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

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PLANS EXAMINER

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State:	**Florida**
Signature:	PFS Mark Severson
Title:	**Staff Plan Reviewer**
Date:	4/6/22

WILLIAM R. HEIDEN III
LICENSE
No. 58523
STATE OF
FLORIDA
PROFESSIONAL ENGINEER

April 4, 2022

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Appendix: (Provided Upon Request) UL-752 Bullet Resistance Testing

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Title:	**Staff Plan Reviewer**
Date:	4/6/22

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

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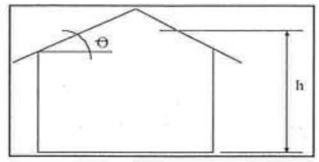
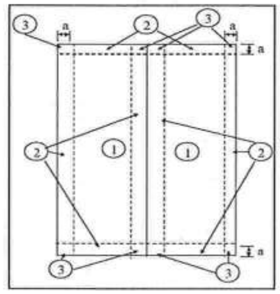
No. 58523
STATE OF FLORIDA
PROFESSIONAL ENGINEER

April 4, 2022

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
Kd	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 Kzt

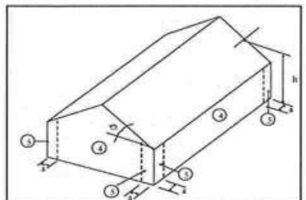




Kzt	1	Topographic factor
h	8.310 ft	Mean roof height
n_a	9.03	Natural frequency
Flexibility	Rigid	Building flexibility
α	9.5	Terrain factor
z_g	900 ft	Terrain factor

Velocity Pressure Exposure Coefficient		
K(z)	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	Agi	Aoi	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

Gust Factor - (26.9)	
G =	0.85

External Pressure Coefficients				
C_{pe}	0.8	See 27.3.3 Roof Overhangs		
Cp	0.8	Windward wall (Use with qz) Fig. 27.3-1		
	-0.339	Leeward wall (wind normal to ridge) (Use with qh)	L/B =	1.80
	-0.500	Leeward wall (wind parallel to ridge) (Use with qh)	L/B =	0.55
	-0.7	Sidewalls (Use with qh) Fig. 27.4-1		

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

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

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

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

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 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	Effective Area					
	10.0 sq ft		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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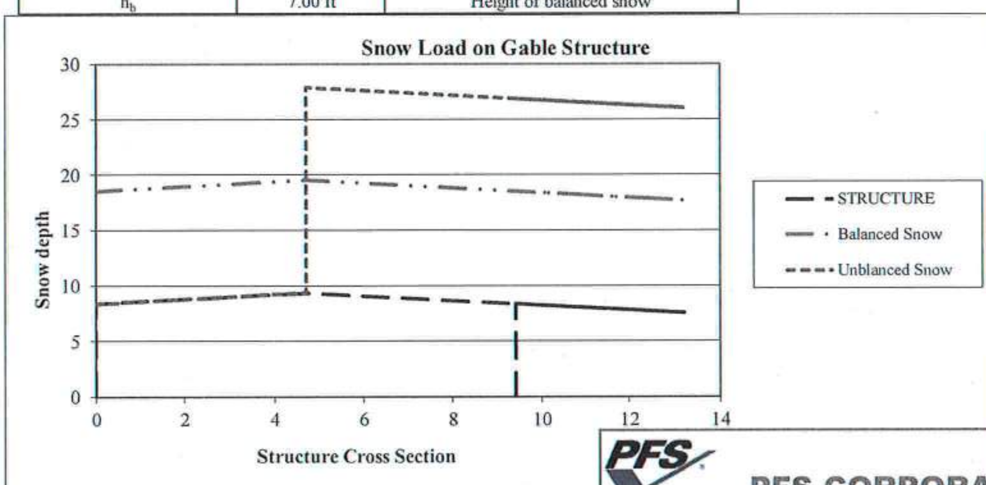
ASCE 7-16 SNOW LOAD CALCULATION

Category	II	IBC TABLE 1604.5: Risk Category of Buildings and Other Structures.
Exposure	C	See § 26.7.3: Exposure Categories, General.
Pg	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
ϑ	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)
Is	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
Cs	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 $P_{leeward1}$ 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.	
Ss	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	
Ω_0	2.5	System Over strength Factor	
C_t	0.02	Approximate period parameter	ASCE Table 12.8-2
x	0.75	Approximate period parameter	ASCE Table 12.8-2
hn	8.52 ft	Height in feet from base to highest level of structure	

				Value 1*	Value 2*
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

*=Used for interpolation

$Sms = Fa * S_S$	0.216 g	Adjusted MCE Spectral Response Acceleration at short periods	*ASCE 11.4-1*
$S_{m1} = F_v * S_1$	0.163 g	Adjusted MCE Spectral Response Acceleration at 1 sec period	*ASCE 11.4-2*

(MCE = Maximum considered earthquake)

$S_{DS} = 2/3\ Sm_s$	0.144 g	Design Spectral Acceleration Parameters	*ASCE 11.4-3*
$S_{D1} = 2/3\ Sm_1$	0.109 g	Design Spectral Acceleration Parameters	*ASCE 11.4-4*

I_E	1	Importance Factor	*ASCE Table 1.5-2*

Seismic Design Category		B
Based on S_{DS}	A	Table 11.6-1
Based on S_{D1}	B	Table 11.6-2

Geotechnical Investigation Report Required? No.

EQUIVALENT LATERAL FORCE PROCEDURE			
$T_a = C_t * hn^x$	0.10 sec	Approximate fundamental period	*ASCE 12.8-7*
$T_s = S_{D1}/S_{DS}$	0.76 sec		
T	0.10 sec	Fundamental period of the structure (can be taken as Ta per ASCE 12.8.2)	

$C_s = S_{DS}/(R/I)$	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_s	0.036		
k	1.000	*ASCE 12.8.3*	
W	51.36 kip		
$V = C_s * W$	4.62 kip	*ASCE 12.8-1*	*Shear with snow load*
$M_o =$	38.8 k-ft		*Overturning Moment with snow load*
$V = C_s * W$	3.89 kip		*Shear without snow load*
$M_o =$	32.4 k-ft		*Overturning Moment without snow load*

WITH SNOW LOAD

Level	Story Height	h_i or h_x	P_f (flat roof snow load)	w_i	$w_i*h_i^k$	C_{vx} (*12.8-12*)	F_x (*12.8-11;11.7*)	V_x (Story shear)	M_x	F_{px} (diaphragm force) (*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	0.0 k-ft	1.77 kip
Walls	0.00 ft	0.00 ft								
Floor	0.21 ft	0.21 ft		20.70 kip	4.3 k-ft	0.016	0.08 kip	4.62 kip	37.8 k-ft	1.19 kip
Base	0 ft	0.00 ft	W=	51.36 kip	265.5 k-ft			M_o =	38.8 k-ft	

WITHOUT SNOW LOAD

Level	Story Height	h_i or h_x	P_f (flat roof snow load)	w_i	$w_i*h_i^k$	C_{vx} (*12.8-12*)	F_x (*12.8-11;11.7*)	V_x (Story shear)	M_x	F_{px} (diaphragm force) (*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	0.0 k-ft	1.30 kip
Walls	0.00 ft	0.00 ft								
Floor	0.21 ft	0.21 ft		20.70 kip	4.3 k-ft	0.022	0.09 kip	3.89 kip	31.6 k-ft	1.19 kip
Base	0 ft	0.00 ft	W=	43.19 kip	195.8 k-ft			Mo =	32.4 k-ft	

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Center of Mass & Rigidity

Cortez CR-1598

					X	Y	
	Upper Left = 0,0		Lower Right		228	123	
Wall	X Relative Stiffness	Y Relative Stiffness	Shear Force lbs	Shear Force plf	Dist to CoRx dx (IN)	Dist to CoRy dy (IN)	
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 X	Top Edge Y	Right Edge X	Bottom Edge Y	Snow/Live (psf)	Center of Gravity X	Center of Gravity Y	Live w snow	Live w/o snow
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)		wgt (w snow)	wgt (w/o snow)
ex	ey					
0.00	4.18	51,365	43,185	roof	30,665	22,485
Center of Gravity				floor	20,700	
X	Y					
114.0	59.5					
Center of Rigidty						
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 Moment (kip-ft)	check	Toward Upper Left Anchor Resistance Moment (kip-ft)	check	Overturning status using just connection to adjacent walls
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 Must investigate ONLY if connection to adjacent walls is insufficient						
Wall	Design Moment (kip-ft)	Toward Lower Right Anchor Resistance Moment (kip-ft)	check	Toward Upper Left Anchor Resistance Moment (kip-ft)	check	Combined Loading Unity Check	Required Tension Capacity per Base Anchor (lb)
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	(4280)
W6	-9.64	25.74	OK	25.74	OK	OK	(4280)

	Wall Overturning Checks Using Base Anchors and Connection to Adjacent Walls Must investigate ONLY if both base anchor alone and adjacent walls alone are insufficient					
	Base Anchor Shear Required	Base Anchor Tension Available	Available Overturning Resistance (kip-ft) From Base Anchors		Overturning Unity Check of Base Anchors	
Wall	(% Capacity)	(% Capacity)	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 R1, R2

Notes:

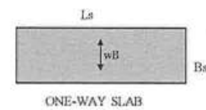
Material Properties

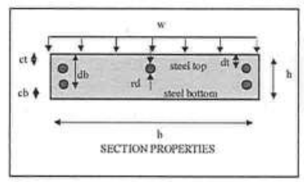
f ' c	5000 psi	
Steel Reinforcement	Plain WWF Grade 80	
Fy	80000 psi	
Lightweight?	No	
C_d (Concrete density)	150 pcf	O.K.
E (Steel)	29000000 psi	
E (Concrete)	4286826 psi	*ACI 8.5.1*
n (modular ratio)	6.76	*ACI 14.0*

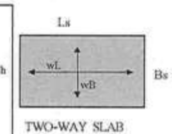
Geometric Properties

Ls (overall length of slab)	10.25 ft		
Bs (overall width of slab)	9.79 ft		
Design will be performed as :	**Two-way slab**		
tfr (roof finish thickness)	0.375 in		
b (section width)	12 in	*(typically 12 inches)*	
h (section thickness)	4.5 in		
ct (cover top)	0 in		
cb (cover bottom)	1 in		
rd (assumed reinf. diameter)	0.319 in	*(if centered enter 0)*	
dt (effective depth top)	0.1595 in		
db (effective depth bottom)	3.181 in		
oh1 (overhang length and qty for Bs)	12 in	1	*(qty of overhangs in Bs direction)*
oh2 (overhang length and qty for ls)	10 in	2	*(qty of overhangs in ls direction)*
Cs (% of DL used for Seismic)	0.036	*(from seismic analysis)*	
NBs (qty of walls in Bs direction)	2	*(walls that support one or more roof panels in the short direction)*	
Nls (qty of walls in ls direction)	2	*(walls that support one or more roof panels in the long direction)*	

fr (rupture modulus)	530.3 psi	*ACI 9.5.2.3*
Ig = (b*h^3)/12	91.125 in^4	
Ag = (b*h)	54 in^4	
Yt = h/2	2.25 in^4	
Mcr	21.48	*ACI 9.5.2.3*
Beta_1	0.8	*ACI 10.2.7.3*
delta_initial	180	
delta_longterm	480	
B	8.830	
kd	0.743 in	
I cr	9.72 in^4	
c	0 in	
a	0.32 in	







bottom mesh

$\rho_{provided}$ (reinforcement ratio provided)		0.0053
top mesh	ω =	0.0848 psi
$\rho_{provided}$ (reinforcement ratio provided)		0.0000
both layers	ω =	0 psi
$\rho_{provided}$ (reinforcement ratio provided)		0.0100
	ω =	0.16 psi

Reinforcement Limits

ρ_l (maximum tensile reinforcement)	0.0166	*ACI 10.3.3*
ρ_{temp} (min. temperature reinforcement)	0.0014	*ACI 7.12.2*
ρ_{min} (minimum tensile reinforcement)	0.0027	*ACI 10.5.1*
ρ_{trial} (trial reinforcement ratio bottom)	0.0033	
ρ_{trial} (trial reinforcement ratio top)	0.0033	

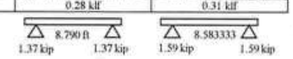
Trial As' req'd	Wire Mesh (Top)		
	Wire Size	None	
	spacing		
	Mesh Area	**0.00 in^2**	= As'

Trial Ast required	Wire Mesh (Bottom)		
	Wire Size	W6.7	
	spacing	4 in	
0.126 in^2	Mesh Area	**0.20 in^2**	= As

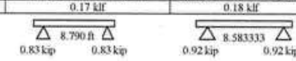
Loading

Design Loads	
Pressure on Slab	w
D (Dead load))	60.938 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

Factored Design Loads		Pressure on Section	Pressure on Section
Factored Loading per ACI equation indicated	Factored Pressure on Slab W	**wB** = W*(L^4 / B^4 + L^4)*be	**wL** = W*(B^4 / B^4 + L^4)*be
ACI eq. 9-3	584.805 psf	0.28 klf	0.31 klf



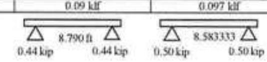
Unfactored Design Loads	Pressure on Section	Pressure on Section
Unfactored Pressure on Slab W	**wB** = W*(L^4 / B^4 + L^4)*be	**wL** = W*(B^4 / B^4 + L^4)*be
353.1375 psf	0.17 klf	0.18 klf



Sustained Loading	
Pressure on slab	W
D (Dead load)	60.938 psf
S (Snow Load)	250 psf
Lr (Live Roof Load)	30 psf

B (Span in the short direction) = Bs-1(oh1)	8.790 ft
L (Span in the long direction) = Ls-2(oh2)	8.583333333 ft

Factored Sustained Loads		Pressure on Section	Pressure on Section
Factored Loading per ACI equation indicated	Factored Pressure on Slab W	**wB** = W*(L^4 / B^4 + L^4)*be	**wL** = W*(B^4 / B^4 + L^4)*be
ASCE7-05 eq CC16	185.938 psf	0.09 klf	0.097 klf



SUMMARY

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

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 ROOF PANELS MARK R1, R2

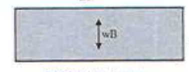
Notes:

Material Properties

f ' c	5000 psi	
Steel Reinforcement	Plain WWF Grade 80	
Fy	80000 psi	
Lightweight?	No	
C_a (Concrete density)	150 pcf	O.K.
E (Steel)	29000000 psi	
E (Concrete)	4286826 psi	ACI 8.5.1
n (modular ratio)	6.76	ACI 14.0

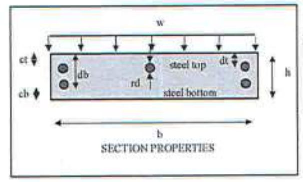
fr (rupture modulus)	530.3 psi	ACI 9.5.2.3
Ig = (b*h^3)/12	91.125 in^4	
Ag = (b*h)	54 in^4	
Yt = h/2	2.25 in^4	
Mcr	21.48	ACI 9.5.2.3
Beta_1	0.8	ACI 10.2.7.3
delta_initial	180	
delta_longterm	480	
B	8.830	
kd	0.743 in	
I.cr	9.72 in^4	
c	0 in	
a	0.32 in	

Ls



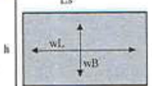
Bs

ONE-WAY SLAB



SECTION PROPERTIES

Ls



Bs

TWO-WAY SLAB

Geometric Properties

Ls (overall length of slab)	10.25 ft		
Bs (overall width of slab)	9.79 ft		
Design will be performed as :	**Two-way slab**		
tfr (roof finish thickness)	0.375 in		
b (section width)	12 in	(typically 12 inches)	
h (section thickness)	4.5 in		
ct (cover top)	0 in		
cb (cover bottom)	1 in		
rd (assumed reinf. diameter)	0.319 in	(if centered enter 0)	
dt (effective depth top)	0.1595 in		
db (effective depth bottom)	3.181 in		
oh1 (overhang length and qty for Bs)	12 in	1	[qty of overhangs in Bs direction]
oh2 (overhang length and qty for ls)	10 in	2	[qty of overhangs in ls direction]
Cs (% of DL used for Seismic)	0.036	[from seismic analysis]	
NBs (qty of walls in Bs direction)	2	[walls that support one or more roof panels in the short direction]	
Nls (qty of walls in ls direction)	2	[walls that support one or more roof panels in the long direction]	

bottom mesh

$\rho_{provided}$ (reinforcement ratio provided)		0.0053
top mesh	ω =	0.0848 psi
$\rho_{provided}$ (reinforcement ratio provided)		0.0000
both layers	ω =	0 psi
$\rho_{provided}$ (reinforcement ratio provided)		0.0100
	ω =	0.16 psi

Flexure

Flexural Moments for Bs	Mu	Tensile Strain	Check ACI 14.8.2.3	ϕb	ϕMn trial = ϕf'cbd^2ω(1-0.59ω)	ΔM = Mu - ϕM	ϕMn =	Check ϕMn > Mu	% allowed
Mpos (positive Moment) = (wB*B^2) / 8	2.7 kip-ft	0.036	Tension	0.9	3.67 kip-ft		3.67 kip-ft	O.K.	73.57%

	Mu	Structural Plain Concrete per ACI 22.5 S Elastic Section Modulus	ϕb	ϕMn = ϕ5*(f'c*S)^0.5	Check ϕMn > Mu	% allowed
Mneg (negative Moment) = (wB*oh1^2) / 2	0.140 kip-ft	0.023 ft^3	0.55	0.941 kip-ft	O.K.	14.88%
Mneg (negative Moment) = (wB*oh2^2) / 2	0.140 kip-ft	0.023 ft^3	0.55	0.941 kip-ft	O.K.	14.88%

Flexural Moments for ls	Mu	Tensile Strain	Check ACI 14.8.2.3	ϕb	ϕMn trial = ϕf'cbd^2ω(1-0.59ω)	ΔM = Mu - ϕM	ϕMn =	Check ϕMn > Mu	% allowed
Mpos (positive Moment) = (wL*L^2) / 8	2.855 kip-ft	0.036	Tension	0.9	3.67 kip-ft		3.67 kip-ft	O.K.	77.79%

	Mu	Structural Plain Concrete per ACI 22.5 S Elastic Section Modulus	ϕb	ϕMn = ϕ5*(f'c*S)^0.5	Check ϕMn > Mu	% allowed
Mneg (negative Moment) = (wL*oh1^2) / 2	0.1076389	0.023 ft^3	0.55	0.941 kip-ft	O.K.	11.44%
Mneg (negative Moment) = (wL*oh2^2) / 2	0.1076389	0.023 ft^3	0.55	0.941 kip-ft	O.K.	11.44%

Shear

Maximum Shear for Bs	Vu	ϕv per ACI 9.3.2.3	Vc per ACI 11.3.1.1	ϕVc	Check ϕVc > Vu	% allowed
Vu = wB (B/2)	1.23 kip	0.85	5.40 kip	4.59 kip	O.K.	26.82%
			per ACI 22.8			
Vu for side overhang 1 = wB*oh1	0.28 kip	0.55	2.98 kip	1.64 kip	O.K.	17.08%
Vu for side overhang 1 = wB*oh1	0.28 kip	0.55	2.98 kip	1.64 kip	O.K.	17.08%

Shear for Ls	Vu	ϕv per ACI 9.3.2.3	Vc per ACI 11.3.1.1	ϕVc	Check ϕVc > Vu	% allowed
Vu = wL (L/2)	0.42 kip	0.85	5.40 kip	4.59 kip	O.K.	9.07%
			per ACI 22.8			
Vu for end overhang 2 = wL*oh2	0.08 kip	0.55	2.98 kip	1.64 kip	O.K.	4.93%
Vu for end overhang 2 = wL*oh2	0.08 kip	0.55	2.98 kip	1.64 kip	O.K.	4.93%

Span type:	K	Sustained Load Duration	Epsilon
Simple span	1	6 months	1.2

Deflection

Span	Service Loads Mserv	M.sus	I.eff.serv	I.eff.sustained	Immediate Deflection Δi	ρ_{comp}	λ	Long-Term Deflection Δl-t	Δ total long-term deflection (Δi + Δl-t)	Δ allow (immediate)	Δ allow (long term)	Check short term deflection	Check total long term deflection	% allowed - short term	% allowed - total long term
B	2.7 kip-ft	0.869 kip-ft	33.43 in^4	91.13 in^4	0.031 in	0.0053	0.9486	0.029 in	0.060 in	0.5860 in	0.2198 in	O.K.	O.K.	5.28%	27.44%
L	2.855 kip-ft	0.893 kip-ft	29.78 in^4	91.13 in^4	0.000 in	0.0053	0.9486	0.029 in	0.060 in	0.5722 in	0.2146 in	O.K.	O.K.	0.00%	28.10%

ID:	Cortez CR-1598
	DESIGN OF WALL MARKED W1

Notes	

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	O.K.
E (Steel)	29000000 psi	
E (Concrete)	4290000 psi	ACI 8.5.1
n (modular ratio)	6.76	

Shear Parameters

Phi.v	0.85	ACI 9.3.2.3
Vc	3.123 kip	ACI 11.3.1.1 & 11.2.1.2
Phi*Vc	2.654 kip	ACI 11.1.1

Minimum Wall Reinforcement Requirements

roe min vert	0.0012	ACI 14.3.2
roe min hor	0.002	ACI 14.3.3
Max Vertical spacing	18 in	ACI 14.3.5
Max Horizontal spacing	18 in	ACI 14.3.5

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-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*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*f'c	ACI 14.8.2.6

Geometric Properties

X Coridinate	16
Y Coridinate	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
lop (length 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)	12 in
h (section thickness)	4 in
ct (cover top)	2 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

Loading

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

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 Wr	584.805

Axial Pressure on Section	
PuB	1.93 kip

Assumption check	
Pu/Ag	40.208 psi
0.06*f'c	300 psi
Check ACI 14.8.2.6	O.K.

Unfactored Axially Applied Loads

Unfactored Pressure on Roof uWr	353.1375 psf

Axial Pressure on Section	
PB	1.26 kip

Factored Laterally Applied Loads

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

Lateral Pressure on Section	
Lw = W*(L^4 / H^4 + L^4)	0 klf
Hw = W*(L^4 / H^4 + L^4)	0.12 klf

Unfactored Laterally Applied Loads

Unfactored Pressure on Wall uWw	75.54 psf

Lateral Pressure on Section	
Lw = W*(L^4 / H^4 + L^4)	0 klf
Hw = W*(L^4 / H^4 + L^4)	0.08 klf

Shear

Factored Loading per ACI	ACI eq. 9-3
Vu = wuB*(Bw-2db) / 2	0
Phi*Vc/2	1.33
Check Shear ACI 11.5.5.1	O.K.

Deflection ACI 14.8.4

Service Loads	
Axial	1.26 kip
Lateral	0 klf
Allowed service deflection	0.65 in
Msa	0.630 kip-in
M	0.633 kip-in
Ds	0.002 in
Check deflection	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
I cr	2.92 in^4
e_c	0.003
e_t	0.005
a	0.33483 psi
c	0.419 in
Ase	0.23 in^2
Icrdeflection	3.61 in^4
Ie	64.00 in^4
delta	150
r_t (maximum tensile reinforcement)	0.0166
r_{temp} (min. temperature reinforcement)	0.0014
r_{min} (minimum tensile reinforcement)	0.0027
r_{trial} (trial reinforcement ratio bottom)	0.0033
$\rho_{provided}$ (reinforcement ratio provided)	0.0090

Flexure

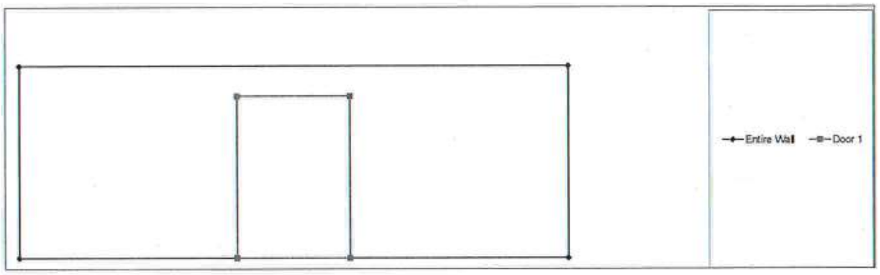
Assumption check		
Span	Hw	Lw
net Tensile Strain	0.010	0.010
Check ACI 14.8.2.3	Tension	Tension
Mua	1.070 kip-ft	

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

ACI 9.3.2		
fb	0.9	0.9
fMn trial = ϕAsFy(dt - 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
Ast = As + As add'l	0.20 in^2	0.20 in^2
fMn = ϕAsFy(db - a/2)	2.016 kip-ft	2.016 kip-ft
Check ϕMn > Mu	O.K.	O.K.
% allowed	63.49%	0.00%

PFS
PFS CORPORATION
Approval Limited to Factory Built Portion Only

State: Florida
Signature: 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)	0.05 ksf

Material Properties	
db (effective depth bottom)	1.84 in
a (block of strain)	0.33483 psi
	a=As * fy / (0.85 * f'c *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
Door 1	6.49 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:	ϕMn = ϕAsFy(db - a/2)	Check ϕMn > Mu
Door 1	0.9	0.006 in^2	No. 3	1	6.69 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 -
in Shear	R - Distance	L - Distance	kip	kip - ft	kip - ft	kip - ft	kip - ft
4	187	187	42.920	91.63	91.63	129.25	129.25

Total Tension	Base Anchors					
14.392	Dist	Tension (kip)	Shear	L - Dist	Moment +	Moment -
Base Anchor 1	9 in	3.56	9.25	187 in	0.128 kip*ft	55.399 kip*ft
Base Anchor 2	59 in	3.64	12.21	137 in	5.648 kip*ft	30.454 kip*ft
Base Anchor 3	137 in	3.64	12.21	59 in	30.454 kip*ft	5.648 kip*ft
Base Anchor 4	187 in	3.56	9.25	9 in	55.399 kip*ft	0.128 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	6.511	46.02%	W3	0	196.000	3.062	0.000	50.013
Wall Connection 2	2	2.703	4.852	50.00%	W5	75.5	120.500	4.852	30.524	48.717
Wall Connection 3	2	2.703	4.852	50.00%	W6	120.5	75.500	4.852	48.717	30.524
Wall Connection 4	2	1.531	6.511	46.02%	W4	196	0.000	3.062	50.013	0.000

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			Reserve Capacity
1659	42920	41261	86	13406	OK	415	(41261)	OK

RIGIDITY

CALCULATED VALUES	66%	Final	8.159612943

	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
Door 1	A'	196	82.28	Y	Y	15.000	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
Door 1	Entire Wall	A'	A'a	-	Deflection	0.014
	A	B	AB	+	Stiffness	9.213
	A'a	AB	Final	+	Deflection	0.123

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State: Florida
Signature: PFS Mark Severson
Title: Staff Plan Reviewer
Date: 4/6/22

ID:	Cortez CR-1598
	DESIGN OF WALL MARKED W2

Notes	

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	O.K.
E (Steel)	29000000 psi	
E (Concrete)	4290000 psi	*ACI 8.5.1*
n (modular ratio)	6.76	

Shear Parameters

Phi.v	0.85	*ACI 9.3.2.3*
Vc	3.123 kip	*ACI 11.3.1.1 & 11.2.1.2*
Phi*Vc	2.654 kip	*ACI 11.1.1*

Minimum Wall Reinforcement Requirements

roe.min.vert	0.0012	*ACI 14.3.2*
roe.min.hor	0.002	*ACI 14.3.3*
Max Vertical spacing	18 in	*ACI 14.3.5*
Max Horizontal spacing	18 in	*ACI 14.3.5*

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-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*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*f'c	*ACI 14.8.2.6*

Geometric Properties

X Corrdinate	16
Y Corrdinate	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
lop (length 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)	12 in
h (section thickness)	4 in
ct (cover top)	2 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

Loading

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

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 Wr	584.805

Axial Pressure on Section	
PuB	1.88 kip

Assumption check	
Pu/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 uWr	353.1375 psf

Axial Pressure on Section	
PB	1.21 kip

Factored Laterally Applied Loads

Factored Loading per ACI	*ACI eq. 9-4*
Factored Pressure on Wall Ww	120.86 psf

Lateral Pressure on Section	
Lw = W*(L^4 / H^4 + L^4)	0 klf
Hw = W*(L^4 / H^4 + L^4)	0.12 klf

Unfactored Laterally Applied Loads

Unfactored Pressure on Wall uWw	75.54 psf

Lateral Pressure on Section	
Lw = W*(L^4 / H^4 + L^4)	0 klf
Hw = W*(L^4 / H^4 + L^4)	0.08 klf

Shear

Factored Loading per ACI	ACI eq. 9-3
Vu = wuH*(Bw-2db) / 2	0
Phi*Vc/2	1.33
Check Shear ACI 11.5.5.1	O.K.

Deflection *ACI 14.8.4*

Service Loads	
Axial	1.21 kip
Lateral	0 klf
Allowed service deflection	0.65 in
Msa	0.605 kip-in
M	0.608 kip-in
Ds	0.002 in
Check deflection	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
I cr	2.92 in^4
e_c	0.003
e_s	0.005
a	0.33483 psi
c	0.419 in
Ase	0.22 in^2
Icrdeflection	3.47 in^4
Ie	64.00 in^4
delta	150
r_t (maximum tensile reinforcement)	0.0166
r_{temp} (min. temperature reinforcement)	0.0014
r_{min} (minimum tensile reinforcement)	0.0027
r_{trial} (trial reinforcement ratio bottom)	0.0033
$\rho_{provided}$ (reinforcement ratio provided)	0.0090

Flexure

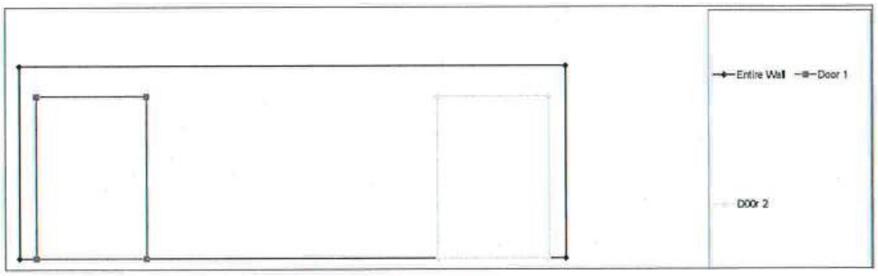
Assumption check		
Span	Hw	Lw
net Tensile Strain	0.010	0.010
Check ACI 14.8.2.3	Tension	Tension
Mua	1.068 kip-ft	

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

ACI 9.3.2		
fb	0.9	0.9
fMn trial = ϕAsFy(dt - 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
Ast = As + As add'l	0.20 in^2	0.20 in^2
fMn = ϕAsFy(db - a/2)	2.016 kip-ft	2.016 kip-ft
Check ϕMn > Mu	O.K.	O.K.
% allowed	63.49%	0.00%

PFS CORPORATION
Approval Limited to Factory Built Portion Only

State: Florida
Signature: 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)	0.05 ksf

Material Properties	
db (effective depth bottom)	1.84 in
a (block of strain)	0.33483 psi
	a=As * fy / (0.85 * f'c *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
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
D00r 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	ϕMn = ϕAsFy(db - a/2)	Check ϕMn > Mu
Door 1	0.9	0.006 in^2	No. 3	1	6.69 kip-ft	O.K.
D00r 2	0.9	0.006 in^2	No. 3	1	6.69 kip-ft	O.K.

CONNECTIONS

			Full Resistance Value				
			Overturning				
Base Anchors			Lateral	Base Anchors		Wall-Wall Connection	
Quantity in Shear	Maximum R - Distance	Maximum L - Distance	Shear kip	Moment + kip - ft	Moment - kip - ft	Moment + kip - ft	Moment - kip - ft
4	193	193	31.085	89.40	89.40	129.25	129.25

Total Tension	Base Anchors					
14.048	Dist	Tension (kip)	Shear	L - Dist	Moment +	Moment -
Base Anchor 1	3 in	3.38	3.33	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	3.38	3.33	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	*s 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	7.637	53.98%	W3	0	196.000	3.062	0.000	50.013
Wall Connection 2	2	2.703	4.852	50.00%	W5	75.5	120.500	4.852	30.524	48.717
Wall Connection 3	2	2.703	4.852	50.00%	W6	120.5	75.500	4.852	48.717	30.524
Wall Connection 4	2	1.531	7.637	53.98%	W4	196	0.000	3.062	50.013	0.000

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			Reserve Capacity
1543	31085	29543	82	12789	OK	386	(29543)	OK

RIGIDITY

CALCULATED VALUES	63%	Final	7.784414098

	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
Door 1	A'	196	82.28	Y	Y	15.000	0.067
	A	6	82.28	Y	N	0.000	0.000
	B	149.92	82.28	Y	Y	11.039	0.091
D00r 2	B'	196	82.28	Y	Y	15.000	0.067
	C	150	82.28	Y	Y	11.046	0.091
	D	5.92	82.28	Y	N	0.000	0.000

	Combine Logic					
	First Segment	Second Segment	Re-Name	Combine/Subtract	Method	Combined
Door 1	Entire Wall	A'	A'a	-	Deflection	0.014
	A	B	AB	+	Stiffness	11.039
	A'a	AB	A'b	+	Deflection	0.105
D00r 2	A'b	B'	B'a	-	Deflection	0.038
	C	D	CD	+	Stiffness	11.046
	B'a	CD	Final	+	Deflection	0.128

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State:	Florida
Signature:	Mark Severson
Title:	**Staff Plan Reviewer**
Date:	4/6/22

ID:	Cortez CR-1598
	DESIGN OF WALL MARKED W3

Notes	

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-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*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*f'c	ACI 14.8.2.6

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	O.K.
E (Steel)	29000000 psi	
E (Concrete)	4290000 psi	ACI 8.5.1
n (modular ratio)	6.76	

Shear Parameters		
Phi.v	0.85	ACI 9.3.2.3
Vc	3.123 kip	ACI 11.3.1.1 & 11.2.1.2
Phi*Vc	2.654 kip	ACI 11.1.1

Minimum Wall Reinforcement Requirements

roe.min.vert	0.0012	ACI 14.3.2
roe.min.hoz	0.002	ACI 14.3.3
Max Vertical spacing	18 in	ACI 14.3.5
Max Horizontal spacing	18 in	ACI 14.3.5

Geometric Properties

X Corridinate	14
Y Corridinate	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
lop (length 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-way slab
b (section width)	12 in
h (section thickness)	4 in
ct (cover top)	2 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

Loading

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

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 Wr	584.805

Axial Pressure on Section	
PuB	1.93 kip

Assumption check	
Pu/Ag	40.208 psi
0.06*f'c	300 psi
Check ACI 14.8.2.6	O.K.

Unfactored Axially Applied Loads

Unfactored Pressure on Roof uWr	353.1375 psf

Axial Pressure on Section	
PB	1.26 kip

Factored Laterally Applied Loads

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

Lateral Pressure on Section	
Lw = W*(L^4 / L^4 + H^4)	0.03 klf
Hw = W*(H^4 / H^4 + L^4)	0.09 klf

Unfactored Laterally Applied Loads

Unfactored Pressure on Wall uWw	75.54 psf

Lateral Pressure on Section	
Lw = W*(L^4 / L^4 + H^4)	0.02 klf
Hw = W*(H^4 / H^4 + L^4)	0.06 klf

Shear

Factored Loading per ACI	ACI eq. 9-3
Vu = wuB*(Bw-2db) / 2	0.09
Phi*Vc/2	1.33
Check Shear ACI 11.5.5.1	O.K.

Deflection ACI 14.8.4

Service Loads	
Axial	1.26 kip
Lateral	0.02 klf
Allowed service deflection	0.58 in
Msa	2.207 kip-in
M	2.215 kip-in
Ds	0.006 in
Check deflection	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
I cr	2.92 in^4
e_c	0.003
e_t	0.005
a	0.33483 psi
c	0.419 in
Ase	0.23 in^2
Icrdeflection	3.60 in^4
Ie	64.00 in^4
delta	150
r_t (maximum tensile reinforcement)	0.0166
r_{temp} (min. temperature reinforcement)	0.0014
r_{min} (minimum tensile reinforcement)	0.0027
r_{trial} (trial reinforcement ratio bottom)	0.0033
$\rho_{provided}$ (reinforcement ratio provided)	0.0090

Flexure

Assumption check		
Span	Hw	Lw
net Tensile Strain	0.010	0.010
Check ACI 14.8.2.3	Tension	Tension
Mua	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
fMn trial = ϕAsFy(dt - 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
Ast = As + As add'l	0.20 in^2	0.20 in^2
fMn = ϕAsFy(db - a/2)	2.016 kip-ft	2.016 kip-ft
Check ϕMn > Mu	O.K.	O.K.
% allowed	38.19%	16.37%

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PFS CORPORATION

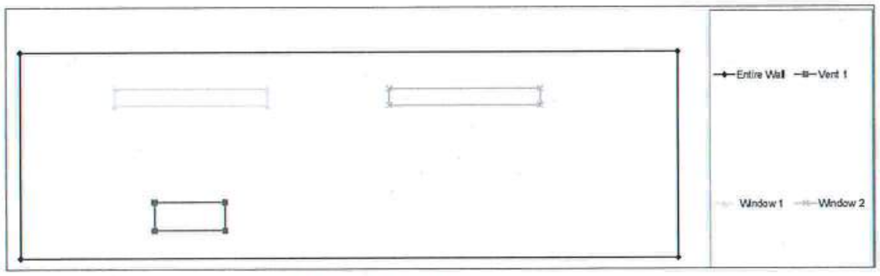
Approval Limited to Factory Built Portion Only

State: Florida

Signature: PFS Mark Peverson

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)	0.05 ksf

Material Properties	
db (effective depth bottom)	1.84 in
a (block of strain)	0.33483 psi
	a=As * fy / (0.85 * f'c *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
Vent 1	1.92 ft	1 ft	1 ft	5.25 ft	50.00	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

Flexure

Opening	ϕb	As req'd	Bar size	qty req'd	ϕMn = ϕAsFy(db - a/2)	Check ϕMn > Mu
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 in Shear	Maximum R - Distance	Maximum L - Distance	Shear kip	Moment + kip - ft	Moment - kip - ft	Moment + kip - ft	Moment - kip - ft
3	98	94	36.627	40.74	39.55	30.73	38.64

Total Tension	Base Anchors					
10.923	Dist	Tension (kip)	Shear	L - Dist	Moment +	Moment -
Base Anchor 1	19 in	3.84	12.21	94 in	1.118 kip*ft	28.521 kip*ft
Base Anchor 2	56.5 in	3.64	12.21	56.5 in	9.883 kip*ft	10.304 kip*ft
Base Anchor 3	98 in	3.64	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	3.826	19.26%	W1	2	111.000	3.826	0.638	35.394
Wall Connection 2	2	2.703	3.541	19.26%	W2	102	11.000	3.541	30.095	3.246

Wall Shear Checks

Shear Connections at Base			Wall Shear Capacity					
Design Force (lb)	Capacity (lb)	Reserve Capacity	Design (PLF)	Resistance (PLF)	check	Required Shear Capacity (lb) per Base Connector		Reserve Capacity
994	36627	35633	90	19239	OK	331	(35633)	OK

RIGIDITY

CALCULATED VALUES	94%	Final	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	12	Y	Y	62.543	0.016
	A	23.04	12	Y	Y	11.739	0.085
	B	77.96	12	Y	Y	42.972	0.023
Window 1	B'	113	6.96	Y	Y	108.101	0.009
	C	16.44	6.96	Y	Y	14.859	0.067
	D	70.52	6.96	Y	Y	67.329	0.015
Window 2	C'	113	6.96	Y	Y	108.101	0.009
	E	63.48	6.96	Y	Y	60.562	0.017
	F	23.48	6.96	Y	Y	21.850	0.046

	Combine Logic					
	First Segment	Second Segment	Re-Name	Combine/Subtract	Method	Combined
Vent 1	Entire Wall	A'	A'a	-	Deflection	0.122
	A	B	AB	+	Stiffness	54.710
	A'a	AB	A'b	+	Deflection	0.141
Window 1	A'b	B'	B'a	-	Deflection	0.131
	C	D	CD	+	Stiffness	82.189
	B'a	CD	B'b	+	Deflection	0.144
Window 2	B'b	C'	C'a	-	Deflection	0.134
	E	F	EF	+	Stiffness	82.412
	C'a	EF	Final	+	Deflection	0.146

PFS
PFS CORPORATION
Approval Limited to Factory Built Portion Only

State: Florida
Signature: Mark Peverson
Title: Staff Plan Reviewer
Date: 4/6/22

ID:	Cortez CR-1598
	DESIGN OF WALL MARKED W4

Notes	

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	O.K.
E (Steel)	29000000 psi	
E (Concrete)	4290000 psi	ACI 8.5.1
n (modular ratio)	6.76	

Shear Parameters		
Phi.v	0.85	ACI 9.3.2.3
Vc	3.123 kip	ACI 11.3.1.1 & 11.2.1.2
Phi*Vc	2.654 kip	ACI 11.1.1

Minimum Wall Reinforcement Requirements		
roe.min.vert	0.0012	ACI 14.3.2
roe.min.hor	0.002	ACI 14.3.3
Max Vertical spacing	18 in	ACI 14.3.5
Max Horizontal spacing	18 in	ACI 14.3.5

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-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*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*f'c	ACI 14.8.2.6

Geometric Properties	
X Corridinate	214
Y Corridinate	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
lop (length 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-way slab
b (section width)	12 in
h (section thickness)	4 in
ct (cover top)	2 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

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

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 Wr	584.805

Axial Pressure on Section	
PuB	1.93 kip

Assumption check	
Pu/Ag	40.208 psi
0.06*f'c	300 psi
Check ACI 14.8.2.6	O.K.

Unfactored Axially Applied Loads	
Unfactored Pressure on Roof uWr	353.1375 psf

Axial Pressure on Section	
PB	1.26 kip

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

Lateral Pressure on Section	
Lw= W*(L^4 / L^4 + H^4)	0.03 klf
Hw= W*(H^4 / H^4 + L^4)	0.09 klf

Unfactored Laterally Applied Loads	
Unfactored Pressure on Wall uWw	75.54 psf

Lateral Pressure on Section	
Lw= W*(L^4 / L^4 + H^4)	0.02 klf
Hw= W*(H^4 / H^4 + L^4)	0.06 klf

Shear	
Factored Loading per ACI	ACI eq. 9-3
Vu = wuB*(Bw-2db) / 2	0.09
Phi*Vc/2	1.33
Check Shear ACI 11.5.5.1	O.K.

Deflection		ACI 14.8.4
Service Loads		
Axial	1.26 kip	
Lateral	0.02 klf	
Allowed service deflection	0.58 in	
Msa	2.207 kip-in	
M	2.215 kip-in	
Ds	0.006 in	
Check deflection	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
I cr	2.92 in^4
e_c	0.003
e_t	0.005
a	0.33483 psi
c	0.419 in
Ase	0.23 in^2
Icrdeflection	3.60 in^4
Ie	64.00 in^4
delta	150
r_t (maximum tensile reinforcement)	0.0166
r_{temp} (min. temperature reinforcement)	0.0014
r_{min} (minimum tensile reinforcement)	0.0027
r_{trial} (trial reinforcement ratio bottom)	0.0033
$\rho_{provided}$ (reinforcement ratio provided)	0.0090

Flexure		
Assumption check		
Span	Hw	Lw
net Tensile Strain	0.010	0.010
Check ACI 14.8.2.3	Tension	Tension
Mua	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
fMn trial = ϕAsFy(dt - 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
Ast = As + As add'l	0.20 in^2	0.20 in^2
fMn = ϕAsFy(db - a/2)	2.016 kip-ft	2.016 kip-ft
Check ϕMn > Mu	O.K.	O.K.
% allowed	38.19%	16.37%

PFS
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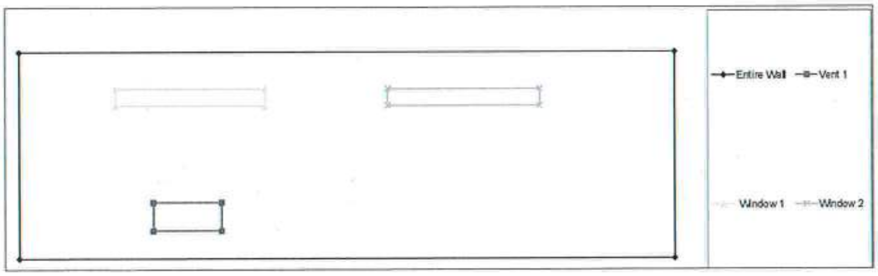
State: Florida
Signature: PFS Mark Severson
Title: Staff Plan Reviewer
Date: 4/6/22



REINFORCEMENT AT OPENINGS

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

Material Properties	
db (effective depth bottom)	1.84 in
a (block of strain)	0.33483 psi
	a=As * fy / (0.85 * f'c *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
Vent 1	1.92 ft	1 ft	1 ft	5.25 ft	50.00	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

Flexure

Opening	ϕb	As req'd	Bar size	qty req'd:	ϕMn = ϕAsFy(db - a/2)	Check ϕMn ~ Mu
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 in Shear	Maximum R - Distance	Maximum L - Distance	Shear kip	Moment + kip - ft	Moment - kip - ft	Moment + kip - ft	Moment - kip - ft
3	98	94	36.627	40.74	39.55	30.73	38.64

Total Tension	Base Anchors					
10.923	16	Tension (kip)	Shear	L - Dist	Moment +	Moment -
Base Anchor 1	19 in	3.64	12.21	94 in	1.118 kip*ft	28.521 kip*ft
Base Anchor 2	56.5 in	3.64	12.21	56.5 in	9.883 kip*ft	10.304 kip*ft
Base Anchor 3	98 in	3.64	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	3.826	19.26%	W1	2	111.000	3.826	0.638	35.394
Wall Connection 2	2	2.703	3.541	19.26%	W2	102	11.000	3.541	30.095	3.246

Wall Shear Checks

Shear Connections at Base			Wall Shear Capacity					
Design Force (lb)	Capacity (lb)	Reserve Capacity	Design (PLF)	Resistance (PLF)	check	Required Shear Capacity (lb) per Base Connector		Reserve Capacity
994	36627	35633	90	19239	OK	331	(35633)	OK

RIGIDITY

CALCULATED VALUES	94%	Final	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	12	Y	Y	62.543	0.016
	A	23.04	12	Y	Y	11.739	0.085
	B	77.96	12	Y	Y	42.972	0.023
Window 1	B'	113	6.96	Y	Y	108.101	0.009
	C	16.44	6.96	Y	Y	14.859	0.067
	D	70.52	6.98	Y	Y	67.329	0.015
Window 2	C'	113	6.96	Y	Y	108.101	0.009
	E	63.48	6.96	Y	Y	60.562	0.017
	F	23.48	6.96	Y	Y	21.850	0.046

	Combine Logic					
	First Segment	Second Segment	Re-Name	Combine/Subtract	Method	Combined
Vent 1	Entire Wall	A'	A'a	-	Deflection	0.122
	A	B	AB	+	Stiffness	54.710
	A'a	AB	A'b	+	Deflection	0.141
Window 1	A'b	B'	B'a	-	Deflection	0.131
	C	D	CD	+	Stiffness	82.189
	B'a	CD	B'b	+	Deflection	0.144
Window 2	B'b	C'	C'a	-	Deflection	0.134
	E	F	EF	+	Stiffness	82.412
	C'a	EF	Final	+	Deflection	0.146

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State: Florida
Signature: Mark Severson
Title: Staff Plan Reviewer
Date: 4/6/22

ID:	Cortez CR-1598
	DESIGN OF WALL MARKED W5

Notes	

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	O.K.
E (Steel)	29000000 psi	
E (Concrete)	4290000 psi	ACI 8.5.1
n (modular ratio)	6.76	

Shear Parameters

Phi.v	0.85	ACI 9.3.2.3
Vc	2.274 kip	ACI 11.3.1.1 & 11.2.1.2
Phi*Vc	1.933 kip	ACI 11.1.1

Minimum Wall Reinforcement Requirements

roe.min.vert	0.0012	ACI 14.3.2
roe.min.hor	0.002	ACI 14.3.3
Max Vertical spacing	18 in	ACI 14.3.5
Max Horizontal spacing	18 in	ACI 14.3.5

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-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*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*f'c	ACI 14.8.2.6

Geometric Properties

X Corridinate	91.5
Y Corridinate	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 that supports 2 roof panels?	No
lop (length of opening on wall)	0 ft
H (height of wall)	87 in
Lh (length of wall)	7.917 ft
Analysis will be performed as	Two-way slab
b (section width)	12 in
h (section thickness)	3 in
ct (cover top)	1 1/2 in
cb (cover bottom)	1 1/2 in
rd (assumed reinf. diameter)	0.319 in
dt (effective depth top)	1.34 in
db (effective depth bottom)	1.34 in
Ca (% of DL used for Seismic)	0.036
Eccentricity - Axial Load	1 in
Is wall Split	No

Loading

Axial Design Loads (pressure from roof)		Lateral Design Loads (pressure on wall)	
D (Dead load) + Ww (Wall weight)	98.44 psf	Dead Load (DL.lat)	0 psf
S (Snow Load)	250 psf	Snow Load (SL.lat)	0 psf
L (Live Load)	0 psf	Live Load (LL.lat)	0 psf
Lr (Live Roof Load)	30 psf	Live Roof Load (LLr.lat)	0 psf
W (Wind Load)	139.6 psf	Wind Load (WL.lat)	73.54 psf
E (Earthquake Load)	2.19 psf	Earthquake Load (EL.lat)	1.35 psf

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 Wr	584.805

Axial Pressure on Section	
PuB	1.92 kip

Assumption check	
Pu/Ag	53.333 psi
0.06*f'c	300 psi
Check ACI 14.8.2.6	O.K.

Unfactored Axially Applied Loads

Unfactored Pressure on Roof uWr	353.1375 psf

Axial Pressure on Section	
PB	1.25 kip

Factored Laterally Applied Loads

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

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

Unfactored Laterally Applied Loads

Unfactored Pressure on Wall uWw	73.54 psf

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

Shear

Factored Loading per ACI	ACI eq. 9-3
Vu = wuB*(Bw-2db) / 2	0.13
Phi*Vc/2	0.97
Check Shear ACI 11.5.5.1	O.K.

Deflection ACI 14.8.4

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

Allowable Capacity

Ig = (b*h^3)/12	27 in^4
Ag = (b*h)	36 in^2
Yt = h/2	1.5
fr (rupture modulus)	530.330 psi
Mcr	9.546 kip-in
Beta_1	0.8
Trial Ast req'd	0.053 in^2
B	8.836162648
kd	0.449 in
I.cr	1.44 in^4
ε_c	0.003
ε_s	0.005
a	0.33483 psi
c	0.419 in
Ase	0.22 in^2
Icrdeflection	1.73 in^4
Ie	27.00 in^4
delta	150
r_t (maximum tensile reinforcement)	0.0166
r_{temp} (min. temperature reinforcement)	0.0014
r_{min} (minimum tensile reinforcement)	0.0027
r_{trial} (trial reinforcement ratio bottom)	0.0033
$\rho_{provided}$ (reinforcement ratio provided)	0.0120

Flexure

Assumption check		
Span	Hw	Lw
net Tensile Strain	0.007	0.007
Check ACI 14.8.2.3	Tension	Tension
Mua	0.540 kip-ft	

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

ACI 9.3.2		
fb	0.9	0.9
fMn trial = ϕAsFy(dt - a 2)	1.410 kip-ft	1.410 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
Ast = As + As add'l	0.20 in^2	0.20 in^2
fMn = ϕAsFy(db - a/2)	1.413 kip-ft	1.413 kip-ft
Check ϕMn > Mu	O.K.	O.K.
% allowed	52.37%	27.60%

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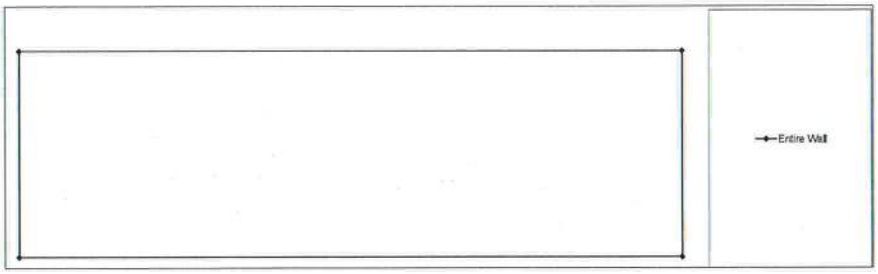
State: Florida
Signature: PFS Mark Severson
Title: Staff Plan Reviewer
Date: 4/6/22



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=As * fy / (0.85 * f'c *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

Flexure

Opening	ϕb	As req'd	Bar size	qty req'd	ϕMn = ϕAsFy(db - a/2)	Check ϕMn > Mu

CONNECTIONS

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

Total Tension	Base Anchors					
7.168	Dist	Tension (kip)	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								Overturning Moment Resistance (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	Up Left	Low Right
Wall Connection 1	2	1.531	6.107	30.74%	W1	0	95.000	3.062	0.000	24.241
Wall Connection 2	2	1.531	5.651	30.74%	W2	95	0.000	3.062	24.241	0.000

Wall Shear Checks

Shear Connections at Base			Wall Shear Capacity					
Design Force (lb)	Capacity (lb)	Reserve Capacity	Design (PLF)	Resistance (PLF)	check	Required Shear Capacity (lb) per Base Connector		Reserve Capacity
645	20474	19829	67	15274	OK	322	(19829)	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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State: Florida
Signature: PFS Mark Severson
Title: Staff Plan Reviewer
Date: 4/6/22

ID:	Cortez CR-1598
	DESIGN OF WALL MARKED W6

Notes	

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-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*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*f'c	ACI 14.8.2.6

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	O.K.
E (Steel)	29000000 psi	
E (Concrete)	4290000 psi	ACI 8.5.1
n (modular ratio)	6.76	

Shear Parameters

Phi.v	0.85	ACI 9.3.2.3
Vc	2.274 kip	ACI 11.3.1.1 & 11.2.1.2
Phi*Vc	1.933 kip	ACI 11.1.1

Minimum Wall Reinforcement Requirements

roe.min.vert	0.0012	ACI 14.3.2
roe.min.hor	0.002	ACI 14.3.3
Max Vertical spacing	18 in	ACI 14.3.5
Max Horizontal spacing	18 in	ACI 14.3.5

Geometric Properties

X Corridinate	136.5
Y Corridinate	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 2 roof panels?	No
lop (length of opening on wall)	0 ft
H (height of wall)	87 in
Lh (length of wall)	7.917 ft
Analysis will be performed as	Two-way slab
b (section width)	12 in
h (section thickness)	3 in
ct (cover top)	1 1/2 in
cb (cover bottom)	1 1/2 in
rd (assumed reinf. diameter)	0.319 in
dt (effective depth top)	1.34 in
db (effective depth bottom)	1.34 in
Cs (% of DL used for Seismic)	0.036
Eccentricity - Axial Load	1 in
Is wall Split	No

Loading

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

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 Wr	584.805

Axial Pressure on Section	
PuB	1.92 kip

Assumption check	
Pu/Ag	53.333 psi
0.06*f'c	300 psi
Check ACI 14.8.2.6	O.K.

Factored Laterally Applied Loads

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

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

Unfactored Laterally Applied Loads

Unfactored Pressure on Wall uWw	75.54 psf

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

Unfactored Axially Applied Loads

Unfactored Pressure on Roof uWr	353.1375 psf

Axial Pressure on Section	
PB	1.25 kip

Shear

Factored Loading per ACI	ACI eq. 9-3
Vu = wuB*(Bw-2db) / 2	0.13
Phi*Vc/2	0.97
Check Shear ACI 11.5.5.1	O.K.

Deflection (ACI 14.8.4)

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

Allowable Capacity

Ig = (b*h^3)/12	27 in^4
Ag = (b*h)	36 in^2
Yt = h/2	1.5
fr (rupture modulus)	530.330 psi
Mcr	9.546 kip-in
Beta_1	0.8
Trial Ast req'd	0.053 in^2
B	8.836162648
kd	0.449 in
I.cr	1.44 in^4
e_c	0.003
e_t	0.005
a	0.33483 psi
c	0.419 in
Ase	0.22 in^2
Icrdeflection	1.73 in^4
Ie	27.00 in^4
delta	150
r_t (maximum tensile reinforcement)	0.0166
r_{temp} (min. temperature reinforcement)	0.0014
r_{min} (minimum tensile reinforcement)	0.0027
r_{trial} (trial reinforcement ratio bottom)	0.0033
$\rho_{provided}$ (reinforcement ratio provided)	0.0120

Flexure

Assumption check		
Span	Hw	Lw
net Tensile Strain	0.007	0.007
Check ACI 14.8.2.3	Tension	Tension
Mua	0.540 kip-ft	

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

ACI 9.3.2		
fb	0.9	0.9
fMn trial = ϕAsFy(dt - a 2)	1.410 kip-ft	1.410 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
Ast = As + As add'l	0.20 in^2	0.20 in^2
fMn = ϕAsFy(db - a/2)	1.413 kip-ft	1.413 kip-ft
Check ϕMn > Mu	O.K.	O.K.
% allowed	52.37%	27.60%

PFS CORPORATION

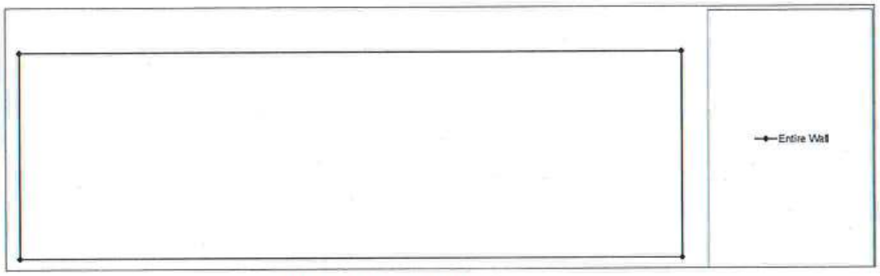
Approval Limited to Factory Built Portion Only

State: Florida

Signature: 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=As * fy / (0.85 * f'c *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

Flexure

Opening	ϕb	As req'd	Bar size	qty req'd	ϕMn = ϕAsFy(db - a/2)	Check ϕMn > Mu

CONNECTIONS

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

Total Tension	Base Anchors					
7.168	Dist	Tension (kip)	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	Overturning Moment Resistance (kip-ft) Low Right
Wall Connection 1	2	1.531	6.107	30.74%	W1	0	95.000	3.062	0.000	24.241
Wall Connection 2	2	1.531	5.651	30.74%	W2	95	0.000	3.062	24.241	0.000

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			Reserve Capacity
645	20474	19829	67	15274	OK	322	(19829)	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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Signature: Mark Severson
Title: Staff Plan Reviewer
Date: 4/6/22

ID: **Cortez CR-1598**

DESIGN OF FLOOR PANEL F1

Material Properties

f ' c	5000 psi	
Steel Reinforcement	Plain WWF Grade 80	
Fy	80000 psi	
Lightweight?	No	
C_d (Concrete density)	150 pcf	O.K.
E (Steel)	29000000 psi	
E (Concrete)	4286826 psi	*ACI 8.5.1*
n (modular ratio)	6.76	*ACI 14.0*

Geometric Properties

Ls (overall length of slab)	17 ft		
Bs (overall width of slab)	9.83 ft		
Design will be performed as :	**Two-way slab**		
tfr (floor finish thickness)	0 in		
b (section width)	12 in	*(typically 12 inches)*	
h (section thickness)	5 in		
ct (cover top)	1 in		
cb (cover bottom)	1 1/2 in		
rd (assumed reinf. diameter)	0.319 in	*(if centered enter 0)*	
l (length of slab for deflection)	29.143 in		
dt (effective depth top)	1.160 in		
db (effective depth bottom)	3.181 in		
oh1 (overhang length and qty for Bs)	0 in	0	*(qty of overhangs in Bs direction)*
oh2 (overhang length and qty for Ls)	0 in	0	*(qty of overhangs in ls direction)*
Cs (% of DL used for Seismic)	0.036	*(from seismic analysis)*	
NsL (Num. of supports along Ls)	8	*(either walls of vaults or enter "8" if no vault)*	
NsB (Num. of supports along Bs)	4	*(either walls of vaults or enter "4" if no vault)*	

Reinforcement Limits

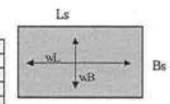
ρ_t (maximum tensile reinforcement)	0.0166	*ACI 10.3.3*
ρ_{temp} (min. temperature reinforcement)	0.0014	*ACI 7.12.2*
ρ_{min} (minimum tensile reinforcement)	0.0027	*ACI 10.5.1*
ρ_{trial} (trial reinforcement ratio bottom)	0.0033	
ρ_{trial} (trial reinforcement ratio top)	0.0033	

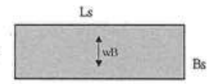
Loading

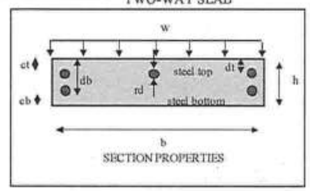
Design Loads	
Pressure on Slab	**w**
D (Dead load))	62.5 psf
S (Snow Load)	0 psf
L (Live Load)	0 psf
LF (Live Floor Load)	400 psf
W (Wind Load)	0 psf
E (Earthquake Load)	2.25 psf

Sustained Loading	
Pressure on slab	**W**
D (Dead load)	62.5 psf
S (Snow Load)	0 psf
Lr (Live Floor Load)	400 psf

Notes:







fr (rupture modulus)	530.3 psi	*ACI 9.5.2.3*
Ig = (b*h^3)/12	125 in^4	
Ag = (b*h)	60 in^4	
Yt = h/2	2.5 in^4	
Mcr	26.51650429	*ACI 9.5.2.3*
Beta_1	0.8	*ACI 10.2.7.3*
delta_initial	180	
delta_longterm	480	
B	8.830	
kd	0.412 in	
I.cr	1.04 in^4	
c	0 in	
a	0.32 in	

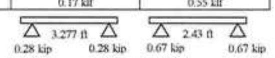
$\rho_{provided}$ (reinforcement ratio provided)	0.0053
ω =	0.0842
$\rho_{provided}$ (reinforcement ratio provided)	0.0154
ω =	0.2469

$\rho_{provided}$ (reinforcement ratio provided)	0.0044
ω =	0.0697
$\rho_{provided}$ (reinforcement ratio provided)	0.0118
ω =	0.1893

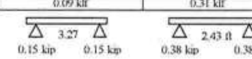
	Wire Mesh (Top)		
	Wire Size	W6.7	
Trial As' req'd	spacing	4 in	
0.13 in^2	Mesh Area	**0.20 in^2**	= As'

	Wire Mesh (Bottom)		
	Wire Size	W6.7	
Trial Ast required	spacing	4 in	
0.126 in^2	Mesh Area	**0.20 in^2**	= As

Factored Design Loads		Pressure on Section	Pressure on Section
Factored Loading per ACI equation indicated	Factored Pressure on Slab **W**	**wB** = W*(L^4 / B^4 + L^4)*be	**wL** = W*(B^4 / B^4 + L^4)*be
ACI eq. 9-3	715 psf	0.17 klf	0.55 klf

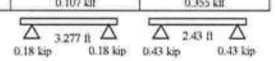


Unfactored Design Loads	Pressure on Section	Pressure on Section
Factored Pressure on Slab **W**	**wB** = W*(L^4 / B^4 + L^4)*be	**wL** = W*(B^4 / B^4 + L^4)*be
400 psf	0.09 klf	0.31 klf



B (Span in the short direction) = (Bs/NsB-1) - 0(oh1)	3.277 ft
L (Span in the long direction) = (Ls/NsL-1) - 0(oh2)	2.43 ft

Factored Sustained Loads		Pressure on Section	Pressure on Section
Factored Loading per ACI equation indicated	Factored Pressure on Slab **W**	**wB** = W*(L^4 / B^4 + L^4)*be	**wL** = W*(B^4 / B^4 + L^4)*be
ASCE7-05 eq CC1a	462.5 psf	0.107 klf	0.355 klf



Eficiency can be enhanced if Ast 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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Date: 4/6/22

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DESIGN OF FLOOR PANEL F1

Notes:

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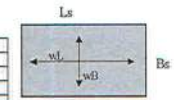
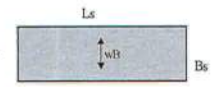


TWO-WAY SLAB



ONE-WAY SLAB

fr (rupture modulus)	530.3 psi	ACI 9.5.2.3
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Ag = (b*h)	60 in^4	
Yt = h/2	2.5 in^4	
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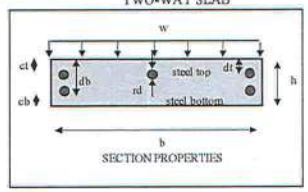
Beta_1	0.8	ACI 10.2.7.3
delta_initial	180	
delta_longterm	480	
B	8.830	
kd	0.412 in	
I cr	1.04 in^4	
c	0 in	
a	0.32 in	



$\rho_{provided}$ (reinforcement ratio provided)	0.0053
ω =	0.0842
$\rho_{provided}$ (reinforcement ratio provided)	0.0154
ω =	0.2469
$\rho_{provided}$ (reinforcement ratio provided)	0.0044
ω =	0.0697
$\rho_{provided}$ (reinforcement ratio provided)	0.0118
ω =	0.1893

Flexure

Flexural Moments for Bs	Mu	Tensile Strain	Check ACI 14.8.2.3	φb	φMn trial = φf'cbd^2ω(1-0.59ω)	ΔM = Mu - φM	φMn =	Check φMn > Mu	% allowed
** Mpos (positive Moment) = (wB*B^2)*0.08	0.15 kip-ft	0.036	Tension	0.9	3.64 kip-ft		3.64 kip-ft	O.K.	4.01%
** Mneg (negative Moment) = (wB*B^2)*0.1	0.18 kip-ft	0.036	Tension	0.9	4.44 kip-ft		4.44 kip-ft	O.K.	4.11%

**continuous beam moment coefficients used

Structural Plain Concrete per ACI 22.5

	Mu	S Elastic Section Modulus	φb	φMn = φ5*(f'c*S)^0.5	Check φMn > Mu	% allowed
Moh1 (Moment at oh1) = 0	0.00 kip-ft	0.029 ft^3	0.55	1.046 kip-ft	O.K.	0.00%

Flexural Moments for Ls	Mu	Tensile Strain	Check ACI 14.8.2.3	φb	φMn trial = φf'cbd^2ω(1-0.59ω)	ΔM = Mu - φM	φMn =	Check φMn > Mu	% allowed
** Mpos (positive Moment) = (wL*L^2)*0.078	0.25 kip-ft	0.036	Tension	0.9	3.64 kip-ft		3.64 kip-ft	O.K.	6.96%
** Mneg (negative Moment) = (wB*B^2)*0.106	0.11 kip-ft	0.036	Tension	0.9	4.44 kip-ft		4.44 kip-ft	O.K.	2.40%

**continuous beam moment coefficients used

Structural Plain Concrete per ACI 22.5

	Mu	S Elastic Section Modulus	φb	φMn = φ5*(f'c*S)^0.5	Check φMn > Mu	% allowed
Moh1 (Moment at oh1) = 0	0.00 kip-ft	0.029 ft^3	0.55	1.046 kip-ft	O.K.	0.00%

Shear

Maximum Shear for Bs	Vu	φv per ACI 9.3.2.3	Vc per ACI 11.3.1.1	φVc	Check φVc > Vu	% allowed
** VuB = wB * B * 0.6	0.33 kip	0.85	5.40 kip	4.59 kip	O.K.	7.28%
			per ACI 22.8			
** Voh1 = 0	0.00 kip	0.55	2.98 kip	1.64 kip	O.K.	0.00%

**continuous beam shear coefficients used

Shear for Ls	Vu	φv per ACI 9.3.2.3	Vc per ACI 11.3.1.1	φVc	Check φVc > Vu	% allowed
** VuL = wL * L * 0.605633802816901	0.52 kip	0.85	5.40 kip	4.59 kip	O.K.	11.39%
			per ACI 22.8			
** Voh2 = 0	0.00 kip	0.55	2.98 kip	1.64 kip	O.K.	0.00%

**continuous beam shear coefficients used

Deflection

Span type:	K	Sustained Load Duration	Epsilon
Simple span	1	6 months	1.2

	Service Loads														
Span	Mserv	M.sus	I.eff.serv	I.eff.sustained	Immediate Deflection Δi	ρ_{comp}	λ	Long-Term Deflection Δl-t	Δ total long-term deflection (Δi + Δl-t)	Δ allow (immediate)	Δ allow (long term)	Check short term deflection	Check long term deflection	% allowed - short term	% allowed - long term
B	0.15 kip-ft	0.14 kip-ft	125.00 in	125.00 in	0.001 in	0.0053	0.9500	0.000 in	0.001 in	0.2184 in	0.0819 in	O.K.	O.K.	0.24%	0.60%
L	0.25 kip-ft	0.26 kip-ft	125.00 in	125.00 in	0.001 in	0.0053	0.9500	0.000 in	0.001 in	0.1620 in	0.0608 in	O.K.	O.K.	0.32%	0.81%

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Signature: PFS Mark Severson

Title: Staff Plan Reviewer

Date: 4/6/22

ID: **Cortez CR-1598**

Geometric properties	
Bs (width of roof panel)	10.25 ft
Ls (Length of roof panel)	19.00 ft
Ar Area of Roof	194.75 ft^2
H (height of building)	9.37 ft
Lb (length of building)	9.42 ft
Wb (width of building)	17 ft
Ab (Area of building)	160.14 ft^2
Nv (quantity of vaults)	0
Avl (Area of Vault Lips)	0.00 ft^2
Av (Area of Vault)	0.00 ft^2
Vh (Vault height)	0 ft
Cab (Closed Area of building)	137.50 ft^2
Hw (depth of floodwater)	1 ft

Loading	
Wv (weight of vault)**	0 lb
Wtr (roof panel weight)	11820 lb
Ww (total walls panel weight)	21330 lb
Fw (floor panel weight)	10035 lb
We (estimated weight of building)	43185 lb
Wev (estimated weight of building w/ vault)	43185 lb
PSFr (roof snow load)	210 psf
PSFf (Floor Live Load)	400 psf
Pmax (Maximum allowable pressure)	1500 psf
Fupmw (MWFRS Uplift Force)	51.69 psf
WLlat (MWFRS lateral wind pressure)	61.91 psf
γw (specific weight of water)	62.4 pcf

**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		Fsreqd	
Factor of Safety	FSwind = Pslide / Vwind	FSwind =	2.3	≥	1.0	O.K.
	FSseismic = Pslide / Vseismic	Fseismic =	17.5	≥	1.0	O.K.
Sliding Resistance with No Snow	Pslide = u*.6*We	Pslide =	10364.4 lb		Fsreqd	
Factor of Safety	FSwind = Pslide / Vwind	Fswind =	1.1	≥	1.0	O.K.
	FSseismic = Pslide / Vseismic	Fseismic =	9.5	≥	1.0	O.K.

CHECK OVERTURNING RESISTANCE

Shear	.7*Otseismic (from seismic analysis with snow)	10.851 kip-ft
	.7*Otseismic (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		Fsreqd	
Factor of Safety	FSwind = Otrsnow / Otwind	FSwind =	2.43	≥	1.0	O.K.
	FSseismic = Otrsnow / Vseismic	Fseismic =	21.45	≥	1.0	O.K.
Overturning Resistance with No Snow	Otr = .6*We*Wb/2	Otr	220.244 kip-ft		Fsreqd	
Factor of Safety	FSwind = Otr / Vwind	Fswind =	2.30	≥	1.0	O.K.
	FSseismic = Otr / Vseismic	Fseismic =	24.28	≥	1.0	O.K.

CHECK BEARING PRESSURE CONDITION

Net Pressure	Pnet = (Wev + PSFr*Ar + PSFf*Af) / Ab	925.06 psf	
Allowable	Pmax ≥ Pnet	1500 psf ≥ 925.06 psf	O.K.

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 O.K.

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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