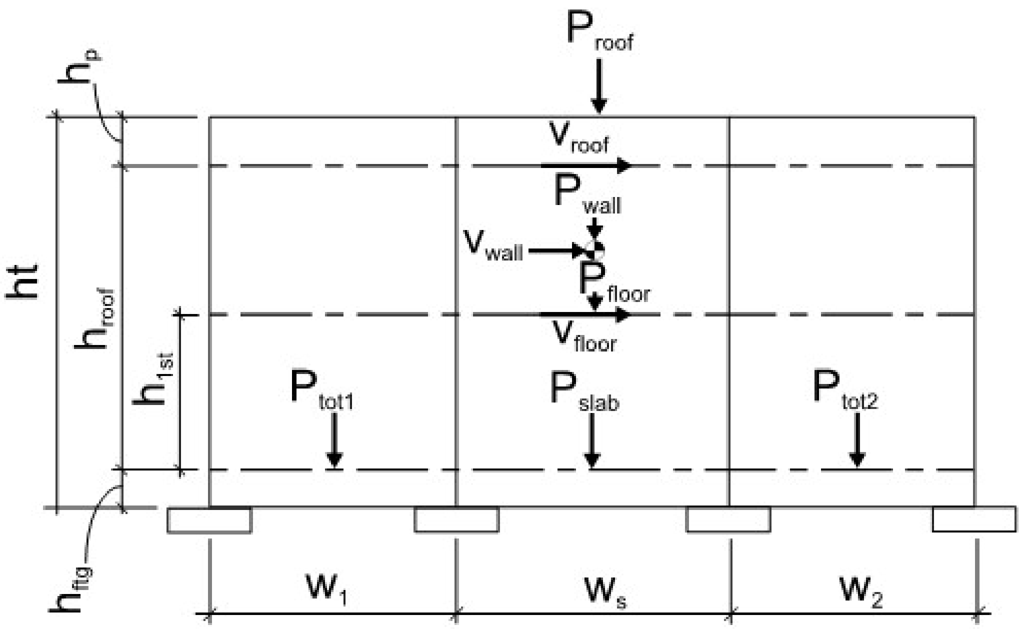
$V_u := \rho \cdot V_{tot} = 99.4\ k$ — Ultimate Panel Shear (ρQ_e)

$$DCR_{vf} := \frac{V_u}{\phi V_{vf\_n}}$$

$$DCR_{vf} = 0.87$$

ACI 318-19 Solid Panel - Panel 23 - Grid 12

In-Plane Shear Solid Panel:



Shear Panel (w_s)

$w_s := 20\ ft$	Panel Width
$h_p := 2.5 \cdot ft$	Parapet Height
$h_{roof} := 38\ ft$	Height of Roof Above FF
$h_{1st} := 0 \cdot ft$	Height of Intermediate Floor Above FF
$h_{ftg} := 1.5 \cdot ft$	FF to T/FTG Height
$ht := h_p + h_{roof} + h_{ftg} = 42\ ft$	Total Panel Height (T/FTG to T/PARAPET)
$t_p := 8.75 \cdot in$	Panel Thickness
$r_v := 0.75 \cdot in$	Reveal Depth
$t := t_p - r_v = 8\ in$	Effective Panel Thickness
$Wt_p := w_c \cdot t_p$	Panel Weight (psf)

Shear Wall Design

Adjacent Panel (w_1)	Adjacent Panel (w_2)	
$w_1 := w_s$	$w_2 := w_s$	Adjacent Panel Width
$h_{p1} := h_p$	$h_{p2} := h_p$	Parapet Height
$h_{roof1} := h_{roof}$	$h_{roof2} := h_{roof}$	Height of Roof Above FF
$h_{ftg1} := h_{ftg}$	$h_{ftg2} := h_{ftg}$	FF to T/FTG Height
$ht_1 := h_{p1} + h_{roof1} + h_{ftg1} = 42\ ft$	$ht_2 := h_{p2} + h_{roof2} + h_{ftg2} = 42\ ft$	Total Panel Ht. (T/FTG to T/PARAPET)
$t_{p1} := t_p$	$t_{p2} := t_p$	Panel Thickness
$Ao_1 := 0 \cdot ft \cdot 0 \cdot ft$	$Ao_2 := 0 \cdot ft \cdot 0 \cdot ft$	Area of Openings

Loading:

Seismic (1.0 Q_e)

$V_{roof} := 41.1\ k$ — Seismic Shear @ Roof- ρ not included

$V_{floor} := 0 \cdot k$ — Seismic Shear @ Floor

$V_{wall} := (ht - h_{ftg}) \cdot w_s \cdot Wt_p \cdot C_s$ — Seismic Shear due to Wall Panel Mass

$V_{tot} := V_{roof} + V_{wall} + V_{floor} = 56\ k$ — Total Seismic Shear

$M_{ot\_roof} := V_{roof} \cdot h_{roof}$ — Overturning Moment due to Roof Shear

$M_{ot\_floor} := V_{floor} \cdot h_{1st}$ — Overturning Moment due to Floor Shear

$M_{ot\_wall} := V_{wall} \cdot \frac{(ht - h_{ftg})}{2}$ — Overturning Moment due to Panel Mass

$M_{ot} := M_{ot\_roof} + M_{ot\_floor} + M_{ot\_wall} = 1864\ k \cdot ft$ — Total Overturning Moment @ Wall Panel

Gravity:

Shear Panel (w_s):

$l_{roof\_trib} := \frac{10}{2} \cdot ft$ — Length of Roof Tributary to Panel

$l_{floor\_trib} := \frac{0}{2} \cdot ft$ — Length of Floor Tributary to Panel

Shear Wall Design

$SL := 0.299\ klf$ — Roof snow load tributary to panel + girder from OOP calc)

$LL := 0.0\ klf$ — Floor live load tributary to panel

$P_{roof} := DL_{sei} \cdot l_{roof\_trib} \cdot w_s$ — Roof DL + girder load from OOP Calc

$P_{floor} := DL_{flr} \cdot l_{floor\_trib} \cdot w_s$ — Floor DL

$P_{wall} := w_s \cdot ht \cdot t_p \cdot w_c$ — Wall Panel DL

$P_{slab} := w_c \cdot t_{SOG} \cdot 8 \cdot ft \cdot w_s$ — Slab-on-Grade DL (assumes 8' width of slab over width of panel available to resist OT)

$P_{tot} := P_{roof} + P_{floor} + P_{wall} = 93\ k$ — Total DL (without slab @ wall considered)

Panel (w_1):

$P_{roof1} := P_{roof} \cdot \left(\frac{w_1}{w_s}\right)$ — Roof DL

$P_{floor1} := P_{floor} \cdot \left(\frac{w_1}{w_s}\right)$ — Floor DL

$P_{wall1} := \left(w_1 \cdot ht_1 - Ao_1\right) \cdot t_{p1} \cdot w_c$ — Wall Panel DL

$P_{slab1} := P_{slab} \cdot \left(\frac{w_1}{w_s}\right)$ — Slab-on-Grade DL

$P_{tot1} := P_{roof1} + P_{floor1} + P_{wall1} + P_{slab1} = 107\ k$ — Total DL (including slab weight)

Panel (w_2):

$P_{roof2} := P_{roof} \cdot \left(\frac{w_2}{w_s}\right)$ — Roof DL

$P_{floor2} := P_{floor} \cdot \frac{w_2}{w_s}$ — Floor DL

$P_{wall2} := \left(w_2 \cdot ht_2 - Ao_2\right) \cdot t_{p2} \cdot w_c$ — Wall Panel DL

$P_{slab2} := P_{slab} \cdot \left(\frac{w_2}{w_s}\right)$ — Slab-on-Grade DL

$P_{tot2} := P_{roof2} + P_{floor2} + P_{wall2} + P_{slab2} = 107\ k$ — Total DL (including slab weight)

Resisting Moment:

$M_{resist} := \left(P_{tot} + P_{slab}\right) \cdot \frac{w_s}{2} = 1070\ k \cdot ft$ — Resisting Moment at Shear Panel

Load Combinations:

ASCE 7-16 2.3.6

LRFD 6: (1.2+0.2Sds)D+1.0ρ Q_e + L + 0.2S

LRFD 7: (0.9 - 0.2Sds)D + 1.0ρ Q_e (Resisting Moment of Slab-on-Grade)

IBC 2021 1605.2 - Foundations

ASD 16-5: D + L + S + E/1.4 (Foundation Bearing)

ASD 16-6: 0.9D + E/1.4 (Foundation Uplift/Weight. Ev = 0 for fdn size, Ev included for hold downs)

Panel Hold Down - IBC 2021 1605.2 Alternative Load Combination ASD 16-6

$$M_{ot\_IBC} := \rho \cdot \frac{M_{ot}}{1.4}$$ ASD Overturning Moment

$$M_{res\_IBC} := \left(0.9 - \frac{0.2 \cdot S_{ds}}{1.4}\right) \cdot M_{resist}$$ ASD Resisting Moment

$$No_{hd} := \#6$$ Rebar Size No.

$$N_{hd} := 3$$ Number of Bars in Hold Down

$$L_{hd} := \text{if}\left(No_{hd_0} = 0.31\ in^2, L_{HD5_{N_{hd}}}, L_{HD6_{N_{hd}}}\right) = 18\ in$$

$$A_{shd} := No_{hd_0} \cdot N_{hd}$$ Area of Steel in Hold Down

$$hd_{check} := M_{ot\_IBC} - M_{res\_IBC}$$ Holdown check

$$T_{hd} := \frac{M_{ot} - (0.9 - 0.2\ S_{ds}) \cdot M_{resist}}{w_s - (1 \cdot ft + 0.5 \cdot L_{hd})} = 59.3\ k$$ LRFD Tension at Hold Down

$$A_{shdreq} := \frac{T_{hd}}{\phi_t \cdot f_y} = 1.1\ in^2$$ Required Area of Steel

$$DCR_{hd} := \frac{A_{shdreq}}{A_{shd}}$$

$$DCR_{hd} = 0.83$$

$$hd\_Results = \text{“USE (3) \#6 BARS AT HOLDOWNS”}$$

Pad Footing Uplift and Seismic Bearing Checks

$l_f := w_s = 20\ ft$ Length of Footing

$w_f := 4.5\ ft$ Width of Footing

$d_f := 1\ ft$ Depth of Footing (Use 2'-0" MIN @ hold down, see footing calcs)

$P_{ftg} := w_c \cdot l_f \cdot w_f \cdot d_f$ Footing Weight

Required Footing Weight - Load Combination ASD 16-6

$$M_{res\_uplift} := M_{res\_IBC} + \left(0.9 \cdot \left(min\left(\frac{P_{tot1}}{2}, \frac{P_{tot2}}{2}\right) \cdot w_s\right)\right)$$

$M_{res\_uplift} = 1797\ k \cdot ft$ Factored Resisting Moment (Including contribution of slab & adjacent panel)

$$Wt_{ftg\_uplift} := \frac{\left(M_{ot\_IBC} - M_{res\_uplift}\right)}{w_s - \left(1\ ft + 0.5 \cdot L_{hd}\right)}$$

$Wt_{ftg\_uplift} = -25.5\ k$ Required Footing Weight

$flag_w := \text{if}\left(Wt_{ftg\_uplift} \le P_{ftg}, \text{“OK”}, \text{“NG!!!”}\right)$ $flag_w = \text{“OK”}$

Shear Wall Design

Wall Shear Design

Factored Wall Axial Forces for M_{pr} Calculation

$$P_{u1} := \left(1.2 + 0.2 \cdot S_{ds}\right) \cdot P_{tot} + \left(0.2\ SL + LL\right)\ w_s = 128\ \boldsymbol{k}$$ ASCE 7-16 12.2.3.6 Eq. 6

$$P_{u2} := \left(0.9 - 0.2\ S_{ds}\right)\ P_{tot} = 67.99\ \boldsymbol{k}$$ ASCE 7-16 12.2.3.6 Eq. 7

Probable Moment and Moment Demand

$$M_{pr} := 11256\ \boldsymbol{k} \cdot \boldsymbol{ft}$$ Largest probable flexural strength of the section from SP Column assuming tensile stress of 1.25*Fy and Φ = 1.0

$$M_u := \rho \cdot M_{ot}$$

$$V_u := \rho \cdot V_{tot} = 56\ \boldsymbol{k}$$ Ultimate Panel Shear (ρ Q_e)

Pier Check - ACI 318-19 Table R18.10.1

If wall section is a pier, requirements of ACI 318-19 18.10.8 or 18.14 must be met.

$$Check_1 := \text{if}\left(\frac{w_s}{t_p} \le 6, \text{“Pier”}, \text{“Segment”}\right)$$

$$PierCheck := \text{if}\left(\frac{h_{roof}}{w_s} \ge 2, Check_1, \text{“Segment”}\right)$$ $$PierCheck = \text{“Segment”}$$

Design Shear Force - ACI 318-19 18.10.3

$$h_{wcs} := h_{roof} + h_p$$ Height of Wall above Critical Section

$$stories_{wcs} := 1$$ Number of Stories above Critical Section

$$n_s := \max\left(stories_{wcs}, 0.007 \cdot \frac{h_{wcs}}{\boldsymbol{in}}\right)$$

$$\omega_v := \begin{array}{|l} \text{if } \dfrac{h_{wcs}}{w_s} < 2 \\ \quad \| \ 1.0 \\ \text{else if } n_s \le 6.0 \\ \quad \| \ 0.9 + \dfrac{n_s}{10} \\ \text{else} \\ \quad \| \ min\left(1.3 + \dfrac{n_s}{30}, 1.8\right) \end{array} = 1.24$$ Dynamic shear amplification factor, ACI 318-19 18.10.3.1.3

$$M_{prratio} := \frac{M_{pr}}{M_u}$$

$$\Omega_v := \text{if}\left(\frac{h_{wcs}}{w_s} \le 1.5, 1.0, \max\left(M_{prratio}, 1.5\right)\right)$$

$$\Omega_v = 6.04$$

Shear Overstrength factor, ACI 318-19 18.10.3.1.2

$$V_{ratio} := min\left(\omega_v \cdot \Omega_v, 3\right) = 3$$

$$V_e := V_{ratio} \cdot V_u$$

$$V_{epiera} := \text{if}\left(\frac{w_s}{t} > 2.5, V_u \cdot \Omega_o, \text{"N/A"}\right)$$

Alternate Ve if pier aspect ratio met
ACI 318-19 18.10.8.1a

$$V_e := \text{if}\left(PierCheck = \text{"Pier"}, \text{if}\left(V_{epiera} = \text{"N/A"}, \frac{M_{pr}}{h_{roof}}, min\left(\frac{M_{pr}}{h_{roof}}, V_{epiera}\right)\right), V_{ratio} \cdot V_u\right)$$

$$V_e = 168.1\ k$$

Design Shear Force,
For Segments: ACI 318-19 18.10.3.1
For Piers: ACI 318-19 18.7.6.1

Shear Steel Requirements - ACI 318-19 18.10.2.1/11.6.1

$$A_{cv} := t \cdot w_s = 1920\ in^2$$

Gross Area of Concrete Section

$$V_{min} := \lambda \cdot \sqrt{\frac{f_c}{psi}} \cdot psi \cdot A_{cv}$$

Shear Strength to determine ρ_{min}

$$\rho_{min} := \text{if}\left(V_u < V_{min}, 0.002, 0.0025\right) = 0.002$$

$$A_{v\_min} := (t) \cdot (12 \cdot in) \cdot \left(\rho_{min}\right)$$

Minimum area of Shear Steel per Foot

Determine number of curtains of shear steel - ACI 318-19 18.10.2.2

$$V_{reinf} := 2 \cdot \lambda \cdot \sqrt{\frac{f_c}{psi}} \cdot psi \cdot A_{cv}$$

Shear Strength to Determine Required Number of Layers

$$Reinforcing := \text{if}\left(V_u > V_{reinf} \lor \frac{ht}{w_s} \ge 2, \text{"(2) CURTAINS REQD"}, \text{"(1) CURTAIN REQD"}\right)$$

$$Reinforcing = \text{"(2) CURTAINS REQD"}$$

Define Shear Steel:

$No_v := \#4$ — Shear Reinforcement Bar Size

$A_{sNov} := No_{v_0}$ $\quad d_{sv} := No_{v_1}$ — Area of Shear Reinforcement Bar

$S := 18 \cdot in$ — Shear Reinforcement Spacing (12" max, typical)

$N_L := 2$ — Number of Layers
(CL = 1, EF = 2)

$$A_{vs} := N_L \cdot \frac{12 \cdot in}{S} \cdot A_{sNov}$$ Area of Shear Reinforcement per Foot

$$\rho_t := \frac{A_{vs}}{(12 \cdot in) \cdot (t)}$$

$$A_{v\_min\_check} := \text{if}\left(A_{v\_min} \le A_{vs}, \text{"OK"}, \text{"REVISE!"}\right)$$ $A_{v\_min\_check} = \text{"OK"}$

$$Check := \text{if}\left(V_u > V_{reinf} \wedge N_L = 1, \text{"REVISE REINF"}, \text{"OK"}\right)$$ $Check = \text{"OK"}$

Shear Wall Strength - ACI 318-19 Eq. 18.10.4.1

$$\alpha_c := \begin{array}{l} \text{if } \frac{ht}{w_s} \le 1.5 \\ \quad 3.0 \\ \text{else if } \frac{ht}{w_s} \ge 2.0 \\ \quad 2.0 \\ \text{else} \\ \quad 2 + \frac{2.0 - \frac{ht}{w_s}}{0.5} \end{array} = 2$$

$$\phi V_n := \phi_{vE} \cdot \left(A_{cv} \cdot \left(\alpha_c \cdot \lambda \cdot \left(\sqrt{\frac{f_c}{psi}} \cdot psi\right) + \rho_t \cdot f_y\right)\right)$$ ACI 318-19 Eq. 18.10.4.1

Define Shear Strength Max - ACI 318-19 18.10.4.4

$$\phi V_{nmax} := \phi_{vE} \cdot \left(8 \cdot \sqrt{\frac{f_c}{psi}} \cdot psi \cdot A_{cv}\right)$$ ACI 318-19 Eq. 18.10.4.4

$$check := \text{if}\left(V_e \le \phi V_{nmax}, \text{"Proceed with Design"}, \text{"Revise!"}\right)$$ $check = \text{"Proceed with Design"}$

$check := \text{if}\left(\phi V_n \le \phi V_{nmax}, \text{"OK"}, \text{"Vn MAX exceeded, reduce reinforcement"}\right)$ $check = \text{"OK"}$

$DCR_v := \dfrac{V_e}{min\left(\phi V_n, \phi V_{nmax}\right)}$ $DCR_v = 0.5$

$Flag_v = \text{"USE (2) LAYER/S OF \#4 BARS AT 18''OC HORIZ MIN"}$

Flexural Steel - ACI 318-19 18.10.6

$No_f := \#5$ Longitudinal Reinforcement Bar Size

$A_{sNof} := No_{f_0}$ $d_{Nof} := No_{f_1}$

$N_{Lf} := 2$ Number of Layers of Longitudinal Reinforcement (CL = 1, EF = 2) at Each End

$N_f := 20$ Number of Longitudinal Reinforcement Bars per Layer at Each End

$d_{solve} := 0.8 \cdot w_s = 16\ ft$ Depth of Lever Arm

Solve Block — Guess Values / Constraints

$A_{sreq} := 1\ in^2$

$\phi_f \cdot A_{sreq} \cdot f_y \cdot \left(d_{solve} - \dfrac{A_{sreq} \cdot f_y}{0.85 \cdot f_c \cdot t \cdot 2}\right) - M_u = 0$

$A_{sreq\_flex} := \text{Find}\left(A_{sreq}\right)$

$A_{sreq\_flex} = 2.19\ in^2$ Required Area of Flexural Steel

$A_{s\_long} := N_f \cdot N_{Lf} \cdot A_{sNof} = 12.4\ in^2$ Area of Flexural Steel

$DCR_f := \dfrac{A_{sreq\_flex}}{A_{s\_long}}$ $DCR_f = 0.18$

$Flag_f = \text{"USE (40) \#5 BARS EACH END MIN"}$

Transverse Reinforcement Requirements - ACI 318-19 18.10.6.5

$$A_{s\_check} := \frac{400}{60000} \cdot (0.2 \cdot w_s) \cdot t = 2.56\ in^2$$

Assumes 0.2*Panel Width for N.A. depth. Input leg into SP Column if req'd to refine results.

$$V_{check} := \lambda \cdot \sqrt{\frac{f_c}{psi}} \cdot psi \cdot A_{cv} = 121.43\ k$$

Check shear against 18.10.6.5a

$$Check_{18.10.6.5a} := \text{if}\left(V_u > V_{check}, \text{“PROVIDE HOOKED HORIZ BARS OR HOOPED ENDS”}, \text{“OK”}\right)$$

$$Check_{18.10.6.5a} = \text{“OK”}$$

$$Check_{18.10.6.5b} := \text{if}\left(A_{sreq\_flex} > A_{s\_check}, \text{“PROVIDE CONFINING HOOPS @ 5'' O.C. MAX per 18.10.6.5”}, \text{“OK”}\right)$$

$$Check_{18.10.6.5b} = \text{“OK”}$$

Basic Boundary Element Check w/ Gross Section Properties - ACI 318-19 18.10.6.3

$$I_g := \frac{(w_s)^3 \cdot t}{12} \qquad A_g := \frac{w_s \cdot t}{2} \qquad S_g := \frac{t \cdot {w_s}^2}{6}$$

$$P_{u1} := \frac{P_{u1}}{2}$$

Factored Axial Load - ASCE 7-16 2.3.6 Load Combo 6

$$\sigma_u := \frac{M_u}{S_g} + \frac{P_{u1}}{A_g}$$

Combined Stress @ Extreme Fiber

$$F_c := 0.2 \cdot f_c$$

$$DCR_\sigma := \frac{\sigma_u}{F_c} \qquad DCR_\sigma = 0.45$$

$$BasicCheck := \text{if}\left(DCR_\sigma > 1, \text{“Perform Alternate Analysis”}, \text{“Okay”}\right) \qquad BasicCheck = \text{“Okay”}$$

If Wall does not meet basic boundary element check, provide boundary elements or perform Alternate check per ACI 318-19 18.10.6.2 ONLY if hwcs/ w_s ≥ 2.

$$AlternateCheck := \begin{array}{|l} \text{if } \dfrac{h_{wcs}}{w_s} \geq 2 \wedge BasicCheck = \text{"Perform Alternate Analysis"} \\ \quad \text{"Proceed"} \\ \text{also if } BasicCheck = \text{"Okay"} \\ \quad \text{"Not Applicable"} \\ \text{else} \\ \quad \text{"Boundary Element Necessary – Alternate Check Not Allowed"} \end{array}$$

$AlternateCheck = \text{"Not Applicable"}$

Verify Section Uncracked

$y_t := \dfrac{w_s}{2}$ Neutral Axis of Gross Section

$M_{cr} := \dfrac{7.5 \cdot \lambda \cdot \sqrt{\dfrac{f_c}{psi}} \cdot psi \cdot I_g}{y_t}$ Cracking Moment - ACI 318-19 19.2.3.1 Eq 19.2.3.1, 24.2.3.5 Eq 24.2.3.5

$M_a := M_u \cdot EL_{asd\_factor}$ Service Moment

$M_{crCheck} := \text{if}\left(M_{cr} < M_a, \text{"Cracked; If necessary adjust relative stiffness and evaluate load"}, \text{"Uncracked"}\right)$

$M_{crCheck} = \text{"Uncracked"}$

Slab Shear Dowel Bars - Shear Friction per ACI 318-19 22.9.4

$\mu := 0.6$ Coefficient of Friction - ACI 318-19 22.9.4.2

$No_{dowel} := \#4$ Dowel Reinforcement Bar Size No.

$A_{s\_Nodowel} := No_{dowel}$

$S_{dowel} := 12 \cdot in$ Dowel Bar Spacing

$A_{dowel} := \dfrac{A_{s\_Nodowel}}{S_{dowel}}$ Dowel Bar Area per Foot

$A_{vf} := A_{dowel} \cdot w_s = 4\ in^2$ Area of Shear Friction Steel Across Panel

$\phi V_{vf\_n} := \phi_{vE} \cdot \left(\mu \cdot A_{vf} \cdot f_y\right)$ Design Shear Capacity - ACI 318-19 22.9.4.2

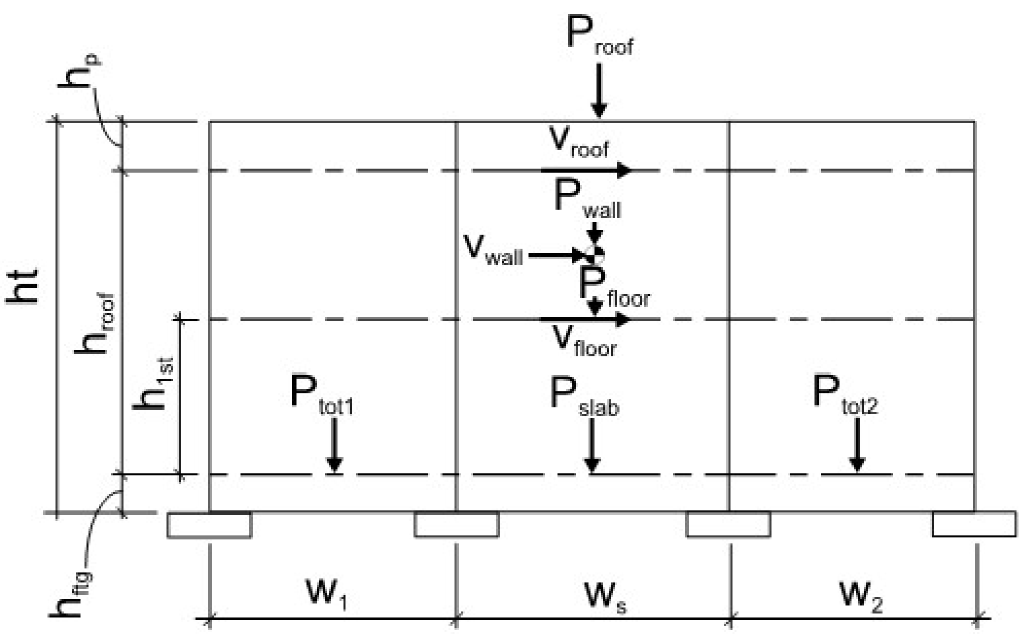
$V_u := \rho \cdot V_{tot} = 56\ k$ Ultimate Panel Shear ($\rho\, Q_e$)

$$DCR_{vf} := \frac{V_u}{\phi V_{vf\_n}}$$

$$DCR_{vf} = 0.65$$

ACI 318-19 Solid Panel - Panel 54 - Grid 1

In-Plane Shear Solid Panel:



Shear Panel (w_s)

$w_s := 26.5\ ft$	Panel Width
$h_p := 1.5 \cdot ft$	Parapet Height
$h_{roof} := 39\ ft$	Height of Roof Above FF
$h_{1st} := 0 \cdot ft$	Height of Intermediate Floor Above FF
$h_{ftg} := 1.5 \cdot ft$	FF to T/FTG Height
$ht := h_p + h_{roof} + h_{ftg} = 42\ ft$	Total Panel Height (T/FTG to T/PARAPET)
$t_p := 8.75 \cdot in$	Panel Thickness
$r_v := 0.75 \cdot in$	Reveal Depth
$t := t_p - r_v = 8\ in$	Effective Panel Thickness
$Wt_p := w_c \cdot t_p$	Panel Weight (psf)

Shear Wall Design

Adjacent Panel (w_1)

$w_1 := w_s$

$h_{p1} := h_p$

$h_{roof1} := h_{roof}$

$h_{ftg1} := h_{ftg}$

$ht_1 := h_{p1} + h_{roof1} + h_{ftg1} = 42\ ft$

$t_{p1} := t_p$

$Ao_1 := 0 \cdot ft \cdot 0 \cdot ft$

Adjacent Panel (w_2)

$w_2 := w_s$ — Adjacent Panel Width

$h_{p2} := h_p$ — Parapet Height

$h_{roof2} := h_{roof}$ — Height of Roof Above FF

$h_{ftg2} := h_{ftg}$ — FF to T/FTG Height

$ht_2 := h_{p2} + h_{roof2} + h_{ftg2} = 42\ ft$ — Total Panel Ht. (T/FTG to T/PARAPET)

$t_{p2} := t_p$ — Panel Thickness

$Ao_2 := 0 \cdot ft \cdot 0 \cdot ft$ — Area of Openings

Loading:

Seismic (1.0 Q_e)

$V_{roof} := 50.9\ k$ — Seismic Shear @ Roof- ρ not included

$V_{floor} := 0 \cdot k$ — Seismic Shear @ Floor

$V_{wall} := \left(ht - h_{ftg}\right) \cdot w_s \cdot Wt_p \cdot C_s$ — Seismic Shear due to Wall Panel Mass

$V_{tot} := V_{roof} + V_{wall} + V_{floor} = 70.7\ k$ — Total Seismic Shear

$M_{ot\_roof} := V_{roof} \cdot h_{roof}$ — Overturning Moment due to Roof Shear

$M_{ot\_floor} := V_{floor} \cdot h_{1st}$ — Overturning Moment due to Floor Shear

$M_{ot\_wall} := V_{wall} \cdot \dfrac{\left(ht - h_{ftg}\right)}{2}$ — Overturning Moment due to Panel Mass

$M_{ot} := M_{ot\_roof} + M_{ot\_floor} + M_{ot\_wall} = 2386\ k \cdot ft$ — Total Overturning Moment @ Wall Panel

Gravity:

Shear Panel (w_s):

$l_{roof\_trib} := \dfrac{10}{2} \cdot ft$ — Length of Roof Tributary to Panel

$l_{floor\_trib} := \dfrac{0}{2} \cdot ft$ — Length of Floor Tributary to Panel

Shear Wall Design

$SL := 0.421\ klf$ Roof snow load tributary to panel

$LL := 0.0\ klf$ Floor live load tributary to panel

$P_{roof} := DL_{sei} \cdot l_{roof\_trib} \cdot w_s$ Roof DL

$P_{floor} := DL_{flr} \cdot l_{floor\_trib} \cdot w_s$ Floor DL

$P_{wall} := w_s \cdot ht \cdot t_p \cdot w_c$ Wall Panel DL

$P_{slab} := w_c \cdot t_{SOG} \cdot 8 \cdot ft \cdot w_s$ Slab-on-Grade DL (assumes 8' width of slab over width of panel available to resist OT)

$P_{tot} := P_{roof} + P_{floor} + P_{wall} = 123.2\ k$ Total DL (without slab @ wall considered)

Panel (w_1):

$P_{roof1} := P_{roof} \cdot \left(\frac{w_1}{w_s}\right)$ Roof DL

$P_{floor1} := P_{floor} \cdot \left(\frac{w_1}{w_s}\right)$ Floor DL

$P_{wall1} := \left(w_1 \cdot ht_1 - Ao_1\right) \cdot t_{p1} \cdot w_c$ Wall Panel DL

$P_{slab1} := P_{slab} \cdot \left(\frac{w_1}{w_s}\right)$ Slab-on-Grade DL

$P_{tot1} := P_{roof1} + P_{floor1} + P_{wall1} + P_{slab1} = 141.7\ k$ Total DL (including slab weight)

Panel (w_2):

$P_{roof2} := P_{roof} \cdot \left(\frac{w_2}{w_s}\right)$ Roof DL

$P_{floor2} := P_{floor} \cdot \frac{w_2}{w_s}$ Floor DL

$P_{wall2} := \left(w_2 \cdot ht_2 - Ao_2\right) \cdot t_{p2} \cdot w_c$ Wall Panel DL

$P_{slab2} := P_{slab} \cdot \left(\frac{w_2}{w_s}\right)$ Slab-on-Grade DL

$P_{tot2} := P_{roof2} + P_{floor2} + P_{wall2} + P_{slab2} = 141.7\ k$ Total DL (including slab weight)

Resisting Moment:

$M_{resist} := \left(P_{tot} + P_{slab}\right) \cdot \frac{w_s}{2} = 1878\ k \cdot ft$ Resisting Moment at Shear Panel

Load Combinations:

ASCE 7-16 2.3.6

LRFD 6: (1.2+0.2Sds)D+1.0ρ Q_e + L + 0.2S

LRFD 7: (0.9 - 0.2Sds)D + 1.0ρ Q_e (Resisting Moment of Slab-on-Grade)

IBC 2021 1605.2 - Foundations

ASD 16-5: D + L + S + E/1.4 (Foundation Bearing)

ASD 16-6: 0.9D + E/1.4 (Foundation Uplift/Weight. Ev = 0 for fdn size, Ev included for hold downs)

Panel Hold Down - IBC 2021 1605.2 Alternative Load Combination ASD 16-6

$$M_{ot\_IBC} := \rho \cdot \frac{M_{ot}}{1.4}$$ ASD Overturning Moment

$$M_{res\_IBC} := \left(0.9 - \frac{0.2 \cdot S_{ds}}{1.4}\right) \cdot M_{resist}$$ ASD Resisting Moment

$$No_{hd} := \#6$$ Rebar Size No.

$$N_{hd} := 2$$ Number of Bars in Hold Down

$$L_{hd} := \text{if}\left(No_{hd_0} = 0.31\ in^2, L_{HD5_{N_{hd}}}, L_{HD6_{N_{hd}}}\right) = 12\ in$$

$$A_{shd} := No_{hd_0} \cdot N_{hd}$$ Area of Steel in Hold Down

$$hd_{check} := M_{ot\_IBC} - M_{res\_IBC}$$ Holdown check

$$T_{hd} := \frac{M_{ot} - \left(0.9 - 0.2\ S_{ds}\right) \cdot M_{resist}}{w_s - \left(1 \cdot ft + 0.5 \cdot L_{hd}\right)} = 40.5\ k$$ LRFD Tension at Hold Down

$$A_{shdreq} := \frac{T_{hd}}{\phi_t \cdot f_y} = 0.75\ in^2$$ Required Area of Steel

$$DCR_{hd} := \frac{A_{shdreq}}{A_{shd}}$$

$$DCR_{hd} = 0.85$$

$hd\_Results$ = "USE (2) #6 BARS AT HOLDOWNS"

Pad Footing Uplift and Seismic Bearing Checks

$l_f := 13.25\ ft$ Length of Footing

$w_f := 4.5\ ft$ Width of Footing

$d_f := 1\ ft$ Depth of Footing (Use 2'-0" MIN @ hold down, see footing calcs)

$P_{ftg} := w_c \cdot l_f \cdot w_f \cdot d_f$ Footing Weight

Required Footing Weight - Load Combination ASD 16-6

$$M_{res\_uplift} := M_{res\_IBC} + \left(0.9 \cdot \left(min\left(\frac{P_{tot1}}{2}, \frac{P_{tot2}}{2}\right) \cdot w_s\right)\right)$$

$M_{res\_uplift} = 3154\ k \cdot ft$ Factored Resisting Moment (Including contribution of slab & adjacent panel)

$$Wt_{ftg\_uplift} := \frac{\left(M_{ot\_IBC} - M_{res\_uplift}\right)}{w_s - \left(1\ ft + 0.5 \cdot L_{hd}\right)}$$

$Wt_{ftg\_uplift} = -58\ k$ Required Footing Weight

$flag_w := \text{if}\left(Wt_{ftg\_uplift} \le P_{ftg}, \text{“OK”}, \text{“NG!!!”}\right)$ $flag_w = \text{“OK”}$

Shear Wall Design

Wall Shear Design

Factored Wall Axial Forces for M_{pr} Calculation

$P_{u1} := (1.2 + 0.2 \cdot S_{ds}) \cdot P_{tot} + (0.2\ SL + LL)\ w_s = 171\ k$ — ASCE 7-16 12.2.3.6 Eq. 6

$P_{u2} := (0.9 - 0.2\ S_{ds})\ P_{tot} = 90.09\ k$ — ASCE 7-16 12.2.3.6 Eq. 7

Probable Moment and Moment Demand

$M_{pr} := 18661\ k \cdot ft$ — Largest probable flexural strength of the section from SP Column assuming tensile stress of 1.25*Fy and Φ = 1.0

$M_u := \rho \cdot M_{ot}$

$V_u := \rho \cdot V_{tot} = 70.7\ k$ — Ultimate Panel Shear ($\rho\, Q_e$)

Pier Check - ACI 318-19 Table R18.10.1

If wall section is a pier, requirements of ACI 318-19 18.10.8 or 18.14 must be met.

$$Check_1 := \text{if}\left(\frac{w_s}{t_p} \le 6, \text{“Pier”}, \text{“Segment”}\right)$$

$$PierCheck := \text{if}\left(\frac{h_{roof}}{w_s} \ge 2, Check_1, \text{“Segment”}\right)$$

$PierCheck = \text{“Segment”}$

Design Shear Force - ACI 318-19 18.10.3

$h_{wcs} := h_{roof} + h_p$ — Height of Wall above Critical Section

$stories_{wcs} := 1$ — Number of Stories above Critical Section

$$n_s := \max\left(stories_{wcs}, 0.007 \cdot \frac{h_{wcs}}{in}\right)$$

$$\omega_v := \left\| \begin{array}{l} \text{if } \dfrac{h_{wcs}}{w_s} < 2 \\ \quad \| \ 1.0 \\ \text{else if } n_s \le 6.0 \\ \quad \| \ 0.9 + \dfrac{n_s}{10} \\ \text{else} \\ \quad \| \ min\left(1.3 + \dfrac{n_s}{30}, 1.8\right) \end{array} \right\| = 1$$

Dynamic shear amplification factor, ACI 318-19 18.10.3.1.3

Shear Wall Design

$$M_{prratio} := \frac{M_{pr}}{M_u}$$

$$\Omega_v := \text{if}\left(\frac{h_{wcs}}{w_s} \le 1.5, 1.0, \max\left(M_{prratio}, 1.5\right)\right)$$

$$\Omega_v = 7.82$$

Shear Overstrength factor, ACI 318-19 18.10.3.1.2

$$V_{ratio} := min\left(\omega_v \cdot \Omega_v, 3\right) = 3$$

$$V_e := V_{ratio} \cdot V_u$$

$$V_{epiera} := \text{if}\left(\frac{w_s}{t} > 2.5, V_u \cdot \Omega_o, \text{“N/A”}\right)$$

Alternate Ve if pier aspect ratio met
ACI 318-19 18.10.8.1a

$$V_e := \text{if}\left(PierCheck = \text{“Pier”}, \text{if}\left(V_{epiera} = \text{“N/A”}, \frac{M_{pr}}{h_{roof}}, min\left(\frac{M_{pr}}{h_{roof}}, V_{epiera}\right)\right), V_{ratio} \cdot V_u\right)$$

$$V_e = 212.1\ \boldsymbol{k}$$

Design Shear Force,
For Segments: ACI 318-19 18.10.3.1
For Piers: ACI 318-19 18.7.6.1

Shear Steel Requirements - ACI 318-19 18.10.2.1/11.6.1

$$A_{cv} := t \cdot w_s = 2544\ \boldsymbol{in}^2$$

Gross Area of Concrete Section

$$V_{min} := \lambda \cdot \sqrt{\frac{f_c}{\boldsymbol{psi}}} \cdot \boldsymbol{psi} \cdot A_{cv}$$

Shear Strength to determine ρ_{min}

$$\rho_{min} := \text{if}\left(V_u < V_{min}, 0.002, 0.0025\right) = 0.002$$

$$A_{v\_min} := (t) \cdot (12 \cdot \boldsymbol{in}) \cdot \left(\rho_{min}\right)$$

Minimum area of Shear Steel per Foot

Determine number of curtains of shear steel - ACI 318-19 18.10.2.2

$$V_{reinf} := 2 \cdot \lambda \cdot \sqrt{\frac{f_c}{\boldsymbol{psi}}} \cdot \boldsymbol{psi} \cdot A_{cv}$$

Shear Strength to Determine Required Number of Layers

$$Reinforcing := \text{if}\left(V_u > V_{reinf} \lor \frac{ht}{w_s} \ge 2, \text{“(2) CURTAINS REQD”}, \text{“(1) CURTAIN REQD”}\right)$$

$$Reinforcing = \text{“(1) CURTAIN REQD”}$$

Define Shear Steel:

$No_v := \#4$ — Shear Reinforcement Bar Size

$A_{sNov} := No_{v_0}$ $\quad d_{sv} := No_{v_1}$ — Area of Shear Reinforcement Bar

$S := 12 \cdot in$ — Shear Reinforcement Spacing (12" max, typical)

$N_L := 1$ — Number of Layers **(CL = 1, EF = 2)**

$$A_{vs} := N_L \cdot \frac{12 \cdot in}{S} \cdot A_{sNov}$$ Area of Shear Reinforcement per Foot

$$\rho_t := \frac{A_{vs}}{(12 \cdot in) \cdot (t)}$$

$A_{v\_min\_check} := \text{if}\left(A_{v\_min} \le A_{vs}, \text{“OK”}, \text{“REVISE!”}\right)$ $\quad A_{v\_min\_check} = \text{“OK”}$

$Check := \text{if}\left(V_u > V_{reinf} \land N_L = 1, \text{“REVISE REINF”}, \text{“OK”}\right)$ $\quad Check = \text{“OK”}$

Shear Wall Strength - ACI 318-19 Eq. 18.10.4.1

$$\alpha_c := \begin{cases} 3.0 & \text{if } \dfrac{ht}{w_s} \le 1.5 \\ 2.0 & \text{else if } \dfrac{ht}{w_s} \ge 2.0 \\ 2 + \dfrac{2.0 - \dfrac{ht}{w_s}}{0.5} & \text{else} \end{cases} = 2.83$$

$$\phi V_n := \phi_{vE} \cdot \left(A_{cv} \cdot \left(\alpha_c \cdot \lambda \cdot \left(\sqrt{\frac{f_c}{psi}} \cdot psi\right) + \rho_t \cdot f_y\right)\right)$$ ACI 318-19 Eq. 18.10.4.1

Define Shear Strength Max - ACI 318-19 18.10.4.4

$$\phi V_{nmax} := \phi_{vE} \cdot \left(8 \cdot \sqrt{\frac{f_c}{psi}} \cdot psi \cdot A_{cv}\right)$$ ACI 318-19 Eq. 18.10.4.4

$check := \text{if}\left(V_e \le \phi V_{nmax}, \text{“Proceed with Design”}, \text{“Revise!”}\right)$ $\quad check = \text{“Proceed with Design”}$

$check := \text{if}\left(\phi V_n \le \phi V_{nmax}, \text{"OK"}, \text{"Vn MAX exceeded, reduce reinforcement"}\right)$ $check = \text{"OK"}$

$DCR_v := \dfrac{V_e}{min\left(\phi V_n, \phi V_{nmax}\right)}$ $DCR_v = 0.46$

$Flag_v = \text{"USE (1) LAYER /S OF \#4 BARS AT 12''OC HORIZ MIN"}$

Flexural Steel - ACI 318-19 18.10.6

$No_f := \#5$ Longitudinal Reinforcement Bar Size

$A_{sNof} := No_{f_0}$ $d_{Nof} := No_{f_1}$

$N_{Lf} := 2$ Number of Layers of Longitudinal Reinforcement (CL = 1, EF = 2) at Each End

$N_f := 26$ Number of Longitudinal Reinforcement Bars per Layer at Each End

$d_{solve} := 0.8 \cdot w_s = 21.2\ ft$ Depth of Lever Arm

Solve Block: Guess Values / Constraints / Solver

$A_{sreq} := 1\ in^2$

$\phi_f \cdot A_{sreq} \cdot f_y \cdot \left(d_{solve} - \dfrac{A_{sreq} \cdot f_y}{0.85 \cdot f_c \cdot t \cdot 2}\right) - M_u = 0$

$A_{sreq\_flex} := \text{Find}\left(A_{sreq}\right)$

$A_{sreq\_flex} = 2.1\ in^2$ Required Area of Flexural Steel

$A_{s\_long} := N_f \cdot N_{Lf} \cdot A_{sNof} = 16.12\ in^2$ Area of Flexural Steel

$DCR_f := \dfrac{A_{sreq\_flex}}{A_{s\_long}}$ $DCR_f = 0.13$

$Flag_f = \text{"USE (52) \#5 BARS EACH END MIN"}$

Transverse Reinforcement Requirements - ACI 318-19 18.10.6.5

$$A_{s\_check} := \frac{400}{60000} \cdot (0.2 \cdot w_s) \cdot t = 3.392\ in^2$$

Assumes 0.2*Panel Width for N.A. depth. Input leg into SP Column if req'd to refine results.

$$V_{check} := \lambda \cdot \sqrt{\frac{f_c}{psi}} \cdot psi \cdot A_{cv} = 160.9\ k$$

Check shear against 18.10.6.5a

$$Check_{18.10.6.5a} := \text{if}\left(V_u > V_{check}, \text{“PROVIDE HOOKED HORIZ BARS OR HOOPED ENDS”}, \text{“OK”}\right)$$

$$Check_{18.10.6.5a} = \text{“OK”}$$

$$Check_{18.10.6.5b} := \text{if}\left(A_{sreq\_flex} > A_{s\_check}, \text{“PROVIDE CONFINING HOOPS @ 5'' O.C. MAX per 18.10.6.5”}, \text{“OK”}\right)$$

$$Check_{18.10.6.5b} = \text{“OK”}$$

Basic Boundary Element Check w/ Gross Section Properties - ACI 318-19 18.10.6.3

$$I_g := \frac{(w_s)^3 \cdot t}{12} \qquad A_g := \frac{w_s \cdot t}{2} \qquad S_g := \frac{t \cdot {w_s}^2}{6}$$

$$P_{u1} := \frac{P_{u1}}{2}$$

Factored Axial Load - ASCE 7-16 2.3.6 Load Combo 6

$$\sigma_u := \frac{M_u}{S_g} + \frac{P_{u1}}{A_g}$$

Combined Stress @ Extreme Fiber

$$F_c := 0.2 \cdot f_c$$

$$DCR_\sigma := \frac{\sigma_u}{F_c} \qquad DCR_\sigma = 0.35$$

$$BasicCheck := \text{if}\left(DCR_\sigma > 1, \text{“Perform Alternate Analysis”}, \text{“Okay”}\right) \qquad BasicCheck = \text{“Okay”}$$

If Wall does not meet basic boundary element check, provide boundary elements or perform Alternate check per ACI 318-19 18.10.6.2 ONLY if hwcs/ w_s ≥ 2.

$$AlternateCheck := \begin{cases} \text{if } \frac{h_{wcs}}{w_s} \geq 2 \wedge BasicCheck = \text{“Perform Alternate Analysis”} \\ \quad \text{“Proceed”} \\ \text{also if } BasicCheck = \text{“Okay”} \\ \quad \text{“Not Applicable”} \\ \text{else} \\ \quad \text{“Boundary Element Necessary – Alternate Check Not Allowed”} \end{cases}$$

$AlternateCheck = \text{“Not Applicable”}$

Verify Section Uncracked

$y_t := \frac{w_s}{2}$ Neutral Axis of Gross Section

$M_{cr} := \frac{7.5 \cdot \lambda \cdot \sqrt{\frac{f_c}{psi}} \cdot psi \cdot I_g}{y_t}$ Cracking Moment - ACI 318-19 19.2.3.1 Eq 19.2.3.1, 24.2.3.5 Eq 24.2.3.5

$M_a := M_u \cdot EL_{asd\_factor}$ Service Moment

$M_{crCheck} := \text{if}\left(M_{cr} < M_a, \text{“Cracked; If necessary adjust relative stiffness and evaluate load”}, \text{“Uncracked”}\right)$

$M_{crCheck} = \text{“Uncracked”}$

Slab Shear Dowel Bars - Shear Friction per ACI 318-19 22.9.4

$\mu := 0.6$ Coefficient of Friction - ACI 318-19 22.9.4.2

$No_{dowel} := \#4$ Dowel Reinforcement Bar Size No.

$A_{s\_Nodowel} := No_{dowel}$

$S_{dowel} := 12 \cdot in$ Dowel Bar Spacing

$A_{dowel} := \frac{A_{s\_Nodowel}}{S_{dowel}}$ Dowel Bar Area per Foot

$A_{vf} := A_{dowel} \cdot w_s = 5.3\ in^2$ Area of Shear Friction Steel Across Panel

$\phi V_{vf\_n} := \phi_{vE} \cdot \left(\mu \cdot A_{vf} \cdot f_y\right)$ Design Shear Capacity - ACI 318-19 22.9.4.2

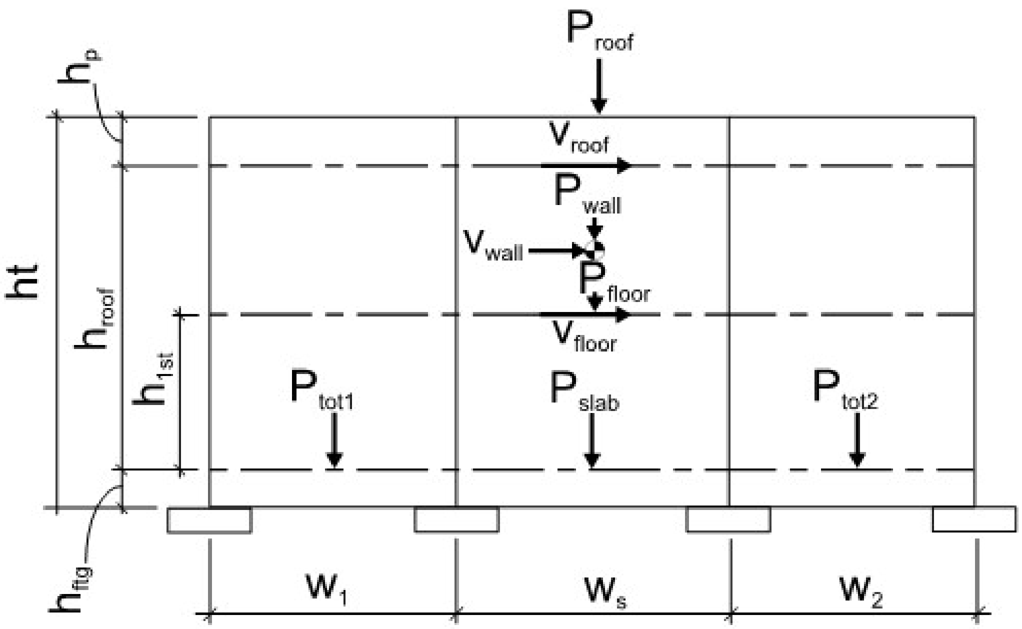
$V_u := \rho \cdot V_{tot} = 70.7\ k$ Ultimate Panel Shear ($\rho\, Q_e$)

$$DCR_{vf} := \frac{V_u}{\phi V_{vf\_n}}$$

$$DCR_{vf} = 0.62$$

ACI 318-19 Solid Panel - Panel 47 - South

In-Plane Shear Solid Panel:



Shear Panel (w_s)

$w_s := 26\ ft$	Panel Width
$h_p := 4.5 \cdot ft$	Parapet Height
$h_{roof} := 36\ ft$	Height of Roof Above FF
$h_{1st} := 0 \cdot ft$	Height of Intermediate Floor Above FF
$h_{ftg} := 1.5 \cdot ft$	FF to T/FTG Height
$ht := h_p + h_{roof} + h_{ftg} = 42\ ft$	Total Panel Height (T/FTG to T/PARAPET)
$t_p := 8 \cdot in$	Panel Thickness
$r_v := 0.75 \cdot in$	Reveal Depth
$t := t_p - r_v = 7.25\ in$	Effective Panel Thickness
$Wt_p := w_c \cdot t_p$	Panel Weight (psf)

Shear Wall Design

Adjacent Panel (w_1)

$w_1 := w_s$

$h_{p1} := h_p$

$h_{roof1} := h_{roof}$

$h_{ftg1} := h_{ftg}$

$ht_1 := h_{p1} + h_{roof1} + h_{ftg1} = 42\ ft$

$t_{p1} := t_p$

$Ao_1 := 0 \cdot ft \cdot 0 \cdot ft$

Adjacent Panel (w_2)

$w_2 := w_s$ — Adjacent Panel Width

$h_{p2} := h_p$ — Parapet Height

$h_{roof2} := h_{roof}$ — Height of Roof Above FF

$h_{ftg2} := h_{ftg}$ — FF to T/FTG Height

$ht_2 := h_{p2} + h_{roof2} + h_{ftg2} = 42\ ft$ — Total Panel Ht. (T/FTG to T/PARAPET)

$t_{p2} := t_p$ — Panel Thickness

$Ao_2 := 0 \cdot ft \cdot 0 \cdot ft$ — Area of Openings

Loading:

Seismic (1.0 Q_e)

$V_{roof} := 11.4\ k$ — Seismic Shear @ Roof- ρ not included

$V_{floor} := 0 \cdot k$ — Seismic Shear @ Floor

$V_{wall} := \left(ht - h_{ftg}\right) \cdot w_s \cdot Wt_p \cdot C_s$ — Seismic Shear due to Wall Panel Mass

$V_{tot} := V_{roof} + V_{wall} + V_{floor} = 29.2\ k$ — Total Seismic Shear

$M_{ot\_roof} := V_{roof} \cdot h_{roof}$ — Overturning Moment due to Roof Shear

$M_{ot\_floor} := V_{floor} \cdot h_{1st}$ — Overturning Moment due to Floor Shear

$M_{ot\_wall} := V_{wall} \cdot \dfrac{\left(ht - h_{ftg}\right)}{2}$ — Overturning Moment due to Panel Mass

$M_{ot} := M_{ot\_roof} + M_{ot\_floor} + M_{ot\_wall} = 770\ k \cdot ft$ — Total Overturning Moment @ Wall Panel

Gravity:

Shear Panel (w_s):

$l_{roof\_trib} := \dfrac{59.33}{2} \cdot ft$ — Length of Roof Tributary to Panel

$l_{floor\_trib} := \dfrac{0}{2} \cdot ft$ — Length of Floor Tributary to Panel

Shear Wall Design

$SL := 0.4\ klf$ Roof snow load tributary to panel

$LL := 0.0\ klf$ Floor live load tributary to panel

$P_{roof} := DL_{sei} \cdot l_{roof\_trib} \cdot w_s$ Roof DL

$P_{floor} := DL_{flr} \cdot l_{floor\_trib} \cdot w_s$ Floor DL

$P_{wall} := w_s \cdot ht \cdot t_p \cdot w_c$ Wall Panel DL

$P_{slab} := w_c \cdot t_{SOG} \cdot 8 \cdot ft \cdot w_s$ Slab-on-Grade DL (assumes 8' width of slab over width of panel available to resist OT)

$P_{tot} := P_{roof} + P_{floor} + P_{wall} = 117.6\ k$ Total DL (without slab @ wall considered)

Panel (w_1):

$P_{roof1} := P_{roof} \cdot \left(\frac{w_1}{w_s}\right)$ Roof DL

$P_{floor1} := P_{floor} \cdot \left(\frac{w_1}{w_s}\right)$ Floor DL

$P_{wall1} := \left(w_1 \cdot ht_1 - Ao_1\right) \cdot t_{p1} \cdot w_c$ Wall Panel DL

$P_{slab1} := P_{slab} \cdot \left(\frac{w_1}{w_s}\right)$ Slab-on-Grade DL

$P_{tot1} := P_{roof1} + P_{floor1} + P_{wall1} + P_{slab1} = 135.8\ k$ Total DL (including slab weight)

Panel (w_2):

$P_{roof2} := P_{roof} \cdot \left(\frac{w_2}{w_s}\right)$ Roof DL

$P_{floor2} := P_{floor} \cdot \frac{w_2}{w_s}$ Floor DL

$P_{wall2} := \left(w_2 \cdot ht_2 - Ao_2\right) \cdot t_{p2} \cdot w_c$ Wall Panel DL

$P_{slab2} := P_{slab} \cdot \left(\frac{w_2}{w_s}\right)$ Slab-on-Grade DL

$P_{tot2} := P_{roof2} + P_{floor2} + P_{wall2} + P_{slab2} = 135.8\ k$ Total DL (including slab weight)

Resisting Moment:

$M_{resist} := \left(P_{tot} + P_{slab}\right) \cdot \frac{w_s}{2} = 1765\ k \cdot ft$ Resisting Moment at Shear Panel

Load Combinations:

ASCE 7-16 2.3.6

LRFD 6: (1.2+0.2Sds)D+1.0ρ Q_e + L + 0.2S

LRFD 7: (0.9 - 0.2Sds)D + 1.0ρ Q_e (Resisting Moment of Slab-on-Grade)

IBC 2021 1605.2 - Foundations

ASD 16-5: D + L + S + E/1.4 (Foundation Bearing)

ASD 16-6: 0.9D + E/1.4 (Foundation Uplift/Weight. Ev = 0 for fdn size, Ev included for hold downs)

Panel Hold Down - IBC 2021 1605.2 Alternative Load Combination ASD 16-6

$$M_{ot\_IBC} := \rho \cdot \frac{M_{ot}}{1.4}$$ ASD Overturning Moment

$$M_{res\_IBC} := \left(0.9 - \frac{0.2 \cdot S_{ds}}{1.4}\right) \cdot M_{resist}$$ ASD Resisting Moment

$$No_{hd} := \#6$$ Rebar Size No.

$$N_{hd} := 2$$ Number of Bars in Hold Down

$$L_{hd} := \text{if}\left(No_{hd_0} = 0.31\ in^2, L_{HD5_{N_{hd}}}, L_{HD6_{N_{hd}}}\right) = 12\ in$$

$$A_{shd} := No_{hd_0} \cdot N_{hd}$$ Area of Steel in Hold Down

$$hd_{check} := M_{ot\_IBC} - M_{res\_IBC}$$ Holdown check

$$T_{hd} := \frac{M_{ot} - (0.9 - 0.2\ S_{ds}) \cdot M_{resist}}{w_s - (1 \cdot ft + 0.5 \cdot L_{hd})} = -21.3\ k$$ LRFD Tension at Hold Down

$$A_{shdreq} := \frac{T_{hd}}{\phi_t \cdot f_y} = -0.39\ in^2$$ Required Area of Steel

$$DCR_{hd} := \frac{A_{shdreq}}{A_{shd}}$$

$$DCR_{hd} = -0.45$$

$$hd\_Results = \text{“NO HOLDOWNS REQD”}$$

Pad Footing Uplift and Seismic Bearing Checks

$l_f := \frac{w_s}{2}$ Length of Footing

$w_f := 4.5\ ft$ Width of Footing

$d_f := 1\ ft$ Depth of Footing (Use 2'-0" MIN @ hold down, see footing calcs)

$P_{ftg} := w_c \cdot l_f \cdot w_f \cdot d_f$ Footing Weight

Required Footing Weight - Load Combination ASD 16-6

$$M_{res\_uplift} := M_{res\_IBC} + \left(0.9 \cdot \left(min\left(\frac{P_{tot1}}{2}, \frac{P_{tot2}}{2}\right) \cdot w_s\right)\right)$$

$M_{res\_uplift} = 2965\ k \cdot ft$ Factored Resisting Moment (Including contribution of slab & adjacent panel)

$$Wt_{ftg\_uplift} := \frac{\left(M_{ot\_IBC} - M_{res\_uplift}\right)}{w_s - \left(1\ ft + 0.5 \cdot L_{hd}\right)}$$

$Wt_{ftg\_uplift} = -98.6\ k$ Required Footing Weight

$flag_w := \text{if}\left(Wt_{ftg\_uplift} \le P_{ftg}, \text{“OK”}, \text{“NG!!!”}\right)$ $flag_w = \text{“OK”}$

Wall Shear Design

Factored Wall Axial Forces for M_{pr} Calculation

$P_{u1} := \left(1.2 + 0.2 \cdot S_{ds}\right) \cdot P_{tot} + \left(0.2\ SL + LL\right)\ w_s = 163\ k$ ASCE 7-16 12.2.3.6 Eq. 6

$P_{u2} := \left(0.9 - 0.2\ S_{ds}\right)\ P_{tot} = 86.02\ k$ ASCE 7-16 12.2.3.6 Eq. 7

Probable Moment and Moment Demand

$M_{pr} := 17000\ k \cdot ft$ Largest probable flexural strength of the section from SP Column assuming tensile stress of 1.25*Fy and Φ = 1.0

$M_u := \rho \cdot M_{ot}$

$V_u := \rho \cdot V_{tot} = 29.2\ k$ Ultimate Panel Shear ($\rho\, Q_e$)

Pier Check - ACI 318-19 Table R18.10.1

If wall section is a pier, requirements of ACI 318-19 18.10.8 or 18.14 must be met.

$$Check_1 := \text{if}\left(\frac{w_s}{t_p} \le 6, \text{“Pier”}, \text{“Segment”}\right)$$

$$PierCheck := \text{if}\left(\frac{h_{roof}}{w_s} \ge 2, Check_1, \text{“Segment”}\right)$$

$PierCheck = \text{“Segment”}$

Design Shear Force - ACI 318-19 18.10.3

$h_{wcs} := h_{roof} + h_p$ Height of Wall above Critical Section

$stories_{wcs} := 1$ Number of Stories above Critical Section

$$n_s := \max\left(stories_{wcs}, 0.007 \cdot \frac{h_{wcs}}{in}\right)$$

$$\omega_v := \begin{Vmatrix} \text{if } \dfrac{h_{wcs}}{w_s} < 2 \\ \quad \Vert\ 1.0 \\ \text{else if } n_s \le 6.0 \\ \quad \Vert\ 0.9 + \dfrac{n_s}{10} \\ \text{else} \\ \quad \Vert\ min\left(1.3 + \dfrac{n_s}{30}, 1.8\right) \end{Vmatrix} = 1$$

Dynamic shear amplification factor, ACI 318-19 18.10.3.1.3

Shear Wall Design

$$M_{prratio} := \frac{M_{pr}}{M_u}$$

$$\Omega_v := \text{if}\left(\frac{h_{wcs}}{w_s} \le 1.5, 1.0, \max\left(M_{prratio}, 1.5\right)\right)$$

$$\Omega_v = 22.08$$

Shear Overstrength factor, ACI 318-19 18.10.3.1.2

$$V_{ratio} := min\left(\omega_v \cdot \Omega_v, 3\right) = 3$$

$$V_e := V_{ratio} \cdot V_u$$

$$V_{epiera} := \text{if}\left(\frac{w_s}{t} > 2.5, V_u \cdot \Omega_o, \text{“N/A”}\right)$$

Alternate Ve if pier aspect ratio met
ACI 318-19 18.10.8.1a

$$V_e := \text{if}\left(PierCheck = \text{“Pier”}, \text{if}\left(V_{epiera} = \text{“N/A”}, \frac{M_{pr}}{h_{roof}}, min\left(\frac{M_{pr}}{h_{roof}}, V_{epiera}\right)\right), V_{ratio} \cdot V_u\right)$$

$$V_e = 87.5\ \boldsymbol{k}$$

Design Shear Force,
For Segments: ACI 318-19 18.10.3.1
For Piers: ACI 318-19 18.7.6.1

Shear Steel Requirements - ACI 318-19 18.10.2.1/11.6.1

$$A_{cv} := t \cdot w_s = 2262\ \boldsymbol{in}^2$$

Gross Area of Concrete Section

$$V_{min} := \lambda \cdot \sqrt{\frac{f_c}{\boldsymbol{psi}}} \cdot \boldsymbol{psi} \cdot A_{cv}$$

Shear Strength to determine ρ_{min}

$$\rho_{min} := \text{if}\left(V_u < V_{min}, 0.002, 0.0025\right) = 0.002$$

$$A_{v\_min} := (t) \cdot (12 \cdot \boldsymbol{in}) \cdot \left(\rho_{min}\right)$$

Minimum area of Shear Steel per Foot

Determine number of curtains of shear steel - ACI 318-19 18.10.2.2

$$V_{reinf} := 2 \cdot \lambda \cdot \sqrt{\frac{f_c}{\boldsymbol{psi}}} \cdot \boldsymbol{psi} \cdot A_{cv}$$

Shear Strength to Determine Required
Number of Layers

$$Reinforcing := \text{if}\left(V_u > V_{reinf} \lor \frac{ht}{w_s} \ge 2, \text{“(2) CURTAINS REQD”}, \text{“(1) CURTAIN REQD”}\right)$$

$$Reinforcing = \text{“(1) CURTAIN REQD”}$$

Define Shear Steel:

$No_v := \#4$ — Shear Reinforcement Bar Size

$A_{sNov} := No_{v_0}$ $d_{sv} := No_{v_1}$ — Area of Shear Reinforcement Bar

$S := 12 \cdot in$ — Shear Reinforcement Spacing (12" max, typical)

$N_L := 1$ — Number of Layers **(CL = 1, EF = 2)**

$A_{vs} := N_L \cdot \frac{12 \cdot in}{S} \cdot A_{sNov}$ — Area of Shear Reinforcement per Foot

$\rho_t := \frac{A_{vs}}{(12 \cdot in) \cdot (t)}$

$A_{v\_min\_check} := \text{if}\left(A_{v\_min} \le A_{vs}, \text{"OK"}, \text{"REVISE!"}\right)$ $A_{v\_min\_check} = \text{"OK"}$

$Check := \text{if}\left(V_u > V_{reinf} \wedge N_L = 1, \text{"REVISE REINF"}, \text{"OK"}\right)$ $Check = \text{"OK"}$

Shear Wall Strength - ACI 318-19 Eq. 18.10.4.1

$$\alpha_c := \begin{cases} 3.0 & \text{if } \frac{ht}{w_s} \le 1.5 \\ 2.0 & \text{else if } \frac{ht}{w_s} \ge 2.0 \\ 2 + \frac{2.0 - \frac{ht}{w_s}}{0.5} & \text{else} \end{cases} = 2.77$$

$$\phi V_n := \phi_{vE} \cdot \left(A_{cv} \cdot \left(\alpha_c \cdot \lambda \cdot \left(\sqrt{\frac{f_c}{psi}} \cdot psi\right) + \rho_t \cdot f_y\right)\right)$$ ACI 318-19 Eq. 18.10.4.1

Define Shear Strength Max - ACI 318-19 18.10.4.4

$$\phi V_{nmax} := \phi_{vE} \cdot \left(8 \cdot \sqrt{\frac{f_c}{psi}} \cdot psi \cdot A_{cv}\right)$$ ACI 318-19 Eq. 18.10.4.4

$check := \text{if}\left(V_e \le \phi V_{nmax}, \text{"Proceed with Design"}, \text{"Revise!"}\right)$ $check = \text{"Proceed with Design"}$

$check := \text{if}\left(\phi V_n \le \phi V_{nmax}, \text{"OK"}, \text{"Vn MAX exceeded, reduce reinforcement"}\right)$ $check = \text{"OK"}$

$DCR_v := \dfrac{V_e}{min\left(\phi V_n, \phi V_{nmax}\right)}$ $DCR_v = 0.21$

$Flag_v = \text{"USE (1) LAYER /S OF \#4 BARS AT 12"OC HORIZ MIN"}$

Flexural Steel - ACI 318-19 18.10.6

$No_f := \#5$ Longitudinal Reinforcement Bar Size

$A_{sNof} := No_{f_0}$ $d_{Nof} := No_{f_1}$

$N_{Lf} := 2$ Number of Layers of Longitudinal Reinforcement (CL = 1, EF = 2) at Each End

$N_f := 22$ Number of Longitudinal Reinforcement Bars per Layer at Each End

$d_{solve} := 0.8 \cdot w_s = 20.8\ ft$ Depth of Lever Arm

Solve Block: Guess Values / Constraints

$A_{sreq} := 1\ in^2$

$\phi_f \cdot A_{sreq} \cdot f_y \cdot \left(d_{solve} - \dfrac{A_{sreq} \cdot f_y}{0.85 \cdot f_c \cdot t \cdot 2}\right) - M_u = 0$

$A_{sreq\_flex} := \text{Find}\left(A_{sreq}\right)$

$A_{sreq\_flex} = 0.69\ in^2$ Required Area of Flexural Steel

$A_{s\_long} := N_f \cdot N_{Lf} \cdot A_{sNof} = 13.64\ in^2$ Area of Flexural Steel

$DCR_f := \dfrac{A_{sreq\_flex}}{A_{s\_long}}$ $DCR_f = 0.05$

$Flag_f = \text{"USE (44) \#5 BARS EACH END MIN"}$

Transverse Reinforcement Requirements - ACI 318-19 18.10.6.5

$$A_{s\_check} := \frac{400}{60000} \cdot (0.2 \cdot w_s) \cdot t = 3.016\ in^2$$

Assumes 0.2*Panel Width for N.A. depth. Input leg into SP Column if req'd to refine results.

$$V_{check} := \lambda \cdot \sqrt{\frac{f_c}{psi}} \cdot psi \cdot A_{cv} = 143.06\ k$$

Check shear against 18.10.6.5a

$$Check_{18.10.6.5a} := \text{if}\left(V_u > V_{check}, \text{“PROVIDE HOOKED HORIZ BARS OR HOOPED ENDS”}, \text{“OK”}\right)$$

$$Check_{18.10.6.5a} = \text{“OK”}$$

$$Check_{18.10.6.5b} := \text{if}\left(A_{sreq\_flex} > A_{s\_check}, \text{“PROVIDE CONFINING HOOPS @ 5'' O.C. MAX per 18.10.6.5”}, \text{“OK”}\right)$$

$$Check_{18.10.6.5b} = \text{“OK”}$$

Basic Boundary Element Check w/ Gross Section Properties - ACI 318-19 18.10.6.3

$$I_g := \frac{(w_s)^3 \cdot t}{12} \qquad A_g := \frac{w_s \cdot t}{2} \qquad S_g := \frac{t \cdot w_s^{\ 2}}{6}$$

$$P_{u1} := \frac{P_{u1}}{2}$$

Factored Axial Load - ASCE 7-16 2.3.6 Load Combo 6

$$\sigma_u := \frac{M_u}{S_g} + \frac{P_{u1}}{A_g}$$

Combined Stress @ Extreme Fiber

$$F_c := 0.2 \cdot f_c$$

$$DCR_\sigma := \frac{\sigma_u}{F_c}$$

$$DCR_\sigma = 0.19$$

$$BasicCheck := \text{if}\left(DCR_\sigma > 1, \text{“Perform Alternate Analysis”}, \text{“Okay”}\right)$$

$$BasicCheck = \text{“Okay”}$$

If Wall does not meet basic boundary element check, provide boundary elements or perform Alternate check per ACI 318-19 18.10.6.2 ONLY if hwcs/ w_s ≥ 2**.**

$$AlternateCheck := \begin{array}{l} \text{if } \frac{h_{wcs}}{w_s} \geq 2 \wedge BasicCheck = \text{“Perform Alternate Analysis”} \\ \quad \text{“Proceed”} \\ \text{also if } BasicCheck = \text{“Okay”} \\ \quad \text{“Not Applicable”} \\ \text{else} \\ \quad \text{“Boundary Element Necessary – Alternate Check Not Allowed”} \end{array}$$

$AlternateCheck = \text{“Not Applicable”}$

Verify Section Uncracked

$y_t := \frac{w_s}{2}$ — Neutral Axis of Gross Section

$M_{cr} := \frac{7.5 \cdot \lambda \cdot \sqrt{\frac{f_c}{psi}} \cdot psi \cdot I_g}{y_t}$ — Cracking Moment - ACI 318-19 19.2.3.1 Eq 19.2.3.1, 24.2.3.5 Eq 24.2.3.5

$M_a := M_u \cdot EL_{asd\_factor}$ — Service Moment

$M_{crCheck} := \text{if}\left(M_{cr} < M_a, \text{“Cracked; If necessary adjust relative stiffness and evaluate load”}, \text{“Uncracked”}\right)$

$M_{crCheck} = \text{“Uncracked”}$

Slab Shear Dowel Bars - Shear Friction per ACI 318-19 22.9.4

$\mu := 0.6$ — Coefficient of Friction - ACI 318-19 22.9.4.2

$No_{dowel} := \#4$ — Dowel Reinforcement Bar Size No.

$A_{s\_Nodowel} := No_{dowel}$

$S_{dowel} := 24 \cdot in$ — Dowel Bar Spacing

$A_{dowel} := \frac{A_{s\_Nodowel}}{S_{dowel}}$ — Dowel Bar Area per Foot

$A_{vf} := A_{dowel} \cdot w_s = 2.6\ in^2$ — Area of Shear Friction Steel Across Panel

$\phi V_{vf\_n} := \phi_{vE} \cdot \left(\mu \cdot A_{vf} \cdot f_y\right)$ — Design Shear Capacity - ACI 318-19 22.9.4.2

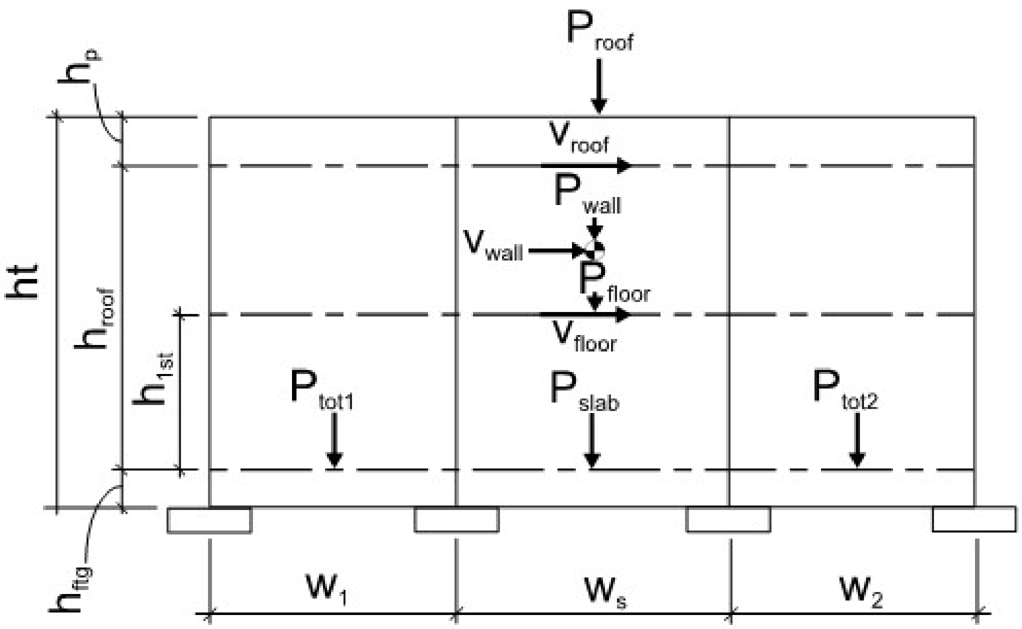
$V_u := \rho \cdot V_{tot} = 29.2\ k$ — Ultimate Panel Shear ($\rho\, Q_e$)

$$DCR_{vf} := \frac{V_u}{\phi V_{vf\_n}}$$

$$DCR_{vf} = 0.52$$

ACI 318-19 Solid Panel - Panel 49 W/ mezzanine - South

In-Plane Shear Solid Panel:



Shear Panel (w_s)

$w_s := 28\ ft$	Panel Width
$h_p := 7.5 \cdot ft$	Parapet Height
$h_{roof} := 36\ ft$	Height of Roof Above FF
$h_{1st} := 12 \cdot ft$	Height of Intermediate Floor Above FF
$h_{ftg} := 1.5 \cdot ft$	FF to T/FTG Height
$ht := h_p + h_{roof} + h_{ftg} = 45\ ft$	Total Panel Height (T/FTG to T/PARAPET)
$t_p := 8 \cdot in$	Panel Thickness
$r_v := 0.75 \cdot in$	Reveal Depth
$t := t_p - r_v = 7.25\ in$	Effective Panel Thickness
$Wt_p := w_c \cdot t_p$	Panel Weight (psf)

Shear Wall Design

Adjacent Panel (w_1)

$w_1 := w_s$ — Adjacent Panel Width

$h_{p1} := h_p$ — Parapet Height

$h_{roof1} := h_{roof}$ — Height of Roof Above FF

$h_{ftg1} := h_{ftg}$ — FF to T/FTG Height

$ht_1 := h_{p1} + h_{roof1} + h_{ftg1} = 45\ ft$ — Total Panel Ht. (T/FTG to T/PARAPET)

$t_{p1} := t_p$ — Panel Thickness

$Ao_1 := 0 \cdot ft \cdot 0 \cdot ft$ — Area of Openings

Adjacent Panel (w_2)

$w_2 := w_s$ — Adjacent Panel Width

$h_{p2} := h_p$ — Parapet Height

$h_{roof2} := h_{roof}$ — Height of Roof Above FF

$h_{ftg2} := h_{ftg}$ — FF to T/FTG Height

$ht_2 := h_{p2} + h_{roof2} + h_{ftg2} = 45\ ft$ — Total Panel Ht. (T/FTG to T/PARAPET)

$t_{p2} := t_p$ — Panel Thickness

$Ao_2 := 0 \cdot ft \cdot 0 \cdot ft$ — Area of Openings

Loading:

Seismic (1.0 Q_e)

$V_{roof} := 12.4\ k$ — Seismic Shear @ Roof- ρ not included

$V_{floor} := 10 \cdot k$ — Seismic Shear @ Floor

$V_{wall} := \left(ht - h_{ftg}\right) \cdot w_s \cdot Wt_p \cdot C_s$ — Seismic Shear due to Wall Panel Mass

$V_{tot} := V_{roof} + V_{wall} + V_{floor} = 42.9\ k$ — Total Seismic Shear

$M_{ot\_roof} := V_{roof} \cdot h_{roof}$ — Overturning Moment due to Roof Shear

$M_{ot\_floor} := V_{floor} \cdot h_{1st}$ — Overturning Moment due to Floor Shear

$M_{ot\_wall} := V_{wall} \cdot \dfrac{\left(ht - h_{ftg}\right)}{2}$ — Overturning Moment due to Panel Mass

$M_{ot} := M_{ot\_roof} + M_{ot\_floor} + M_{ot\_wall} = 1013\ k \cdot ft$ — Total Overturning Moment @ Wall Panel

Gravity:

Shear Panel (w_s):

$l_{roof\_trib} := \dfrac{59.33}{2} \cdot ft$ — Length of Roof Tributary to Panel

$l_{floor\_trib} := \dfrac{20}{2} \cdot ft$ — Length of Floor Tributary to Panel

Shear Wall Design

$SL := 0.4\ klf$ Roof snow load tributary to panel

$LL := 0.65\ klf$ Floor live load tributary to panel

$P_{roof} := DL_{sei} \cdot l_{roof\_trib} \cdot w_s$ Roof DL

$P_{floor} := DL_{flr} \cdot l_{floor\_trib} \cdot w_s$ Floor DL

$P_{wall} := w_s \cdot ht \cdot t_p \cdot w_c$ Wall Panel DL

$P_{slab} := w_c \cdot t_{SOG} \cdot 8 \cdot ft \cdot w_s$ Slab-on-Grade DL (assumes 8' width of slab over width of panel available to resist OT)

$P_{tot} := P_{roof} + P_{floor} + P_{wall} = 141.8\ k$ Total DL (without slab @ wall considered)

Panel (w_1):

$P_{roof1} := P_{roof} \cdot \left(\frac{w_1}{w_s}\right)$ Roof DL

$P_{floor1} := P_{floor} \cdot \left(\frac{w_1}{w_s}\right)$ Floor DL

$P_{wall1} := \left(w_1 \cdot ht_1 - Ao_1\right) \cdot t_{p1} \cdot w_c$ Wall Panel DL

$P_{slab1} := P_{slab} \cdot \left(\frac{w_1}{w_s}\right)$ Slab-on-Grade DL

$P_{tot1} := P_{roof1} + P_{floor1} + P_{wall1} + P_{slab1} = 161.4\ k$ Total DL (including slab weight)

Panel (w_2):

$P_{roof2} := P_{roof} \cdot \left(\frac{w_2}{w_s}\right)$ Roof DL

$P_{floor2} := P_{floor} \cdot \frac{w_2}{w_s}$ Floor DL

$P_{wall2} := \left(w_2 \cdot ht_2 - Ao_2\right) \cdot t_{p2} \cdot w_c$ Wall Panel DL

$P_{slab2} := P_{slab} \cdot \left(\frac{w_2}{w_s}\right)$ Slab-on-Grade DL

$P_{tot2} := P_{roof2} + P_{floor2} + P_{wall2} + P_{slab2} = 161.4\ k$ Total DL (including slab weight)

Resisting Moment:

$M_{resist} := \left(P_{tot} + P_{slab}\right) \cdot \frac{w_s}{2} = 2259\ k \cdot ft$ Resisting Moment at Shear Panel

Load Combinations:

ASCE 7-16 2.3.6

LRFD 6: (1.2+0.2Sds)D+1.0ρ Q_e + L + 0.2S

LRFD 7: (0.9 - 0.2Sds)D + 1.0ρ Q_e (Resisting Moment of Slab-on-Grade)

IBC 2021 1605.2 - Foundations

ASD 16-5: D + L + S + E/1.4 (Foundation Bearing)

ASD 16-6: 0.9D + E/1.4 (Foundation Uplift/Weight. Ev = 0 for fdn size, Ev included for hold downs)

Panel Hold Down - IBC 2021 1605.2 Alternative Load Combination ASD 16-6

$$M_{ot\_IBC} := \rho \cdot \frac{M_{ot}}{1.4}$$ ASD Overturning Moment

$$M_{res\_IBC} := \left(0.9 - \frac{0.2 \cdot S_{ds}}{1.4}\right) \cdot M_{resist}$$ ASD Resisting Moment

$$No_{hd} := \#6$$ Rebar Size No.

$$N_{hd} := 2$$ Number of Bars in Hold Down

$$L_{hd} := \text{if}\left(No_{hd_0} = 0.31\ in^2, L_{HD5_{N_{hd}}}, L_{HD6_{N_{hd}}}\right) = 12\ in$$

$$A_{shd} := No_{hd_0} \cdot N_{hd}$$ Area of Steel in Hold Down

$$hd_{check} := M_{ot\_IBC} - M_{res\_IBC}$$ Holdown check

$$T_{hd} := \frac{M_{ot} - (0.9 - 0.2\ S_{ds}) \cdot M_{resist}}{w_s - (1 \cdot ft + 0.5 \cdot L_{hd})} = -24.1\ k$$ LRFD Tension at Hold Down

$$A_{shdreq} := \frac{T_{hd}}{\phi_t \cdot f_y} = -0.45\ in^2$$ Required Area of Steel

$$DCR_{hd} := \frac{A_{shdreq}}{A_{shd}}$$

$$DCR_{hd} = -0.51$$

$$hd\_Results = \text{"NO HOLDOWNS REQD"}$$

Pad Footing Uplift and Seismic Bearing Checks

$l_f := \frac{w_s}{2}$ Length of Footing

$w_f := 4.5\ ft$ Width of Footing

$d_f := 1\ ft$ Depth of Footing (Use 2'-0" MIN @ hold down, see footing calcs)

$P_{ftg} := w_c \cdot l_f \cdot w_f \cdot d_f$ Footing Weight

Required Footing Weight - Load Combination ASD 16-6

$$M_{res\_uplift} := M_{res\_IBC} + \left(0.9 \cdot \left(min\left(\frac{P_{tot1}}{2}, \frac{P_{tot2}}{2}\right) \cdot w_s\right)\right)$$

$M_{res\_uplift} = 3795\ k \cdot ft$ Factored Resisting Moment (Including contribution of slab & adjacent panel)

$$Wt_{ftg\_uplift} := \frac{\left(M_{ot\_IBC} - M_{res\_uplift}\right)}{w_s - \left(1\ ft + 0.5 \cdot L_{hd}\right)}$$

$Wt_{ftg\_uplift} = -115.9\ k$ Required Footing Weight

$flag_w := \text{if}\left(Wt_{ftg\_uplift} \le P_{ftg}, \text{“OK”}, \text{“NG!!!”}\right)$ $flag_w = \text{“OK”}$

Wall Shear Design

Factored Wall Axial Forces for M_{pr} Calculation

$$P_{u1} := \left(1.2 + 0.2 \cdot S_{ds}\right) \cdot P_{tot} + \left(0.2\ SL + LL\right)\ w_s = 214\ \boldsymbol{k}$$ ASCE 7-16 12.2.3.6 Eq. 6

$$P_{u2} := \left(0.9 - 0.2\ S_{ds}\right)\ P_{tot} = 103.69\ \boldsymbol{k}$$ ASCE 7-16 12.2.3.6 Eq. 7

Probable Moment and Moment Demand

$$M_{pr} := 20000\ \boldsymbol{k} \cdot \boldsymbol{ft}$$ Largest probable flexural strength of the section from SP Column assuming tensile stress of 1.25*Fy and Φ = 1.0

$$M_u := \rho \cdot M_{ot}$$

$$V_u := \rho \cdot V_{tot} = 42.9\ \boldsymbol{k}$$ Ultimate Panel Shear (ρ Q_e)

Pier Check - ACI 318-19 Table R18.10.1

If wall section is a pier, requirements of ACI 318-19 18.10.8 or 18.14 must be met.

$$Check_1 := \text{if}\left(\frac{w_s}{t_p} \le 6, \text{"Pier"}, \text{"Segment"}\right)$$

$$PierCheck := \text{if}\left(\frac{h_{roof}}{w_s} \ge 2, Check_1, \text{"Segment"}\right)$$ $$PierCheck = \text{"Segment"}$$

Design Shear Force - ACI 318-19 18.10.3

$$h_{wcs} := h_{roof} + h_p$$ Height of Wall above Critical Section

$$stories_{wcs} := 1$$ Number of Stories above Critical Section

$$n_s := \max\left(stories_{wcs}, 0.007 \cdot \frac{h_{wcs}}{\boldsymbol{in}}\right)$$

$$\omega_v := \begin{array}{l} \text{if } \dfrac{h_{wcs}}{w_s} < 2 \\ \quad 1.0 \\ \text{else if } n_s \le 6.0 \\ \quad 0.9 + \dfrac{n_s}{10} \\ \text{else} \\ \quad min\left(1.3 + \dfrac{n_s}{30}, 1.8\right) \end{array} = 1$$ Dynamic shear amplification factor, ACI 318-19 18.10.3.1.3

$$M_{prratio} := \frac{M_{pr}}{M_u}$$

$$\Omega_v := \text{if}\left(\frac{h_{wcs}}{w_s} \le 1.5, 1.0, \max\left(M_{prratio}, 1.5\right)\right)$$

$$\Omega_v = 19.74$$

Shear Overstrength factor, ACI 318-19 18.10.3.1.2

$$V_{ratio} := min\left(\omega_v \cdot \Omega_v, 3\right) = 3$$

$$V_e := V_{ratio} \cdot V_u$$

$$V_{epiera} := \text{if}\left(\frac{w_s}{t} > 2.5, V_u \cdot \Omega_o, \text{“N/A”}\right)$$

Alternate Ve if pier aspect ratio met
ACI 318-19 18.10.8.1a

$$V_e := \text{if}\left(PierCheck = \text{“Pier”}, \text{if}\left(V_{epiera} = \text{“N/A”}, \frac{M_{pr}}{h_{roof}}, min\left(\frac{M_{pr}}{h_{roof}}, V_{epiera}\right)\right), V_{ratio} \cdot V_u\right)$$

$$V_e = 128.8\ k$$

Design Shear Force,
For Segments: ACI 318-19 18.10.3.1
For Piers: ACI 318-19 18.7.6.1

Shear Steel Requirements - ACI 318-19 18.10.2.1/11.6.1

$$A_{cv} := t \cdot w_s = 2436\ in^2$$

Gross Area of Concrete Section

$$V_{min} := \lambda \cdot \sqrt{\frac{f_c}{psi}} \cdot psi \cdot A_{cv}$$

Shear Strength to determine ρ_{min}

$$\rho_{min} := \text{if}\left(V_u < V_{min}, 0.002, 0.0025\right) = 0.002$$

$$A_{v\_min} := (t) \cdot (12 \cdot in) \cdot \left(\rho_{min}\right)$$

Minimum area of Shear Steel per Foot

Determine number of curtains of shear steel - ACI 318-19 18.10.2.2

$$V_{reinf} := 2 \cdot \lambda \cdot \sqrt{\frac{f_c}{psi}} \cdot psi \cdot A_{cv}$$

Shear Strength to Determine Required
Number of Layers

$$Reinforcing := \text{if}\left(V_u > V_{reinf} \lor \frac{ht}{w_s} \ge 2, \text{“(2) CURTAINS REQD”}, \text{“(1) CURTAIN REQD”}\right)$$

$$Reinforcing = \text{“(1) CURTAIN REQD”}$$

Define Shear Steel:

$No_v := \#4$ — Shear Reinforcement Bar Size

$A_{sNov} := No_{v_0}$ $\quad d_{sv} := No_{v_1}$ — Area of Shear Reinforcement Bar

$S := 12 \cdot in$ — Shear Reinforcement Spacing (12" max, typical)

$N_L := 1$ — Number of Layers **(CL = 1, EF = 2)**

$A_{vs} := N_L \cdot \frac{12 \cdot in}{S} \cdot A_{sNov}$ — Area of Shear Reinforcement per Foot

$\rho_t := \frac{A_{vs}}{(12 \cdot in) \cdot (t)}$

$A_{v\_min\_check} := \text{if}\left(A_{v\_min} \le A_{vs}, \text{“OK”}, \text{“REVISE!”}\right)$ $\quad A_{v\_min\_check} = \text{“OK”}$

$Check := \text{if}\left(V_u > V_{reinf} \wedge N_L = 1, \text{“REVISE REINF”}, \text{“OK”}\right)$ $\quad Check = \text{“OK”}$

Shear Wall Strength - ACI 318-19 Eq. 18.10.4.1

$$\alpha_c := \begin{array}{l} \text{if } \frac{ht}{w_s} \le 1.5 \\ \quad 3.0 \\ \text{else if } \frac{ht}{w_s} \ge 2.0 \\ \quad 2.0 \\ \text{else} \\ \quad 2 + \frac{2.0 - \frac{ht}{w_s}}{0.5} \end{array} = 2.79$$

$\phi V_n := \phi_{vE} \cdot \left(A_{cv} \cdot \left(\alpha_c \cdot \lambda \cdot \left(\sqrt{\frac{f_c}{psi}} \cdot psi\right) + \rho_t \cdot f_y\right)\right)$ — ACI 318-19 Eq. 18.10.4.1

Define Shear Strength Max - ACI 318-19 18.10.4.4

$\phi V_{nmax} := \phi_{vE} \cdot \left(8 \cdot \sqrt{\frac{f_c}{psi}} \cdot psi \cdot A_{cv}\right)$ — ACI 318-19 Eq. 18.10.4.4

$check := \text{if}\left(V_e \le \phi V_{nmax}, \text{“Proceed with Design”}, \text{“Revise!”}\right)$ $\quad check = \text{“Proceed with Design”}$

$check := \text{if}\left(\phi V_n \le \phi V_{nmax}, \text{"OK"}, \text{"Vn MAX exceeded, reduce reinforcement"}\right)$ $check = \text{"OK"}$

$DCR_v := \dfrac{V_e}{min\left(\phi V_n, \phi V_{nmax}\right)}$ $DCR_v = 0.28$

$Flag_v = \text{"USE (1) LAYER /S OF \#4 BARS AT 12"OC HORIZ MIN"}$

Flexural Steel - ACI 318-19 18.10.6

$No_f := \#5$ Longitudinal Reinforcement Bar Size

$A_{sNof} := No_{f_0}$ $d_{Nof} := No_{f_1}$

$N_{Lf} := 2$ Number of Layers of Longitudinal Reinforcement (CL = 1, EF = 2) at Each End

$N_f := 28$ Number of Longitudinal Reinforcement Bars per Layer at Each End

$d_{solve} := 0.8 \cdot w_s = 22.4\ ft$ Depth of Lever Arm

Solve Block — Guess Values / Constraints

$A_{sreq} := 1\ in^2$

$\phi_f \cdot A_{sreq} \cdot f_y \cdot \left(d_{solve} - \dfrac{A_{sreq} \cdot f_y}{0.85 \cdot f_c \cdot t \cdot 2}\right) - M_u = 0$

$A_{sreq\_flex} := \text{Find}\left(A_{sreq}\right)$

$A_{sreq\_flex} = 0.84\ in^2$ Required Area of Flexural Steel

$A_{s\_long} := N_f \cdot N_{Lf} \cdot A_{sNof} = 17.36\ in^2$ Area of Flexural Steel

$DCR_f := \dfrac{A_{sreq\_flex}}{A_{s\_long}}$ $DCR_f = 0.05$

$Flag_f = \text{"USE (56) \#5 BARS EACH END MIN"}$

Transverse Reinforcement Requirements - ACI 318-19 18.10.6.5

$$A_{s\_check} := \frac{400}{60000} \cdot (0.2 \cdot w_s) \cdot t = 3.248\ in^2$$

Assumes 0.2*Panel Width for N.A. depth. Input leg into SP Column if req'd to refine results.

$$V_{check} := \lambda \cdot \sqrt{\frac{f_c}{psi}} \cdot psi \cdot A_{cv} = 154.07\ k$$

Check shear against 18.10.6.5a

$$Check_{18.10.6.5a} := \text{if}\left(V_u > V_{check}, \text{"PROVIDE HOOKED HORIZ BARS OR HOOPED ENDS"}, \text{"OK"}\right)$$

$$Check_{18.10.6.5a} = \text{"OK"}$$

$$Check_{18.10.6.5b} := \text{if}\left(A_{sreq\_flex} > A_{s\_check}, \text{"PROVIDE CONFINING HOOPS @ 5'' O.C. MAX per 18.10.6.5"}, \text{"OK"}\right)$$

$$Check_{18.10.6.5b} = \text{"OK"}$$

Basic Boundary Element Check w/ Gross Section Properties - ACI 318-19 18.10.6.3

$$I_g := \frac{(w_s)^3 \cdot t}{12} \qquad A_g := \frac{w_s \cdot t}{2} \qquad S_g := \frac{t \cdot {w_s}^2}{6}$$

$$P_{u1} := \frac{P_{u1}}{2}$$

Factored Axial Load - ASCE 7-16 2.3.6
Load Combo 6

$$\sigma_u := \frac{M_u}{S_g} + \frac{P_{u1}}{A_g}$$

Combined Stress @ Extreme Fiber

$$F_c := 0.2 \cdot f_c$$

$$DCR_\sigma := \frac{\sigma_u}{F_c} \qquad DCR_\sigma = 0.22$$

$$BasicCheck := \text{if}\left(DCR_\sigma > 1, \text{"Perform Alternate Analysis"}, \text{"Okay"}\right) \qquad BasicCheck = \text{"Okay"}$$

If Wall does not meet basic boundary element check, provide boundary elements or perform Alternate check per ACI 318-19 18.10.6.2 <u>ONLY</u> if hwcs/ w_s ≥ 2**.**

$$AlternateCheck := \left\| \text{if } \frac{h_{wcs}}{w_s} \geq 2 \wedge BasicCheck = \text{"Perform Alternate Analysis"} \right.$$

"Proceed"
also if $BasicCheck = $ "Okay"
"Not Applicable"
else
"Boundary Element Necessary – Alternate Check Not Allowed"

$AlternateCheck = $ "Not Applicable"

Verify Section Uncracked

$y_t := \frac{w_s}{2}$ — Neutral Axis of Gross Section

$M_{cr} := \frac{7.5 \cdot \lambda \cdot \sqrt{\frac{f_c}{psi}} \cdot psi \cdot I_g}{y_t}$ — Cracking Moment - ACI 318-19 19.2.3.1 Eq 19.2.3.1, 24.2.3.5 Eq 24.2.3.5

$M_a := M_u \cdot EL_{asd\_factor}$ — Service Moment

$M_{crCheck} :=$ if $\left(M_{cr} < M_a\right.$, "Cracked; If necessary adjust relative stiffness and evaluate load", "Uncracked"$\left.\right)$

$M_{crCheck} = $ "Uncracked"

Slab Shear Dowel Bars - Shear Friction per ACI 318-19 22.9.4

$\mu := 0.6$ — Coefficient of Friction - ACI 318-19 22.9.4.2

$No_{dowel} := \#4$ — Dowel Reinforcement Bar Size No.

$A_{s\_Nodowel} := No_{dowel}$

$S_{dowel} := 24 \cdot in$ — Dowel Bar Spacing

$A_{dowel} := \frac{A_{s\_Nodowel}}{S_{dowel}}$ — Dowel Bar Area per Foot

$A_{vf} := A_{dowel} \cdot w_s = 2.8\ in^2$ — Area of Shear Friction Steel Across Panel

$\phi V_{vf\_n} := \phi_{vE} \cdot \left(\mu \cdot A_{vf} \cdot f_y\right)$ — Design Shear Capacity - ACI 318-19 22.9.4.2

$V_u := \rho \cdot V_{tot} = 42.9\ k$ — Ultimate Panel Shear (ρQ_e)

$DCR_{vf} := \frac{V_u}{\phi V_{vf\_n}}$

$DCR_{vf} = 0.71$

Shear Wall Design

Shear Wall Design

Protected Area

Purpose Statement: The purpose of this calculation is in-plane design of concrete tilt panels as special reinforced shear walls.

Referenced Standards:

IBC 2021
ASCE 7-16
ACI 318-19
Amrhein/Yellow Book method
NEHRP Tech Brief 6

Inputs
Verify
Outputs

PROJECT PARAMETERS

$f_c := 4000 \cdot psi$	Concrete Strength
$q_{all} := 2500 \cdot psf$	Allowable Soil Pressure
$\gamma_{temp} := \frac{4}{3}$	Short-term Loading Increase for Soil
$q_{sei} := \gamma_{temp} \cdot q_{all}$	Short-term Allowable Soil Pressure
$DL_{sei} := 10.9 \cdot psf$	Roof DL
$DL_{flr} := 65 \cdot psf$	Intermediate Floor DL
$t_{SOG} := 7 \cdot in$	Slab-on-Grade Thickness
$\phi_{vE} := 0.6$	Seismic shear strength reduction factor - ACI 318-19 21.2.4.1
$\phi_f := 0.9$	Flexural Strength Reduction Factor
$\phi_t := 0.9$	Tensile Strength Reduction Factor
$EL_{asd\_factor} := 0.7$	ASD Factor
$f_y := 60 \cdot ksi$	Reinforcing Yield Strength
$E_s := 29000 \cdot ksi$	Steel Modulus of Elasticity
$w_c := 150 \cdot pcf$	Unit Weight of Concrete
$\lambda :=$ Normal Weight	Concrete weight modification factor - ACI 318-19 Table 19.2.4.1 (b)

Shear Wall Design

$S_{ds} := 0.843$ Seismic Coefficient

$\rho := 1.0$ Redundancy Factor - ASCE 7-16 12.3.4

$I_e := 1.0$ Seismic Importance Factor - ASCE 7-16 Table 1.5-2

Design Coefficients and Factors for Special Reinforced Concrete Shear Walls - ASCE 7-16 12.2

$R := 5$ Response Modification Factor

$C_d := 5$ Deflection Amplification Factor

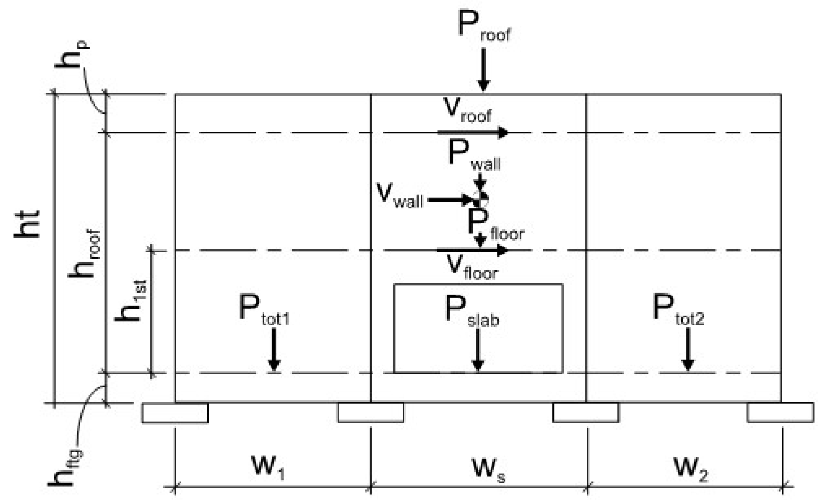
$\Omega_o := 2.0$ Special Concrete Shear Wall Overstrength Factor

$$C_s := \frac{S_{ds}}{\left(\frac{R}{I_e}\right)} = 0.1686$$

Seismic Coefficient (C_s) - ASCE 7-16 12.8.1

Shear Wall Design

318-19 Panel w/ (1) Opening - Panel 2 - North



Shear Panel Input (w_s)

$w_s := 26 \cdot ft$	Panel Width
$h_p := 4.5 \cdot ft$	Parapet Height
$h_{roof} := 36 \cdot ft$	Height of Roof Above FF
$h_{1st} := 0 \cdot ft$	Height of Floor Above FF
$h_{ftg} := 1.5 \cdot ft$	FF to T/FTG Height
$ht := h_p + h_{roof} + h_{ftg} = 42\ ft$	Total Panel Height (T/FTG to T/PARAPET)
$ht_o := 12 \cdot ft$	Opening Height
$w_o := 9 \cdot ft$	Opening Width
$lega := 8.5\ ft$	Leg A Width
$legb := w_s - w_o - lega = 8.5\ ft$	Leg B Width
$t_p := 8 \cdot in$	Panel Thickness
$r_v := 0.75 \cdot in$	Reveal Depth
$t := t_p - r_v = 7.3\ in$	Effective Panel Thickness
$Wt_p := w_c \cdot t_p$	Panel Wt. (psf)

Relative Panel Leg Stiffnesses (w_s)

Amrhein/Yellow book method

$BC_{lega} :=$ Boundary Condition: Fixed−Fixed

$BC_{legb} :=$ Boundary Condition: Fixed−Fixed

Leg A & B boundary conditions
BC_{leg} = 1 for fixed-fixed
BC_{leg} = 4 for fixed-pinned

$hd_{ratioa} := \frac{ht_o}{lega} = 1.41$ Leg A aspect ratio

$hd_{ratiob} := \frac{ht_o}{legb} = 1.41$ Leg B aspect ratio

$P_k := 100000\ lb$ $t_k := 1\ in$ $E_k := 1000000\ psi$

$$\Delta_{ka} := \frac{P_k}{E_k \cdot t_k} \cdot \left(BC_{lega} \cdot \left(hd_{ratioa}\right)^3 + 3 \cdot \left(hd_{ratioa}\right)\right)$$

$$\Delta_{kb} := \frac{P_k}{E_k \cdot t_k} \cdot \left(BC_{legb} \cdot \left(hd_{ratiob}\right)^3 + 3 \cdot \left(hd_{ratiob}\right)\right)$$

$R_{ka} := \frac{1}{\Delta_{ka}}$ $R_{kb} := \frac{1}{\Delta_{kb}}$

$k_a := \frac{R_{ka}}{R_{ka} + R_{kb}} = 0.5$ Leg A Relative Stiffness

$k_b := \frac{R_{kb}}{R_{ka} + R_{kb}} = 0.5$ Leg B Relative Stiffness

$Stiffness := \text{if}\left(\left|1.0 - \left(k_a + k_b\right)\right| > 0.001, \text{"CHECK"}, \text{"OK"}\right)$ $Stiffness = \text{"OK"}$

Adjacent Panel Input (w_1) Adjacent Panel Input (w_2)

$w_1 := w_s$	$w_2 := w_s$	Adjacent Panel Width
$h_{p1} := h_p$	$h_{p2} := h_p$	Parapet Height
$h_{roof1} := h_{roof}$	$h_{roof2} := h_{roof}$	Height of Roof Above FF
$h_{ftg1} := h_{ftg}$	$h_{ftg2} := h_{ftg}$	FF to T/FTG Height
$ht_1 := h_{p1} + h_{roof1} + h_{ftg1} = 42\ ft$	$ht_2 := h_{p2} + h_{roof2} + h_{ftg2} = 42\ ft$	Total Panel Height (T/FTG to T/ PARAPET)

Shear Wall Design

$t_{p1} := t_p$	$t_{p2} := t_p$	Panel Thickness
$Ao_1 := 12 \cdot ft \cdot 9 \cdot ft$	$Ao_2 := 12 \cdot ft \cdot 9 \cdot ft$	Area of Openings

Loading:

Seismic (1.0 Q_e)

$V_{roof} := 18.6 \cdot k$ — Seismic Shear @ Roof - ρ not included

$V_{floor} := 0 \cdot k$ — Seismic Shear @ Floor

$V_{wall} := \left(\left(ht - h_{ftg}\right) \cdot w_s - ht_o \cdot w_o\right) \cdot Wt_p \cdot C_s$ — Seismic Shear due to Wall Panel Mass

$V_{tot} := V_{roof} + V_{wall} + V_{floor} = 34.5\ k$ — Total Seismic Shear

$M_{ot\_roof} := V_{roof} \cdot h_{roof}$ — Overturning Moment due to Roof Shear

$M_{ot\_floor} := V_{floor} \cdot h_{1st}$ — Overturning Moment due to Floor Shear

$M_{ot\_wall} := V_{wall} \cdot \frac{\left(ht - h_{ftg}\right)}{2}$ — Overturning Moment Due to Panel Mass

$M_{ot} := M_{ot\_roof} + M_{ot\_floor} + M_{ot\_wall} = 992.2\ k \cdot ft$ — Total Overturning Moment @ Wall Panel

Gravity:

Shear Panel (w_s):

$l_{roof\_trib} := \frac{60}{2} \cdot ft$ — Length of Roof Tributary to Panel

$l_{floor\_trib} := \frac{0}{2} \cdot ft$ — Length of Floor Tributary to Panel

$SL := 1.24\ klf$ — Roof snow load tributary to panel

$LL := 0.0\ klf$ — Floor live load tributary to panel

$P_{roof} := DL_{sei} \cdot l_{roof\_trib} \cdot w_s$ — Roof DL

$P_{floor} := DL_{flr} \cdot l_{floor\_trib} \cdot w_s$ — Floor DL

$P_{wall} := \left(ht \cdot w_s - ht_o \cdot w_o\right) \cdot Wt_p$ — Wall Panel DL

$P_{slab} := w_c \cdot t_{SOG} \cdot 8 \cdot ft \cdot w_s$ — Slab-on-Grade DL (assumes 8' width of slab over width of panel available to resist OT)

Shear Wall Design

$P_{tot} := P_{roof} + P_{floor} + P_{wall} = 106.9\ k$ Total DL (without slab @ wall considered)

Panel (w_1):

$P_{roof1} := P_{roof}$ Roof DL

$P_{floor1} := P_{floor}$ Floor DL

$P_{wall1} := (w_1 \cdot ht_1 - Ao_1) \cdot t_{p1} \cdot w_c$ Wall Panel DL

$P_{slab1} := P_{slab} \cdot \left(\dfrac{w_1}{w_s}\right)$ Slab-on-Grade DL

$P_{tot1} := P_{roof1} + P_{floor1} + P_{wall1} + P_{slab1} = 125.1\ k$ Total DL (including slab weight)

Panel (w_2):

$P_{roof2} := P_{roof}$ Roof DL

$P_{floor2} := P_{floor}$ Floor DL

$P_{wall2} := (w_2 \cdot ht_2 - Ao_2) \cdot t_{p2} \cdot w_c$ Wall Panel DL

$P_{slab2} := P_{slab} \cdot \left(\dfrac{w_2}{w_s}\right)$ Slab-on-Grade DL

$P_{tot2} := P_{roof2} + P_{floor2} + P_{wall2} + P_{slab2} = 125.1\ k$ Total DL (including slab weight)

Resisting Moment:

$M_{resist} := (P_{tot} + P_{slab}) \cdot \dfrac{w_s}{2} = 1626.3\ k \cdot ft$ Resisting Moment at Shear Panel

Load Combinations:

ASCE 7-16 2.3.6

LRFD 6: (1.2+0.2Sds)D+1.0ρ Q_e + L + 0.2S

LRFD 7: (0.9 - 0.2Sds)D + 1.0ρ Q_e (Resisting Moment of Slab-on-Grade)

IBC 2021 1605.2 - Foundations

ASD 16-5: D + L + S + E/1.4 (Foundation Bearing)

ASD 16-6: 0.9D + E/1.4 (Foundation Uplift/Weight. Ev = 0 for fdn size, Ev included for holdowns)

Panel Hold Down - IBC 2021 1605.2 - Alternative Load Combination ASD 16-6

$M_{ot\_IBC} := \rho \cdot \dfrac{M_{ot}}{1.4}$ ASD Overturning Moment

$M_{res\_IBC} := \left(0.9 - \dfrac{0.2 \cdot S_{ds}}{1.4}\right) \cdot M_{resist}$ ASD Resisting Moment

$No_{hd} := \#5$ Rebar Size No.

$N_{hd} := 2$ Number of Bars in Hold Down

$L_{hd} := \text{if}\left(No_{hd_0} = 0.31\ in^2, L_{HD5_{N_{hd}}}, L_{HD6_{N_{hd}}}\right) = 8\ in$ Length of Hold Down

$A_{shd} := No_{hd_0} \cdot N_{hd}$ Area of Steel in Hold Down

$T_{hd} := \dfrac{M_{ot\_IBC} - M_{res\_IBC}}{w_s - (1 \cdot ft + 0.5 \cdot L_{hd})} = -22.7\ k$ ASD Tension at Hold Down

$A_{shdreq} := \dfrac{\left(\dfrac{T_{hd}}{EL_{asd\_factor}}\right)}{\phi_t \cdot f_y} = -0.6\ in^2$ Required Area of Steel

$DCR_{hd} := \dfrac{A_{shdreq}}{A_{shd}}$ $DCR_{hd} = -1$

$hd\_Results =$ “NO HOLDOWNS REQD”

Footing Uplift and Seismic Bearing Checks

$l_f := 10.5\ ft$ — Length of Footing

$w_f := 2.5 \cdot ft$ — Width of Footing

$d_f := 1 \cdot ft$ — Depth of Footing (Use 2'-0" Min @ hold down, see footing calcs)

$P_{ftg} := w_c \cdot l_f \cdot w_f \cdot d_f$ — Footing Weight

Required Footing Weight - Load Combination ASD 16-6

$$M_{res\_uplift} := M_{res\_IBC} + \left(0.9 \cdot \left(min\left(\frac{P_{tot1}}{2}, \frac{P_{tot2}}{2}\right) \cdot w_s\right)\right)$$

Factored Resisting Moment (Including contribution of slab & adjacent panel)

$M_{res\_uplift} = 2731.53\ k \cdot ft$

$$Wt_{ftg\_uplift} := \frac{\left(M_{ot\_IBC} - M_{res\_uplift}\right)}{w_s - \left(1\ ft + 0.5 \cdot L_{hd}\right)}$$

Required Footing Weight

$Wt_{ftg\_uplift} = -82\ k$

$flag_w := \text{if}\left(Wt_{ftg\_uplift} \le P_{ftg}, \text{“OK”}, \text{“NG!!!”}\right)$ $\quad flag_w = \text{“OK”}$

------------------------ VERIFY SECTION BELOW APPLIES------------------------

Maximum Seismic Bearing, PAD Footing Condition - Load Combination ASD 16-6

$$C_{ot\_comp} := \frac{M_{ot\_IBC}}{w_s}$$

Factored Axial Compression Force due to Overturning

$P_{grav\_comp} := 0.5 \cdot \left(P_{tot}\right)$

$P_{grav\_comp} = 53.45\ k$ — Factored Axial Gravity Force for 1/2 of panel

$A_{ftg} := l_f \cdot w_f$ — Footing Area

$$A_{ftg\_reqd} := \frac{C_{ot\_comp} + P_{grav\_comp}}{q_{sei}} = 24.21\ ft^2$$

Required Footing Area

$flag_A := \text{if}\left(A_{ftg\_reqd} \le A_{ftg}, \text{“OK”}, \text{“NG!!!”}\right)$ $\quad flag_A = \text{“OK”}$

$$q_s := \frac{C_{ot\_comp} + P_{grav\_comp}}{A_{ftg}} = 3074.68\ psf$$

Seismic Bearing Pressure

$flag_B := \text{if}\left(q_s < q_{sei}, \text{“OK”}, \text{“NG!!!”}\right)$ $\quad flag_B = \text{“OK”}$

Check Wall Segment Above/Below Opening

Wall Shear Design

$P_{u1c} := (1.2 + 0.2 \cdot S_{ds}) \cdot P_{tot} + (0.2\ SL + LL)\ w_s = 153\ k$

Factored Wall Axial Forces (for Mpr calculation) ASCE 7-16 12.2.3.6 Eq. 6

$P_{u2c} := (0.9 - 0.2\ S_{ds}) \cdot P_{tot} = 78.19\ k$

ASCE 7-16 12.2.3.6 Eq. 7

$M_{pr} := 19000\ k \cdot ft$

Largest probable flexural strength of the section from SP Column assuming tensile stress of 1.25*Fy and Φ = 1.0

$M_u := \rho \cdot M_{ot} = 992\ k \cdot ft$

$V_u := \rho \cdot V_{tot} = 34.5\ k$

Ultimate Panel Shear ($\rho\, Q_e$)

$h_{wcs} := 40.5\ ft$

Height of wall measured from critical section to parapet (see NTE diagram)

Pier Check - ACI 318-19 Table R18.10.1

If wall section is a pier, requirements of ACI 318-19 18.10.8 or 18.14 must be met.

$$Check_1 := \text{if}\left(\frac{w_s}{t_p} \le 6, \text{“Pier”}, \text{“Segment”}\right)$$

$$PierCheck := \text{if}\left(\frac{h_{wcs}}{w_s} \ge 2, Check_1, \text{“Segment”}\right)$$

$PierCheck = \text{“Segment”}$

Design Shear Force - ACI 318-19 18.10.3

$stories_{wcs} := 1$

Number of Stories above Critical Section

$$n_s := \max\left(stories_{wcs}, 0.007 \cdot \frac{h_{wcs}}{in}\right)$$

$$\omega_v := \begin{cases} 1.0 & \text{if } \frac{h_{wcs}}{w_s} < 2 \\ 0.9 + \frac{n_s}{10} & \text{else if } n_s \le 6.0 \\ min\left(1.3 + \frac{n_s}{30}, 1.8\right) & \text{else} \end{cases} = 1$$

Dynamic shear amplification factor, ACI 318-19 18.10.3.1.3

$$M_{prratio} := \frac{M_{pr}}{M_u}$$

$$\Omega_v := \text{if}\left(\frac{h_{wcs}}{w_s} \le 1.5, 1.0, \max\left(M_{prratio}, 1.5\right)\right)$$

$$\Omega_v = 19.15$$

Shear Overstrength factor, ACI 318-19 18.10.3.1.2

$$V_{ratio} := min\left(\omega_v \cdot \Omega_v, 3\right) = 3$$

$$V_e := V_{ratio} \cdot V_u$$

$$V_{epiera} := \text{if}\left(\frac{w_s}{t} > 2.5, V_u \cdot \Omega_o, \text{“N/A”}\right)$$

Alternate Ve if pier aspect ratio met
ACI 318-19 18.10.8.1a

$$V_e := \text{if}\left(PierCheck = \text{“Pier”}, \text{if}\left(V_{epiera} = \text{“N/A”}, \frac{M_{pr}}{h_{roof}}, min\left(\frac{M_{pr}}{h_{roof}}, V_{epiera}\right)\right), V_{ratio} \cdot V_u\right)$$

$$V_e = 103.6\ k$$

Design Shear Force,
For Segments: ACI 318-19 18.10.3.1
For Piers: ACI 318-19 18.7.6.1

Shear Steel Requirements - ACI 318-19 18.10.2.1/11.6.1

$$A_{cv} := t \cdot w_s = 2262\ in^2$$

Gross Area of Concrete Section

$$V_{min} := \lambda \cdot \sqrt{\frac{f_c}{psi}} \cdot psi \cdot A_{cv}$$

Shear Strength to determine ρ_{min}

$$\rho_{min} := \text{if}\left(V_u < V_{min}, 0.002, 0.0025\right) = 0.002$$

$$A_{v\_min} := t \cdot (12 \cdot in) \cdot \rho_{min}$$

Minimum area of Shear Steel per Foot

Determine number of curtains of shear steel - ACI 318-19 18.10.2.2

$$V_{reinf} := 2 \cdot \lambda \cdot \sqrt{\frac{f_c}{psi}} \cdot psi \cdot A_{cv}$$

Shear Strength to Determine Required Number of Layers

$$Reinforcing := \text{if}\left(V_u > V_{reinf} \lor \frac{ht}{w_s} \ge 2, \text{“(2) CURTAINS REQD”}, \text{“(1) CURTAIN REQD”}\right)$$

$$Reinforcing = \text{“(1) CURTAIN REQD”}$$

Define Shear Steel:

$No_v := \#4$ — Shear Reinforcement Bar Size

$A_{sNov} := No_{v_0}$ $\quad d_{sv} := No_{v_1}$ — Area of Shear Reinforcement Bar

$S := 12 \cdot in$ — Shear Reinforcement Spacing (12" max, typical)

$N_L := 1$ — Number of Layers **(CL = 1, EF = 2)**

$A_{vs} := N_L \cdot \frac{12 \cdot in}{S} \cdot A_{sNov}$ — Area of Shear Reinforcement per Foot

$$\rho_t := \frac{A_{vs}}{(12 \cdot in) \cdot (t)}$$

$A_{v\_min\_check} := \text{if}\left(A_{v\_min} \le A_{vs}, \text{"OK"}, \text{"REVISE!"}\right)$ $\quad A_{v\_min\_check} = \text{"OK"}$

$Check := \text{if}\left(V_u > V_{reinf} \wedge N_L = 1, \text{"REVISE REINF"}, \text{"OK"}\right)$ $\quad Check = \text{"OK"}$

Shear Wall Strength - ACI 318-19 Eq. 18.10.4.1

$$\alpha_c := \begin{array}{l} \text{if } \frac{ht}{w_s} \le 1.5 \\ \quad 3.0 \\ \text{else if } \frac{ht}{w_s} \ge 2.0 \\ \quad 2.0 \\ \text{else} \\ \quad 2 + \frac{2.0 - \frac{ht}{w_s}}{0.5} \end{array} = 2.77$$

$$\phi V_n := \phi_{vE} \cdot \left(A_{cv} \cdot \left(\alpha_c \cdot \lambda \cdot \left(\sqrt{\frac{f_c}{psi}} \cdot psi\right) + \rho_t \cdot f_y\right)\right)$$ ACI 318-19 Eq. 18.10.4.1

Define Shear Strength Max - ACI 318-19 18.10.4.4

$$\phi V_{nmax} := \phi_{vE} \cdot \left(8 \cdot \sqrt{\frac{f_c}{psi}} \cdot psi \cdot A_{cv}\right)$$ ACI 318-19 Eq. 18.10.4.4

$check := \text{if}\left(V_e \le \phi V_{nmax}, \text{"Proceed with Design"}, \text{"Revise!"}\right)$ $\quad check = \text{"Proceed with Design"}$

$check := \text{if}\left(\phi V_n \le \phi V_{nmax}, \text{"OK"}, \text{"Vn MAX exceeded, reduce reinforcement"}\right)$ $check = \text{"OK"}$

$$DCR_v := \frac{V_e}{min\left(\phi V_n, \phi V_{nmax}\right)}$$ $DCR_v = 0.24$

$Flag_v = \text{"USE (1) LAYER /S OF \#4 BARS AT 12''OC HORIZ MIN"}$

Flexural Steel - ACI 318-19 18.10.6

$No_f := \#5$ Longitudinal Reinforcement Bar Size

$A_{sNof} := No_{f_0}$ $d_{Nof} := No_{f_1}$

$N_{Lf} := 2$ Number of Layers of Longitudinal Reinforcement (CL = 1, EF = 2) at Each End

$N_f := 18$ Number of Longitudinal Reinforcement Bars per Layer at Each End

$d_{solve} := 0.8 \cdot w_s = 20.8\ ft$ Depth of Lever Arm

Solve Block: Guess Values / Constraints

$A_{sreq} := 1\ in^2$

$$\phi_f \cdot A_{sreq} \cdot f_y \cdot \left(d_{solve} - \frac{A_{sreq} \cdot f_y}{0.85 \cdot f_c \cdot t \cdot 2}\right) - M_u = 0$$

$A_{sreq\_flex} := \text{Find}\left(A_{sreq}\right)$

$A_{sreq\_flex} = 0.89\ in^2$ Required Area of Flexural Steel

$A_{s\_long} := N_f \cdot N_{Lf} \cdot A_{sNof} = 11.16\ in^2$ Area of Flexural Steel

$$DCR_f := \frac{A_{sreq\_flex}}{A_{s\_long}}$$ $DCR_f = 0.08$

$Flag_f = \text{"USE (36) \#5 BARS EACH END MIN"}$

----------------------- VERIFY SECTION BELOW APPLIES. IT IS NOT A REQUIREMENT FOR PRECAST WALLS OR TILT UP PANELS PER ACI 318-19 18.11.2.1-----------------------

Req'd Long Reinf at Wall Ends- ACI 318-19 18.10.2.4

If wall section hw/lw >2, requirements of ACI 318-19 18.10.2.4 must be met.

$$WallEndCheck := \text{if}\left(\frac{h_{roof}}{w_s} \ge 2, \text{“Provide Req'd Reinf at Wall End”}, \text{“Not Applicable”}\right)$$

$$WallEndCheck = \text{“Not Applicable”}$$

18.10.6.5 Where special boundary elements are not required by 18.10.6.2 or 18.10.6.3, (a) and (b) shall be satisfied:

(a) Except where V_u in the plane of the wall is less than $\lambda\sqrt{f_c'}A_{cv}$, horizontal reinforcement terminating at the edges of structural walls without boundary elements shall have a standard hook engaging the edge reinforcement or the edge reinforcement shall be enclosed in U-stirrups having the same size and spacing as, and spliced to, the horizontal reinforcement.

(b) If the maximum longitudinal reinforcement ratio at the wall boundary exceeds $400/f_y$, boundary transverse reinforcement shall satisfy 18.7.5.2(a) through (e) over the distance calculated in accordance with 18.10.6.4(a). The vertical spacing of transverse reinforcement at the wall boundary shall be in accordance with Table 18.10.6.5(b).

The amount of reinforcement required for leg in-plane moment is usually lower than the 400/fy ratio stated in 18.10.6.5 (OOP demands typically govern for these bars) and the minimum spacing requirements in Table 18.10.6.5b should not need to be met in that case.

Transverse Reinforcement Requirements - ACI 318-19 18.10.6.5

$$A_{s\_check} := \frac{400}{60000} \cdot (0.2 \cdot w_s) \cdot t$$

Assumes 0.2*Panel Width for N.A. depth. Input leg into SP Column if req'd to refine results.

$$V_{check} := \lambda \cdot \sqrt{\frac{f_c}{psi}} \cdot psi \cdot A_{cv}$$

Check shear against 18.10.6.5a

$$Check_{18.10.6.5a} := \text{if}\left(V_e > V_{check}, \text{“PROVIDE HOOKED HORIZ BARS OR HOOPED ENDS”}, \text{“OK”}\right)$$

$$Check_{18.10.6.5a} = \text{“OK”}$$

$$Check_{18.10.6.5b} := \text{if}\left(A_{sreq\_flex} > A_{s\_check}, \text{“PROVIDE CONFINING HOOPS @ 5'' O.C. MAX per 18.10.6.5”}, \text{“OK”}\right)$$

$$Check_{18.10.6.5b} = \text{“OK”}$$

Basic Boundary Element Check w/ Gross Section Properties - ACI 318-19 18.10.6.3

$I_g := \frac{(w_s)^3 \cdot t}{12}$ $\quad A_g := \frac{w_s \cdot t}{2}$ $\quad S_g := \frac{t \cdot w_s^2}{6}$

$P_{u1} := P_{u1c} = 152.8\ k$

Factored Axial Load - ASCE 7-16 2.3.6 Load Combo 6

$\sigma_u := \frac{M_u}{S_g} + \frac{P_{u1}}{A_g}$

Combined Stress @ Extreme Fiber

$F_c := 0.2 \cdot f_c$

$DCR_\sigma := \frac{\sigma_u}{F_c}$ $\quad DCR_\sigma = 0.3$

$BasicCheck := \text{if}\left(DCR_\sigma > 1, \text{"Perform Alternate Analysis"}, \text{"Okay"}\right)$ $\quad BasicCheck = \text{"Okay"}$

If Wall does not meet basic boundary element check, provide boundary elements or perform Alternate check per ACI 318-19 18.10.6.2 ONLY if hwcs/ w_s ≥ 2**.**

$$AlternateCheck := \begin{array}{l} \text{if } \frac{h_{wcs}}{w_s} \geq 2 \wedge BasicCheck = \text{"Perform Alternate Analysis"} \\ \quad \text{"Proceed"} \\ \text{also if } BasicCheck = \text{"Okay"} \\ \quad \text{"Not Applicable"} \\ \text{else} \\ \quad \text{"Boundary Element Necessary – Alternate Check Not Allowed"} \end{array}$$

$AlternateCheck = \text{"Not Applicable"}$

Verify Section Is Uncracked

$y_t := \frac{w_s}{2}$

Neutral Axis of Gross Section

$$M_{cr} := \frac{7.5 \cdot \lambda \cdot \sqrt{\frac{f_c}{psi}} \cdot psi \cdot I_g}{y_t}$$

Cracking Moment - ACI 318-19 19.2.3.1 Eq 19.2.3.1, 24.2.3.5 Eq 24.2.3.5

$M_a := M_u \cdot EL_{asd\_factor}$

Service Moment

$M_{crCheck} := \text{if}\left(M_{cr} < M_a, \text{"Cracked; If necessary adjust relative stiffness and evaluate load"}, \text{"Uncracked"}\right)$

$M_{crCheck} = \text{"Uncracked"}$

Panel 2 - Leg Design

Wall Shear Design - Leg

$Label_{leg} := A$

$leg := leg_{temp_{Label_{leg}}} = 8.5\ ft$ $k_{leg} := k_{temp_{Label_{leg}}} = 0.50$

$$P_{u1} := \left(\left(1.2 + 0.2 \cdot S_{ds}\right) \cdot P_{tot} + \left(0.2 \cdot SL + LL\right)\ w_s\right) \left(\frac{leg + \frac{w_o}{2}}{w_s}\right) = 76.4\ k$$

ASCE 7-16 12.2.3.6 Eq 6

$$P_{u2} := \left(0.9 - 0.2 \cdot S_{ds}\right)\ P_{tot} \left(\frac{leg + \frac{w_o}{2}}{w_s}\right) = 39.1\ k$$

ASCE 7-16 12.2.3.6 Eq 7

$M_{pr} := 2966\ k \cdot ft$

Largest probable flexural strength of the section from SP Column assuming tensile stress of 1.25*Fy and Φ = 1.0

Pier Check - ACI 318-19 Table R18.10.1

If wall section is a pier, requirements of ACI 318-19 18.10.8 or 18.14 must be met

$BC_{pier} :=$ Boundary condition: Fixed–Fixed

Boundary Condition Pier Factor

BC_{pier} **= 0.5 for fixed-fixed,**

BC_{pier} **= 1.0 for fixed-pinned**

$Check_1 := \text{if}\left(\frac{leg}{t_p} \le 6, \text{“Pier”}, \text{“Segment”}\right)$ $Check_1 = \text{“Segment”}$

$PierCheck := \text{if}\left(\frac{ht_o}{leg} \ge 2, Check_1, \text{“Segment”}\right)$ $PierCheck = \text{“Segment”}$

Design Shear Force - ACI 318-19 18.10.3

$V_u := \rho \cdot V_{tot} \cdot k_{leg} = 17.27\ k$ — Ultimate Leg Shear (ρQ_e)

$h_{wcsleg} := h_{roof} = 36\ ft$ — Height of panel leg above critical section

$stories_{wcs} := 1$ — Number of Stories above Critical Section

$$n_s := \max\left(stories_{wcs}, 0.007 \cdot \frac{h_{wcsleg}}{in}\right)$$

$$\omega_v := \begin{cases} 1.0 & \text{if } \frac{h_{wcsleg}}{leg} < 2 \\ 0.9 + \frac{n_s}{10} & \text{else if } n_s \le 6.0 \\ min\left(1.3 + \frac{n_s}{30}, 1.8\right) & \text{else} \end{cases} = 1.2$$

Dynamic shear amplification factor, ACI 318-19 18.10.3.1.3

$M_u := V_u \cdot BC_{pier} \cdot ht_o = 103.6\ k \cdot ft$

$$M_{prratio} := \frac{M_{pr}}{M_u}$$

$$\Omega_v := \text{if}\left(\frac{h_{wcsleg}}{leg} \le 1.5, 1.0, \max\left(M_{prratio}, 1.5\right)\right)$$

$\Omega_v = 28.63$ — Overstrength factor, ACI 318-19 18.10.3.1.2

$V_{ratio} := min\left(\omega_v \cdot \Omega_v, 3\right) = 3$

$$V_{epiera} := \text{if}\left(\frac{leg}{t} > 2.5, V_u \cdot \Omega_o, \text{“N/A”}\right)$$

Alternate Ve if pier aspect ratio met
ACI 318-19 18.10.8.1a

$$V_e := \text{if}\left(PierCheck = \text{“Pier”}, \text{if}\left(V_{epiera} = \text{“N/A”}, \frac{M_{pr}}{BC_{pier} \cdot ht_o}, min\left(\frac{M_{pr}}{BC_{pier} \cdot ht_o}, V_{epiera}\right)\right), V_{ratio} \cdot V_u\right)$$

$V_e = 51.8\ k$

Design Shear Force,
For Segments: ACI 318-19 18.10.3.1
For Piers: ACI 318-19 18.7.6.1 or 18.10.8.1

Shear Wall Design

$Check_{Ve18.10.8.1} = \text{"Not applicable, proceed with design"}$

ACI 318-19 18.10.8.1 - Transverse reinforcement with seismic hooks shall be provided to resist shear determined per 18.7.6.1.1. Transverse reinf. spacing 6" max OC. Extend reinf. minimum 12" above/ below opening.

Shear Steel Requirements - ACI 318-19 18.10.2.1/11.6.1

$A_{cv} := t \cdot leg = 739.5\ in^2$ — Gross Area of Concrete Section

$V_{min} := \lambda \cdot \sqrt{\frac{f_c}{psi}} \cdot psi \cdot A_{cv}$ — Shear Strength to determine ρ_{min} - ACI 318-19 18.10.2.1

$\rho_{min} := \text{if}\left(V_u < V_{min}, 0.002, 0.0025\right) = 0.002$

$A_{v\_min} := (t) \cdot (12 \cdot in) \cdot (\rho_{min})$ — Minimum area of Shear Steel per Foot

Determine number of curtains of shear steel - ACI 318-19 18.10.2.2

$V_{reinf} := 2 \cdot \lambda \sqrt{\frac{f_c}{psi}} \cdot psi \cdot A_{cv}$ — Shear Strength to Determine Required Number of Layers

$Reinforcing := \text{if}\left(V_u > V_{reinf}, \text{"(2) CURTAINS REQD"}, \text{"(1) CURTAIN REQD"}\right)$

$Reinforcing = \text{"(1) CURTAIN REQD"}$

Define Shear Steel:

$No_v := \#4$ — Shear Reinforcement Bar Size

$A_{sNov} := No_{v_0}$ $\quad d_{sv} := No_{v_1}$ — Area of Shear Reinforcement Bar

$S := 12 \cdot in$ — Shear Reinforcement Spacing (12" max, typical)

$N_L := 1$ — Number of Layers **(CL = 1, EF = 2)**

$A_{vs} := N_L \cdot \frac{12 \cdot in}{S} \cdot A_{sNov}$ — Area of Shear Reinforcement per Foot

$\rho_t := \frac{A_{vs}}{(12 \cdot in) \cdot (t)}$

$A_{v\_min\_check} := \text{if}\left(A_{v\_min} \le A_{vs}, \text{"OK"}, \text{"REVISE!"}\right)$ $\quad A_{v\_min\_check} = \text{"OK"}$

$Check := \text{if}\left(V_u > V_{reinf} \land N_L = 1, \text{"REVISE REINF"}, \text{"OK"}\right)$ $\quad Check = \text{"OK"}$

Shear Wall Strength - ACI 318-19 Eq. 18.10.4.1

$$h_{w\_max} := \max\left(\frac{ht}{w_s}, \frac{ht_o}{leg}\right) = 1.62$$

ACI 318-19 18.10.4.2 - Ratio for the greatest of wall or segment needs to be considered

$$\alpha_c := \begin{array}{l} \text{if } h_{w\_max} < 1.5 \\ \quad 3.0 \\ \text{else if } h_{w\_max} \geq 2.0 \\ \quad 2.0 \\ \text{else} \\ \quad 2 + \dfrac{2.0 - h_{w\_max}}{0.5} \end{array} = 2.77$$

$$\phi V_n := \phi_{vE} \cdot \left(A_{cv} \cdot \left(\alpha_c \cdot \lambda \cdot \left(\sqrt{\frac{f_c}{psi}} \cdot psi\right) + \rho_t \cdot f_y\right)\right)$$

ACI 318-19 Eq. 18.10.4.1

Define Shear Strength Max - ACI 318-19 18.10.4.4

$$\phi V_{nmax} := \phi_{vE} \cdot \left(8 \cdot \sqrt{\frac{f_c}{psi}} \cdot psi \cdot A_{cv}\right)$$

ACI 318-19 Eq. 18.10.4.4

$check := \text{if}\left(V_e \leq \phi V_{nmax}, \text{“Proceed with Design”}, \text{“Revise!”}\right)$

$check = \text{“Proceed with Design”}$

$check := \text{if}\left(\phi V_n \leq \phi V_{nmax}, \text{“OK”}, \text{“Vn MAX exceeded, reduce reinforcement”}\right)$

$check = \text{“OK”}$

$$\phi V_n := min\left(\phi V_n, \phi V_{nmax}\right) = 138.91\ k$$

$$DCR_v := \frac{V_e}{\phi V_n}$$

$DCR_v = 0.37$

$Flag_v = \text{“USE (1) LAYER /S OF \#4 BARS AT 12″OC HORIZ MIN”}$

Leg Compression Force: Per Load Case ASCE 7-16 12.2.3.6. Default is for outer legs receiving half of total panel load in addition to seismic.

$$P_{uleg} := P_{u1} + \frac{\rho \cdot M_{ot}}{w_s - \left(\dfrac{leg}{2}\right)} = 122\ k$$

Flexural Steel - ACI 318-19 18.10.6

$No_f := \#5$ Longitudinal Reinforcement Bar Size

$A_{sNof} := No_{f_0}$ $d_{Nof} := No_{f_1}$ Area of Flexural Reinforcement Bar
Diameter of Flexural Reinforcement Bar

$N_{Lf} := 2$ Number of Layers of Longitudinal Reinforcement (CL = 1, EF = 2)

$N_f := 18$ Number of Longitudinal Reinforcement Bars per Layer

$A_{s\_leg} := N_f \cdot N_{Lf} \cdot A_{sNof} = 11.16\ in^2$ Total Area of Steel Provided in Leg

$M_u := V_u \cdot BC_{pier} \cdot ht_o = 103.6\ k \cdot ft$ Ultimate Overturning Moment ($\rho\, Q_e$)

$d_{solve} := 0.8 \cdot leg = 6.8\ ft$ Depth of Lever Arm, use variables from above where possible

Solve Block — Guess Values / Constraints

$$A_{sreq} := 1\ in^2$$

$$\phi_f \cdot A_{sreq} \cdot f_y \cdot \left(d_{solve} - \frac{A_{sreq} \cdot f_y}{0.85 \cdot f_c \cdot t \cdot 2}\right) - M_u = 0$$

$$A_{sreq\_flex} := \text{Find}\left(A_{sreq}\right)$$

$A_{sreq\_flex} = 0.28\ in^2$ Required Area of Flexural Steel

$$A_{sreq\_ten} := \frac{\rho \cdot M_{ot}}{\phi_f \cdot (w_s - 1.5 \cdot ft) \cdot f_y} = 0.75\ in^2$$ Required Area of Steel to Resist Panel Overturning

$A_{stot} := A_{sreq\_flex} \cdot 2 + A_{sreq\_ten} = 1.32\ in^2$ Required Area of Steel for Leg Flexure and Panel Overturning

$$DCR_f := \frac{A_{stot}}{A_{s\_leg}} = 0.12$$

$DCR_f = 0.12$

$Flag_f =$ "USE (36) #5 BARS EACH END MIN"

Transverse Reinforcement Requirements- ACI 318-19 18.10.6.5

$$A_{s\_check} := \frac{400}{\frac{f_y}{psi}} \cdot (0.2 \cdot leg) \cdot t$$ Assumes 0.2*leg for N.A. depth. Input leg into SP Column if req'd to refine results.

$$V_{check} := \lambda \cdot \sqrt{\frac{f_c}{psi}} \cdot psi \cdot A_{cv}$$

Check shear against 18.10.6.5a

$$Check_{18.10.6.5a} := \text{if}\left(V_u > V_{check}, \text{"PROVIDE HOOKED HORIZ BARS OR CLOSED HOOPS"}, \text{"OK"}\right)$$

$$Check_{18.10.6.5a} = \text{"OK"}$$

$$Check_{18.10.6.5b} := \text{if}\left(A_{sreq\_flex} > A_{s\_check}, \text{"PROVIDE CONFINING HOOPS @ 5'' O.C. MAX per 18.10.6.5"}, \text{"OK"}\right)$$

$$Check_{18.10.6.5b} = \text{"OK"}$$

Basic Boundary Element Check w/ Gross Section Properties - ACI 318-19 18.10.6.3

$$I_g := \frac{leg^3 \cdot t}{12} \qquad A_g := leg \cdot t \qquad S_g := \frac{t \cdot leg^2}{6}$$

$$P_u := P_{uleg}$$

Factored Axial Load - Load Combination
ASCE 7-16 12.2.3.6 Eq. 6

$$\sigma_u := \frac{M_u}{S_g} + \frac{P_{uleg}}{A_g}$$

Combined Stress at Extreme Fiber

$$F_c := 0.2 \cdot f_c$$

$$DCR_\sigma := \frac{\sigma_u}{F_c}$$

$$DCR_\sigma = 0.33$$

$$BasicCheck := \text{if}\left(F_c > \sigma_u, \text{"Okay"}, \text{"Perform Alternate Analysis"}\right)$$

$$BasicCheck = \text{"Okay"}$$

If Wall does not meet basic boundary element check, provide boundary elements or perform Alternate check per ACI 318-19 18.10.6.2 <u>ONLY</u> if hwcs/ w_s > 2.

$$AlternateCheck := \begin{array}{l} \text{if } \frac{h_{wcsleg}}{leg} \geq 2 \land BasicCheck = \text{"Perform Alternate Analysis"} \\ \quad \text{"Proceed"} \\ \text{also if } BasicCheck = \text{"Okay"} \\ \quad \text{"Not Applicable"} \\ \text{else} \\ \quad \text{"Boundary Element Necessary – Alternate Check Not Allowed"} \end{array}$$

$$AlternateCheck = \text{"Not Applicable"}$$

Verify Section Uncracked

$y_t := \frac{leg}{2}$ Neutral Axis of Gross Section

$M_{cr} := \frac{7.5 \cdot \lambda \cdot \sqrt{\frac{f_c}{psi}} \cdot psi \cdot I_g}{y_t}$ Cracking Moment - ACI 318-19 19.2.3.1 Eq 19.2.3.1, 24.2.3.5 Eq 24.2.3.5

$M_a := M_u \cdot EL_{asd\_factor}$ Service Moment

$M_{crCheck} := \text{if}\left(M_{cr} < M_a, \text{"Cracked; If necessary adjust relative stiffness and evaluate load"}, \text{"Uncracked"}\right)$

$M_{crCheck} = \text{"Uncracked"}$

----------------------- VERIFY SECTION BELOW APPLIES-----------------------

Slab Shear Dowel Bars (Leg A) - Shear Friction per ACI 318-19 22.9.4

$\mu := 0.6$ Coefficient of Friction - ACI 318-19 22.9.4.2

$L_{dowel} := leg$ Length of wall including dowels (leg length at storefront condition)

$No_{dowel} := \#5$ Dowel Reinforcement Bar Size

$A_{sNodowel} := No_{dowel}$

$N_{dowel} := 4$ Number of Dowels per Leg - (4) Min

$A_{vf} := A_{sNodowel} \cdot N_{dowel} = 1.24\ in^2$ Area of Shear Friction Steel Across Panel

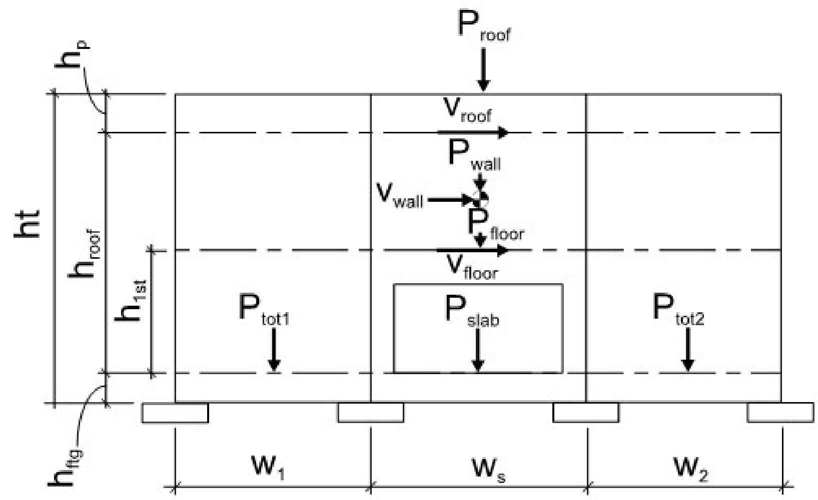
$\phi V_{vf\_n} := \phi_{vE} \cdot \left(\mu \cdot A_{vf} \cdot f_y\right)$ Allowable Shear Capacity - ACI 318-19 22.9.4.2

$V_u := \rho \cdot V_{tot} \cdot k_a = 17.27\ k$ Ultimate Panel Shear (ρQ_e)

$DCR_{vf} := \frac{V_u}{\phi V_{vf\_n}}$ $DCR_{vf} = 0.64$

$flag := \text{if}\left(\phi V_{vf\_n} \ge V_u, \text{"OK"}, \text{"NG!"}\right)$ $flag = \text{"OK"}$

318-19 Panel w/ (1) Opening - Panel 18 - North



Shear Panel Input (w_s)

$w_s := 26 \cdot ft$	Panel Width
$h_p := 0 \cdot ft$	Parapet Height
$h_{roof} := 35.75 \cdot ft$	Height of Roof Above FF
$h_{1st} := 0 \cdot ft$	Height of Floor Above FF
$h_{ftg} := 1.5 \cdot ft$	FF to T/FTG Height
$ht := h_p + h_{roof} + h_{ftg} = 37.3\ ft$	Total Panel Height (T/FTG to T/PARAPET)
$ht_o := 14 \cdot ft$	Opening Height
$w_o := 16 \cdot ft$	Opening Width
$lega := 5\ ft$	Leg A Width
$legb := w_s - w_o - lega = 5\ ft$	Leg B Width
$t_p := 9.5 \cdot in$	Panel Thickness
$r_v := 0.75 \cdot in$	Reveal Depth
$t := t_p - r_v = 8.8\ in$	Effective Panel Thickness
$Wt_p := w_c \cdot t_p$	Panel Wt. (psf)

Relative Panel Leg Stiffnesses (w_s)

Amrhein/Yellow book method

$BC_{lega} :=$ Boundary Condition: Fixed–Fixed

$BC_{legb} :=$ Boundary Condition: Fixed–Fixed

Leg A & B boundary conditions
BC_{leg} = 1 for fixed-fixed
BC_{leg} = 4 for fixed-pinned

$hd_{ratioa} := \frac{ht_o}{lega} = 2.8$

Leg A aspect ratio

$hd_{ratiob} := \frac{ht_o}{legb} = 2.8$

Leg B aspect ratio

$P_k := 100000\ lb$ $t_k := 1\ in$ $E_k := 1000000\ psi$

$$\Delta_{ka} := \frac{P_k}{E_k \cdot t_k} \cdot \left(BC_{lega} \cdot \left(hd_{ratioa}\right)^3 + 3 \cdot \left(hd_{ratioa}\right)\right)$$

$$\Delta_{kb} := \frac{P_k}{E_k \cdot t_k} \cdot \left(BC_{legb} \cdot \left(hd_{ratiob}\right)^3 + 3 \cdot \left(hd_{ratiob}\right)\right)$$

$R_{ka} := \frac{1}{\Delta_{ka}}$ $R_{kb} := \frac{1}{\Delta_{kb}}$

$k_a := \frac{R_{ka}}{R_{ka} + R_{kb}} = 0.5$

Leg A Relative Stiffness

$k_b := \frac{R_{kb}}{R_{ka} + R_{kb}} = 0.5$

Leg B Relative Stiffness

$Stiffness := \text{if}\left(\left|1.0 - \left(k_a + k_b\right)\right| > 0.001, \text{“CHECK”}, \text{“OK”}\right)$ $Stiffness = \text{“OK”}$

Adjacent Panel Input (w_1) / Adjacent Panel Input (w_2)

Adjacent Panel Input (w_1)	Adjacent Panel Input (w_2)	
$w_1 := w_s$	$w_2 := w_s$	Adjacent Panel Width
$h_{p1} := h_p$	$h_{p2} := h_p$	Parapet Height
$h_{roof1} := h_{roof}$	$h_{roof2} := h_{roof}$	Height of Roof Above FF
$h_{ftg1} := h_{ftg}$	$h_{ftg2} := h_{ftg}$	FF to T/FTG Height
$ht_1 := h_{p1} + h_{roof1} + h_{ftg1} = 37.3\ ft$	$ht_2 := h_{p2} + h_{roof2} + h_{ftg2} = 37.3\ ft$	Total Panel Height (T/FTG to T/PARAPET)

$t_{p1} := t_p$ | $t_{p2} := t_p$ | Panel Thickness

$Ao_1 := 0 \cdot ft \cdot 0 \cdot ft$ | $Ao_2 := 0\ ft \cdot 0\ ft$ | Area of Openings

Loading:

Seismic (1.0 Q_e)

$V_{roof} := 11.5 \cdot k$ — Seismic Shear @ Roof - ρ not included

$V_{floor} := 0 \cdot k$ — Seismic Shear @ Floor

$V_{wall} := \left(\left(ht - h_{ftg}\right) \cdot w_s - ht_o \cdot w_o\right) \cdot Wt_p \cdot C_s$ — Seismic Shear due to Wall Panel Mass

$V_{tot} := V_{roof} + V_{wall} + V_{floor} = 25.6\ k$ — Total Seismic Shear

$M_{ot\_roof} := V_{roof} \cdot h_{roof}$ — Overturning Moment due to Roof Shear

$M_{ot\_floor} := V_{floor} \cdot h_{1st}$ — Overturning Moment due to Floor Shear

$M_{ot\_wall} := V_{wall} \cdot \dfrac{\left(ht - h_{ftg}\right)}{2}$ — Overturning Moment Due to Panel Mass

$M_{ot} := M_{ot\_roof} + M_{ot\_floor} + M_{ot\_wall} = 663.6\ k \cdot ft$ — Total Overturning Moment @ Wall Panel

Gravity:

Shear Panel (w_s):

$l_{roof\_trib} := \dfrac{60}{2} \cdot ft$ — Length of Roof Tributary to Panel

$l_{floor\_trib} := \dfrac{0}{2} \cdot ft$ — Length of Floor Tributary to Panel

$SL := 0.57\ klf$ (Balanced W/ Rain on Snow) — Roof snow load tributary to panel

$LL := 0.0\ klf$ — Floor live load tributary to panel

$P_{roof} := DL_{sei} \cdot l_{roof\_trib} \cdot w_s$ — Roof DL

$P_{floor} := DL_{flr} \cdot l_{floor\_trib} \cdot w_s$ — Floor DL

$P_{wall} := \left(ht \cdot w_s - ht_o \cdot w_o\right) \cdot Wt_p$ — Wall Panel DL

$P_{slab} := w_c \cdot t_{SOG} \cdot 8 \cdot ft \cdot w_s$ — Slab-on-Grade DL (assumes 8' width of slab over width of panel available to resist OT)

$P_{tot} := P_{roof} + P_{floor} + P_{wall} = 96.9\ k$ Total DL (without slab @ wall considered)

Panel (w_1):

$P_{roof1} := P_{roof}$ Roof DL

$P_{floor1} := P_{floor}$ Floor DL

$P_{wall1} := \left(w_1 \cdot ht_1 - Ao_1\right) \cdot t_{p1} \cdot w_c$ Wall Panel DL

$P_{slab1} := P_{slab} \cdot \left(\dfrac{w_1}{w_s}\right)$ Slab-on-Grade DL

$P_{tot1} := P_{roof1} + P_{floor1} + P_{wall1} + P_{slab1} = 141.7\ k$ Total DL (including slab weight)

Panel (w_2):

$P_{roof2} := P_{roof}$ Roof DL

$P_{floor2} := P_{floor}$ Floor DL

$P_{wall2} := \left(w_2 \cdot ht_2 - Ao_2\right) \cdot t_{p2} \cdot w_c$ Wall Panel DL

$P_{slab2} := P_{slab} \cdot \left(\dfrac{w_2}{w_s}\right)$ Slab-on-Grade DL

$P_{tot2} := P_{roof2} + P_{floor2} + P_{wall2} + P_{slab2} = 141.7\ k$ Total DL (including slab weight)

Resisting Moment:

$M_{resist} := \left(P_{tot} + P_{slab}\right) \cdot \dfrac{w_s}{2} = 1496.4\ k \cdot ft$ Resisting Moment at Shear Panel

Load Combinations:

ASCE 7-16 2.3.6

LRFD 6: (1.2+0.2Sds)D+1.0ρ Q_e + L + 0.2S

LRFD 7: (0.9 - 0.2Sds)D + 1.0ρ Q_e (Resisting Moment of Slab-on-Grade)

IBC 2021 1605.2 - Foundations

ASD 16-5: D + L + S + E/1.4 (Foundation Bearing)

ASD 16-6: 0.9D + E/1.4 (Foundation Uplift/Weight. Ev = 0 for fdn size, Ev included for holdowns)

Panel Hold Down - IBC 2021 1605.2 - Alternative Load Combination ASD 16-6

$M_{ot\_IBC} := \rho \cdot \dfrac{M_{ot}}{1.4}$ ASD Overturning Moment

$M_{res\_IBC} := \left(0.9 - \dfrac{0.2 \cdot S_{ds}}{1.4}\right) \cdot M_{resist}$ ASD Resisting Moment

$No_{hd} := \#5$ Rebar Size No.

$N_{hd} := 2$ Number of Bars in Hold Down

$L_{hd} := \text{if}\left(No_{hd_0} = 0.31\ in^2, L_{HD5_{N_{hd}}}, L_{HD6_{N_{hd}}}\right) = 8\ in$ Length of Hold Down

$A_{shd} := No_{hd_0} \cdot N_{hd}$ Area of Steel in Hold Down

$T_{hd} := \dfrac{M_{ot\_IBC} - M_{res\_IBC}}{w_s - (1 \cdot ft + 0.5 \cdot L_{hd})} = -28.1\ k$ ASD Tension at Hold Down

$A_{shdreq} := \dfrac{\left(\dfrac{T_{hd}}{EL_{asd\_factor}}\right)}{\phi_t \cdot f_y} = -0.74\ in^2$ Required Area of Steel

$DCR_{hd} := \dfrac{A_{shdreq}}{A_{shd}}$ $DCR_{hd} = -1.2$

$hd\_Results =$ "NO HOLDOWNS REQD"

Footing Uplift and Seismic Bearing Checks

$l_f := 6\ ft$ — Length of Footing

$w_f := 4.5 \cdot ft$ — Width of Footing

$d_f := 1 \cdot ft$ — Depth of Footing (Use 2'-0" Min @ hold down, see footing calcs)

$P_{ftg} := w_c \cdot l_f \cdot w_f \cdot d_f$ — Footing Weight

Required Footing Weight - Load Combination ASD 16-6

$$M_{res\_uplift} := M_{res\_IBC} + \left(0.9 \cdot \left(min\left(\frac{P_{tot1}}{2}, \frac{P_{tot2}}{2}\right) \cdot w_s\right)\right)$$

Factored Resisting Moment (Including contribution of slab & adjacent panel)

$M_{res\_uplift} = 2824.61\ k \cdot ft$

$$Wt_{ftg\_uplift} := \frac{\left(M_{ot\_IBC} - M_{res\_uplift}\right)}{w_s - \left(1\ ft + 0.5 \cdot L_{hd}\right)}$$

Required Footing Weight

$Wt_{ftg\_uplift} = -95.29\ k$

$flag_w := \text{if}\left(Wt_{ftg\_uplift} \le P_{ftg}, \text{"OK"}, \text{"NG!!!"}\right)$ $\quad flag_w = \text{"OK"}$

------------------------ VERIFY SECTION BELOW APPLIES------------------------

Maximum Seismic Bearing, PAD Footing Condition - Load Combination ASD 16-6

$$C_{ot\_comp} := \frac{M_{ot\_IBC}}{w_s}$$

Factored Axial Compression Force due to Overturning

$P_{grav\_comp} := 0.5 \cdot \left(P_{tot}\right)$

$P_{grav\_comp} = 48.46\ k$ — Factored Axial Gravity Force for 1/2 of panel

$A_{ftg} := l_f \cdot w_f$ — Footing Area

$$A_{ftg\_reqd} := \frac{C_{ot\_comp} + P_{grav\_comp}}{q_{sei}} = 20.01\ ft^2$$

Required Footing Area

$flag_A := \text{if}\left(A_{ftg\_reqd} \le A_{ftg}, \text{"OK"}, \text{"NG!!!"}\right)$ $\quad flag_A = \text{"OK"}$

$$q_s := \frac{C_{ot\_comp} + P_{grav\_comp}}{A_{ftg}} = 2469.88\ psf$$

Seismic Bearing Pressure

$flag_B := \text{if}\left(q_s < q_{sei}, \text{"OK"}, \text{"NG!!!"}\right)$ $\quad flag_B = \text{"OK"}$

Check Wall Segment Above/Below Opening

Wall Shear Design

$P_{u1c} := \left(1.2 + 0.2 \cdot S_{ds}\right) \cdot P_{tot} + \left(0.2\ SL + LL\right)\ w_s = 136\ k$

Factored Wall Axial Forces (for Mpr calculation) ASCE 7-16 12.2.3.6 Eq. 6

$P_{u2c} := \left(0.9 - 0.2\ S_{ds}\right) \cdot P_{tot} = 70.88\ k$

ASCE 7-16 12.2.3.6 Eq. 7

$M_{pr} := 18500\ k \cdot ft$

Largest probable flexural strength of the section from SP Column assuming tensile stress of 1.25*Fy and Φ = 1.0

$M_u := \rho \cdot M_{ot} = 664\ k \cdot ft$

$V_u := \rho \cdot V_{tot} = 25.6\ k$

Ultimate Panel Shear ($\rho\, Q_e$)

$h_{wcs} := 40.5\ ft$

Height of wall measured from critical section to parapet (see NTE diagram)

Pier Check - ACI 318-19 Table R18.10.1

If wall section is a pier, requirements of ACI 318-19 18.10.8 or 18.14 must be met.

$$Check_1 := \text{if}\left(\frac{w_s}{t_p} \le 6, \text{“Pier”}, \text{“Segment”}\right)$$

$$PierCheck := \text{if}\left(\frac{h_{wcs}}{w_s} \ge 2, Check_1, \text{“Segment”}\right)$$

$PierCheck = \text{“Segment”}$

Design Shear Force - ACI 318-19 18.10.3

$stories_{wcs} := 1$

Number of Stories above Critical Section

$$n_s := \max\left(stories_{wcs}, 0.007 \cdot \frac{h_{wcs}}{in}\right)$$

$$\omega_v := \left\| \begin{array}{l} \text{if } \dfrac{h_{wcs}}{w_s} < 2 \\ \quad \| \ 1.0 \\ \text{else if } n_s \le 6.0 \\ \quad \left\| \ 0.9 + \dfrac{n_s}{10} \right. \\ \text{else} \\ \quad \left\| \ min\left(1.3 + \dfrac{n_s}{30}, 1.8\right) \right. \end{array} \right\| = 1$$

Dynamic shear amplification factor, ACI 318-19 18.10.3.1.3

$$M_{prratio} := \frac{M_{pr}}{M_u}$$

$$\Omega_v := \text{if}\left(\frac{h_{wcs}}{w_s} \le 1.5, 1.0, \max\left(M_{prratio}, 1.5\right)\right)$$

$$\Omega_v = 27.88$$

Shear Overstrength factor, ACI 318-19 18.10.3.1.2

$$V_{ratio} := min\left(\omega_v \cdot \Omega_v, 3\right) = 3$$

$$V_e := V_{ratio} \cdot V_u$$

$$V_{epiera} := \text{if}\left(\frac{w_s}{t} > 2.5, V_u \cdot \Omega_o, \text{“N/A”}\right)$$

Alternate Ve if pier aspect ratio met
ACI 318-19 18.10.8.1a

$$V_e := \text{if}\left(PierCheck = \text{“Pier”}, \text{if}\left(V_{epiera} = \text{“N/A”}, \frac{M_{pr}}{h_{roof}}, min\left(\frac{M_{pr}}{h_{roof}}, V_{epiera}\right)\right), V_{ratio} \cdot V_u\right)$$

$$V_e = 76.9\ k$$

Design Shear Force,
For Segments: ACI 318-19 18.10.3.1
For Piers: ACI 318-19 18.7.6.1

Shear Steel Requirements - ACI 318-19 18.10.2.1/11.6.1

$$A_{cv} := t \cdot w_s = 2730\ in^2$$

Gross Area of Concrete Section

$$V_{min} := \lambda \cdot \sqrt{\frac{f_c}{psi}} \cdot psi \cdot A_{cv}$$

Shear Strength to determine ρ_{min}

$$\rho_{min} := \text{if}\left(V_u < V_{min}, 0.002, 0.0025\right) = 0.002$$

$$A_{v\_min} := t \cdot (12 \cdot in) \cdot \rho_{min}$$

Minimum area of Shear Steel per Foot

Determine number of curtains of shear steel - ACI 318-19 18.10.2.2

$$V_{reinf} := 2 \cdot \lambda \cdot \sqrt{\frac{f_c}{psi}} \cdot psi \cdot A_{cv}$$

Shear Strength to Determine Required
Number of Layers

$$Reinforcing := \text{if}\left(V_u > V_{reinf} \vee \frac{ht}{w_s} \ge 2, \text{“(2) CURTAINS REQD”}, \text{“(1) CURTAIN REQD”}\right)$$

$$Reinforcing = \text{“(1) CURTAIN REQD”}$$

Define Shear Steel:

$No_v := \#4$ — Shear Reinforcement Bar Size

$A_{sNov} := No_{v_0}$ $\quad d_{sv} := No_{v_1}$ — Area of Shear Reinforcement Bar

$S := 9 \cdot in$ — Shear Reinforcement Spacing (12" max, typical)

$N_L := 1$ — Number of Layers
(CL = 1, EF = 2)

$$A_{vs} := N_L \cdot \frac{12 \cdot in}{S} \cdot A_{sNov}$$ Area of Shear Reinforcement per Foot

$$\rho_t := \frac{A_{vs}}{(12 \cdot in) \cdot (t)}$$

$A_{v\_min\_check} := \text{if}\left(A_{v\_min} \le A_{vs}, \text{“OK”}, \text{“REVISE!”}\right)$ $\quad A_{v\_min\_check} = \text{“OK”}$

$Check := \text{if}\left(V_u > V_{reinf} \land N_L = 1, \text{“REVISE REINF”}, \text{“OK”}\right)$ $\quad Check = \text{“OK”}$

Shear Wall Strength - ACI 318-19 Eq. 18.10.4.1

$$\alpha_c := \left\| \begin{array}{l} \text{if } \dfrac{ht}{w_s} \le 1.5 \\ \quad \| \; 3.0 \\ \text{else if } \dfrac{ht}{w_s} \ge 2.0 \\ \quad \| \; 2.0 \\ \text{else} \\ \quad \left\| \; 2 + \dfrac{2.0 - \dfrac{ht}{w_s}}{0.5} \right. \end{array} \right\| = 3$$

$$\phi V_n := \phi_{vE} \cdot \left(A_{cv} \cdot \left(\alpha_c \cdot \lambda \cdot \left(\sqrt{\frac{f_c}{psi}} \cdot psi\right) + \rho_t \cdot f_y\right)\right)$$ ACI 318-19 Eq. 18.10.4.1

Define Shear Strength Max - ACI 318-19 18.10.4.4

$$\phi V_{nmax} := \phi_{vE} \cdot \left(8 \cdot \sqrt{\frac{f_c}{psi}} \cdot psi \cdot A_{cv}\right)$$ ACI 318-19 Eq. 18.10.4.4

$check := \text{if}\left(V_e \le \phi V_{nmax}, \text{“Proceed with Design”}, \text{“Revise!”}\right)$ $\quad check = \text{“Proceed with Design”}$

$check := \text{if}\left(\phi V_n \le \phi V_{nmax}, \text{“OK”}, \text{“Vn MAX exceeded, reduce reinforcement”}\right)$ $check = \text{“OK”}$

$DCR_v := \frac{V_e}{min\left(\phi V_n, \phi V_{nmax}\right)}$ $DCR_v = 0.14$

$Flag_v = \text{“USE (1) LAYER /S OF \#4 BARS AT 9”OC HORIZ MIN”}$

Flexural Steel - ACI 318-19 18.10.6

$No_f := \#5$ Longitudinal Reinforcement Bar Size

$A_{sNof} := No_{f_0}$ $d_{Nof} := No_{f_1}$

$N_{Lf} := 2$ Number of Layers of Longitudinal Reinforcement (CL = 1, EF = 2) at Each End

$N_f := 16$ Number of Longitudinal Reinforcement Bars per Layer at Each End

$d_{solve} := 0.8 \cdot w_s = 20.8\ ft$ Depth of Lever Arm

Solve Block: Guess Values / Constraints / Solver

$A_{sreq} := 1\ in^2$

$\phi_f \cdot A_{sreq} \cdot f_y \cdot \left(d_{solve} - \frac{A_{sreq} \cdot f_y}{0.85 \cdot f_c \cdot t \cdot 2}\right) - M_u = 0$

$A_{sreq\_flex} := \text{Find}\left(A_{sreq}\right)$

$A_{sreq\_flex} = 0.59\ in^2$ Required Area of Flexural Steel

$A_{s\_long} := N_f \cdot N_{Lf} \cdot A_{sNof} = 9.92\ in^2$ Area of Flexural Steel

$DCR_f := \frac{A_{sreq\_flex}}{A_{s\_long}}$ $DCR_f = 0.06$

$Flag_f = \text{“USE (32) \#5 BARS EACH END MIN”}$

----------------------- VERIFY SECTION BELOW APPLIES. IT IS NOT A REQUIREMENT FOR PRECAST WALLS OR TILT UP PANELS PER ACI 318-19 18.11.2.1-----------------------

Req'd Long Reinf at Wall Ends- ACI 318-19 18.10.2.4

If wall section hw/lw >2, requirements of ACI 318-19 18.10.2.4 must be met.

$$WallEndCheck := \text{if}\left(\frac{h_{roof}}{w_s} \ge 2, \text{"Provide Req'd Reinf at Wall End"}, \text{"Not Applicable"}\right)$$

$$WallEndCheck = \text{"Not Applicable"}$$

18.10.6.5 Where special boundary elements are not required by 18.10.6.2 or 18.10.6.3, (a) and (b) shall be satisfied:

(a) Except where V_u in the plane of the wall is less than $\lambda\sqrt{f_c'}A_{cv}$, horizontal reinforcement terminating at the edges of structural walls without boundary elements shall have a standard hook engaging the edge reinforcement or the edge reinforcement shall be enclosed in U-stirrups having the same size and spacing as, and spliced to, the horizontal reinforcement.

(b) If the maximum longitudinal reinforcement ratio at the wall boundary exceeds **$400/f_y$**, boundary transverse reinforcement shall satisfy 18.7.5.2(a) through (e) over the distance calculated in accordance with 18.10.6.4(a). The vertical spacing of transverse reinforcement at the wall boundary shall be in accordance with Table 18.10.6.5(b).

The amount of reinforcement required for leg in-plane moment is usually lower than the 400/fy ratio stated in 18.10.6.5 (OOP demands typically govern for these bars) and the minimum spacing requirements in Table 18.10.6.5b should not need to be met in that case.

Transverse Reinforcement Requirements - ACI 318-19 18.10.6.5

$$A_{s\_check} := \frac{400}{60000} \cdot (0.2 \cdot w_s) \cdot t$$

Assumes 0.2*Panel Width for N.A. depth. Input leg into SP Column if req'd to refine results.

$$V_{check} := \lambda \cdot \sqrt{\frac{f_c}{psi}} \cdot psi \cdot A_{cv}$$

Check shear against 18.10.6.5a

$$Check_{18.10.6.5a} := \text{if}\left(V_e > V_{check}, \text{"PROVIDE HOOKED HORIZ BARS OR HOOPED ENDS"}, \text{"OK"}\right)$$

$$Check_{18.10.6.5a} = \text{"OK"}$$

$$Check_{18.10.6.5b} := \text{if}\left(A_{sreq\_flex} > A_{s\_check}, \text{"PROVIDE CONFINING HOOPS @ 5'' O.C. MAX per 18.10.6.5"}, \text{"OK"}\right)$$

$$Check_{18.10.6.5b} = \text{"OK"}$$

Basic Boundary Element Check w/ Gross Section Properties - ACI 318-19 18.10.6.3

$$I_g := \frac{(w_s)^3 \cdot t}{12} \qquad A_g := \frac{w_s \cdot t}{2} \qquad S_g := \frac{t \cdot {w_s}^2}{6}$$

$$P_{u1} := P_{u1c} = 135.6\ k$$

Factored Axial Load - ASCE 7-16 2.3.6
Load Combo 6

$$\sigma_u := \frac{M_u}{S_g} + \frac{P_{u1}}{A_g}$$

Combined Stress @ Extreme Fiber

$$F_c := 0.2 \cdot f_c$$

$$DCR_\sigma := \frac{\sigma_u}{F_c} \qquad DCR_\sigma = 0.19$$

$$BasicCheck := \text{if}\left(DCR_\sigma > 1, \text{“Perform Alternate Analysis”}, \text{“Okay”}\right) \qquad BasicCheck = \text{“Okay”}$$

If Wall does not meet basic boundary element check, provide boundary elements or perform Alternate check per ACI 318-19 18.10.6.2 <u>ONLY</u> if hwcs/ w_s ≥ 2**.**

$$AlternateCheck := \left\| \begin{array}{l} \text{if } \dfrac{h_{wcs}}{w_s} \geq 2 \wedge BasicCheck = \text{“Perform Alternate Analysis”} \\ \quad \left\| \text{“Proceed”} \right. \\ \text{also if } BasicCheck = \text{“Okay”} \\ \quad \left\| \text{“Not Applicable”} \right. \\ \text{else} \\ \quad \left\| \text{“Boundary Element Necessary − Alternate Check Not Allowed”} \right. \end{array} \right.$$

$$AlternateCheck = \text{“Not Applicable”}$$

Verify Section Is Uncracked

$$y_t := \frac{w_s}{2}$$

Neutral Axis of Gross Section

$$M_{cr} := \frac{7.5 \cdot \lambda \cdot \sqrt{\dfrac{f_c}{psi}} \cdot psi \cdot I_g}{y_t}$$

Cracking Moment - ACI 318-19 19.2.3.1 Eq 19.2.3.1, 24.2.3.5 Eq 24.2.3.5

$$M_a := M_u \cdot EL_{asd\_factor}$$

Service Moment

$$M_{crCheck} := \text{if}\left(M_{cr} < M_a, \text{“Cracked; If necessary adjust relative stiffness and evaluate load”}, \text{“Uncracked”}\right)$$

$$M_{crCheck} = \text{“Uncracked”}$$

Panel 18 - Leg Design

Wall Shear Design - Leg

$Label_{leg} := \text{A}$

$leg := leg_{temp_{Label_{leg}}} = 5\ ft$ $\quad k_{leg} := k_{temp_{Label_{leg}}} = 0.50$

$$P_{u1} := \left(\left(1.2 + 0.2 \cdot S_{ds}\right) \cdot P_{tot} + \left(0.2 \cdot SL + LL\right)\ w_s\right) \left(\frac{leg + \frac{w_o}{2}}{w_s}\right) = 67.8\ k$$

ASCE 7-16 12.2.3.6 Eq 6

$$P_{u2} := \left(0.9 - 0.2 \cdot S_{ds}\right)\ P_{tot} \left(\frac{leg + \frac{w_o}{2}}{w_s}\right) = 35.4\ k$$

ASCE 7-16 12.2.3.6 Eq 7

$M_{pr} := 1325\ k \cdot ft$

Largest probable flexural strength of the section from SP Column assuming tensile stress of 1.25*Fy and Φ = 1.0

Pier Check - ACI 318-19 Table R18.10.1

If wall section is a pier, requirements of ACI 318-19 18.10.8 or 18.14 must be met

$BC_{pier} := \text{Boundary condition: Fixed−Fixed}$

Boundary Condition Pier Factor

BC_{pier} **= 0.5 for fixed-fixed,**

BC_{pier} **= 1.0 for fixed-pinned**

$$Check_1 := \text{if}\left(\frac{leg}{t_p} \le 6, \text{“Pier”}, \text{“Segment”}\right)$$

$Check_1 = \text{“Segment”}$

$$PierCheck := \text{if}\left(\frac{ht_o}{leg} \ge 2, Check_1, \text{“Segment”}\right)$$

$PierCheck = \text{“Segment”}$

Design Shear Force - ACI 318-19 18.10.3

$V_u := \rho \cdot V_{tot} \cdot k_{leg} = 12.81\ k$ — Ultimate Leg Shear (ρQ_e)

$h_{wcsleg} := h_{roof} = 35.75\ ft$ — Height of panel leg above critical section

$stories_{wcs} := 1$ — Number of Stories above Critical Section

$$n_s := \max\left(stories_{wcs}, 0.007 \cdot \frac{h_{wcsleg}}{in}\right)$$

$$\omega_v := \begin{array}{l} \text{if } \dfrac{h_{wcsleg}}{leg} < 2 \\ \quad 1.0 \\ \text{else if } n_s \le 6.0 \\ \quad 0.9 + \dfrac{n_s}{10} \\ \text{else} \\ \quad min\left(1.3 + \dfrac{n_s}{30}, 1.8\right) \end{array} = 1.2$$

Dynamic shear amplification factor, ACI 318-19 18.10.3.1.3

$M_u := V_u \cdot BC_{pier} \cdot ht_o = 89.7\ k \cdot ft$

$$M_{prratio} := \frac{M_{pr}}{M_u}$$

$$\Omega_v := \text{if}\left(\frac{h_{wcsleg}}{leg} \le 1.5, 1.0, \max\left(M_{prratio}, 1.5\right)\right)$$

$\Omega_v = 14.77$ — Overstrength factor, ACI 318-19 18.10.3.1.2

$V_{ratio} := min\left(\omega_v \cdot \Omega_v, 3\right) = 3$

$$V_{epiera} := \text{if}\left(\frac{leg}{t} > 2.5, V_u \cdot \Omega_o, \text{“N/A”}\right)$$

Alternate Ve if pier aspect ratio met
ACI 318-19 18.10.8.1a

$$V_e := \text{if}\left(PierCheck = \text{“Pier”}, \text{if}\left(V_{epiera} = \text{“N/A”}, \frac{M_{pr}}{BC_{pier} \cdot ht_o}, min\left(\frac{M_{pr}}{BC_{pier} \cdot ht_o}, V_{epiera}\right)\right), V_{ratio} \cdot V_u\right)$$

$V_e = 38.4\ k$

Design Shear Force,
For Segments: ACI 318-19 18.10.3.1
For Piers: ACI 318-19 18.7.6.1 or 18.10.8.1

Shear Wall Design

$Check_{Ve18.10.8.1} = \text{“Not applicable, proceed with design”}$

ACI 318-19 18.10.8.1 - Transverse reinforcement with seismic hooks shall be provided to resist shear determined per 18.7.6.1.1. Transverse reinf. spacing 6" max OC. Extend reinf. minimum 12" above/ below opening.

Shear Steel Requirements - ACI 318-19 18.10.2.1/11.6.1

$A_{cv} := t \cdot leg = 525\ in^2$ — Gross Area of Concrete Section

$V_{min} := \lambda \cdot \sqrt{\frac{f_c}{psi}} \cdot psi \cdot A_{cv}$ — Shear Strength to determine ρ_{min} - ACI 318-19 18.10.2.1

$\rho_{min} := \text{if}\left(V_u < V_{min}, 0.002, 0.0025\right) = 0.002$

$A_{v\_min} := (t) \cdot (12 \cdot in) \cdot (\rho_{min})$ — Minimum area of Shear Steel per Foot

Determine number of curtains of shear steel - ACI 318-19 18.10.2.2

$V_{reinf} := 2 \cdot \lambda \sqrt{\frac{f_c}{psi}} \cdot psi \cdot A_{cv}$ — Shear Strength to Determine Required Number of Layers

$Reinforcing := \text{if}\left(V_u > V_{reinf}, \text{“(2) CURTAINS REQD”}, \text{“(1) CURTAIN REQD”}\right)$

$Reinforcing = \text{“(1) CURTAIN REQD”}$

Define Shear Steel:

$No_v := \#4$ — Shear Reinforcement Bar Size

$A_{sNov} := No_{v_0}$ $\quad d_{sv} := No_{v_1}$ — Area of Shear Reinforcement Bar

$S := 18 \cdot in$ — Shear Reinforcement Spacing (12" max, typical)

$N_L := 2$ — Number of Layers **(CL = 1, EF = 2)**

$A_{vs} := N_L \cdot \frac{12 \cdot in}{S} \cdot A_{sNov}$ — Area of Shear Reinforcement per Foot

$\rho_t := \frac{A_{vs}}{(12 \cdot in) \cdot (t)}$

$A_{v\_min\_check} := \text{if}\left(A_{v\_min} \le A_{vs}, \text{“OK”}, \text{“REVISE!”}\right)$ $\quad A_{v\_min\_check} = \text{“OK”}$

$Check := \text{if}\left(V_u > V_{reinf} \wedge N_L = 1, \text{“REVISE REINF”}, \text{“OK”}\right)$ $\quad Check = \text{“OK”}$

Shear Wall Strength - ACI 318-19 Eq. 18.10.4.1

$$h_{w\_max} := \max\left(\frac{ht}{w_s}, \frac{ht_o}{leg}\right) = 2.8$$

ACI 318-19 18.10.4.2 - Ratio for the greatest of wall or segment needs to be considered

$$\alpha_c := \begin{array}{l} \text{if } h_{w\_max} < 1.5 \\ \quad 3.0 \\ \text{else if } h_{w\_max} \geq 2.0 \\ \quad 2.0 \\ \text{else} \\ \quad 2 + \dfrac{2.0 - h_{w\_max}}{0.5} \end{array} = 2$$

$$\phi V_n := \phi_{vE} \cdot \left(A_{cv} \cdot \left(\alpha_c \cdot \lambda \cdot \left(\sqrt{\frac{f_c}{psi}} \cdot psi\right) + \rho_t \cdot f_y\right)\right)$$

ACI 318-19 Eq. 18.10.4.1

Define Shear Strength Max - ACI 318-19 18.10.4.4

$$\phi V_{nmax} := \phi_{vE} \cdot \left(8 \cdot \sqrt{\frac{f_c}{psi}} \cdot psi \cdot A_{cv}\right)$$

ACI 318-19 Eq. 18.10.4.4

$check := \text{if}\left(V_e \leq \phi V_{nmax}, \text{“Proceed with Design”}, \text{“Revise!”}\right)$

$check = \text{“Proceed with Design”}$

$check := \text{if}\left(\phi V_n \leq \phi V_{nmax}, \text{“OK”}, \text{“Vn MAX exceeded, reduce reinforcement”}\right)$

$check = \text{“OK”}$

$$\phi V_n := min\left(\phi V_n, \phi V_{nmax}\right) = 87.84\ k$$

$$DCR_v := \frac{V_e}{\phi V_n}$$

$DCR_v = 0.44$

$Flag_v = \text{“USE (2) LAYER /S OF \#4 BARS AT 18''OC HORIZ MIN”}$

Leg Compression Force: Per Load Case ASCE 7-16 12.2.3.6. Default is for outer legs receiving half of total panel load in addition to seismic.

$$P_{uleg} := P_{u1} + \frac{\rho \cdot M_{ot}}{w_s - \left(\dfrac{leg}{2}\right)} = 96\ k$$

Flexural Steel - ACI 318-19 18.10.6

$No_f := \#5$ Longitudinal Reinforcement Bar Size

$A_{sNof} := No_{f_0}$ $d_{Nof} := No_{f_1}$ Area of Flexural Reinforcement Bar
Diameter of Flexural Reinforcement Bar

$N_{Lf} := 2$ Number of Layers of Longitudinal Reinforcement (CL = 1, EF = 2)

$N_f := 16$ Number of Longitudinal Reinforcement Bars per Layer

$A_{s\_leg} := N_f \cdot N_{Lf} \cdot A_{sNof} = 9.92\ in^2$ Total Area of Steel Provided in Leg

$M_u := V_u \cdot BC_{pier} \cdot ht_o = 89.7\ k \cdot ft$ Ultimate Overturning Moment ($\rho\, Q_e$)

$d_{solve} := 0.8 \cdot leg = 4\ ft$ Depth of Lever Arm, use variables from above where possible

Solver Constraints Guess Values

$A_{sreq} := 1\ in^2$

$$\phi_f \cdot A_{sreq} \cdot f_y \cdot \left(d_{solve} - \frac{A_{sreq} \cdot f_y}{0.85 \cdot f_c \cdot t \cdot 2}\right) - M_u = 0$$

$A_{sreq\_flex} := \text{Find}(A_{sreq})$

$A_{sreq\_flex} = 0.42\ in^2$ Required Area of Flexural Steel

$$A_{sreq\_ten} := \frac{\rho \cdot M_{ot}}{\phi_f \cdot (w_s - 1.5 \cdot ft) \cdot f_y} = 0.5\ in^2$$ Required Area of Steel to Resist Panel Overturning

$A_{stot} := A_{sreq\_flex} \cdot 2 + A_{sreq\_ten} = 1.34\ in^2$ Required Area of Steel for Leg Flexure and Panel Overturning

$$DCR_f := \frac{A_{stot}}{A_{s\_leg}} = 0.14$$

$DCR_f = 0.14$

$Flag_f =$ "USE (32) #5 BARS EACH END MIN"

----------------------- VERIFY SECTION BELOW APPLIES. IT IS NOT A REQUIREMENT FOR PRECAST WALLS OR TILT UP PANELS PER ACI 318-19 18.11.2.1-----------------------

Req'd Long Reinf at Wall Ends- ACI 318-19 18.10.2.4

If wall section hw/lw >2, requirements of ACI 318-19 18.10.2.4 must be met.

$$WallEndCheck := \text{if}\left(\frac{ht_o}{leg}, \text{"Provide Req'd Reinf at Wall End"}, \text{"Not Applicable"}\right)$$

$$WallEndCheck = \text{"Provide Req'd Reinf at Wall End"}$$

$N_{f18.10.2.4} := 3$ Number of Longitudinal Reinforcement Bars per Layer at Each End (0.15*lw)

$A_{s\_end} := N_{f18.10.2.4} \cdot N_{Lf} \cdot A_{sNof}$ Area of Longitudinal Steel at Wall End

$$\rho_{lreq} := 6 \cdot \sqrt{\frac{f_c}{psi}} \cdot \frac{psi}{f_y}$$

$A_{lreq} := \rho_{lreq} \cdot 0.15 \cdot leg \cdot t$ Required Steel Area within 0.15 lw from end of Wall

$Check := \text{if}\left(A_{s\_end} > A_{lreq}, \text{"OK"}, \text{"More Long Steel Req'd!"}\right)$ $Check = \text{"OK"}$

Transverse Reinforcement Requirements- ACI 318-19 18.10.6.5

$$A_{s\_check} := \frac{400}{\frac{f_y}{psi}} \cdot (0.2 \cdot leg) \cdot t$$

Assumes 0.2*leg for N.A. depth. Input leg into SP Column if req'd to refine results.

$$V_{check} := \lambda \cdot \sqrt{\frac{f_c}{psi}} \cdot psi \cdot A_{cv}$$

Check shear against 18.10.6.5a

$$Check_{18.10.6.5a} := \text{if}\left(V_u > V_{check}, \text{"PROVIDE HOOKED HORIZ BARS OR CLOSED HOOPS"}, \text{"OK"}\right)$$

$$Check_{18.10.6.5a} = \text{"OK"}$$

$$Check_{18.10.6.5b} := \text{if}\left(A_{sreq\_flex} > A_{s\_check}, \text{"PROVIDE CONFINING HOOPS @ 5'' O.C. MAX per 18.10.6.5"}, \text{"OK"}\right)$$

$$Check_{18.10.6.5b} = \text{"OK"}$$

Basic Boundary Element Check w/ Gross Section Properties - ACI 318-19 18.10.6.3

$$I_g := \frac{leg^3 \cdot t}{12} \qquad A_g := leg \cdot t \qquad S_g := \frac{t \cdot leg^2}{6}$$

$$P_u := P_{uleg}$$

Factored Axial Load - Load Combination ASCE 7-16 12.2.3.6 Eq. 6

$$\sigma_u := \frac{M_u}{S_g} + \frac{P_{uleg}}{A_g}$$

Combined Stress at Extreme Fiber

$$F_c := 0.2 \cdot f_c$$

$$DCR_\sigma := \frac{\sigma_u}{F_c}$$

$$DCR_\sigma = 0.48$$

$$BasicCheck := \text{if}\left(F_c > \sigma_u, \text{"Okay"}, \text{"Perform Alternate Analysis"}\right)$$

$$BasicCheck = \text{"Okay"}$$

If Wall does not meet basic boundary element check, provide boundary elements or perform Alternate check per ACI 318-19 18.10.6.2 <u>ONLY</u> if hwcs/ w_s > 2.

$$AlternateCheck := \left\| \begin{array}{l} \text{if } \dfrac{h_{wcsleg}}{leg} \ge 2 \land BasicCheck = \text{"Perform Alternate Analysis"} \\ \quad \| \text{"Proceed"} \\ \text{also if } BasicCheck = \text{"Okay"} \\ \quad \| \text{"Not Applicable"} \\ \text{else} \\ \quad \| \text{"Boundary Element Necessary – Alternate Check Not Allowed"} \end{array} \right.$$

$$AlternateCheck = \text{"Not Applicable"}$$

Verify Section Uncracked

$$y_t := \frac{leg}{2}$$

Neutral Axis of Gross Section

$$M_{cr} := \frac{7.5 \cdot \lambda \cdot \sqrt{\dfrac{f_c}{psi}} \cdot psi \cdot I_g}{y_t}$$

Cracking Moment - ACI 318-19 19.2.3.1 Eq 19.2.3.1, 24.2.3.5 Eq 24.2.3.5

$$M_a := M_u \cdot EL_{asd\_factor}$$

Service Moment

$$M_{crCheck} := \text{if}\left(M_{cr} < M_a, \text{"Cracked; If necessary adjust relative stiffness and evaluate load"}, \text{"Uncracked"}\right)$$

$$M_{crCheck} = \text{"Uncracked"}$$

------------------------ VERIFY SECTION BELOW APPLIES------------------------

Pier Shear Reinforcing (Leg A) - ACI 318-19 18.10.8 - This section applies where the wall segment is a pier and does not meet the exceptions listed in the code.

$PierCheck = \text{"Segment"}$

ACI 318-19 18.10.8.2 - Transverse reinforcement with seismic hooks shall be provided to resist shear determined per 18.7.6.1.1. Transverse reinf. spacing 6" max OC. Extend reinf. minimum 12" above/below opening.

$V_{Check\_18.10.8.2} = \text{"n/a"}$ — Minimum shear requirement check per ACI 318-19 18.10.8.2

------------------------ VERIFY SECTION BELOW APPLIES------------------------

Slab Shear Dowel Bars (Leg A) - Shear Friction per ACI 318-19 22.9.4

$\mu := 0.6$ — Coefficient of Friction - ACI 318-19 22.9.4.2

$L_{dowel} := leg$ — Length of wall including dowels (leg length at storefront condition)

$No_{dowel} := \#5$ — Dowel Reinforcement Bar Size

$A_{sNodowel} := No_{dowel}$

$N_{dowel} := 3$ — Number of Dowels per Leg - (4) Min

$A_{vf} := A_{sNodowel} \cdot N_{dowel} = 0.93\ in^2$ — Area of Shear Friction Steel Across Panel

$\phi V_{vf\_n} := \phi_{vE} \cdot \left(\mu \cdot A_{vf} \cdot f_y\right)$ — Allowable Shear Capacity - ACI 318-19 22.9.4.2

$V_u := \rho \cdot V_{tot} \cdot k_a = 12.81\ k$ — Ultimate Panel Shear ($\rho\, Q_e$)

$$DCR_{vf} := \frac{V_u}{\phi V_{vf\_n}} \qquad DCR_{vf} = 0.64$$

$$flag := \text{if}\left(\phi V_{vf\_n} \ge V_u, \text{"OK"}, \text{"NG!"}\right) \qquad flag = \text{"OK"}$$

Shear Wall Design

Protected Area

Purpose Statement: The purpose of this calculation is in-plane design of concrete tilt panels as special reinforced shear walls.

Referenced Standards:
IBC 2021
ASCE 7-16
ACI 318-19
Amrhein/Yellow Book method
NEHRP Tech Brief 6

Inputs
Verify
Outputs

PROJECT PARAMETERS

$f_c := 4000 \cdot psi$	Concrete Strength
$q_{all} := 2500 \cdot psf$	Allowable Soil Pressure
$\gamma_{temp} := \frac{4}{3}$	Short-term Loading Increase for Soil
$q_{sei} := \gamma_{temp} \cdot q_{all}$	Short-term Allowable Soil Pressure
$DL_{sei} := 10.9 \cdot psf$	Roof DL
$DL_{flr} := 25 \cdot psf$	Intermediate Floor DL
$t_{SOG} := 7 \cdot in$	Slab-on-Grade Thickness
$\phi_{vE} := 0.6$	Seismic shear strength reduction factor - ACI 318-19 21.2.4.1
$\phi_f := 0.9$	Flexural Strength Reduction Factor
$\phi_t := 0.9$	Tensile Strength Reduction Factor
$EL_{asd\_factor} := 0.7$	ASD Factor
$f_y := 60 \cdot ksi$	Reinforcing Yield Strength
$E_s := 29000 \cdot ksi$	Steel Modulus of Elasticity
$w_c := 150 \cdot pcf$	Unit Weight of Concrete
$\lambda :=$ Normal Weight	Concrete weight modification factor - ACI 318-19 Table 19.2.4.1 (b)

Shear Wall Design

$S_{ds} := 0.843$ Seismic Coefficient

$\rho := 1.0$ Redundancy Factor - ASCE 7-16 12.3.4

$I_e := 1.0$ Seismic Importance Factor - ASCE 7-16 Table 1.5-2

Design Coefficients and Factors for Special Reinforced Concrete Shear Walls - ASCE 7-16 12.2

$R := 5$ Response Modification Factor

$C_d := 5$ Deflection Amplification Factor

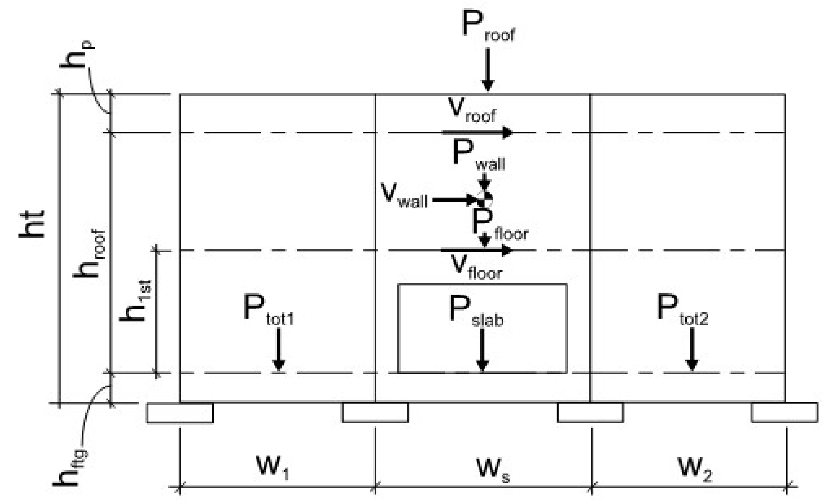
$\Omega_o := 2.0$ Special Concrete Shear Wall Overstrength Factor

$$C_s := \frac{S_{ds}}{\left(\frac{R}{I_e}\right)} = 0.1686$$

Seismic Coefficient (C_s) - ASCE 7-16 12.8.1

Shear Wall Design

318-19 Panel w/ (2) Opening - Panel 17 - North



Shear Panel Input (w_s)

$w_s := 26 \cdot ft$	Panel Width
$h_p := 0 \cdot ft$	Parapet Height
$h_{roof} := 35.75 \cdot ft$	Height of Roof Above FF
$h_{1st} := 0 \cdot ft$	Height of Floor Above FF
$h_{ftg} := 5 \cdot ft$	FF to T/FTG Height
$ht := h_p + h_{roof} + h_{ftg} = 40.8\ ft$	Total Panel Height (T/FTG to T/PARAPET)
$ht_{o1} := 10 \cdot ft$	Opening 1 Height
$ht_{o2} := 10\ ft$	Opening 2 Height
$w_{o1} := 9 \cdot ft$	Opening 1 Width
$w_{o2} := 9 \cdot ft$	Opening 1 Width
$lega := 2\ ft$	Leg A Width
$legb := 4\ ft$	Leg B Width
$legc := 2\ ft$	Leg C Width
$t_p := 9.5 \cdot in$	Panel Thickness

$r_v := 0.75 \cdot in$ Reveal Depth

$t := t_p - r_v = 8.8\ in$ Effective Panel Thickness

$Wt_p := w_c \cdot t_p$ Panel Wt. (psf)

Relative Panel Leg Stiffnesses (w_s)

Amrhein/Yellow book method

$BC_{lega} :=$ Boundary Condition: Fixed–Fixed

$BC_{legb} :=$ Boundary Condition: Fixed–Fixed

$BC_{legc} :=$ Boundary Condition: Fixed–Fixed

Leg A & B boundary conditions
BC_{leg} = 1 for fixed-fixed
BC_{leg} = 4 for fixed-pinned

$hd_{ratioa} := \frac{ht_{o1}}{lega} = 5$ Leg A aspect ratio

$hd_{ratiob} := \frac{\max\left(ht_{o1}, ht_{o2}\right)}{legb} = 2.5$ Leg B aspect ratio

$hd_{ratioc} := \frac{ht_{o2}}{legc} = 5$ Leg C aspect ratio

$P_k := 100000\ lb$ $t_k := 1\ in$ $E_k := 1000000\ psi$

$$\Delta_{ka} := \frac{P_k}{E_k \cdot t_k} \cdot \left(BC_{lega} \cdot \left(hd_{ratioa}\right)^3 + 3 \cdot \left(hd_{ratioa}\right)\right)$$

$$\Delta_{kb} := \frac{P_k}{E_k \cdot t_k} \cdot \left(BC_{legb} \cdot \left(hd_{ratiob}\right)^3 + 3 \cdot \left(hd_{ratiob}\right)\right)$$

$$\Delta_{kc} := \frac{P_k}{E_k \cdot t_k} \cdot \left(BC_{legc} \cdot \left(hd_{ratioc}\right)^3 + 3 \cdot \left(hd_{ratioc}\right)\right)$$

$R_{ka} := \frac{1}{\Delta_{ka}}$ $R_{kb} := \frac{1}{\Delta_{kb}}$ $R_{kc} := \frac{1}{\Delta_{kc}}$

$k_a := \frac{R_{ka}}{R_{ka} + R_{kb} + R_{kc}} = 0.124$ Leg A Relative Stiffness

$k_b := \frac{R_{kb}}{R_{ka} + R_{kb} + R_{kc}} = 0.752$ Leg B Relative Stiffness

$$k_c := \frac{R_{kc}}{R_{ka} + R_{kb} + R_{kc}} = 0.124$$ Leg B Relative Stiffness

$$Stiffness := \text{if}\left(\left|1.0 - \left(k_a + k_b + k_c\right)\right| > 0.0001, \text{“Check”}, \text{“OK”}\right)$$ $Stiffness = \text{“OK”}$

Adjacent Panel Input (w_1)	**Adjacent Panel Input (w_2)**	
$w_1 := w_s$	$w_2 := w_s$	Adjacent Panel Width
$h_{p1} := h_p$	$h_{p2} := h_p$	Parapet Height
$h_{roof1} := h_{roof}$	$h_{roof2} := h_{roof}$	Height of Roof Above FF
$h_{ftg1} := h_{ftg}$	$h_{ftg2} := h_{ftg}$	FF to T/FTG Height
$ht_1 := h_{p1} + h_{roof1} + h_{ftg1} = 40.8\ ft$	$ht_2 := h_{p2} + h_{roof2} + h_{ftg2} = 40.8\ ft$	Total Panel Height (T/FTG to T/PARAPET)
$t_{p1} := t_p$	$t_{p2} := t_p$	Panel Thickness
$Ao_1 := 2 \cdot 10 \cdot ft \cdot 9 \cdot ft$	$Ao_2 := 2 \cdot 10 \cdot ft \cdot 9 \cdot ft$	Area of Openings

Loading:

Seismic (1.0 Q_e)

$V_{roof} := 8.87 \cdot k$ Seismic Shear @ Roof - ρ not included

$V_{floor} := 0 \cdot k$ Seismic Shear @ Floor

$V_{wall} := \left(\left(ht - h_{ftg}\right) \cdot w_s - \left(ht_{o1} \cdot w_{o1} + ht_{o2} \cdot w_{o2}\right)\right) \cdot Wt_p \cdot C_s$ Seismic Shear due to Wall Panel Mass

$V_{tot} := V_{roof} + V_{wall} + V_{floor} = 23.9\ k$ Total Seismic Shear

$M_{ot\_roof} := V_{roof} \cdot h_{roof}$ Overturning Moment due to Roof Shear

$M_{ot\_floor} := V_{floor} \cdot h_{1st}$ Overturning Moment due to Floor Shear

$$M_{ot\_wall} := V_{wall} \cdot \frac{\left(ht - h_{ftg}\right)}{2}$$ Overturning Moment Due to Panel Mass

$M_{ot} := M_{ot\_roof} + M_{ot\_floor} + M_{ot\_wall} = 585.3\ k \cdot ft$ Total Overturning Moment @ Wall Panel

Gravity:

Shear Panel (w_s):

$l_{roof\_trib} := \frac{60}{2} \cdot ft$ Length of Roof Tributary to Panel

$l_{floor\_trib} := \frac{0}{2} \cdot ft$ Length of Floor Tributary to Panel

$SL := 0.6\ klf$ (Construction LL) Roof snow load tributary to panel

$LL := 0.0\ klf$ Floor live load tributary to panel

$P_{roof} := DL_{sei} \cdot l_{roof\_trib} \cdot w_s$ Roof DL

$P_{floor} := DL_{flr} \cdot l_{floor\_trib} \cdot w_s$ Floor DL

$P_{wall} := \left(ht \cdot w_s - \left(ht_{o1} \cdot w_{o1} + ht_{o2} \cdot w_{o2}\right)\right) \cdot Wt_p$ Wall Panel DL

$P_{slab} := w_c \cdot t_{SOG} \cdot 8 \cdot ft \cdot w_s$ Slab-on-Grade DL (assumes 8' width of slab over width of panel available to resist OT)

$P_{tot} := P_{roof} + P_{floor} + P_{wall} = 112.9\ k$ Total DL (without slab @ wall considered)

Panel (w_1):

$P_{roof1} := P_{roof}$ Roof DL

$P_{floor1} := P_{floor}$ Floor DL

$P_{wall1} := \left(w_1 \cdot ht_1 - Ao_1\right) \cdot t_{p1} \cdot w_c$ Wall Panel DL

$P_{slab1} := P_{slab} \cdot \left(\frac{w_1}{w_s}\right)$ Slab-on-Grade DL

$P_{tot1} := P_{roof1} + P_{floor1} + P_{wall1} + P_{slab1} = 131.1\ k$ Total DL (including slab weight)

Panel (w_2):

$P_{roof2} := P_{roof}$ Roof DL

$P_{floor2} := P_{floor}$ Floor DL

$P_{wall2} := \left(w_2 \cdot ht_2 - Ao_2\right) \cdot t_{p2} \cdot w_c$ Wall Panel DL

$P_{slab2} := P_{slab} \cdot \left(\frac{w_2}{w_s}\right)$ Slab-on-Grade DL

$P_{tot2} := P_{roof2} + P_{floor2} + P_{wall2} + P_{slab2} = 131.1\ k$ Total DL (including slab weight)

Resisting Moment:

$M_{resist} := (P_{tot} + P_{slab}) \cdot \frac{w_s}{2} = 1704.9\ k \cdot ft$ Resisting Moment at Shear Panel

Load Combinations:

ASCE 7-16 2.3.6

LRFD 6: (1.2+0.2Sds)D+1.0ρ Q_e + L + 0.2S

LRFD 7: (0.9 - 0.2Sds)D + 1.0ρ Q_e (Resisting Moment of Slab-on-Grade)

IBC 2021 1605.2 - Foundations

ASD 16-5: D + L + S + E/1.4 (Foundation Bearing)

ASD 16-6: 0.9D + E/1.4 (Foundation Uplift/Weight. Ev = 0 for fdn size, Ev included for holdowns)

Panel Hold Down - IBC 2021 1605.2 - Alternative Load Combination ASD 16-6

$M_{ot\_IBC} := \rho \cdot \frac{M_{ot}}{1.4}$ ASD Overturning Moment

$M_{res\_IBC} := \left(0.9 - \frac{0.2 \cdot S_{ds}}{1.4}\right) \cdot M_{resist}$ ASD Resisting Moment

$No_{hd} := \#5$ Rebar Size No.

$N_{hd} := 2$ Number of Bars in Hold Down

$L_{hd} := \text{if}\left(No_{hd_0} = 0.31\ in^2, L_{HD5_{N_{hd}}}, L_{HD6_{N_{hd}}}\right) = 8\ in$ Length of Hold Down

$A_{shd} := No_{hd_0} \cdot N_{hd}$ Area of Steel in Hold Down

$T_{hd} := \frac{M_{ot\_IBC} - M_{res\_IBC}}{w_s - (1 \cdot ft + 0.5 \cdot L_{hd})} = -36.9\ k$ ASD Tension at Hold Down

$A_{shdreq} := \frac{\left(\frac{T_{hd}}{EL_{asd\_factor}}\right)}{\phi_t \cdot f_y} = -0.98\ in^2$ Required Area of Steel

$DCR_{hd} := \frac{A_{shdreq}}{A_{shd}}$ $DCR_{hd} = -1.6$

$hd\_Results =$ "NO HOLDOWNS REQD"

Footing Uplift and Seismic Bearing Checks

$l_f := \frac{w_s}{2}$ Length of Footing

$w_f := 2.5 \cdot ft$ Width of Footing

$d_f := 1 \cdot ft$ Depth of Footing (Use 2'-0" Min @ hold down, see footing calcs)

$P_{ftg} := w_c \cdot l_f \cdot w_f \cdot d_f$ Footing Weight

Required Footing Weight - Load Combination ASD 16-6

$M_{res\_uplift} := M_{res\_IBC} + \left(0.9 \cdot \left(min\left(\frac{P_{tot1}}{2}, \frac{P_{tot2}}{2}\right) \cdot w_s\right)\right)$ Factored Resisting Moment (Including contribution of slab & adjacent panel)

$M_{res\_uplift} = 2863.42\ k \cdot ft$

$Wt_{ftg\_uplift} := \frac{\left(M_{ot\_IBC} - M_{res\_uplift}\right)}{w_s - \left(1\ ft + 0.5 \cdot L_{hd}\right)}$ Required Footing Weight

$Wt_{ftg\_uplift} = -99.13\ k$

$flag_w := \text{if}\left(Wt_{ftg\_uplift} \le P_{ftg}, \text{“OK”}, \text{“NG!!!”}\right)$ $flag_w = \text{“OK”}$

Check Wall Segment Above/Below Opening

Wall Shear Design

$P_{u1c} := (1.2 + 0.2 \cdot S_{ds}) \cdot P_{tot} + (0.5\ SL + LL)\ w_s = 162\ k$

Factored Wall Axial Forces (for Mpr calculation)
ASCE 7-16 12.2.3.6 Eq. 6

$P_{u2c} := (0.9 - 0.2\ S_{ds}) \cdot P_{tot} = 82.61\ k$

ASCE 7-16 12.2.3.6 Eq. 7

$M_{pr} := 12440\ k \cdot ft$

$M_u := \rho \cdot M_{ot} = 585\ k \cdot ft$

Largest probable flexural strength of the section from SP Column assuming tensile stress of 1.25*Fy and Φ = 1.0

$V_u := \rho \cdot V_{tot} = 23.9\ k$

Ultimate Panel Shear ($\rho\, Q_e$)

$h_{wcs} := 35.75\ ft$

Height of wall measured from critical section to parapet (see NTE diagram)

Pier Check - ACI 318-19 Table R18.10.1

If wall section is a pier, requirements of ACI 318-19 18.10.8 or 18.14 must be met.

$$Check_1 := \text{if}\left(\frac{w_s}{t_p} \le 6, \text{“Pier”}, \text{“Segment”}\right)$$

$$PierCheck := \text{if}\left(\frac{h_{wcs}}{w_s} \ge 2, Check_1, \text{“Segment”}\right)$$

$PierCheck = \text{“Segment”}$

Design Shear Force - ACI 318-19 18.10.3

$stories_{wcs} := 1$

Number of Stories above Critical Section

$$n_s := \max\left(stories_{wcs}, 0.007 \cdot \frac{h_{wcs}}{in}\right)$$

$$\omega_v := \begin{Vmatrix} \text{if } \dfrac{h_{wcs}}{w_s} < 2 \\ \quad \| \ 1.0 \\ \text{else if } n_s \le 6.0 \\ \quad \| \ 0.9 + \dfrac{n_s}{10} \\ \text{else} \\ \quad \| \ min\left(1.3 + \dfrac{n_s}{30}, 1.8\right) \end{Vmatrix} = 1$$

Dynamic shear amplification factor, ACI 318-19 18.10.3.1.3

Shear Wall Design

$$M_{prratio} := \frac{M_{pr}}{M_u}$$

$$\Omega_v := \text{if}\left(\frac{h_{wcs}}{w_s} \le 1.5, 1.0, \max\left(M_{prratio}, 1.5\right)\right)$$

$$\Omega_v = 1$$

Shear Overstrength factor, ACI 318-19 18.10.3.1.2

$$V_{ratio} := min\left(\omega_v \cdot \Omega_v, 3\right) = 1$$

$$V_e := V_{ratio} \cdot V_u$$

$$V_{epiera} := \text{if}\left(\frac{w_s}{t} > 2.5, V_u \cdot \Omega_o, \text{"N/A"}\right)$$

Alternate Ve if pier aspect ratio met
ACI 318-19 18.10.8.1a

$$V_e := \text{if}\left(PierCheck = \text{"Pier"}, \text{if}\left(V_{epiera} = \text{"N/A"}, \frac{M_{pr}}{h_{roof}}, min\left(\frac{M_{pr}}{h_{roof}}, V_{epiera}\right)\right), V_{ratio} \cdot V_u\right)$$

$$V_e = 23.9\ k$$

Design Shear Force,
For Segments: ACI 318-19 18.10.3.1
For Piers: ACI 318-19 18.7.6.1

Shear Steel Requirements - ACI 318-19 18.10.2.1/11.6.1

$$A_{cv} := t \cdot w_s = 2730\ in^2$$

Gross Area of Concrete Section

$$V_{min} := \lambda \cdot \sqrt{\frac{f_c}{psi}} \cdot psi \cdot A_{cv}$$

Shear Strength to determine ρ_{min}

$$\rho_{min} := \text{if}\left(V_u < V_{min}, 0.002, 0.0025\right) = 0.002$$

$$A_{v\_min} := t \cdot (12 \cdot in) \cdot \rho_{min}$$

Minimum area of Shear Steel per Foot

Determine number of curtains of shear steel - ACI 318-19 18.10.2.2

$$V_{reinf} := 2 \cdot \lambda \cdot \sqrt{\frac{f_c}{psi}} \cdot psi \cdot A_{cv}$$

Shear Strength to Determine Required Number of Layers

$$Reinforcing := \text{if}\left(V_u > V_{reinf} \vee \frac{ht}{w_s} \ge 2, \text{"(2) CURTAINS REQD"}, \text{"(1) CURTAIN REQD"}\right)$$

$$Reinforcing = \text{"(1) CURTAIN REQD"}$$

Define Shear Steel:

$No_v := \#4$ — Shear Reinforcement Bar Size

$A_{sNov} := No_{v_0}$ $\quad d_{sv} := No_{v_1}$ — Area of Shear Reinforcement Bar

$S := 9 \cdot in$ — Shear Reinforcement Spacing (12" max, typical)

$N_L := 1$ — Number of Layers **(CL = 1, EF = 2)**

$A_{vs} := N_L \cdot \dfrac{12 \cdot in}{S} \cdot A_{sNov}$ — Area of Shear Reinforcement per Foot

$$\rho_t := \frac{A_{vs}}{(12 \cdot in) \cdot (t)}$$

$A_{v\_min\_check} := \text{if}\left(A_{v\_min} \le A_{vs}, \text{“OK”}, \text{“REVISE!”}\right)$ $\quad A_{v\_min\_check} = \text{“OK”}$

$Check := \text{if}\left(V_u > V_{reinf} \wedge N_L = 1, \text{“REVISE REINF”}, \text{“OK”}\right)$ $\quad Check = \text{“OK”}$

Shear Wall Strength - ACI 318-19 Eq. 18.10.4.1

$$\alpha_c := \begin{cases} 3.0 & \text{if } \dfrac{ht}{w_s} \le 1.5 \\ 2.0 & \text{else if } \dfrac{ht}{w_s} \ge 2.0 \\ 2 + \dfrac{2.0 - \dfrac{ht}{w_s}}{0.5} & \text{else} \end{cases} = 2.87$$

$$\phi V_n := \phi_{vE} \cdot \left(A_{cv} \cdot \left(\alpha_c \cdot \lambda \cdot \left(\sqrt{\frac{f_c}{psi}} \cdot psi\right) + \rho_t \cdot f_y\right)\right)$$

ACI 318-19 Eq. 18.10.4.1

Define Shear Strength Max - ACI 318-19 18.10.4.4

$$\phi V_{nmax} := \phi_{vE} \cdot \left(8 \cdot \sqrt{\frac{f_c}{psi}} \cdot psi \cdot A_{cv}\right)$$

ACI 318-19 Eq. 18.10.4.4

$check := \text{if}\left(V_e \le \phi V_{nmax}, \text{“Proceed with Design”}, \text{“Revise!”}\right)$ $\quad check = \text{“Proceed with Design”}$

$check := \text{if}\left(\phi V_n \le \phi V_{nmax}, \text{“OK”}, \text{“Vn MAX exceeded, reduce reinforcement”}\right)$ $check = \text{“OK”}$

$DCR_v := \dfrac{V_e}{min\left(\phi V_n, \phi V_{nmax}\right)}$ $DCR_v = 0.04$

$Flag_v = \text{“USE (1) LAYER /S OF \#4 BARS AT 9”OC HORIZ MIN”}$

Flexural Steel - ACI 318-19 18.10.6

$No_f := \#5$ Longitudinal Reinforcement Bar Size

$A_{sNof} := No_{f_0}$ $d_{Nof} := No_{f_1}$

$N_{Lf} := 2$ Number of Layers of Longitudinal Reinforcement (CL = 1, EF = 2) at Each End

$N_f := 8$ Number of Longitudinal Reinforcement Bars per Layer at Each End

$d_{solve} := 0.8 \cdot w_s = 20.8\ ft$ Depth of Lever Arm

Solve Constraints Guess Values

$A_{sreq} := 1\ in^2$

$\phi_f \cdot A_{sreq} \cdot f_y \cdot \left(d_{solve} - \dfrac{A_{sreq} \cdot f_y}{0.85 \cdot f_c \cdot t \cdot 2}\right) - M_u = 0$

$A_{sreq\_flex} := \text{Find}\left(A_{sreq}\right)$

$A_{sreq\_flex} = 0.52\ in^2$ Required Area of Flexural Steel

$A_{s\_long} := N_f \cdot N_{Lf} \cdot A_{sNof} = 4.96\ in^2$ Area of Flexural Steel

$DCR_f := \dfrac{A_{sreq\_flex}}{A_{s\_long}}$ $DCR_f = 0.11$

$Flag_f = \text{“USE (16) \#5 BARS EACH END MIN”}$

----------------------- VERIFY SECTION BELOW APPLIES. IT IS NOT A REQUIREMENT FOR PRECAST WALLS OR TILT UP PANELS PER ACI 318-19 18.11.2.1-----------------------

Req'd Long Reinf at Wall Ends- ACI 318-19 18.10.2.4

If wall section hw/lw >2, requirements of ACI 318-19 18.10.2.4 must be met.

$$WallEndCheck := \text{if}\left(\frac{h_{roof}}{w_s} \geq 2, \text{“Provide Req'd Reinf at Wall End”}, \text{“Not Applicable”}\right)$$

$$WallEndCheck = \text{“Not Applicable”}$$

18.10.6.5 Where special boundary elements are not required by 18.10.6.2 or 18.10.6.3, (a) and (b) shall be satisfied:

(a) Except where V_u in the plane of the wall is less than $\lambda\sqrt{f_c'}A_{cv}$, horizontal reinforcement terminating at the edges of structural walls without boundary elements shall have a standard hook engaging the edge reinforcement or the edge reinforcement shall be enclosed in U-stirrups having the same size and spacing as, and spliced to, the horizontal reinforcement.

(b) If the maximum longitudinal reinforcement ratio at the wall boundary exceeds $400/f_y$, boundary transverse reinforcement shall satisfy 18.7.5.2(a) through (e) over the distance calculated in accordance with 18.10.6.4(a). The vertical spacing of transverse reinforcement at the wall boundary shall be in accordance with Table 18.10.6.5(b).

The amount of reinforcement required for leg in-plane moment is usually lower than the 400/fy ratio stated in 18.10.6.5 (OOP demands typically govern for these bars) and the minimum spacing requirements in Table 18.10.6.5b should not need to be met in that case.

Transverse Reinforcement Requirements - ACI 318-19 18.10.6.5

$$A_{s\_check} := \frac{400}{60000} \cdot \left(0.2 \cdot w_s\right) \cdot t$$

Assumes 0.2*Panel Width for N.A. depth. Input leg into SP Column if req'd to refine results.

$$V_{check} := \lambda \cdot \sqrt{\frac{f_c}{psi}} \cdot psi \cdot A_{cv}$$

Check shear against 18.10.6.5a

$$Check_{18.10.6.5a} := \text{if}\left(V_e > V_{check}, \text{“PROVIDE HOOKED HORIZ BARS OR HOOPED ENDS”}, \text{“OK”}\right)$$

$$Check_{18.10.6.5a} = \text{“OK”}$$

$$Check_{18.10.6.5b} := \text{if}\left(A_{sreq\_flex} > A_{s\_check}, \text{“PROVIDE CONFINING HOOPS @ 5'' O.C. MAX per 18.10.6.5”}, \text{“OK”}\right)$$

$$Check_{18.10.6.5b} = \text{“OK”}$$

Basic Boundary Element Check w/ Gross Section Properties - ACI 318-19 18.10.6.3

$$I_g := \frac{(w_s)^3 \cdot t}{12} \qquad A_g := \frac{w_s \cdot t}{2} \qquad S_g := \frac{t \cdot w_s^{\,2}}{6}$$

$$P_{u1} := P_{u1c} = 162.4\ k$$

Factored Axial Load - ASCE 7-16 2.3.6
Load Combo 6

$$\sigma_u := \frac{M_u}{S_g} + \frac{P_{u1}}{A_g}$$

Combined Stress @ Extreme Fiber

$$F_c := 0.2 \cdot f_c$$

$$DCR_\sigma := \frac{\sigma_u}{F_c} \qquad DCR_\sigma = 0.21$$

$$BasicCheck := \text{if}\left(DCR_\sigma > 1, \text{“Perform Alternate Analysis”}, \text{“Okay”}\right) \qquad BasicCheck = \text{“Okay”}$$

If Wall does not meet basic boundary element check, provide boundary elements or perform Alternate check per ACI 318-19 18.10.6.2 ONLY if hwcs/ w_s ≥ 2**.**

$$AlternateCheck := \begin{array}{l} \text{if } \dfrac{h_{wcs}}{w_s} \ge 2 \wedge BasicCheck = \text{“Perform Alternate Analysis”} \\ \quad \text{“Proceed”} \\ \text{also if } BasicCheck = \text{“Okay”} \\ \quad \text{“Not Applicable”} \\ \text{else} \\ \quad \text{“Boundary Element Necessary – Alternate Check Not Allowed”} \end{array}$$

$$AlternateCheck = \text{“Not Applicable”}$$

Verify Section Is Uncracked

$$y_t := \frac{w_s}{2}$$

Neutral Axis of Gross Section

$$M_{cr} := \frac{7.5 \cdot \lambda \cdot \sqrt{\dfrac{f_c}{psi}} \cdot psi \cdot I_g}{y_t}$$

Cracking Moment - ACI 318-19 19.2.3.1 Eq 19.2.3.1, 24.2.3.5 Eq 24.2.3.5

$$M_a := M_u \cdot EL_{asd\_factor}$$

Service Moment

$$M_{crCheck} := \text{if}\left(M_{cr} < M_a, \text{“Cracked; If necessary adjust relative stiffness and evaluate load”}, \text{“Uncracked”}\right)$$

$$M_{crCheck} = \text{“Uncracked”}$$

Panel 17 - Leg A (2'-0") Design

Wall Shear Design - Leg

$Label_{leg} := \text{A}$

$leg := leg_{temp_{Label_{leg}}} = 2\ ft$

$k_{leg} := k_{temp_{Label_{leg}}} = 0.12$

$w_o := w_{temp_{Label_{leg}}} = 9\ ft$

$ht_o := ht_{temp_{Label_{leg}}} = 10\ ft$

$$P_{u1} := \left(\left(1.2 + 0.2 \cdot S_{ds}\right) \cdot P_{tot} + \left(0.2 \cdot SL + LL\right)\ w_s\right) \left(\frac{leg + \frac{w_o}{2}}{w_s}\right) = 39.4\ k$$

ASCE 7-16 12.2.3.6 Eq 6

$$P_{u2} := \left(0.9 - 0.2 \cdot S_{ds}\right)\ P_{tot} \left(\frac{leg + \frac{w_o}{2}}{w_s}\right) = 20.7\ k$$

ASCE 7-16 12.2.3.6 Eq 7

$M_{pr} := 269\ k \cdot ft$

Largest probable flexural strength of the section from SP Column assuming tensile stress of 1.25*Fy and Φ = 1.0

Pier Check - ACI 318-19 Table R18.10.1

If wall section is a pier, requirements of ACI 318-19 18.10.8 or 18.14 must be met

$BC_{pier} := \text{Boundary condition: Fixed–Fixed}$

Boundary Condition Pier Factor

BC_{pier} **= 0.5 for fixed-fixed,**

BC_{pier} **= 1.0 for fixed-pinned**

$$Check_1 := \text{if}\left(\frac{leg}{t_p} \le 6, \text{“Pier”}, \text{“Segment”}\right)$$

$Check_1 = \text{“Pier”}$

$$PierCheck := \text{if}\left(\frac{ht_o}{leg} \ge 2, Check_1, \text{“Segment”}\right)$$

$PierCheck = \text{“Pier”}$

Design Shear Force - ACI 318-19 18.10.3

$V_u := \rho \cdot V_{tot} \cdot k_{leg} = 2.96\ k$ — Ultimate Leg Shear (ρQ_e)

$h_{wcsleg} := h_{roof} = 35.75\ ft$ — Height of panel leg above critical section

$stories_{wcs} := 1$ — Number of Stories above Critical Section

$$n_s := \max\left(stories_{wcs}, 0.007 \cdot \frac{h_{wcsleg}}{in}\right)$$

$$\omega_v := \begin{array}{l} \text{if } \dfrac{h_{wcsleg}}{leg} < 2 \\ \quad 1.0 \\ \text{else if } n_s \le 6.0 \\ \quad 0.9 + \dfrac{n_s}{10} \\ \text{else} \\ \quad min\left(1.3 + \dfrac{n_s}{30}, 1.8\right) \end{array} = 1.2$$

Dynamic shear amplification factor,
ACI 318-19 18.10.3.1.3

$M_u := V_u \cdot BC_{pier} \cdot ht_o = 14.8\ k \cdot ft$

$$M_{prratio} := \frac{M_{pr}}{M_u}$$

$$\Omega_v := \text{if}\left(\frac{h_{wcsleg}}{leg} \le 1.5, 1.0, \max\left(M_{prratio}, 1.5\right)\right)$$

$\Omega_v = 18.15$ — Overstrength factor, ACI 318-19 18.10.3.1.2

$V_{ratio} := min\left(\omega_v \cdot \Omega_v, 3\right) = 3$

$$V_{epier} := \text{if}\left(\frac{leg}{t} > 2.5, V_u \cdot \Omega_o, \text{"N/A"}\right)$$

Alternate Ve if pier aspect ratio met
ACI 318-19 18.10.8.1a

$$V_e := \text{if}\left(PierCheck = \text{"Pier"}, \text{if}\left(V_{epier} = \text{"N/A"}, \frac{M_{pr}}{BC_{pier} \cdot ht_o}, min\left(\frac{M_{pr}}{BC_{pier} \cdot ht_o}, V_{epier}\right)\right), V_{ratio} \cdot V_u\right)$$

$V_e = 5.9\ k$

Design Shear Force,
For Segments: ACI 318-19 18.10.3.1
For Piers: ACI 318-19 18.7.6.1 or 18.10.8.1

$Check_{Ve18.10.8.1} = \text{“Refer to 18.10.8.1 for additional transverse reinforcement requirements”}$

ACI 318-19 18.10.8.1 - Transverse reinforcement with seismic hooks shall be provided to resist shear determined per 18.7.6.1.1. Transverse reinf. spacing 6" max OC. Extend reinf. minimum 12" above/ below opening.

Shear Steel Requirements - ACI 318-19 18.10.2.1/11.6.1

$A_{cv} := t \cdot leg = 210\ in^2$ — Gross Area of Concrete Section

$V_{min} := \lambda \cdot \sqrt{\frac{f_c}{psi}} \cdot psi \cdot A_{cv}$ — Shear Strength to determine ρ_{min} - ACI 318-19 18.10.2.1

$\rho_{min} := \text{if}\left(V_u < V_{min}, 0.002, 0.0025\right) = 0.002$

$A_{v\_min} := (t) \cdot (12 \cdot in) \cdot (\rho_{min})$ — Minimum area of Shear Steel per Foot

Determine number of curtains of shear steel - ACI 318-19 18.10.2.2

$V_{reinf} := 2 \cdot \lambda \sqrt{\frac{f_c}{psi}} \cdot psi \cdot A_{cv}$ — Shear Strength to Determine Required Number of Layers

$Reinforcing := \text{if}\left(V_u > V_{reinf}, \text{“(2) CURTAINS REQD”}, \text{“(1) CURTAIN REQD”}\right)$

$Reinforcing = \text{“(1) CURTAIN REQD”}$

Define Shear Steel:

$No_v := \#4$ — Shear Reinforcement Bar Size

$A_{sNov} := No_{v_0} \qquad d_{sv} := No_{v_1}$ — Area of Shear Reinforcement Bar

$S := 6 \cdot in$ — Shear Reinforcement Spacing (12" max, typical)

$N_L := 2$ — Number of Layers **(CL = 1, EF = 2)**

$A_{vs} := N_L \cdot \frac{12 \cdot in}{S} \cdot A_{sNov}$ — Area of Shear Reinforcement per Foot

$\rho_t := \frac{A_{vs}}{(12 \cdot in) \cdot (t)}$

$A_{v\_min\_check} := \text{if}\left(A_{v\_min} \le A_{vs}, \text{“OK”}, \text{“REVISE!”}\right)$ $\qquad A_{v\_min\_check} = \text{“OK”}$

$Check := \text{if}\left(V_u > V_{reinf} \wedge N_L = 1, \text{“REVISE REINF”}, \text{“OK”}\right)$ $\qquad Check = \text{“OK”}$

Shear Wall Strength - ACI 318-19 Eq. 18.10.4.1

$$h_{w\_max} := \max\left(\frac{ht}{w_s}, \frac{ht_o}{leg}\right) = 5$$

ACI 318-19 18.10.4.2 - Ratio for the greatest of wall or segment needs to be considered

$$\alpha_c := \begin{array}{l} \text{if } h_{w\_max} < 1.5 \\ \quad 3.0 \\ \text{else if } h_{w\_max} \geq 2.0 \\ \quad 2.0 \\ \text{else} \\ \quad 2 + \dfrac{2.0 - h_{w\_max}}{0.5} \end{array} = 2$$

$$\phi V_n := \phi_{vE} \cdot \left(A_{cv} \cdot \left(\alpha_c \cdot \lambda \cdot \left(\sqrt{\frac{f_c}{psi}} \cdot psi\right) + \rho_t \cdot f_y\right)\right)$$

ACI 318-19 Eq. 18.10.4.1

Define Shear Strength Max - ACI 318-19 18.10.4.4

$$\phi V_{nmax} := \phi_{vE} \cdot \left(10 \cdot \sqrt{\frac{f_c}{psi}} \cdot psi \cdot A_{cv}\right)$$

ACI 318-19 Eq. 18.10.4.4

$check := \text{if}\left(V_e \leq \phi V_{nmax}, \text{“Proceed with Design”}, \text{“Revise!”}\right)$

$check = \text{“Proceed with Design”}$

$check := \text{if}\left(\phi V_n \leq \phi V_{nmax}, \text{“OK”}, \text{“Vn MAX exceeded, reduce reinforcement”}\right)$

$check = \text{“OK”}$

$\phi V_n := min\left(\phi V_n, \phi V_{nmax}\right) = 73.54\ k$

$$DCR_v := \frac{V_e}{\phi V_n}$$

$DCR_v = 0.08$

$Flag_v = \text{“USE (2) LAYER /S OF \#4 BARS AT 6''OC HORIZ MIN”}$

Leg Compression Force: Per Load Case ASCE 7-16 12.2.3.6. Default is for outer legs receiving half of total panel load in addition to seismic.

$$P_{uleg} := P_{u1} + \frac{\rho \cdot M_{ot}}{w_s - \left(\dfrac{leg}{2}\right)} = 62.8\ k$$

Flexural Steel - ACI 318-19 18.10.6

$No_f := \#5$ — Longitudinal Reinforcement Bar Size

$A_{sNof} := No_{f_0}$ $\quad d_{Nof} := No_{f_1}$ — Area of Flexural Reinforcement Bar; Diameter of Flexural Reinforcement Bar

$N_{Lf} := 2$ — Number of Layers of Longitudinal Reinforcement (CL = 1, EF = 2)

$N_f := 8$ — Number of Longitudinal Reinforcement Bars per Layer

$A_{s\_leg} := N_f \cdot N_{Lf} \cdot A_{sNof} = 4.96\ in^2$ — Total Area of Steel Provided in Leg

$M_u := V_u \cdot BC_{pier} \cdot ht_o = 14.8\ k \cdot ft$ — Ultimate Overturning Moment ($\rho\, Q_e$)

$d_{solve} := 0.8 \cdot leg = 1.6\ ft$ — Depth of Lever Arm, use variables from above where possible

Solver / Guess Values / Constraints

$A_{sreq} := 1\ in^2$

$$\phi_f \cdot A_{sreq} \cdot f_y \cdot \left(d_{solve} - \frac{A_{sreq} \cdot f_y}{0.85 \cdot f_c \cdot t \cdot 2}\right) - M_u = 0$$

$A_{sreq\_flex} := \text{Find}\left(A_{sreq}\right)$

$A_{sreq\_flex} = 0.17\ in^2$ — Required Area of Flexural Steel

$$A_{sreq\_ten} := \frac{\rho \cdot M_{ot}}{\phi_f \cdot \left(w_s - 1.5 \cdot ft\right) \cdot f_y} = 0.44\ in^2$$

Required Area of Steel to Resist Panel Overturning

$A_{stot} := A_{sreq\_flex} \cdot 2 + A_{sreq\_ten} = 0.79\ in^2$ — Required Area of Steel for Leg Flexure and Panel Overturning

$$DCR_f := \frac{A_{stot}}{A_{s\_leg}} = 0.16$$

$DCR_f = 0.16$

$Flag_f =$ "USE (16) #5 BARS EACH END MIN"

Transverse Reinforcement Requirements- ACI 318-19 18.10.6.5

$$A_{s\_check} := \frac{400}{\frac{f_y}{psi}} \cdot (0.2 \cdot leg) \cdot t$$

Assumes 0.2*leg for N.A. depth. Input leg into SP Column if req'd to refine results.

$$V_{check} := \lambda \cdot \sqrt{\frac{f_c}{psi}} \cdot psi \cdot A_{cv}$$

Check shear against 18.10.6.5a

$$Check_{18.10.6.5a} := \text{if}\left(V_u > V_{check}, \text{"PROVIDE HOOKED HORIZ BARS OR CLOSED HOOPS"}, \text{"OK"}\right)$$

$$Check_{18.10.6.5a} = \text{"OK"}$$

$$Check_{18.10.6.5b} := \text{if}\left(A_{sreq\_flex} > A_{s\_check}, \text{"PROVIDE CONFINING HOOPS @ 5'' O.C. MAX per 18.10.6.5"}, \text{"OK"}\right)$$

$$Check_{18.10.6.5b} = \text{"OK"}$$

Basic Boundary Element Check w/ Gross Section Properties - ACI 318-19 18.10.6.3

$$I_g := \frac{leg^3 \cdot t}{12} \qquad A_g := leg \cdot t \qquad S_g := \frac{t \cdot leg^2}{6}$$

$$P_u := P_{uleg}$$

Factored Axial Load - Load Combination ASCE 7-16 12.2.3.6 Eq. 6

$$\sigma_u := \frac{M_u}{S_g} + \frac{P_{uleg}}{A_g}$$

Combined Stress at Extreme Fiber

$$F_c := 0.2 \cdot f_c$$

$$DCR_\sigma := \frac{\sigma_u}{F_c}$$

$$DCR_\sigma = 0.64$$

$$BasicCheck := \text{if}\left(F_c > \sigma_u, \text{"Okay"}, \text{"Perform Alternate Analysis"}\right)$$

$$BasicCheck = \text{"Okay"}$$

If Wall does not meet basic boundary element check, provide boundary elements or perform Alternate check per ACI 318-19 18.10.6.2 ONLY if hwcs/ w_s > 2.

$$AlternateCheck := \begin{cases} \text{if } \dfrac{h_{wcsleg}}{leg} \ge 2 \wedge BasicCheck = \text{"Perform Alternate Analysis"} \\ \quad \text{"Proceed"} \\ \text{also if } BasicCheck = \text{"Okay"} \\ \quad \text{"Not Applicable"} \\ \text{else} \\ \quad \text{"Boundary Element Necessary – Alternate Check Not Allowed"} \end{cases}$$

$AlternateCheck = \text{"Not Applicable"}$

Verify Section Uncracked

$$y_t := \frac{leg}{2}$$ Neutral Axis of Gross Section

$$M_{cr} := \frac{7.5 \cdot \lambda \cdot \sqrt{\frac{f_c}{psi}} \cdot psi \cdot I_g}{y_t}$$ Cracking Moment - ACI 318-19 19.2.3.1 Eq 19.2.3.1, 24.2.3.5 Eq 24.2.3.5

$$M_a := M_u \cdot EL_{asd\_factor}$$ Service Moment

$$M_{crCheck} := \text{if}\left(M_{cr} < M_a, \text{"Cracked; If necessary adjust relative stiffness and evaluate load"}, \text{"Uncracked"}\right)$$

$M_{crCheck} = \text{"Uncracked"}$

------------------------ VERIFY SECTION BELOW APPLIES------------------------

Pier Shear Reinforcing - ACI 318-19 18.10.8 - This section applies where the wall segment is a pier and does not meet the exceptions listed in the code.

$PierCheck = \text{"Pier"}$

ACI 318-19 18.10.8.2 - Transverse reinforcement with seismic hooks shall be provided to resist shear determined per 18.7.6.1.1. Transverse reinf. spacing 6" max OC. Extend reinf. minimum 12" above/ below opening.

$V_{Check\_18.10.8.2} = \text{"PROVIDE 180–DEGREE HOOKS AT 6'' MAX o.c."}$

Minimum shear requirement check per ACI 318-19 18.10.8.2

$M_{pr} = 269\ k \cdot ft$ Per SP Column

$$V_{epier} := \frac{M_{pr}}{ht_o \cdot BC_{pier}} = 53.8\ k$$ Shear associated with M_{pr}

$$Flag := \text{if}\left(\phi V_n > V_{epier}, \text{"OK"}, \text{"Revise Reinf"}\right)$$ $Flag = \text{"OK"}$

------------------------ VERIFY SECTION BELOW APPLIES------------------------

Slab Shear Dowel Bars (Leg A) - Shear Friction per ACI 318-19 22.9.4

$\mu := 0.6$ — Coefficient of Friction - ACI 318-19 22.9.4.2

$L_{dowel} := leg$ — Length of wall including dowels (leg length at storefront condition)

$No_{dowel} := \#4$ — Dowel Reinforcement Bar Size

$A_{sNodowel} := No_{dowel}$

$N_{dowel} := 2$ — Number of Dowels per Leg - (4) Min

$A_{vf} := A_{sNodowel} \cdot N_{dowel} = 0.4\ in^2$ — Area of Shear Friction Steel Across Panel

$\phi V_{vf\_n} := \phi_{vE} \cdot (\mu \cdot A_{vf} \cdot f_y)$ — Allowable Shear Capacity - ACI 318-19 22.9.4.2

$V_u := \rho \cdot V_{tot} \cdot k_a = 2.96\ k$ — Ultimate Panel Shear ($\rho\, Q_e$)

$$DCR_{vf} := \frac{V_u}{\phi V_{vf\_n}}$$ $DCR_{vf} = 0.34$

$flag := \text{if}\left(\phi V_{vf\_n} \ge V_u, \text{“OK”}, \text{“NG!”}\right)$ $flag = \text{“OK”}$

Panel 17 - Leg B (4'-0") Design

Wall Shear Design - Leg

$Label_{leg} := B$

$leg := leg_{temp_{Label_{leg}}} = 4\ ft$ $k_{leg} := k_{temp_{Label_{leg}}} = 0.75$

$w_o := w_{temp_{Label_{leg}}} = 18\ ft$ $ht_o := ht_{temp_{Label_{leg}}} = 10\ ft$

$$P_{u1} := \left(\left(1.2 + 0.2 \cdot S_{ds}\right) \cdot P_{tot} + \left(0.5 \cdot SL + LL\right) w_s\right) \left(\frac{leg + \frac{w_o}{2}}{w_s}\right) = 81.2\ k$$ ASCE 7-16 12.2.3.6 Eq 6

$$P_{u2} := \left(0.9 - 0.2 \cdot S_{ds}\right) P_{tot} \left(\frac{leg + \frac{w_o}{2}}{w_s}\right) = 41.3\ k$$ ASCE 7-16 12.2.3.6 Eq 7

$M_{pr} := 1006\ k \cdot ft$ — Largest probable flexural strength of the section from SP Column assuming tensile stress of 1.25*Fy and Φ = 1.0

Pier Check - ACI 318-19 Table R18.10.1

If wall section is a pier, requirements of ACI 318-19 18.10.8 or 18.14 must be met

$BC_{pier} :=$ Boundary condition: Fixed–Fixed

Boundary Condition Pier Factor

BC_{pier} **= 0.5 for fixed-fixed,**

BC_{pier} **= 1.0 for fixed-pinned**

$$Check_1 := \text{if}\left(\frac{leg}{t_p} \le 6, \text{“Pier”}, \text{“Segment”}\right)$$

$Check_1 = \text{“Pier”}$

$$PierCheck := \text{if}\left(\frac{ht_o}{leg} \ge 2, Check_1, \text{“Segment”}\right)$$

$PierCheck = \text{“Pier”}$

Design Shear Force - ACI 318-19 18.10.3

$V_u := \rho \cdot V_{tot} \cdot k_{leg} = 17.95\ k$ — Ultimate Leg Shear (ρQ_e)

$h_{wcsleg} := h_{roof} = 35.75\ ft$ — Height of panel leg above critical section

$stories_{wcs} := 1$ — Number of Stories above Critical Section

$$n_s := \max\left(stories_{wcs}, 0.007 \cdot \frac{h_{wcsleg}}{in}\right)$$

$$\omega_v := \begin{cases} 1.0 & \text{if } \frac{h_{wcsleg}}{leg} < 2 \\ 0.9 + \frac{n_s}{10} & \text{else if } n_s \le 6.0 \\ min\left(1.3 + \frac{n_s}{30}, 1.8\right) & \text{else} \end{cases} = 1.2$$

Dynamic shear amplification factor, ACI 318-19 18.10.3.1.3

$M_u := V_u \cdot BC_{pier} \cdot ht_o = 89.7\ k \cdot ft$

$M_{prratio} := \frac{M_{pr}}{M_u}$

$\Omega_v := \text{if}\left(\frac{h_{wcsleg}}{leg} \le 1.5, 1.0, \max\left(M_{prratio}, 1.5\right)\right)$

$\Omega_v = 11.21$ — Overstrength factor, ACI 318-19 18.10.3.1.2

$V_{ratio} := min\left(\omega_v \cdot \Omega_v, 3\right) = 3$

$V_{epier} := \text{if}\left(\frac{leg}{t} > 2.5, V_u \cdot \Omega_o, \text{"N/A"}\right)$ — Alternate Ve if pier aspect ratio met ACI 318-19 18.10.8.1a

$V_e := \text{if}\left(PierCheck = \text{"Pier"}, \text{if}\left(V_{epier} = \text{"N/A"}, \frac{M_{pr}}{BC_{pier} \cdot ht_o}, min\left(\frac{M_{pr}}{BC_{pier} \cdot ht_o}, V_{epier}\right)\right), V_{ratio} \cdot V_u\right)$

$V_e = 35.9\ k$ — Design Shear Force,
For Segments: ACI 318-19 18.10.3.1
For Piers: ACI 318-19 18.7.6.1 or 18.10.8.1

$Check_{Ve18.10.8.1} = \text{"Refer to 18.10.8.1 for additional transverse reinforcement requirements"}$

ACI 318-19 18.10.8.1 - Transverse reinforcement with seismic hooks shall be provided to resist shear determined per 18.7.6.1.1. Transverse reinf. spacing 6" max OC. Extend reinf. minimum 12" above/below opening.

Shear Steel Requirements - ACI 318-19 18.10.2.1/11.6.1

$A_{cv} := t \cdot leg = 420\ in^2$ — Gross Area of Concrete Section

$V_{min} := \lambda \cdot \sqrt{\frac{f_c}{psi}} \cdot psi \cdot A_{cv}$ — Shear Strength to determine ρ_{min} - ACI 318-19 18.10.2.1

$\rho_{min} := \text{if}\left(V_u < V_{min}, 0.002, 0.0025\right) = 0.002$

$A_{v\_min} := (t) \cdot (12 \cdot in) \cdot \left(\rho_{min}\right)$ — Minimum area of Shear Steel per Foot

Determine number of curtains of shear steel - ACI 318-19 18.10.2.2

$V_{reinf} := 2 \cdot \lambda \sqrt{\frac{f_c}{psi}} \cdot psi \cdot A_{cv}$ — Shear Strength to Determine Required Number of Layers

$Reinforcing := \text{if}\left(V_u > V_{reinf}, \text{"(2) CURTAINS REQD"}, \text{"(1) CURTAIN REQD"}\right)$

$Reinforcing = \text{"(1) CURTAIN REQD"}$

Define Shear Steel:

$No_v := \#3$ — Shear Reinforcement Bar Size

$A_{sNov} := No_{v_0} \qquad d_{sv} := No_{v_1}$ — Area of Shear Reinforcement Bar

$S := 6 \cdot in$ — Shear Reinforcement Spacing (12" max, typical)

$N_L := 2$ — Number of Layers **(CL = 1, EF = 2)**

$A_{vs} := N_L \cdot \dfrac{12 \cdot in}{S} \cdot A_{sNov}$ — Area of Shear Reinforcement per Foot

$\rho_t := \dfrac{A_{vs}}{(12 \cdot in) \cdot (t)}$

$A_{v\_min\_check} := \text{if}\left(A_{v\_min} \le A_{vs}, \text{"OK"}, \text{"REVISE!"}\right)$ $\qquad A_{v\_min\_check} = \text{"OK"}$

$Check := \text{if}\left(V_u > V_{reinf} \wedge N_L = 1, \text{"REVISE REINF"}, \text{"OK"}\right)$ $\qquad Check = \text{"OK"}$

Shear Wall Strength - ACI 318-19 Eq. 18.10.4.1

$h_{w\_max} := \max\left(\dfrac{ht}{w_s}, \dfrac{ht_o}{leg}\right) = 2.5$ — ACI 318-19 18.10.4.2 - Ratio for the greatest of wall or segment needs to be considered

$$\alpha_c := \begin{cases} 3.0 & \text{if } h_{w\_max} < 1.5 \\ 2.0 & \text{else if } h_{w\_max} \ge 2.0 \\ 2 + \dfrac{2.0 - h_{w\_max}}{0.5} & \text{else} \end{cases} = 2$$

$\phi V_n := \phi_{vE} \cdot \left(A_{cv} \cdot \left(\alpha_c \cdot \lambda \cdot \left(\sqrt{\dfrac{f_c}{psi}} \cdot psi\right) + \rho_t \cdot f_y\right)\right) = 95.24\ k$ — ACI 318-19 Eq. 18.10.4.1

Define Shear Strength Max - ACI 318-19 18.10.4.4

$\phi V_{nmax} := \phi_{vE} \cdot \left(10 \cdot \sqrt{\dfrac{f_c}{psi}} \cdot psi \cdot A_{cv}\right)$ — ACI 318-19 Eq. 18.10.4.4

$check := \text{if}\left(V_e \le \phi V_{nmax}, \text{"Proceed with Design"}, \text{"Revise!"}\right)$ $check = \text{"Proceed with Design"}$

$check := \text{if}\left(\phi V_n \le \phi V_{nmax}, \text{"OK"}, \text{"Vn MAX exceeded, reduce reinforcement"}\right)$

$check = \text{"OK"}$

$\phi V_n := min\left(\phi V_n, \phi V_{nmax}\right) = 95.24\ k$

$DCR_v := \dfrac{V_e}{\phi V_n}$ $DCR_v = 0.38$

$Flag_v = \text{"USE (2) LAYER /S OF \#3 BARS AT 6''OC HORIZ MIN"}$

Leg Compression Force: Per Load Case ASCE 7-16 12.2.3.6. Default is for outer legs receiving half of total panel load in addition to seismic.

$$P_{uleg} := P_{u1} + \frac{\rho \cdot M_{ot}}{w_s - \left(\dfrac{leg}{2}\right)} = 105.6\ k$$

Flexural Steel - ACI 318-19 18.10.6

$No_f := \#5$ Longitudinal Reinforcement Bar Size

$A_{sNof} := No_{f_0}$ $d_{Nof} := No_{f_1}$ Area of Flexural Reinforcement Bar; Diameter of Flexural Reinforcement Bar

$N_{Lf} := 2$ Number of Layers of Longitudinal Reinforcement (CL = 1, EF = 2)

$N_f := 14$ Number of Longitudinal Reinforcement Bars per Layer

$A_{s\_leg} := N_f \cdot N_{Lf} \cdot A_{sNof} = 8.68\ in^2$ Total Area of Steel Provided in Leg

$M_u := V_u \cdot BC_{pier} \cdot ht_o = 89.7\ k \cdot ft$ Ultimate Overturning Moment ($\rho\, Q_e$)

$d_{solve} := 0.8 \cdot leg = 3.2\ ft$ Depth of Lever Arm, use variables from above where possible

Guess Values / Constraints / Solver

$A_{sreq} := 1\ in^2$

$$\phi_f \cdot A_{sreq} \cdot f_y \cdot \left(d_{solve} - \frac{A_{sreq} \cdot f_y}{0.85 \cdot f_c \cdot t \cdot 2}\right) - M_u = 0$$

$A_{sreq\_flex} := \text{Find}\left(A_{sreq}\right)$

$$A_{sreq\_flex} = 0.53\ in^2$$

Required Area of Flexural Steel

$$A_{sreq\_ten} := \frac{\rho \cdot M_{ot}}{\phi_f \cdot (w_s - 1.5 \cdot ft) \cdot f_y} = 0.44\ in^2$$

Required Area of Steel to Resist Panel Overturning

$$A_{stot} := A_{sreq\_flex} \cdot 2 + A_{sreq\_ten} = 1.5\ in^2$$

Required Area of Steel for Leg Flexure and Panel Overturning

$$DCR_f := \frac{A_{stot}}{A_{s\_leg}} = 0.17$$

$$DCR_f = 0.17$$

$$Flag_f = \text{“USE (28) \#5 BARS EACH END MIN”}$$

Transverse Reinforcement Requirements- ACI 318-19 18.10.6.5

$$A_{s\_check} := \frac{400}{\frac{f_y}{psi}} \cdot (0.2 \cdot leg) \cdot t$$

Assumes 0.2*leg for N.A. depth. Input leg into SP Column if req'd to refine results.

$$V_{check} := \lambda \cdot \sqrt{\frac{f_c}{psi}} \cdot psi \cdot A_{cv}$$

Check shear against 18.10.6.5a

$$Check_{18.10.6.5a} := \text{if}\left(V_u > V_{check}, \text{“PROVIDE HOOKED HORIZ BARS OR CLOSED HOOPS”}, \text{“OK”}\right)$$

$$Check_{18.10.6.5a} = \text{“OK”}$$

$$Check_{18.10.6.5b} := \text{if}\left(A_{sreq\_flex} > A_{s\_check}, \text{“PROVIDE CONFINING HOOPS @ 5'' O.C. MAX per 18.10.6.5”}, \text{“OK”}\right)$$

$$Check_{18.10.6.5b} = \text{“OK”}$$

Basic Boundary Element Check w/ Gross Section Properties - ACI 318-19 18.10.6.3

$I_g := \frac{leg^3 \cdot t}{12}$ $\quad A_g := leg \cdot t$ $\quad S_g := \frac{t \cdot leg^2}{6}$

$P_u := P_{uleg}$ — Factored Axial Load - Load Combination ASCE 7-16 12.2.3.6 Eq. 6

$\sigma_u := \frac{M_u}{S_g} + \frac{P_{uleg}}{A_g}$ — Combined Stress at Extreme Fiber

$F_c := 0.2 \cdot f_c$

$DCR_\sigma := \frac{\sigma_u}{F_c}$ $\quad DCR_\sigma = 0.71$

$BasicCheck := \text{if}\left(F_c > \sigma_u, \text{"Okay"}, \text{"Perform Alternate Analysis"}\right)$

$BasicCheck = \text{"Okay"}$

If Wall does not meet basic boundary element check, provide boundary elements or perform Alternate check per ACI 318-19 18.10.6.2 ONLY if hwcs/ w_s > 2.

$$AlternateCheck := \left\| \begin{array}{l} \text{if } \frac{h_{wcsleg}}{leg} \geq 2 \wedge BasicCheck = \text{"Perform Alternate Analysis"} \\ \quad \| \text{"Proceed"} \\ \text{also if } BasicCheck = \text{"Okay"} \\ \quad \| \text{"Not Applicable"} \\ \text{else} \\ \quad \| \text{"Boundary Element Necessary – Alternate Check Not Allowed"} \end{array} \right\|$$

$AlternateCheck = \text{"Not Applicable"}$

Verify Section Uncracked

$y_t := \frac{leg}{2}$ — Neutral Axis of Gross Section

$M_{cr} := \frac{7.5 \cdot \lambda \cdot \sqrt{\frac{f_c}{psi}} \cdot psi \cdot I_g}{y_t}$ — Cracking Moment - ACI 318-19 19.2.3.1 Eq 19.2.3.1, 24.2.3.5 Eq 24.2.3.5

$M_a := M_u \cdot EL_{asd\_factor}$ — Service Moment

$M_{crCheck} := \text{if}\left(M_{cr} < M_a, \text{"Cracked; If necessary adjust relative stiffness and evaluate load"}, \text{"Uncracked"}\right)$

$M_{crCheck} = \text{"Uncracked"}$

------------------------ VERIFY SECTION BELOW APPLIES------------------------

Pier Shear Reinforcing - ACI 318-19 18.10.8 - This section applies where the wall segment is a pier and does not meet the exceptions listed in the code.

$PierCheck = \text{"Pier"}$

ACI 318-19 18.10.8.2 - Transverse reinforcement with seismic hooks shall be provided to resist shear determined per 18.7.6.1.1. Transverse reinf. spacing 6" max OC. Extend reinf. minimum 12" above/ below opening.

$V_{Check\_18.10.8.2} = \text{"PROVIDE 180–DEGREE HOOKS AT 6'' MAX o.c."}$

Minimum shear requirement check per ACI 318-19 18.10.8.2

$M_{pr} = 1006\ k \cdot ft$ Per SP Column

$$V_{epier} := \frac{M_{pr}}{ht_o \cdot BC_{pier}} = 201.2\ k$$

Shear associated with M_{pr}

$Flag := \text{if}\left(\phi V_n > V_e, \text{"OK"}, \text{"Revise Reinf"}\right)$ $Flag = \text{"OK"}$

------------------------ VERIFY SECTION BELOW APPLIES------------------------

Slab Shear Dowel Bars (Leg A) - Shear Friction per ACI 318-19 22.9.4

$\mu := 0.6$ Coefficient of Friction - ACI 318-19 22.9.4.2

$L_{dowel} := leg$ Length of wall including dowels (leg length at storefront condition)

$No_{dowel} := \#4$ Dowel Reinforcement Bar Size

$A_{sNodowel} := No_{dowel}$

$N_{dowel} := 3$ Number of Dowels per Leg - (4) Min

$A_{vf} := A_{sNodowel} \cdot N_{dowel} = 0.6\ in^2$ Area of Shear Friction Steel Across Panel

$\phi V_{vf\_n} := \phi_{vE} \cdot \left(\mu \cdot A_{vf} \cdot f_y\right)$ Allowable Shear Capacity - ACI 318-19 22.9.4.2

$V_u := \rho \cdot V_{tot} \cdot k_a = 2.96\ k$ Ultimate Panel Shear ($\rho\, Q_e$)

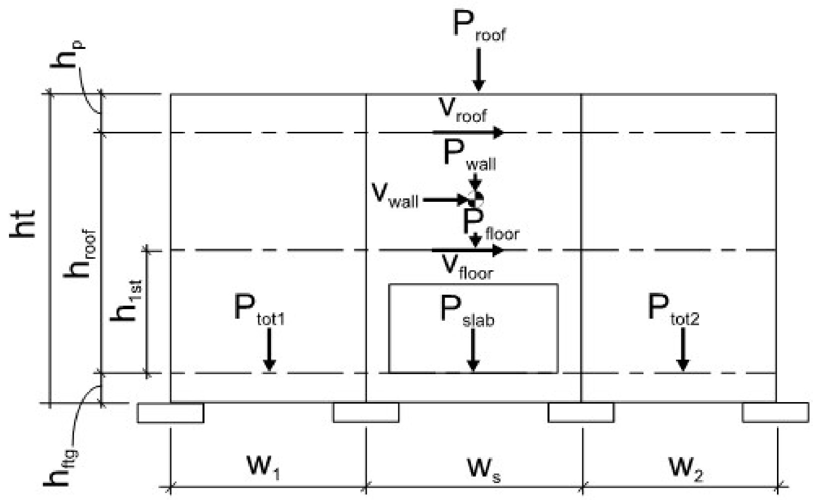
$$DCR_{vf} := \frac{V_u}{\phi V_{vf\_n}}$$

$DCR_{vf} = 0.23$

$flag := \text{if}\left(\phi V_{vf\_n} \ge V_u, \text{"OK"}, \text{"NG!"}\right)$ $flag = \text{"OK"}$

Shear Wall Design

318-19 Panel w/ (2) Opening - Panel 15 - North



Shear Panel Input (w_s)

$w_s := 26 \cdot ft$	Panel Width
$h_p := 0 \cdot ft$	Parapet Height
$h_{roof} := 35.75 \cdot ft$	Height of Roof Above FF
$h_{1st} := 0 \cdot ft$	Height of Floor Above FF
$h_{ftg} := 5 \cdot ft$	FF to T/FTG Height
$ht := h_p + h_{roof} + h_{ftg} = 40.8\ ft$	Total Panel Height (T/FTG to T/PARAPET)
$ht_{o1} := 7.167 \cdot ft$	Opening 1 Height
$ht_{o2} := 10\ ft$	Opening 2 Height
$w_{o1} := 3.33 \cdot ft$	Opening 1 Width
$w_{o2} := 9 \cdot ft$	Opening 1 Width
$lega := 4\ ft + 10\ in$	Leg A Width
$legb := 6\ ft + 10\ in$	Leg B Width
$legc := 2\ ft$	Leg C Width
$t_p := 9.5 \cdot in$	Panel Thickness

$r_v := 0.75 \cdot in$ Reveal Depth

$t := t_p - r_v = 8.8\ in$ Effective Panel Thickness

$Wt_p := w_c \cdot t_p$ Panel Wt. (psf)

Relative Panel Leg Stiffnesses (w_s)

Amrhein/Yellow book method

$BC_{lega} :=$ Boundary Condition: Fixed–Fixed

Leg A & B boundary conditions

BC_{leg} = 1 for fixed-fixed

BC_{leg} = 4 for fixed-pinned

$BC_{legb} :=$ Boundary Condition: Fixed–Fixed

$BC_{legc} :=$ Boundary Condition: Fixed–Fixed

$$hd_{ratioa} := \frac{ht_{o1}}{lega} = 1.48$$

Leg A aspect ratio

$$hd_{ratiob} := \frac{\max\left(ht_{o1}, ht_{o2}\right)}{legb} = 1.46$$

Leg B aspect ratio

$$hd_{ratioc} := \frac{ht_{o2}}{legc} = 5$$

Leg C aspect ratio

$P_k := 100000\ lb$ $t_k := 1\ in$ $E_k := 1000000\ psi$

$$\Delta_{ka} := \frac{P_k}{E_k \cdot t_k} \cdot \left(BC_{lega} \cdot \left(hd_{ratioa}\right)^3 + 3 \cdot \left(hd_{ratioa}\right)\right)$$

$$\Delta_{kb} := \frac{P_k}{E_k \cdot t_k} \cdot \left(BC_{legb} \cdot \left(hd_{ratiob}\right)^3 + 3 \cdot \left(hd_{ratiob}\right)\right)$$

$$\Delta_{kc} := \frac{P_k}{E_k \cdot t_k} \cdot \left(BC_{legc} \cdot \left(hd_{ratioc}\right)^3 + 3 \cdot \left(hd_{ratioc}\right)\right)$$

$R_{ka} := \frac{1}{\Delta_{ka}}$ $R_{kb} := \frac{1}{\Delta_{kb}}$ $R_{kc} := \frac{1}{\Delta_{kc}}$

$$k_a := \frac{R_{ka}}{R_{ka} + R_{kb} + R_{kc}} = 0.481$$

Leg A Relative Stiffness

$$k_b := \frac{R_{kb}}{R_{ka} + R_{kb} + R_{kc}} = 0.493$$

Leg B Relative Stiffness

Shear Wall Design

$k_c := \frac{R_{kc}}{R_{ka}+R_{kb}+R_{kc}} = 0.026$ Leg B Relative Stiffness

$Stiffness := \text{if}\left(\left|1.0-\left(k_a+k_b+k_c\right)\right| > 0.0001, \text{"Check"}, \text{"OK"}\right)$ $Stiffness = \text{"OK"}$

Adjacent Panel Input (w_1)	**Adjacent Panel Input (w_2)**	
$w_1 := w_s$	$w_2 := w_s$	Adjacent Panel Width
$h_{p1} := h_p$	$h_{p2} := h_p$	Parapet Height
$h_{roof1} := h_{roof}$	$h_{roof2} := h_{roof}$	Height of Roof Above FF
$h_{ftg1} := h_{ftg}$	$h_{ftg2} := h_{ftg}$	FF to T/FTG Height
$ht_1 := h_{p1}+h_{roof1}+h_{ftg1} = 40.8\ ft$	$ht_2 := h_{p2}+h_{roof2}+h_{ftg2} = 40.8\ ft$	Total Panel Height (T/FTG to T/PARAPET)
$t_{p1} := t_p$	$t_{p2} := t_p$	Panel Thickness
$Ao_1 := 2 \cdot 10 \cdot ft \cdot 9 \cdot ft$	$Ao_2 := 2 \cdot 10 \cdot ft \cdot 9 \cdot ft$	Area of Openings

Loading:

Seismic (1.0 Q_e)

$V_{roof} := 15.72 \cdot k$ Seismic Shear @ Roof - ρ not included

$V_{floor} := 0 \cdot k$ Seismic Shear @ Floor

$V_{wall} := \left(\left(ht-h_{ftg}\right) \cdot w_s - \left(ht_{o1} \cdot w_{o1} + ht_{o2} \cdot w_{o2}\right)\right) \cdot Wt_p \cdot C_s$ Seismic Shear due to Wall Panel Mass

$V_{tot} := V_{roof}+V_{wall}+V_{floor} = 32.1\ k$ Total Seismic Shear

$M_{ot\_roof} := V_{roof} \cdot h_{roof}$ Overturning Moment due to Roof Shear

$M_{ot\_floor} := V_{floor} \cdot h_{1st}$ Overturning Moment due to Floor Shear

$M_{ot\_wall} := V_{wall} \cdot \frac{\left(ht-h_{ftg}\right)}{2}$ Overturning Moment Due to Panel Mass

$M_{ot} := M_{ot\_roof}+M_{ot\_floor}+M_{ot\_wall} = 853.9\ k \cdot ft$ Total Overturning Moment @ Wall Panel

Shear Wall Design

Gravity:

Shear Panel (w_s):

$l_{roof\_trib} := \frac{60}{2} \cdot ft$ — Length of Roof Tributary to Panel

$l_{floor\_trib} := \frac{0}{2} \cdot ft$ — Length of Floor Tributary to Panel

$SL := 0.6\ klf$ (Construction LL) — Roof snow load tributary to panel

$LL := 0.0\ klf$ — Floor live load tributary to panel

$P_{roof} := DL_{sei} \cdot l_{roof\_trib} \cdot w_s$ — Roof DL

$P_{floor} := DL_{flr} \cdot l_{floor\_trib} \cdot w_s$ — Floor DL

$P_{wall} := \left(ht \cdot w_s - \left(ht_{o1} \cdot w_{o1} + ht_{o2} \cdot w_{o2}\right)\right) \cdot Wt_p$ — Wall Panel DL

$P_{slab} := w_c \cdot t_{SOG} \cdot 8 \cdot ft \cdot w_s$ — Slab-on-Grade DL (assumes 8' width of slab over width of panel available to resist OT)

$P_{tot} := P_{roof} + P_{floor} + P_{wall} = 120.8\ k$ — Total DL (without slab @ wall considered)

Panel (w_1):

$P_{roof1} := P_{roof}$ — Roof DL

$P_{floor1} := P_{floor}$ — Floor DL

$P_{wall1} := \left(w_1 \cdot ht_1 - Ao_1\right) \cdot t_{p1} \cdot w_c$ — Wall Panel DL

$P_{slab1} := P_{slab} \cdot \left(\frac{w_1}{w_s}\right)$ — Slab-on-Grade DL

$P_{tot1} := P_{roof1} + P_{floor1} + P_{wall1} + P_{slab1} = 131.1\ k$ — Total DL (including slab weight)

Panel (w_2):

$P_{roof2} := P_{roof}$ — Roof DL

$P_{floor2} := P_{floor}$ — Floor DL

$P_{wall2} := \left(w_2 \cdot ht_2 - Ao_2\right) \cdot t_{p2} \cdot w_c$ — Wall Panel DL

$P_{slab2} := P_{slab} \cdot \left(\frac{w_2}{w_s}\right)$ — Slab-on-Grade DL

$P_{tot2} := P_{roof2} + P_{floor2} + P_{wall2} + P_{slab2} = 131.1\ k$ — Total DL (including slab weight)

Resisting Moment:

$$M_{resist} := (P_{tot} + P_{slab}) \cdot \frac{w_s}{2} = 1806.9\ k \cdot ft$$

Resisting Moment at Shear Panel

Load Combinations:

ASCE 7-16 2.3.6

LRFD 6: (1.2+0.2Sds)D+1.0ρ Q_e + L + 0.2S

LRFD 7: (0.9 - 0.2Sds)D + 1.0ρ Q_e (Resisting Moment of Slab-on-Grade)

IBC 2021 1605.2 - Foundations

ASD 16-5: D + L + S + E/1.4 (Foundation Bearing)

ASD 16-6: 0.9D + E/1.4 (Foundation Uplift/Weight. Ev = 0 for fdn size, Ev included for holdowns)

Panel Hold Down - IBC 2021 1605.2 - Alternative Load Combination ASD 16-6

$$M_{ot\_IBC} := \rho \cdot \frac{M_{ot}}{1.4}$$

ASD Overturning Moment

$$M_{res\_IBC} := \left(0.9 - \frac{0.2 \cdot S_{ds}}{1.4}\right) \cdot M_{resist}$$

ASD Resisting Moment

$$No_{hd} := \#5$$

Rebar Size No.

$$N_{hd} := 2$$

Number of Bars in Hold Down

$$L_{hd} := \text{if}\left(No_{hd_0} = 0.31\ in^2, L_{HD5_{N_{hd}}}, L_{HD6_{N_{hd}}}\right) = 8\ in$$

Length of Hold Down

$$A_{shd} := No_{hd_0} \cdot N_{hd}$$

Area of Steel in Hold Down

$$T_{hd} := \frac{M_{ot\_IBC} - M_{res\_IBC}}{w_s - (1 \cdot ft + 0.5 \cdot L_{hd})} = -32.4\ k$$

ASD Tension at Hold Down

$$A_{shdreq} := \frac{\left(\frac{T_{hd}}{EL_{asd\_factor}}\right)}{\phi_t \cdot f_y} = -0.86\ in^2$$

Required Area of Steel

$$DCR_{hd} := \frac{A_{shdreq}}{A_{shd}}$$

$$DCR_{hd} = -1.4$$

$hd\_Results =$ "NO HOLDOWNS REQD"

Footing Uplift and Seismic Bearing Checks

$l_f := \frac{w_s}{2}$ Length of Footing

$w_f := 2.5 \cdot ft$ Width of Footing

$d_f := 1 \cdot ft$ Depth of Footing (Use 2'-0" Min @ hold down, see footing calcs)

$P_{ftg} := w_c \cdot l_f \cdot w_f \cdot d_f$ Footing Weight

Required Footing Weight - Load Combination ASD 16-6

$M_{res\_uplift} := M_{res\_IBC} + \left(0.9 \cdot \left(min\left(\frac{P_{tot1}}{2}, \frac{P_{tot2}}{2}\right) \cdot w_s\right)\right)$ Factored Resisting Moment (Including contribution of slab & adjacent panel)

$M_{res\_uplift} = 2943.01\ k \cdot ft$

$Wt_{ftg\_uplift} := \frac{\left(M_{ot\_IBC} - M_{res\_uplift}\right)}{w_s - \left(1\ ft + 0.5 \cdot L_{hd}\right)}$ Required Footing Weight

$Wt_{ftg\_uplift} = -94.58\ k$

$flag_w := \text{if}\left(Wt_{ftg\_uplift} \le P_{ftg}, \text{“OK”}, \text{“NG!!!”}\right)$ $flag_w = \text{“OK”}$

------------------------ VERIFY SECTION BELOW APPLIES------------------------

Maximum Seismic Bearing, PAD Footing Condition - Load Combination ASD 16-6

$C_{ot\_comp} := \frac{M_{ot\_IBC}}{w_s}$ Factored Axial Compression Force due to Overturning

$P_{grav\_comp} := 0.5 \cdot \left(P_{tot}\right)$

$P_{grav\_comp} = 60.4\ k$ Factored Axial Gravity Force for 1/2 of panel

$A_{ftg} := l_f \cdot w_f$ Footing Area

$A_{ftg\_reqd} := \frac{C_{ot\_comp} + P_{grav\_comp}}{q_{sei}} = 25.16\ ft^2$ Required Footing Area

$flag_A := \text{if}\left(A_{ftg\_reqd} \le A_{ftg}, \text{“OK”}, \text{“NG!!!”}\right)$ $flag_A = \text{“OK”}$

$q_s := \frac{C_{ot\_comp} + P_{grav\_comp}}{A_{ftg}} = 2580.2\ psf$ Seismic Bearing Pressure

$flag_B := \text{if}\left(q_s < q_{sei}, \text{“OK”}, \text{“NG!!!”}\right)$ $flag_B = \text{“OK”}$

Check Wall Segment Above/Below Opening

Wall Shear Design

Factored Wall Axial Forces (for Mpr calculation)

$$P_{u1c} := \left(1.2 + 0.2 \cdot S_{ds}\right) \cdot P_{tot} + \left(0.5\ SL + LL\right)\ w_s = 173\ k$$

ASCE 7-16 12.2.3.6 Eq. 6

$$P_{u2c} := \left(0.9 - 0.2\ S_{ds}\right) \cdot P_{tot} = 88.35\ k$$

ASCE 7-16 12.2.3.6 Eq. 7

$$M_{pr} := 16000\ k \cdot ft$$

Largest probable flexural strength of the section from SP Column assuming tensile stress of 1.25*Fy and Φ = 1.0

$$M_u := \rho \cdot M_{ot} = 854\ k \cdot ft$$

$$V_u := \rho \cdot V_{tot} = 32.1\ k$$

Ultimate Panel Shear (ρ Q_e)

$$h_{wcs} := 35.75\ ft$$

Height of wall measured from critical section to parapet (see NTE diagram)

Pier Check - ACI 318-19 Table R18.10.1

If wall section is a pier, requirements of ACI 318-19 18.10.8 or 18.14 must be met.

$$Check_1 := \text{if}\left(\frac{w_s}{t_p} \le 6, \text{“Pier”}, \text{“Segment”}\right)$$

$$PierCheck := \text{if}\left(\frac{h_{wcs}}{w_s} \ge 2, Check_1, \text{“Segment”}\right)$$

$$PierCheck = \text{“Segment”}$$

Design Shear Force - ACI 318-19 18.10.3

$$stories_{wcs} := 1$$

Number of Stories above Critical Section

$$n_s := \max\left(stories_{wcs}, 0.007 \cdot \frac{h_{wcs}}{in}\right)$$

$$\omega_v := \begin{array}{|l} \text{if } \frac{h_{wcs}}{w_s} < 2 \\ \quad 1.0 \\ \text{else if } n_s \le 6.0 \\ \quad 0.9 + \frac{n_s}{10} \\ \text{else} \\ \quad min\left(1.3 + \frac{n_s}{30}, 1.8\right) \end{array} = 1$$

Dynamic shear amplification factor, ACI 318-19 18.10.3.1.3

$$M_{prratio} := \frac{M_{pr}}{M_u}$$

$$\Omega_v := \text{if}\left(\frac{h_{wcs}}{w_s} \le 1.5, 1.0, \max\left(M_{prratio}, 1.5\right)\right)$$

$$\Omega_v = 1$$

Shear Overstrength factor, ACI 318-19 18.10.3.1.2

$$V_{ratio} := min\left(\omega_v \cdot \Omega_v, 3\right) = 1$$

$$V_e := V_{ratio} \cdot V_u$$

$$V_{epiera} := \text{if}\left(\frac{w_s}{t} > 2.5, V_u \cdot \Omega_o, \text{"N/A"}\right)$$

Alternate Ve if pier aspect ratio met
ACI 318-19 18.10.8.1a

$$V_e := \text{if}\left(PierCheck = \text{"Pier"}, \text{if}\left(V_{epiera} = \text{"N/A"}, \frac{M_{pr}}{h_{roof}}, min\left(\frac{M_{pr}}{h_{roof}}, V_{epiera}\right)\right), V_{ratio} \cdot V_u\right)$$

$$V_e = 32.1\ k$$

Design Shear Force,
For Segments: ACI 318-19 18.10.3.1
For Piers: ACI 318-19 18.7.6.1

Shear Steel Requirements - ACI 318-19 18.10.2.1/11.6.1

$$A_{cv} := t \cdot w_s = 2730\ in^2$$

Gross Area of Concrete Section

$$V_{min} := \lambda \cdot \sqrt{\frac{f_c}{psi}} \cdot psi \cdot A_{cv}$$

Shear Strength to determine ρ_{min}

$$\rho_{min} := \text{if}\left(V_u < V_{min}, 0.002, 0.0025\right) = 0.002$$

$$A_{v\_min} := t \cdot \left(12 \cdot in\right) \cdot \rho_{min}$$

Minimum area of Shear Steel per Foot

Determine number of curtains of shear steel - ACI 318-19 18.10.2.2

$$V_{reinf} := 2 \cdot \lambda \cdot \sqrt{\frac{f_c}{psi}} \cdot psi \cdot A_{cv}$$

Shear Strength to Determine Required Number of Layers

$$Reinforcing := \text{if}\left(V_u > V_{reinf} \lor \frac{ht}{w_s} \ge 2, \text{"(2) CURTAINS REQD"}, \text{"(1) CURTAIN REQD"}\right)$$

$$Reinforcing = \text{"(1) CURTAIN REQD"}$$

Define Shear Steel:

$No_v := \#4$ — Shear Reinforcement Bar Size

$A_{sNov} := No_{v_0} \qquad d_{sv} := No_{v_1}$ — Area of Shear Reinforcement Bar

$S := 18 \cdot in$ — Shear Reinforcement Spacing (12" max, typical)

$N_L := 2$ — Number of Layers **(CL = 1, EF = 2)**

$A_{vs} := N_L \cdot \frac{12 \cdot in}{S} \cdot A_{sNov}$ — Area of Shear Reinforcement per Foot

$$\rho_t := \frac{A_{vs}}{(12 \cdot in) \cdot (t)}$$

$A_{v\_min\_check} := \text{if}\left(A_{v\_min} \le A_{vs}, \text{“OK”}, \text{“REVISE!”}\right)$ — $A_{v\_min\_check} = \text{“OK”}$

$Check := \text{if}\left(V_u > V_{reinf} \wedge N_L = 1, \text{“REVISE REINF”}, \text{“OK”}\right)$ — $Check = \text{“OK”}$

Shear Wall Strength - ACI 318-19 Eq. 18.10.4.1

$$\alpha_c := \begin{cases} 3.0 & \text{if } \frac{ht}{w_s} \le 1.5 \\ 2.0 & \text{else if } \frac{ht}{w_s} \ge 2.0 \\ 2 + \frac{2.0 - \frac{ht}{w_s}}{0.5} & \text{else} \end{cases} = 2.87$$

$$\phi V_n := \phi_{vE} \cdot \left(A_{cv} \cdot \left(\alpha_c \cdot \lambda \cdot \left(\sqrt{\frac{f_c}{psi}} \cdot psi\right) + \rho_t \cdot f_y\right)\right)$$ ACI 318-19 Eq. 18.10.4.1

Define Shear Strength Max - ACI 318-19 18.10.4.4

$$\phi V_{nmax} := \phi_{vE} \cdot \left(8 \cdot \sqrt{\frac{f_c}{psi}} \cdot psi \cdot A_{cv}\right)$$ ACI 318-19 Eq. 18.10.4.4

$check := \text{if}\left(V_e \le \phi V_{nmax}, \text{“Proceed with Design”}, \text{“Revise!”}\right)$ — $check = \text{“Proceed with Design”}$

$check := \text{if}\left(\phi V_n \le \phi V_{nmax}, \text{“OK”}, \text{“Vn MAX exceeded, reduce reinforcement”}\right)$ $check = \text{“OK”}$

$DCR_v := \dfrac{V_e}{min\left(\phi V_n, \phi V_{nmax}\right)}$ $DCR_v = 0.06$

$Flag_v = \text{“USE (2) LAYER /S OF \#4 BARS AT 18''OC HORIZ MIN”}$

Flexural Steel - ACI 318-19 18.10.6

$No_f := \#5$ Longitudinal Reinforcement Bar Size

$A_{sNof} := No_{f_0}$ $d_{Nof} := No_{f_1}$

$N_{Lf} := 2$ Number of Layers of Longitudinal Reinforcement (CL = 1, EF = 2) at Each End

$N_f := 8$ Number of Longitudinal Reinforcement Bars per Layer at Each End

$d_{solve} := 0.8 \cdot w_s = 20.8\ ft$ Depth of Lever Arm

Solve Block — Guess Values / Constraints

$A_{sreq} := 1\ in^2$

$\phi_f \cdot A_{sreq} \cdot f_y \cdot \left(d_{solve} - \dfrac{A_{sreq} \cdot f_y}{0.85 \cdot f_c \cdot t \cdot 2}\right) - M_u = 0$

$A_{sreq\_flex} := \text{Find}\left(A_{sreq}\right)$

$A_{sreq\_flex} = 0.76\ in^2$ Required Area of Flexural Steel

$A_{s\_long} := N_f \cdot N_{Lf} \cdot A_{sNof} = 4.96\ in^2$ Area of Flexural Steel

$DCR_f := \dfrac{A_{sreq\_flex}}{A_{s\_long}}$ $DCR_f = 0.15$

$Flag_f = \text{“USE (16) \#5 BARS EACH END MIN”}$

----------------------- VERIFY SECTION BELOW APPLIES. IT IS NOT A REQUIREMENT FOR PRECAST WALLS OR TILT UP PANELS PER ACI 318-19 18.11.2.1-----------------------

Req'd Long Reinf at Wall Ends- ACI 318-19 18.10.2.4

If wall section hw/lw >2, requirements of ACI 318-19 18.10.2.4 must be met.

$$WallEndCheck := \text{if}\left(\frac{h_{roof}}{w_s} \geq 2, \text{“Provide Req'd Reinf at Wall End”}, \text{“Not Applicable”}\right)$$

$$WallEndCheck = \text{“Not Applicable”}$$

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The amount of reinforcement required for leg in-plane moment is usually lower than the 400/fy ratio stated in 18.10.6.5 (OOP demands typically govern for these bars) and the minimum spacing requirements in Table 18.10.6.5b should not need to be met in that case.

Transverse Reinforcement Requirements - ACI 318-19 18.10.6.5

$$A_{s\_check} := \frac{400}{60000} \cdot \left(0.2 \cdot w_s\right) \cdot t$$

Assumes 0.2*Panel Width for N.A. depth. Input leg into SP Column if req'd to refine results.

$$V_{check} := \lambda \cdot \sqrt{\frac{f_c}{psi}} \cdot psi \cdot A_{cv}$$

Check shear against 18.10.6.5a

$$Check_{18.10.6.5a} := \text{if}\left(V_e > V_{check}, \text{“PROVIDE HOOKED HORIZ BARS OR HOOPED ENDS”}, \text{“OK”}\right)$$

$$Check_{18.10.6.5a} = \text{“OK”}$$

$$Check_{18.10.6.5b} := \text{if}\left(A_{sreq\_flex} > A_{s\_check}, \text{“PROVIDE CONFINING HOOPS @ 5'' O.C. MAX per 18.10.6.5”}, \text{“OK”}\right)$$

$$Check_{18.10.6.5b} = \text{“OK”}$$

Basic Boundary Element Check w/ Gross Section Properties - ACI 318-19 18.10.6.3

$$I_g := \frac{(w_s)^3 \cdot t}{12} \qquad A_g := \frac{w_s \cdot t}{2} \qquad S_g := \frac{t \cdot w_s^2}{6}$$

$P_{u1} := P_{u1c} = 173.1\ k$

Factored Axial Load - ASCE 7-16 2.3.6
Load Combo 6

$$\sigma_u := \frac{M_u}{S_g} + \frac{P_{u1}}{A_g}$$

Combined Stress @ Extreme Fiber

$$F_c := 0.2 \cdot f_c$$

$$DCR_\sigma := \frac{\sigma_u}{F_c}$$

$DCR_\sigma = 0.25$

$BasicCheck :=$ if $(DCR_\sigma > 1, \text{"Perform Alternate Analysis"}, \text{"Okay"})$

$BasicCheck = \text{"Okay"}$

If Wall does not meet basic boundary element check, provide boundary elements or perform Alternate check per ACI 318-19 18.10.6.2 ONLY if hwcs/ w_s ≥ 2.

$AlternateCheck :=$
if $\frac{h_{wcs}}{w_s} \ge 2 \wedge BasicCheck = \text{"Perform Alternate Analysis"}$
  "Proceed"
also if $BasicCheck = \text{"Okay"}$
  "Not Applicable"
else
  "Boundary Element Necessary – Alternate Check Not Allowed"

$AlternateCheck = \text{"Not Applicable"}$

Verify Section Is Uncracked

$$y_t := \frac{w_s}{2}$$

Neutral Axis of Gross Section

$$M_{cr} := \frac{7.5 \cdot \lambda \cdot \sqrt{\frac{f_c}{psi}} \cdot psi \cdot I_g}{y_t}$$

Cracking Moment - ACI 318-19 19.2.3.1 Eq 19.2.3.1, 24.2.3.5 Eq 24.2.3.5

$$M_a := M_u \cdot EL_{asd\_factor}$$

Service Moment

$M_{crCheck} :=$ if $(M_{cr} < M_a, \text{"Cracked; If necessary adjust relative stiffness and evaluate load"}, \text{"Uncracked"})$

$M_{crCheck} = \text{"Uncracked"}$

Panel 15 - Leg A (4'-10") Design

Wall Shear Design - Leg

$Label_{leg} := $ A

$leg := leg_{temp_{Label_{leg}}} = 4.83\ ft$ $\quad k_{leg} := k_{temp_{Label_{leg}}} = 0.48$

$w_o := w_{temp_{Label_{leg}}} = 3.33\ ft$ $\quad ht_o := ht_{temp_{Label_{leg}}} = 7.17\ ft$

$$P_{u1} := \left(\left(1.2 + 0.2 \cdot S_{ds}\right) \cdot P_{tot} + \left(0.2 \cdot SL + LL\right)\ w_s\right) \left(\frac{leg + \frac{w_o}{2}}{w_s}\right) = 42.1\ k$$

ASCE 7-16 12.2.3.6 Eq 6

$$P_{u2} := \left(0.9 - 0.2 \cdot S_{ds}\right)\ P_{tot} \left(\frac{leg + \frac{w_o}{2}}{w_s}\right) = 22.1\ k$$

ASCE 7-16 12.2.3.6 Eq 7

$M_{pr} := 978\ k \cdot ft$

Largest probable flexural strength of the section from SP Column assuming tensile stress of 1.25*Fy and Φ = 1.0

Pier Check - ACI 318-19 Table R18.10.1

If wall section is a pier, requirements of ACI 318-19 18.10.8 or 18.14 must be met

$BC_{pier} := $ Boundary condition: Fixed–Fixed

Boundary Condition Pier Factor

BC_{pier} **= 0.5 for fixed-fixed,**

BC_{pier} **= 1.0 for fixed-pinned**

$$Check_1 := \text{if}\left(\frac{leg}{t_p} \le 6, \text{"Pier"}, \text{"Segment"}\right)$$

$Check_1 = $ "Segment"

$$PierCheck := \text{if}\left(\frac{ht_o}{leg} \ge 2, Check_1, \text{"Segment"}\right)$$

$PierCheck = $ "Segment"

Design Shear Force - ACI 318-19 18.10.3

$V_u := \rho \cdot V_{tot} \cdot k_{leg} = 15.41\ k$ — Ultimate Leg Shear (ρQ_e)

$h_{wcsleg} := h_{roof} = 35.75\ ft$ — Height of panel leg above critical section

$stories_{wcs} := 1$ — Number of Stories above Critical Section

$$n_s := \max\left(stories_{wcs}, 0.007 \cdot \frac{h_{wcsleg}}{in}\right)$$

$$\omega_v := \begin{cases} \text{if } \dfrac{h_{wcsleg}}{leg} < 2 \\ \quad 1.0 \\ \text{else if } n_s \le 6.0 \\ \quad 0.9 + \dfrac{n_s}{10} \\ \text{else} \\ \quad min\left(1.3 + \dfrac{n_s}{30}, 1.8\right) \end{cases} = 1.2$$

Dynamic shear amplification factor, ACI 318-19 18.10.3.1.3

$M_u := V_u \cdot BC_{pier} \cdot ht_o = 55.2\ k \cdot ft$

$$M_{prratio} := \frac{M_{pr}}{M_u}$$

$$\Omega_v := \text{if}\left(\frac{h_{wcsleg}}{leg} \le 1.5, 1.0, \max\left(M_{prratio}, 1.5\right)\right)$$

$\Omega_v = 17.71$ — Overstrength factor, ACI 318-19 18.10.3.1.2

$V_{ratio} := min\left(\omega_v \cdot \Omega_v, 3\right) = 3$

$$V_{epiera} := \text{if}\left(\frac{leg}{t} > 2.5, V_u \cdot \Omega_o, \text{“N/A”}\right)$$

Alternate Ve if pier aspect ratio met
ACI 318-19 18.10.8.1a

$$V_e := \text{if}\left(PierCheck = \text{“Pier”}, \text{if}\left(V_{epiera} = \text{“N/A”}, \frac{M_{pr}}{BC_{pier} \cdot ht_o}, min\left(\frac{M_{pr}}{BC_{pier} \cdot ht_o}, V_{epiera}\right)\right), V_{ratio} \cdot V_u\right)$$

$V_e = 46.2\ k$

Design Shear Force,
For Segments: ACI 318-19 18.10.3.1
For Piers: ACI 318-19 18.7.6.1 or 18.10.8.1

$Check_{Ve18.10.8.1} = \text{"Not applicable, proceed with design"}$

ACI 318-19 18.10.8.1 - Transverse reinforcement with seismic hooks shall be provided to resist shear determined per 18.7.6.1.1. Transverse reinf. spacing 6" max OC. Extend reinf. minimum 12" above/ below opening.

Shear Steel Requirements - ACI 318-19 18.10.2.1/11.6.1

$A_{cv} := t \cdot leg = 507.5\ in^2$ — Gross Area of Concrete Section

$V_{min} := \lambda \cdot \sqrt{\frac{f_c}{psi}} \cdot psi \cdot A_{cv}$ — Shear Strength to determine ρ_{min} - ACI 318-19 18.10.2.1

$\rho_{min} := \text{if}\left(V_u < V_{min}, 0.002, 0.0025\right) = 0.002$

$A_{v\_min} := (t) \cdot (12 \cdot in) \cdot (\rho_{min})$ — Minimum area of Shear Steel per Foot

Determine number of curtains of shear steel - ACI 318-19 18.10.2.2

$V_{reinf} := 2 \cdot \lambda \sqrt{\frac{f_c}{psi}} \cdot psi \cdot A_{cv}$ — Shear Strength to Determine Required Number of Layers

$Reinforcing := \text{if}\left(V_u > V_{reinf}, \text{"(2) CURTAINS REQD"}, \text{"(1) CURTAIN REQD"}\right)$

$Reinforcing = \text{"(1) CURTAIN REQD"}$

Define Shear Steel:

$No_v := \#4$ — Shear Reinforcement Bar Size

$A_{sNov} := No_{v_0} \qquad d_{sv} := No_{v_1}$ — Area of Shear Reinforcement Bar

$S := 12 \cdot in$ — Shear Reinforcement Spacing (12" max, typical)

$N_L := 2$ — Number of Layers **(CL = 1, EF = 2)**

$A_{vs} := N_L \cdot \frac{12 \cdot in}{S} \cdot A_{sNov}$ — Area of Shear Reinforcement per Foot

$\rho_t := \frac{A_{vs}}{(12 \cdot in) \cdot (t)}$

$A_{v\_min\_check} := \text{if}\left(A_{v\_min} \le A_{vs}, \text{"OK"}, \text{"REVISE!"}\right)$ — $A_{v\_min\_check} = \text{"OK"}$

$Check := \text{if}\left(V_u > V_{reinf} \wedge N_L = 1, \text{"REVISE REINF"}, \text{"OK"}\right)$ — $Check = \text{"OK"}$

Shear Wall Strength - ACI 318-19 Eq. 18.10.4.1

$$h_{w\_max} := \max\left(\frac{ht}{w_s}, \frac{ht_o}{leg}\right) = 1.57$$

ACI 318-19 18.10.4.2 - Ratio for the greatest of wall or segment needs to be considered

$$\alpha_c := \begin{cases} \text{if } h_{w\_max} < 1.5 \\ \quad 3.0 \\ \text{else if } h_{w\_max} \geq 2.0 \\ \quad 2.0 \\ \text{else} \\ \quad 2 + \dfrac{2.0 - h_{w\_max}}{0.5} \end{cases} = 2.87$$

$$\phi V_n := \phi_{vE} \cdot \left(A_{cv} \cdot \left(\alpha_c \cdot \lambda \cdot \left(\sqrt{\frac{f_c}{psi}} \cdot psi\right) + \rho_t \cdot f_y\right)\right)$$

ACI 318-19 Eq. 18.10.4.1

Define Shear Strength Max - ACI 318-19 18.10.4.4

$$\phi V_{nmax} := \phi_{vE} \cdot \left(10 \cdot \sqrt{\frac{f_c}{psi}} \cdot psi \cdot A_{cv}\right)$$

ACI 318-19 Eq. 18.10.4.4

$check := \text{if}\left(V_e \leq \phi V_{nmax}, \text{“Proceed with Design”}, \text{“Revise!”}\right)$

$check = \text{“Proceed with Design”}$

$check := \text{if}\left(\phi V_n \leq \phi V_{nmax}, \text{“OK”}, \text{“Vn MAX exceeded, reduce reinforcement”}\right)$

$check = \text{“OK”}$

$$\phi V_n := min\left(\phi V_n, \phi V_{nmax}\right) = 124.78\ k$$

$$DCR_v := \frac{V_e}{\phi V_n}$$

$DCR_v = 0.37$

$Flag_v = \text{“USE (2) LAYER /S OF \#4 BARS AT 12''OC HORIZ MIN”}$

Leg Compression Force: Per Load Case ASCE 7-16 12.2.3.6. Default is for outer legs receiving half of total panel load in addition to seismic.

$$P_{uleg} := P_{u1} + \frac{\rho \cdot M_{ot}}{w_s - \left(\dfrac{leg}{2}\right)} = 78.3\ k$$

Shear Wall Design

Flexural Steel - ACI 318-19 18.10.6

$No_f := \#5$ — Longitudinal Reinforcement Bar Size

$A_{sNof} := No_{f_0}$ $\quad d_{Nof} := No_{f_1}$ — Area of Flexural Reinforcement Bar; Diameter of Flexural Reinforcement Bar

$N_{Lf} := 2$ — Number of Layers of Longitudinal Reinforcement (CL = 1, EF = 2)

$N_f := 10$ — Number of Longitudinal Reinforcement Bars per Layer

$A_{s\_leg} := N_f \cdot N_{Lf} \cdot A_{sNof} = 6.2\ in^2$ — Total Area of Steel Provided in Leg

$M_u := V_u \cdot BC_{pier} \cdot ht_o = 55.2\ k \cdot ft$ — Ultimate Overturning Moment ($\rho\, Q_e$)

$d_{solve} := 0.8 \cdot leg = 3.87\ ft$ — Depth of Lever Arm, use variables from above where possible

Solver / Guess Values / Constraints

$$A_{sreq} := 1\ in^2$$

$$\phi_f \cdot A_{sreq} \cdot f_y \cdot \left(d_{solve} - \frac{A_{sreq} \cdot f_y}{0.85 \cdot f_c \cdot t \cdot 2} \right) - M_u = 0$$

$$A_{sreq\_flex} := \text{Find}\left(A_{sreq}\right)$$

$A_{sreq\_flex} = 0.27\ in^2$ — Required Area of Flexural Steel

$$A_{sreq\_ten} := \frac{\rho \cdot M_{ot}}{\phi_f \cdot \left(w_s - 1.5 \cdot ft\right) \cdot f_y} = 0.65\ in^2$$

Required Area of Steel to Resist Panel Overturning

$A_{stot} := A_{sreq\_flex} \cdot 2 + A_{sreq\_ten} = 1.18\ in^2$ — Required Area of Steel for Leg Flexure and Panel Overturning

$$DCR_f := \frac{A_{stot}}{A_{s\_leg}} = 0.19$$

$DCR_f = 0.19$

$Flag_f =$ "USE (20) #5 BARS EACH END MIN"

Transverse Reinforcement Requirements- ACI 318-19 18.10.6.5

$$A_{s\_check} := \frac{400}{\frac{f_y}{psi}} \cdot (0.2 \cdot leg) \cdot t$$

Assumes 0.2*leg for N.A. depth. Input leg into SP Column if req'd to refine results.

$$V_{check} := \lambda \cdot \sqrt{\frac{f_c}{psi}} \cdot psi \cdot A_{cv}$$

Check shear against 18.10.6.5a

$$Check_{18.10.6.5a} := \text{if}\left(V_u > V_{check}, \text{“PROVIDE HOOKED HORIZ BARS OR CLOSED HOOPS”}, \text{“OK”}\right)$$

$$Check_{18.10.6.5a} = \text{“OK”}$$

$$Check_{18.10.6.5b} := \text{if}\left(A_{sreq\_flex} > A_{s\_check}, \text{“PROVIDE CONFINING HOOPS @ 5″ O.C. MAX per 18.10.6.5”}, \text{“OK”}\right)$$

$$Check_{18.10.6.5b} = \text{“OK”}$$

Basic Boundary Element Check w/ Gross Section Properties - ACI 318-19 18.10.6.3

$$I_g := \frac{leg^3 \cdot t}{12} \qquad A_g := leg \cdot t \qquad S_g := \frac{t \cdot leg^2}{6}$$

$$P_u := P_{uleg}$$

Factored Axial Load - Load Combination ASCE 7-16 12.2.3.6 Eq. 6

$$\sigma_u := \frac{M_u}{S_g} + \frac{P_{uleg}}{A_g}$$

Combined Stress at Extreme Fiber

$$F_c := 0.2 \cdot f_c$$

$$DCR_\sigma := \frac{\sigma_u}{F_c} \qquad DCR_\sigma = 0.36$$

$$BasicCheck := \text{if}\left(F_c > \sigma_u, \text{“Okay”}, \text{“Perform Alternate Analysis”}\right)$$

$$BasicCheck = \text{“Okay”}$$

If Wall does not meet basic boundary element check, provide boundary elements or perform Alternate check per ACI 318-19 18.10.6.2 <u>ONLY</u> if hwcs/ w_s > 2**.**

$$AlternateCheck := \begin{array}{l} \text{if } \dfrac{h_{wcsleg}}{leg} \ge 2 \wedge BasicCheck = \text{"Perform Alternate Analysis"} \\ \quad \text{"Proceed"} \\ \text{also if } BasicCheck = \text{"Okay"} \\ \quad \text{"Not Applicable"} \\ \text{else} \\ \quad \text{"Boundary Element Necessary – Alternate Check Not Allowed"} \end{array}$$

$AlternateCheck = \text{"Not Applicable"}$

Verify Section Uncracked

$y_t := \dfrac{leg}{2}$ Neutral Axis of Gross Section

$M_{cr} := \dfrac{7.5 \cdot \lambda \cdot \sqrt{\dfrac{f_c}{psi}} \cdot psi \cdot I_g}{y_t}$ Cracking Moment - ACI 318-19 19.2.3.1 Eq 19.2.3.1, 24.2.3.5 Eq 24.2.3.5

$M_a := M_u \cdot EL_{asd\_factor}$ Service Moment

$M_{crCheck} := \text{if}\left(M_{cr} < M_a, \text{"Cracked; If necessary adjust relative stiffness and evaluate load"}, \text{"Uncracked"}\right)$

$M_{crCheck} = \text{"Uncracked"}$

----------------------- VERIFY SECTION BELOW APPLIES-----------------------

Pier Shear Reinforcing - ACI 318-19 18.10.8 - This section applies where the wall segment is a pier and does not meet the exceptions listed in the code.

$PierCheck = \text{"Segment"}$

ACI 318-19 18.10.8.2 - Transverse reinforcement with seismic hooks shall be provided to resist shear determined per 18.7.6.1.1. Transverse reinf. spacing 6" max OC. Extend reinf. minimum 12" above/below opening.

$V_{Check\_18.10.8.2} = \text{"n/a"}$ Minimum shear requirement check per ACI 318-19 18.10.8.2

------------------------ VERIFY SECTION BELOW APPLIES------------------------

Slab Shear Dowel Bars (Leg A) - Shear Friction per ACI 318-19 22.9.4

$\mu := 0.6$ — Coefficient of Friction - ACI 318-19 22.9.4.2

$L_{dowel} := leg$ — Length of wall including dowels (leg length at storefront condition)

$No_{dowel} := \#5$ — Dowel Reinforcement Bar Size

$A_{sNodowel} := No_{dowel}$

$N_{dowel} := 4$ — Number of Dowels per Leg - (4) Min

$A_{vf} := A_{sNodowel} \cdot N_{dowel} = 1.24\ in^2$ — Area of Shear Friction Steel Across Panel

$\phi V_{vf\_n} := \phi_{vE} \cdot (\mu \cdot A_{vf} \cdot f_y)$ — Allowable Shear Capacity - ACI 318-19 22.9.4.2

$V_u := \rho \cdot V_{tot} \cdot k_a = 15.41\ k$ — Ultimate Panel Shear ($\rho\, Q_e$)

$$DCR_{vf} := \frac{V_u}{\phi V_{vf\_n}}$$

$DCR_{vf} = 0.58$

$flag := \text{if}\left(\phi V_{vf\_n} \ge V_u, \text{“OK”}, \text{“NG!”}\right)$

$flag = \text{“OK”}$

Panel 15 - Leg B (6'-10") Design

Wall Shear Design - Leg

$Label_{leg} := \text{B}$

$leg := leg_{temp_{Label_{leg}}} = 6.83\ ft$ $\quad k_{leg} := k_{temp_{Label_{leg}}} = 0.49$

$w_o := w_{temp_{Label_{leg}}} = 12.33\ ft$ $\quad ht_o := ht_{temp_{Label_{leg}}} = 10\ ft$

$$P_{u1} := \left(\left(1.2 + 0.2 \cdot S_{ds}\right) \cdot P_{tot} + \left(0.2 \cdot SL + LL\right)\ w_s\right) \left(\frac{leg + \frac{w_o}{2}}{w_s}\right) = 84.2\ k$$

ASCE 7-16 12.2.3.6 Eq 6

$$P_{u2} := \left(0.9 - 0.2 \cdot S_{ds}\right)\ P_{tot} \left(\frac{leg + \frac{w_o}{2}}{w_s}\right) = 44.2\ k$$

ASCE 7-16 12.2.3.6 Eq 7

$M_{pr} := 2360\ k \cdot ft$

Largest probable flexural strength of the section from SP Column assuming tensile stress of 1.25*Fy and Φ = 1.0

Pier Check - ACI 318-19 Table R18.10.1

If wall section is a pier, requirements of ACI 318-19 18.10.8 or 18.14 must be met

$BC_{pier} := \text{Boundary condition: Fixed−Fixed}$

Boundary Condition Pier Factor
BC_{pier} = 0.5 for fixed-fixed,
BC_{pier} = 1.0 for fixed-pinned

$Check_1 := \text{if}\left(\frac{leg}{t_p} \le 6, \text{"Pier"}, \text{"Segment"}\right)$ $\qquad Check_1 = \text{"Segment"}$

$PierCheck := \text{if}\left(\frac{ht_o}{leg} \ge 2, Check_1, \text{"Segment"}\right)$ $\qquad PierCheck = \text{"Segment"}$

Design Shear Force - ACI 318-19 18.10.3

$V_u := \rho \cdot V_{tot} \cdot k_{leg} = 15.79\ k$ — Ultimate Leg Shear ($\rho\, Q_e$)

$h_{wcsleg} := h_{roof} = 35.75\ ft$ — Height of panel leg above critical section

$stories_{wcs} := 1$ — Number of Stories above Critical Section

$$n_s := \max\left(stories_{wcs}, 0.007 \cdot \frac{h_{wcsleg}}{in}\right)$$

$$\omega_v := \begin{array}{l} \text{if } \dfrac{h_{wcsleg}}{leg} < 2 \\ \quad 1.0 \\ \text{else if } n_s \le 6.0 \\ \quad 0.9 + \dfrac{n_s}{10} \\ \text{else} \\ \quad min\left(1.3 + \dfrac{n_s}{30}, 1.8\right) \end{array} = 1.2$$

Dynamic shear amplification factor,
ACI 318-19 18.10.3.1.3

$M_u := V_u \cdot BC_{pier} \cdot ht_o = 78.9\ k \cdot ft$

$$M_{prratio} := \frac{M_{pr}}{M_u}$$

$$\Omega_v := \text{if}\left(\frac{h_{wcsleg}}{leg} \le 1.5, 1.0, \max\left(M_{prratio}, 1.5\right)\right)$$

$\Omega_v = 29.89$ — Overstrength factor, ACI 318-19 18.10.3.1.2

$V_{ratio} := min\left(\omega_v \cdot \Omega_v, 3\right) = 3$

$$V_{epiera} := \text{if}\left(\frac{leg}{t} > 2.5, V_u \cdot \Omega_o, \text{“N/A”}\right)$$

Alternate Ve if pier aspect ratio met
ACI 318-19 18.10.8.1a

$$V_e := \text{if}\left(PierCheck = \text{“Pier”}, \text{if}\left(V_{epiera} = \text{“N/A”}, \frac{M_{pr}}{BC_{pier} \cdot ht_o}, min\left(\frac{M_{pr}}{BC_{pier} \cdot ht_o}, V_{epiera}\right)\right), V_{ratio} \cdot V_u\right)$$

$V_e = 47.4\ k$

Design Shear Force,
For Segments: ACI 318-19 18.10.3.1
For Piers: ACI 318-19 18.7.6.1 or 18.10.8.1

$Check_{Ve18.10.8.1} = \text{"Not applicable, proceed with design"}$

ACI 318-19 18.10.8.1 - Transverse reinforcement with seismic hooks shall be provided to resist shear determined per 18.7.6.1.1. Transverse reinf. spacing 6" max OC. Extend reinf. minimum 12" above/below opening.

Shear Steel Requirements - ACI 318-19 18.10.2.1/11.6.1

$A_{cv} := t \cdot leg = 717.5\ in^2$ — Gross Area of Concrete Section

$V_{min} := \lambda \cdot \sqrt{\dfrac{f_c}{psi}} \cdot psi \cdot A_{cv}$ — Shear Strength to determine ρ_{min} - ACI 318-19 18.10.2.1

$\rho_{min} := \text{if}\left(V_u < V_{min}, 0.002, 0.0025\right) = 0.002$

$A_{v\_min} := (t) \cdot (12 \cdot in) \cdot (\rho_{min})$ — Minimum area of Shear Steel per Foot

Determine number of curtains of shear steel - ACI 318-19 18.10.2.2

$V_{reinf} := 2 \cdot \lambda \sqrt{\dfrac{f_c}{psi}} \cdot psi \cdot A_{cv}$ — Shear Strength to Determine Required Number of Layers

$Reinforcing := \text{if}\left(V_u > V_{reinf}, \text{"(2) CURTAINS REQD"}, \text{"(1) CURTAIN REQD"}\right)$

$Reinforcing = \text{"(1) CURTAIN REQD"}$

Define Shear Steel:

$No_v := \#4$ — Shear Reinforcement Bar Size

$A_{sNov} := No_{v_0} \qquad d_{sv} := No_{v_1}$ — Area of Shear Reinforcement Bar

$S := 12 \cdot in$ — Shear Reinforcement Spacing (12" max, typical)

$N_L := 2$ — Number of Layers **(CL = 1, EF = 2)**

$A_{vs} := N_L \cdot \dfrac{12 \cdot in}{S} \cdot A_{sNov}$ — Area of Shear Reinforcement per Foot

$\rho_t := \dfrac{A_{vs}}{(12 \cdot in) \cdot (t)}$

$A_{v\_min\_check} := \text{if}\left(A_{v\_min} \le A_{vs}, \text{"OK"}, \text{"REVISE!"}\right)$ — $A_{v\_min\_check} = \text{"OK"}$

$Check := \text{if}\left(V_u > V_{reinf} \wedge N_L = 1, \text{"REVISE REINF"}, \text{"OK"}\right)$ — $Check = \text{"OK"}$

Shear Wall Strength - ACI 318-19 Eq. 18.10.4.1

$$h_{w\_max} := \max\left(\frac{ht}{w_s}, \frac{ht_o}{leg}\right) = 1.57$$

ACI 318-19 18.10.4.2 - Ratio for the greatest of wall or segment needs to be considered

$$\alpha_c := \begin{cases} \text{if } h_{w\_max} < 1.5 \\ \quad 3.0 \\ \text{else if } h_{w\_max} \geq 2.0 \\ \quad 2.0 \\ \text{else} \\ \quad 2 + \dfrac{2.0 - h_{w\_max}}{0.5} \end{cases} = 2.87$$

$$\phi V_n := \phi_{vE} \cdot \left(A_{cv} \cdot \left(\alpha_c \cdot \lambda \cdot \left(\sqrt{\frac{f_c}{psi}} \cdot psi\right) + \rho_t \cdot f_y\right)\right)$$

ACI 318-19 Eq. 18.10.4.1

Define Shear Strength Max - ACI 318-19 18.10.4.4

$$\phi V_{nmax} := \phi_{vE} \cdot \left(10 \cdot \sqrt{\frac{f_c}{psi}} \cdot psi \cdot A_{cv}\right)$$

ACI 318-19 Eq. 18.10.4.4

$check := \text{if}\left(V_e \leq \phi V_{nmax}, \text{“Proceed with Design”}, \text{“Revise!”}\right)$

$check = \text{“Proceed with Design”}$

$check := \text{if}\left(\phi V_n \leq \phi V_{nmax}, \text{“OK”}, \text{“Vn MAX exceeded, reduce reinforcement”}\right)$

$check = \text{“OK”}$

$$\phi V_n := min\left(\phi V_n, \phi V_{nmax}\right) = 176.42\ k$$

$$DCR_v := \frac{V_e}{\phi V_n}$$

$DCR_v = 0.27$

$Flag_v = \text{“USE (2) LAYER /S OF \#4 BARS AT 12''OC HORIZ MIN”}$

Leg Compression Force: Per Load Case ASCE 7-16 12.2.3.6. Default is for outer legs receiving half of total panel load in addition to seismic.

$$P_{uleg} := P_{u1} + \frac{\rho \cdot M_{ot}}{w_s - \left(\dfrac{leg}{2}\right)} = 122\ k$$

Flexural Steel - ACI 318-19 18.10.6

$No_f := \#5$ — Longitudinal Reinforcement Bar Size

$A_{sNof} := No_{f_0}$ $\quad d_{Nof} := No_{f_1}$ — Area of Flexural Reinforcement Bar; Diameter of Flexural Reinforcement Bar

$N_{Lf} := 2$ — Number of Layers of Longitudinal Reinforcement (CL = 1, EF = 2)

$N_f := 18$ — Number of Longitudinal Reinforcement Bars per Layer

$A_{s\_leg} := N_f \cdot N_{Lf} \cdot A_{sNof} = 11.16\ in^2$ — Total Area of Steel Provided in Leg

$M_u := V_u \cdot BC_{pier} \cdot ht_o = 78.9\ k \cdot ft$ — Ultimate Overturning Moment ($\rho\, Q_e$)

$d_{solve} := 0.8 \cdot leg = 5.47\ ft$ — Depth of Lever Arm, use variables from above where possible

Solve Block: Guess Values / Constraints

$$A_{sreq} := 1\ in^2$$

$$\phi_f \cdot A_{sreq} \cdot f_y \cdot \left(d_{solve} - \frac{A_{sreq} \cdot f_y}{0.85 \cdot f_c \cdot t \cdot 2}\right) - M_u = 0$$

$$A_{sreq\_flex} := \text{Find}\left(A_{sreq}\right)$$

$A_{sreq\_flex} = 0.27\ in^2$ — Required Area of Flexural Steel

$A_{sreq\_ten} := \dfrac{\rho \cdot M_{ot}}{\phi_f \cdot (w_s - 1.5 \cdot ft) \cdot f_y} = 0.65\ in^2$ — Required Area of Steel to Resist Panel Overturning

$A_{stot} := A_{sreq\_flex} \cdot 2 + A_{sreq\_ten} = 1.18\ in^2$ — Required Area of Steel for Leg Flexure and Panel Overturning

$DCR_f := \dfrac{A_{stot}}{A_{s\_leg}} = 0.11$ $\qquad DCR_f = 0.11$

$Flag_f =$ "USE (36) #5 BARS EACH END MIN"

Transverse Reinforcement Requirements- ACI 318-19 18.10.6.5

$$A_{s\_check} := \frac{400}{\frac{f_y}{psi}} \cdot (0.2 \cdot leg) \cdot t$$

Assumes 0.2*leg for N.A. depth. Input leg into SP Column if req'd to refine results.

$$V_{check} := \lambda \cdot \sqrt{\frac{f_c}{psi}} \cdot psi \cdot A_{cv}$$

Check shear against 18.10.6.5a

$$Check_{18.10.6.5a} := \text{if}\left(V_u > V_{check}, \text{"PROVIDE HOOKED HORIZ BARS OR CLOSED HOOPS"}, \text{"OK"}\right)$$

$$Check_{18.10.6.5a} = \text{"OK"}$$

$$Check_{18.10.6.5b} := \text{if}\left(A_{sreq\_flex} > A_{s\_check}, \text{"PROVIDE CONFINING HOOPS @ 5'' O.C. MAX per 18.10.6.5"}, \text{"OK"}\right)$$

$$Check_{18.10.6.5b} = \text{"OK"}$$

Basic Boundary Element Check w/ Gross Section Properties - ACI 318-19 18.10.6.3

$$I_g := \frac{leg^3 \cdot t}{12} \qquad A_g := leg \cdot t \qquad S_g := \frac{t \cdot leg^2}{6}$$

$$P_u := P_{uleg}$$

Factored Axial Load - Load Combination ASCE 7-16 12.2.3.6 Eq. 6

$$\sigma_u := \frac{M_u}{S_g} + \frac{P_{uleg}}{A_g}$$

Combined Stress at Extreme Fiber

$$F_c := 0.2 \cdot f_c$$

$$DCR_\sigma := \frac{\sigma_u}{F_c}$$

$$DCR_\sigma = 0.33$$

$$BasicCheck := \text{if}\left(F_c > \sigma_u, \text{"Okay"}, \text{"Perform Alternate Analysis"}\right)$$

$$BasicCheck = \text{"Okay"}$$

If Wall does not meet basic boundary element check, provide boundary elements or perform Alternate check per ACI 318-19 18.10.6.2 ONLY if hwcs/ w_s > 2.

$$AlternateCheck := \begin{cases} \text{if } \dfrac{h_{wcsleg}}{leg} \geq 2 \wedge BasicCheck = \text{“Perform Alternate Analysis”} \\ \quad \text{“Proceed”} \\ \text{also if } BasicCheck = \text{“Okay”} \\ \quad \text{“Not Applicable”} \\ \text{else} \\ \quad \text{“Boundary Element Necessary – Alternate Check Not Allowed”} \end{cases}$$

$$AlternateCheck = \text{“Not Applicable”}$$

Verify Section Uncracked

$$y_t := \frac{leg}{2}$$ Neutral Axis of Gross Section

$$M_{cr} := \frac{7.5 \cdot \lambda \cdot \sqrt{\dfrac{f_c}{psi}} \cdot psi \cdot I_g}{y_t}$$ Cracking Moment - ACI 318-19 19.2.3.1 Eq 19.2.3.1, 24.2.3.5 Eq 24.2.3.5

$$M_a := M_u \cdot EL_{asd\_factor}$$ Service Moment

$$M_{crCheck} := \text{if}\left(M_{cr} < M_a, \text{“Cracked; If necessary adjust relative stiffness and evaluate load”}, \text{“Uncracked”}\right)$$

$$M_{crCheck} = \text{“Uncracked”}$$

----------------------- VERIFY SECTION BELOW APPLIES-----------------------

Pier Shear Reinforcing - ACI 318-19 18.10.8 - This section applies where the wall segment is a pier and does not meet the exceptions listed in the code.

$$PierCheck = \text{“Segment”}$$

ACI 318-19 18.10.8.2 - Transverse reinforcement with seismic hooks shall be provided to resist shear determined per 18.7.6.1.1. Transverse reinf. spacing 6" max OC. Extend reinf. minimum 12" above/ below opening.

$$V_{Check\_18.10.8.2} = \text{“n/a”}$$ Minimum shear requirement check per ACI 318-19 18.10.8.2

----------------------- VERIFY SECTION BELOW APPLIES-----------------------

Slab Shear Dowel Bars (Leg A) - Shear Friction per ACI 318-19 22.9.4

$\mu := 0.6$ — Coefficient of Friction - ACI 318-19 22.9.4.2

$L_{dowel} := leg$ — Length of wall including dowels (leg length at storefront condition)

$No_{dowel} := \#5$ — Dowel Reinforcement Bar Size

$A_{sNodowel} := No_{dowel}$

$N_{dowel} := 4$ — Number of Dowels per Leg - (4) Min

$A_{vf} := A_{sNodowel} \cdot N_{dowel} = 1.24\ in^2$ — Area of Shear Friction Steel Across Panel

$\phi V_{vf\_n} := \phi_{vE} \cdot \left(\mu \cdot A_{vf} \cdot f_y\right)$ — Allowable Shear Capacity - ACI 318-19 22.9.4.2

$V_u := \rho \cdot V_{tot} \cdot k_a = 15.41\ k$ — Ultimate Panel Shear ($\rho\, Q_e$)

$DCR_{vf} := \dfrac{V_u}{\phi V_{vf\_n}}$ $\qquad DCR_{vf} = 0.58$

$flag := \text{if}\left(\phi V_{vf\_n} \ge V_u, \text{“OK”}, \text{“NG!”}\right)$ $\qquad flag = \text{“OK”}$

Panel 15 - Leg C (2'-0") Design

Wall Shear Design - Leg

$Label_{leg} := \text{C}$

$leg := leg_{temp_{Label_{leg}}} = 2\ ft$

$k_{leg} := k_{temp_{Label_{leg}}} = 0.03$

$w_o := w_{temp_{Label_{leg}}} = 9\ ft$

$ht_o := ht_{temp_{Label_{leg}}} = 10\ ft$

$$P_{u1} := \left(\left(1.2 + 0.2 \cdot S_{ds}\right) \cdot P_{tot} + \left(0.5 \cdot SL + LL\right)\ w_s\right) \left(\frac{leg + \frac{w_o}{2}}{w_s}\right) = 43.3\ k$$

ASCE 7-16 12.2.3.6 Eq 6

$$P_{u2} := \left(0.9 - 0.2 \cdot S_{ds}\right)\ P_{tot} \left(\frac{leg + \frac{w_o}{2}}{w_s}\right) = 22.1\ k$$

ASCE 7-16 12.2.3.6 Eq 7

$M_{pr} := 297\ k \cdot ft$

Largest probable flexural strength of the section from SP Column assuming tensile stress of 1.25*Fy and Φ = 1.0

Pier Check - ACI 318-19 Table R18.10.1

If wall section is a pier, requirements of ACI 318-19 18.10.8 or 18.14 must be met

$BC_{pier} := \text{Boundary condition: Fixed–Fixed}$

Boundary Condition Pier Factor

BC_{pier} **= 0.5 for fixed-fixed,**

BC_{pier} **= 1.0 for fixed-pinned**

$$Check_1 := \text{if}\left(\frac{leg}{t_p} \le 6, \text{“Pier”}, \text{“Segment”}\right)$$

$Check_1 = \text{“Pier”}$

$$PierCheck := \text{if}\left(\frac{ht_o}{leg} \ge 2, Check_1, \text{“Segment”}\right)$$

$PierCheck = \text{“Pier”}$

Shear Wall Design

Design Shear Force - ACI 318-19 18.10.3

$V_u := \rho \cdot V_{tot} \cdot k_{leg} = 0.85\ \boldsymbol{k}$ — Ultimate Leg Shear (ρQ_e)

$h_{wcsleg} := h_{roof} = 35.75\ \boldsymbol{ft}$ — Height of panel leg above critical section

$stories_{wcs} := 1$ — Number of Stories above Critical Section

$$n_s := \max\left(stories_{wcs}, 0.007 \cdot \frac{h_{wcsleg}}{\boldsymbol{in}}\right)$$

$$\omega_v := \begin{Vmatrix} \text{if } \dfrac{h_{wcsleg}}{leg} < 2 \\ \quad \| \ 1.0 \\ \text{else if } n_s \le 6.0 \\ \quad \| \ 0.9 + \dfrac{n_s}{10} \\ \text{else} \\ \quad \| \ min\left(1.3 + \dfrac{n_s}{30}, 1.8\right) \end{Vmatrix} = 1.2$$

Dynamic shear amplification factor, ACI 318-19 18.10.3.1.3

$M_u := V_u \cdot BC_{pier} \cdot ht_o = 4.2\ \boldsymbol{k} \cdot \boldsymbol{ft}$

$$M_{prratio} := \frac{M_{pr}}{M_u}$$

$$\Omega_v := \text{if}\left(\frac{h_{wcsleg}}{leg} \le 1.5, 1.0, \max\left(M_{prratio}, 1.5\right)\right)$$

$\Omega_v = 70$ — Overstrength factor, ACI 318-19 18.10.3.1.2

$V_{ratio} := min\left(\omega_v \cdot \Omega_v, 3\right) = 3$

$$V_{epiera} := \text{if}\left(\frac{leg}{t} > 2.5, V_u \cdot \Omega_o, \text{"N/A"}\right)$$

Alternate Ve if pier aspect ratio met
ACI 318-19 18.10.8.1a

$$V_e := \text{if}\left(PierCheck = \text{"Pier"}, \text{if}\left(V_{epiera} = \text{"N/A"}, \frac{M_{pr}}{BC_{pier} \cdot ht_o}, min\left(\frac{M_{pr}}{BC_{pier} \cdot ht_o}, V_{epiera}\right)\right), V_{ratio} \cdot V_u\right)$$

$V_e = 1.7\ \boldsymbol{k}$

Design Shear Force,
For Segments: ACI 318-19 18.10.3.1
For Piers: ACI 318-19 18.7.6.1 or 18.10.8.1

$Check_{Ve18.10.8.1} = \text{"Refer to 18.10.8.1 for additional transverse reinforcement requirements"}$

ACI 318-19 18.10.8.1 - Transverse reinforcement with seismic hooks shall be provided to resist shear determined per 18.7.6.1.1. Transverse reinf. spacing 6" max OC. Extend reinf. minimum 12" above/ below opening.

Shear Steel Requirements - ACI 318-19 18.10.2.1/11.6.1

$A_{cv} := t \cdot leg = 210\ in^2$ — Gross Area of Concrete Section

$V_{min} := \lambda \cdot \sqrt{\frac{f_c}{psi}} \cdot psi \cdot A_{cv}$ — Shear Strength to determine ρ_{min} - ACI 318-19 18.10.2.1

$\rho_{min} := \text{if}\left(V_u < V_{min}, 0.002, 0.0025\right) = 0.002$

$A_{v\_min} := (t) \cdot (12 \cdot in) \cdot (\rho_{min})$ — Minimum area of Shear Steel per Foot

Determine number of curtains of shear steel - ACI 318-19 18.10.2.2

$V_{reinf} := 2 \cdot \lambda \sqrt{\frac{f_c}{psi}} \cdot psi \cdot A_{cv}$ — Shear Strength to Determine Required Number of Layers

$Reinforcing := \text{if}\left(V_u > V_{reinf}, \text{"(2) CURTAINS REQD"}, \text{"(1) CURTAIN REQD"}\right)$

$Reinforcing = \text{"(1) CURTAIN REQD"}$

Define Shear Steel:

$No_v := \#3$ — Shear Reinforcement Bar Size

$A_{sNov} := No_{v_0}$ $\quad d_{sv} := No_{v_1}$ — Area of Shear Reinforcement Bar

$S := 6 \cdot in$ — Shear Reinforcement Spacing (12" max, typical)

$N_L := 2$ — Number of Layers **(CL = 1, EF = 2)**

$A_{vs} := N_L \cdot \frac{12 \cdot in}{S} \cdot A_{sNov}$ — Area of Shear Reinforcement per Foot

$\rho_t := \frac{A_{vs}}{(12 \cdot in) \cdot (t)}$

$A_{v\_min\_check} := \text{if}\left(A_{v\_min} \le A_{vs}, \text{"OK"}, \text{"REVISE!"}\right)$ $\quad A_{v\_min\_check} = \text{"OK"}$

$Check := \text{if}\left(V_u > V_{reinf} \wedge N_L = 1, \text{"REVISE REINF"}, \text{"OK"}\right)$ $\quad Check = \text{"OK"}$

Shear Wall Strength - ACI 318-19 Eq. 18.10.4.1

$$h_{w\_max} := \max\left(\frac{ht}{w_s}, \frac{ht_o}{leg}\right) = 5$$

ACI 318-19 18.10.4.2 - Ratio for the greatest of wall or segment needs to be considered

$$\alpha_c := \begin{array}{l} \text{if } h_{w\_max} < 1.5 \\ \quad 3.0 \\ \text{else if } h_{w\_max} \geq 2.0 \\ \quad 2.0 \\ \text{else} \\ \quad 2 + \dfrac{2.0 - h_{w\_max}}{0.5} \end{array} = 2$$

$$\phi V_n := \phi_{vE} \cdot \left(A_{cv} \cdot \left(\alpha_c \cdot \lambda \cdot \left(\sqrt{\frac{f_c}{psi}} \cdot psi\right) + \rho_t \cdot f_y\right)\right) = 47.62\ k$$

ACI 318-19 Eq. 18.10.4.1

Define Shear Strength Max - ACI 318-19 18.10.4.4

$$\phi V_{nmax} := \phi_{vE} \cdot \left(10 \cdot \sqrt{\frac{f_c}{psi}} \cdot psi \cdot A_{cv}\right)$$

ACI 318-19 Eq. 18.10.4.4

$check := \text{if}\left(V_e \leq \phi V_{nmax}, \text{“Proceed with Design”}, \text{“Revise!”}\right)$

$check = \text{“Proceed with Design”}$

$check := \text{if}\left(\phi V_n \leq \phi V_{nmax}, \text{“OK”}, \text{“Vn MAX exceeded, reduce reinforcement”}\right)$

$check = \text{“OK”}$

$$\phi V_n := min\left(\phi V_n, \phi V_{nmax}\right) = 47.62\ k$$

$$DCR_v := \frac{V_e}{\phi V_n}$$

$DCR_v = 0.04$

$Flag_v = \text{“USE (2) LAYER /S OF \#3 BARS AT 6''OC HORIZ MIN”}$

Leg Compression Force: Per Load Case ASCE 7-16 12.2.3.6. Default is for outer legs receiving half of total panel load in addition to seismic.

$$P_{uleg} := P_{u1} + \frac{\rho \cdot M_{ot}}{w_s - \left(\dfrac{leg}{2}\right)} = 77.4\ k$$

Flexural Steel - ACI 318-19 18.10.6

$No_f := \#5$ Longitudinal Reinforcement Bar Size

$A_{sNof} := No_{f_0}$ $d_{Nof} := No_{f_1}$ Area of Flexural Reinforcement Bar
Diameter of Flexural Reinforcement Bar

$N_{Lf} := 2$ Number of Layers of Longitudinal Reinforcement (CL = 1, EF = 2)

$N_f := 8$ Number of Longitudinal Reinforcement Bars per Layer

$A_{s\_leg} := N_f \cdot N_{Lf} \cdot A_{sNof} = 4.96\ in^2$ Total Area of Steel Provided in Leg

$M_u := V_u \cdot BC_{pier} \cdot ht_o = 4.2\ k \cdot ft$ Ultimate Overturning Moment ($\rho\, Q_e$)

$d_{solve} := 0.8 \cdot leg = 1.6\ ft$ Depth of Lever Arm, use variables from above where possible

Solve Block: Guess Values / Constraints

$$A_{sreq} := 1\ in^2$$

$$\phi_f \cdot A_{sreq} \cdot f_y \cdot \left(d_{solve} - \frac{A_{sreq} \cdot f_y}{0.85 \cdot f_c \cdot t \cdot 2}\right) - M_u = 0$$

$$A_{sreq\_flex} := \text{Find}\left(A_{sreq}\right)$$

$A_{sreq\_flex} = 0.05\ in^2$ Required Area of Flexural Steel

$$A_{sreq\_ten} := \frac{\rho \cdot M_{ot}}{\phi_f \cdot \left(w_s - 1.5 \cdot ft\right) \cdot f_y} = 0.65\ in^2$$

Required Area of Steel to Resist Panel Overturning

$A_{stot} := A_{sreq\_flex} \cdot 2 + A_{sreq\_ten} = 0.74\ in^2$ Required Area of Steel for Leg Flexure and Panel Overturning

$$DCR_f := \frac{A_{stot}}{A_{s\_leg}} = 0.15$$

$DCR_f = 0.15$

$Flag_f =$ "USE (16) #5 BARS EACH END MIN"

Transverse Reinforcement Requirements- ACI 318-19 18.10.6.5

$$A_{s\_check} := \frac{400}{\frac{f_y}{psi}} \cdot (0.2 \cdot leg) \cdot t$$

Assumes 0.2*leg for N.A. depth. Input leg into SP Column if req'd to refine results.

$$V_{check} := \lambda \cdot \sqrt{\frac{f_c}{psi}} \cdot psi \cdot A_{cv}$$

Check shear against 18.10.6.5a

$$Check_{18.10.6.5a} := \text{if}\left(V_u > V_{check}, \text{“PROVIDE HOOKED HORIZ BARS OR CLOSED HOOPS”}, \text{“OK”}\right)$$

$$Check_{18.10.6.5a} = \text{“OK”}$$

$$Check_{18.10.6.5b} := \text{if}\left(A_{sreq\_flex} > A_{s\_check}, \text{“PROVIDE CONFINING HOOPS @ 5'' O.C. MAX per 18.10.6.5”}, \text{“OK”}\right)$$

$$Check_{18.10.6.5b} = \text{“OK”}$$

Basic Boundary Element Check w/ Gross Section Properties - ACI 318-19 18.10.6.3

$$I_g := \frac{leg^3 \cdot t}{12} \qquad A_g := leg \cdot t \qquad S_g := \frac{t \cdot leg^2}{6}$$

$$P_u := P_{uleg}$$

Factored Axial Load - Load Combination ASCE 7-16 12.2.3.6 Eq. 6

$$\sigma_u := \frac{M_u}{S_g} + \frac{P_{uleg}}{A_g}$$

Combined Stress at Extreme Fiber

$$F_c := 0.2 \cdot f_c$$

$$DCR_\sigma := \frac{\sigma_u}{F_c}$$

$$DCR_\sigma = 0.54$$

$$BasicCheck := \text{if}\left(F_c > \sigma_u, \text{“Okay”}, \text{“Perform Alternate Analysis”}\right)$$

$$BasicCheck = \text{“Okay”}$$

If Wall does not meet basic boundary element check, provide boundary elements or perform Alternate check per ACI 318-19 18.10.6.2 ONLY if hwcs/ w_s > 2.

$$AlternateCheck := \begin{cases} \text{if } \frac{h_{wcsleg}}{leg} \geq 2 \wedge BasicCheck = \text{"Perform Alternate Analysis"} \\ \quad \text{"Proceed"} \\ \text{also if } BasicCheck = \text{"Okay"} \\ \quad \text{"Not Applicable"} \\ \text{else} \\ \quad \text{"Boundary Element Necessary – Alternate Check Not Allowed"} \end{cases}$$

$AlternateCheck = \text{"Not Applicable"}$

Verify Section Uncracked

$y_t := \frac{leg}{2}$ Neutral Axis of Gross Section

$$M_{cr} := \frac{7.5 \cdot \lambda \cdot \sqrt{\frac{f_c}{psi}} \cdot psi \cdot I_g}{y_t}$$

Cracking Moment - ACI 318-19 19.2.3.1 Eq 19.2.3.1, 24.2.3.5 Eq 24.2.3.5

$M_a := M_u \cdot EL_{asd\_factor}$ Service Moment

$M_{crCheck} := \text{if}\left(M_{cr} < M_a, \text{"Cracked; If necessary adjust relative stiffness and evaluate load"}, \text{"Uncracked"}\right)$

$M_{crCheck} = \text{"Uncracked"}$

------------------------ VERIFY SECTION BELOW APPLIES------------------------

Pier Shear Reinforcing - ACI 318-19 18.10.8 - This section applies where the wall segment is a pier and does not meet the exceptions listed in the code.

$PierCheck = \text{"Pier"}$

ACI 318-19 18.10.8.2 - Transverse reinforcement with seismic hooks shall be provided to resist shear determined per 18.7.6.1.1. Transverse reinf. spacing 6" max OC. Extend reinf. minimum 12" above/ below opening.

$V_{Check\_18.10.8.2} = \text{"PROVIDE 180–DEGREE HOOKS AT 6'' MAX o.c."}$

Minimum shear requirement check per ACI 318-19 18.10.8.2

$M_{pr} = 297\ k \cdot ft$ Per SP Column

$$V_{epier} := \frac{M_{pr}}{ht_o \cdot BC_{pier}} = 59.4\ k$$

Shear associated with M_{pr}

$Flag := \text{if}\left(\phi V_n > V_e, \text{"OK"}, \text{"Revise Reinf"}\right)$ $Flag = \text{"OK"}$

----------------------- VERIFY SECTION BELOW APPLIES-----------------------

Slab Shear Dowel Bars (Leg A) - Shear Friction per ACI 318-19 22.9.4

$\mu := 0.6$ Coefficient of Friction - ACI 318-19 22.9.4.2

$L_{dowel} := leg$ Length of wall including dowels (leg length at storefront condition)

$No_{dowel} := \#5$ Dowel Reinforcement Bar Size

$A_{sNodowel} := No_{dowel}$

$N_{dowel} := 4$ Number of Dowels per Leg - (4) Min

$A_{vf} := A_{sNodowel} \cdot N_{dowel} = 1.24\ in^2$ Area of Shear Friction Steel Across Panel

$\phi V_{vf\_n} := \phi_{vE} \cdot \left(\mu \cdot A_{vf} \cdot f_y\right)$ Allowable Shear Capacity - ACI 318-19 22.9.4.2

$V_u := \rho \cdot V_{tot} \cdot k_a = 15.41\ k$ Ultimate Panel Shear ($\rho\, Q_e$)

$DCR_{vf} := \dfrac{V_u}{\phi V_{vf\_n}}$ $DCR_{vf} = 0.58$

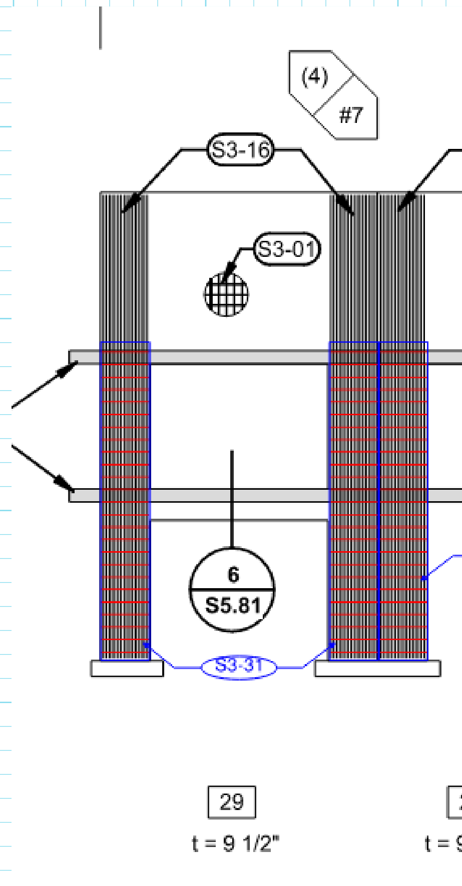
$flag := \text{if}\left(\phi V_{vf\_n} \geq V_u, \text{“OK”}, \text{“NG!”}\right)$ $flag = \text{“OK”}$

STOREFRONT PANEL LEG

P29:

disp = 0.36 in

Induced demand:
Pu = 171.3 k
Vu = 46.9 k
Mu = 515.9 k-ft



lw = 4.75 ft = 57 in
bw = 9.5 in (NO REVEAL)
hw = 12 ft

hw/lw = 12 ft / 4.75 ft - 2.53
lw/bw = 57 in / 9.5 in = 6 ⟶ SEGMENT

ac = 2

Vc = 2 sqrt(f'c)Acv = 2 sqrt(4000 psi) * (54 in x 9.5 in) = 64.9 k

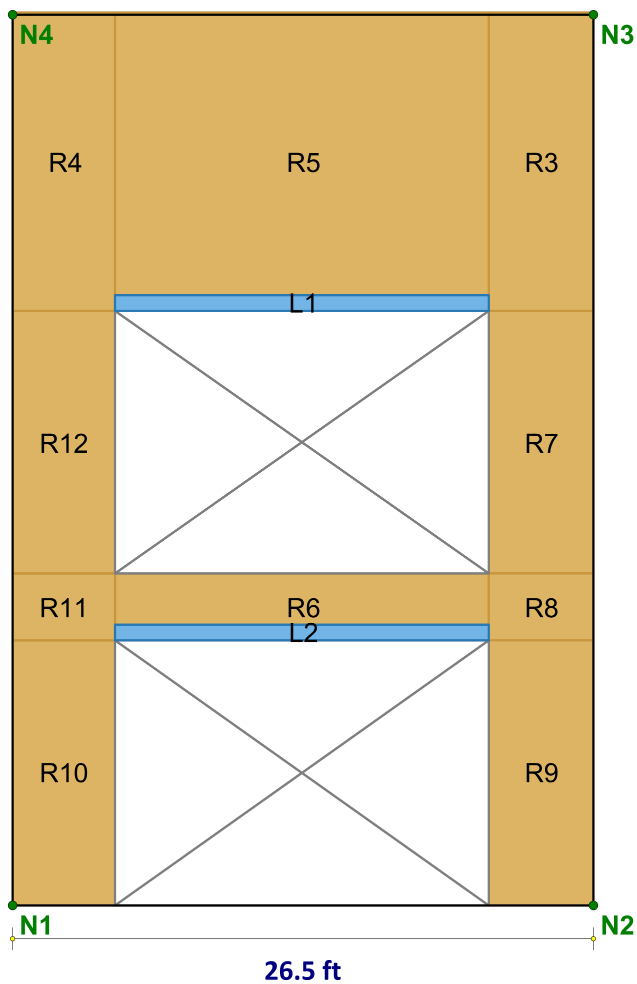
Vs = 2*0.11 in2 * 60 ksi * 54 in / 6 in = 118.8 k

phi*Vn = 0.6*(64.9 k + 118.8 k) = 110.2 k

$$DCR = \frac{46.9\ k}{110.2\ k} = 0.42$$

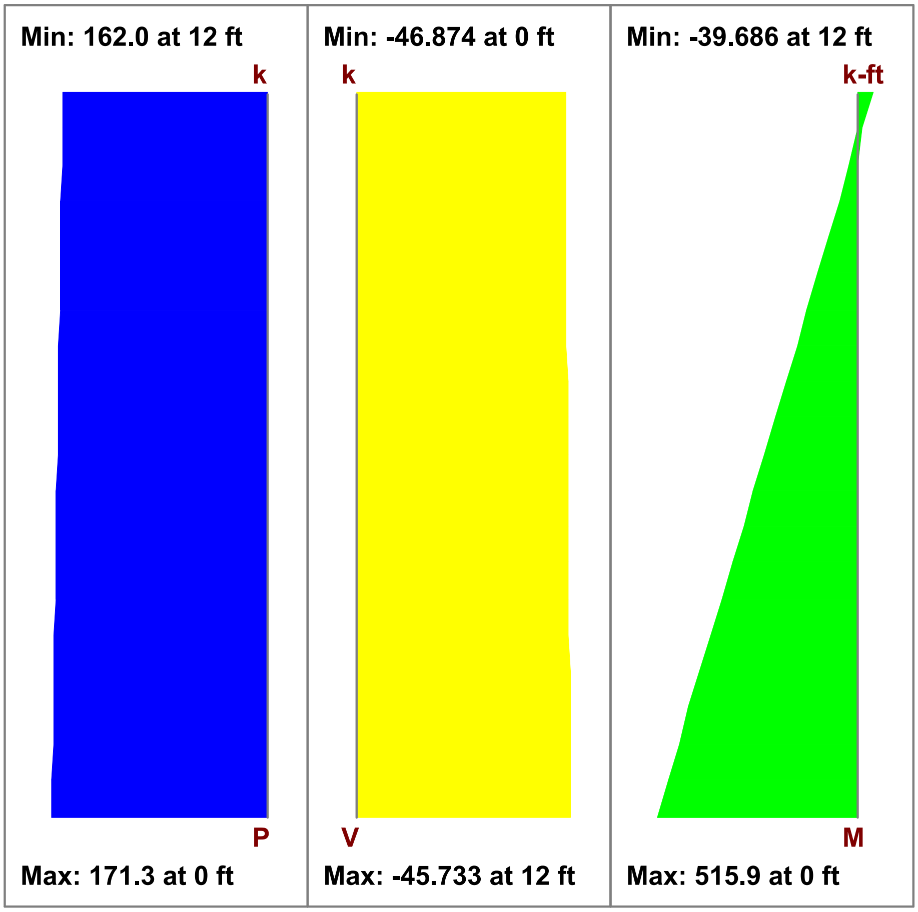
See result from SpCol attached below

CRITERIA		MATERIALS			GEOMETRY		
Code	: ACI 318-19	Material Set	: Conc4000NW		Total Height	: 40.5	ft
Design Rule	: Typical	Concrete f'c	: 4	ksi	Total Length	: 26.5	ft
Loc of r/f	: Each Face	Concrete E	: 3644	ksi	Thickness	: 9.5	in
Outer Bars	: Vertical	Concrete G	: 1584	ksi			
		Conc Density	: 0.15	k/ft^3	Int Cover (-z)	: 0.75	in
Vert Bar Size	: #5	Lambda	: 1		Ext Cover (+z)	: 1.5	in
Horz Bar Size	: #4	Conc Str Blk	: Rectangular		Cover Open/Edge	2	in
					K	: 1	
Transfer In?	: No	Vert Bar Fy	: 60	ksi	Use Cracked?	: Yes	
Transfer Out?	: No	Horz Bar Fy	: 60	ksi	In Icr Factor	: 0.7	
Group Wall?	: No	Steel E	: 29000	ksi			



CRITERIA			MATERIALS			GEOMETRY		
Code	: ACI 318-19		Material Set	: Conc4000NW		Total Height	: 12	ft
Design Rule	: Typical		Concrete f'c	: 4	ksi	Total Length	: 4.75	ft
Loc of r/f	: Each Face		Concrete E	: 3644	ksi	Thickness	: 9.5	in
Outer Bars	: Vertical		Concrete G	: 1584	ksi			
			Conc Density	: 0.15	k/ft^3	Int Cover (-z)	: 0.75	in
Vert Bar Size	: #5		Lambda	: 1		Ext Cover (+z)	: 1.5	in
Horz Bar Size	: #4		Conc Str Blk	: Rectangular		Cover Open/Edge	2	in
						K	: 1	
Vert Bar Spac	: 10	in	Vert Bar Fy	: 60	ksi	Use Cracked?	: Yes	
Horz Bar Spac	: 18	in	Horz Bar Fy	: 60	ksi	Icr Factor	: 0.7	
Group Wall?	: No		Steel E	: 29000	ksi			

ENVELOPE DIAGRAMS



ACI 318-19 Code Check

AXIAL/BENDING DETAILS

UC Max	: 0.581	
Location	: 0	ft
Gov Pu	: 171.288	k
phi*Pn	: 294.877	k
Gov Mu	: 515.906	k-ft
phi*Mn	: 888.144	k-ft
phi eff.	: 0.9	
Gov LC	: 6	

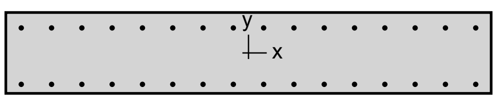
SHEAR DETAILS

UC Max	: 0.437	
Location	: 0	ft
Gov Vu	: -46.874	k
phi*Vn	: 107.331	k
Vnmax	: 273.98	k
Vc	: 68.495	k
Vs	: 74.613	k
Gov LC	: 6	

DEFLECTION DETAILS

Delta max	: 0.121	in
Deflection Ratio	: H/100	
Location	: 40.5	ft
Gov LC	: 6	

spColumn v10.00 (TM)
Computer program for the Strength Design of Reinforced Concrete Sections



Contents

List of Figures

1. General Information

File Name	H:\Projects\222029000\Production\ ...\P29 Leg.colx
Project	---
Column	---
Engineer	---
Code	ACI 318-19
Bar Set	ASTM A615
Units	English
Run Option	Investigation
Run Axis	Y - axis
Slenderness	Not Considered
Column Type	Structural
Capacity Method	Moment capacity

2. Material Properties

2.1. Concrete

Type	Standard	
f'_c	4	ksi
E_c	3605	ksi
f_c	3.4	ksi
ε_u	0.003	in/in
β_1	0.85	

2.2. Steel

Type	Standard	
f_y	60	ksi
E_s	29000	ksi
ε_{ty}	0.00206897	in/in

3. Section

3.1. Shape and Properties

Type	Rectangular	
Width	57	in
Depth	9.5	in
A_g	541.5	in^2
I_x	4072.53	in^4
I_y	146611	in^4
r_x	2.74241	in
r_y	16.4545	in
X_o	0	in
Y_o	0	in

3.2. Section Figure

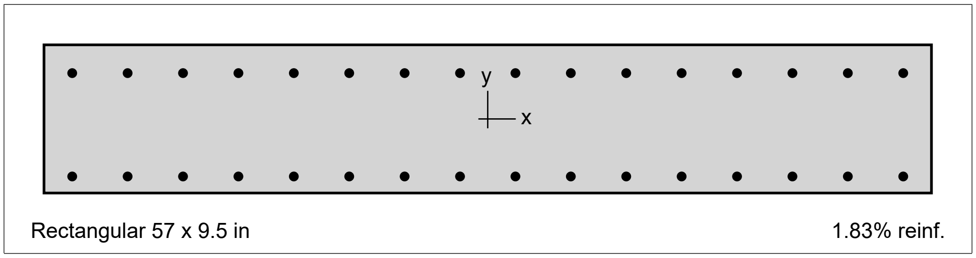


Figure 1: Column section

4. Reinforcement

4.1. Bar Set: ASTM A615

Bar	Diameter	Area	Bar	Diameter	Area	Bar	Diameter	Area
	in	in²		in	in²		in	in²
#3	0.38	0.11	#4	0.50	0.20	#5	0.63	0.31
#6	0.75	0.44	#7	0.88	0.60	#8	1.00	0.79
#9	1.13	1.00	#10	1.27	1.27	#11	1.41	1.56
#14	1.69	2.25	#18	2.26	4.00			

4.2. Confinement and Factors

Confinement type	Tied
For #10 bars or less	#3 ties
For larger bars	#4 ties
Capacity Reduction Factors	
Axial compression, (a)	0.8
Tension controlled ϕ, (b)	0.9
Compression controlled ϕ, (c)	0.65

4.3. Arrangement

Pattern	Sides different
Bar layout	Rectangular
Cover to	Longitudal bars
Clear cover	---
Bars	---
Total steel area, A_s	9.92 in²
Rho	1.83 %
Minimum clear spacing	2.93 in

4.4. Bars Provided

		Bars	Clear cover
			in
Top	16	#5	1.5
Bottom	16	#5	0.75
Left	0	#5	1.5
Right	0	#5	1.5

5. Factored Loads and Moments with Corresponding Capacity Ratios

NOTE: Calculations are based on "Moment Capacity" Method.

No.	Demand		Capacity		Parameters at Capacity			Capacity
	P_u	M_{uy}	ϕP_n	ϕM_{ny}	NA Depth	ε_t	ϕ	Ratio
	kip	k-ft	kip	k-ft	in			
1	171.30	515.90	171.30	1201.45	16.41	0.00709	0.900	0.43

STOREFRONT PANEL COUPLING BEAM

P29:

disp = 0.36 in

Induced demand:
Pu = 1.74 k
Vu = 31.8 k
Mu = 245.0 k-ft

d = 33.8125 in
bw = 9.5 in - 0.75 in = 8.75 in

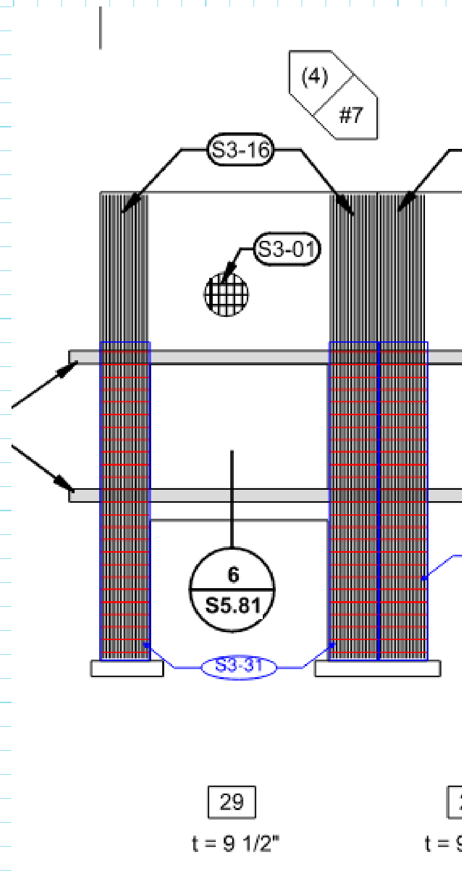
Vc = 2 sqrt(f'c)Acv = 2 sqrt(4000 psi) * (33.8 in x 8.75 in) = 37.4 k

Vs = 2*0.11 in2 * 60 ksi * 33.8 in / 15 in = 29.8 k

phi*Vn = 0.6*(37.4 k + 29.8 k) = 40.3 k

$$\text{DCR} = \frac{31.8\text{ k}}{40.3\text{ k}} = 0.78$$

See result from SpCol attached below



PROJECT NAME

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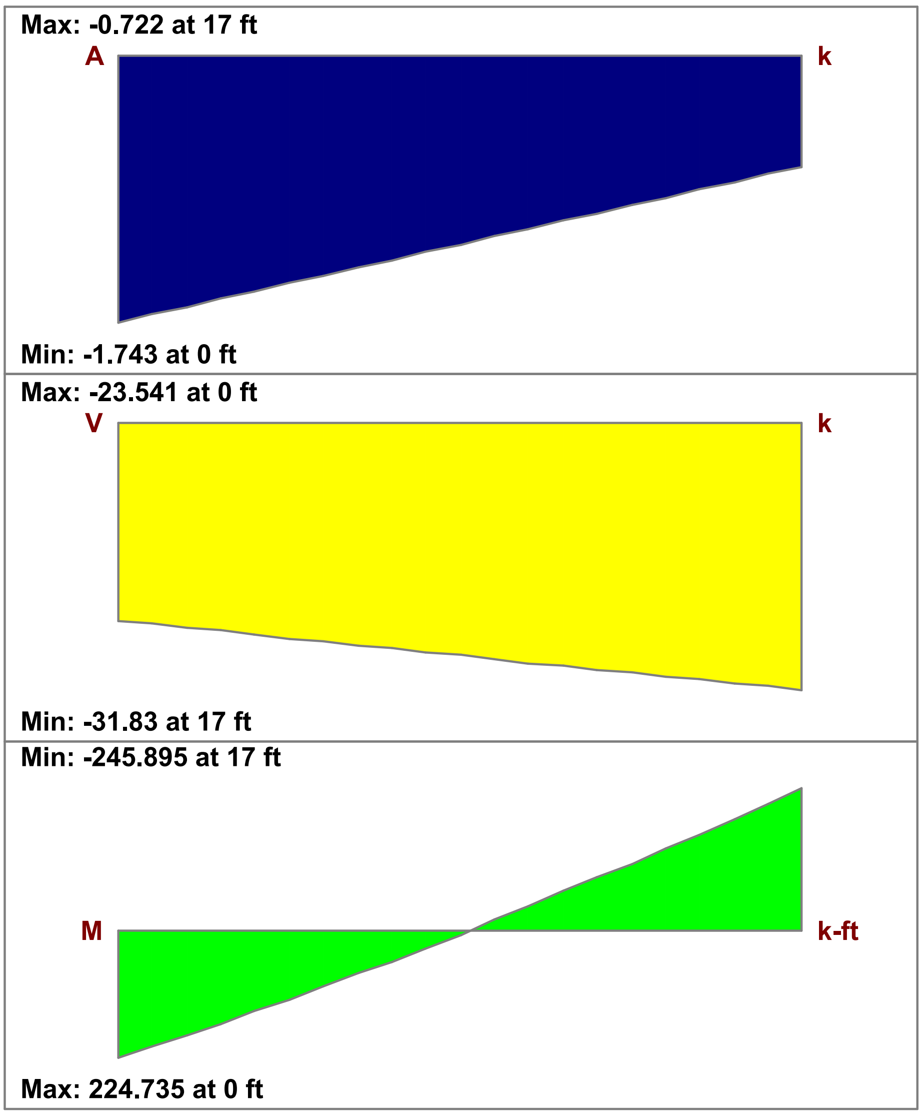
Date X/X/2023
Job# XXXXXX.XX
Sht. 1 of XX

CRITERIA		
Code	: ACI 318-19	
Design Rule	: Typical	
Loc of r/f	: Each Face	
Outer Bars	: Vertical	
Vert Bar Size	: #5	
Horz Bar Size	: #4	

MATERIALS		
Material Set	: Conc4000NW	
Concrete f'c	: 4	ksi
Concrete E	: 3644	ksi
Concrete G	: 1584	ksi
Conc Density	: 0.15	k/ft^3
Lambda	: 1	
Conc Str Blk	: Rectangular	
Vert Bar Fy	: 60	ksi
Horz Bar Fy	: 60	ksi
Steel E	: 29000	ksi

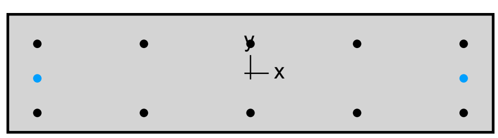
GEOMETRY		
Total Length	: 17	ft
Thickness	: 9.5	in
Int Cover (-z)	: 0.75	in
Ext Cover (+z)	: 1.5	in
Cover Open/Edge	2	in

ENVELOPE DIAGRAMS



Note: Lintel design is not currently available for concrete wall panel openings.

spColumn v10.00 (TM)
Computer program for the Strength Design of Reinforced Concrete Sections



StructurePoint

Contents

List of Figures

1. General Information

File Name	H:\Projects\222029000\P...\P29 Coupling Beam.colx
Project	---
Column	---
Engineer	---
Code	ACI 318-19
Bar Set	ASTM A615
Units	English
Run Option	Investigation
Run Axis	Y - axis
Slenderness	Not Considered
Column Type	Structural
Capacity Method	Moment capacity

2. Material Properties

2.1. Concrete

Type	Standard	
f'_c	4	ksi
E_c	3605	ksi
f_c	3.4	ksi
ε_u	0.003	in/in
β_1	0.85	

2.2. Steel

Type	Standard	
f_y	60	ksi
E_s	29000	ksi
ε_{ty}	0.00206897	in/in

3. Section

3.1. Shape and Properties

Type	Rectangular	
Width	36	in
Depth	8.75	in
A_g	315	in^2
I_x	2009.77	in^4
I_y	34020	in^4
r_x	2.52591	in
r_y	10.3923	in
X_o	0	in
Y_o	0	in

3.2. Section Figure

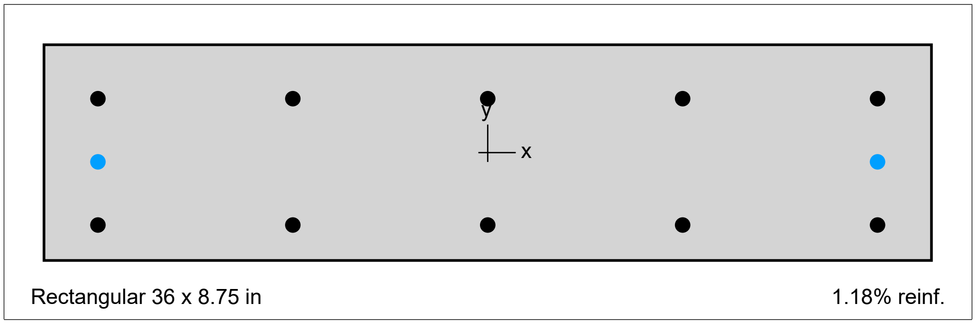


Figure 1: Column section

4. Reinforcement

4.1. Bar Set: ASTM A615

Bar	Diameter	Area	Bar	Diameter	Area	Bar	Diameter	Area
	in	in^2		in	in^2		in	in^2
#3	0.38	0.11	#4	0.50	0.20	#5	0.63	0.31
#6	0.75	0.44	#7	0.88	0.60	#8	1.00	0.79
#9	1.13	1.00	#10	1.27	1.27	#11	1.41	1.56
#14	1.69	2.25	#18	2.26	4.00			

4.2. Confinement and Factors

Confinement type	Tied
For #10 bars or less	#3 ties
For larger bars	#4 ties
Capacity Reduction Factors	
Axial compression, (a)	0.8
Tension controlled ϕ, (b)	0.9
Compression controlled ϕ, (c)	0.65

4.3. Arrangement

Pattern	Sides different
Bar layout	Rectangular
Cover to	Transverse bars
Clear cover	---
Bars	---
Total steel area, A_s	3.72 in^2
Rho	1.18 %
Minimum clear spacing	1.93 in

4.4. Bars Provided

		Bars	Clear cover
			in
Top	5	#5	1.5
Bottom	5	#5	0.75
Left	1	#5	1.5
Right	1	#5	1.5

5. Factored Loads and Moments with Corresponding Capacity Ratios

NOTE: Calculations are based on "Moment Capacity" Method.

No.	Demand		Capacity		Parameters at Capacity			Capacity
	P_u	M_{uy}	ϕP_n	ϕM_{ny}	NA Depth	ε_t	ϕ	Ratio
	kip	k-ft	kip	k-ft	in			
1	0.00	245.00	0.00	265.47	4.96	0.01746	0.900	0.92

09 MISCELLANEOUS

CANOPY LOADING

GRAVITY

$DL = 7.5\ psf$ (2.2 psf Deck, 4 psf framing, 1.3 psf misc, lighting)

$SL = 16.8\ psf$ + varies drift

$W_{TRIB} = 21.67\ ft/2 = 10.833\ ft$

$V = 97\ MPH$

$h = 37\ ft$

$q_h = 15.2\ psf$ → FROM WIND CALC

$$GC_{pn} = \begin{cases} 0.8 & \text{DOWN} \\ -0.5 & \text{UP} \end{cases}$$

$$p = \begin{cases} 12.2\ psf & \text{DOWN} \\ -7.6\ psf & \text{UP} \end{cases}$$

SEISMIC COEF

$$C_{s,p} = \frac{0.4\, a_p S_{DS}}{\frac{R_p}{I_p}}\left(1 + 2\frac{z}{h}\right) = \frac{0.4(2.5)(0.843)}{\left(\frac{2.5}{1}\right)}\left(1 + 2\left(\tfrac{1}{2}\right)\right)$$

$$= 0.674$$

WIND

C&C

$A_{eff} = (4\ ft)(4\ ft/3) = 5.33\ ft^2 \Rightarrow 10\ ft^2$

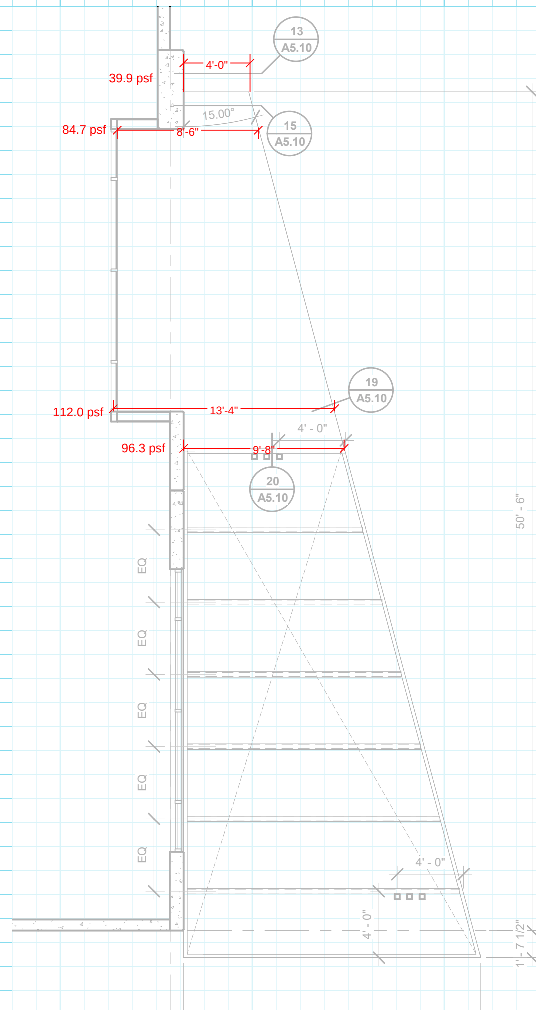
$P_{net} = 24.1\ psf \Rightarrow 30.125\ plf$ (Channel face)

By

Date

Job#

CANOPY DESIGN (EAST ELEVATION):



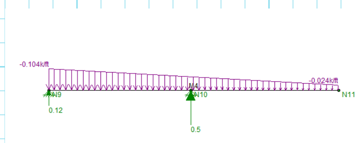
DECK DESIGN (20 GA PLB-36):

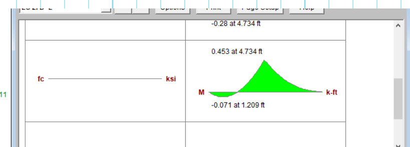
S_f = 0.230 in^3/ft
F_y = 50 ksi

M_n = 0.230 in^3/ft x 50 ksi = 11.5 k-in/ft
M_n/Omega = 11.5 k-in/ft / 1.67 = 6.9 k-in/ft => 578 lb-ft/ft

M_a = 453 lb-ft/ft

DCR = 0.78 $\longrightarrow$ OKAY





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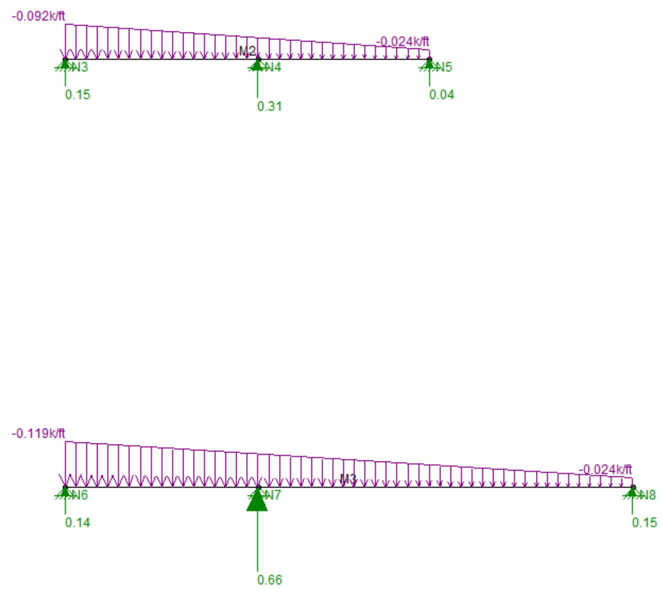
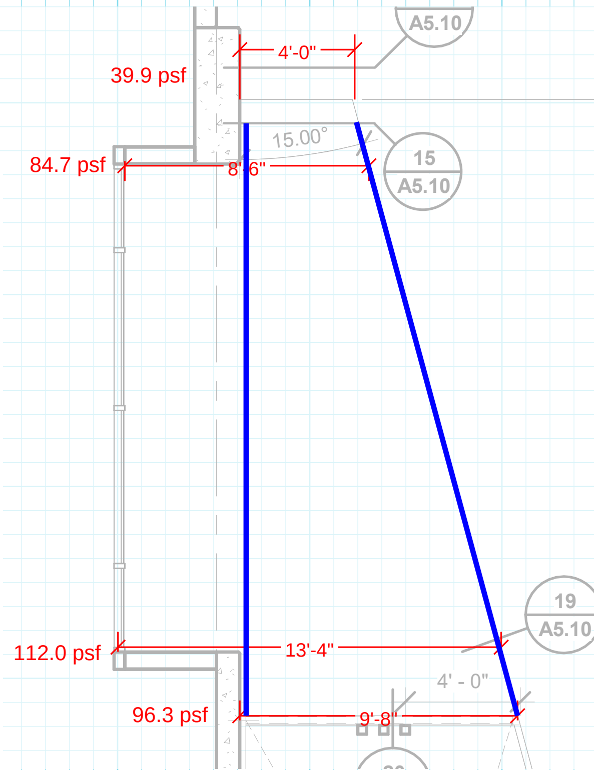
By SHA
Date 6/15/2023
Job# 2220290.00
Sht. 1 of XX

CANOPY DESIGN (EAST ELEVATION):

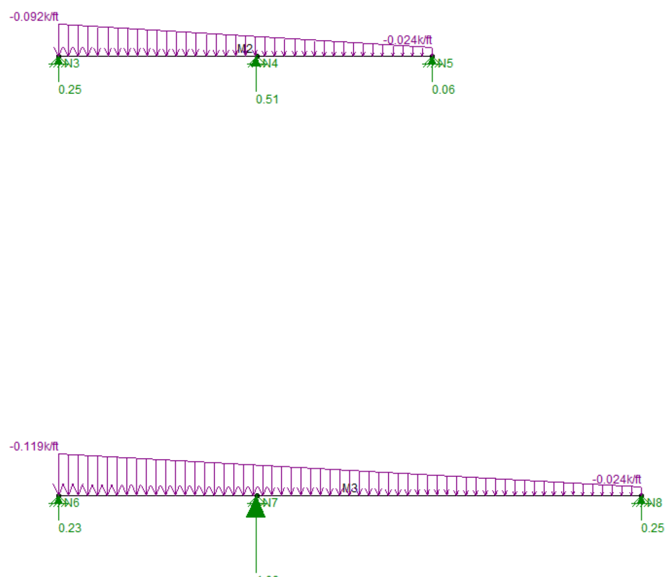
DESIGN OF HSS SPANNING ACROSS RECCESS ENTRY

w_a @ HSS





w_u @ HSS



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CANOPY DESIGN (EAST ELEVATION):

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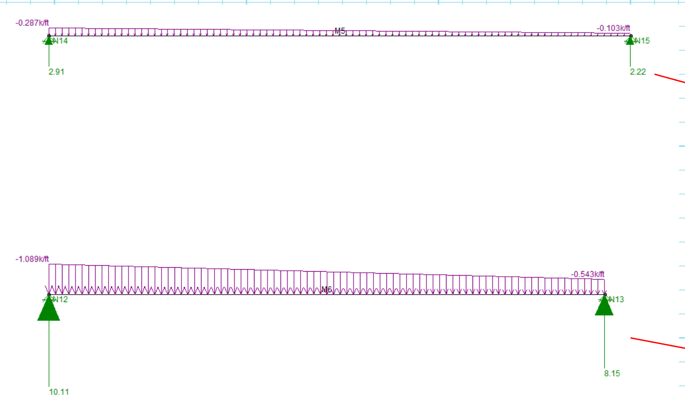
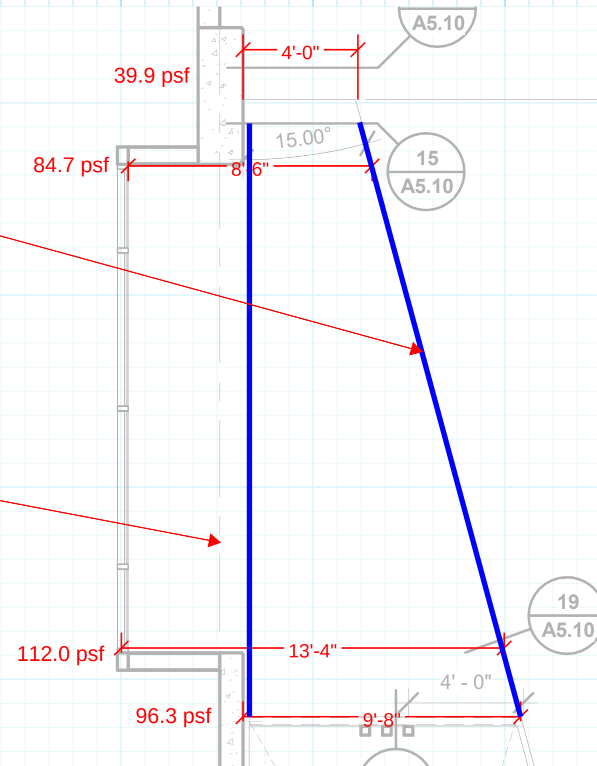


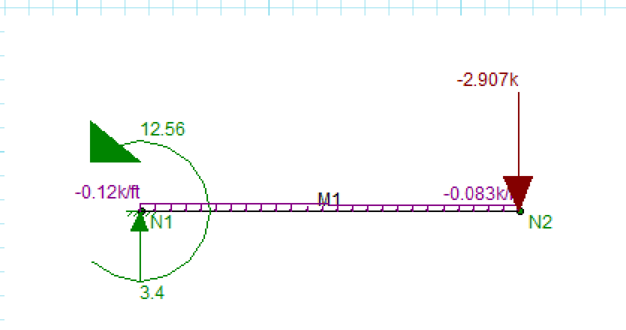


HSS10x5x5/16 PASSES BOTH STRENGTH AND DEFLECTION CHECKS

CONN DESIGN:
Vu = 10.1 k

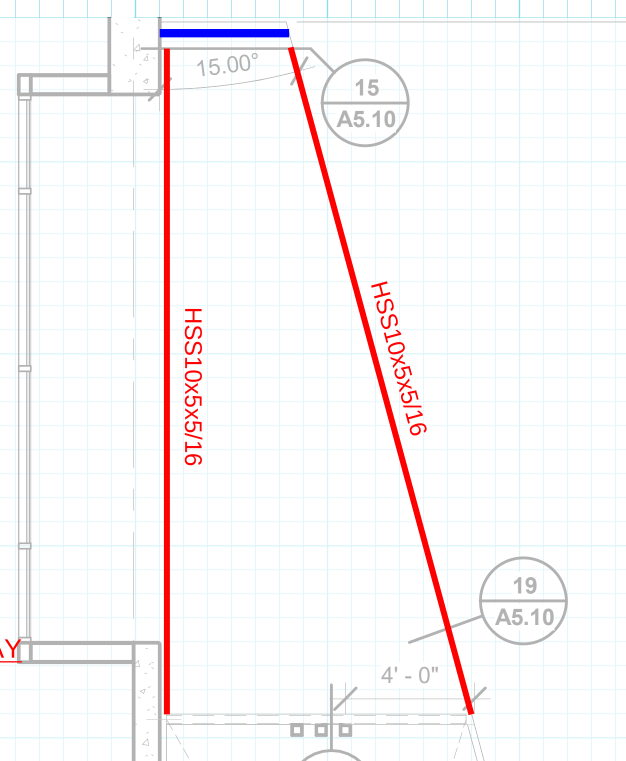
phiRn = 2 x 1.392(4)(5 in)(1.5) = 83.5 k

CANTILEVERING HSS DESIGN (4'-0" PROJECTION)



HSS5x5x1/4 PASSES BOTH STRENGTH AND DEFLECTION CHECKS

CONN DESIGN:
Vu = 3.4 k
Mu = 12.6 k-ft
C = T = 12.6 k-ft / 5 in = 30.2 k
phiRn = 2 x 1.392(4)(3 in)(1.5) = 50.1 k
phiTn = 0.9(36 ksi)(3 in)(0.375 in) = 36.5 k → OKAY



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Job# 2220290.00
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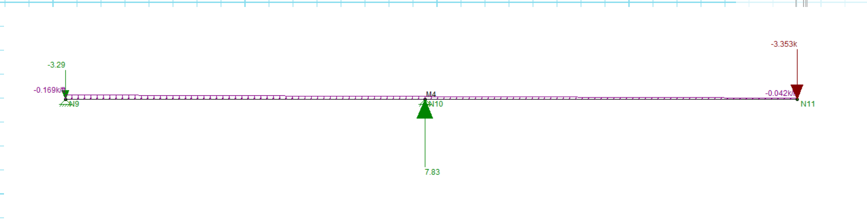
CANOPY DESIGN (EAST ELEVATION):

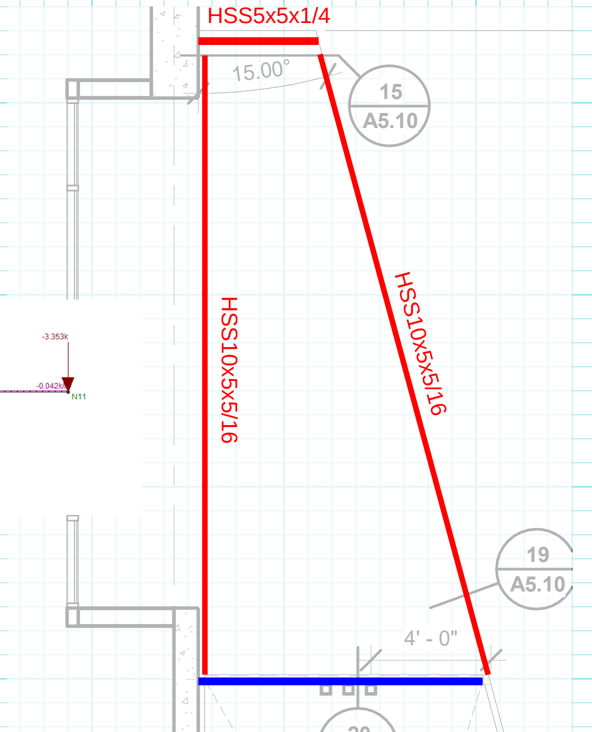
HSS WITH COL AT MID SPAN

LOAD FROM OTHER SIDE OF STEEL (FROM CHANNEL AND HSS):

DL = 4 PSF x 200 ft^2 / 31.75 ft + 34 plf = 59.2 plf

VDL = 59.2 plf * 31.75 ft / 2 = 940 lb





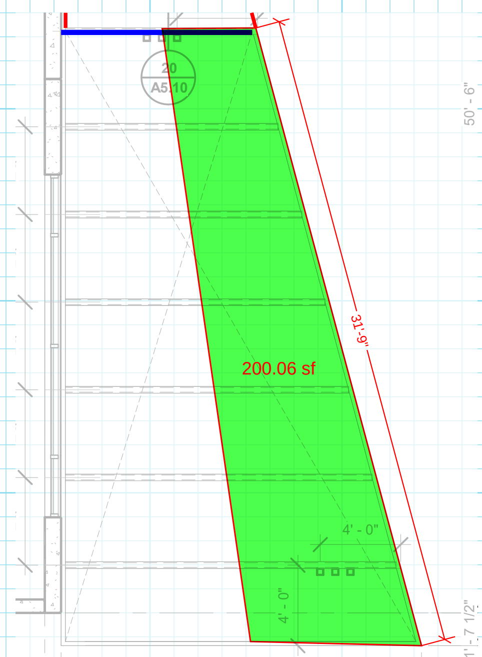
HSS5x5x1/4 PASSES BOTH STRENGTH AND DEFLECTION CHECKS

CONN DESIGN:
Vu = 3.3 k
phiRn = 2 x 1.392(4)(3 in) = 33.4 k

HSS COLUMN DESIGN:

Pu = 7.83 k
H = 17 ft

phiPn = 85.5 k → OKAY



FDN DESIGN:
Pa = 5.65 k

q = 2.5 ksf

Areqd = 2.26 ft^2 => 1.75 ft x 1.75 ft = 3.0625 ft^2

CANOPY DESIGN (EAST ELEVATION):

Lateral Load Path:
The HSS on the face of the panel will directly transfer shear into the panels. The force from the portion of the canopy without a diaphragm will be dragged into the metal diaphragm through the channel element. Since the direct shear only occurs along the HSS at the face, the eccentricity in the diaphragm support will be resolved in a "racking" mechanism at the HSS outriggers on the ends of the metal deck.

LATERAL LOAD TRANSFER:

Wdrag = 200 ft^2 x 4 psf + 10 ft x 18 ft x 12 psf / 2 + 34 plf x 31.75 ft = 2209 lb
Fp,drag = (0.674)(2290 lb) = 1490 lb

OmegaFp,drag = (2.5)(1490 lb) = 3725 lb of drag load

C15x33.9 can act as drag element. Has enough capacity

Wcanopy = 145 ft^2 x 7.5 psf + 2 x 29 plf x 22.5 ft + 34 plf X 22.5 ft= 3160 lb
Fp,canopy = 0.674 x 3160 lb = 2130 lb

M = 2130 lb / 2 x 7 ft = 7500 lb
C = T = 7500 lb / 21.5 ft = 350 lb

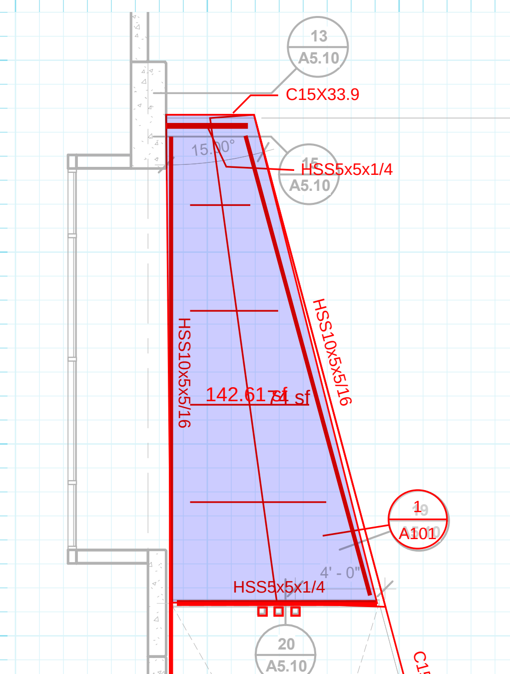
design embeds at ends including axial from "racking"

Vtot = 0.674(220 ft^2 x 7.5 psf + 29 plf x 53 ft + 29 plf x 22.5 ft + 34 plf x 22.5 ft) + 1490 lb = 4.6 k

phi*Rn = 6 x 1.392 x 4 x 5 = 167 k

total shear through the continuous HSS along the face of the panel. resolved via horiz weld along embed

Fw = (18.7 psf + 15.3 psf)x(10 ft x 18 ft) / 2 = 3060 lb -> Seismic governs



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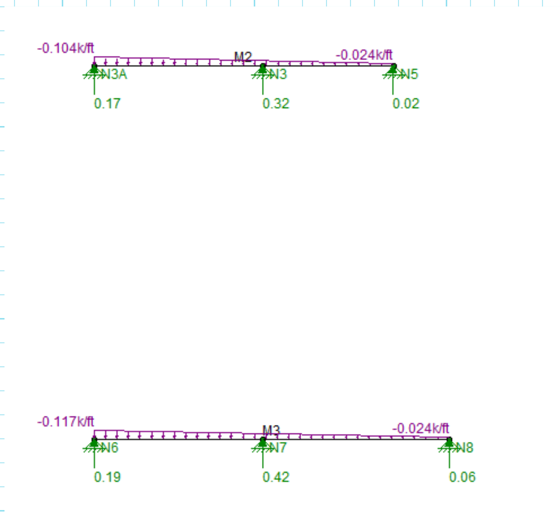
By SHA
Date 6/15/2023
Job# 2220290.00
Sht. 1 of XX

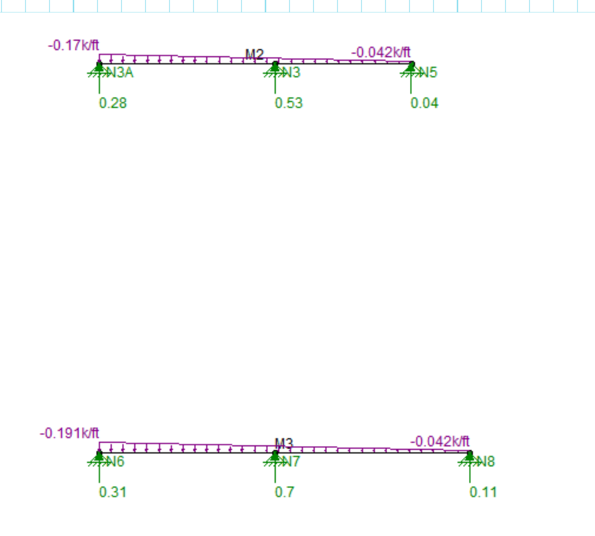
CANOPY DESIGN WEST ELEVATION:

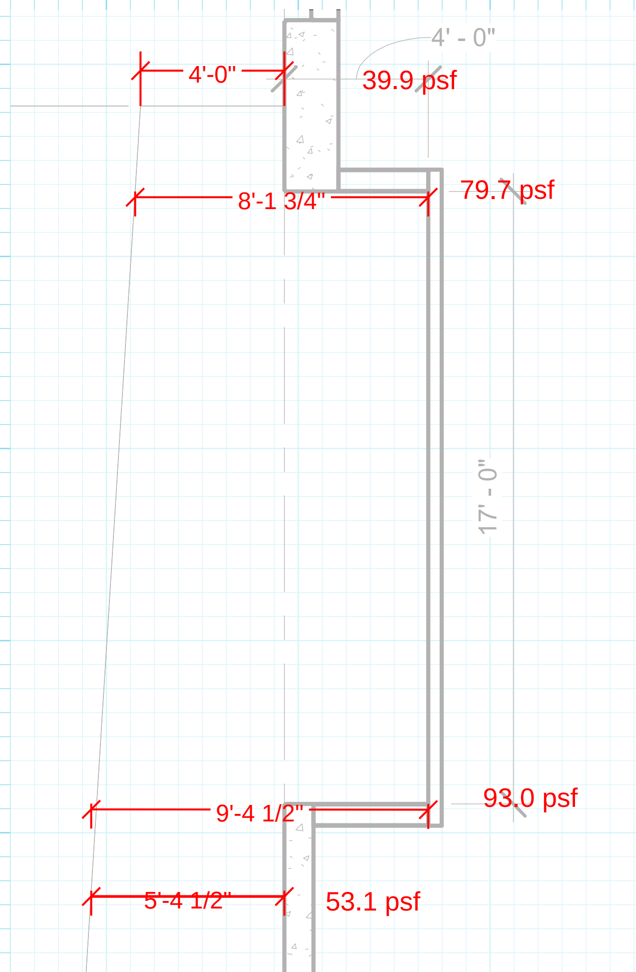
DESIGN OF HSS SPANNING ACROSS RECCESS ENTRY

w_a @ HSS



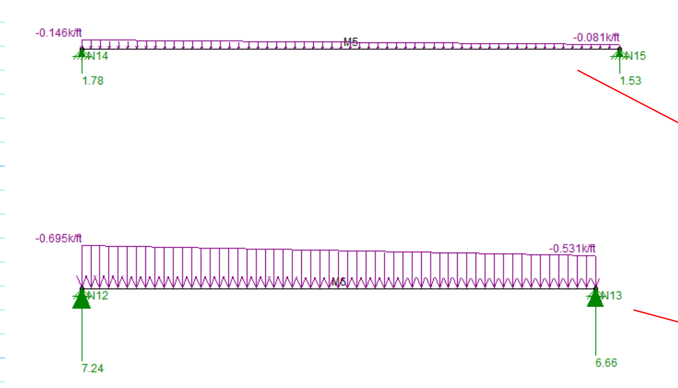
w_u @ HSS

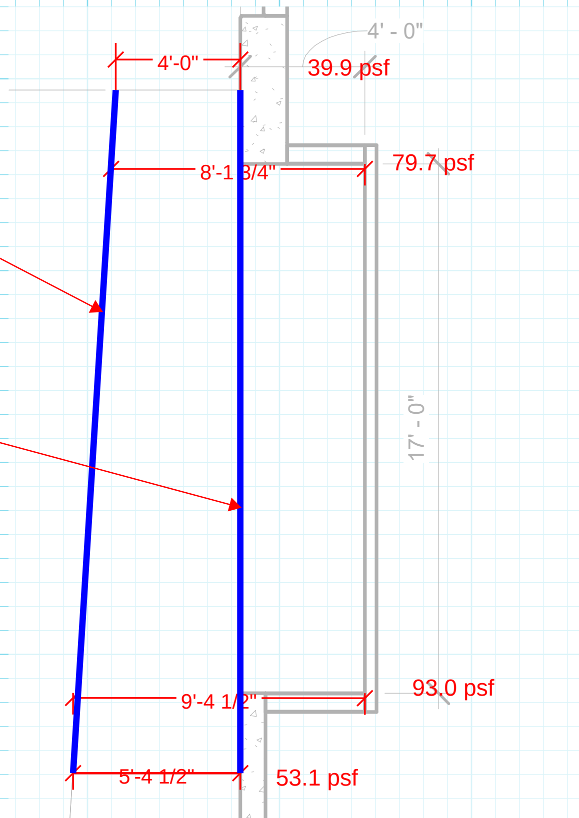




CANOPY DESIGN:

DESIGN OF HSS SPANNING ACROSS
RECCESS ENTRY





HSS10x5x5/16 PASSES BOTH STRENGTH AND
DEFLECTION CHECKS

CONN DESIGN:
Vu = 7.24 k

phiRn = 2 x 1.392(4)(5 in)(1.5) = 83.5 k

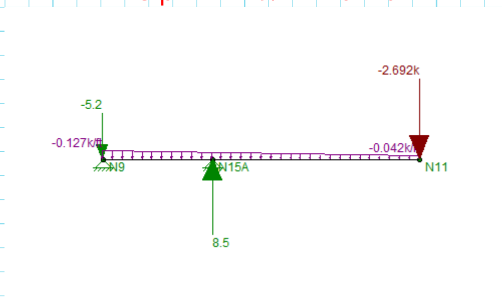
CANOPY DESIGN:

CANTILEVERING HSS DESIGN (5'-6" PROJECTION)

LOAD FROM OTHER SIDE OF STEEL (FROM CHANNEL AND HSS):

DL = 4 PSF x 70 ft^2 / 21 ft + 34 plf = 47.3 plf

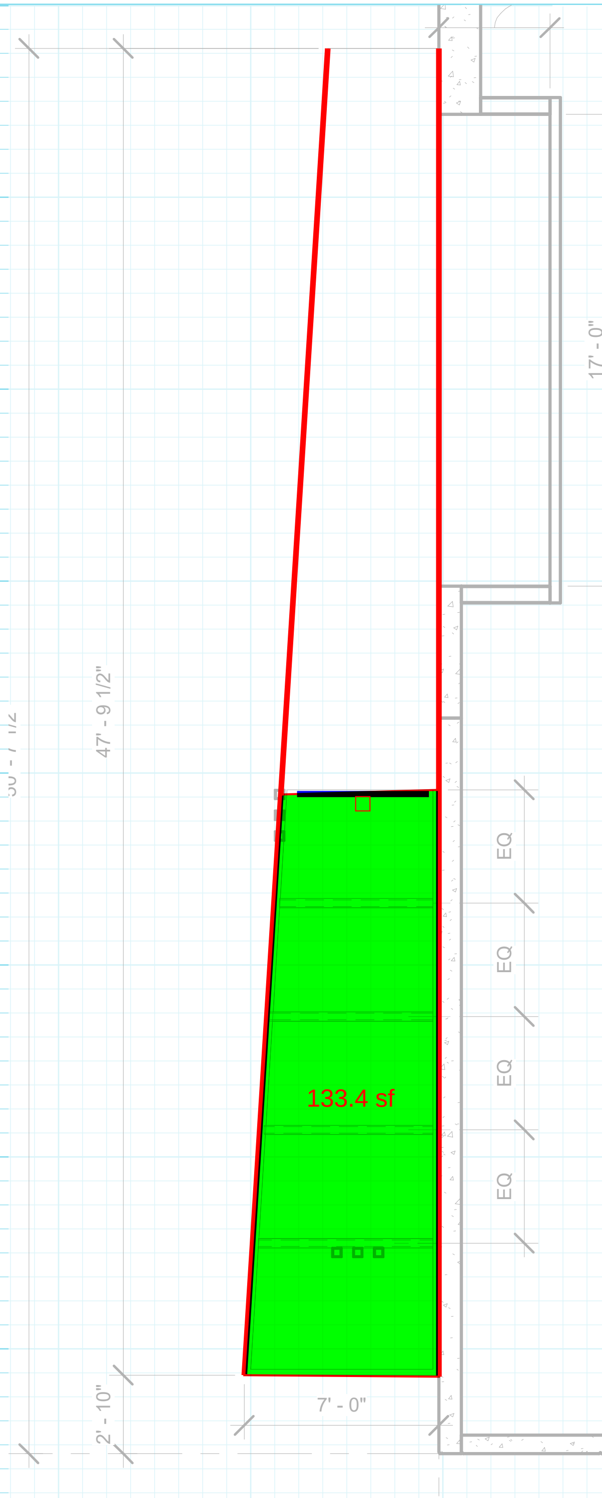
VDL = 47.3 plf * 21 ft / 2 =497 lb



HSS5x5x1/4 PASSES BOTH STRENGTH AND DEFLECTION CHECKS

CONN DESIGN:
Vu = 5.2 k

phi*Rn = 2*1.392*(4)(3 in) = 33.4 k → OKAY



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By SHA
Date 6/15/2023
Job# 2220290.00
Sht. 1 of XX

Snow Drift Geometry and Loading

Purpose Statement: Snow drift calculations for Lower Roofs, Adjacent Structures, Rooftop Projections and Parapets. Calculates snow drift only. Ground snow load and minimum roof snow load to be determined outside of this program.

Referenced Standards: 2021 IBC
ASCE 7-16

Basic Input

Basic ground snow load for use with snow drift calculations

$p_g := 20\ psf$ *Note that this is not the same as the minimum roof snow load. The minimum roof snow load is calculated outside of this program*

Snow exposure factor as defined in *ASCE 7-16, Table 7.3-1*

$Surface\_Roughness\_Category :=$ B Terrain (exposure) category as defined in *ASCE 7-16, Ch. 26.7*

$Exposure\_of\_Roof :=$ Partially Exposed

Protected Area

$C_e = 1$

Thermal factor as defined in *ASCE 7-16, Table 7.3-2*

$C_t :=$ Unheated and open air structures $\quad C_t = 1.2$

Importance factor *ASCE 7-16, Table 1.5-2*

$I_s :=$ Occupancy category II $\quad I_s = 1$

Sloped roof factor (If applicable) *ASCE 7-16, Figure 7.4-1*

$C_s := 1.0$

Calculated flat roof snow load *(ASCE 7-16, Eq. 7.3-1)*

$p_f := 0.7 \cdot C_e \cdot C_t \cdot I_s \cdot p_g \qquad p_f = 16.8\ psf$

Balanced roof snow load *(ASCE 7-16, Eq. 7.4-1)*

$p_s := C_s \cdot p_f \qquad p_s = 16.8\ psf$

Density of snow *(ASCE 7-16, Eq. 7.7-1)*

$$\gamma := min\left(30 \cdot pcf, \frac{0.13}{ft} \cdot p_g + 14.0 \cdot pcf\right) \qquad \gamma = 16.6\ pcf$$

Height of calculated minimum snow load *ASCE 7-16, 7.7.1*

$$h_b := \frac{p_s}{\gamma} \qquad h_b = 1.01\ ft$$

Snow Drift Geometry and Loading

Canopy on the East Side (9'-8" Projection)

CASE I: DRIFT LOAD FOR LOWER ROOF

(ASCE 7-16 Section 7.7.1 and Figure 7.7-2)

$l_u := 550\ ft$ — Horizontal length of upper roof

$l_l := 9.67\ ft$ — Horizontal length of lower roof

$h_r := 28.5\ ft$ — Difference in height between upper and lower roofs

Adjusted difference in roof heights (Removes height of flat roof snow load)

$h_c := h_r - h_b$ $h_c = 27.49\ ft$

$Check_{7.7.1a} := \text{if}\left(\frac{h_c}{h_b} < 0.2, \text{"Drift does not apply"}, \text{"Drift applies"}\right) = \text{"Drift applies"}$

Minimum length of roof upwind

$l_u := \text{if}\left(l_u \ge 20 \cdot ft, l_u, 20\ ft\right)$ $l_u = 550\ ft$

Leeward drift height

$h_{dl} := min\left(\sqrt{I_s} \cdot \left(0.43 \cdot \sqrt[3]{\frac{l_u}{ft}} \cdot \sqrt[4]{\frac{p_g}{psf} + 10} - 1.5\right) \cdot ft, 0.6 \cdot l_l\right)$ (ASCE 7-16 Fig. 7.6-1) $h_{dl} = 5.8\ ft$

Windward drift height

$h_{dw} := \frac{3}{4} \cdot \sqrt{I_s} \cdot \left(0.43 \cdot \sqrt[3]{\frac{l_l}{ft}} \cdot \sqrt[4]{\frac{p_g}{psf} + 10} - 1.5\right) \cdot ft$ (ASCE 7-16 7.7.1) $h_{dw} = 0.48\ ft$

Controlling drift height

$h'_d := \max\left(h_{dl}, h_{dw}\right)$ $h'_d = 5.80\ ft$

Check drift height against hc

$h_d := \text{if}\left(h'_d \le h_c, h'_d, h_c\right)$ **Drift height** $h_d = 5.80\ ft$

Calculate drift width

$w_1 := \text{if}\left(h_d \le h_c, 4 \cdot h_d, 4 \cdot \frac{h_d^{\,2}}{h_c}\right)$ $w_1 = 23.21\ ft$

$w := min\left(w_1, 8 \cdot h_c, l_l\right)$ **Drift width** $w = 9.67\ ft$

Snow Drift Geometry and Loading

Maximum intensity of snow load

$p_m := (h_d) \cdot \gamma + p_f$ $\qquad p_m = 113.11\ psf$

$p_d := p_m - p_f$ $\qquad$ **Maximum drift surcharge load** $\qquad p_d = 96.31\ psf$

Snow Drift Geometry and Loading

Canopy on the East Side (4'-0" Projection)

CASE I: DRIFT LOAD FOR LOWER ROOF

(ASCE 7-16 Section 7.7.1 and Figure 7.7-2)

$l_u := 550\ ft$ Horizontal length of upper roof

$l_l := 4\ ft$ Horizontal length of lower roof

$h_r := 28.5\ ft$ Difference in height between upper and lower roofs

Adjusted difference in roof heights (Removes height of flat roof snow load)

$h_c := h_r - h_b$ $h_c = 27.49\ ft$

$Check_{7.7.1a} := \text{if}\left(\frac{h_c}{h_b} < 0.2, \text{“Drift does not apply”}, \text{“Drift applies”}\right) = \text{“Drift applies”}$

Minimum length of roof upwind

$l_u := \text{if}\left(l_u \geq 20 \cdot ft, l_u, 20\ ft\right)$ $l_u = 550\ ft$

Leeward drift height

$h_{dl} := min\left(\sqrt{I_s} \cdot \left(0.43 \cdot \sqrt[3]{\frac{l_u}{ft}} \cdot \sqrt[4]{\frac{p_g}{psf} + 10} - 1.5\right) \cdot ft, 0.6 \cdot l_l\right)$ *(ASCE 7-16 Fig. 7.6-1)* $h_{dl} = 2.4\ ft$

Windward drift height

$h_{dw} := \frac{3}{4} \cdot \sqrt{I_s} \cdot \left(0.43 \cdot \sqrt[3]{\frac{l_l}{ft}} \cdot \sqrt[4]{\frac{p_g}{psf} + 10} - 1.5\right) \cdot ft$ *(ASCE 7-16 7.7.1)* $h_{dw} = 0.07\ ft$

Controlling drift height

$h'_d := \max\left(h_{dl}, h_{dw}\right)$ $h'_d = 2.40\ ft$

Check drift height against hc

$h_d := \text{if}\left(h'_d \leq h_c, h'_d, h_c\right)$ **Drift height** $h_d = 2.40\ ft$

Calculate drift width

$w_1 := \text{if}\left(h_d \leq h_c, 4 \cdot h_d, 4 \cdot \frac{{h_d}^2}{h_c}\right)$ $w_1 = 9.60\ ft$

$w := min\left(w_1, 8 \cdot h_c, l_l\right)$ **Drift width** $w = 4.00\ ft$

Snow Drift Geometry and Loading

Maximum intensity of snow load

$p_m := (h_d) \cdot \gamma + p_f$ $p_m = 56.64\ psf$

$p_d := p_m - p_f$ **Maximum drift surcharge load** $p_d = 39.84\ psf$

Snow Drift Geometry and Loading

Canopy on the East Side (13'-3" Projection)

CASE I: DRIFT LOAD FOR LOWER ROOF

(ASCE 7-16 Section 7.7.1 and Figure 7.7-2)

$l_u := 550\ ft$ Horizontal length of upper roof

$l_l := 13.25\ ft$ Horizontal length of lower roof

$h_r := 28.5\ ft$ Difference in height between upper and lower roofs

Adjusted difference in roof heights (Removes height of flat roof snow load)

$h_c := h_r - h_b$ $h_c = 27.49\ ft$

$Check_{7.7.1a} := \text{if}\left(\frac{h_c}{h_b} < 0.2, \text{“Drift does not apply”}, \text{“Drift applies”}\right) = \text{“Drift applies”}$

Minimum length of roof upwind

$l_u := \text{if}\left(l_u \ge 20 \cdot ft, l_u, 20\ ft\right)$ $l_u = 550\ ft$

Leeward drift height

$h_{dl} := min\left(\sqrt{I_s} \cdot \left(0.43 \cdot \sqrt[3]{\frac{l_u}{ft}} \cdot \sqrt[4]{\frac{p_g}{psf} + 10} - 1.5\right) \cdot ft, 0.6 \cdot l_l\right)$ *(ASCE 7-16 Fig. 7.6-1)* $h_{dl} = 6.75\ ft$

Windward drift height

$h_{dw} := \frac{3}{4} \cdot \sqrt{I_s} \cdot \left(0.43 \cdot \sqrt[3]{\frac{l_l}{ft}} \cdot \sqrt[4]{\frac{p_g}{psf} + 10} - 1.5\right) \cdot ft$ *(ASCE 7-16 7.7.1)* $h_{dw} = 0.66\ ft$

Controlling drift height

$h'_d := \max\left(h_{dl}, h_{dw}\right)$ $h'_d = 6.75\ ft$

Check drift height against hc

$h_d := \text{if}\left(h'_d \le h_c, h'_d, h_c\right)$ **Drift height** $h_d = 6.75\ ft$

Calculate drift width

$w_1 := \text{if}\left(h_d \le h_c, 4 \cdot h_d, 4 \cdot \frac{{h_d}^2}{h_c}\right)$ $w_1 = 26.98\ ft$

$w := min\left(w_1, 8 \cdot h_c, l_l\right)$ **Drift width** $w = 13.25\ ft$

Snow Drift Geometry and Loading

Maximum intensity of snow load

$p_m := (h_d) \cdot \gamma + p_f$ $\qquad p_m = 128.77\ psf$

$p_d := p_m - p_f$ **Maximum drift surcharge load** $p_d = 111.97\ psf$

Snow Drift Geometry and Loading

Canopy on the East Side (8'-6" Projection)

CASE I: DRIFT LOAD FOR LOWER ROOF

(ASCE 7-16 Section 7.7.1 and Figure 7.7-2)

$l_u := 550\ ft$ — Horizontal length of upper roof

$l_l := 8.5\ ft$ — Horizontal length of lower roof

$h_r := 28.5\ ft$ — Difference in height between upper and lower roofs

Adjusted difference in roof heights (Removes height of flat roof snow load)

$h_c := h_r - h_b$ $\qquad h_c = 27.49\ ft$

$$Check_{7.7.1a} := \text{if}\left(\frac{h_c}{h_b} < 0.2, \text{“Drift does not apply”}, \text{“Drift applies”}\right) = \text{“Drift applies”}$$

Minimum length of roof upwind

$l_u := \text{if}\left(l_u \geq 20 \cdot ft, l_u, 20\ ft\right)$ $\qquad l_u = 550\ ft$

Leeward drift height

$$h_{dl} := min\left(\sqrt{I_s} \cdot \left(0.43 \cdot \sqrt[3]{\frac{l_u}{ft}} \cdot \sqrt[4]{\frac{p_g}{psf} + 10} - 1.5\right) \cdot ft, 0.6 \cdot l_l\right)$$ *(ASCE 7-16 Fig. 7.6-1)*

$h_{dl} = 5.1\ ft$

Windward drift height

$$h_{dw} := \frac{3}{4} \cdot \sqrt{I_s} \cdot \left(0.43 \cdot \sqrt[3]{\frac{l_l}{ft}} \cdot \sqrt[4]{\frac{p_g}{psf} + 10} - 1.5\right) \cdot ft$$ *(ASCE 7-16 7.7.1)* $\qquad h_{dw} = 0.42\ ft$

Controlling drift height

$h'_d := \max\left(h_{dl}, h_{dw}\right)$ $\qquad h'_d = 5.10\ ft$

Check drift height against hc

$h_d := \text{if}\left(h'_d \leq h_c, h'_d, h_c\right)$ **Drift height** $h_d = 5.10\ ft$

Calculate drift width

$$w_1 := \text{if}\left(h_d \leq h_c, 4 \cdot h_d, 4 \cdot \frac{h_d^{\ 2}}{h_c}\right)$$ $\qquad w_1 = 20.40\ ft$

$w := min\left(w_1, 8 \cdot h_c, l_l\right)$ **Drift width** $w = 8.50\ ft$

Snow Drift Geometry and Loading

Maximum intensity of snow load

$p_m := (h_d) \cdot \gamma + p_f$ $\quad p_m = 101.46\ psf$

$p_d := p_m - p_f$ **Maximum drift surcharge load** $p_d = 84.66\ psf$

Snow Drift Geometry and Loading

Canopy on the West Side (5'-4" Projection)

CASE I: DRIFT LOAD FOR LOWER ROOF

(ASCE 7-16 Section 7.7.1 and Figure 7.7-2)

$l_u := 550\ ft$ Horizontal length of upper roof

$l_l := 5.33\ ft$ Horizontal length of lower roof

$h_r := 28.5\ ft$ Difference in height between upper and lower roofs

Adjusted difference in roof heights (Removes height of flat roof snow load)

$h_c := h_r - h_b$ $\qquad h_c = 27.49\ ft$

$Check_{7.7.1a} := \text{if}\left(\frac{h_c}{h_b} < 0.2, \text{“Drift does not apply”}, \text{“Drift applies”}\right) = \text{“Drift applies”}$

Minimum length of roof upwind

$l_u := \text{if}\left(l_u \ge 20 \cdot ft, l_u, 20\ ft\right)$ $\qquad l_u = 550\ ft$

Leeward drift height

$h_{dl} := min\left(\sqrt{I_s} \cdot \left(0.43 \cdot \sqrt[3]{\frac{l_u}{ft}} \cdot \sqrt[4]{\frac{p_g}{psf} + 10} - 1.5\right) \cdot ft, 0.6 \cdot l_l\right)$ *(ASCE 7-16 Fig. 7.6-1)* $\qquad h_{dl} = 3.2\ ft$

Windward drift height

$h_{dw} := \frac{3}{4} \cdot \sqrt{I_s} \cdot \left(0.43 \cdot \sqrt[3]{\frac{l_l}{ft}} \cdot \sqrt[4]{\frac{p_g}{psf} + 10} - 1.5\right) \cdot ft$ *(ASCE 7-16 7.7.1)* $\qquad h_{dw} = 0.19\ ft$

Controlling drift height

$h'_d := \max\left(h_{dl}, h_{dw}\right)$ $\qquad h'_d = 3.20\ ft$

Check drift height against hc

$h_d := \text{if}\left(h'_d \le h_c, h'_d, h_c\right)$ **Drift height** $h_d = 3.20\ ft$

Calculate drift width

$w_1 := \text{if}\left(h_d \le h_c, 4 \cdot h_d, 4 \cdot \frac{{h_d}^2}{h_c}\right)$ $\qquad w_1 = 12.79\ ft$

$w := min\left(w_1, 8 \cdot h_c, l_l\right)$ **Drift width** $w = 5.33\ ft$

Snow Drift Geometry and Loading

Maximum intensity of snow load

$p_m := (h_d) \cdot \gamma + p_f$

$p_m = 69.89\ psf$

$p_d := p_m - p_f$

Maximum drift surcharge load $p_d = 53.09\ psf$

Snow Drift Geometry and Loading

Canopy on the West Side (9'-4" Projection)

CASE I: DRIFT LOAD FOR LOWER ROOF

(ASCE 7-16 Section 7.7.1 and Figure 7.7-2)

$l_u := 550\ ft$ Horizontal length of upper roof

$l_l := 9.33\ ft$ Horizontal length of lower roof

$h_r := 28.5\ ft$ Difference in height between upper and lower roofs

Adjusted difference in roof heights (Removes height of flat roof snow load)

$h_c := h_r - h_b$ $h_c = 27.49\ ft$

$Check_{7.7.1a} := \text{if}\left(\frac{h_c}{h_b} < 0.2, \text{“Drift does not apply”}, \text{“Drift applies”}\right) = \text{“Drift applies”}$

Minimum length of roof upwind

$l_u := \text{if}\left(l_u \ge 20 \cdot ft, l_u, 20\ ft\right)$ $l_u = 550\ ft$

Leeward drift height

$h_{dl} := min\left(\sqrt{I_s} \cdot \left(0.43 \cdot \sqrt[3]{\frac{l_u}{ft}} \cdot \sqrt[4]{\frac{p_g}{psf} + 10} - 1.5\right) \cdot ft, 0.6 \cdot l_l\right)$ *(ASCE 7-16 Fig. 7.6-1)* $h_{dl} = 5.6\ ft$

Windward drift height

$h_{dw} := \frac{3}{4} \cdot \sqrt{I_s} \cdot \left(0.43 \cdot \sqrt[3]{\frac{l_l}{ft}} \cdot \sqrt[4]{\frac{p_g}{psf} + 10} - 1.5\right) \cdot ft$ *(ASCE 7-16 7.7.1)* $h_{dw} = 0.46\ ft$

Controlling drift height

$h'_d := \max\left(h_{dl}, h_{dw}\right)$ $h'_d = 5.60\ ft$

Check drift height against hc

$h_d := \text{if}\left(h'_d \le h_c, h'_d, h_c\right)$ **Drift height** $h_d = 5.60\ ft$

Calculate drift width

$w_1 := \text{if}\left(h_d \le h_c, 4 \cdot h_d, 4 \cdot \frac{{h_d}^2}{h_c}\right)$ $w_1 = 22.39\ ft$

$w := min\left(w_1, 8 \cdot h_c, l_l\right)$ **Drift width** $w = 9.33\ ft$

Snow Drift Geometry and Loading

Maximum intensity of snow load

$p_m := (h_d) \cdot \gamma + p_f$ $p_m = 109.73\ psf$

$p_d := p_m - p_f$ **Maximum drift surcharge load** $p_d = 92.93\ psf$

Snow Drift Geometry and Loading

Canopy on the West Side (8'-0" Projection)

CASE I: DRIFT LOAD FOR LOWER ROOF

(ASCE 7-16 Section 7.7.1 and Figure 7.7-2)

$l_u := 550\ ft$ Horizontal length of upper roof

$l_l := 8.0\ ft$ Horizontal length of lower roof

$h_r := 28.5\ ft$ Difference in height between upper and lower roofs

Adjusted difference in roof heights (Removes height of flat roof snow load)

$h_c := h_r - h_b$ $h_c = 27.49\ ft$

$Check_{7.7.1a} := \text{if}\left(\frac{h_c}{h_b} < 0.2, \text{“Drift does not apply”}, \text{“Drift applies”}\right) = \text{“Drift applies”}$

Minimum length of roof upwind

$l_u := \text{if}\left(l_u \ge 20 \cdot ft, l_u, 20\ ft\right)$ $l_u = 550\ ft$

Leeward drift height

$h_{dl} := min\left(\sqrt{I_s} \cdot \left(0.43 \cdot \sqrt[3]{\frac{l_u}{ft}} \cdot \sqrt[4]{\frac{p_g}{psf} + 10} - 1.5\right) \cdot ft, 0.6 \cdot l_l\right)$ (ASCE 7-16 Fig. 7.6-1) $h_{dl} = 4.8\ ft$

Windward drift height

$h_{dw} := \frac{3}{4} \cdot \sqrt{I_s} \cdot \left(0.43 \cdot \sqrt[3]{\frac{l_l}{ft}} \cdot \sqrt[4]{\frac{p_g}{psf} + 10} - 1.5\right) \cdot ft$ (ASCE 7-16 7.7.1) $h_{dw} = 0.38\ ft$

Controlling drift height

$h'_d := \max\left(h_{dl}, h_{dw}\right)$ $h'_d = 4.80\ ft$

Check drift height against hc

$h_d := \text{if}\left(h'_d \le h_c, h'_d, h_c\right)$ **Drift height** $h_d = 4.80\ ft$

Calculate drift width

$w_1 := \text{if}\left(h_d \le h_c, 4 \cdot h_d, 4 \cdot \frac{h_d^{\,2}}{h_c}\right)$ $w_1 = 19.20\ ft$

$w := min\left(w_1, 8 \cdot h_c, l_l\right)$ **Drift width** $w = 8.00\ ft$

Snow Drift Geometry and Loading

Maximum intensity of snow load

$p_m := (h_d) \cdot \gamma + p_f$ $\qquad p_m = 96.48\ psf$

$p_d := p_m - p_f$ $\qquad$ **Maximum drift surcharge load** $\qquad p_d = 79.68\ psf$

Snow Drift Geometry and Loading

Canopy on the North Side (3'-9" Projection)

CASE I: DRIFT LOAD FOR LOWER ROOF

(ASCE 7-16 Section 7.7.1 and Figure 7.7-2)

$l_u := 240 \cdot ft$ Horizontal length of upper roof

$l_l := 3.75 \cdot ft$ Horizontal length of lower roof

$h_r := 28.5 \cdot ft$ Difference in height between upper and lower roofs

Adjusted difference in roof heights (Removes height of flat roof snow load)

$h_c := h_r - h_b$ $h_c = 27.49\ ft$

$Check_{7.7.1a} := \text{if}\left(\frac{h_c}{h_b} < 0.2, \text{“Drift does not apply”}, \text{“Drift applies”}\right) = \text{“Drift applies”}$

Minimum length of roof upwind

$l_u := \text{if}\left(l_u \geq 20 \cdot ft, l_u, 20\ ft\right)$ $l_u = 240\ ft$

Leeward drift height

$h_{dl} := min\left(\sqrt{I_s} \cdot \left(0.43 \cdot \sqrt[3]{\frac{l_u}{ft}} \cdot \sqrt[4]{\frac{p_g}{psf} + 10} - 1.5\right) \cdot ft, 0.6 \cdot l_l\right)$ *(ASCE 7-16 Fig. 7.6-1)* $h_{dl} = 2.25\ ft$

Windward drift height

$h_{dw} := \frac{3}{4} \cdot \sqrt{I_s} \cdot \left(0.43 \cdot \sqrt[3]{\frac{l_l}{ft}} \cdot \sqrt[4]{\frac{p_g}{psf} + 10} - 1.5\right) \cdot ft$ *(ASCE 7-16 7.7.1)* $h_{dw} = 0.05\ ft$

Controlling drift height

$h'_d := \max\left(h_{dl}, h_{dw}\right)$ $h'_d = 2.25\ ft$

Check drift height against hc

$h_d := \text{if}\left(h'_d \leq h_c, h'_d, h_c\right)$ **Drift height** $h_d = 2.25\ ft$

Calculate drift width

$w_1 := \text{if}\left(h_d \leq h_c, 4 \cdot h_d, 4 \cdot \frac{{h_d}^2}{h_c}\right)$ $w_1 = 9.00\ ft$

$w := min\left(w_1, 8 \cdot h_c, l_l\right)$ **Drift width** $w = 3.75\ ft$

Snow Drift Geometry and Loading

Maximum intensity of snow load

$p_m := (h_d) \cdot \gamma + p_f$ $\qquad p_m = 54.15\ psf$

$p_d := p_m - p_f$ $\qquad$ **Maximum drift surcharge load** $\qquad p_d = 37.35\ psf$

Company:		Date:	6/15/2023
Engineer:		Page:	1/5
Project:			
Address:			
Phone:			
E-mail:			

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 318-14
Units: Imperial units

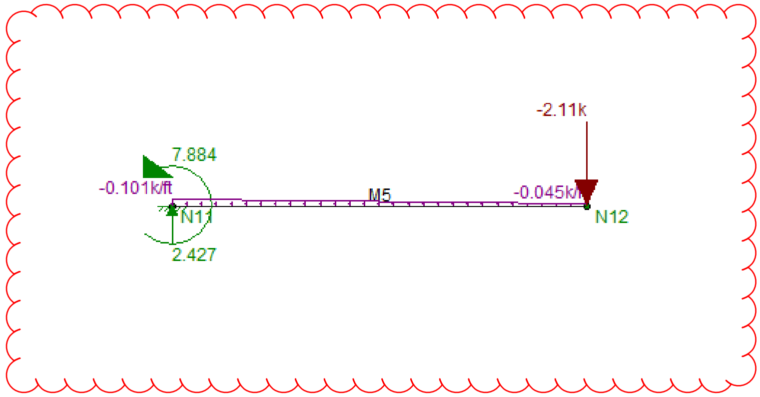
Anchor Information:
Anchor type: Cast-in-place
Material: AWS Type A
Diameter (inch): 0.625
Effective Embedment depth, h_{ef} (inch): 6.000
Anchor category: -
Anchor ductility: Yes
h_{min} (inch): 7.38
C_{min} (inch): 1.38
S_{min} (inch): 2.50

Base Material
Concrete: Normal-weight
Concrete thickness, h (inch): 9.50
State: Cracked
Compressive strength, f'_c (psi): 4000
$\Psi_{c,V}$: 1.0
Reinforcement condition: B tension, B shear
Supplemental reinforcement: Not applicable
Reinforcement provided at corners: No
Ignore concrete breakout in tension: No
Ignore concrete breakout in shear: No
Ignore 6do requirement: No
Build-up grout pad: No

Base Plate
Length x Width x Thickness (inch): 8.00 x 8.00 x 0.38

Recommended Anchor
Anchor Name: Headed Stud - 5/8"Ø AWS Type A Headed Stud

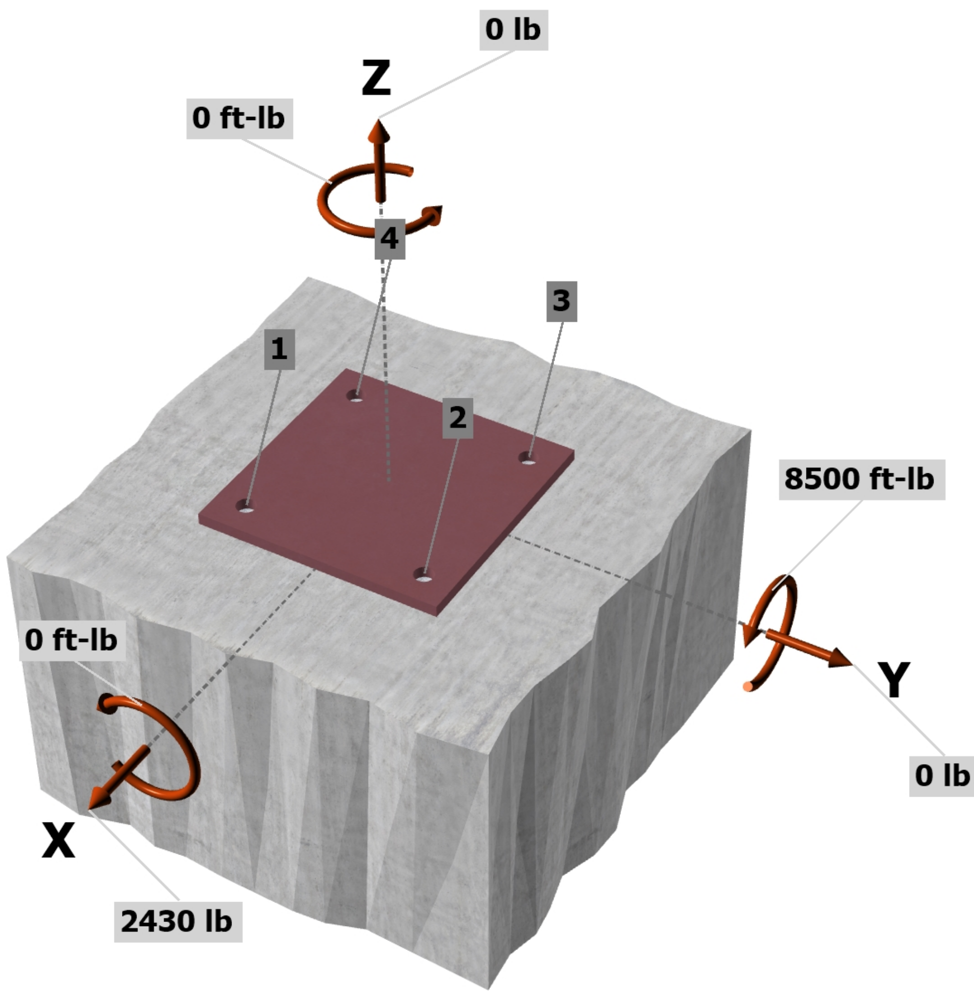
Load and Geometry
Load factor source: ACI 318 Section 5.3
Load combination: not set
Seismic design: No
Anchors subjected to sustained tension: Not applicable
Apply entire shear load at front row: No
Anchors only resisting wind and/or seismic loads: No



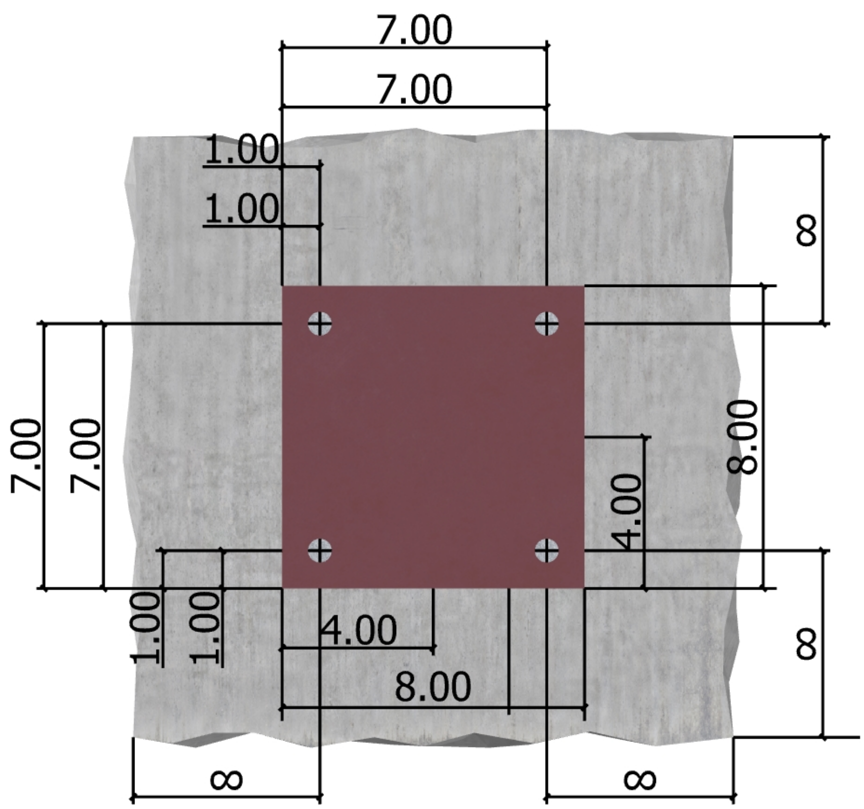
Strength level loads:

N_{ua} [lb]: 0
V_{uax} [lb]: 2430
V_{uay} [lb]: 0
M_{ux} [ft-lb]: 0
M_{uy} [ft-lb]: 8500 includes gravity loading w/ eccentricity (3")
M_{uz} [ft-lb]: 0

<Figure 1>



<Figure 2>

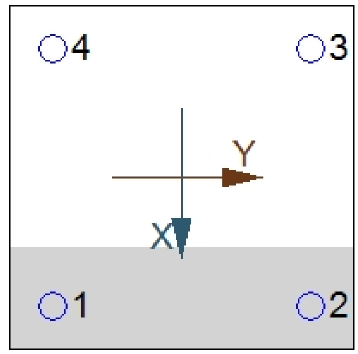


3. Resulting Anchor Forces

Anchor	Tension load, N_{ua} (lb)	Shear load x, V_{uax} (lb)	Shear load y, V_{uay} (lb)	Shear load combined, $\sqrt{(V_{uax})^2+(V_{uay})^2}$ (lb)
1	0.0	607.5	0.0	607.5
2	0.0	607.5	0.0	607.5
3	8219.6	607.5	0.0	607.5
4	8219.6	607.5	0.0	607.5
Sum	16439.2	2430.0	0.0	2430.0

Maximum concrete compression strain (‰): 0.40
Maximum concrete compression stress (psi): 1722
Resultant tension force (lb): 16439
Resultant compression force (lb): 16439
Eccentricity of resultant tension forces in x-axis, e'_{Nx} (inch): 0.00
Eccentricity of resultant tension forces in y-axis, e'_{Ny} (inch): 0.00
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. 17.4.1)

N_{sa} (lb)	ϕ	ϕN_{sa} (lb)
18715	0.75	14036

5. Concrete Breakout Strength of Anchor in Tension (Sec. 17.4.2)

$N_b = k_c \lambda_a \sqrt{f'_c} h_{ef}^{1.5}$ (Eq. 17.4.2.2a)

k_c	λ_a	f'_c (psi)	h_{ef} (in)	N_b (lb)
24.0	1.00	4000	6.000	22308

$\phi N_{cbg} = \phi\,(A_{Nc} / A_{Nco})\,\Psi_{ec,N}\,\Psi_{ed,N}\,\Psi_{c,N}\,\Psi_{cp,N} N_b$ (Sec. 17.3.1 & Eq. 17.4.2.1b)

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	ϕN_{cbg} (lb)
432.00	324.00	-	1.000	1.000	1.00	1.000	22308	0.70	20821

6. Pullout Strength of Anchor in Tension (Sec. 17.4.3)

$\phi N_{pn} = \phi \Psi_{c,P} N_p = \phi \Psi_{c,P} 8 A_{brg} f'_c$ (Sec. 17.3.1, Eq. 17.4.3.1 & 17.4.3.4)

$\Psi_{c,P}$	A_{brg} (in^2)	f'_c (psi)	ϕ	ϕN_{pn} (lb)
1.0	0.92	4000	0.70	20608

8. Steel Strength of Anchor in Shear (Sec. 17.5.1)

V_{sa} (lb)	ϕ_{grout}	ϕ	$\phi_{grout}\phi V_{sa}$ (lb)
18715	1.0	0.65	12165

10. Concrete Pryout Strength of Anchor in Shear (Sec. 17.5.3)

$\phi V_{cpg} = \phi k_{cp} N_{cbg} = \phi k_{cp}(A_{Nc} / A_{Nco})\Psi_{ec,N}\Psi_{ed,N}\Psi_{c,N}\Psi_{cp,N}N_b$ (Sec. 17.3.1 & Eq. 17.5.3.1b)

k_{cp}	A_{Nc} (in^2)	A_{Nco} (in^2)	$\Psi_{ec,N}$	$\Psi_{ed,N}$	$\Psi_{c,N}$	$\Psi_{cp,N}$	N_b (lb)	ϕ	ϕV_{cpg} (lb)
2.0	576.00	324.00	1.000	1.000	1.000	1.000	22308	0.70	55523

11. Results

Interaction of Tensile and Shear Forces (Sec. 17.6.)

Tension	Factored Load, N_{ua} (lb)	Design Strength, $øN_n$ (lb)	Ratio	Status
Steel	8220	14036	0.59	Pass
Concrete breakout	**16439**	**20821**	**0.79**	**Pass (Governs)**
Pullout	8220	20608	0.40	Pass

Shear	Factored Load, V_{ua} (lb)	Design Strength, $øV_n$ (lb)	Ratio	Status
Steel	**608**	**12165**	**0.05**	**Pass (Governs)**
Pryout	2430	55523	0.04	Pass

Interaction check	$N_{ua}/\phi N_n$	$V_{ua}/\phi V_n$	Combined Ratio	Permissible	Status
Sec. 17.6..1	0.79	0.00	79.0%	1.0	Pass

5/8"Ø AWS Type A Headed Stud with hef = 6.000 inch meets the selected design criteria.

12. Warnings

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

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 318-14
Units: Imperial units

Anchor Information:
Anchor type: Cast-in-place
Material: AWS Type A
Diameter (inch): 0.625
Effective Embedment depth, h_{ef} (inch): 6.000
Anchor category: -
Anchor ductility: Yes
h_{min} (inch): 7.38
C_{min} (inch): 1.38
S_{min} (inch): 2.50

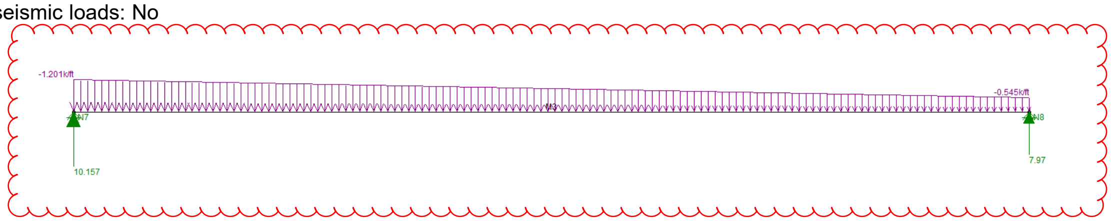
Base Material
Concrete: Normal-weight
Concrete thickness, h (inch): 9.50
State: Cracked
Compressive strength, f'_c (psi): 4000
$\Psi_{c,V}$: 1.0
Reinforcement condition: B tension, B shear
Supplemental reinforcement: Not applicable
Reinforcement provided at corners: No
Ignore concrete breakout in tension: No
Ignore concrete breakout in shear: No
Ignore 6do requirement: No
Build-up grout pad: No

Base Plate
Length x Width x Thickness (inch): 6.00 x 12.00 x 0.38

Recommended Anchor
Anchor Name: Headed Stud - 5/8"Ø AWS Type A Headed Stud

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

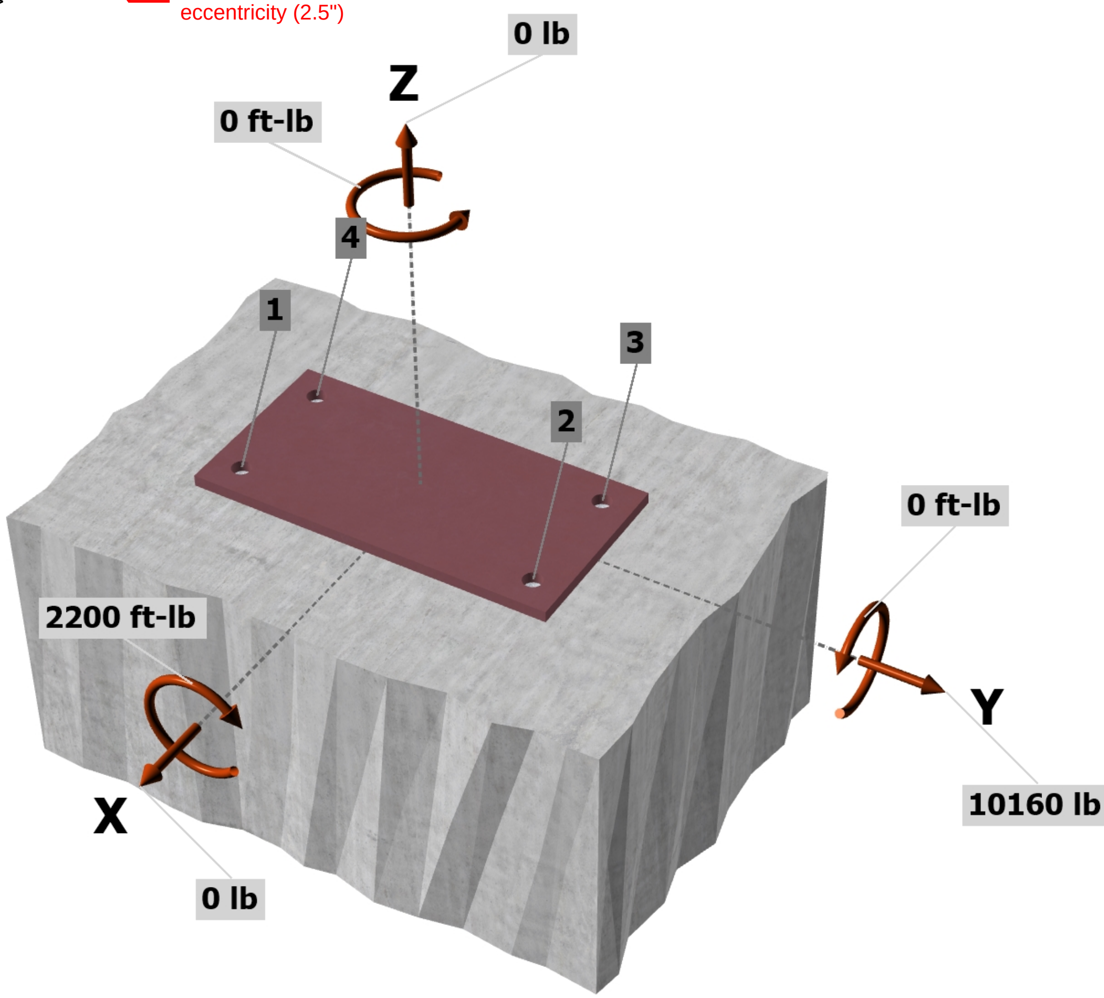
Strength level loads:



N_{ua} [lb]: 0
V_{uax} [lb]: 0
V_{uay} [lb]: 10160
M_{ux} [ft-lb]: -2200
M_{uy} [ft-lb]: 0
M_{uz} [ft-lb]: 0

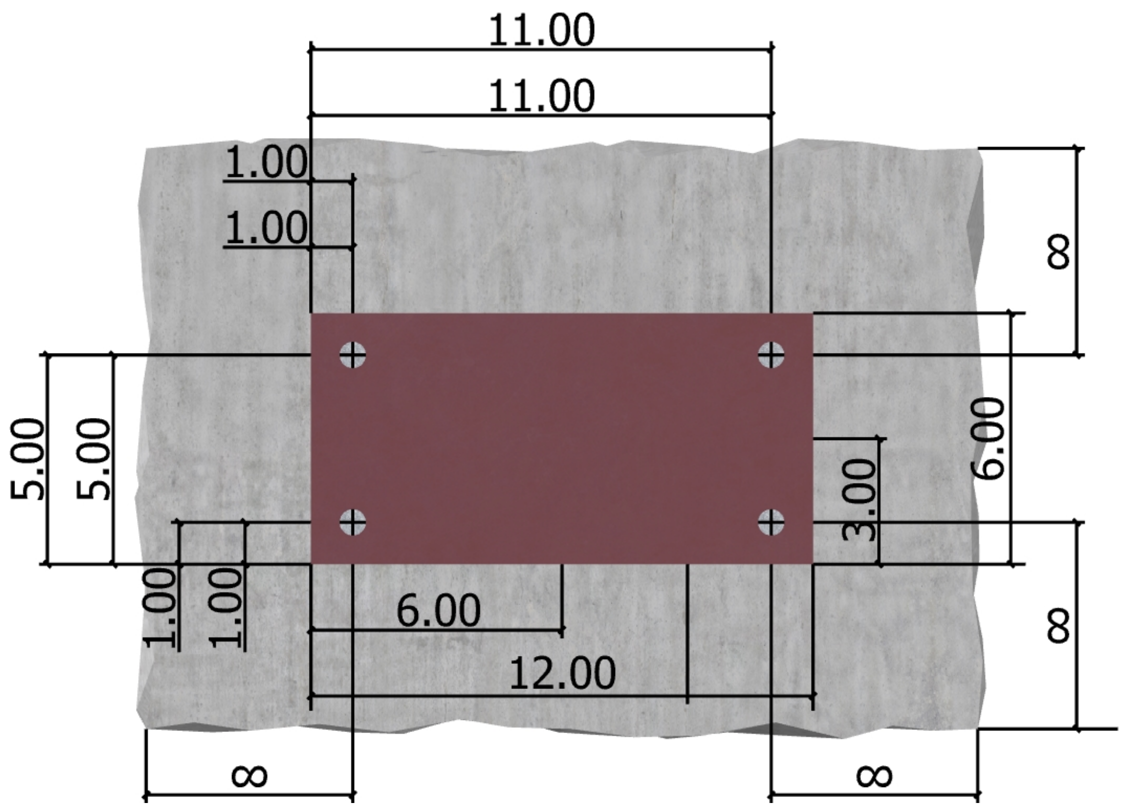
includes gravity loading w/ eccentricity (2.5")

<Figure 1>



Company:		Date:	6/15/2023
Engineer:		Page:	3/5
Project:			
Address:			
Phone:			
E-mail:			

<Figure 2>

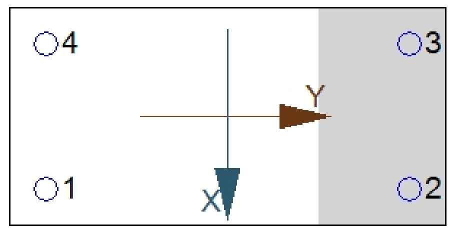


3. Resulting Anchor Forces

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	1342.7	0.0	2540.0	2540.0
2	0.0	0.0	2540.0	2540.0
3	0.0	0.0	2540.0	2540.0
4	1342.7	0.0	2540.0	2540.0
Sum	2685.4	0.0	10160.0	10160.0

Maximum concrete compression strain (‰): 0.06
Maximum concrete compression stress (psi): 255
Resultant tension force (lb): 2685
Resultant compression force (lb): 2685
Eccentricity of resultant tension forces in x-axis, e'_{Nx} (inch): 0.00
Eccentricity of resultant tension forces in y-axis, e'_{Ny} (inch): 0.00
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. 17.4.1)

N_{sa} (lb)	ϕ	ϕN_{sa} (lb)
18715	0.75	14036

5. Concrete Breakout Strength of Anchor in Tension (Sec. 17.4.2)

$N_b = k_c \lambda_a \sqrt{f'_c} h_{ef}^{1.5}$ (Eq. 17.4.2.2a)

k_c	λ_a	f'_c (psi)	h_{ef} (in)	N_b (lb)
24.0	1.00	4000	6.000	22308

$\phi N_{cbg} = \phi\,(A_{Nc} / A_{Nco})\, \Psi_{ec,N} \Psi_{ed,N} \Psi_{c,N} \Psi_{cp,N} N_b$ (Sec. 17.3.1 & Eq. 17.4.2.1b)

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	ϕN_{cbg} (lb)
396.00	324.00	-	1.000	1.000	1.00	1.000	22308	0.70	19086

6. Pullout Strength of Anchor in Tension (Sec. 17.4.3)

$\phi N_{pn} = \phi \Psi_{c,P} N_p = \phi \Psi_{c,P} 8 A_{brg} f'_c$ (Sec. 17.3.1, Eq. 17.4.3.1 & 17.4.3.4)

$\Psi_{c,P}$	A_{brg} (in^2)	f'_c (psi)	ϕ	ϕN_{pn} (lb)
1.0	0.92	4000	0.70	20608

8. Steel Strength of Anchor in Shear (Sec. 17.5.1)

V_{sa} (lb)	ϕ_{grout}	ϕ	$\phi_{grout}\phi V_{sa}$ (lb)
18715	1.0	0.65	12165

10. Concrete Pryout Strength of Anchor in Shear (Sec. 17.5.3)

$\phi V_{cpg} = \phi k_{cp} N_{cbg} = \phi k_{cp}(A_{Nc} / A_{Nco}) \Psi_{ec,N} \Psi_{ed,N} \Psi_{c,N} \Psi_{cp,N} N_b$ (Sec. 17.3.1 & Eq. 17.5.3.1b)

k_{cp}	A_{Nc} (in^2)	A_{Nco} (in^2)	$\Psi_{ec,N}$	$\Psi_{ed,N}$	$\Psi_{c,N}$	$\Psi_{cp,N}$	N_b (lb)	ϕ	ϕV_{cpg} (lb)
2.0	616.00	324.00	1.000	1.000	1.000	1.000	22308	0.70	59379

11. Results

Interaction of Tensile and Shear Forces (Sec. 17.6.)

Tension	Factored Load, N_{ua} (lb)	Design Strength, $øN_n$ (lb)	Ratio	Status
Steel	1343	14036	0.10	Pass
Concrete breakout	**2685**	**19086**	**0.14**	**Pass (Governs)**
Pullout	1343	20608	0.07	Pass

Shear	Factored Load, V_{ua} (lb)	Design Strength, $øV_n$ (lb)	Ratio	Status
Steel	**2540**	**12165**	**0.21**	**Pass (Governs)**
Pryout	10160	59379	0.17	Pass

Interaction check	$N_{ua}/\phi N_n$	$V_{ua}/\phi V_n$	Combined Ratio	Permissible	Status
Sec. 17.6..2	0.00	0.21	20.9%	1.0	Pass

5/8"Ø AWS Type A Headed Stud with hef = 6.000 inch meets the selected design criteria.

12. Warnings

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

ENTRY VESTIBULE FRAMING

TOP OF VESTIBULE = 27'

BOTTOM OF UPPER STOREFRONT = 15'

TOP OF LOWER STOREFRONT = 12'

V = 97 MPH

WIND COMPONENTS AND CLADDING

qh = (0.00256)(Kz)(Kzt)(Kd)(Ke)(V^2)

qh = (0.00256)(0.74)(1.0)(1.0)(0.85)(97^2) = 21.3 PSF

MAXIMUM BEAM SPAN = 17'

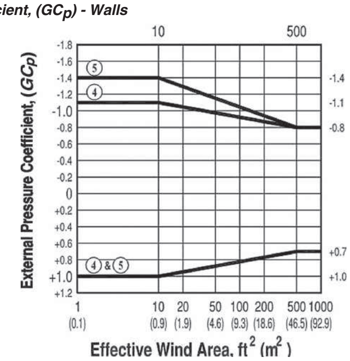
EFFECTIVE WIND AREA = (17')(17'/3) = 96.33 FT2

GCp = +0.8, -1.05
GCpi = +/-0.18

p = (qh)(GCp - GCpi)

p = (15.2 PSF)(-1.05 - 0.18) = 18.8 PSF

External Pressure Coefficient, (GCp) - Walls



Notes

1. Vertical scale denotes (GCp) to be used with qh.
2. Horizontal scale denotes effective wind area, in ft² (m²).
3. Plus and minus signs signify pressures acting toward and away from the surfaces, respectively.
4. Each component shall be designed for maximum positive and negative pressures.
5. Values of (GCp) for walls shall be reduced by 10% when θ ≤ 10°.

DETERMINE SEISMIC LOADING

$$Fp = \frac{(0.4)(ap)(Sds)(Wp)}{\frac{Rp}{Ip}}(1 + 2\,(z/h))$$

Table 13.5-1 Coefficients for Architectural Components

Architectural Component	a_p[a]	R_p	Ω_0[b]
Exterior nonstructural wall elements and connections[b]			
Wall element	1	2½	NA
Body of wall panel connections	1	2½	NA
Fasteners of the connecting system	1¼	1	1

$$Fp = \frac{(0.4)(1)(1.0)(15\ PSF)}{\frac{2.5}{1.0}}(1 + 2\,(0.5))$$

Fp = 4.8 PSF << 18.8 PSF

WIND C&C LOAD GOVERNS

LOWER MID STOREFRONT TRIB WIDTH = (12'/2) + (3'/2) = 7.5'

(7.5')(18.8 PSF)(0.6) = 85 PLF

DETERMINE DEFLECTION OOP

$$\delta = \frac{(5)(0.085\ KLF)(17')^4(1728)}{(384)(29{,}000\ KSI)(I_y)} = 1.13",\ I_y = 4.9\ in^4\ (L/180)$$

$I_y = 21.5\ in^4$ (L/360)

HSS6x5x5/16 (22.3 in^4)

VERTICAL DEFLECTION (STOREFRONT):

(15')(15 PSF) = 225 PLF

$$\delta = \frac{(5)(0.225\ KLF)(17')^4(1728)}{(384)(29{,}000\ KSI)(I_x)} = 0.57",\ I_x = 25.7\ in^4\ (L/360)$$

INCLUDE MEMBER SELF WEIGHT, ~21 PLF, LIMIT TO 1/2" DEFLECTION, TRY HSS6x5x5/16 (90.1 in4)

$$\delta = \frac{(5)(0.246\ KLF)(17')^4(1728)}{(384)(29{,}000\ KSI)(29.6\ in^4)} = 0.54",\ (L/1275)$$

DESIGN FOR BIAXIAL BENDING, WIND + GRAVITY LOADING 1.2D + 1.0W

Mux = (1.2)(0.246 KLF)$(17')^2$/8 = 10.8 k-ft

Sx = 9.85 in^3

(18.8 PSF)(7.5') = 141 KLF

Muy = (0.141 KLF)$(17')^2$/8 = 5.1 k-ft

Sy = 8.91 in^3

σ_1 = (10.8 k-ft)(12 in/ft)/9.85 in^3 = 13.2 KSI

σ_2 = (5.1 k-ft)(12 in/ft)/8.91 in^3 = 6.9 KSI

σ_{TOTAL} = 13.2 KSI + 6.9 KSI = 20 KSI

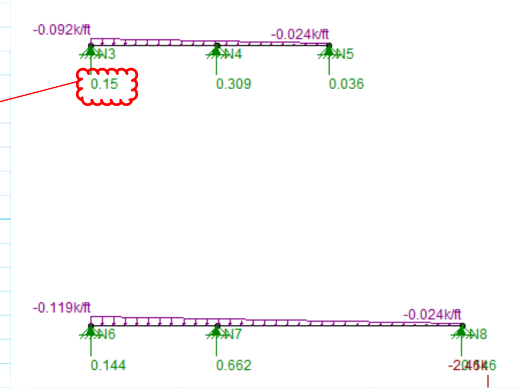
σ_{ALLOW} = (0.9)(46 KSI) = 41.4 KSI > 20 KSI, OK

UPPER STOREFRONT TRIB WIDTH = (12'/2) + (3'/2) = 7.5'

(7.5')(18.8 PSF)(0.6) = 85 PLF

DETERMINE DEFLECTION

$$\delta = \frac{(5)(0.085\ \text{KLF})(17')^4(1728)}{(384)(29{,}000\ \text{KSI})(I_y)} = 1.13",\ I_y = 4.18\ \text{in}^4\ (L/180)$$



VERTICAL DEFLECTION (STOREFRONT):

(15')(15 PSF) +150 PLF = 375 PLF

$$\delta = \frac{(5)(0.405\ \text{KLF})(17')^4(1728)}{(384)(29{,}000\ \text{KSI})(I_x)} = 0.5",\ I_x = 52.5\ \text{in}^4\ \text{TRY HSS10x5x5/16}$$

DESIGN FOR BIAXIAL BENDING, WIND + GRAVITY LOADING 1.2D + 1.6L + 0.5W

Mux = ((1.2)(0.255 KLF)+0.246 KLF)(17')^2/8 = 19.9 k-ft

Sx = 20.8 in^3

(0.5 x 18.8 PSF)(7.5') = 70.5 KLF

Muy = (0.0705 KLF)(17')^2/8 = 2.55 k-ft

Sy = 14.1 in^3

σ_1 = (19.9 k-ft)(12 in/ft)/20.8 in^3 = 11.5 KSI

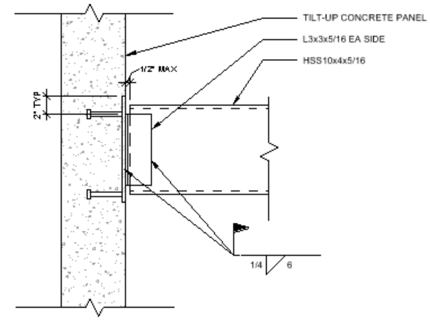
σ_2 = (2.55 k-ft)(12 in/ft)/14.1 in^3 = 2.1 KSI

σ_{TOTAL} = 11.5 KSI + 2.1 KSI = 13.6 KSI

σ_{ALLOW} = (0.9)(46 KSI) = 41.4 KSI > 13.6 KSI, OK

CHECK TIEBACK TO CONCRETE PANEL

(18.8 PSF)(17'/2)(27'/2) = 2.1 k FROM WIND



13 A5.15 HSS TO PANEL CONNECTION
1 1/2" = 1'-0"

LOAD DUE TO GRAVITY:

(15')(15 PSF)(4'/2) = 450 LB

1.2(D) = (1.2)(450 LB) = 540 LB

TOTAL DEMAND ON WELD FROM SHEAR/AXIAL:

sqrt($(2.1\ k)^2 + (0.6k)^2$) = 2.3 k

WELD PROVIDED:

(1.392)(4)(8") = 44.5k >> 2.3 k, OK

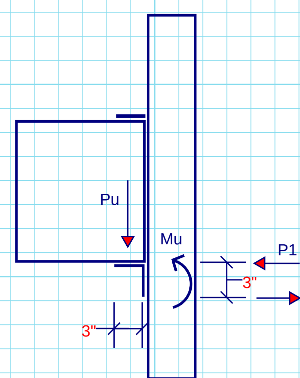
SEE HILTI FOR EMBED DESIGN

BEAM TO POST CONNECTION

Pu = (0.552 klf)(17')/2 = 4.7 k

Mu = (4.7 k)(3") = 14.1 k-in

P1 = 14.1 k-in / 3" = 4.7 k T/C



FORCE FROM WIND = (18.8 PSF)(27'/2)(17'/2) = 2.2 k

3 WELDS TO POST: 2.2 k/3 = 0.73 k

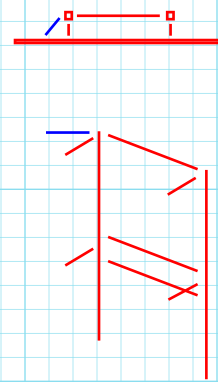
COMBINED TENSION/SHEAR ON WELD:

sqrt($(4.7\ k)^2 + (0.73\ k)^2$) = 4.8 k

(1.392)(4)(3) = 16.7k > 4.8 k, OK

WIND/SEISMIC FOR IN PLANE KICKER FORCE

$$F_p = \frac{(0.4)(1)(1.0)(15\ PSF)}{\frac{2.5}{1.0}}(1 + 2(0.5))$$



Fp = 4.8 PSF

AREA OF ENTRY FRAMING TRIB TO KICKER

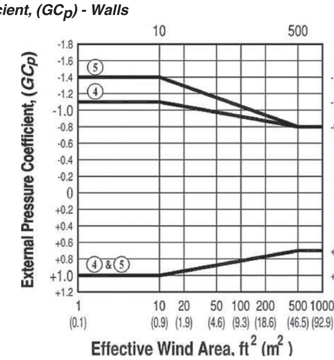
(17' + 2' + 2')(27'/2) = 284 FT2

(4.8 PSF)(284 FT2) = 1.4 k

DETERMINE WIND FORCE, EFFECTIVE WIND AREA:

(27'/2)(2') = 27 FT2

External Pressure Coefficient, (GC_p) - Walls



Notes

1. Vertical scale denotes (GCp) to be used with qh.
2. Horizontal scale denotes effective wind area, in ft^2 (m^2).
3. Plus and minus signs signify pressures acting toward and away from the surfaces, respectively.
4. Each component shall be designed for maximum positive and negative pressures.
5. Values of (GCp) for walls shall be reduced by 10% when $\theta \leq 10°$.

GCp = 0.92, -1.25
GCpi = +/-0.18

WINDWARD:

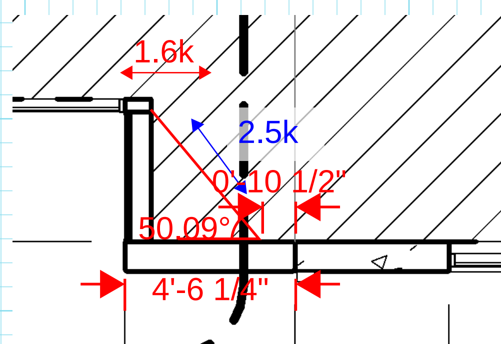
p = (15.2 PSF)(0.92 + 0.18) = 16.8 PSF

LEEWARD:

p = (15.2 PSF)(-1.25 - 0.18) = 21.7 PSF

TOTAL FORCE:

(16.8 PSF + 21.7 PSF)(4'/2)(27'/2) = 1.0 k



KICKER AT 50 DEGREES

1.0 k/cos(50) = 1.5k

4" WELD LENGTH

(1.392)(4)(4) = 22.2k >> 1.5k, OK

FORCE NORMAL TO WALL FACE

1.0k(sin(50)) = 3 k

SEE HILTI FOR EMBED PLATE CHECK

www.hilti.com

Company:		Page:	1
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Concrete - Dec 13, 2022	Date:	12/13/2022
Fastening point:			

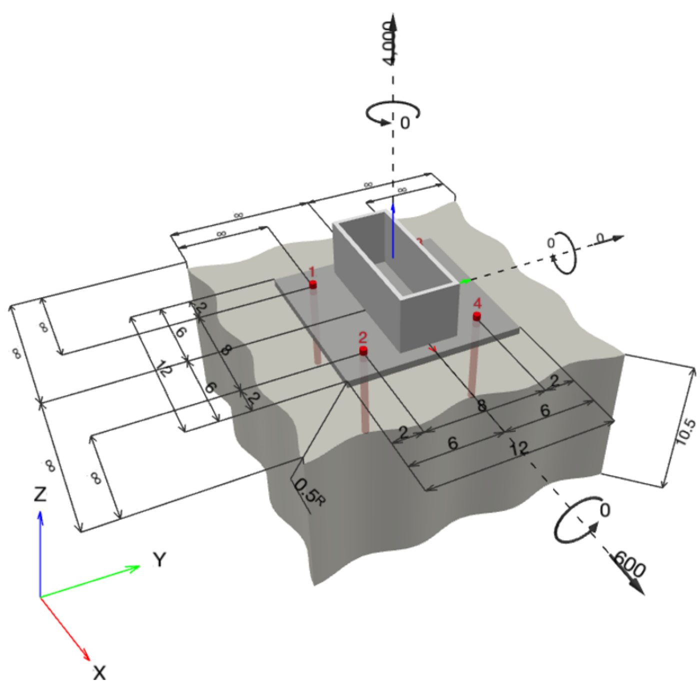
Specifier's comments:

1 Input data

Anchor type and diameter:	**Hex Head ASTM F 1554 GR. 36 1/2**
Item number:	not available
Effective embedment depth:	h_{ef} = 6.000 in.
Material:	ASTM F 1554
Evaluation Service Report:	Hilti Technical Data
Issued I Valid:	- | -
Proof:	Design Method ACI 318-14 / CIP
Stand-off installation:	e_b = 0.000 in. (no stand-off); t = 0.500 in.
Anchor plate[R] :	l_x x l_y x t = 12.000 in. x 12.000 in. x 0.500 in.; (Recommended plate thickness: not calculated)
Profile:	Rectangular HSS (AISC), HSS10X4X.3125; (L x W x T) = 10.000 in. x 4.000 in. x 0.312 in.
Base material:	cracked concrete, 4000, f_c' = 4,000 psi; h = 10.500 in.
Reinforcement:	tension: condition B, shear: condition B;
	edge reinforcement: none or < No. 4 bar

[R] - The anchor calculation is based on a rigid anchor plate assumption.

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



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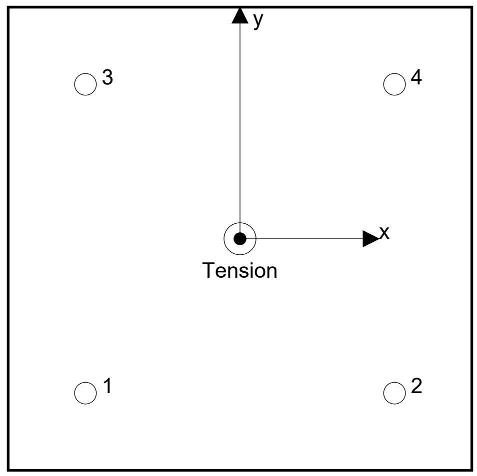
1.1 Design results

Case	Description	Forces [lb] / Moments [in.lb]	Seismic	Max. Util. Anchor [%]
1	Combination 1	N = 4,000; V_x = 600; V_y = 0; M_x = 0; M_y = 0; M_z = 0;	no	17

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	1,000	150	150	0
2	1,000	150	150	0
3	1,000	150	150	0
4	1,000	150	150	0

max. concrete compressive strain: - [‰]
max. concrete compressive stress: - [psi]
resulting tension force in (x/y)=(0.000/0.000): 4,000 [lb]
resulting compression force in (x/y)=(0.000/0.000): 0 [lb]

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

3 Tension load

	Load N_{ua} [lb]	Capacity ϕ N_n [lb]	Utilization $\beta_N = N_{ua}/\phi N_n$	Status
Steel Strength*	1,000	6,177	17	OK
Pullout Strength*	1,000	6,518	16	OK
Concrete Breakout Failure**	4,000	32,581	13	OK
Concrete Side-Face Blowout, direction **	N/A	N/A	N/A	N/A

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

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3.1 Steel Strength

$N_{sa} = A_{se,N}\, f_{uta}$ ACI 318-14 Eq. (17.4.1.2)
$\phi\ N_{sa} \geq N_{ua}$ ACI 318-14 Table 17.3.1.1

Variables

$A_{se,N}$ [in.^{2}]	f_{uta} [psi]
0.14	58,000

Calculations

N_{sa} [lb]
8,236

Results

N_{sa} [lb]	ϕ_{steel}	$\phi\ N_{sa}$ [lb]	N_{ua} [lb]
8,236	0.750	6,177	1,000

3.2 Pullout Strength

$N_{pN} = \psi_{c,p}\, N_p$ ACI 318-14 Eq. (17.4.3.1)
$N_p = 8\, A_{brg}\, f'_c$ ACI 318-14 Eq. (17.4.3.4)
$\phi\ N_{pN} \geq N_{ua}$ ACI 318-14 Table 17.3.1.1

Variables

$\psi_{c,p}$	A_{brg} [in.^{2}]	λ_a	f'_c [psi]
1.000	0.29	1.000	4,000

Calculations

N_p [lb]
9,312

Results

N_{pn} [lb]	$\phi_{concrete}$	$\phi\ N_{pn}$ [lb]	N_{ua} [lb]
9,312	0.700	6,518	1,000

3.3 Concrete Breakout Failure

$N_{cbg} = \left(\frac{A_{Nc}}{A_{Nc0}}\right) \psi_{ec,N}\ \psi_{ed,N}\ \psi_{c,N}\ \psi_{cp,N}\ N_b$	ACI 318-14 Eq. (17.4.2.1b)
$\phi\ N_{cbg} \geq N_{ua}$	ACI 318-14 Table 17.3.1.1
A_{Nc} see ACI 318-14, Section 17.4.2.1, Fig. R 17.4.2.1(b)	
$A_{Nc0} = 9\ h_{ef}^2$	ACI 318-14 Eq. (17.4.2.1c)
$\psi_{ec,N} = \left(\frac{1}{1 + \frac{2\ e'_N}{3\ h_{ef}}}\right) \leq 1.0$	ACI 318-14 Eq. (17.4.2.4)
$\psi_{ed,N} = 0.7 + 0.3 \left(\frac{c_{a,min}}{1.5h_{ef}}\right) \leq 1.0$	ACI 318-14 Eq. (17.4.2.5b)
$\psi_{cp,N} = \mathrm{MAX}\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5h_{ef}}{c_{ac}}\right) \leq 1.0$	ACI 318-14 Eq. (17.4.2.7b)
$N_b = k_c\ \lambda_a\ \sqrt{f'_c}\ h_{ef}^{1.5}$	ACI 318-14 Eq. (17.4.2.2a)

Variables

h_{ef} [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]	$\psi_{c,N}$
6.000	0.000	0.000	∞	1.000

c_{ac} [in.]	k_c	λ_a	f'_c [psi]
-	24	1.000	4,000

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ec1,N}$	$\psi_{ec2,N}$	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [lb]
676.00	324.00	1.000	1.000	1.000	1.000	22,308

Results

N_{cbg} [lb]	$\phi_{concrete}$	$\phi\ N_{cbg}$ [lb]	N_{ua} [lb]
46,545	0.700	32,581	4,000

4 Shear load

	Load V_{ua} [lb]	Capacity $\phi\ V_n$ [lb]	Utilization $\beta_V = V_{ua}/\phi\ V_n$	Status
Steel Strength*	150	3,212	5	OK
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout Strength**	600	65,162	1	OK
Concrete edge failure in direction **	N/A	N/A	N/A	N/A

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

4.1 Steel Strength

$V_{sa} = 0.6\ A_{se,V}\ f_{uta}$ ACI 318-14 Eq. (17.5.1.2b)

$\phi\ V_{steel} \geq V_{ua}$ ACI 318-14 Table 17.3.1.1

Variables

$A_{se,V}$ [in.^{2}]	f_{uta} [psi]
0.14	58,000

Calculations

V_{sa} [lb]
4,942

Results

V_{sa} [lb]	ϕ_{steel}	$\phi\ V_{sa,eq}$ [lb]	V_{ua} [lb]
4,942	0.650	3,212	150

4.2 Pryout Strength

$V_{cpg} = k_{cp} \left[\left(\frac{A_{Nc}}{A_{Nc0}} \right) \psi_{ec,N}\ \psi_{ed,N}\ \psi_{c,N}\ \psi_{cp,N}\ N_b \right]$ ACI 318-14 Eq. (17.5.3.1b)

$\phi\ V_{cpg} \geq V_{ua}$ ACI 318-14 Table 17.3.1.1

A_{Nc} see ACI 318-14, Section 17.4.2.1, Fig. R 17.4.2.1(b)

$A_{Nc0} = 9\ h_{ef}^2$ ACI 318-14 Eq. (17.4.2.1c)

$\psi_{ec,N} = \left(\frac{1}{1 + \frac{2\ e'_N}{3\ h_{ef}}} \right) \leq 1.0$ ACI 318-14 Eq. (17.4.2.4)

$\psi_{ed,N} = 0.7 + 0.3 \left(\frac{c_{a,min}}{1.5h_{ef}} \right) \leq 1.0$ ACI 318-14 Eq. (17.4.2.5b)

$\psi_{cp,N} = \text{MAX}\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5h_{ef}}{c_{ac}} \right) \leq 1.0$ ACI 318-14 Eq. (17.4.2.7b)

$N_b = k_c\ \lambda_a\ \sqrt{f'_c}\ h_{ef}^{1.5}$ ACI 318-14 Eq. (17.4.2.2a)

Variables

k_{cp}	h_{ef} [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]
2	6.000	0.000	0.000	∞

$\psi_{c,N}$	c_{ac} [in.]	k_c	λ_a	f'_c [psi]
1.000	∞	24	1.000	4,000

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ec1,N}$	$\psi_{ec2,N}$	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [lb]
676.00	324.00	1.000	1.000	1.000	1.000	22,308

Results

V_{cpg} [lb]	$\phi_{concrete}$	$\phi\ V_{cpg}$ [lb]	V_{ua} [lb]
93,089	0.700	65,162	600

5 Combined tension and shear loads

β_N	β_V	ζ	Utilization $\beta_{N,V}$ [%]	Status
0.162	0.047	5/3	6	OK

$\beta_{NV} = \beta_N^{\zeta} + \beta_V^{\zeta} <= 1$

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6 Warnings

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

Fastening meets the design criteria!

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7 Installation data

Profile: Rectangular HSS (AISC), HSS10X4X.3125; (L x W x T) = 10.000 in. x 4.000 in. x 0.312 in.

Hole diameter in the fixture: d_f = 0.562 in.

Plate thickness (input): 0.500 in.

Recommended plate thickness: not calculated

Anchor type and diameter: Hex Head ASTM F 1554 GR. 36 1/2

Item number: not available

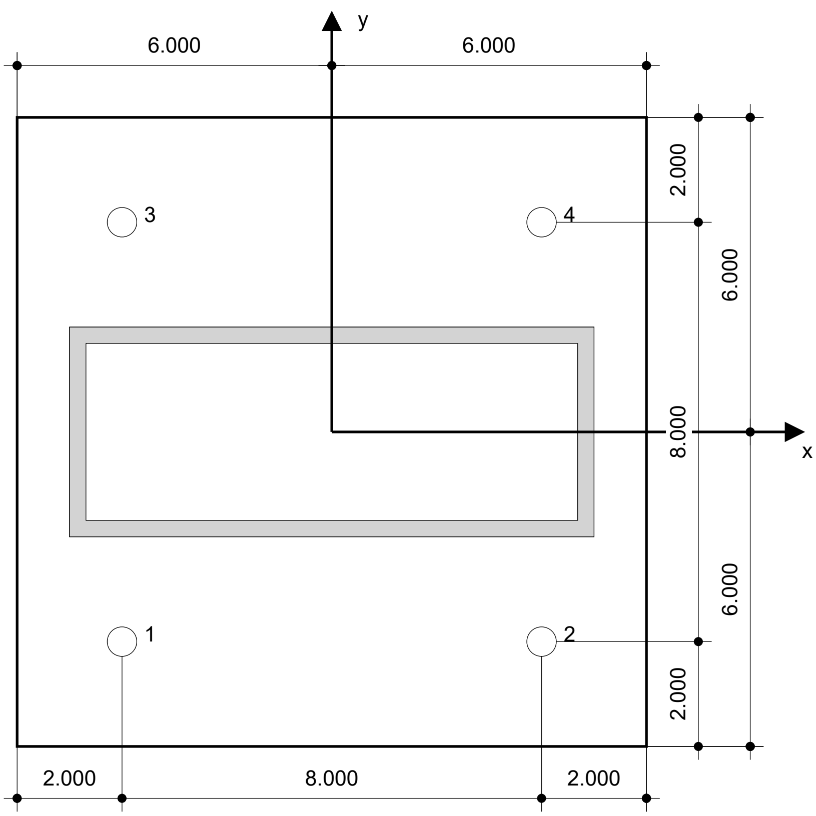
Maximum installation torque: -

Hole diameter in the base material: - in.

Hole depth in the base material: 6.000 in.

Minimum thickness of the base material: 6.844 in.

Hilti Hex Head headed stud anchor with 6 in embedment, 1/2, Steel galvanized, installation per instruction for use



Coordinates Anchor [in.]

Anchor	x	y	c_{-x}	c_{+x}	c_{-y}	c_{+y}
1	-4.000	-4.000	-	-	-	-
2	4.000	-4.000	-	-	-	-
3	-4.000	4.000	-	-	-	-
4	4.000	4.000	-	-	-	-

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8 Remarks; Your Cooperation Duties

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

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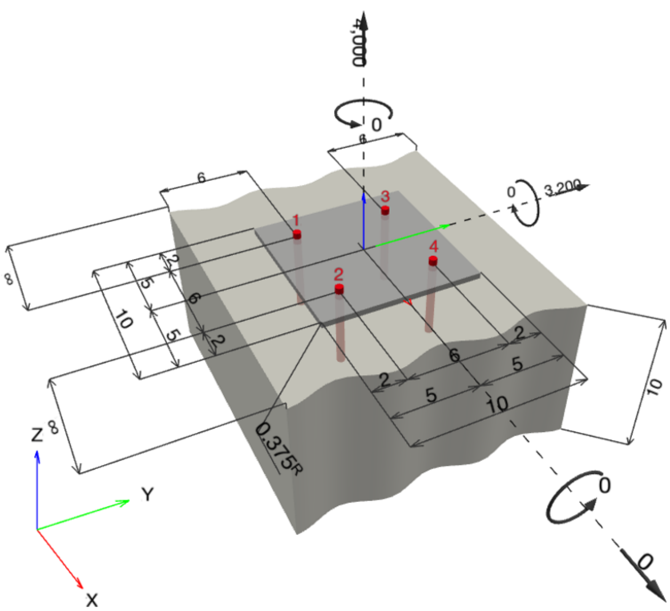
Specifier's comments:

1 Input data

Anchor type and diameter:	**Hex Head ASTM F 1554 GR. 36 1/2**
Item number:	not available
Effective embedment depth:	h_{ef} = 4.724 in.
Material:	ASTM F 1554
Evaluation Service Report:	Hilti Technical Data
Issued I Valid:	- | -
Proof:	Design Method ACI 318-14 / CIP
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 = 10.000 in. x 10.000 in. x 0.375 in.; (Recommended plate thickness: not calculated)
Profile:	no profile
Base material:	cracked concrete, 4000, f_c' = 4,000 psi; h = 10.000 in.
Reinforcement:	tension: condition B, shear: condition B;
	edge reinforcement: none or < No. 4 bar
Seismic loads (cat. C, D, E, or F)	Tension load: yes (17.2.3.4.3 (d))
	Shear load: yes (17.2.3.5.3 (c))

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

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



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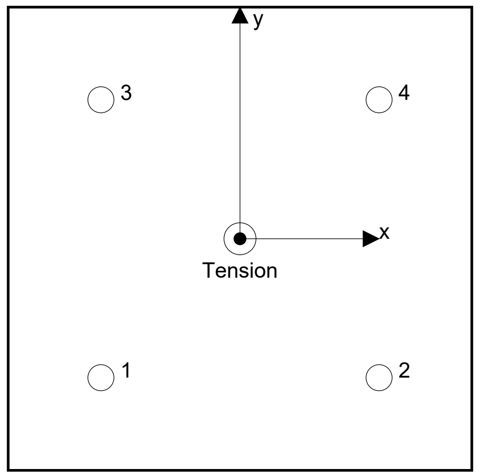
1.1 Design results

Case	Description	Forces [lb] / Moments [in.lb]	Seismic	Max. Util. Anchor [%]
1	Combination 1	N = 4,000; V_x = 0; V_y = 3,200; M_x = 0; M_y = 0; M_z = 0;	yes	50

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	1,000	800	0	800
2	1,000	800	0	800
3	1,000	800	0	800
4	1,000	800	0	800

max. concrete compressive strain: - [‰]
max. concrete compressive stress: - [psi]
resulting tension force in (x/y)=(0.000/0.000): 4,000 [lb]
resulting compression force in (x/y)=(0.000/0.000): 0 [lb]

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

3 Tension load

	Load N_{ua} [lb]	Capacity ϕ N_n [lb]	Utilization $\beta_N = N_{ua}/\phi\ N_n$	Status
Steel Strength*	1,000	6,177	17	OK
Pullout Strength*	1,000	4,889	21	OK
Concrete Breakout Failure**	4,000	14,112	29	OK
Concrete Side-Face Blowout, direction **	N/A	N/A	N/A	N/A

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

3.1 Steel Strength

$N_{sa} = A_{se,N}\, f_{uta}$ ACI 318-14 Eq. (17.4.1.2)
$\phi\, N_{sa} \geq N_{ua}$ ACI 318-14 Table 17.3.1.1

Variables

$A_{se,N}$ [in.^{2}]	f_{uta} [psi]
0.14	58,000

Calculations

N_{sa} [lb]
8,236

Results

N_{sa} [lb]	ϕ_{steel}	$\phi\, N_{sa}$ [lb]	N_{ua} [lb]
8,236	0.750	6,177	1,000

3.2 Pullout Strength

$N_{pN} = \psi_{c,p}\, N_p$ ACI 318-14 Eq. (17.4.3.1)
$N_p = 8\, A_{brg}\, f'_c$ ACI 318-14 Eq. (17.4.3.4)
$\phi\, N_{pN} \geq N_{ua}$ ACI 318-14 Table 17.3.1.1

Variables

$\psi_{c,p}$	A_{brg} [in.^{2}]	λ_a	f'_c [psi]
1.000	0.29	1.000	4,000

Calculations

N_p [lb]
9,312

Results

N_{pn} [lb]	$\phi_{concrete}$	$\phi_{seismic}$	$\phi_{nonductile}$	$\phi\, N_{pn}$ [lb]	N_{ua} [lb]
9,312	0.700	0.750	1.000	4,889	1,000

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3.3 Concrete Breakout Failure

$N_{cbg} = \left(\frac{A_{Nc}}{A_{Nc0}}\right) \psi_{ec,N}\ \psi_{ed,N}\ \psi_{c,N}\ \psi_{cp,N}\ N_b$ — ACI 318-14 Eq. (17.4.2.1b)

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

A_{Nc} see ACI 318-14, Section 17.4.2.1, Fig. R 17.4.2.1(b)

$A_{Nc0} = 9\ h_{ef}^2$ — ACI 318-14 Eq. (17.4.2.1c)

$\psi_{ec,N} = \left(\frac{1}{1 + \frac{2\ e'_N}{3\ h_{ef}}}\right) \leq 1.0$ — ACI 318-14 Eq. (17.4.2.4)

$\psi_{ed,N} = 0.7 + 0.3 \left(\frac{c_{a,min}}{1.5h_{ef}}\right) \leq 1.0$ — ACI 318-14 Eq. (17.4.2.5b)

$\psi_{cp,N} = \mathrm{MAX}\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5h_{ef}}{c_{ac}}\right) \leq 1.0$ — ACI 318-14 Eq. (17.4.2.7b)

$N_b = k_c\ \lambda_a\ \sqrt{f'_c}\ h_{ef}^{1.5}$ — ACI 318-14 Eq. (17.4.2.2a)

Variables

h_{ef} [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]	$\psi_{c,N}$
4.724	0.000	0.000	6.000	1.000

c_{ac} [in.]	k_c	λ_a	f'_c [psi]
-	24	1.000	4,000

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ec1,N}$	$\psi_{ec2,N}$	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [lb]
363.12	200.88	1.000	1.000	0.954	1.000	15,587

Results

N_{cbg} [lb]	$\phi_{concrete}$	$\phi_{seismic}$	$\phi_{nonductile}$	$\phi\ N_{cbg}$ [lb]	N_{ua} [lb]
26,879	0.700	0.750	1.000	14,112	4,000

4 Shear load

	Load V_{ua} [lb]	Capacity $\phi\ V_n$ [lb]	Utilization $\beta_V = V_{ua}/\phi\ V_n$	Status
Steel Strength*	800	3,212	25	OK
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout Strength**	3,200	37,631	9	OK
Concrete edge failure in direction y+**	3,200	6,509	50	OK

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

4.1 Steel Strength

$V_{sa} = 0.6\ A_{se,V}\ f_{uta}$ ACI 318-14 Eq. (17.5.1.2b)

$\phi\ V_{steel} \geq V_{ua}$ ACI 318-14 Table 17.3.1.1

Variables

$A_{se,V}$ [in.^{2}]	f_{uta} [psi]
0.14	58,000

Calculations

V_{sa} [lb]
4,942

Results

V_{sa} [lb]	ϕ_{steel}	$\phi\ V_{sa,eq}$ [lb]	V_{ua} [lb]
4,942	0.650	3,212	800

4.2 Pryout Strength

$V_{cpg} = k_{cp} \left[\left(\frac{A_{Nc}}{A_{Nc0}} \right) \psi_{ec,N}\ \psi_{ed,N}\ \psi_{c,N}\ \psi_{cp,N}\ N_b \right]$ ACI 318-14 Eq. (17.5.3.1b)

$\phi\ V_{cpg} \geq V_{ua}$ ACI 318-14 Table 17.3.1.1

A_{Nc} see ACI 318-14, Section 17.4.2.1, Fig. R 17.4.2.1(b)

$A_{Nc0} = 9\ h_{ef}^2$ ACI 318-14 Eq. (17.4.2.1c)

$\psi_{ec,N} = \left(\frac{1}{1 + \frac{2\ e'_N}{3\ h_{ef}}} \right) \leq 1.0$ ACI 318-14 Eq. (17.4.2.4)

$\psi_{ed,N} = 0.7 + 0.3 \left(\frac{c_{a,min}}{1.5h_{ef}} \right) \leq 1.0$ ACI 318-14 Eq. (17.4.2.5b)

$\psi_{cp,N} = \mathrm{MAX}\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5h_{ef}}{c_{ac}} \right) \leq 1.0$ ACI 318-14 Eq. (17.4.2.7b)

$N_b = k_c\ \lambda_a\ \sqrt{f'_c}\ h_{ef}^{1.5}$ ACI 318-14 Eq. (17.4.2.2a)

Variables

k_{cp}	h_{ef} [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]
2	4.724	0.000	0.000	6.000

$\psi_{c,N}$	c_{ac} [in.]	k_c	λ_a	f'_c [psi]
1.000	∞	24	1.000	4,000

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ec1,N}$	$\psi_{ec2,N}$	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [lb]
363.12	200.88	1.000	1.000	0.954	1.000	15,587

Results

V_{cpg} [lb]	$\phi_{concrete}$	$\phi_{seismic}$	$\phi_{nonductile}$	$\phi\ V_{cpg}$ [lb]	V_{ua} [lb]
53,759	0.700	1.000	1.000	37,631	3,200

4.3 Concrete edge failure in direction y+

$$V_{cbg} = \left(\frac{A_{Vc}}{A_{Vc0}}\right) \psi_{ec,V}\ \psi_{ed,V}\ \psi_{c,V}\ \psi_{h,V}\ \psi_{parallel,V}\ V_b$$ ACI 318-14 Eq. (17.5.2.1b)

$\phi\ V_{cbg} \geq V_{ua}$ ACI 318-14 Table 17.3.1.1

A_{Vc} see ACI 318-14, Section 17.5.2.1, Fig. R 17.5.2.1(b)

$A_{Vc0} = 4.5\ c_{a1}^2$ ACI 318-14 Eq. (17.5.2.1c)

$$\psi_{ec,V} = \left(\frac{1}{1 + \frac{2e_V'}{3c_{a1}}}\right) \leq 1.0$$ ACI 318-14 Eq. (17.5.2.5)

$$\psi_{ed,V} = 0.7 + 0.3\left(\frac{c_{a2}}{1.5c_{a1}}\right) \leq 1.0$$ ACI 318-14 Eq. (17.5.2.6b)

$$\psi_{h,V} = \sqrt{\frac{1.5c_{a1}}{h_a}} \geq 1.0$$ ACI 318-14 Eq. (17.5.2.8)

$$V_b = \left(7\left(\frac{l_e}{d_a}\right)^{0.2}\sqrt{d_a}\right) \lambda_a\ \sqrt{f_c'}\ c_{a1}^{1.5}$$ ACI 318-14 Eq. (17.5.2.2a)

Variables

c_{a1} [in.]	c_{a2} [in.]	e_{cV} [in.]	$\psi_{c,V}$	h_a [in.]
6.000	-	0.000	1.000	10.000

l_e [in.]	λ_a	d_a [in.]	f_c' [psi]	$\psi_{parallel,V}$
4.000	1.000	0.500	4,000	1.000

Calculations

A_{Vc} [in.^{2}]	A_{Vc0} [in.^{2}]	$\psi_{ec,V}$	$\psi_{ed,V}$	$\psi_{h,V}$	V_b [lb]
216.00	162.00	1.000	1.000	1.000	6,974

Results

V_{cbg} [lb]	$\phi_{concrete}$	$\phi_{seismic}$	$\phi_{nonductile}$	$\phi\ V_{cbg}$ [lb]	V_{ua} [lb]
9,298	0.700	1.000	1.000	6,509	3,200

5 Combined tension and shear loads

β_N	β_V	ζ	Utilization $\beta_{N,V}$ [%]	Status
0.283	0.492	5/3	43	OK

$\beta_{NV} = \beta_N^{\zeta} + \beta_V^{\zeta} <= 1$

6 Warnings

- The anchor design methods in PROFIS Engineering require rigid anchor plates per current regulations (AS 5216:2021, ETAG 001/Annex C, EOTA TR029 etc.). This means load re-distribution on the anchors due to elastic deformations of the anchor plate are not considered - the anchor plate is assumed to be sufficiently stiff, in order not to be deformed when subjected to the design loading. PROFIS Engineering calculates the minimum required anchor plate thickness with CBFEM to limit the stress of the anchor plate based on the assumptions explained above. The proof if the rigid anchor plate assumption is valid is not carried out by PROFIS Engineering. Input data and results must be checked for agreement with the existing conditions and for plausibility!
- Condition A applies where the potential concrete failure surfaces are crossed by supplementary reinforcement proportioned to tie the potential concrete failure prism into the structural member. Condition B applies where such supplementary reinforcement is not provided, or where pullout or pryout strength governs.
- For additional information about ACI 318 strength design provisions, please go to https://submittals.us.hilti.com/PROFISAnchorDesignGuide/
- An anchor design approach for structures assigned to Seismic Design Category C, D, E or F is given in ACI 318-14, Chapter 17, Section 17.2.3.4.3 (a) that requires the governing design strength of an anchor or group of anchors be limited by ductile steel failure. If this is NOT the case, the connection design (tension) shall satisfy the provisions of Section 17.2.3.4.3 (b), Section 17.2.3.4.3 (c), or Section 17.2.3.4.3 (d). The connection design (shear) shall satisfy the provisions of Section 17.2.3.5.3 (a), Section 17.2.3.5.3 (b), or Section 17.2.3.5.3 (c).
- Section 17.2.3.4.3 (b) / Section 17.2.3.5.3 (a) require the attachment the anchors are connecting to the structure be designed to undergo ductile yielding at a load level corresponding to anchor forces no greater than the controlling design strength. Section 17.2.3.4.3 (c) / Section 17.2.3.5.3 (b) waive the ductility requirements and require the anchors to be designed for the maximum tension / shear that can be transmitted to the anchors by a non-yielding attachment. Section 17.2.3.4.3 (d) / Section 17.2.3.5.3 (c) waive the ductility requirements and require the design strength of the anchors to equal or exceed the maximum tension / shear obtained from design load combinations that include E, with E increased by ω_0.

Fastening meets the design criteria!

Company:		Page:	9
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Concrete - Dec 16, 2022	Date:	12/17/2022
Fastening point:			

7 Installation data

Profile: no profile

Hole diameter in the fixture: d_f = 0.562 in.

Plate thickness (input): 0.375 in.

Recommended plate thickness: not calculated

Anchor type and diameter: Hex Head ASTM F 1554 GR. 36 1/2

Item number: not available

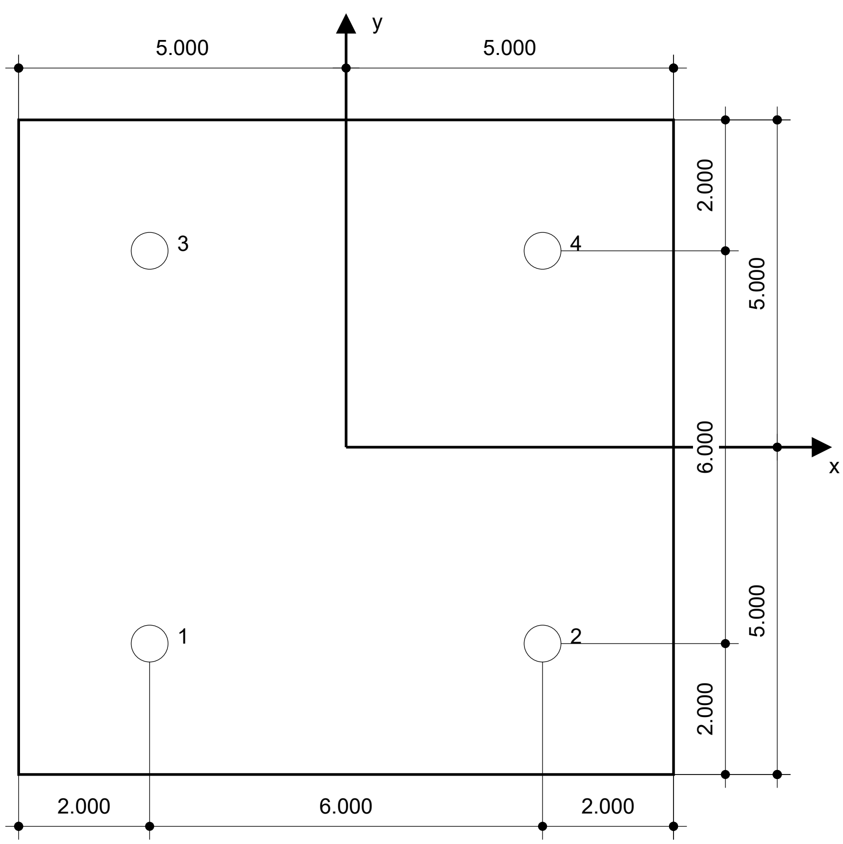
Maximum installation torque: -

Hole diameter in the base material: - in.

Hole depth in the base material: 4.724 in.

Minimum thickness of the base material: 5.568 in.

Hilti Hex Head headed stud anchor with 4.724409 in embedment, 1/2, Steel galvanized, installation per instruction for use



Coordinates Anchor [in.]

Anchor	x	y	c_{-x}	c_{+x}	c_{-y}	c_{+y}
1	-3.000	-3.000	-	-	6.000	12.000
2	3.000	-3.000	-	-	6.000	12.000
3	-3.000	3.000	-	-	12.000	6.000
4	3.000	3.000	-	-	12.000	6.000

Company:		Page:	10
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Concrete - Dec 16, 2022	Date:	12/17/2022
Fastening point:			

8 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.

Dock Retaining Wall

Purpose Statement: The purpose of this calculation is to design a cantilevered Concrete Masonry wall for Out-Of-Plane Forces and Continuous Footing

Referenced Standards: IBC 2018
ASCE 7-16
ACI 318-19

Inputs
Verify
Outputs

General

Dimensions:

$ht_w := 9 \cdot ft$ Height of wall to top of footing

$t := 5.5\ in$ Wall Thickness

$ht_{drive} := 5.5\ ft$ Height from top of footing to upper driveway

$L_1 := 4\ ft$ Length to inside face of wall

$d_{ftg} := 1\ ft$ Thickness of the footing

$W := 6 \cdot ft$ Total width of the wall footing

$h_{apron} := 15\ in$ Height to middle of apron from top of footing

Material Parameters:

$f'c := 3\ ksi$ Concrete strength

$f_y := 60\ ksi$ Rebar Strength

$soil := 125\ pcf$ Soil unit weight

$q_{bearing} := 2500 \cdot psf$ Allowable Soil Bearing Pressure

$q_{increase} := \frac{4}{3}$ Short term lateral load bearing increase

$q_{allow} := q_{bearing} \cdot q_{increase} = 3333.3\ psf$

Lateral Load Parameters:

$S_{DS} := 0.843$ Design Short Term Spectral Response Acceleration Coefficient

Dock Retaining Wall

Loading/Force Parameters:

$H_{LL} := 1000\ lb$ — Horizontal force onto wall due to Live Load per Unit Length (250 PSF*4')

$H_{soil} := 0.5 \cdot 35\ pcf \cdot (ht_{drive})^2 \cdot 1\ ft$

$H_{soil} = 529.4\ lb$ — Horizontal force due to Soil Load per Unit Length*

$M_{OTLL} := H_{LL} \cdot (ht_{drive} - h_{apron}) = 4250\ lb \cdot ft$ — Unfactored Overturning Moment due to Live Load per Unit Length*

$M_{OTSOIL} := H_{soil} \cdot \dfrac{ht_{drive} - h_{apron}}{2} = 1124.9\ lb \cdot ft$ — Unfactored Overturning Moment due to Soil Load per Unit Length*

$R := 1.25$ $\quad I := 1.0$ — Parameters per Table 15.4-2 (Cantilevered Walls)

$W_p := t \cdot 150\ pcf \cdot (ht_{drive} - h_{apron}) = 292.2\ plf$

$$C_s := \frac{S_{DS}}{\left(\dfrac{R}{I}\right)}$$

$E_{h1} := C_s \cdot W_p \cdot 1\ ft = 197.1\ lb$ — Base Shear per Unit Length from Wall OOP (Eq 12.8-1)

$E_{h2} := 8 \cdot \left(\dfrac{ht_{drive}}{ft}\right)^2 \cdot lb = 242\ lb$ — Force due to Soil Seismic*

$M_{EQ1} := E_{h1} \cdot \dfrac{ht_w - h_{apron}}{2} = 763.6\ ft \cdot lb$ — Moment due to Wall OOP per Unit Length

$M_{EQ2} := E_{h2} \cdot \dfrac{ht_{drive} - h_{apron}}{2} = 514.3\ lb \cdot ft$ — Moment due to soil seismic effect*

$M_{OTEQ} := M_{EQ1} + M_{EQ2} = 1277.8\ lb \cdot ft$ — Unfactored Overturning Moment due to Total Seismic

*Equations to calculate these values will vary based on formulas given by the geotechnical engineer. Calculate externally then plug in above

Check DL + LL Load Combination

$W_w := t \cdot 150\ pcf \cdot ht_w \cdot 1\ ft = 618.8\ lb$

$W_{ftg} := 150\ pcf \cdot W \cdot d_{ftg} \cdot 1\ ft = 900\ lb$

$W_{soil} := soil \cdot ht_{drive} \cdot L_1 \cdot 1\ ft = 2750\ lb$

$S_x := \dfrac{1\ ft \cdot W^2}{6} = 6\ ft^3$ Section Modulus per Unit Length

$\sigma_{gravity} := \dfrac{W_w + W_{ftg} + W_{soil}}{1\ ft \cdot W} = 711.5\ psf$

$M_{RES} := W_w \cdot \left(L_1 + \dfrac{t}{2}\right) = 2616.8\ lb \cdot ft$ Restoring Moment

$M_{OTnet} := M_{OTLL} + M_{OTSOIL} - M_{RES} = 2758.1\ lb \cdot ft$ Net Overturning Moment

$\sigma_{net} := \dfrac{M_{OTnet}}{S_x} = 459.7\ psf$ Maximum tension/compression in soil from overturning

$Check_1 := \text{if}\left(\sigma_{net} < \sigma_{gravity}, \text{“NO NET TENSION”}, \text{“NG”}\right) = \text{“NO NET TENSION”}$

$Check_2 := \text{if}\left(\sigma_{net} + \sigma_{gravity} < q_{bearing}, \text{“OK”}, \text{“NG”}\right) = \text{“OK”}$

Check DL + 0.7E Load Combination

$DL := W_w + W_{ftg} + W_{soil} = 4268.8\ lb$

$E_v := 0.2 \cdot S_{DS} \cdot DL = 719.7\ lb$ Seismic Vertical Component

$E_h := E_{h1} + E_{h2} = 439.1\ lb$ Seismic Horizontal Component

$R := 0.7\ E_h + H_{soil} = 836.7\ lb$ Reaction from Lower Truck Apron

$\sigma_{gravity} := \dfrac{DL - (0.7\ E_v)}{1\ ft \cdot W} = 627.5\ psf$

$M_{OT} := M_{OTSOIL} + 0.7\ M_{OTEQ} = 2019.4\ lb \cdot ft$ Overturning Moment

$M_{RES} := R \cdot h_{apron} = 1045.9\ lb \cdot ft$ Restoring Moment

$M_{OTnet} := M_{OT} - M_{RES} = 973.5\ lb \cdot ft$ Net Overturning Moment

$\sigma_{net} := \frac{M_{OTnet}}{S_x} = 162.3\ psf$ Maximum tension/compression in soil from overturning

$Check_3 := \text{if}\left(\sigma_{net} < \sigma_{gravity}, \text{"NO NET TENSION"}, \text{"NG"}\right) = \text{"NO NET TENSION"}$

$Check_4 := \text{if}\left(\sigma_{net} + \sigma_{gravity} < q_{allow}, \text{"OK"}, \text{"NG"}\right) = \text{"OK"}$

Check DL + 0.525E + 0.75LL Load Combination

$\sigma_{gravity} := \frac{DL - (0.525\ E_v)}{1\ ft \cdot W} = 648.5\ psf$

$M_{OT} := M_{OTSOIL} + 0.525\ M_{OTEQ} + 0.75\ M_{OTLL} = 4983.3\ lb \cdot ft$ Overturning Moment

$R := 0.525\ E_h + H_{soil} + 0.75\ H_{LL} = 1509.9\ lbf$ Reaction from Lower Truck Apron

$M_{RES} := R \cdot h_{apron} = 1887.3\ lb \cdot ft$ Restoring Moment

$M_{OTnet} := M_{OT} - M_{RES} = 3095.9\ lb \cdot ft$ Net Overturning Moment

$\sigma_{net} := \frac{M_{OTnet}}{S_x} = 516\ psf$ Maximum tension/compression in soil from overturning

$Check_5 := \text{if}\left(\sigma_{net} < \sigma_{gravity}, \text{"NO NET TENSION"}, \text{"NG"}\right) = \text{"NO NET TENSION"}$

$Check_6 := \text{if}\left(\sigma_{net} + \sigma_{gravity} < q_{allow}, \text{"OK"}, \text{"NG"}\right) = \text{"OK"}$

Check Alternate IBC Load Combination [0.9-(0.2)(Sds)]DL + E/1.4

$\sigma_{gravity} := \frac{(0.9 - (0.2\ S_{DS})) \cdot DL}{1\ ft \cdot W} = 520.4\ psf$

$R := \frac{E_h}{1.4} + 0.9\ H_{soil} = 790\ lb$ Reaction from Lower Truck Apron

$M_{OT} := 0.9\ M_{OTSOIL} + \frac{M_{OTEQ}}{1.4} = 1925.2\ lb \cdot ft$ Overturning Moment

$M_{RES} := R \cdot h_{apron} = 987.6\ lb \cdot ft$ Restoring Moment

$M_{OTnet} := M_{OT} - M_{RES} = 937.6\ lb \cdot ft$ Net Overturning Moment

$\sigma_{net} := \frac{M_{OTnet}}{S_x} = 156.3\ psf$ Maximum tension/compression in soil from overturning

$Check_7 := \text{if}\left(\sigma_{net} < \sigma_{gravity}, \text{“NO NET TENSION”}, \text{“NG”}\right) = \text{“NO NET TENSION”}$

$Check_8 := \text{if}\left(\sigma_{net} + \sigma_{gravity} < q_{allow}, \text{“OK”}, \text{“NG”}\right) = \text{“OK”}$

Design Footing

1.6H + 1.6L

$Mu_1 := 1.6\ \left(M_{OTSOIL} + M_{OTLL}\right) = 8.6\ k \cdot ft$

$R_1 := 1.6\ \left(H_{LL} + H_{soil}\right) = 2447\ lbf$

$M_{RES1} := R_1 \cdot h_{apron} = 3.1\ k \cdot ft$

$Mu_{f1} := Mu_1 - M_{RES1} = 5.5\ k \cdot ft$

1.6H + 1.0Eh + 1.0L

$Mu_2 := 1.6 \cdot \left(M_{OTSOIL}\right) + M_{EQ1} + M_{EQ2} + M_{OTLL} = 7.3\ k \cdot ft$

$R_2 := 1.6 \cdot H_{soil} + E_{h1} + E_{h2} + H_{LL} = 2286.1\ lbf$

$M_{RES2} := R_2 \cdot h_{apron} = 2.9\ k \cdot ft$

$Mu_{f2} := Mu_2 - M_{RES2} = 4.5\ k \cdot ft$

$Mu_{ftg} := \max\left(Mu_{f1}, Mu_{f2}\right) = 5.5\ k \cdot ft$

$\sigma_{Mot} := \frac{Mu_{ftg}}{S_x} = 923.5\ psf$

$P_u := 1.2\ \left(\left(t \cdot ht_w + W \cdot d_{ftg}\right)\ 150\ pcf \cdot 1\ ft\right) + 1.0\ \left(ht_{drive} \cdot L_1 \cdot 1\ ft \cdot soil\right) = 4.6\ k$

$\sigma_{Pu} := \frac{P_u}{W \cdot 1\ ft} = 762.1\ psf$

$Mu_{ftg} := \left(\sigma_{Mot} + \sigma_{Pu}\right) \cdot 1\ ft \cdot \frac{\left(W - t - L_1\right)^2}{2} = 2\ k \cdot ft$

$A_{sftg} :=$ Reinforcement size: #5 $= 0.31\ in^2$

$sp_{ftg} := 12\ in$

$d := d_{ftg} - 3\ in - (1.5 \cdot bar_{diaftg}) = 8.1\ in$

Flexure (per 1' width) - ACI 318-14 13.2.6.4, 13.2.7.1

$$f(A_{sreq}) := A_{sreq} \cdot f_y \cdot \left(d_{ftg} - \frac{A_{sreq} \cdot f_y}{.85 \cdot f'c \cdot (1 \cdot ft) \cdot 2}\right) \cdot .9 - Mu_{ftg}$$

Find required area of steel

Guess Values

$A_{sreq} := 1\ in^2$

Constraints

$$A_{sreq} \cdot f_y \cdot \left(d_{ftg} - \frac{A_{sreq} \cdot f_y}{.85 \cdot f'c \cdot 1\ ft \cdot 2}\right) \cdot .9 - Mu_{ftg} = 0$$

Solver

$A_{sreq} := \text{Find}(A_{sreq})$ $\quad A_{sreq} = 0.04\ in^2$

Required area of steel for footing flexure

$A_{smin} := 0.0018 \cdot t \cdot (1\ ft) = 0.12\ in^2$

Min flexural reinforcement per ACI 318-14 8.6.1.1

$$A_s := \frac{A_{sftg} \cdot (1\ ft)}{sp_{ftg}} = 0.31\ in^2$$

Area of reinforcement provided per foot of width

$A_{sreq} := \max(A_{sreq}, A_{smin}) = 0.12\ in^2$

$$\frac{A_{sreq}}{A_s} = 0.4$$

$Flag_f := \text{if}(A_s \geq A_{sreq}, \text{“OK”}, \text{“NG!!”})$

$Flag_f = \text{“OK”}$

Design Wall

1.6H + 1.0Eh + 1.0L

$Mu_{wall} := 1.6 \cdot (M_{OTSOIL}) + M_{EQ1} + M_{EQ2} + M_{OTLL} = 7.3\ k \cdot ft$

$A_{sw} :=$ Reinforcement size: #6 $= 0.44\ in^2$

$sp := 6\ in$

$d_w := t - 1.5\ in - 0.625\ in - (0.5 \cdot bar_{diaw}) = 3\ in$

Flexure (per 1' width) - ACI 318-14 13.2.6.4, 13.2.7.1

$f(A_{sreq}) := A_{sreq} \cdot f_y \cdot \left(d_w - \frac{A_{sreq} \cdot f_y}{.85 \cdot f'c \cdot (1 \cdot ft) \cdot 2}\right) \cdot .9 - Mu_{wall}$ Find required area of steel

Guess Values

$A_{sreq} := 1\ in^2$

Constraints

$A_{sreq} \cdot f_y \cdot \left(d_w - \frac{A_{sreq} \cdot f_y}{.85 \cdot f'c \cdot 1\ ft \cdot 2}\right) \cdot .9 - Mu_{wall} = 0$

Solver

$A_{sreq} := \text{Find}(A_{sreq})$ $A_{sreq} = 0.71\ in^2$ Required area of steel for footing flexure

$A_{smin} := 0.0018 \cdot t \cdot (1\ ft) = 0.12\ in^2$ Min flexural reinforcement per ACI 318-14 8.6.1.1

$A_s := \frac{A_{sw} \cdot (1\ ft)}{sp} = 0.88\ in^2$ Area of reinforcement provided per foot of width

$A_{sreq} := \max(A_{sreq}, A_{smin}) = 0.71\ in^2$ $\frac{A_{sreq}}{A_s} = 0.8$

$Flag_f := \text{if}\left(A_s \ge A_{sreq}, \text{“OK”}, \text{“NG!!”}\right)$ $Flag_f = \text{“OK”}$

Dock Retaining Wall

Purpose Statement: The purpose of this calculation is to design a cantilevered Concrete Masonry wall for Out-Of-Plane Forces and Continuous Footing

Referenced Standards: IBC 2018
ASCE 7-16
ACI 318-19

Inputs
Verify
Outputs

General

Dimensions:

$ht_w := 6 \cdot ft$	Height of wall to top of footing
$t := 5.5\ in$	Wall Thickness
$ht_{drive} := 4\ ft$	Height from top of footing to upper driveway
$L_1 := 2\ ft$	Length to inside face of wall
$d_{ftg} := 1\ ft$	Thickness of the footing
$W := 4 \cdot ft$	Total width of the wall footing
$h_{apron} := 15\ in$	Height to middle of apron from top of footing

Material Parameters:

$f'c := 3\ ksi$	Concrete strength
$f_y := 60\ ksi$	Rebar Strength
$soil := 125\ pcf$	Soil unit weight
$q_{bearing} := 2500 \cdot psf$	Allowable Soil Bearing Pressure
$q_{increase} := \frac{4}{3}$	Short term lateral load bearing increase

$$q_{allow} := q_{bearing} \cdot q_{increase} = 3333.3\ psf$$

Lateral Load Parameters:

$S_{DS} := 0.843$ — Design Short Term Spectral Response Acceleration Coefficient

Dock Retaining Wall

Loading/Force Parameters:

$H_{LL} := 625\ lb$ — Horizontal force onto wall due to Live Load per Unit Length (250 PSF*2.5')

$H_{soil} := 0.5 \cdot 35\ pcf \cdot (ht_{drive})^2 \cdot 1\ ft$

$H_{soil} = 280\ lb$ — Horizontal force due to Soil Load per Unit Length*

$M_{OTLL} := H_{LL} \cdot (ht_{drive} - h_{apron}) = 1718.8\ lb \cdot ft$ — Unfactored Overturning Moment due to Live Load per Unit Length*

$M_{OTSOIL} := H_{soil} \cdot \dfrac{ht_{drive} - h_{apron}}{2} = 385\ lb \cdot ft$ — Unfactored Overturning Moment due to Soil Load per Unit Length*

$R := 1.25$ $\quad I := 1.0$ — Parameters per Table 15.4-2 (Cantilevered Walls)

$W_p := t \cdot 150\ pcf \cdot (ht_{drive} - h_{apron}) = 189.1\ plf$

$$C_s := \frac{S_{DS}}{\left(\dfrac{R}{I}\right)}$$

$E_{h1} := C_s \cdot W_p \cdot 1\ ft = 127.5\ lb$ — Base Shear per Unit Length from Wall OOP (Eq 12.8-1)

$E_{h2} := 8 \cdot \left(\dfrac{ht_{drive}}{ft}\right)^2 \cdot lb = 128\ lb$ — Force due to Soil Seismic*

$M_{EQ1} := E_{h1} \cdot \dfrac{ht_w - h_{apron}}{2} = 302.8\ ft \cdot lb$ — Moment due to Wall OOP per Unit Length

$M_{EQ2} := E_{h2} \cdot \dfrac{ht_{drive} - h_{apron}}{2} = 176\ lb \cdot ft$ — Moment due to soil seismic effect*

$M_{OTEQ} := M_{EQ1} + M_{EQ2} = 478.8\ lb \cdot ft$ — Unfactored Overturning Moment due to Total Seismic

*Equations to calculate these values will vary based on formulas given by the geotechnical engineer. Calculate externally then plug in above

Check DL + LL Load Combination

$W_w := t \cdot 150\ pcf \cdot ht_w \cdot 1\ ft = 412.5\ lb$

$W_{ftg} := 150\ pcf \cdot W \cdot d_{ftg} \cdot 1\ ft = 600\ lb$

$W_{soil} := soil \cdot ht_{drive} \cdot L_1 \cdot 1\ ft = 1000\ lb$

$S_x := \dfrac{1\ ft \cdot W^2}{6} = 2.67\ ft^3$ — Section Modulus per Unit Length

$\sigma_{gravity} := \dfrac{W_w + W_{ftg} + W_{soil}}{1\ ft \cdot W} = 503.1\ psf$

$M_{RES} := W_w \cdot \left(L_1 + \dfrac{t}{2}\right) = 919.5\ lb \cdot ft$ — Restoring Moment

$M_{OTnet} := M_{OTLL} + M_{OTSOIL} - M_{RES} = 1184.2\ lb \cdot ft$ — Net Overturning Moment

$\sigma_{net} := \dfrac{M_{OTnet}}{S_x} = 444.1\ psf$ — Maximum tension/compression in soil from overturning

$Check_1 := \text{if}\left(\sigma_{net} < \sigma_{gravity}, \text{“NO NET TENSION”}, \text{“NG”}\right) = \text{“NO NET TENSION”}$

$Check_2 := \text{if}\left(\sigma_{net} + \sigma_{gravity} < q_{bearing}, \text{“OK”}, \text{“NG”}\right) = \text{“OK”}$

Check DL + 0.7E Load Combination

$DL := W_w + W_{ftg} + W_{soil} = 2012.5\ lb$

$E_v := 0.2 \cdot S_{DS} \cdot DL = 339.3\ lb$ — Seismic Vertical Component

$E_h := E_{h1} + E_{h2} = 255.5\ lb$ — Seismic Horizontal Component

$R := 0.7\ E_h + H_{soil} = 458.9\ lb$ — Reaction from Lower Truck Apron

$\sigma_{gravity} := \dfrac{DL - (0.7\ E_v)}{1\ ft \cdot W} = 443.7\ psf$

$M_{OT} := M_{OTSOIL} + 0.7\ M_{OTEQ} = 720.2\ lb \cdot ft$ — Overturning Moment

$M_{RES} := R \cdot h_{apron} = 573.6\ lb \cdot ft$ — Restoring Moment

$M_{OTnet} := M_{OT} - M_{RES} = 146.6\ lb \cdot ft$ Net Overturning Moment

$\sigma_{net} := \dfrac{M_{OTnet}}{S_x} = 55\ psf$ Maximum tension/compression in soil from overturning

$Check_3 := \text{if}\left(\sigma_{net} < \sigma_{gravity}, \text{“NO NET TENSION”}, \text{“NG”}\right) = \text{“NO NET TENSION”}$

$Check_4 := \text{if}\left(\sigma_{net} + \sigma_{gravity} < q_{allow}, \text{“OK”}, \text{“NG”}\right) = \text{“OK”}$

Check DL + 0.525E + 0.75LL Load Combination

$\sigma_{gravity} := \dfrac{DL - (0.525\ E_v)}{1\ ft \cdot W} = 458.6\ psf$

$M_{OT} := M_{OTSOIL} + 0.525\ M_{OTEQ} + 0.75\ M_{OTLL} = 1925.4\ lb \cdot ft$ Overturning Moment

$R := 0.525\ E_h + H_{soil} + 0.75\ H_{LL} = 882.9\ lbf$ Reaction from Lower Truck Apron

$M_{RES} := R \cdot h_{apron} = 1103.6\ lb \cdot ft$ Restoring Moment

$M_{OTnet} := M_{OT} - M_{RES} = 821.8\ lb \cdot ft$ Net Overturning Moment

$\sigma_{net} := \dfrac{M_{OTnet}}{S_x} = 308.2\ psf$ Maximum tension/compression in soil from overturning

$Check_5 := \text{if}\left(\sigma_{net} < \sigma_{gravity}, \text{“NO NET TENSION”}, \text{“NG”}\right) = \text{“NO NET TENSION”}$

$Check_6 := \text{if}\left(\sigma_{net} + \sigma_{gravity} < q_{allow}, \text{“OK”}, \text{“NG”}\right) = \text{“OK”}$

Check Alternate IBC Load Combination [0.9-(0.2)(Sds)]DL + E/1.4

$\sigma_{gravity} := \dfrac{(0.9 - (0.2\ S_{DS})) \cdot DL}{1\ ft \cdot W} = 368\ psf$

$R := \dfrac{E_h}{1.4} + 0.9\ H_{soil} = 434.5\ lb$ Reaction from Lower Truck Apron

$M_{OT} := 0.9\ M_{OTSOIL} + \dfrac{M_{OTEQ}}{1.4} = 688.5\ lb \cdot ft$ Overturning Moment

$M_{RES} := R \cdot h_{apron} = 543.1\ lb \cdot ft$ Restoring Moment

$M_{OTnet} := M_{OT} - M_{RES} = 145.4\ lb \cdot ft$ Net Overturning Moment

$\sigma_{net} := \dfrac{M_{OTnet}}{S_x} = 54.5\ psf$ Maximum tension/compression in soil from overturning

$Check_7 := \text{if}\left(\sigma_{net} < \sigma_{gravity}, \text{“NO NET TENSION”}, \text{“NG”}\right) = \text{“NO NET TENSION”}$

$Check_8 := \text{if}\left(\sigma_{net} + \sigma_{gravity} < q_{allow}, \text{“OK”}, \text{“NG”}\right) = \text{“OK”}$

Design Footing

1.6H + 1.6L

$Mu_1 := 1.6\ \left(M_{OTSOIL} + M_{OTLL}\right) = 3.4\ k \cdot ft$

$R_1 := 1.6\ \left(H_{LL} + H_{soil}\right) = 1448\ lbf$

$M_{RES1} := R_1 \cdot h_{apron} = 1.8\ k \cdot ft$

$Mu_{f1} := Mu_1 - M_{RES1} = 1.6\ k \cdot ft$

1.6H + 1.0Eh + 1.0L

$Mu_2 := 1.6 \cdot \left(M_{OTSOIL}\right) + M_{EQ1} + M_{EQ2} + M_{OTLL} = 2.8\ k \cdot ft$

$R_2 := 1.6 \cdot H_{soil} + E_{h1} + E_{h2} + H_{LL} = 1328.5\ lbf$

$M_{RES2} := R_2 \cdot h_{apron} = 1.7\ k \cdot ft$

$Mu_{f2} := Mu_2 - M_{RES2} = 1.2\ k \cdot ft$

$Mu_{ftg} := \max\left(Mu_{f1}, Mu_{f2}\right) = 1.6\ k \cdot ft$

$\sigma_{Mot} := \dfrac{Mu_{ftg}}{S_x} = 583.5\ psf$

$P_u := 1.2\ \left(\left(t \cdot ht_w + W \cdot d_{ftg}\right)\ 150\ pcf \cdot 1\ ft\right) + 1.0\ \left(ht_{drive} \cdot L_1 \cdot 1\ ft \cdot soil\right) = 2.2\ k$

$\sigma_{Pu} := \dfrac{P_u}{W \cdot 1\ ft} = 553.8\ psf$

$Mu_{ftg} := \left(\sigma_{Mot} + \sigma_{Pu}\right) \cdot 1\ ft \cdot \dfrac{\left(W - t - L_1\right)^2}{2} = 1.4\ k \cdot ft$

$A_{sftg} :=$ Reinforcement size: #5 $= 0.31\ in^2$

$sp_{ftg} := 12\ in$

$d := d_{ftg} - 3\ in - (1.5 \cdot bar_{diaftg}) = 8.1\ in$

Flexure (per 1' width) - ACI 318-14 13.2.6.4, 13.2.7.1

$$f(A_{sreq}) := A_{sreq} \cdot f_y \cdot \left(d_{ftg} - \frac{A_{sreq} \cdot f_y}{.85 \cdot f'c \cdot (1 \cdot ft) \cdot 2}\right) \cdot .9 - Mu_{ftg}$$

Find required area of steel

Guess Values

$A_{sreq} := 1\ in^2$

Constraints

$$A_{sreq} \cdot f_y \cdot \left(d_{ftg} - \frac{A_{sreq} \cdot f_y}{.85 \cdot f'c \cdot 1\ ft \cdot 2}\right) \cdot .9 - Mu_{ftg} = 0$$

Solver

$A_{sreq} := \text{Find}(A_{sreq})$ $\quad A_{sreq} = 0.03\ in^2$

Required area of steel for footing flexure

$A_{smin} := 0.0018 \cdot t \cdot (1\ ft) = 0.12\ in^2$

Min flexural reinforcement per ACI 318-14 8.6.1.1

$$A_s := \frac{A_{sftg} \cdot (1\ ft)}{sp_{ftg}} = 0.31\ in^2$$

Area of reinforcement provided per foot of width

$A_{sreq} := \max(A_{sreq}, A_{smin}) = 0.12\ in^2$

$$\frac{A_{sreq}}{A_s} = 0.4$$

$Flag_f := \text{if}(A_s \ge A_{sreq}, \text{“OK”}, \text{“NG!!”})$

$Flag_f = \text{“OK”}$

Design Wall

1.6H + 1.0Eh + 1.0L

$Mu_{wall} := 1.6 \cdot (M_{OTSOIL}) + M_{EQ1} + M_{EQ2} + M_{OTLL} = 2.8\ k \cdot ft$

$A_{sw} :=$ Reinforcement size: #5 $= 0.31\ in^2$

$sp := 12\ in$

$d_w := t - 1.5\ in - (1.5 \cdot bar_{diaw}) = 3.1\ in$

Flexure (per 1' width) - ACI 318-14 13.2.6.4, 13.2.7.1

$f(A_{sreq}) := A_{sreq} \cdot f_y \cdot \left(d_w - \dfrac{A_{sreq} \cdot f_y}{.85 \cdot f'c \cdot (1 \cdot ft) \cdot 2}\right) \cdot .9 - Mu_{wall}$ Find required area of steel

Guess Values

$A_{sreq} := 1\ in^2$

Constraints

$$A_{sreq} \cdot f_y \cdot \left(d_w - \frac{A_{sreq} \cdot f_y}{.85 \cdot f'c \cdot 1\ ft \cdot 2}\right) \cdot .9 - Mu_{wall} = 0$$

Solver

$A_{sreq} := \text{Find}(A_{sreq})$ $A_{sreq} = 0.22\ in^2$ Required area of steel for footing flexure

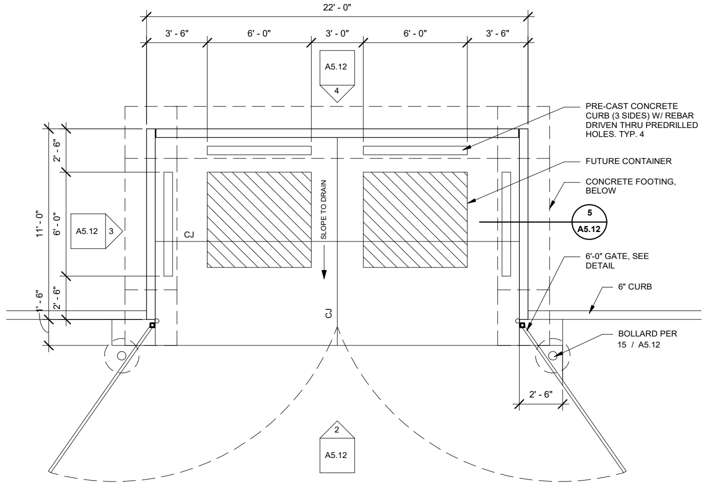
$A_{smin} := 0.0018 \cdot t \cdot (1\ ft) = 0.12\ in^2$ Min flexural reinforcement per ACI 318-14 8.6.1.1

$A_s := \dfrac{A_{sw} \cdot (1\ ft)}{sp} = 0.31\ in^2$ Area of reinforcement provided per foot of width

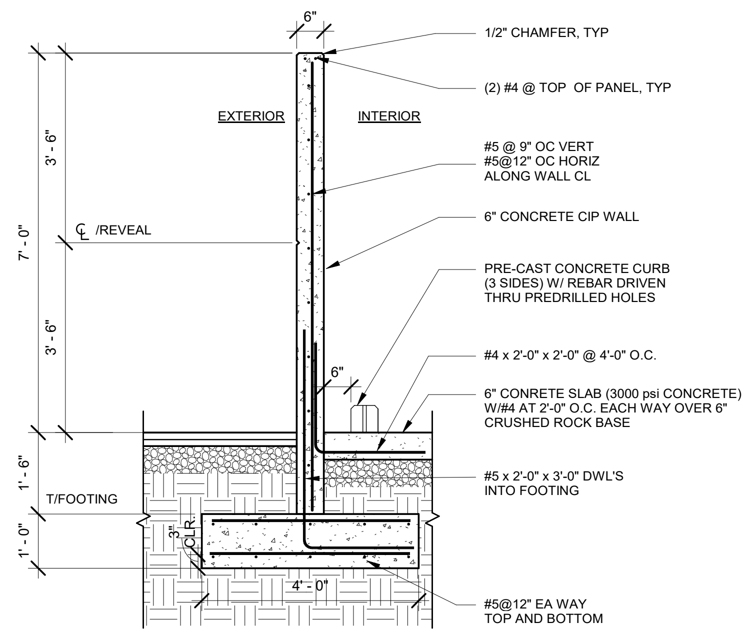
$A_{sreq} := \max(A_{sreq}, A_{smin}) = 0.22\ in^2$ $\dfrac{A_{sreq}}{A_s} = 0.71$

$Flag_f := \text{if}(A_s \ge A_{sreq}, \text{"OK"}, \text{"NG!!"})$ $Flag_f = \text{"OK"}$

TRASH ENCLOSURE DESIGN



1 A5.12 TRASH ENCLOSURE PLAN
1/4" = 1'-0"



5 A5.12 TRASH ENCLOSURE WALL
1/2" = 1'-0"

CONTROLLING LOCATION IS AT THE CORNER OF WALL WITH THE GATE

DETERMINE SEISMIC LOADS ON WALL AND GATE:

PER ASCE 7 TABLE 15.4-2, CALCULATE Cs USING 12.8

Nonbuilding Structure Type	Detailing Requirements[c]	R	Ω_0	C_d
Ground-supported cantilever walls or fences	Sec. 15.6.8	1.25	2	2.5

CONTROLLING LOCATION IS AT THE CORNER OF WALL WITH THE GATE

DETERMINE SEISMIC LOADS ON WALL AND GATE:

PER ASCE 7 TABLE 15.4-2, CALCULATE Cs USING 12.8

Cs = (Sds)/((R)(Ie)) = (0.843)/((1.25)/(1.0)) = 0.675

MIN Cs = (0.044)(Sds)(Ie) = (0.044)(0.843)(1.0) = 0.037 < 0.675, OK (S1 = 0.435 < 0.6)

V = (Cs)(W) = (0.675)(0.5')(150 PCF) = 50.6 PSF

CALCULATE SEISMIC EFFECT OF GATE SEPARATELY AS AN ARCHITECTURAL COMPONENT-APPENDAGES AND ORNAMENTATIONS

WEIGHT OF GATE, SAY 5 PSF

(10.5')(6')(5 PSF) = 315 LB

USE EQUATIONS IN CHAPTER 13

Architectural Component	a_p[a]	R_p	Ω_0[b]
Appendages and ornamentations	2½	2½	2

$$F_p = \frac{0.4 a_p S_{DS} W_p}{\left(\frac{R_p}{I_p}\right)}\left(1 + 2\frac{z}{h}\right) \quad (13.3\text{-}1)$$

F_p is not required to be taken as greater than

$$F_p = 1.6 S_{DS} I_p W_p \quad (13.3\text{-}2)$$

and F_p shall not be taken as less than

$$F_p = 0.3 S_{DS} I_p W_p \quad (13.3\text{-}3)$$

Fp = [((0.4)(2.5)(0.843)(315 LB))/((2.5)/(1.0))](1 + 2(0))

Fp = 106.3 LB, FORCE ACTS 4' ABOVE GROUND

OVERTURNING FROM SELF-WEIGHT OF GATE (DEAD LOAD COMPONENT)

M_{OT} = (315 LB)(10.5'/2) = 1654 LB-FT

OVERTURNING FROM SEISMIC FORCES ON WALL AND GATE

M_{OT} = (50.6 PSF)(7')(7'/2) + (106.3 LB)(4')
M_{OT} = 1239.7 LB-FT + 425.2 LB-FT = 1665 LB-FT

DETERMINE GOVERNING OOP MOMENT INTO WALL BASED ON LOAD COMBINATION

(1.2 + (0.2)(Sds))D + E

(1.2 + (0.2)(0.843))(D) + E

(1.37)(1654 LB-FT) + 1665 LB-FT = 3931 LB-FT

$$\rho = \frac{(0.85)(f'c)}{fy}\left[1 - \sqrt{1 - \frac{(4)(Mu)}{(1.7)(\phi)(f'c)(b)(d^2)}}\right]$$

$$\rho = \frac{(0.85)(4\ KSI)}{60\ KSI}\left[1 - \sqrt{1 - \frac{(4)(3931\ LB\text{-}FT)(12\ IN/FT)}{(1.7)(0.9)(4000\ PSI)(12\ IN)((2.75\ IN)^2)}}\right]$$

ρ = 0.008

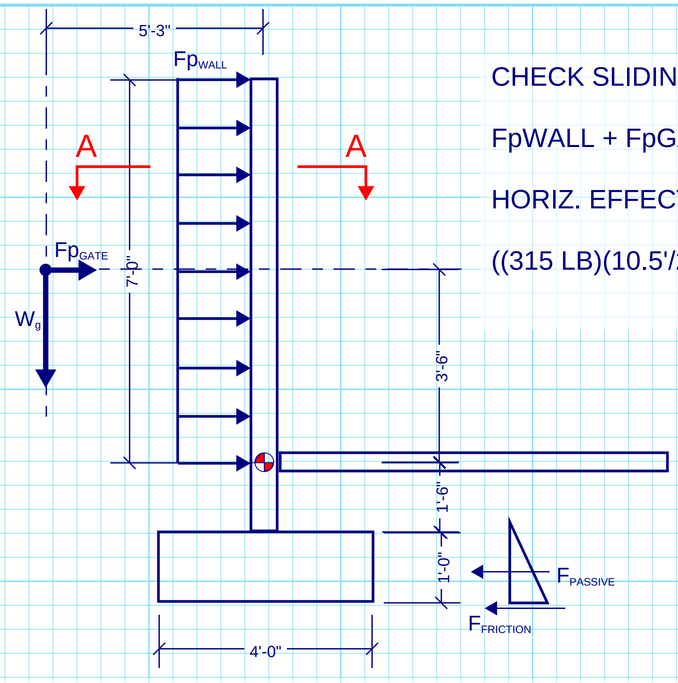
As = (0.0083)(12 IN)(2.75 IN) = 0.26 IN^2/FT

@5@12" OC VERT, OK

HORIZ REINF:

(12")(6")(0.0018) = 0.13 IN2/FT MIN FOR SHRINKAGE

USE #4@12" HORIZ CL

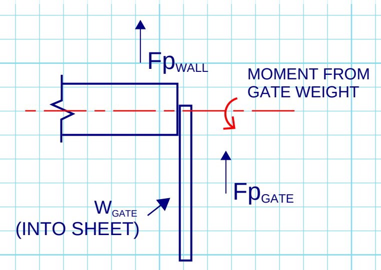


CHECK SLIDING, TOTAL OF LATERAL FORCES FROM SEISMIC:

FpWALL + FpGATE = (50.6 PSF)(7') + 106.3 LB = 354.2 LB + 106.3 LB = 460.5 LB

HORIZ. EFFECT OF GATE SELFWEIGHT (DL):

((315 LB)(10.5'/2))/(1.5'+1') = 661.5 LB



A-A (PART PLAN)

USING ALTERNATE IBC COMBINATION
(0.9-(0.2)(Sds))(D) + E/1.4

(0.9 - (0.2)(0.843))(661.5 LB) + (460.5 LB)/1.4 = 813 LB OF SLIDING

SAY FOOTING IS 12"x48", BOTT. OF FTG. 24" FROM BOTT. OF SOG; 2' OVERHANG FROM EDGE OF WALL

TOTAL WEIGHT = (8.5')(0.5')(150 PCF) + (1')(3')(4')(150 PCF) = 637.5 LB + 1800 LB = 2437 LB

TOTAL FRICTION = (2437 LB)(0.35) = 853 LB

For designing foundations to resist lateral loads, a base friction coefficient of 0.35 can be used. Passive earth pressures acting on the sides of the footings can also be considered. We recommend calculating this lateral resistance using an equivalent fluid weight of 300 pounds per cubic foot (pcf). We do not recommend including the upper 12 inches of soil in this computation because it can be affected by weather or disturbed by future grading activity. This value assumes the foundation will be constructed neat against competent native soil or backfilled with structural fill, as described in Section 4.2 of this report. The values recommended include a safety factor of 1.5.

4.6 Lateral Earth Pressures for Retaining Walls

The magnitude of earth pressure development on below-grade walls, such as basement or retaining walls, will partly depend on the quality of the wall backfill. We recommend placing and compacting wall backfill as structural fill as described in Section 4.2 of this report. To guard against hydrostatic pressure development, drainage must be installed behind the wall. A typical wall drainage detail is shown on Figure 4.

With wall backfill placed and compacted as recommended and drainage properly installed, unrestrained walls can be designed for an active earth pressure equivalent to a fluid weighing 35 pcf. For restrained walls, an additional uniform lateral pressure of 100 psf should be included. For evaluating the walls under seismic loading, a uniform earth pressure equivalent to 8H psf, where H is the height of the retained earth in feet, can be used. These values assume a horizontal backfill condition and that no other surcharge loading, such as traffic, sloping embankments, or adjacent buildings, will act on the wall. If such conditions exist, then the imposed loading must be included in the wall design.

Friction at the base of the wall foundation and passive earth pressure will provide resistance to these lateral loads. Values for these parameters are provided in Section 4.5.

FORCE FROM PASSIVE SOIL PRESSURE, 350 PSF FLUID PRESSURE PER GEOTECH

(2.5' - 1')(300 PSF) = 450 PSF

AREA OF TRIANGLE = (0.5)(1.5')(450 PSF) = 337.5 LB

PASSIVE PRESSURE FROM FOOTING OVERHANG

(0.5')(300 PSF) = 175 PSF

(.5)(175 PSF + 350 PSF)(1')(2') = 525 LB

337.5 LB + 525 LB = 862.5 LB FROM PASSIVE PRESSURE

(0.9 - (0.2)(0.843))(853 LB + 862.5 LB) = 1254.7 LB > 813 LB OF TOTAL SLIDING, OK

CHECK "INTERIOR" PORTION OF WALL AWAY FROM GATE W/ 3' FOOTING

W = (9.5')(0.5')(150 PCF) + (1')(3')(1')(150 PCF) = 1162.5 LB

USING ALTERNATE IBC COMBINATION
(0.9-(0.2)(Sds))(D) + E/1.4

(50.6 PSF)(8')/1.4 = 253 LB OF SLIDING

UNIT WEIGHT PER LINEAR FT:

(8.5')(.5')(150 PCF) + (4')(1')(150 PCF) = 1237 LB

(1237 LB)(0.35) = 433.1 LB

(0.9 + (0.2)(0.843))(433.1 LB) = 316.8 LB > 253 LB, OK

AT INTERIOR, FRICTION ALONE IS ADEQUATE TO RESIST SLIDING, OK

CHECK GLOBAL OVERTURNING:

ASD M_{OT} = (0.9 - (0.2)(0.843))(1654 LB-FT) + (1665 LB-FT)/1.4 = 2400 LB-FT

ASD RESTORING MOMENT = [(0.9 - (0.2)(0.843)](1237 LB + (2')(4')(150 PCF))

= (1782 LB)(2') = 3564 LB-FT > 2400 LB-FT, OK FROM WEIGHT ALONE

CHECK FOOTING FOR BEARING/REINFORCEMENT:

(4/3)(2500 PSF) = 3333 PSF SHORT TERM BEARING

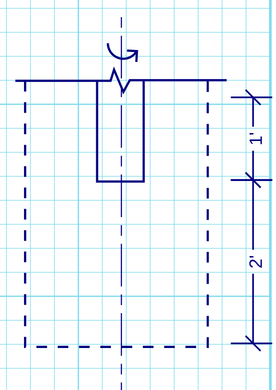
CONSIDER 1' END OF WALL + GATE AND FOOTING SECTION OVERHANG BELOW

ASD M_{OT} = (1.0 + (0.14)(0.843))(1654 LB-FT) + (0.7)(1665 LB-FT) = 3014.7 LB-FT

DL = (1.0 + (0.14)(0.843))(2437 LB) = 2724.6 LB

e = (3014.7 LB-FT)/2724.6 LB = 1.1' (OUTSIDE KERN = 0.67')

σ_{MAX} = 2P/[3(0.5L - e)(B)] = (2)(2724.6 LB)/[(3)((0.5)(4') - 1.1')(3')] = 672.7 PSF << 3333 PSF, OK



CONSIDER CANTILEVERED END OF FOOTING

USE MAXIMUM FACTORED SOIL PRESSURE (CONSERVATIVE)

(4/3)(25000 PSF)(1.6) = 5.33 KSF MAXIMUM ULTIMATE SOIL BEARING PRESSURE

Mu = (5.33 K-FT/FT)(2')(2'/2) = 10.7 K-FT/FT

$$\rho = \frac{(0.85)(3\text{ KSI})}{60\text{ KSI}}\left[1 - \sqrt{1 - \frac{(4)(10700\text{ LB-FT})(12\text{ IN/FT})}{(1.7)(0.9)(3000\text{ PSI})(12\text{ IN})((8\text{ IN})^2)}}\right]$$

ρ = 0.00322 (APPROX SAME AS MINIMAL FLEXURE, OK)

As = (0.00322)(12")(8") = 0.31 IN^2/FT

USE #5@12" T&B EACH WAY

CHECK SHEAR PER ACI318-19

Pu = (1.2 + (0.2)(0.843))(2437 LB) = 3336 LB

ULTIMATE M_{OT} = (1.2 + (0.2)(0.843))(1654 LB-FT) + 1665 LB-FT = 3929 LB-FT

e = 3929 LB-FT/3336 LB = 1.18'

σ_{MAX} = 2P/[3(0.5L - e)(B)] = (2)(3336 LB)/[(3)((0.5)(4') - 1.18')(3')] = 904 PSF (VERY SMALL, FTG SIZED FOR SLIDING)

Vu = (0.904 KSF)(2' - 0.67') = 1.2 K/FT

ρ_w = 0.31/(12")(8") = 0.00323

Vc = [8(λ_s)(λ)(ρ_w)sqrt(f'c)](b)(d) = [(8)(1.0)(1.0)(0.00323)$^{1/3}$sqrt(3000 PSI)](8")(12") = 6.2k (NO SHEAR REINF)

ϕVc = (0.6)(6.2k) = 3.7k >> 1.2k, OK