

CXT Inc. (Precast Division)

Calculations

Cortez CR-1598
Structural Analysis

Design Loads

400 psf Live Floor Load
250 psf Ground Snow Load
Wind Speed – 170 mph Exp. C
Seismic Design Category: B

Design Standards

2020 Florida Building Code (2018 IBC)
ASCE 7-16/ ACI 318-14

UL-752 Bullet Resistance
Classification: Level IV
Report #: 2012-647



PFS CORPORATION

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State:

Florida

Signature:

 *Mark Severson*

Title:

Staff Plan Reviewer

Date:

4/6/22

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April 4, 2022

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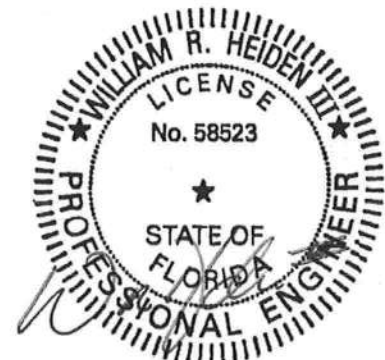
<u>Description</u>	<u>Page(s)</u>
<u>2018 International Building Code</u>	
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Appendix: (Provided Upon Request) UL-752 Bullet Resistance Testing

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All attached documents are for reference only and designed or approved by others.

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Main Wind Force Resisting System Loads (ASCE 7-16)

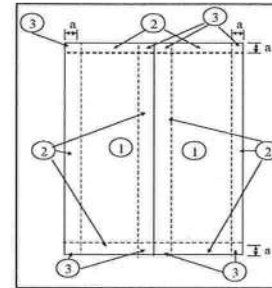
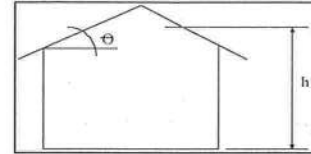
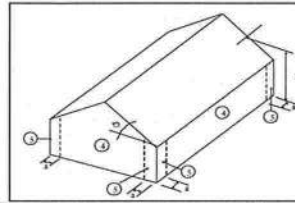
Cortez CR-1598		
Category	II	IBC TABLE 1604.5: Risk Category of Buildings and Other Structures.
Exposure	C	See § 26.7.3: Exposure Categories, General.
Velocity	170 mph	See Figure 26.5-1A thru 26.5-2D: Basic Wind Speed (3 second Gust)
h _{wind}	7.25 ft	Windward wall height
h _{lee}	7.25 ft	Leeward wall height
W _{building}	17 ft	Width of the building
L _{building}	9.42 ft	Length of the building
H _{building}	9.37 ft	Height of the building (to the ridge). Enter 0 if unknown.
Roof Rise	2.5625	Roof pitch (per foot)
θ	12.05 deg	Roof Angle
K _d	0.85	Wind directionality factor. 0.85 when using load combinations, 1.0 otherwise.
K ₁	0.00	
K ₂	0.00	
K ₃	0.00	See Figure 26.8-1: Multipliers for Obtaining Topographical Factor K _{zt}

K _{zt}	1	Topographic factor
h	8.310 ft	Mean roof height
n _s	9.03	Natural frequency
Flexibility	Rigid	Building flexibility
α	9.5	Terrain factor
z _g	900 ft	Terrain factor

Velocity Pressure Exposure Coefficient	
K _z (z)	0.849 at windward eave

Velocity Pressure (27.3.2)	
q _z	53.38 psf

Gable ☐ Type of Roof - Gable or Hip? ☐



Partially Enclosed if the building meets both of the following conditions:

1. Total area of openings in one wall exceeds area of openings in the balance of the building by more than 10%.
2. Total area of openings in one wall exceeds 4 sq. ft. or 1% of area of that wall and the total area of openings in the balance of the building does not exceed 20% of the area in the balance of the building.

Zone	Opening Area	Gross Area	A _{gi}	A _{oi}	Condition 1	Condition 2	Condition 3	Condition 4	Type:
Windward sidewall	0 sq ft	68.3 sq ft	511.0 sq ft	0 sq ft	0.00	0.00	0.00	0.00	Enclosed
Windward endwall	0 sq ft	141.3 sq ft	438.0 sq ft	0 sq ft	0.00	0.00	0.00	0.00	Enclosed
Leeward sidewall	0 sq ft	68.3 sq ft	511.0 sq ft	0 sq ft	0.00	0.00	0.00	0.00	Enclosed
Leeward endwall	0 sq ft	141.3 sq ft	438.0 sq ft	0 sq ft	0.00	0.00	0.00	0.00	Enclosed
Roof	0 sq ft	160.1 sq ft	419.1 sq ft	0 sq ft	0.00	0.00	0.00	0.00	Enclosed

Enclosed

External Pressure Coefficients	
C _{pe}	0.8 See 27.3.3 Roof Overhangs
C _p	0.8 Windward wall (Use with q _z) Fig. 27.3-1
	-0.339 Leeward wall (wind normal to ridge) (Use with q _h)
	-0.500 Leeward wall (wind parallel to ridge) (Use with q _h)
	-0.7 Sidewalls (Use with q _h) Fig. 27.4-1

L/B =	1.80
L/B =	0.53

Gust Factor - (26.9)	
G =	0.85

Internal Pressures:	
Negative:	-9.61 psf
Positive:	9.61 psf

Roof Pressure Coefficients (Fig 27.3-1) Normal to Ridge when Theta ≥ 10degrees	Pos. Windward	Neg. Windward	Leeward
	-0.177	-0.809	-0.495

Roof Pressure Coefficients (Fig 27.3-1) Normal to Ridge when Theta < 10 deg	0 to h/2	h/2 to h	h to 2h	> 2h
	-0.90	-0.90	-0.50	-0.30
Roof Pressure Coefficients (Fig 27.3-1) PARALLEL to Ridge	0 to h/2	h/2 to h	h to 2h	> 2h
	-1.21	-0.75	-0.65	-0.61

Wall Pressures:	w/ Negative	w/ Positive Internal
Windward	45.91 psf	26.69 psf
Leeward (wind normal)	-16.00 psf	-24.99 psf
Leeward (wind parallel)	-16.00 psf	-32.30 psf
Side Wall	-22.15 psf	-41.37 psf

Additional Overhang Pressure:	36.30 psf
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Roof Pressures: Wind Parallel to ridge for all roof slopes:	
Location	w/ Positive Internal
0 to h/2	-64.32 psf
h/2 to h	-43.51 psf
h to 2h	-39.23 psf
Over 2h	-37.09 psf

Roof Pressures Wind Perpendicular to Ridge w/ θ ≥ 10 deg	
w/ Negative Internal	1.59 psf
w/ Positive Internal	-46.31 psf
*WORST CASE LOADING	

Roof Pressures: Wind Perpendicular to ridge for θ < 10 deg:	
Location	w/ Positive Internal
0 to h/2	0.00 psf
h/2 to h	0.00 psf
h to 2h	0.00 psf
Over 2h	0.00 psf

Wind Speed:	170 mph	Roof Slope:	2.56 : 12	COMPONENTS & CLADDING			
Exposure:	C	Mean Roof Height:	8.31 ft				
Zone	10.0 sq ft		Effective Area 100.0 sq ft		500.0 sq ft		
1	-48.84 psf	25.89 psf	-43.51 psf	15.21 psf	-43.51 psf	15.21 psf	
2	-91.55 psf	25.89 psf	-64.86 psf	15.21 psf	-64.86 psf	15.21 psf	
2oh	-117.44 psf	-	-117.44 psf	-	-117.44 psf	-	
3	-139.60 psf	25.89 psf	-107.57 psf	15.21 psf	-107.57 psf	15.21 psf	
3oh	-197.52 psf	-	-133.46 psf	-	-133.46 psf	-	
4	-59.52 psf	52.58 psf	-48.84 psf	43.51 psf	-43.51 psf	36.57 psf	
5	-75.54 psf	52.58 psf	-59.52 psf	43.51 psf	-43.51 psf	36.57 psf	
a:	3.00 ft						

Higher pressures at the ridge line only applies to roof pitches > 7 degrees



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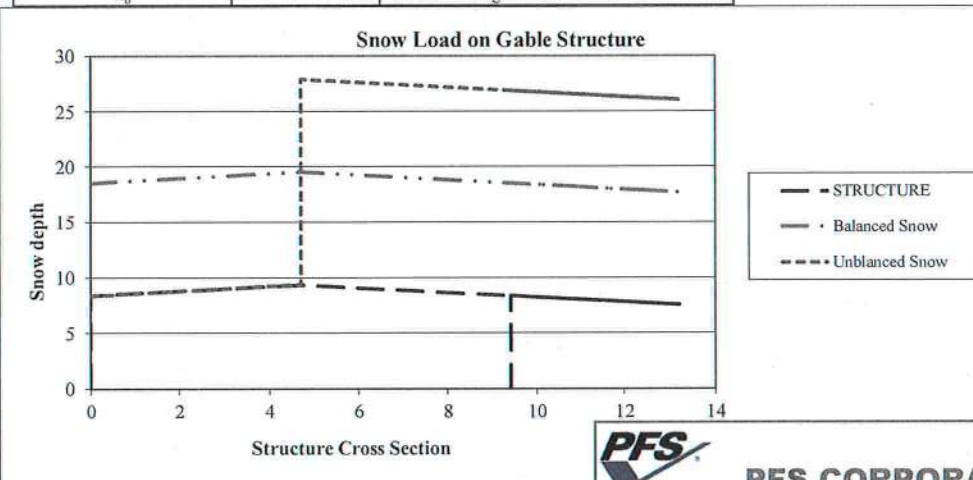
ASCE 7-16 SNOW LOAD CALCULATION

Category	II	IBC TABLE 1604.5: Risk Category of Buildings and Other Structures.
Exposure	C	See § 26.7.3: Exposure Categories, General.
P _g	250 psf	See ASCE Figure 7.2-1: Ground Snow Load
W. building	17 ft	Length of the building
L. building	9.42 ft	Width of the building
H. building	9.37 ft	Height of the building (to the ridge). Enter 0 if unknown.
Roof Rise (per foot)	2.5625	Roof pitch
9	12.05 deg	Roof Angle

ASCE Table 7.3-2 - Thermal Condition:	C _t
All structures except as indicated below:	1.0
Structures kept just above freezing and others with cold, ventilated roofs in which the thermal resistance (R-value) between the ventilated space and the heated space exceeds 25*h (deg*sq ft/BTU).	1.1
Unheated and open air structures	1.2
Structures intentionally kept below freezing	1.3
Continuously heated greenhouses with a roof having a thermal resistance value (R-value) less than 2.0*h (deg*sq ft/BTU).	0.85

C _t	1.2	(Choose from table above)
I _s	1	ASCE Table 1.5-2
Surface	Unobstructed	ASCE § 7.4
Roof type	Gable	
Hor. Eave to Ridge Distance - windward	8.5 ft	
Roof Exposure	Partially exposed	ASCE Table 7.3-1
C _e	1	ASCE Table 7.3-1
C _s	1	Slope Factor from Figure 7.4-1
Low Sloped?:	Yes	ASCE § 7.3.4
P _f	210.00 psf	Flat Roof Snow Load
P _s	210.00 psf	Sloped Roof Snow Load
Use unbalanced?	Yes	ASCE § 7.6.1
P _{windward}	0.00 psf	ASCE § 7.6.1
P _{leeward 1}	250.00 psf	ASCE § 7.6.1
P _{leeward 2}	250.00 psf	ASCE § 7.6.1
Distance from Ridge to Edge of P _{leeward 1} loading	8.5 ft	ASCE Figure 7.6-2

γ	30.00 pcf	Snow density	Eq. 7.7-1 of ASCE 7
S	4.682926829	Run per rise of 1	ASCE § 7.1
h _d	10.19 ft	Height of drifting snow on leeward side	
h _b	7.00 ft	Height of balanced snow	

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Seismic Loads (ASCE 7-16)

Cortez CR-1598			
Category	II	IBC TABLE 1604.5: Risk Category of Buildings and Other Structures.	
S_s	0.14 g	Max. Earthquake Ground Motion of 0.2 sec Spectral Response Acceleration	ASCE Figure 22-1
S_1	0.07 g	Max. Earthquake Ground Motion of 1.0 sec Spectral Response Acceleration	ASCE Figure 22-2
Site Class	D (Default)	Site classification (Use D if unknown unless jurisdiction, or geotechnical data determines Site Class E or F.)	ASCE 20.1
T_L	16.0 sec	Long Period Transition Period	ASCE Figure 22-14
Seismic Force Resisting System	A.5	Intermediate precast shear walls	ASCE Table 12.2-1
R	4.00	Response Modification Factor	
C_D	2.5	System Over strength Factor	
C_i	0.02	Approximate period parameter	ASCE Table 12.8-2
α	0.75	Approximate period parameter	ASCE Table 12.8-2
h_n	8.52 ft	Height in feet from base to highest level of structure	

			Value 1*	Value 2*	*=Used for interpolation
F_a	1.6	Interpolated Value	ASCE Table 11.4-1	1.6	1.6
F_v	2.4	Interpolated Value	ASCE Table 11.4-2	2.4	2.4

$S_{ms} = F_a * S_s$	0.216 g	Adjusted MCE Spectral Response Acceleration at short periods	ASCE 11.4-1
$S_{mi} = F_v * S_1$	0.163 g	Adjusted MCE Spectral Response Acceleration at 1 sec period	ASCE 11.4-2
(MCE = Maximum considered earthquake)			

$S_{DS} = 2/3 S_{ms}$	0.144 g	Design Spectral Acceleration Parameters	ASCE 11.4-3
$S_{DI} = 2/3 S_{mi}$	0.109 g	Design Spectral Acceleration Parameters	ASCE 11.4-4

I_E	1	Importance Factor	ASCE Table 1.5-2
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Seismic Design Category		B
Based on S_{DS}	A	Table 11.6-1
Based on S_{DI}	B	Table 11.6-2

Geotechnical Investigation Report Required?

No.

EQUIVALENT LATERAL FORCE PROCEDURE

$T_1 = C_t * h_n^x$	0.10 sec	Approximate fundamental period	ASCE 12.8-7
$T_1 = S_{DI} / S_{DS}$	0.76 sec		
T	0.10 sec	Fundamental period of the structure (can be taken as T_1 per ASCE 12.8.2)	

$C_s = S_{DS} / (R/I_E)$	0.036	ASCE 12.8-2
$C_{s, min}$	0.010	ASCE 12.8-5 & 12.8-6
$C_{s, max}$	0.273	ASCE 12.8-3 & 12.8-4
C_a	0.036	
k	1,000	ASCE 12.8.3
W	51.36 kip	
$V = C_s * W$	4.62 kip	ASCE 12.8-1
$M_o =$	38.8 k-ft	
$V = C_s * W$	3.89 kip	
$M_o =$	32.4 k-ft	

Shear with snow load
Overturning Moment with snow load
Shear without snow load
Overturning Moment without snow load

WITH SNOW LOAD

Level	Story Height	h_i or h_n	P_f (flat roof snow load)	w_i	$w_i * h_i^k$	12.8-12	12.8-11; 11.7	12.10-1
Roof	8.31 ft	8.52 ft	210 psf	30.66 kip	261.2 k-ft	0.984	4.55 kip	4.55 kip
Walls	0.00 ft	0.00 ft						0.0 k-ft
Floor	0.21 ft	0.21 ft		20.70 kip	4.3 k-ft	0.016	0.08 kip	4.62 kip
Base	0 ft	0.00 ft	$W =$	51.36 kip	265.5 k-ft			38.8 k-ft

WITHOUT SNOW LOAD

Level	Story Height	h_i or h_n	P_f (flat roof snow load)	w_i	$w_i * h_i^k$	12.8-12	12.8-11; 11.7	12.10-1
Roof	8.31 ft	8.52 ft	0 psf	22.49 kip	191.5 k-ft	0.978	3.80 kip	3.80 kip
Walls	0.00 ft	0.00 ft						0.0 k-ft
Floor	0.21 ft	0.21 ft		20.70 kip	4.3 k-ft	0.022	0.09 kip	3.89 kip
Base	0 ft	0.00 ft	$W =$	43.19 kip	195.8 k-ft			31.6 k-ft



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Center of Mass & Rigidity

Cortez CR-1598

Wall	Upper Left = 0.0		Lower Right		X		Y	
	X Relative		Y Relative		228		123	
	Stiffness	Stiffness	Stiffness	Stiffness	lbs	plf	Dist to CoRy	Dist to CoRx
W1	0.00%	51.18%	565	35	0.021	48.335		
W2	0.00%	48.82%	539	33	0.029	50.665		
W3	30.78%	0.00%	340	36	100.000	6.737		
W4	30.78%	0.00%	340	36	100.000	6.737		
W5	19.22%	0.00%	212	27	22.500	1.165		
W6	19.22%	0.00%	212	27	22.500	1.165		

Slab	Thickness	Weight	Left Edge		Top Edge		Right Edge		Bottom Edge		Snow/Live (psf)	Center of Gravity		Live w/snow	Live w/o snow
			X	Y	X	Y	X	Y	X	Y		X	Y		
F1	5	10035	12	5	216	123	400	114.0	64.0	10035	0				
R1	4.5	5910	0	0	114	123	210	57.0	61.5	10000	5910				
R2	4.5	5910	114	0	228	123	210	171.0	61.5	10000	5910				
Totals		21330						114.0	59.5						

Torsional Eccentricity		Wgt (w/snow)	Wgt (w/o snow)	roof	floor
ex	ey				
0.00	4.18	51,365	43,185	30,655	22,485
Center of Gravity					
X	Y				
114.0	59.5				
Center of Rigidity					
X	Y				
114.0	55.3				

Wall Overturning Checks Using Weight of Adjacent Walls
Force Transferred by Connections Between Walls

Wall	Anchorage Required to Resist Overturning From Design Moment (kip-ft)	Toward Lower Right Anchor Resistance		Toward Upper Left Anchor Resistance		Overturning status using just connection to adjacent walls
		Moment (kip-ft)	check	Moment (kip-ft)	check	
W1	-46.82	129.25	OK	129.25	OK	None Required
W2	-43.29	129.25	OK	129.25	OK	None Required
W3	-13.36	30.73	OK	38.64	OK	None Required
W4	-13.36	30.73	OK	38.64	OK	None Required
W5	-9.64	24.24	OK	24.24	OK	None Required
W6	-9.64	24.24	OK	24.24	OK	None Required

Overturning resistance considers only the weight of the wall, the weight of the roof supported by the wall, and connection to adjacent walls. Roof weight supported by other walls has not been considered. Connection to adjacent walls is taken as the connection capacity, not to exceed that portion of the adjacent wall weight that can be reasonably attributed to the connection.

Wall Overturning Checks Using Base Anchors Only

Wall	Design Moment (kip-ft)	Toward Lower Right Anchor Resistance		Toward Upper Left Anchor Resistance		Combined Loading Unity Check	Required Tension Capacity per Base Anchor (lb)
		Moment (kip-ft)	check	Moment (kip-ft)	check		
W1	-46.82	91.63	OK	91.63	OK	OK	(5390)
W2	-43.29	89.40	OK	89.40	OK	OK	(5282)
W3	-13.36	40.74	OK	39.55	OK	OK	(3049)
W4	-13.36	40.74	OK	39.55	OK	OK	(3049)
W5	-9.64	25.74	OK	25.74	OK	OK	(4290)
W6	-9.64	25.74	OK	25.74	OK	OK	(4290)

Wall Overturning Checks Using Base Anchors and Connection to Adjacent Walls

Wall	Base Anchor Shear Required (% Capacity)	Base Anchor Tension Available (% Capacity)	Available Overturning Resistance (kip-ft) From Base Anchors		Overturning Unity Check of Base Anchors	
			Lower Right	Upper Left	Lower Right	Upper Left
W1	3.9%	100.0%	220.88	220.88	OK	OK
W2	5.0%	100.0%	218.66	218.66	OK	OK
W3	2.7%	100.0%	71.47	78.19	OK	OK
W4	2.7%	100.0%	71.47	78.19	OK	OK
W5	3.1%	100.0%	49.98	49.98	OK	OK
W6	3.1%	100.0%	49.98	49.98	OK	OK



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ID#	Cortez CR-1598	
	DESIGN OF ROOF PANELS MARK RI, R2	
Material Properties		
Steel Reinforcement	4000 psi	
Joist	Plain WWF Grade 80	
Sp	80000 psi	
$f_{\text{splitwall}}^*$	no	
C_r (Concrete density)	150 pcf	
ρ (Steel)	2000000 psi	
n (modular ratio)	15.7	2.44
	15.7	18.4

Geometric Properties	10/25 ft
ls (overall length of slab)	9.79 ft
bs (overall width of slab)	Two-way slab
tf (total thickness)	0.375 in
bf (nominal thickness)	12 in
ls (section length)	4.5 m
bs (cover top)	0 in
bs (bottom reinforcement)	1 in
ds (nominal depth)	0.319 m
ds (effective depth)	0.526 m
ds (effective depth)	3.181 m
ds (effective depth and gtr for H ₂)	12 in
ds (effective depth and gtr for H ₃)	18 in
ds (effective depth and gtr for H ₄)	24 in
ds (effective depth and gtr for H ₅)	30 in
ds (effective depth and gtr for H ₆)	36 in
ds (effective depth and gtr for H ₇)	42 in
ds (effective depth and gtr for H ₈)	48 in
ds (effective depth and gtr for H ₉)	54 in
ds (effective depth and gtr for H ₁₀)	60 in
ds (effective depth and gtr for H ₁₁)	66 in
ds (effective depth and gtr for H ₁₂)	72 in
ds (effective depth and gtr for H ₁₃)	78 in
ds (effective depth and gtr for H ₁₄)	84 in
ds (effective depth and gtr for H ₁₅)	90 in
ds (effective depth and gtr for H ₁₆)	96 in
ds (effective depth and gtr for H ₁₇)	102 in
ds (effective depth and gtr for H ₁₈)	108 in
ds (effective depth and gtr for H ₁₉)	114 in
ds (effective depth and gtr for H ₂₀)	120 in
ds (effective depth and gtr for H ₂₁)	126 in
ds (effective depth and gtr for H ₂₂)	132 in
ds (effective depth and gtr for H ₂₃)	138 in
ds (effective depth and gtr for H ₂₄)	144 in
ds (effective depth and gtr for H ₂₅)	150 in
ds (effective depth and gtr for H ₂₆)	156 in
ds (effective depth and gtr for H ₂₇)	162 in
ds (effective depth and gtr for H ₂₈)	168 in
ds (effective depth and gtr for H ₂₉)	174 in
ds (effective depth and gtr for H ₃₀)	180 in
ds (effective depth and gtr for H ₃₁)	186 in
ds (effective depth and gtr for H ₃₂)	192 in
ds (effective depth and gtr for H ₃₃)	198 in
ds (effective depth and gtr for H ₃₄)	204 in
ds (effective depth and gtr for H ₃₅)	210 in
ds (effective depth and gtr for H ₃₆)	216 in
ds (effective depth and gtr for H ₃₇)	222 in
ds (effective depth and gtr for H ₃₈)	228 in
ds (effective depth and gtr for H ₃₉)	234 in
ds (effective depth and gtr for H ₄₀)	240 in
ds (effective depth and gtr for H ₄₁)	246 in
ds (effective depth and gtr for H ₄₂)	252 in
ds (effective depth and gtr for H ₄₃)	258 in
ds (effective depth and gtr for H ₄₄)	264 in
ds (effective depth and gtr for H ₄₅)	270 in
ds (effective depth and gtr for H ₄₆)	276 in
ds (effective depth and gtr for H ₄₇)	282 in
ds (effective depth and gtr for H ₄₈)	288 in
ds (effective depth and gtr for H ₄₉)	294 in
ds (effective depth and gtr for H ₅₀)	300 in
ds (effective depth and gtr for H ₅₁)	306 in
ds (effective depth and gtr for H ₅₂)	312 in
ds (effective depth and gtr for H ₅₃)	318 in
ds (effective depth and gtr for H ₅₄)	324 in
ds (effective depth and gtr for H ₅₅)	330 in
ds (effective depth and gtr for H ₅₆)	336 in
ds (effective depth and gtr for H ₅₇)	342 in
ds (effective depth and gtr for H ₅₈)	348 in
ds (effective depth and gtr for H ₅₉)	354 in
ds (effective depth and gtr for H ₆₀)	360 in
ds (effective depth and gtr for H ₆₁)	366 in
ds (effective depth and gtr for H ₆₂)	372 in
ds (effective depth and gtr for H ₆₃)	378 in
ds (effective depth and gtr for H ₆₄)	384 in
ds (effective depth and gtr for H ₆₅)	390 in
ds (effective depth and gtr for H ₆₆)	396 in
ds (effective depth and gtr for H ₆₇)	402 in
ds (effective depth and gtr for H ₆₈)	408 in
ds (effective depth and gtr for H ₆₉)	414 in
ds (effective depth and gtr for H ₇₀)	420 in
ds (effective depth and gtr for H ₇₁)	426 in
ds (effective depth and gtr for H ₇₂)	432 in
ds (effective depth and gtr for H ₇₃)	438 in
ds (effective depth and gtr for H ₇₄)	444 in
ds (effective depth and gtr for H ₇₅)	450 in
ds (effective depth and gtr for H ₇₆)	456 in
ds (effective depth and gtr for H ₇₇)	462 in
ds (effective depth and gtr for H ₇₈)	468 in
ds (effective depth and gtr for H ₇₉)	474 in
ds (effective depth and gtr for H ₈₀)	480 in
ds (effective depth and gtr for H ₈₁)	486 in
ds (effective depth and gtr for H ₈₂)	492 in
ds (effective depth and gtr for H ₈₃)	498 in
ds (effective depth and gtr for H ₈₄)	504 in
ds (effective depth and gtr for H ₈₅)	510 in
ds (effective depth and gtr for H ₈₆)	516 in
ds (effective depth and gtr for H ₈₇)	522 in
ds (effective depth and gtr for H ₈₈)	528 in
ds (effective depth and gtr for H ₈₉)	534 in
ds (effective depth and gtr for H ₉₀)	540 in
ds (effective depth and gtr for H ₉₁)	546 in
ds (effective depth and gtr for H ₉₂)	552 in
ds (effective depth and gtr for H ₉₃)	558 in
ds (effective depth and gtr for H ₉₄)	564 in
ds (effective depth and gtr for H ₉₅)	570 in
ds (effective depth and gtr for H ₉₆)	576 in
ds (effective depth and gtr for H ₉₇)	582 in
ds (effective depth and gtr for H ₉₈)	588 in
ds (effective depth and gtr for H ₉₉)	594 in
ds (effective depth and gtr for H ₁₀₀)	600 in
ds (effective depth and gtr for H ₁₀₁)	606 in
ds (effective depth and gtr for H ₁₀₂)	612 in
ds (effective depth and gtr for H ₁₀₃)	618 in
ds (effective depth and gtr for H ₁₀₄)	624 in
ds (effective depth and gtr for H ₁₀₅)	630 in
ds (effective depth and gtr for H ₁₀₆)	636 in
ds (effective depth and gtr for H ₁₀₇)	642 in
ds (effective depth and gtr for H ₁₀₈)	648 in
ds (effective depth and gtr for H ₁₀₉)	654 in
ds (effective depth and gtr for H ₁₁₀)	660 in
ds (effective depth and gtr for H ₁₁₁)	666 in
ds (effective depth and gtr for H ₁₁₂)	672 in
ds (effective depth and gtr for H ₁₁₃)	678 in
ds (effective depth and gtr for H ₁₁₄)	684 in
ds (effective depth and gtr for H ₁₁₅)	690 in
ds (effective depth and gtr for H ₁₁₆)	696 in
ds (effective depth and gtr for H ₁₁₇)	702 in
ds (effective depth and gtr for H ₁₁₈)	708 in
ds (effective depth and gtr for H ₁₁₉)	714 in
ds (effective depth and gtr for H ₁₂₀)	720 in
ds (effective depth and gtr for H ₁₂₁)	726 in
ds (effective depth and gtr for H ₁₂₂)	732 in
ds (effective depth and gtr for H ₁₂₃)	738 in
ds (effective depth and gtr for H ₁₂₄)	744 in
ds (effective depth and gtr for H ₁₂₅)	750 in
ds (effective depth and gtr for H ₁₂₆)	756 in
ds (effective depth and gtr for H ₁₂₇)	762 in
ds (effective depth and gtr for H ₁₂₈)	768 in
ds (effective depth and gtr for H ₁₂₉)	774 in
ds (effective depth and gtr for H ₁₃₀)	780 in
ds (effective depth and gtr for H ₁₃₁)	786 in
ds (effective depth and gtr for H ₁₃₂)	792 in
ds (effective depth and gtr for H ₁₃₃)	798 in
ds (effective depth and gtr for H ₁₃₄)	804 in
ds (effective depth and gtr for H ₁₃₅)	810 in
ds (effective depth and gtr for H ₁₃₆)	816 in
ds (effective depth and gtr for H ₁₃₇)	822 in
ds (effective depth and gtr for H ₁₃₈)	828 in
ds (effective depth and gtr for H ₁₃₉)	834 in
ds (effective depth and gtr for H ₁₄₀)	840 in
ds (effective depth and gtr for H ₁₄₁)	846 in
ds (effective depth and gtr for H ₁₄₂)	852 in
ds (effective depth and gtr for H ₁₄₃)	858 in
ds (effective depth and gtr for H ₁₄₄)	864 in
ds (effective depth and gtr for H ₁₄₅)	870 in
ds (effective depth and gtr for H ₁₄₆)	876 in
ds (effective depth and gtr for H ₁₄₇)	882 in
ds (effective depth and gtr for H ₁₄₈)	888 in
ds (effective depth and gtr for H ₁₄₉)	894 in
ds (effective depth and gtr for H ₁₅₀)	900 in
ds (effective depth and gtr for H ₁₅₁)	906 in
ds (effective depth and gtr for H ₁₅₂)	912 in
ds (effective depth and gtr for H ₁₅₃)	918 in
ds (effective depth and gtr for H ₁₅₄)	924 in
ds (effective depth and gtr for H ₁₅₅)	930 in
ds (effective depth and gtr for H ₁₅₆)	936 in
ds (effective depth and gtr for H ₁₅₇)	942 in
ds (effective depth and gtr for H ₁₅₈)	948 in
ds (effective depth and gtr for H ₁₅₉)	954 in
ds (effective depth and gtr for H ₁₆₀)	960 in
ds (effective depth and gtr for H ₁₆₁)	966 in
ds (effective depth and gtr for H ₁₆₂)	972 in
ds (effective depth and gtr for H ₁₆₃)	978 in
ds (effective depth and gtr for H ₁₆₄)	984 in
ds (effective depth and gtr for H ₁₆₅)	990 in
ds (effective depth and gtr for H ₁₆₆)	996 in
ds (effective depth and gtr for H ₁₆₇)	1002 in
ds (effective depth and gtr for H ₁₆₈)	1008 in
ds (effective depth and gtr for H ₁₆₉)	1014 in
ds (effective depth and gtr for H ₁₇₀)	1020 in
ds (effective depth and gtr for H ₁₇₁)	1026 in
ds (effective depth and gtr for H ₁₇₂)	1032 in
ds (effective depth and gtr for H ₁₇₃)	1038 in
ds (effective depth and gtr for H ₁₇₄)	1044 in
ds (effective depth and gtr for H ₁₇₅)	1050 in
ds (effective depth and gtr for H ₁₇₆)	1056 in
ds (effective depth and gtr for H ₁₇₇)	1062 in
ds (effective depth and gtr for H ₁₇₈)	1068 in
ds (effective depth and gtr for H ₁₇₉)	1074 in
ds (effective depth and gtr for H ₁₈₀)	1080 in
ds (effective depth and gtr for H ₁₈₁)	1086 in
ds (effective depth and gtr for H ₁₈₂)	1092 in
ds (effective depth and gtr for H ₁₈₃)	1098 in
ds (effective depth and gtr for H ₁₈₄)	1104 in
ds (effective depth and gtr for H ₁₈₅)	1110 in
ds (effective depth and gtr for H ₁₈₆)	1116 in
ds (effective depth and gtr for H ₁₈₇)	1122 in
ds (effective depth and gtr for H ₁₈₈)	1128 in
ds (effective depth and gtr for H ₁₈₉)	1134 in
ds (effective depth and gtr for H ₁₉₀)	1140 in
ds (effective depth and gtr for H ₁₉₁)	1146 in
ds (effective depth and gtr for H ₁₉₂)	1152 in
ds (effective depth and gtr for H ₁₉₃)	1158 in
ds (effective depth and gtr for H ₁₉₄)	1164 in
ds (effective depth and gtr for H ₁₉₅)	1170 in
ds (effective depth and gtr for H ₁₉₆)	1176 in
ds (effective depth and gtr for H ₁₉₇)	1182 in
ds (effective depth and gtr for H ₁₉₈)	1188 in
ds (effective depth and gtr for H ₁₉₉)	1194 in
ds (effective depth and gtr for H ₂₀₀)	1200 in
ds (effective depth and gtr for H ₂₀₁)	1206 in
ds (effective depth and gtr for H ₂₀₂)	1212 in
ds (effective depth and gtr for H ₂₀₃)	1218 in
ds (effective depth and gtr for H ₂₀₄)	1224 in
ds (effective depth and gtr for H ₂₀₅)	1230 in
ds (effective depth and gtr for H ₂₀₆)	1236 in
ds (effective depth and gtr for H ₂₀₇)	1242 in
ds (effective depth and gtr for H ₂₀₈)	1248 in
ds (effective depth and gtr for H ₂₀₉)	1254 in
ds (effective depth and gtr for H ₂₁₀)	1260 in
ds (effective depth and gtr for H ₂₁₁)	1266 in
ds (effective depth and gtr for H ₂₁₂)	1272 in
ds (effective depth and gtr for H ₂₁₃)	1278 in
ds (effective depth and gtr for H ₂₁₄)	1284 in
ds (effective depth and gtr for H ₂₁₅)	1290 in
ds (effective depth and gtr for H ₂₁₆)	1296 in
ds (effective depth and gtr for H ₂₁₇)	1302 in
ds (effective depth and gtr for H ₂₁₈)	1308 in
ds (effective depth and gtr for H ₂₁₉)	1314 in
ds (effective depth and gtr for H ₂₂₀)	1320 in
ds (effective depth and gtr for H ₂₂₁)	1326 in
ds (effective depth and gtr for H ₂₂₂)	1332 in
ds (effective depth and gtr for H ₂₂₃)	1338 in
ds (effective depth and gtr for H ₂₂₄)	1344 in
ds (effective depth and gtr for H ₂₂₅)	1350 in
ds (effective depth and gtr for H ₂₂₆)	1356 in
ds (effective depth and gtr for H ₂₂₇)	1362 in
ds (effective depth and gtr for H ₂₂₈)	1368 in
ds (effective depth and gtr for H ₂₂₉)	1374 in
ds (effective depth and gtr for H ₂₃₀)	1380 in
ds (effective depth and gtr for H ₂₃₁)	1386 in
ds (effective depth and gtr for H ₂₃₂)	1392 in
ds (effective depth and gtr for H ₂₃₃)	1398 in
ds (effective depth and gtr for H ₂₃₄)	1404 in
ds (effective depth and gtr for H ₂₃₅)	1410 in
ds (effective depth and gtr for H ₂₃₆)	1416 in
ds (effective depth and gtr for H ₂₃₇)	1422 in
ds (effective depth and gtr for H ₂₃₈)	1428 in
ds (effective depth and gtr for H ₂₃₉)	1434 in
ds (effective depth and gtr for H ₂₄₀)	1440 in
ds (effective depth and gtr for H ₂₄₁)	1446 in
ds (effective depth and gtr for H ₂₄₂)	1452 in
ds (effective depth and gtr for H ₂₄₃)	1458 in
ds (effective depth and gtr for H ₂₄₄)	1464 in
ds (effective depth and gtr for H ₂₄₅)	1470 in
ds (effective depth and gtr for H ₂₄₆)	1476 in
ds (effective depth and gtr for H ₂₄₇)	1482 in
ds (effective depth and gtr for H ₂₄₈)	1488 in
ds (effective depth and gtr for H ₂₄₉)	1494 in
ds (effective depth and gtr for H ₂₅₀)	1500 in
ds (effective depth and gtr for H ₂₅₁)	1506 in
ds (effective depth and gtr for H ₂₅₂)	1512 in
ds (effective depth and gtr for H ₂₅₃)	1518 in
ds (effective depth and gtr for H ₂₅₄)	1524 in
ds (effective depth and gtr for H ₂₅₅)	1530 in
ds (effective depth and gtr for H ₂₅₆)	1536 in
ds (effective depth and gtr for H ₂₅₇)	1542 in
ds (effective depth and gtr for H ₂₅₈)	1548 in
ds (effective depth and gtr for H ₂₅₉)	1554 in
ds (effective depth and gtr for H ₂₆₀)	1560 in
ds (effective depth and gtr for H ₂₆₁)	1566 in
ds (effective depth and gtr for H ₂₆₂)	1572 in
ds (effective depth and gtr for H ₂₆₃)	1578 in
ds (effective depth and gtr for H ₂₆₄)	1584 in
ds (effective depth and gtr for H ₂₆₅)	1590 in
ds (effective depth and gtr for H ₂₆₆)	1596 in
ds (effective depth and gtr for H ₂₆₇)	1602 in
ds (effective depth and gtr for H ₂₆₈)	1608 in
ds (effective depth and gtr for H ₂₆₉)	1614 in
ds (effective depth and gtr for H ₂₇₀)	1620 in
ds (effective depth and gtr for H ₂₇₁)	1626 in
ds (effective depth and gtr for H ₂₇₂)	1632 in
ds (effective depth and gtr for H ₂₇₃)	1638 in
ds (effective depth and gtr for H ₂₇₄)	1644 in
ds (effective depth and gtr for H ₂₇₅)	1650 in
ds (effective depth and gtr for H ₂₇₆)	1656 in
ds (effective depth and gtr for H ₂₇₇)	1662 in
ds (effective depth and gtr for H ₂₇₈)	1668 in
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ds (effective depth and gtr for H ₂₈₀)	1680 in
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ds (effective depth and gtr for H ₂₈₄)	1704 in
ds (effective depth and gtr for H ₂₈₅)	1710 in
ds (effective depth and gtr for H ₂₈₆)	1716 in
ds (effective depth and gtr for H ₂₈₇)	1722 in
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ds (effective depth and gtr for H ₂₉₂)	1752 in
ds (effective depth and gtr for H ₂₉₃)	1758 in
ds (effective depth and gtr for H ₂₉₄)	1764 in
ds (effective depth and gtr for H ₂₉₅)	1770 in
ds (effective depth and gtr for H ₂₉₆)	1776 in
ds (effective depth and gtr for H ₂₉₇)	1782 in
ds (effective depth and gtr for H ₂₉₈)	1788 in
ds (effective depth and gtr for H ₂₉₉)	1794 in
ds (effective depth and gtr for H ₃₀₀)	1800 in
ds (effective depth and gtr for H ₃₀₁)	1806 in
ds (effective depth and gtr for H ₃₀₂)	1812 in
ds (effective depth and gtr for H ₃₀₃)	1818 in
ds (effective depth and gtr for H ₃₀₄)	1824 in
ds (effective depth and gtr for H ₃₀₅)	1830 in
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ds (effective depth and gtr for H ₃₀₇)	1842 in
ds (effective depth and gtr for H ₃₀₈)	1848 in
ds (effective depth and gtr for H ₃₀₉)	1854 in
ds (effective depth and gtr for H ₃₁₀)	1860 in
ds (effective depth and gtr for H ₃₁₁)	1866 in
ds (effective depth and gtr for H ₃₁₂)	1872 in
ds (effective depth and gtr for H ₃₁₃)	1878 in
ds (effective depth and gtr for H ₃₁₄)	1884 in
ds (effective depth and gtr for H ₃₁₅)	1890 in
ds (effective depth and gtr for H ₃₁₆)	1896 in
ds (effective depth and gtr for H ₃₁₇)	1902 in
ds (effective depth and gtr for H ₃₁₈)	1908 in
ds (effective depth and gtr for H ₃₁₉)	1914 in
ds (effective depth and gtr for H ₃₂₀)	1920 in
ds (effective depth and gtr for H ₃₂₁)	1926 in
ds (effective depth and gtr for H ₃₂₂)	1932 in
ds (effective depth and gtr for H ₃₂₃)	1938 in
ds (effective depth and gtr for H ₃₂₄)	1944 in
ds (effective depth and gtr for H ₃₂₅)	1950 in
ds (effective depth and gtr for H ₃₂₆)	1956 in
ds (effective depth and gtr for H ₃₂₇)	1962 in
ds (effective depth and gtr for H ₃₂₈)	1968 in
ds (effective depth and gtr for H ₃₂₉)	1974 in
ds (effective depth and gtr for H ₃₃₀)	1980 in
ds (effective depth and gtr for H ₃₃₁)	1986 in
ds (effective depth and gtr for H ₃₃₂)	1992 in
ds (effective depth and gtr for H ₃₃₃)	1998 in
ds (effective depth and gtr for H ₃₃₄)	2004 in
ds (effective depth and gtr for H ₃₃₅)	2010 in
ds (effective depth and gtr for H ₃₃₆)	2016 in
ds (effective depth and gtr for H ₃₃₇	

Reinforcement Limits	
P_t (maximum tensile reinforcement)	0.0166
P_{max} (min. temperature reinforcement)	0.0014
P_{min} (minimum tensile reinforcement)	0.0027

Design Loads	w	
	Pressure on Slab	
D (Dead Load)	60/338 psf	
S (Snow Load)	250 psf	
L (Live Load)	0 psf	
Lr (Live Roof Load)	30 psf	
W (Wind Load)	139.6 psf	

Sustained Loading	
Pressure on slab	W
D (Dead load)	60.938 kN/m ²
S (Snow load)	250 kN/m ²

Notes:

β (fracture modulus)	520 J m ⁻²	0.7 ± 0.2, 3
β (fracture toughness)	91 MPa m ^{1/2}	0.7 ± 0.2, 3
β (fracture energy)	54 J m ⁻²	0.7 ± 0.2, 3
β (fracture energy)	2.25 mJ m ⁻²	0.7 ± 0.2, 3
β (fracture energy)	1.48	0.7 ± 0.2, 3
β (fracture energy)	0.8	0.7 ± 0.2, 3
β (fracture energy)	180	0.7 ± 0.2, 3
β (fracture energy)	480	0.7 ± 0.2, 3
β (fracture energy)	8.830	0.7 ± 0.2, 3
β (fracture energy)	0.743 m	0.7 ± 0.2, 3
β (fracture energy)	9.72 m ³	0.7 ± 0.2, 3
β (fracture energy)	0 m	0.7 ± 0.2, 3
β (fracture energy)	0 m	0.7 ± 0.2, 3

	P_{penalty} (reinforcement ratio provided)	m	0.0053
bottom mesh			
top mesh			
	P_{penalty} (reinforcement ratio provided)	m	0.0048 per
both layers			0.0000
	P_{penalty} (reinforcement ratio provided)	m	0 per
	P_{penalty} (reinforcement ratio provided)	m	0.0100
	P_{penalty} (reinforcement ratio provided)	m	0.0050

Wire Mesh (Top)	
Wire Size	Note
spacing	
Mesh Area	0.00 m ²

Factored Design Loads		
Factored loading per ACI equation indicated	Factored Pressure on Slab W'	Pressure on Section
4.72 kN/m^2	584.80 kN/m^2	0.29 MPa
	$W' = L + B' + L' + B'$	$W' = B' + B' + L + L'$
	$W' = 1.4 + 1.4 + 1.4 + 1.4$	$W' = 1.4 + 1.4 + 1.4 + 1.4$
	$W' = 5.6$	$W' = 5.6$
	$W' = 584.80 \text{ kN/m}^2$	$W' = 584.80 \text{ kN/m}^2$
	$W' = 0.29 \text{ MPa}$	$W' = 0.29 \text{ MPa}$

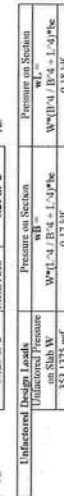
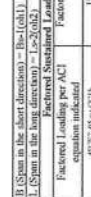
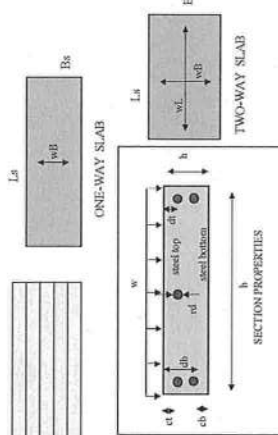
B (Span in the short direction) = $E_{\text{eff}} I_{\text{eff}} / (ohL)$
L (Span in the long direction) = $1.5 \cdot 2 / (ohL)$

Unfactored Design Loads	Pressure on Section on Slab W	Pressure on Section on Slab W	Pressure on Section on Slab W
Unfactored Pressure	$w_u(1.4 + B/4 + 1.4) \text{ kN/m}^2$	$w_u(1.4 + B/4 + 1.4) \text{ kN/m}^2$	$w_u(1.4 + B/4 + 1.4) \text{ kN/m}^2$
3.2.3 (13.2) mm ²	3.2.3 (13.2) mm ²	3.2.3 (13.2) mm ²	3.2.3 (13.2) mm ²

355,127 g	0.77 kN	8.790 N	0.83 kN	0.83 kg	0.92 kN	8.58333	0.92 kN
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SUMMARY

Use 1 Layer of Wire Mesh on Bottom



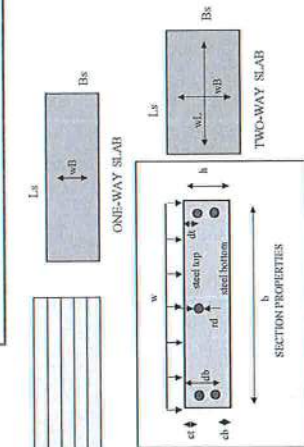
PFS CORPORATION
Approval Limited to Factory Built Portion Only

State: Florida

Signature: *Mark Severson*

Title: Staff Plan Reviewer

Date: 4/6/22



Franchise model(s)	430.1 mil	467.032.5
As - (24%)	91,125 m ² d	
As - (24%)	54 m ² d	
Vr - 16.2	2,15 m ² d	467.032.5
Mar	21.48	467.032.5
Item 1	0.8	
della unit	180	
della ingegner	480	
il	8.850	
del	0.742 m	
1.er	9.72 m ² d	
le	0.12 m	

DESIGN OF ROOF PANELS MARK R1, R2	
Properties	
C	5000 psi
F_c	Plain WWT Grade 80
Steel Reinforcement	80000 psi
ly	No
Lightweight?	No
C_a (Concrete density)	150 pcf
E (Steel)	29000000 psi
E (Concrete)	42800000 psi
n (modular ratio)	8.75
	472.85.1
	87.14.0
Parameters	
L_x (overall length of slab)	10.25 ft
B_x (overall width of slab)	9.79 ft
Design will be performed as :	Two-way slab
hr (roof, finish, thickness)	0.375 m
b (section width)	12 in
h (section thickness)	4.5 in
er (cover top)	0 in
	typically 2 inch deep

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	Structural Plain Concrete per ACI 22.5			
	Mu	S Elastic Section Modulus	qb	qbM = qbS/1.5
Mavg (average Moment) = (wl ²)/8 (2) 1/2	0.140 kN-m	0.023 m ³	0.55	0.077 kN-m
Mmax (negative Moment) = -wl ² /12 (2) 1/2	0.140 kN-m	0.023 m ³	0.55	0.077 kN-m
Mmax (positive Moment) = wl ² /16 (2) 1/2	0.140 kN-m	0.023 m ³	0.55	0.077 kN-m
Flexural Moments for Is	Mu	Tensile Strain	qb	qbM = qbS/1.5
Mpos (positive Moment) = (wl ²)/8	2.855 kN-m	0.036	0.9	0.077 kN-m
Mneg (negative Moment) = -wl ² /12	2.855 kN-m	0.036	0.9	0.077 kN-m
Mneg (positive Moment) = wl ² /16	2.855 kN-m	0.036	0.9	0.077 kN-m

[illegible][illegible]

	2.855 kPa(-)	0.869 kPa(-)	33.4 m ² /s	91.13 m ² /s	0.031 m	0.053	0.0486	0.029 m	0.020 m	0.5722 m	0.2166 m
II	2.7 kPa(-)	0.869 kPa(-)	33.4 m ² /s	91.13 m ² /s	0.031 m	0.053	0.0486	0.029 m	0.020 m	0.5722 m	0.2166 m
I	2.855 kPa(-)	0.869 kPa(-)	29.78 m ² /s	91.13 m ² /s	0.050 m	0.053	0.0486	0.029 m	0.020 m	0.5722 m	0.2166 m

ID:	Cortez CR-1598
	DESIGN OF WALL MARKED W1

Notes	
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Material Properties	
f'_c	5000 psi
Steel Reinforcement	Plain WWF Grade 80
Fy wire mesh	80000 psi
Fy rebar	60000 pcf
Lightweight?	No
Concrete density	150 pcf
E (Steel)	2900000 psi
E (Concrete)	4290000 psi
n (modular ratio)	6.75

O.K.

ACI 14.8.1

Shear Parameters	
Φv	0.83
V_c	3.123 kip
ΦV_c	2.634 kip

ACI 9.3.2.3

ACI 11.3.1.1.6 11.2.2

ACI 11.3.1

Minimum Wall Reinforcement Requirements	
roc min vert	0.0012
roc min hor	0.002
Max Vertical spacing	18 in
Max Horizontal spacing	18 in

ACI 14.8.2

ACI 14.8.3

ACI 14.8.3

Loading	
Axial Design Loads (pressure from roof)	
D (Dead Load) = Ww (Wall weight)	110.94 psf
S (Snow Load)	250 psf
L (Live Load)	0 psf
Lr (Live Roof Load)	30 psf
W (Wind Load)	139.6 psf
E (Earthquake Load)	2.19 psf

Lateral Design Loads (pressure on wall)	
Dead Load (DL lat)	0 psf
Snow Load (SL lat)	0 psf
Live Load (LL lat)	0 psf
Live Roof Load (LLr lat)	0 psf
Wind Load (WL lat)	75.34 psf
Earthquake Load (EL lat)	1.8 psf

Factored Axially Applied Loads	
Factored Loading per ACI	ACI eq. 9-3
Factored Pressure on Roof Ww	384.803

Axial Pressure on Section	
P_u/B	1.93 kip

Assumption check	
P_u/Ag	40.208 psi
$0.05F'_c$	300 psi
Check ACI 14.8.2.6	O.K.

Unfactored Axially Applied Loads	
Unfactored Pressure on Roof wW	353.1375 psf

Axial Pressure on Section	
P_B	1.26 kip

Shear	
Factored Loading per ACI	ACI eq. 9-3
$V_u = w_u l (Bw - 2d_u) / 2$	0
ΦV_c	1.33
Check Shear ACI 11.5.5.1	O.K.

Allowable Capacity	
$I_p = (b^3 h^3) / 12$	64 in ⁴
$A_g = (b^2 h)$	48 in ²
$Y_t = h/2$	2
f_r (rupture modulus)	530,330 psi
Mer	16,971 kip-in
Beta 1	0.8
Tril Ast req'd	0.073 in ²
B	8.836162648
kd	0.542 in
l _{ei}	2.92 in ⁴
e_s	0.003
e_s	0.005
a	0.33483 psi
c	0.419 in
Asw	0.23 in ²
l _{er} deflection	3.61 in ⁴
l _e	64.00 in ⁴
delta	150
r_s (maximum tensile reinforcement)	0.0166
r_{temp} (min. temperature reinforcement)	0.0014
r_{min} (minimum tensile reinforcement)	0.0027
r_{tril} (trial reinforcement ratio bottom)	0.0033
$P_{balance}$ (reinforcement ratio provided)	0.0090

ACT's Alternate Design of Slender Walls	ACI 14.8
Assumptions from this methodology:	
Wall panel shall be simply supported, axially loaded, and subject to out-of-phase uniform lateral loading where maximum moments and deflections occur at mid-height of the wall.	ACI 14.8.2.1
The cross section is constant over the height of the wall panel.	ACI 14.8.2.2
The wall cross sections shall be tension controlled.	ACI 14.8.2.3
$\Phi M_n \geq M_{cr}$	ACI 14.8.2.4
Concentrated gravity loads are distributed over the wall length.	ACI 14.8.2.5
The vertical stress P_u/Ag at mid-height shall not exceed $0.06F'_c$.	ACI 14.8.2.6

Geometric Properties	
X Coordinate	16
Y Coordinate	7
Direction of Wall	X
Center of gravity X	114.021
Center of gravity Y	7.000
Wall Weight	5475.000 lbs
Central wall?	Yes
Wall that supports 2 roof panels?	Yes
kip (depth of opening on wall)	0 ft
H (height of wall)	97.4 in
Lh (length of wall)	16.333 ft
Analysis will be performed as	One-way slab
b (section width)	4 in
h (section thickness)	12 in
cl (cover top)	3 in
cb (cover bottom)	2 in
rd (assumed reinf. diameter)	0.319 in
dt (effective depth top)	1.84 in
db (effective depth bottom)	1.84 in
Cs (% of DL used for seismic)	0.036
Eccentricity - Axial Load	1 in
Is wall Split	No

Wire Mesh	
Wire Size	W6.7
spacing	4 in
Mesh Area	0.20 in ²

= As

Factored Laterally Applied Loads	
Factored Loading per ACI	ACI eq. 9-4
Factored Pressure on Wall Ww	120.86 psf

Lateral Pressure on Section	
$L_w = W_w (L/4 + H/4 + L/4)$	0 klf
$H_w = W_w (L/4 + H/4 + L/4)$	0.12 klf

Unfactored Laterally Applied Loads	
Unfactored Pressure on Wall wW	75.34 psf

Lateral Pressure on Section	
$L_w = W_w (L/4 + H/4 + L/4)$	0 klf
$H_w = W_w (L/4 + H/4 + L/4)$	0.08 klf

Deflection	
Service Loads	
Axial	1.26 kip
Lateral	0 klf
Allowed service deflection	0.45 in
M _{max}	0.630 kip-in
M _{min}	0.633 kip-in
D _u	0.002 in
Check deflection	O.K.

ACI 14.8.4

Flexure	
Assumption check	
Span	Hw
net Tensile Strain	0.010
Check ACI 14.8.2.3	Tension
M _{max}	1.070 kip-ft

ACI eq. (14-6)	
M _u	1.280 kip-ft
	0.000 kip-ft

ACI 9.3.3	
fb	0.9
$(M_n)_{trial} = \Phi A_s f_y (d_t - a/2)$	2.020 kip-ft
DM = M _{pos} - ΦM_n	0.000 kip-ft
As Add'l req'd	0.00 in ²
Additional reinf req'd	0.00 in ²
Add'l bar size	3
qty req'd	0
or spacing of	0
As add'l	0.000 kip-ft
As _{tr} = As + As add'l	0.20 in ²
$(M_n) = \Phi A_s f_y (d_t - a/2)$	2.016 kip-ft
Check $\Phi M_n > M_u$	O.K.
ϵ_s allowed	63.49%
	0.00%



PFS CORPORATION
Approval Limited to Factory Built Portion Only

State:

Florida

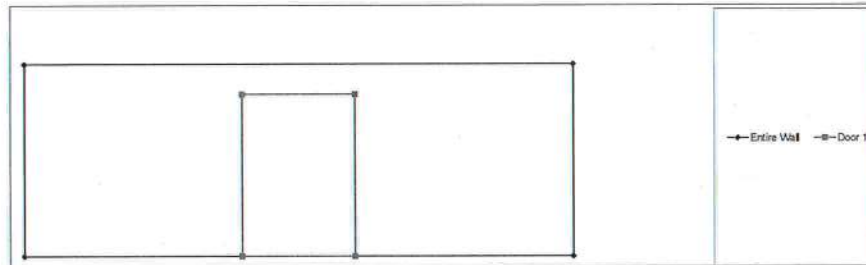
Signature:

Title:

Staff Plan Reviewer

Date:

4/6/22



REINFORCEMENT AT OPENINGS

Loading	
Po (factored load from roof)	0.31 klf
Ww (weight of panel per sq ft)	0.05 ksf

Material Properties	
dh (effective depth bottom)	1.84 in
a (block of strain)	0.33483 psi
$\phi = A_s \cdot f_y / (0.85 \cdot f_c \cdot b)$	

Factorized Moment							
Opening	Horizontal Location	Vertical Location	Length of opening	H height above opening	(-) Weight of Opening (LBS)	Pw total factorized panel load	Mu total factorized load (wu*L/2)/12
Door 1	6.49 ft	0 ft	3.34 ft	1.26 ft	1145.06	0.06 klf	0.37 klf

Reinure							
Opening	dh	As req'd	Bar size	qty req'd	$\phi M_n = \phi A_s F_y (d_h - a/2)$	Check $\phi M_n > M_u$	
Door 1	0.9	0.006 in ²	No. 3	1	6.69 kip-ft	O.K.	

CONNECTIONS

Full Resistance Value							
Base Anchors				Overturning			
Quantity	Maximum R - Distance	Maximum L - Distance	Maximum Shear	Moment + kip-ft	Moment - kip-ft	Moment + kip-ft	Moment - kip-ft
4	187	187	42,920	91.63	91.63	129.25	129.25

Total Tension							
Base Anchors				Base Anchors			
Base Anchor 1	Dist	Tension (kip)	Shear	Base Anchor 1	Dist	Moment +	Moment -
Base Anchor 1	9 in	3.56	9.25	Base Anchor 1	187 in	0.128 kip-ft	55.399 kip-ft
Base Anchor 2	59 in	3.64	12.21	Base Anchor 2	137 in	5.648 kip-ft	30.454 kip-ft
Base Anchor 3	137 in	3.64	12.21	Base Anchor 3	59 in	30.454 kip-ft	5.648 kip-ft
Base Anchor 4	187 in	3.56	9.25	Base Anchor 4	9 in	55.399 kip-ft	0.128 kip-ft

Wall Connections									
Quantity of Anchors	Capacity of each Anchor	Countering Dead Load from Adjacent Wall	% of wall to use	Adjoining Wall	Dist (inches)	L - Dist	Allowable Force	Overturning Moment Resistance (kip-ft)	Up Left / Low Right
Wall Connection 1	2	7,531	6.511	W3	0	196,000	3,062	0.000	50,013
Wall Connection 2	2	2,793	4.852	W5	75.5	120,500	4,852	30,524	48,717
Wall Connection 3	2	2,793	4.852	W6	120.5	75,500	4,852	48,717	30,524
Wall Connection 4	2	7,531	6.511	W4	196	0.000	3,062	50,013	0.000

Shear Connections at Base						Required Shear Capacity (lb) per Base Connector		Reserve Capacity
Design Force (lb)	Capacity (lb)	Reserve Capacity	Design (PLF)	Wall Shear Capacity Resistance (PLF)	check	415	(41261)	OK
1659	42920	41261	86	13406	OK			

RIGIDITY

CALCULATED VALUES							8.139612343
Pier Label	Length (inches)	Height (inches)	Fixed Top? (Y/N)	Useable? (Y/N)	Stiffness (k) (1000 kip / IN)	Deflection (in / 1000 kip)	
Entire Wall	196	97.4	Y	Y	12.395	0.081	
A	196	82.28	Y	Y	13.000	0.067	
A	77.88	82.28	Y	Y	4.599	0.217	
B	78.04	82.28	Y	Y	4.614	0.217	

Combine Logic						
First Segment	Second Segment	Re-Name	Combine/Subtract	Method	Combined	
Entire Wall	A	Aa	-	Deflection	0.014	
A	B	AB	+	Stiffness	9.213	
Aa	AB	Final	+	Deflection	0.123	



PFS CORPORATION

Approval Limited to Factory Built Portion Only

State:

Florida

Signature:

Mark Severson

Title:

Staff Plan Reviewer

Date:

4/6/22

ID:	Cortez CR-1598
	DESIGN OF WALL MARKED W2

Notes	
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Material Properties	
f_c	5000 psi
Steel Reinforcement	Plain WWF Grade 80
F_y wire mesh	80000 psi
F_y rebar	60000 psi
Lightweight?	No
Concrete density	150 pcf
E (Steel)	29000000 psi
E (Concrete)	4290000 psi
n (modular ratio)	6.76

Shear Parameters	
Φu_v	0.85
V_c	3.123 kip
$\Phi u_v V_c$	2.654 kip

Minimum Wall Reinforcement Requirements	
rise min. vert.	0.0012
rise min. hor.	0.002
Max Vertical spacing	18 in
Max Horizontal spacing	18 in

Loadings	
Axial Design Loads (pressure from roof)	Lateral Design Loads (pressure on wall)
D (Dead load) - Ww (Wall weight)	Dead Load (DL lat)
S (Snow Load)	Snow Load (SL lat)
L (Live Load)	Live Load (LL lat)
Lr (Live Roof Load)	Live Roof Load (LLr lat)
W (Wind Load)	Wind Load (WL lat)
E (Earthquake Load)	Earthquake Load (EL lat)

Factored Axially Applied Loads	
Factored Loading per ACI	ACI eq. 9-3
Factored Pressure on Roof Ww	584.805
Axial Pressure on Section	
P_u	1.88 kip
Assumption check	
P_u/Ag	39.167 psi
$0.06 f_c$	300 psi
Check ACI 14.8.2.6	O.K.

Unfactored Axially Applied Loads	
Unfactored Pressure on Roof wW	333.1375 psf
Axial Pressure on Section	
P	1.21 kip

Shear	
Factored Loading per ACI	ACI eq. 9-3
$V_u = wW \cdot (H_w - 2d_b) / 2$	0
$\Phi u_v V_c / 2$	1.33
Check Shear ACI 11.5.5.1	O.K.

Allowable Capacity	
$J_g = (b \cdot h^3) / 12$	64 in ⁴
$A_g = (b \cdot h)$	48 in ²
$Y_t = h/2$	2
f_r (rupture modulus)	530.330 psi
M_{cr}	16.971 kip-in
$Beta_1$	0.8
Trial Ast req'd	0.073 in ²
B	8.836162648
k_d	0.542 in
I_{cr}	2.92 in ⁴
e_s	0.003
e_c	0.005
a	0.33483 psi
c	0.419 in
A_{sc}	0.22 in ²
$I_{ndeflection}$	3.47 in ⁴
I_g	64.00 in ⁴
d_{ba}	150
ρ_t (maximum tensile reinforcement)	0.0166
ρ_{temp} (min. temperature reinforcement)	0.0014
ρ_{min} (minimum tensile reinforcement)	0.0027
ρ_{trd} (trial reinforcement ratio bottom)	0.0033
$\rho_{provided}$ (reinforcement ratio provided)	0.0090

ACT's Alternate Design of Slender Walls	ACT 14.8
Assumptions from this methodology:	
Wall panel shall be simply supported, axially loaded, and subject to out-of-plane uniform lateral loading where maximum moments and deflections occur at mid-height of the wall.	ACT 14.8.2.2
The cross section is constant over the height of the wall panel.	ACT 14.8.2.3
The wall cross sections shall be tension controlled.	ACT 14.8.2.4
Concentrated gravity loads are distributed over the wall length.	ACT 14.8.2.5
The vertical stress P_u/Ag at mid-height shall not exceed $0.06 f_c$.	ACT 14.8.2.6

Geometric Properties	
X Coordinate	16
Y Coordinate	106
Direction of Wall	X
Center of gravity X	113.971
Center of gravity Y	106.000
Wall Weight	4335.000 lbs
Central wall?	Yes
Wall that supports 2 roof panels?	Yes
kip (depth of opening in wall)	0 ft
H (height of wall)	97.4 in
Lh (length of wall)	16.333 ft
Analysis will be performed as	One-way slab
b (section width)	12 in
h (section thickness)	4 in
c1 (cover top)	2 in
c2 (cover bottom)	2 in
rd (assumed reinf. diameter)	0.319 in
dt (effective depth top)	1.84 in
db (effective depth bottom)	1.84 in
Cs (% of DL used for Seismic)	0.036
Eccentricity - Axial Load	1 in
Is wall Split	No

Wire Mesh	
Wire Size	W6.7
spacing	4 in
Mesh Area	0.20 in ²

- As

Factored Laterally Applied Loads	
Factored Loading per ACI	ACT eq. 9-4
Factored Pressure on Wall Ww	120.86 psf

Lateral Pressure on Section	
$L_w = W \cdot (L/4 + H/4 + L/4)$	0 klf
$H_w = W \cdot (L/4 + H/4 + L/4)$	0.12 klf

Unfactored Laterally Applied Loads	
Unfactored Pressure on Wall wW	75.54 psf

Lateral Pressure on Section	
$L_w = W \cdot (L/4 + H/4 + L/4)$	0 klf
$H_w = W \cdot (L/4 + H/4 + L/4)$	0.08 klf

Deflection	
Service Loads	
Axial	1.21 kip
Lateral	0 klf
Allowed service deflection	0.65 in
M_{sn}	0.605 kip-in
M	0.608 kip-in
Δ	0.002 in
Check deflection	O.K.

Flexure	
Assumption check	
Span	Hw
net Tensile Strain	0.010
Check ACI 14.8.2.3	Tension
M_{ua}	1.068 kip-ft

ACI eq. (14-6)	
M_u	1.280 kip-ft
	0.000 kip-ft

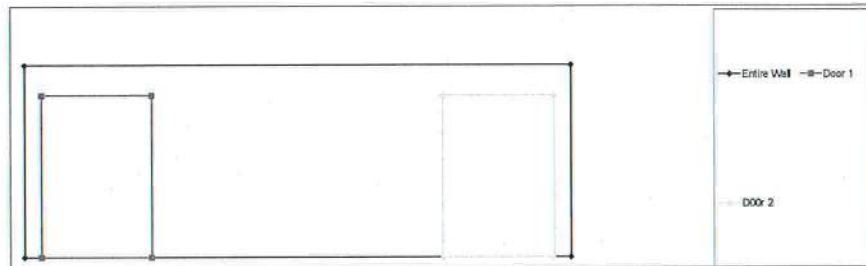
ACI 19.3.2	
ρ_b	0.9
$(M_n)_{trial} = \phi A_s f_y (d_t - a/2)$	2.020 kip-ft
$(M_n)_{Mpos} = \phi M$	0.000 kip-ft
A_s Add'l req'd	0.00 in ²
Additional reinf req'd	0.00 in ²
Add'l bar size	3
qty req'd	0
or spacing of	0
A_s add'l =	0.000 kip-ft
$A_{st} = A_s + A_s$ add'l	0.20 in ²
$(M_n) = \phi A_s f_y (d_b - a/2)$	2.016 kip-ft
Check $\phi M_n > M_u$	O.K.
ρ_s allowed	63.49%
	0.00%



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 Mark Severson
Staff Plan Reviewer
4/6/22



REINFORCEMENT AT OPENINGS

Loading	
Pu (factorized load from roof)	0.31 klf
Ww (weight of panel per sq ft)	0.05 ksf

Material Properties	
dh (effective depth bottom)	1.84 in
a (block of strain)	0.33483 psi
$\phi = A_s \cdot f_y / (0.85 \cdot f_c \cdot b)$	

Factorized Moment								
Opening	Horizontal Location	Vertical Location	L length of opening	H height above opening	(-) Weight of Opening (LBS)	Pu total factorized panel load	Ww total factorized load	Mu (wu*L ² /12)
Door 1	0.5 ft	0 ft	3.34 ft	1.26 ft	1145.06	0.06 klf	0.37 klf	0.34 kip-ft
Door 2	12.5 ft	0 ft	3.34 ft	1.26 ft	1145.06	0.06 klf	0.37 klf	0.34 kip-ft

Flexure							
Opening	ϕb	As req'd	Bar size	qty req'd	$\phi Mn = \phi As f_y (d - a/2)$	Check	$\phi Mn > Mu$
Door 1	0.9	0.006 in ²	No. 3	1	6.69 kip-ft	O.K.	
Door 2	0.9	0.006 in ²	No. 3	1	6.69 kip-ft	O.K.	

CONNECTIONS

Full Resistance Value							
Base Anchors				Overturning			
Quantity	Maximum	Maximum	Lateral	Shear	Moment +	Moment -	Wall-Wall Connection
4	R - Distance	L - Distance	31.065	85.40	89.40	129.25	129.25

Base Anchors							
Total Tension	Dist	Tension (kip)	Shear	L - Dist	Moment +	Moment -	
Base Anchor 1	3 in	3.28	3.23	193 in	0.013 kip-ft	54.410 kip-ft	
Base Anchor 2	59 in	3.64	12.21	137 in	5.473 kip-ft	29.507 kip-ft	
Base Anchor 3	137 in	3.64	12.21	59 in	29.507 kip-ft	5.473 kip-ft	
Base Anchor 4	193 in	3.28	3.23	3 in	54.410 kip-ft	0.013 kip-ft	

Wall Connections									
Quantity of Anchors	Capacity of each Anchor	Countering Dead Load from Adjoining Wall	% of wall to use	Adjoining Wall	Dist (inches)	L - Dist	Allowable Force	Overturing Moment Resistance (kip-ft)	
Wall Connection 1	2	7.537	53.98%	W3	0	196.000	3.062	0.000	50.013
Wall Connection 2	2	2.703	4.852	W3	75.5	120.500	4.852	30.524	48.717
Wall Connection 3	2	2.703	4.852	W6	120.5	75.500	4.852	48.717	30.524
Wall Connection 4	2	7.537	53.98%	W4	196	0.000	3.062	50.013	0.000

Wall Shear Checks						Required Shear Capacity (lb) per Base Connector	Reserve Capacity
Design Force (lb)	Capacity (lb)	Reserve Capacity	Design (PLF)	Wall Shear Capacity Resistance (PLF)	check		
1543	31065	29543	82	12789	OK	386	OK

RIGIDITY

CALCULATED VALUES							Final
Pier	Length (inches)	Height (inches)	Fixed Top?	Useable?	Stiffness (k)	Deflection (in / 1000 kip)	
Door 1	Entire Wall	196	97.4	Y	12.395	0.081	
	A'	196	82.28	Y	15.000	0.067	
	A	8	82.28	Y	0.000	0.000	
Door 2	B	149.92	82.28	Y	11.039	0.091	
	B'	196	82.28	Y	15.000	0.067	
	C	150	82.28	Y	11.046	0.091	
	D	5.92	82.28	Y	0.000	0.000	

Combine Logic					
First Segment	Second Segment	Re-Name	Combine/Subtract	Method	Combined
Door 1	Entire Wall	A	-	Deflection	0.014
		B	+	Stiffness	11.039
	A'	AB	+	Deflection	0.105
	Ab	B'	-	Deflection	0.038
Door 2	C	D	+	Stiffness	11.046
	B'a	CD	+	Deflection	0.128



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4/6/22

ID:	Cortez CR-1598
	DESIGN OF WALL MARKED W3

Notes	
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Material Properties	
F _c	5000 psi
Steel Reinforcement	Plain WWP Grade 80
F _y wire mesh	80000 psi
F _y rebar	60000 psi
Lightweight?	No
Concrete density	150 pcf
E (Steel)	29000000 psi
E (Concrete)	4290000 psi
n (modular ratio)	6.76

O.K.

ACI 8.3.1

Shear Parameters	
Phi v	0.85
V _c	3.123 kip
Phi*V _c	2.654 kip

ACI 9.3.2.3

ACI 11.3.1.6 11.2.1.2

ACI 11.1.1

Minimum Wall Reinforcement Requirements	
rho min vert	0.0012
rho min hor	0.002
Max Vertical spacing	18 in
Max Horizontal spacing	18 in

ACI 14.3.2

ACI 14.3.3

ACI 14.3.3

ACI 14.3.3

Loading	
Axial Design Loads (pressure on roof)	
D (Dead Load) = Ww (Wall weight)	110.94 psf
S (Snow Load)	250 psf
L (Live Load)	0 psf
Lr (Live Roof Load)	30 psf
W (Wind Load)	139.6 psf
E (Earthquake Load)	2.19 psf

ACI 14.3.2

ACI 14.3.3

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ACI 14.3.3

ACT's Alternate Design of Slender Walls	ACI 14.8
Assumptions from this methodology:	
Wall panel shall be simply supported, axially loaded, and subject to out-of-phase uniform lateral loading where maximum moments and deflections occur at mid-height of the wall.	ACI 14.8.2.1
The cross section is constant over the height of the wall panel.	ACI 14.8.2.2
The wall cross sections shall be tension controlled.	ACI 14.8.2.3
Phi*Mn >= Mcr	ACI 14.8.2.4
Concentrated gravity loads are distributed over the wall length.	ACI 14.8.2.5
The vertical stress Pu/Ag at mid-height shall not exceed 0.06*fc	ACI 14.8.2.6

Geometric Properties	
X Coordinate	14
Y Coordinate	5
Direction of Wall	Y
Center of gravity X	14.000
Center of gravity Y	62.073
Wall Weight	3225.000 lbs
Central wall?	Yes
Wall that supports 2 roof panels?	No
kip (depth of opening in wall)	0 ft
H (height of wall)	87 in
Lh (length of wall)	9.417 ft
Analysis will be performed as	Two-way slab
b (section width)	12 in
h (section thickness)	4 in
cl (cover top)	2 in
ch (cover bottom)	2 in
rd (assumed reinf. diameter)	0.319 in
dt (effective depth top)	1.84 in
db (effective depth bottom)	1.84 in
Cs (% of DL used for Seismic)	0.036
Eccentricity - Axial Load	1 in
Is wall Split?	No

Wire Mesh	
Wire Size	W6.7
spacing	4 in
Mesh Area	0.20 in ²

= As

Factored Axially Applied Loads	
Factored Loading per ACI	ACI eq. 9-3
Factored Pressure on Roof Ww	584.803
Axial Pressure on Section	
Pu/h	1.93 kip
Assumption check	
Pu/Ag	40.208 psi
0.06*Fc	300 psi
Check ACI 14.8.2.6	O.K.

Unfactored Axially Applied Loads	
Unfactored Pressure on Roof wW	353.1375 psf
Axial Pressure on Section	
Pu	1.26 kip

Shear	
Factored Loading per ACI	ACI eq. 9-3
Vu = wW*(Bw-2db)/2	0.09
Phi*Vc/2	1.33
Check Shear ACI 11.5.3.1	O.K.

Allowable Capacity	
Ig = (b*h ³)/12	64 in ⁴
Ag = (b*h)	48 in ²
Yt = h/2	2
fr (rupture modulus)	530.330 psi
Mcr	16.971 kip-in
Beta 1	0.8
Trial Ast req'd	0.073 in ²
B	8.836162648
kd	0.542 in
lcr	2.92 in ⁴
ec	0.003
ec	0.005
a	0.33483 psi
c	0.419 in
Asw	0.23 in ²
Icr (deflection)	3.60 in ⁴
Ic	64.00 in ⁴
delta	150
ts (maximum tensile reinforcement)	0.0166
tfmax (min. temperature reinforcement)	0.0014
tfmin (minimum tensile reinforcement)	0.0027
tfmax (trial reinforcement ratio bottom)	0.0033
P provided (reinforcement ratio provided)	0.0090

Factored Laterally Applied Loads	
Factored Loading per ACI	ACI eq. 9-4
Factored Pressure on Wall Ww	120.86 psf

Laterally Pressure on Section	
Lw = W*(L/4 + L/4 + H/4)	0.03 kip
Hw = W*(H/4 + H/4 + L/4)	0.09 kip

Unfactored Laterally Applied Loads	
Unfactored Pressure on Wall wW	75.54 psf

Laterally Pressure on Section	
Lw = W*(L/4 + L/4 + H/4)	0.02 kip
Hw = W*(H/4 + H/4 + L/4)	0.06 kip

Deflection	
Service Loads	
Axial	1.26 kip
Laterally	0.02 kip
Allowed service deflection	0.58 in
Max	2.207 kip-in
M	2.213 kip-in
Delta	0.006 in
Check deflection	O.K.

ACI 14.8.4

Flexure	Assumption check		
	Span	Hw	Lw
	net Tensile Strain	0.010	0.010
	Check ACI 14.8.2.3	Tension	Tension
	Max	0.670 Kip-ft	

ACI eq. (14-6)		
Mu	0.770 kip-ft	0.330 kip-ft

ACI 9.3.2		
fb	0.9	0.9
(Mn trial = $\phi As F_y (d - a/2)$)	2.020 kip-ft	2.020 kip-ft
DM = Mpos - ϕM	0.000 kip-ft	0.000 kip-ft
As Add'l req'd	0.00 in ²	0.00 in ²
Additional reinf req'd	0.00 in ²	0.00 in ²
Add'l bar size	3	3
spc req'd or spacing of	0	0
As add'l	0.000 kip-ft	0.000 kip-ft
Asl = As + As add'l	0.20 in ²	0.20 in ²
(Mn = $\phi As F_y (d - a/2)$)	2.016 kip-ft	2.016 kip-ft
Check $\phi M_n > M_u$	O.K.	O.K.
% allowed	38.19%	16.37%



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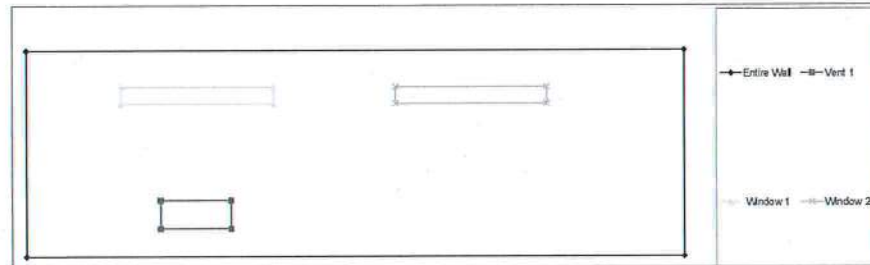
PFS Mark Severson

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Date:

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REINFORCEMENT AT OPENINGS

Loading	
Pu (factored load from roof)	0.31 klf
Ww (weight of panel per sq ft)	0.05 klf

Material Properties	
db (effective depth bottom)	1.84 in
a (block of steel)	0.33483 psi
$a = A_s \cdot f_y / (0.85 \cdot f'_c \cdot b)$	

Factored Moment								
Opening	Horizontal Location	Vertical Location	L length of opening	H height above opening	(-) Weight of Opening (LBS)	Pw total factored panel load	ww total factored load	Mu (wu*L/2y12)
Vent 1	1.92 ft	1 ft	1 ft	5.25 ft	50.00	0.36 klf	0.37 klf	0.05 kip-ft
Window 1	1.37 ft	5.38 ft	2.17 ft	1.29 ft	62.93	0.06 klf	0.37 klf	0.15 kip-ft
Window 2	5.29 ft	5.38 ft	2.17 ft	1.29 ft	62.93	0.06 klf	0.37 klf	0.15 kip-ft

Reinforce							
Opening	db	As req'd	Bar size	qty req'd	$\phi M_n = \phi A_s f_y (d - a/2)$	Check	$\phi M_n > M_u$
Vent 1	0.9	0 in ²	No. 3	0	0 kip-ft	O.K.	
Window 1	0.9	0.003 in ²	No. 3	1	6.86 kip-ft	O.K.	
Window 2	0.9	0.003 in ²	No. 3	1	6.86 kip-ft	O.K.	

CONNECTIONS

Base Anchors		Full Resistance Value			
		Overturning		Wall-Wall Connection	
Quantity	Maximum R - Distance	Maximum L - Distance	Shear kip	Moment + kip-ft	Moment - kip-ft
3	98	94	36.627	40.74	39.55

Total Tension		Base Anchors			
Quantity	Dist	Tension (kip)	Shear	L - Dist	Moment +
Base Anchor 1	19 in	3.94	12.21	94 in	1.118 kip-ft
Base Anchor 2	56.5 in	3.94	12.21	56.5 in	9.883 kip-ft
Base Anchor 3	98 in	3.94	12.21	15 in	20.735 kip-ft

Quantity of Anchors		Capacity of each Anchor	Countering Dead Load from Adjoining Wall	% of wall to use	Adjoining Wall	Dist (inches)	L - Dist	Allowable Force	Overturning Moment Resistance (kip-ft)
Wall Connection 1	2	2.703	3.525	19.26%	W1	2	111.000	3.826	0.638
Wall Connection 2	2	2.703	3.541	19.26%	W2	102	11.000	3.541	30.095

Shear Checks at Base		Wall Shear Capacity		Required Shear Capacity (lb) per Base Connector	
Design Force (lb)	Capacity (lb)	Design (PLF)	Resistance (PLF)	check	
994	35627	35633	90	OK	331

Reserve Capacity OK

RIGIDITY

CALCULATED VALUES		94%	Final
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6.830857562

Pier Label	Length (inches)	Height (inches)	Fixed Top? (Y/N)	Useable? (Y/N)	Stiffness (k) (1000 kip/in)	Deflection (in / 1000 kip)
Entire Wall	113	87	Y	Y	7.230	0.138
Vent 1	A'	113	Y	Y	62.543	0.016
	A	23.04	Y	Y	11.739	0.085
	B	77.96	Y	Y	42.872	0.023
Window 1	B'	113	Y	Y	108.101	0.009
	C	16.44	Y	Y	14.859	0.067
	D	70.52	Y	Y	67.329	0.015
Window 2	C'	113	Y	Y	108.101	0.009
	E	63.48	Y	Y	80.562	0.017
	F	23.48	Y	Y	21.850	0.048

First Segment		Second Segment		Combine Logic		Method		Combined	
Vent 1	Entire Wall	A'	Aa	-	-	Deflection	0.122	Stiffness	54.710
		A	Ab	+	+	Deflection	0.141	Stiffness	82.159
Window 1		B'	Bb	+	+	Deflection	0.144	Stiffness	82.412
		C	Cc	-	-	Deflection	0.134	Stiffness	82.412
Window 2		D	Dd	+	+	Deflection	0.148	Stiffness	82.412
		E	Ee	-	-	Deflection	0.148	Stiffness	82.412
		F	Ff	+	+	Deflection	0.148	Stiffness	82.412
			Final	+	+	Deflection	0.148	Stiffness	82.412



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PFS Mark Peterson

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Staff Plan Reviewer

Date:

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ID:	Cortez CR-1598
	DESIGN OF WALL MARKED W4

Notes	
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Material Properties	
f'_c	5000 psi
Steel Reinforcement	Plain WWP Grade 80
Fy wire mesh	80000 psi
Fy rebar	60000 psi
Lightweight?	No
Concrete density	150 pcf
E (Steel)	29000000 psi
E (Concrete)	4290000 psi
n (modular ratio)	6.76

O.K.

ACI 8.3.1

Shear Parameters	
Φv	0.85
V_c	3.123 kip
$\Phi v V_c$	2.654 kip

ACI 9.3.3

ACI 11.1.1.4, 11.2.1.2

ACI 11.1.1

Minimum Wall Reinforcement Requirements	
min. vert.	0.0012
min. hor.	0.002
Max Vertical spacing	18 in
Max Horizontal spacing	18 in

ACI 14.3.2

ACI 14.3.3

ACI 14.3.3

ACI 14.3.3

Loading	
Axial Design Loads (pressure from roof)	
D (Dead load) - Ww (Wall weight)	110.94 psf
S (Snow Load)	250 psf
L (Live Load)	0 psf
Lr (Live Roof Load)	30 psf
W (Wind Load)	139.6 psf
E (Earthquake Load)	2.19 psf
Lateral Design Loads (pressure on wall)	
Dead Load (DL lat)	0 psf
Snow Load (SL lat)	0 psf
Live Load (LL lat)	0 psf
Live Roof Load (LLr lat)	0 psf
Wind Load (WL lat)	75.54 psf
Earthquake Load (EL lat)	1.8 psf

Factored Axially Applied Loads	
Factored Loading per ACI	ACI eq. 9-3
Factored Pressure on Roof Ww	384.805

Axial Pressure on Section	
P_u	1.93 kip

Assumption check	
P_u/A_g	40.208 psi
$0.004 f'_c$	300 psi
Check ACI 14.3.2.6	O.K.

Unfactored Axially Applied Loads	
Unfactored Pressure on Roof wW	353.1375 psf

Axial Pressure on Section	
P	1.26 kip

Shear	
Factored Loading per ACI	ACI eq. 9-3
$V_u = w_u h (b_w - d_w) / 2$	0.09
$\Phi v V_u / 2$	1.33
Check Shear ACI 11.5.3.1	O.K.

Allowable Capacity	
$J_R = (b^3 h^3) / 12$	64 in ⁴
$A_g = (b^2 h)$	48 in ²
$Y_1 = h/2$	2
f_r (rupture modulus)	530.330 psi
M_u	16.921 kip-in
β_{eff}	0.8
Trial Ast req'd	0.073 in ²
I_R	8.836162648
$k d$	0.542 in
I_{cr}	2.92 in ⁴
a_s	0.003
a_s	0.005
a	0.3483 psi
c	0.419 in
As req'd	0.23 in ²
I_{cr} deflection	3.60 in ⁴
I_c	64.00 in ⁴
δ_{defl}	150
ϵ_s (maximum tensile reinforcement)	0.0166
ϵ_{temp} (min. temperature reinforcement)	0.0014
ϵ_{min} (minimum tensile reinforcement)	0.0027
ϵ_{min} (trial reinforcement ratio bottom)	0.0033
ϵ_{min} (reinforcement ratio provided)	0.0090

ACI's Alternate Design of Slender Walls	ACI 14.8
Assumptions from this methodology:	
Wall panel shall be simply supported, axially loaded, and subject to out-of-phase uniform lateral loading where maximum moments and deflections occur at mid-height of the wall.	ACI 14.8.2.1
The cross section is constant over the height of the wall panel.	ACI 14.8.2.2
The wall cross sections shall be tension controlled.	ACI 14.8.2.3
$\Phi M_u \geq M_{cr}$	ACI 14.8.2.4
Concentrated gravity loads are distributed over the wall length.	ACI 14.8.2.5
The vertical stress P_u/A_g at mid-height shall not exceed $0.06 f'_c$.	ACI 14.8.2.6

Geometric Properties	
X Coordinate	214
Y Coordinate	5
Direction of Wall	Y
Center of gravity X	214.000
Center of gravity Y	62.073
Wall Weight	3225.000 lbs
Central wall?	Yes
Wall that supports 2 roof panels?	No
top (depth of opening on wall)	0 ft
H (height of wall)	87 in
Lh (length of wall)	9.417 ft
Analysis will be performed as	Two-story slab
b (section width)	12 in
b (section thickness)	4 in
et (cover top)	2 in
eb (cover bottom)	2 in
rd (assumed reinf. diameter)	0.319 in
dt (effective depth top)	1.84 in
db (effective depth bottom)	1.84 in
C_u (% of I_R used for Seismic)	0.036
Eccentricity - Axial Load	1 in
Is wall Split	No

Wire Mesh	
Wire Size	W6.7
spacing	4 in
Mesh Area	0.20 in ²

= As

Factored Laterally Applied Loads	
Factored Loading per ACI	ACI eq. 9-4
Factored Pressure on Wall Ww	120.86 psf

Lateral Pressure on Section	
$L_w = W_u (L/4 + L/4 + H/4)$	0.03 klf
$H_w = W_u (H/4 + H/4 + L/4)$	0.09 klf

Unfactored Laterally Applied Loads	
Unfactored Pressure on Wall wW	75.54 psf

Lateral Pressure on Section	
$L_w = W_u (L/4 + L/4 + H/4)$	0.02 klf
$H_w = W_u (H/4 + H/4 + L/4)$	0.06 klf

Deflection	
Service Loads	
Axial	1.26 kip
Lateral	0.02 klf
Allowed service deflection	0.58 in
M_u	2.207 kip-in
M	2.315 kip-in
D_u	0.006 in
Check deflection	O.K.

ACI 14.8.4

Flexure			
Assumption check			
Spun	Hw	Lw	
net Tensile Strain	0.010	0.010	
Check ACI 14.8.2.3	Tension	Tension	
Mu	0.670 kip-ft		
ACI eq. (14-6)			
Mu	0.770 kip-ft	0.330 kip-ft	
ACI 9.3.2			
fb	0.9	0.9	
(Mu trial = φAsfy(d - a/2)	2.020 kip-ft	2.020 kip-ft	
DM = Mpos - φM	0.000 kip-ft	0.000 kip-ft	
As Add'l req'd	0.00 in ²	0.00 in ²	
Additional reinf req'd	0.00 in ²	0.00 in ²	
Add'l bar size	3	3	
qty req'd	0	0	
or spacing of	0	0	
As add'l =	0.000 kip-ft	0.000 kip-ft	
As = As + As add'l	0.20 in ²	0.20 in ²	
(Mu = φAsfy(d - a/2)	2.016 kip-ft	2.016 kip-ft	
Check φMu > Mu	O.K.	O.K.	
εs allowed	38.19%	16.37%	



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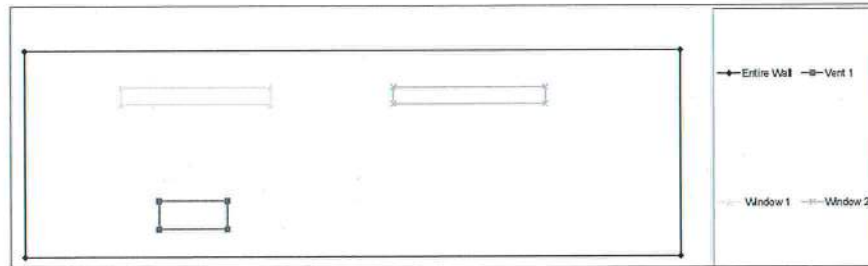
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4/6/22



REINFORCEMENT AT OPENINGS

Loading	
Pu (factorized load from roof)	0.31 klf
Ww (weight of panel per sq ft)	0.05 ksf

Material Properties	
db (effective depth bottom)	1.84 in
a (block of strain)	0.33483 psi
$a \cdot A_s \cdot f_y / (0.85 \cdot f_c \cdot b)$	

Factorized Moment								
Opening	Horizontal Location	Vertical Location	L length of opening	H height above opening	(-) Weight of Opening (LBS)	Pw total factorized panel load	wu total factorized load	Mu (wu*L ² /2)/12
Vent 1	1.92 ft	1 ft	1 ft	5.25 ft	59.00	0.26 klf	0.37 klf	0.05 kip-ft
Window 1	1.37 ft	5.38 ft	2.17 ft	1.29 ft	62.93	0.06 klf	0.37 klf	0.15 kip-ft
Window 2	5.29 ft	5.38 ft	2.17 ft	1.29 ft	62.93	0.06 klf	0.37 klf	0.15 kip-ft

Reinure						
Opening	db	As req'd	Bar size	qty req'd	$\phi M_n = \phi(A_s f_y d_b - a/2)$	Check $\phi M_n \sim M_u$
Vent 1	0.9	0 in ²	No. 3	0	0 kip-ft	O.K.
Window 1	0.9	0.003 in ²	No. 3	1	6.86 kip-ft	O.K.
Window 2	0.9	0.003 in ²	No. 3	1	6.86 kip-ft	O.K.

CONNECTIONS

Full Resistance Value									
Overturning									
Base Anchors			Lateral	Base Anchors		Wall-Wall Connection			
Quantity	Maximum	Maximum	Shear	Moment +	Moment -	Moment +	Moment -	Moment -	
in Shear	R - Distance	L - Distance	kip	kip - ft	kip - ft	kip - ft	kip - ft	kip - ft	
3	68	84	36.627	40.74	39.55	39.73	38.64		

Total Tension									
10.923	16	Tension (kip)	Shear	L - Dist	Moment +	Moment -			
Base Anchor 1	19 in	3.54	12.21	94 in	1.118 kip*ft	28.521 kip*ft			
Base Anchor 2	56.5 in	3.54	12.21	56.5 in	9.883 kip*ft	10.304 kip*ft			
Base Anchor 3	98 in	3.54	12.21	15 in	29.735 kip*ft	0.726 kip*ft			

Wall Connections									
Quantity of Anchors	Capacity of each Anchor	Countering Dead Load from Adjoining Wall	% of wall to use	Adjoining Wall	Dist (inches)	L - Dist	Allowable Force	Overturning Moment Resistance (kip-ft)	
								Up Left	Low Right
Wall Connection 1	2	2.703	3.626	19.26%	W1	2	111.000	3.826	35.384
Wall Connection 2	2	2.703	3.541	19.26%	W2	102	11.000	3.541	3.246

Wall Shear Checks									
Shear Connections at Base			Wall Shear Capacity		Required Shear Capacity (lb) per		Reserve Capacity		
Design Force (lb)	Capacity (lb)	Reserve Capacity (lb)	Design (PLF)	Resistance (PLF)	check	Base Connector	(35633)	OK	OK
994	35627	35633	90	19239	OK	331			

RIGIDITY

CALCULATED VALUES						
	94%	Final	6.830857562			
Pier	Length (inches)	Height (inches)	Fixed Top?	Useable?	Stiffness (k)	Deflection (in / 1000 kip)
Entire Wall	113	87	Y	Y	7.230	0.136
Vent 1	A	113	Y	Y	82.543	0.016
	A'	23.04	Y	Y	11.739	0.085
	B	77.96	Y	Y	42.972	0.023
Window 1	B'	113	Y	Y	108.101	0.009
	C	16.44	Y	Y	14.859	0.067
	D	70.52	Y	Y	67.329	0.015
Window 2	C'	113	Y	Y	108.101	0.009
	E	63.48	Y	Y	60.562	0.017
	F	23.48	Y	Y	21.850	0.046

Combine Logic						
First Segment	Second Segment	Re-Name	Combine/Subtract	Method	Combined	
Vent 1	Entire Wall	A'	-	Deflection	0.122	
		B	-	Stiffness	54.710	
		Aa	+	Deflection	0.141	
		Ab	-	Deflection	0.131	
		C	+	Stiffness	82.189	
		B'a	+	Deflection	0.144	
		B'b	-	Deflection	0.134	
Window 2		C'	+	Stiffness	82.412	
		C'a	+	Deflection	0.148	



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ID:	Cortez CR-1598
	DESIGN OF WALL MARKED W5

Notes	
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Material Properties	
F _c	5000 psi
Steel Reinforcement	Plain WWF Grade 80
F _y wire mesh	80000 psi
F _y rebar	60000 pcf
Lightweight?	No
Concrete density	150 pcf
E (Steel)	29000000 psi
E (Concrete)	4290000 psi
n (modular ratio)	6.76

O.K.

ACI 8.5.1

Shear Parameters	
Phi _v	0.85
V _e	2.274 kip
Phi*V _e	1.933 kip

ACI 8.5.2.2

ACI 11.3.1.4 11.2.1.2

ACI 11.1.1

Minimum Wall Reinforcement Requirements	
ρ _{min} vert	0.0012
ρ _{min} hor	0.002
Max Vertical spacing	18 in
Max Horizontal spacing	18 in

ACI 14.3.2

ACI 14.3.3

ACI 14.3.3

Loadings	
Axial Design Loads (pressure from roof)	Lateral Design Loads (pressure on wall)
D (Dead load) + W _w (Wall weight)	Dead Load (DL lat)
S (Snow Load)	Snow Load (SL lat)
L (Live Load)	Live Load (LL lat)
L _r (Live Roof Load)	Live Roof Load (LL _r lat)
W (Wind Load)	Wind Load (WL lat)
E (Earthquake Load)	Earthquake Load (EL lat)

Factored Axially Applied Loads	
Factored Loading per ACI	ACI eq. 9-3
Factored Pressure on Roof W _w	584.805

Axial Pressure on Section	
P _u /b	1.92 kip

Assumption check	
P _u /A _g	53.333 psi
0.0045 f _c	300 psi
Check ACI 14.8.2.4	O.K.

Unfactored Axially Applied Loads	
Unfactored Pressure on Roof w _w	353.1375 psf

Axial Pressure on Section	
P _b /b	1.25 kip

Shear	
Factored Loading per ACI	ACI eq. 9-3
V _u = w _w *(B _w -2b _w)/2	0.13
Phi*V _e /2	0.97
Check Shear ACI 11.5.5.1	O.K.

Allowable Capacity	
I _g = (b*h ³)/12	27 in ⁴
A _g = (b*h)	36 in ²
γ _t = h/2	1.5
f _r (rupture modulus)	530.330 psi
M _{cr}	9.545 kip-in
Beta ₁	0.8
Trial Ast req'd	0.053 in ²
B	8.836162648
k _d	0.449 in
I _{cr}	1.44 in ⁴
e _s	0.003
e _s	0.003
a	0.3483 psi
c	0.419 in
Ass	0.22 in ²
I _{cr} deflection	1.73 in ⁴
I _e	27.00 in ⁴
delta	150
ρ _t (maximum tensile reinforcement)	0.0166
ρ _{max} (min. temperature reinforcement)	0.0014
ρ _{min} (minimum tensile reinforcement)	0.0027
ρ _{max} (trial reinforcement ratio bottom)	0.0033
ρ _{provided} (reinforcement ratio provided)	0.0120

ACT's Alternate Design of Slender Walls	ACI 14.8
Assumptions from this methodology:	
Wall panel shall be simply supported, axially loaded, and subject to out-of-plane uniform lateral loading where maximum moments and deflections occur at mid-height of the wall.	ACI 14.8.2.1
The cross section is constant over the height of the wall panel.	ACI 14.8.2.2
The wall cross sections shall be tension controlled.	ACI 14.8.2.3
Phi*M _n >= M _{cr}	ACI 14.8.2.4
Concentrated gravity loads are distributed over the wall length.	ACI 14.8.2.5
The vertical stress P ₀ /A _g at mid-height shall not exceed 0.06*f _c	ACI 14.8.2.6

Geometric Properties	
X Coordinate	91.5
Y Coordinate	9
Direction of Wall	Y
Center of gravity X	91.500
Center of gravity Y	56.500
Wall Weight	2535.000 lbs
Central wall?	Yes
Wall does support 2 roof panels?	No
h _p (length of opening on wall)	0 ft
H (height of wall)	87 in
L _h (length of wall)	7.917 ft
Analysis will be performed as	Two-way slab
b (section width)	12 in
h (section thickness)	3 in
c _t (cover top)	1 1/2 in
c _b (cover bottom)	1 1/2 in
rd (assumed reinf. diameter)	0.319 in
d _t (effective depth top)	1.34 in
d _b (effective depth bottom)	1.34 in
C _u (% of DL used for Seismic)	0.036
Eccentricity - Axial Load	1 in
Is wall Split	No

Wire Mesh	
Wire Size	W6.7
spacing	4 in
Mesh Area	0.20 in ²

= As

Factored Laterally Applied Loads	
Factored Loading per ACI	ACI eq. 9-4
Factored Pressure on Wall W _w	120.86 psf

Lateral Pressure on Section	
L _w = W _w *(L/4 + L/4 + H/4)	0.05 klf
H _w = W _w *(H/4 + H/4 + L/4)	0.07 klf

Unfactored Laterally Applied Loads	
Unfactored Pressure on Wall w _w	75.54 psf

Lateral Pressure on Section	
L _w = W _w *(L/4 + L/4 + H/4)	0.03 klf
H _w = W _w *(H/4 + H/4 + L/4)	0.04 klf

Deflection	
Service Loads	
Axial	1.25 kip
Lateral	0.03 klf
Allowed service deflection	0.58 in
M _{max}	2.990 kip-in
M _i	3.016 kip-in
D _u	0.021 in
Check deflection	O.K.

ACI 14.8.4

Flexure	Assumption check		
	Span	Hw	Lw
	net Tensile Strain	0.007	0.007
	Check ACI 14.8.2.3	Tmax<Tmin	Tmax<Tmin
	Mmax	0.540 kip-ft	

ACI eq. (14-6)		
M _u	0.740 kip-ft	0.390 kip-ft

ACI 9.3.2		
fb	0.9	0.9
(M _u trial - φAsF _y (d - a/2))	1.410 kip-ft	1.410 kip-ft
DM - M _{pos} - φM _i	0.000 kip-ft	0.000 kip-ft
As add'l req'd	0.00 in ²	0.00 in ²
Additional reinf req'd	0.00 in ²	0.00 in ²
Add'l bar size	3	3
qty req'd	0	0
or spacing of	0	0
As add'l -	0.000 kip-ft	0.000 kip-ft
As _t = As + As add'l	0.20 in ²	0.20 in ²
(M _u - φAsF _y (d _b - a/2))	1.413 kip-ft	1.413 kip-ft
Check φM _u > M _u	O.K.	O.K.
% allowed	52.37%	27.60%



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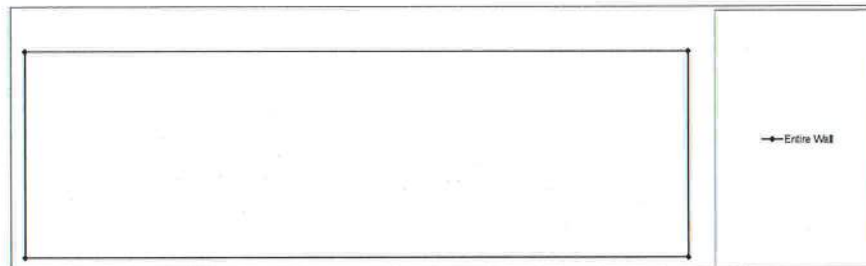
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REINFORCEMENT AT OPENINGS

Loading	
Pu (factored load from roof)	0.31 klf
Ww (weight of panel per sq ft)	0.04 ksf

Material Properties	
db (effective depth bottom)	1.34 in
a (block of strain)	0.33483 psi
$a = A_s \cdot f_y / (0.85 \cdot f'_c \cdot b)$	

Factorized Moment							
Opening	Horizontal Location	Vertical Location	L length of opening	H height above opening	(-) Weight of Opening (LBS)	Pw total factorized panel load	Mu total factorized load
							(wu * L * 2) / 12

Flexure							
Opening	db	As req'd	Bar size	qty req'd	$\phi M_n = \phi A_s f_y (d - a/2)$	Check	$\phi M_n > M_u$

CONNECTIONS

Full Resistance Value							
Base Anchors				Overturning			
Quantity	Maximum	Maximum	Maximum	Lateral	Base Anchors	Wall-Wall Connection	
	R - Distance	L - Distance		Shear	Moment +	Moment -	Moment +
2	65	85		kips	kips-ft	kips-ft	kips-ft
				20,474	25.74	25.74	24.24

Total Tension							
Quantity	Dist	Tension (kip)	Base Anchors	Shear	L - Dist	Moment +	Moment -
Base Anchor 1	10 in	3.58		10.24	85 in	0.351 kip-ft	25.387 kip-ft
Base Anchor 2	85 in	3.58		10.24	10 in	25.387 kip-ft	0.351 kip-ft

Wall Connections									
Quantity of Anchors	Capacity of each Anchor	Countering Dead Load from Adjoining Wall	% of wall to use	Adjoining Wall	Dist (inches)	L - Dist	Allowable Force	Overturning Moment Resistance (kip-ft)	
								Up Left	Low Right
Wall Connection 1	2	1.63T	6.10T	30.74%	W1	0	95,000	3,092	0,000
Wall Connection 2	2	1.63T	5.65T	30.74%	W2	95	0,000	3,092	24,241

Wall Shear Checks							
Shear Connections at Base			Wall Shear Capacity			Required Shear Capacity (lb) per Base Connector	
Design Force (lb)	Capacity (lb)	Reserve Capacity	Design (PLF)	Resistance (PLF)	check		
845	20474	19929	67	15274	OK	322	(16629) Reserve Capacity OK

RIGIDITY

CALCULATED VALUES			100%	Final	4.366923869
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Pier Label	Length (inches)	Height (inches)	Fixed Top? (Y/N)	Useable? (Y/N)	Stiffness (k) (1000 kip / in)	Deflection (in / 1000 kip)
Entire Wall	95	87	Y	Y	4.267	0.234

Combine Logic					
First Segment	Second Segment	Re-Name	Combine/Subtract	Method	Combined
Entire Wall	0	Final			4.267



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ID:	Cortez CR-1598
	DESIGN OF WALL MARKED W6

Notes	
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Material Properties	
F _c	5000 psi
Steel Reinforcement	Phia WWF Grade 80
F _y wire mesh	80000 psi
F _y rebar	60000 psi
Lightweight?	No
Concrete density	150 pcf
E (steel)	29000000 psi
E (concrete)	4290000 psi
n (modular ratio)	6.76

Shear Parameters	
Phi v	0.85
V _c	2.274 kip
Phi*V _c	1.933 kip

Minimum Wall Reinforcement Requirements	
roe min vert	0.0012
roe min hor	0.002
Max Vertical spacing	18 in
Max Horizontal spacing	18 in

Loading	
Axial Design Loads (pressure on roof)	
D (Dead Load) + Wu (Wall weight)	98.44 psf
S (Snow Load)	250 psf
L (Live Load)	0 psf
L _r (Live Roof Load)	30 psf
W (Wind Load)	139.6 psf
E (Earthquake Load)	2.19 psf
Lateral Design Loads (pressure on wall)	
Dead Load (DL lat)	0 psf
Snow Load (SL lat)	0 psf
Live Load (LL lat)	0 psf
Wind Load (WL lat)	75.54 psf
Earthquake Load (EL lat)	1.35 psf

Factored Axially Applied Loads	
Factored Loading per ACI	ACI eq. 9-3
Factored Pressure on Roof W _f	354.803

Axial Pressure on Section	
P _u	1.92 kip
Assumption check	
P _u /A _g	53.333 psi
0.09F _c	300 psi
Check ACI 14.8.5.2	O.K.

Unfactored Axially Applied Loads	
Unfactored Pressure on Roof w _f	353.1375 psf
Axial Pressure on Section	
P _u	1.25 kip

Shear	
Factored Loading per ACI	ACI eq. 9-3
V _u = w _u l*(b _w -2d _f)/2	0.13
Phi*V _c /2	0.97
Check Shear ACI 11.5.3.1	O.K.

Allowable Capacity	
I _g = (b ³ h ³)/12	27 in ⁴
A _g = (b ² h)	36 in ²
Y _t = h/2	1.5
f _r (rupture modulus)	530.330 psi
M _{cr}	9.546 kip-in
Beta ₁	0.8
Trial Ast req'd	0.033 in ²
B _t	8.836162648
kd	0.449 in
I _{cr}	1.44 in ⁴
e _s	0.003
e _s	0.005
a	0.33483 psi
c	0.419 in
As _{cr}	0.22 in ²
I _{er} deflection	1.73 in ⁴
I _e	27.00 in ⁴
delta	150
f _s (maximum tensile reinforcement)	0.0166
f _{temp} (min. temperature reinforcement)	0.0014
f _{max} (minimum tensile reinforcement)	0.0027
f _{total} (trial reinforcement ratio bottom)	0.0033
P _{provided} (reinforcement ratio provided)	0.0120

ACT's Alternate Design of Slender Walls	
Assumptions from this methodology:	
Wall panel shall be simply supported, axially loaded, and subject to out-of-plane uniform lateral loading where maximum moments and deflections occur at mid-height of the wall.	
The cross section is constant over the height of the wall panel.	
The wall cross sections shall be tension controlled.	
Phi*M _n > M _{cr}	
Concentrated gravity loads are distributed over the wall length	
The vertical stress P _u /A _g at mid-height shall not exceed 0.06*F _c	

Geometric Properties	
X Coordinate	136.5
Y Coordinate	9
Direction of Wall	Y
Center of gravity X	136.500
Center of gravity Y	56.500
Wall Weight	2535.000 lbs
Central wall?	Yes
Wall that supports roof panel?	No
kip depth of opening on wall	0 ft
H (height of wall)	87 in
L _h (length of wall)	7.917 ft
Analysis will be performed as	Two-way slab
b (section width)	12 in
h (section thickness)	3 in
cd (cover top)	1 1/2 in
cd (cover bottom)	1 1/2 in
cd (assumed reinf. diameter)	0.319 in
dt (effective depth top)	1.34 in
db (effective depth bottom)	1.34 in
Cx (% of I _e used for Slender)	0.036
Eccentricity - Axial Load	1 in
Is wall Split	No

Wire Mesh	
Wire Size	W6.7
spacing	4 in
Mesh Area	0.20 in ²

Factored Laterally Applied Loads	
Factored Loading per ACI	ACI eq. 9-4
Factored Pressure on Wall W _w	120.86 psf

Lateral Pressure on Section	
L _w = W _w (L/4 + L/4 + H/4)	0.03 klf
H _w = W _w (H/4 + H/4 + L/4)	0.07 klf

Unfactored Laterally Applied Loads	
Unfactored Pressure on Wall w _w	75.54 psf

Lateral Pressure on Section	
L _w = W _w (L/4 + L/4 + H/4)	0.03 klf
H _w = W _w (H/4 + H/4 + L/4)	0.04 klf

Deflection	
Service Loads	
Axial	1.25 kip
Lateral	0.03 klf
Allowed service deflection	0.58 in
Delta	2.990 kip-in
M	3.016 kip-in
Delta	0.021 in
Check deflection	O.K.

Flexure	
Assumption check	
Spun	Hw
net Tensile Strain	0.007
Check ACI 14.8.2.3	Tension
M _u	0.540 kip-ft

ACI eq. (14-6)	
M _u	0.740 kip-ft
	0.390 kip-ft

ACI 9.3.2	
fb	0.9
fb	0.9
fMn trial = phi*As*fy*(d - a/2)	1.410 kip-ft
DM = Mpos - phi*M	0.000 kip-ft
As add'l req'd	0.00 in ²
Additional reinf req'd	0.00 in ²
Additional reinf req'd	0.00 in ²
As add'l req'd	0
or spacing of	0
As add'l -	0.000 kip-ft
As add'l -	0.20 in ²
fMn = phi*As*fy*(d - a/2)	1.413 kip-ft
Check phi*Mn > Mu	O.K.
% allowed	52.37%
	27.60%



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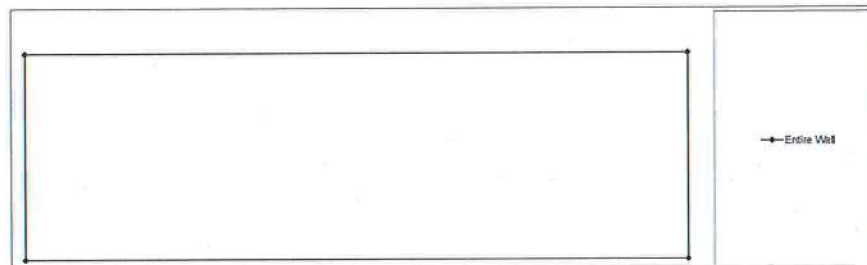
PFS Mark Severson

Title:

Staff Plan Reviewer

Date:

4/6/22



REINFORCEMENT AT OPENINGS

Loading	
Pu (factorized load from roof)	0.31 klf
Ww (weight of panel per sq ft)	0.04 ksf

Material Properties	
db (effective depth bottom)	1.34 in
a (block of strain)	0.33483 psi
$a = A_s \cdot f_y / (0.85 \cdot f'_c \cdot b)$	

Factorized Moment								
Opening	Horizontal Location	Vertical Location	L length of opening	H height above opening	(-) Weight of Opening (LBS)	Pw total factorized panel load	wu total factorized load	Mu (wu*L/2)/12

Reinure							
Opening	db	As req'd	Bar size	qty req'd	$\phi M_n = \phi A_s f_y (d - a/2)$	Check	$\phi M_n \geq M_u$

CONNECTIONS

Base Anchors		Full Resistance Value			
		Overturning		Wall-Wall Connection	
Quantity	Maximum	Maximum	Lateral Shear	Moment +	Moment -
in Shear	R - Distance	L - Distance	kip	kip-ft	kip-ft
2	85	85	20.474	25.74	24.24

Total Tension		Base Anchors			
7.16k	Dist	Tension (kip)	Shear	L - Dist	Moment +
Base Anchor 1	10 in	3.58	10.24	85 in	0.351 kip-ft
Base Anchor 2	85 in	3.58	10.24	10 in	25.387 kip-ft

Wall Connections													
	Quantity of Anchors	Capacity of each Anchor	Countering Dead Load from Adjoining Wall	% of wall to use	Adjoining Wall	Dist (inches)	L - Dist	Allowable Force	Overturning Moment Resistance (kip-ft)				
									Up Left	Low Right			
Wall Connection 1	2	1.531	6.107	30.74%	W1	0	95.000	3.062	0.000	24.241	24.241		
Wall Connection 2	2	1.531	5.651	30.74%	W2	95	0.000	3.062	24.241	0.000	24.241		

Shear Connections at Base		Wall Shear Capacity		Required Shear Capacity (lb) per Base Connector		Reserve Capacity
Design Force (lb)	Capacity (lb)	Reserve Capacity	Design (PLF)	Resistance (PLF)	check	
645	20474	19529	67	15274	OK	(19529) OK

RIGIDITY

CALCULATED VALUES			100%	Final	4.266922869	
Pier Label	Length (inches)	Height (inches)	Fixed Top? (Y/N)	Useable? (Y/N)	Stiffness (k) (1000 kip / in)	Deflection (in / 1000 kip)
Entire Wall	95	87	Y	Y	4.267	0.234

Combine Logic					
First Segment	Second Segment	Re-Name	Combine/Subtract	Method	Combined
Entire Wall	0	Final			4.267



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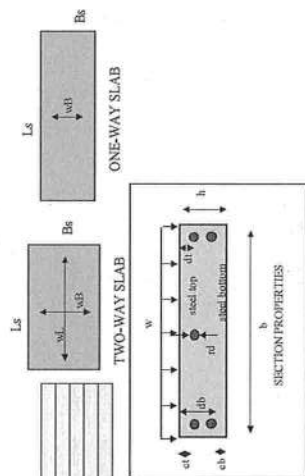
Mark Severson

Title:

Staff Plan Reviewer

Date:

4/6/22

[illegible]

$P_{\text{protest}}(\text{reinforcement info provided})$	$\sigma =$	0.0053
$P_{\text{protest}}(\text{reinforcement info provided})$	$\sigma =$	0.0842
$P_{\text{protest}}(\text{reinforcement info provided})$	$\sigma =$	0.0154
$P_{\text{protest}}(\text{reinforcement info provided})$	$\sigma =$	0.2469
$P_{\text{protest}}(\text{reinforcement info provided})$	$\sigma =$	0.0042
$P_{\text{protest}}(\text{reinforcement info provided})$	$\sigma =$	0.0097
$P_{\text{protest}}(\text{reinforcement info provided})$	$\sigma =$	0.0118
$P_{\text{protest}}(\text{reinforcement info provided})$	$\sigma =$	0.1803

		Wire Mesh (Top)
	Wire Size	W6.7
	spacing	4 in
	Mesh Area	0.20 in ²

Factored Loading per ACI Load combination indicated	Factored Pressure on Slab W	Pressure on Section	Pressure on Section	Pressure on Section
407 psf	W _U = 1.4 D + 1.7 L + 1.7 P	0.177 ksi	0.28 ksi	0.67 ksi
462.5 psf	W _U = 1.4 D + 1.4 L + 1.7 P	0.107 ksi	0.28 ksi	0.67 ksi
407 psf	W _U = 1.4 D + 1.7 L + 1.7 P	0.177 ksi	0.28 ksi	0.67 ksi
462.5 psf	W _U = 1.4 D + 1.4 L + 1.7 P	0.107 ksi	0.28 ksi	0.67 ksi

Wire Mesh (Bottom)			As	Pressure on Section	
Wire Size	Wx7	Wx7		Wx7	Wx7
Spacing	4 in				
Mesh Area	0.20 in ²				
Total Air Required	0.126 in ²				
Design Load			Pressure on Section	Pressure on Section	
Factored Pressure on Area	440 psf			Wx7	Wx7
			0.091 ksf	0.231 ksf	
			3.27	2.43	
			0.15 kN	0.38 kN	

3,277 ft		$W^*B/4 = B^3/4 \cdot L/4 \cdot \rho \cdot g$ W^*B	Pressure on Section $W^*B/4 = L/4 \cdot \rho \cdot g$
2.43 ft			
0.18 kip		0.107 klf	0.353 klf
0.18 kip		$W^*B/4 = B^3/4 \cdot L/4 \cdot \rho \cdot g$ W^*B	Pressure on Section $W^*B/4 = L/4 \cdot \rho \cdot g$
3,277 ft			
0.18 kip		2.43 ft	0.43 kip
0.18 kip		0.43 kip	0.43 kip

Efficiency can be enhanced if A_{st} is diminished

SUMMARY

Use 1 Layer of Wire Mesh on Top W6.7 x W6.7 x 4 x 4

Use 1 Layer of Wire Mesh on Bottom W6.7 x W6.7 x 4 x 4

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Signature: *Mark Severson*

Title: Staff Plan Reviewer

Date: 4/6/22

ID: Cortez CR-1598

Geometric properties		Loading	
Bs (width of roof panel)	10.25 ft	Wv (weight of vault)**	0 lb
Ls (Length of roof panel)	19.00 ft	Wtr (roof panel weight)	11820 lb
Ar (Area of Roof)	194.75 ft ²	Ww (total walls panel weight)	21330 lb
H (height of building)	9.37 ft	Fw (floor panel weight)	10035 lb
Lb (length of building)	9.42 ft	We (estimated weight of building)	43185 lb
Wb (width of building)	17 ft	Wew (estimated weight of building w/ vault)	43185 lb
Ab (Area of building)	160.14 ft ²	PSFr (roof snow load)	210 psf
Nv (quantity of vaults)	0	PSFF (Floor Live Load)	400 psf
Avl (Area of Vault Lips)	0.00 ft ²	Pmax (Maximum allowable pressure)	1500 psf
Av (Area of Vault)	0.00 ft ²	Fupmw (MWFRS Uplift Force)	51.69 psf
Vh (Vault height)	0 ft	WLlat (MWFRS lateral wind pressure)	61.91 psf
Cab (Closed Area of building)	137.50 ft ²	γw (specific weight of water)	62.4 pcf
Hw (depth of floodwater)	1 ft	**Weight of vault is not considered in sliding resistance	
μ (sliding factor)	0.40	FS (factor of safety required)	1.00

CHECK SLIDING RESISTANCE

Shear	.7*Vseismic (from seismic analysis with snow)	1294.4 lb
	.7*Vseismic (from seismic analysis without snow)	1088.3 lb
	Vwind = WLlat * max(Wb, Lb) * H	9861.6 lb

* Load adjustment per IBC 1605.3 load combinations.

Sliding Resistance with Snow	Pslide = $u * (.6 * We + .75 * PSFr * Ar)$	Pslide =	22633.65 lb		
Factor of Safety	FSwind = Pslide / Vwind	FSwind =	2.3	≥	1.0
	FSseismic = Pslide / Vseismic	FSseismic =	17.5	≥	1.0
Sliding Resistance with No Snow	Pslide = $u * .6 * We$	Pslide =	10364.4 lb		
Factor of Safety	FSwind = Pslide / Vwind	FSwind =	1.1	≥	1.0
	FSseismic = Pslide / Vseismic	FSseismic =	9.5	≥	1.0

CHECK OVERTURNING RESISTANCE

Shear	.7*Otsseismic (from seismic analysis with snow)	10.851 kip-ft
	.7*Otsseismic (from seismic analysis without snow)	9.071 kip-ft
	Otwind = $(WLlat * Lb * H^2 / 2) + (Fupmw * Lb * Wb^2 / 2)$	95.956 kip-ft

* Load adjustment per IBC 1605.3 load combinations.

Overturning Resistance with Snow	Otrsnow = $(.6 * We + .75 * PSFr * Ar) * (Wb / 2)$	Otrsnow =	232.788 kip-ft		
Factor of Safety	FSwind = Otrsnow / Otwind	FSwind =	2.43	≥	1.0
	FSseismic = Otrsnow / Vseismic	FSseismic =	21.45	≥	1.0
Overturning Resistance with No Snow	Otr = $.6 * We * Wb / 2$	Otr =	220.244 kip-ft		
Factor of Safety	FSwind = Otr / Vwind	FSwind =	2.30	≥	1.0
	FSseismic = Otr / Vseismic	FSseismic =	24.28	≥	1.0

CHECK BEARING PRESSURE CONDITION

Net Pressure	Pnet = $(Wew + PSFr * Ar + PSFF * Af) / Ab$	925.06 psf		
Allowable	Pmax ≥ Pnet	1500 psf ≥ 925.06 psf		

By observation, if the building is placed on a properly prepared well drained granular sub-base, the design is sufficient for lateral and vertical loads.

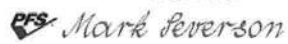
CHECK BUOYANCY FORCE CONDITION

Buoyant Force	Fb = $γw * Av * Hw + γw * Cab * (Hw - Vh)$	Fb =	8580.00 lb		
Factor of Safety	FSb = We / Fb	FSb =	5.03	≥	1.00

The weight of the building exceeds the buoyant force due to hydrostatic pressure acting on the horizontal surface of the vault, therefore, the design is sufficient against buoyancy.

Floor Design Information:

- 1) The referenced building is made of flood damage resistant 5000 psi reinforced concrete.
- 2) The vault system, if existing, is designed to minimize infiltration into system and can be considered water tight to a height of 17"
- 3) Flood Ventilation is available at threshold level and flood ventilation exceeding 1" per sq. ft. of floor area is provided no more than 12" A.F.F.

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