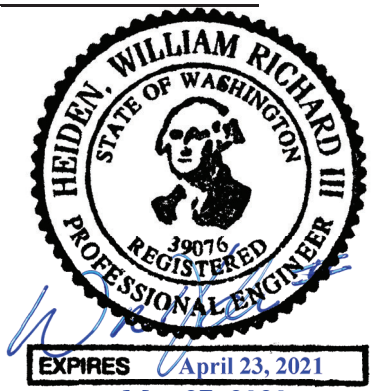
CXT Inc. (Precast Division)

Calculations
Ozark I OZI-252
Structural Analysis

Design Loads
400 psf Live Floor Load
250 psf Ground Snow Load
Wind Speed – 150 mph Exp. C
Seismic Design Category: D

Design Standards
2015 International Building Code
ASCE 7-10/ ACI 318-14

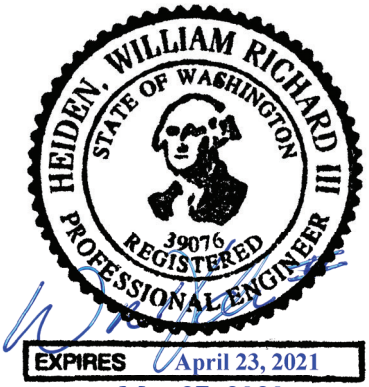
UL-752 Bullet Resistance
Classification: Level IV
Report #: 2012-647



May 27, 2020

Table of Contents

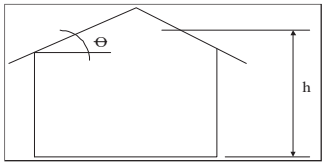
All attached documents are for reference only and designed or approved by others.



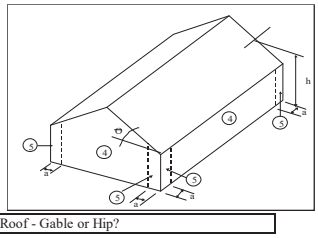
May 27, 2020

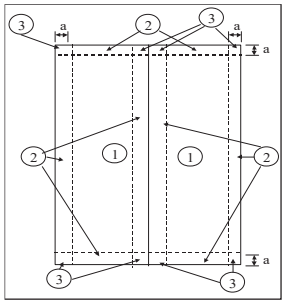
Main Wind Force Resisting System Loads (ASCE 7-10)

Ozark I OZI-252		
Category	II	IBC Table 1604.5: Risk Category of Buildings and Other Structures.
Exposure	C	See § 26.7.3: Exposure Categories, General.
Velocity	150 mph	See Figure 26.5A thru 26.5C: Basic Wind Speed (3 second Gust)
h.wind	8.00 ft	Windward wall height
h.lee	8.00 ft	Leeward wall height
W.building	10.5 ft	Width of the building
L.building	12 ft	Length of the building
H.building	9.75 ft	Height of the building (to the ridge). Enter 0 if unknown.
Roof Rise	3	Roof pitch (per foot)
ϑ	14.04 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	8.875 ft	Mean roof height
n_a	8.45	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	41.56 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	96.0 sq ft	408.4 sq ft	0 sq ft	0.00	0.00	0.00	0.00	Enclosed
Windward endwall	0 sq ft	93.2 sq ft	411.2 sq ft	0 sq ft	0.00	0.00	0.00	0.00	Enclosed
Leeward sidewall	0 sq ft	96.0 sq ft	408.4 sq ft	0 sq ft	0.00	0.00	0.00	0.00	Enclosed
Leeward endwall	0 sq ft	93.2 sq ft	411.2 sq ft	0 sq ft	0.00	0.00	0.00	0.00	Enclosed
Roof	0 sq ft	126.0 sq ft	378.4 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_{po}	0.8	See 27.4.4 Roof Overhangs		
Cp	0.8	Windward wall (Use with qz) Fig. 27.4-1		
	-0.500	Leeward wall (wind normal to ridge) (Use with qh)	L/B =	0.88
	-0.471	Leeward wall (wind parallel to ridge) (Use with qh)	L/B =	1.14
	-0.7	Sidewalls (Use with qh) Fig. 27.4-1		

Internal Pressures:	
Negative:	-7.48 psf
Positive:	7.48 psf

	Pos. Windward	Neg. Windward	Leeward
Roof Pressure Coefficients (Fig 27.4-1) Normal to Ridge when Theta >= 10degrees	-0.180	-0.959	-0.582

Roof Pressures Wind Perpendicular to Ridge w/ ϑ >= 10 deg	
w/ Negative Internal	1.12 psf
w/ Positive Internal	-41.36 psf

*WORST CASE LOADING

	0 to h/2	h/2 to h	h to 2h	> 2h
Roof Pressure Coefficients (Fig 27.4-1) Normal to Ridge when Theta < 10 deg.	-1.18	-0.76	-0.64	-0.58
Roof Pressure Coefficients (Fig 27.4-1) PARALLEL to Ridge	-1.09	-0.80	-0.60	-0.49

Wall Pressures:	w/ Negative	w/ Positive Internal
Windward	35.74 psf	20.78 psf
Leeward (wind normal)	-16.00 psf	-25.14 psf
Leeward (wind parallel)	-16.00 psf	-24.14 psf
Side Wall	-17.25 psf	-32.21 psf

Additional Overhang Pressure:	28.26 psf

Roof Pressures: Wind Parallel to ridge for all roof slopes:	
Location	w/ Positive Internal
0 to h/2	-46.05 psf
h/2 to h	-35.89 psf
h to 2h	-28.53 psf
Over 2h	-24.85 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:	150 mph	Roof Slope:	3.00 : 12	COMPONENTS & CLADDING	
Exposure:	C	Mean Roof Height:	8.88 ft		
	Effective Area				
Zone	10.0 sq ft		100.0 sq ft		500.0 sq ft	
1	-38.21 psf	19.98 psf	-34.05 psf	11.67 psf	-34.05 psf	11.67 psf
2	-71.45 psf	19.98 psf	-50.67 psf	11.67 psf	-50.67 psf	11.67 psf
2oh	-91.44 psf	-	-91.44 psf	-	-91.44 psf	-
3	-108.86 psf	19.98 psf	-83.92 psf	11.67 psf	-83.92 psf	11.67 psf
3oh	-153.78 psf	-	-103.90 psf	-	-103.90 psf	-
4	-46.52 psf	40.76 psf	-38.21 psf	33.70 psf	-34.05 psf	28.29 psf
5	-58.99 psf	40.76 psf	-46.52 psf	33.70 psf	-34.05 psf	28.29 psf
a:	3.00 ft					

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

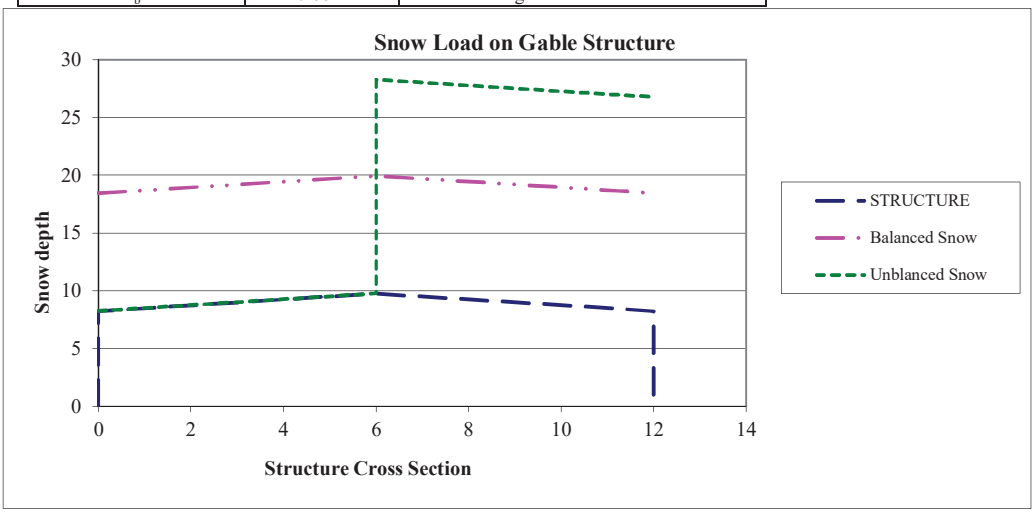
ASCE 7-10 SNOW LOAD CALCULATION

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 7-1: Ground Snow Load
W.building	10.5 ft	Length of the building
L.building	12 ft	Width of the building
H.building	9.75 ft	Height of the building (to the ridge). Enter 0 if unknown.
Roof Rise (per foot)	3	Roof pitch
ϑ	14.04 deg	Roof Angle

ASCE Table 7-3 - 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	5.25 ft	
Roof Exposure	Partially exposed	ASCE Table 7-2
C_e	1	ASCE Table 7-2
Cs	1	Slope Factor from Figure 7-2
Low Sloped?:	Yes	ASCE § 7-4
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	5.3 ft	ASCE Figure 7-5

γ	30.00 pcf	Snow density	Eq. 7.7-1 of ASCE 7
S	4	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	



Seismic Loads (ASCE 7-10)

Ozark I OZI-252			
Category	II	IBC Table 1604.5: Risk Category of Buildings and Other Structures.	
Ss	1.67 g	Max. Earthquake Ground Motion of 0.2 sec Spectral Response Acceleration	*ASCE Figure 22-1*
S_1	0.75 g	Max. Earthquake Ground Motion of 1.0 sec Spectral Response Acceleration	*ASCE Figure 22-2*
Site Class	D	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 22-12*
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	9.08 ft	Height in feet from base to highest level of structure	

				Value 1*	Value 2*	
F_a	1	Interpolated Value	*ASCE Table 11.4-1*	1	1	*=Used for interpolation
F_v	1.5	Interpolated Value	*ASCE Table 11.4-2*	1.5	1.5	

$S_{MS} = F_a * S_S$	1.669 g	Adjusted MCE Spectral Response Acceleration at short periods	*ASCE 11.4-1*
$S_{M1} = F_v * S_1$	1.122 g	Adjusted MCE Spectral Response Acceleration at 1 sec period	*ASCE 11.4-2*
		(MCE = Maximum considered earthquake)	

$S_{DS} = 2/3\ S_{M_s}$	1.113 g	Design Spectral Acceleration Parameters	*ASCE 11.4-3*
$S_{D1} = 2/3\ S_{M_1}$	0.748 g	Design Spectral Acceleration Parameters	*ASCE 11.4-4*

I_E	1	Importance Factor	*ASCE Table 1.5-2*

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

Geotechnical Investigation Report Required? **Yes per ASCE 11.8.2 and 11.8.3, IBC 1803**

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

$C_s = S_{DS}/(R/I)$	0.278	*ASCE 12.8-2*
$C_{s,min}$	0.094	*ASCE 12.8-5 & 12.8-6*
$C_{s,max}$	1.787	*ASCE 12.8-3 & 12.8-4*
C_s	0.278	
k	1.000	*ASCE 12.8.3*
W	45.14 kip	
$V = C_s * W$	31.39 kip	*ASCE 12.8-1* — *Shear **with** snow load*
M_o =	281.2 k-ft	*Overturning Moment **with** snow load*
$V = C_s * W$	26.17 kip	*Shear **without** snow load*
M_o =	171.1 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	$w_i * h_i^k$	*12.8-12* C_{vx}	*12.8-11;11.7* F_x	V_x (Story shear)	M_x	*12.10-1* F_{px} (diaphragm force)
Roof	8.88 ft	9.08 ft	210 psf	27.79 kip	252.4 k-ft	0.986	30.95 kip	30.95 kip	0.0 k-ft	12.37 kip
Walls	0.00 ft	0.00 ft								
Floor	0.21 ft	0.21 ft		17.35 kip	3.6 k-ft	0.014	0.44 kip	31.39 kip	274.6 k-ft	7.72 kip
Base	0 ft	0.00 ft	W=	45.14 kip	256.1 k-ft			M_o =	281.2 k-ft	

WITHOUT SNOW LOAD

Level	Story Height	h_i or h_x	P_f (flat roof snow load)	w_i	$w_i * h_i^k$	*12.8-12* C_{vx}	*12.8-11;11.7* F_x	V_x (Story shear)	M_x	*12.10-1* F_{px} (diaphragm force)
Roof	8.88 ft	9.08 ft	0 psf	20.29 kip	184.3 k-ft	0.720	18.83 kip	18.83 kip	0.0 k-ft	9.03 kip
Walls	0.00 ft	0.00 ft								
Floor	0.21 ft	0.21 ft		17.35 kip	3.6 k-ft	0.014	0.37 kip	19.20 kip	167.1 k-ft	7.72 kip
Base	0 ft	0.00 ft	W=	37.63 kip	187.9 k-ft			M_o =	171.1 k-ft	

Center of Mass & Rigidity

Ozark I OZI-252

			X	Y
	Upper Left = 0,0	Lower Right	180	143

Wall	X Relative Stiffness	Y Relative Stiffness	Shear Force lbs	Shear Force plf	Dist to CoRx dx (IN)	Dist to CoRy dy (IN)
W1	26.66%	0.00%	2,061	210	76.366	0.100
W2	43.84%	0.00%	3,389	345	3.634	0.053
W3	29.50%	0.00%	2,280	232	63.634	1.110
W4	0.00%	50.04%	3,869	322	5.384	60.947
W5	0.00%	49.96%	3,862	322	4.789	61.053

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
R1	4.5	5295	0	71.5	180	143	210	90.0	107.3	9049	5295
R2	4.5	5295	0	0	180	71.5	210	90.0	35.8	9049	5295
F1	5	7650	18	8.5	162	134.5	400	90.0	71.5	7650	0
Totals		19390						91.2	71.6		

Torsional Eccentricity ex	Torsional Eccentricity ey	Wgt (w snow)	Wgt (w/o snow)		wgt (w snow)	wgt (w/o snow)
5.17	0.13	45,138	37,630	roof	27,793	20,285
Center of Gravity X	Center of Gravity Y			floor	17,345	
91.2	71.6					
Center of Rigidty X	Center of Rigidty Y					
96.4	71.4					

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)	Toward Lower Right Anchor Resistance check	Toward Upper Left Anchor Resistance Moment (kip-ft)	Toward Upper Left Anchor Resistance check	Overturning status using just connection to adjacent walls
W1	26.83	44.21	OK	44.21	OK	None Required
W2	53.52	45.16	Need More	45.16	Need More	TRY BASE ANCHORS
W3	31.54	33.68	OK	33.68	OK	None Required
W4	49.76	148.29	OK	135.86	OK	None Required
W5	49.62	148.29	OK	135.86	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)	Toward Lower Right Anchor Resistance check	Toward Upper Left Anchor Resistance Moment (kip-ft)	Toward Upper Left Anchor Resistance check	Combined Loading Unity Check	Required Tension Capacity per Base Anchor (lb)
W1	26.83	55.72	OK	55.72	OK	OK	(884)
W2	53.52	42.09	Try Both	42.09	Try Both	Try Both	567
W3	31.54	55.72	OK	55.72	OK	OK	(109)
W4	49.76	52.30	OK	52.30	OK	Try Both	(4783)
W5	49.62	52.30	OK	52.30	OK	Try Both	(4791)

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

Wall	Base Anchor Shear Required (% Capacity)	Base Anchor Tension Available (% Capacity)	Available Overturning Resistance (kip-ft) From Base Anchors Lower Right	Available Overturning Resistance (kip-ft) From Base Anchors Upper Left	Overturning Unity Check of Base Anchors Lower Right	Overturning Unity Check of Base Anchors Upper Left
W1	15.2%	100.0%	99.92	99.92	OK	OK
W2	34.6%	85.4%	74.52	74.52	OK	OK
W3	16.5%	100.0%	89.40	89.40	OK	OK
W4	30.7%	89.3%	179.19	168.08	OK	OK
W5	30.6%	89.4%	179.28	168.17	OK	OK

ID: **Ozark I OZI-252**

DESIGN OF ROOF PANELS MARK R1, R2

Notes:

Material Properties

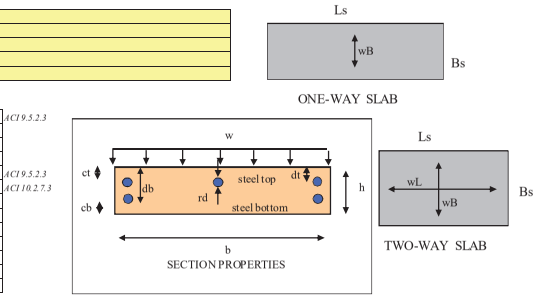
f ' c	5000 psi	
Steel Reinforcement	I. Plain WWF >=	W1.2-A185
Fy	65000 psi	
Lightweight?	No	
C_d (Concrete density)	150 pcf	O.K.
E (Steel)	29000000 psi	
E (Concrete)	4286826 psi	*ACI 8. 5. 1*
n (modular ratio)	6.76	*ACI 14.0*

Geometric Properties

Ls (overall length of slab)	15 ft		
Bs (overall width of slab)	6.09 ft		
Design will be performed as :	**One-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)	1 1/4 in		
cb (cover bottom)	3/4 in		
rd (assumed reinf. diameter)	0.319 in		*(if centered enter 0)*
dt (effective depth top)	1.4095 in		
db (effective depth bottom)	3.431 in		
oh1 (overhang length and qty for Bs)	8.5 in	1	*{qty of overhangs in Bs direction}*
oh2 (overhang length and qty for ls)	18 in	2	*{qty of overhangs in ls direction}*
Cs (% of DL used for Seismic)	0.278		*{from seismic analysis}*
NBs (qty of walls in Bs direction)	1		*{walls that support one or more roof panels in the short direction}*
Nls (qty of walls in ls direction)	3		*{walls that support one or more roof panels in the long direction}*

Reinforcement Limits

ρ_t (maximum tensile reinforcement)	0.0225	*ACI 10.3.3*
ρ_{temp} (min. tempenature reinforcement)	0.0017	*ACI 7. 12.2*
ρ_{min} (minimum tensile reinforcement)	0.0033	*ACI 10.5.1*
ρ_{trial} (trial reinforcement ratio bottom)	0.0033	
ρ_{trial} (trial reinforcement ratio top)	0.0033	



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	7.398
kd	0.837 in
I.cr	13.26 in^4
c	0 in
a	0.31 in

bottom mesh		
$\rho_{provided}$ (reinforcement ratio provided)		0.0058
top mesh	ω =	0.0754 psi
$\rho_{provided}$ (reinforcement ratio provided)		0.0142
both layers	ω =	0.1846 psi
$\rho_{provided}$ (reinforcement ratio provided)		0.0165
	ω =	0.2145 psi

	Wire Mesh (Top)		
	Wire Size	W8	
Trial As' req'd	spacing	4 in	
	Mesh Area	**0.24 in^2**	**= As'**

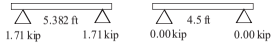
	Wire Mesh (Bottom)		
	Wire Size	W8	
Trial Ast required	spacing	4 in	
0.136 in^2	Mesh Area	**0.24 in^2**	**= As**

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)	108.86 psf
E (Earthquake Load)	16.95 psf

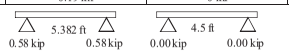
Sustained Loading	
Pressure on slab	**W**
D (Dead load)	60.938 psf
S (Snow Load)	250 psf
Lr (Live Roof Load)	30 psf

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	560.213 psf	0.56 klf	0 klf



B (Span in the short direction) = Bs-1(oh1)	5.382 ft
L (Span in the long direction) = Ls-2(oh2)	4.5 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 CC1b	185.938 psf	0.19 klf	0 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
330.0825 psf	0.89 klf	0 klf

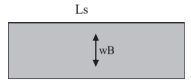
SUMMARY

Use 1 Layer of Wire Mesh on Top	W8 x W8 x 4 x 4
Use 1 Layer of Wire Mesh on Bottom	W8 x W8 x 4 x 4

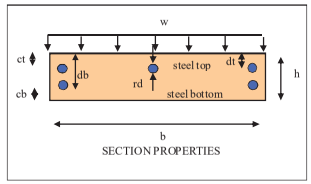
ID: **Ozark I OZI-252**

DESIGN OF ROOF PANELS MARK R1, R2

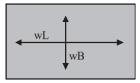
Notes:



ONE-WAY SLAB



SECTION PROPERTIES



TWO-WAY SLAB

Material Properties

f ' c	5000 psi	
Steel Reinforcement	I. Plain WWF >=	W1.2-A185
Fy	65000 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*

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	7.398	
kd	0.837 in	
I.cr	13.26 in^4	
c	0 in	
a	0.31 in	

Geometric Properties

Ls (overall length of slab)	15 ft		
Bs (overall width of slab)	6.09 ft		
Design will be performed as :	**One-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)	1 1/4 in		
cb (cover bottom)	3/4 in		
rd (assumed reinf. diameter)	0.319 in		*(if centered enter 0)*
dt (effective depth top)	1.4095 in		
db (effective depth bottom)	3.431 in		
oh1 (overhang length and qty for Bs)	8.5 in	1	*(qty of overhangs in Bs direction)*
oh2 (overhang length and qty for ls)	18 in	2	*(qty of overhangs in ls direction)*
Cs (% of DL used for Seismic)	0.278		*(from seismic analysis)*
NBs (qty of walls in Bs direction)	1		*(walls that support one or more roof panels in the short direction)*
Nls (qty of walls in ls direction)	3		*(walls that support one or more roof panels in the long direction)*

bottom mesh

$\rho_{provided}$ (reinforcement ratio provided)		0.0058
top mesh	ω =	0.0754 psi
$\rho_{provided}$ (reinforcement ratio provided)		0.0142
both layers	ω =	0.1846 psi
$\rho_{provided}$ (reinforcement ratio provided)		0.0165
	ω =	0.2145 psi

Flexure

One-way slab	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	2.03 kip-ft	0.045	Tension	0.9	3.82 kip-ft		3.82 kip-ft	O.K.	53.14%

	Structural Plain Concrete per ACI 22.5					
	Mu	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.	14.93%
Mneg (negative Moment) = (wB*oh2^2) / 2	0.140 kip-ft	0.023 ft^3	0.55	0.941 kip-ft	O.K.	14.93%

	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	0.000 kip-ft	0.045	Tension	0.9	3.82 kip-ft		3.82 kip-ft	O.K.	0.00%

	Structural Plain Concrete per ACI 22.5					
	Mu	S Elastic Section Modulus	ϕb	ϕMn = ϕ5*(f'c*S)^0.5	Check ϕMn > Mu	% allowed
Mneg (negative Moment) = (wB*oh1^2) / 2	0.630 kip-ft	0.023 ft^3	0.55	0.941 kip-ft	O.K.	66.95%
Mneg (negative Moment) = (wB*oh2^2) / 2	0.630 kip-ft	0.023 ft^3	0.55	0.941 kip-ft	O.K.	66.95%

Shear

One-way slab	Vu	ϕv per ACI 9.3.2.3	Vc per ACI 11.3.1.1	ϕVc	Check ϕVc > Vu	% allowed
Vu = wB (B/2)	1.51 kip	0.85	5.82 kip	4.95 kip	O.K.	30.45%
			per ACI 22.8			
Vu for side overhang 1 = wB*oh1	0.40 kip	0.55	2.98 kip	1.64 kip	O.K.	24.19%
Vu for side overhang 1 = wB*oh1	0.40 kip	0.55	2.98 kip	1.64 kip	O.K.	24.19%

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)	0.00 kip	0.85	5.82 kip	4.95 kip	O.K.	0.00%
			per ACI 22.8			
Vu for end overhang 2 = wL*oh2	0.00 kip	0.55	2.98 kip	1.64 kip	O.K.	0.00%
Vu for end overhang 2 = wL*oh2	0.00 kip	0.55	2.98 kip	1.64 kip	O.K.	0.00%

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

Deflection

	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 total long term deflection	% allowed - short term	% allowed - total long term
B	2.03 kip-ft	0.688 kip-ft	66.63 in^4	91.13 in^4	0.009 in	0.0058	0.9302	0.009 in	0.018 in	0.3588 in	0.1345 in	O.K.	O.K.	2.56%	13.17%
L	0 kip-ft	0 kip-ft	0.00 in^4	0.00 in^4	0.000 in	0.0058	0.9302	0.009 in	0.018 in	0.3000 in	0.1125 in	O.K.	O.K.	0.00%	15.75%

ID:	Ozark I OZI-252
	DESIGN OF WALL MARKED W1

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 >= W1.2-A185	
Fy wire mesh	65000 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*

Geometric Properties

X Corridinate	20
Y Corridinate	12.5
Direction of Wall	Y
Center of gravity X	20.000
Center of gravity Y	71.547
Wall Weight	3090.000 lbs.
Central wall?	Yes
Wall that supports 2 roof panels?	Yes
lop (length of opening on wall)	0 ft
H (height of wall)	103.37 in
Lh (length of wall)	9.833 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)	0.278
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)	108.86 psf	Wind Load (WL.lat)	58.99 psf
E (Earthquake Load)	16.95 psf	Earthquake Load (EL.lat)	13.91 psf

Wire Mesh		
Wire Size	W8	
spacing	4 in	
Mesh Area	0.24 in^2	= As

Factored Axially Applied Loads

Factored Loading per ACI	ACI eq. 9-3
Factored Pressure on Roof Wr	560.213

Axial Pressure on Section	
PuB	2.51 kip

Assumption check	
Pu/Ag	52.292 psi
0.06*f`c	300 psi
Check ACI 14.8.2.6	O.K.

Unfactored Axially Applied Loads

Unfactored Pressure on Roof uWr	330.0825 psf

Axial Pressure on Section	
PB	1.57 kip

Factored Laterally Applied Loads

Factored Loading per ACI	*ACI eq. 9-4*
Factored Pressure on Wall Ww	94.38 psf

Lateral Pressure on Section	
Lw = W*(L^4 / L^4 + H^4)	0.03 klf
Hw = W*(H^4 / H^4 + L^4)	0.06 klf

Unfactored Laterally Applied Loads

Unfactored Pressure on Wall uWw	58.99 psf

Lateral Pressure on Section	
Lw = W*(L^4 / L^4 + H^4)	0.02 klf
Hw = W*(H^4 / H^4 + L^4)	0.04 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	1.57 kip
Lateral	0.02 klf
Allowed service deflection	0.69 in
Msa	3.011 kip-in
M	3.030 kip-in
Ds	0.012 in
Check deflection	O.K.

Allowable Capacity

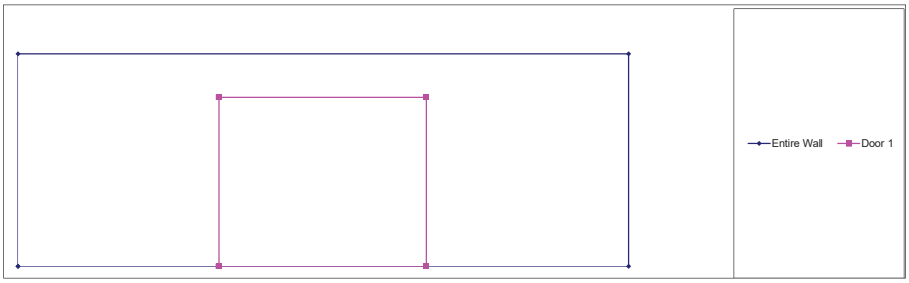
Ig = (b*h^3)/12	64 in^4
Ag = (b*h)	48 in^2
Yt = h/2	2
fr (rupture modulus)	530.330 psi
Mcr	16.971 kip-in
Beta_1	0.8
Trial Ast req'd	0.073 in^2
B	7.403686795
kd	0.583 in
I.cr	3.35 in^4
e_c	0.003
e_s	0.005
a	0.32469 psi
c	0.406 in
Asc	0.28 in^2
Icrdeflection	4.35 in^4
Ie	64.00 in^4
delta	150
r_t (maximum tensile reinforcement)	0.0225
r_{temp} (min. temperature reinforcement)	0.0017
r_{min} (minimum tensile reinforcement)	0.0033
r_{trial} (trial reinforcement ratio bottom)	0.0033
$\rho_{provided}$ (reinforcement ratio provided)	0.0110

Flexure

Assumption check		
Span	Hw	Lw
net Tensile Strain	0.011	0.011
Check ACI 14.8.2.3	Tension	Tension
Mua	0.665 kip-ft	

ACI eq. (14-6)		
Mu	0.830 kip-ft	0.360 kip-ft

ACI 9.3.2		
fb	0.9	0.9
fMn trial = ϕAsFy(dt - a/2)	1.960 kip-ft	1.960 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	0.24 in^2	0.24 in^2
fMn = ϕAsFy(db - a/2)	1.961 kip-ft	1.961 kip-ft
Check ϕMn > Mu	O.K.	O.K.
% allowed	42.33%	18.36%



REINFORCEMENT AT OPENINGS

Loading	
Pu (factorized load from roof)	0.56 klf
Ww (weight of panel per sq ft)	0.05 ksf

Material Properties	
db (effective depth bottom)	1.84 in
a (block of strain)	0.32469 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	3.24 ft	0 ft	3.34 ft	1.76 ft	1144.65	0.09 klf	0.65 klf	0.6 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	0.007 in^2	No. 3	1	9.67 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	110	110	40.946	55.72	55.72	44.21	44.21

Total Tension	Base Anchors					
14.334	Dist	Tension (kip)	Shear	L - Dist	Moment +	Moment -
Base Anchor 1	8 in	***3.53***	***8.26***	110 in	0.171 kip*ft	32.322 kip*ft
Base Anchor 2	32 in	3.64	12.21	86 in	2.825 kip*ft	20.401 kip*ft
Base Anchor 3	86 in	3.64	12.21	32 in	20.401 kip*ft	2.825 kip*ft
Base Anchor 4	110 in	***3.53***	***8.26***	8 in	32.322 kip*ft	0.171 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	3	***1.531***	4.495	29.17%	W4	0	118.000	4.495	0.000	44.205
Wall Connection 2	3	***1.531***	4.495	29.17%	W5	118	0.000	4.495	44.205	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
6228	40946	34718	524	7688	OK	1557	(34718)	OK

RIGIDITY

	CALCULATED VALUES	38%	Final	2.287854839

	Pier	Length	Height	Fixed Top?	Useable?	Stiffness (k)	Deflection
	Label	(inches)	(inches)	(Y/N)	(Y/N)	(1000 kip / IN)	(in / 1000 kip)
Door 1	Entire Wall	118	103.37	Y	Y	6.060	0.165
	A'	118	82.25	Y	Y	8.231	0.121
	A	38.88	82.25	Y	Y	1.265	0.791
	B	39.04	82.25	Y	Y	1.276	0.784

	Combine Logic					
	First Segment	Second Segment	Re-Name	Combine/Subtract	Method	Combined
Door 1	Entire Wall	A'	A'a	-	Deflection	0.044
	A	B	AB	+	Stiffness	2.541
	A'a	AB	Final	+	Deflection	0.437

ID:	Ozark I OZI-252
	DESIGN OF WALL MARKED W2

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 >= W1.2-A185	
Fy wire mesh	65000 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*

Geometric Properties

X Corridinate	100
Y Corridinate	12.5
Direction of Wall	Y
Center of gravity X	100.000
Center of gravity Y	71.500
Wall Weight	4230.000 lbs.
Central wall?	Yes
Wall that supports 2 roof panels?	Yes
lop (length of opening on wall)	0 ft
H (height of wall)	103.37 in
Lh (length of wall)	9.833 ft
Analyisis 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)	0.278
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)	108.86 psf	Wind Load (WL.lat)	58.99 psf
E (Earthquake Load)	16.95 psf	Earthquake Load (EL.lat)	13.91 psf

Wire Mesh		
Wire Size	W8	
spacing	4 in	
Mesh Area	0.24 in^2	= As

Factored Axially Applied Loads

Factored Loading per ACI	ACI eq. 9-3
Factored Pressure on Roof Wr	560.213

Axial Pressure on Section	
PuB	0.3 kip

Assumption check	
Pu/Ag	6.250 psi
0.06*f`c	300 psi
Check ACI 14.8.2.6	O.K.

Unfactored Axially Applied Loads

Unfactored Pressure on Roof uWr	330.0825 psf

Axial Pressure on Section	
PB	0.3 kip

Factored Laterally Applied Loads

Factored Loading per ACI	*ACI eq. 9-4*
Factored Pressure on Wall Ww	94.38 psf

Lateral Pressure on Section	
Lw = W*(L^4 / L^4 + H^4)	0.03 klf
Hw = W*(H^4 / H^4 + L^4)	0.06 klf

Unfactored Laterally Applied Loads

Unfactored Pressure on Wall uWw	58.99 psf

Lateral Pressure on Section	
Lw = W*(L^4 / L^4 + H^4)	0.02 klf
Hw = W*(H^4 / H^4 + L^4)	0.04 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	0.30 kip
Lateral	0.02 klf
Allowed service deflection	0.69 in
Msa	2.376 kip-in
M	2.379 kip-in
Ds	0.010 in
Check deflection	O.K.

Allowable Capacity

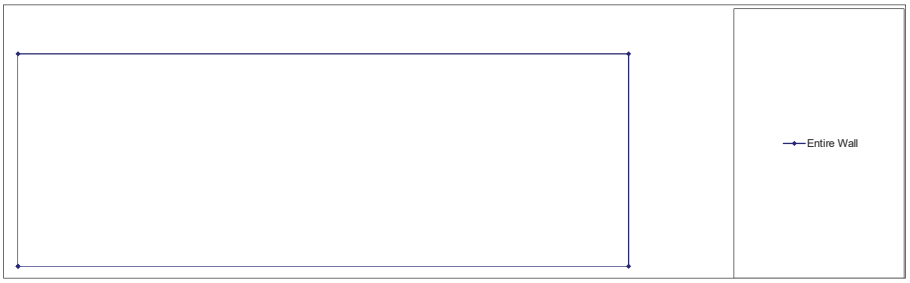
Ig = (b*h^3)/12	64 in^4
Ag = (b*h)	48 in^2
Yt = h/2	2
fr (rupture modulus)	530.330 psi
Mcr	16.971 kip-in
Beta_1	0.8
Trial Ast req'd	0.073 in^2
B	7.403686795
kd	0.583 in
I.cr	3.35 in^4
e_c	0.003
e_s	0.005
a	0.32469 psi
c	0.406 in
Asc	0.24 in^2
Icrdeflection	3.79 in^4
Ie	64.00 in^4
delta	150
r_t (maximum tensile reinforcement)	0.0225
r_{temp} (min. temperature reinforcement)	0.0017
r_{min} (minimum tensile reinforcement)	0.0033
r_{trial} (trial reinforcement ratio bottom)	0.0033
$\rho_{provided}$ (reinforcement ratio provided)	0.0110

Flexure

Assumption check		
Span	Hw	Lw
net Tensile Strain	0.011	0.011
Check ACI 14.8.2.3	Tension	Tension
Mua	0.573 kip-ft	

ACI eq. (14-6)		
Mu	0.590 kip-ft	0.360 kip-ft

ACI 9.3.2		
fb	0.9	0.9
fMn trial = ϕAsFy(dt - a/2)	1.960 kip-ft	1.960 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	0.24 in^2	0.24 in^2
fMn = ϕAsFy(db - a/2)	1.961 kip-ft	1.961 kip-ft
Check ϕMn > Mu	O.K.	O.K.
% allowed	30.09%	18.36%



REINFORCEMENT AT OPENINGS

Loading	
Pu (factorized load from roof)	0.56 klf
Ww (weight of panel per sq ft)	0.05 ksf

Material Properties	
db (effective depth bottom)	1.84 in
a (block of strain)	0.32469 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
3	110	110	28.737	42.09	42.09	45.16	45.16

Total Tension	Base Anchors					
10.693	Dist	Tension (kip)	Shear	L - Dist	Moment +	Moment -
Base Anchor 1	8 in	***3.53***	***8.26***	110 in	0.171 kip*ft	32.322 kip*ft
Base Anchor 2	59 in	3.64	12.21	59 in	9.602 kip*ft	9.602 kip*ft
Base Anchor 3	110 in	***3.53***	***8.26***	8 in	32.322 kip*ft	0.171 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	3	***1.531***	7.492	48.61%	W4	0	118.000	4.593	0.000	45.165
Wall Connection 2	3	***1.531***	7.492	48.61%	W5	118	0.000	4.593	45.165	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
9944	28737	18793	862	20365	OK	3315	(18793)	OK

RIGIDITY

CALCULATED VALUES	100%	Final	3.761454265

Pier	Length	Height	Fixed Top?	Useable?	Stiffness (k)	Deflection
Label	(inches)	(inches)	(Y/N)	(Y/N)	(1000 kip / IN)	(in / 1000 kip)
Entire Wall	118	103.37	N	Y	3.761	0.266

Combine Logic					
First Segment	Second Segment	Re-Name	Combine/Subtract	Method	Combined
Entire Wall	0	Final			3.761

ID:	Ozark I OZI-252
	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 >= W1.2-A185	
Fy wire mesh	65000 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*

Geometric Properties

X Corridinate	160
Y Corridinate	12.5
Direction of Wall	Y
Center of gravity X	160.000
Center of gravity Y	72.557
Wall Weight	3090.000 lbs.
Central wall?	Yes
Wall that supports 2 roof panels?	Yes
lop (length of opening on wall)	0 ft
H (height of wall)	103.37 in
Lh (length of wall)	9.833 ft
Analyisis 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)	0.278
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)	108.86 psf	Wind Load (WL.lat)	58.99 psf
E (Earthquake Load)	16.95 psf	Earthquake Load (EL.lat)	13.91 psf

Wire Mesh		
Wire Size	W8	
spacing	4 in	
Mesh Area	0.24 in^2	= As

Factored Axially Applied Loads

Factored Loading per ACI	ACI eq. 9-3
Factored Pressure on Roof Wr	560.213

Axial Pressure on Section

PuB	2.51 kip

Assumption check

Pu/Ag	52.292 psi
0.06*f`c	300 psi
Check ACI 14.8.2.6	O.K.

Unfactored Axially Applied Loads

Unfactored Pressure on Roof uWr	330.0825 psf

Axial Pressure on Section

PB	1.57 kip

Factored Laterally Applied Loads

Factored Loading per ACI	*ACI eq. 9-4*
Factored Pressure on Wall Ww	94.38 psf

Lateral Pressure on Section

Lw = W*(L^4 / L^4 + H^4)	0.03 klf
Hw = W*(H^4 / H^4 + L^4)	0.06 klf

Unfactored Laterally Applied Loads

Unfactored Pressure on Wall uWw	58.99 psf

Lateral Pressure on Section

Lw = W*(L^4 / L^4 + H^4)	0.02 klf
Hw = W*(H^4 / H^4 + L^4)	0.04 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	1.57 kip
Lateral	0.02 klf
Allowed service deflection	0.69 in
Msa	3.011 kip-in
M	3.030 kip-in
Ds	0.012 in
Check deflection	O.K.

Allowable Capacity

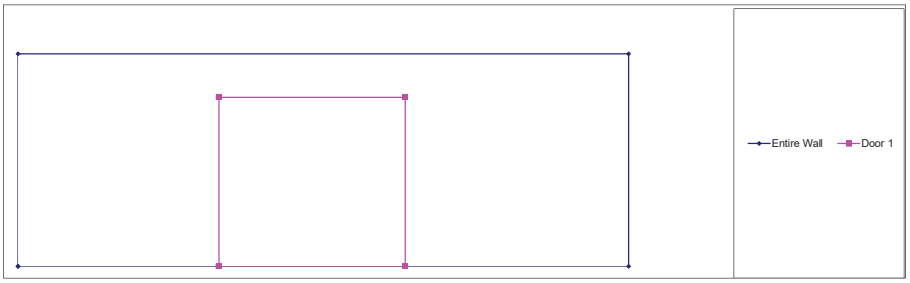
Ig = (b*h^3)/12	64 in^4
Ag = (b*h)	48 in^2
Yt = h/2	2
fr (rupture modulus)	530.330 psi
Mcr	16.971 kip-in
Beta_1	0.8
Trial Ast req'd	0.073 in^2
B	7.403686795
kd	0.583 in
I.cr	3.35 in^4
e_c	0.003
e_s	0.005
a	0.32469 psi
c	0.406 in
Asc	0.28 in^2
Icrdeflection	4.35 in^4
Ie	64.00 in^4
delta	150
r_t (maximum tensile reinforcement)	0.0225
r_{temp} (min. temperature reinforcement)	0.0017
r_{min} (minimum tensile reinforcement)	0.0033
r_{trial} (trial reinforcement ratio bottom)	0.0033
$\rho_{provided}$ (reinforcement ratio provided)	0.0110

Flexure

Assumption check		
Span	Hw	Lw
net Tensile Strain	0.011	0.011
Check ACI 14.8.2.3	Tension	Tension
Mua	0.665 kip-ft	

ACI eq. (14-6)		
Mu	0.830 kip-ft	0.360 kip-ft

ACI 9.3.2		
fb	0.9	0.9
fMn trial = ϕAsFy(dt - a/2)	1.960 kip-ft	1.960 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	0.24 in^2	0.24 in^2
fMn = ϕAsFy(db - a/2)	1.961 kip-ft	1.961 kip-ft
Check ϕMn > Mu	O.K.	O.K.
% allowed	42.33%	18.36%



REINFORCEMENT AT OPENINGS

Loading	
Pu (factorized load from roof)	0.56 klf
Ww (weight of panel per sq ft)	0.05 ksf

Material Properties	
db (effective depth bottom)	1.84 in
a (block of strain)	0.32469 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	3.24 ft	0 ft	3 ft	1.76 ft	1028.13	0.09 klf	0.65 klf	0.49 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	9.67 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	110	110	40.946	55.72	55.72	33.68	33.68

Total Tension	Base Anchors					
14.334	Dist	Tension (kip)	Shear	L - Dist	Moment +	Moment -
Base Anchor 1	8 in	***3.53***	***8.26***	110 in	0.171 kip*ft	32.322 kip*ft
Base Anchor 2	32 in	3.64	12.21	86 in	2.825 kip*ft	20.401 kip*ft
Base Anchor 3	86 in	3.64	12.21	32 in	20.401 kip*ft	2.825 kip*ft
Base Anchor 4	110 in	***3.53***	***8.26***	8 in	32.322 kip*ft	0.171 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	3	***1.531***	3.425	22.22%	W4	0	118.000	3.425	0.000	33.680
Wall Connection 2	3	***1.531***	3.425	22.22%	W5	118	0.000	3.425	33.680	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
6775	40946	34171	580	8505	OK	1694	(34171)	OK

RIGIDITY

	CALCULATED VALUES	42%	Final	2.530853042

	Pier Label	Length (inches)	Height (inches)	Fixed Top? (Y/N)	Useable? (Y/N)	Stiffness (k) (1000 kip / IN)	Deflection (in / 1000 kip)
	Entire Wall	118	103.37	Y	Y	6.060	0.165
Door 1	A'	118	82.25	Y	Y	8.231	0.121
	A	38.88	82.25	Y	Y	1.265	0.791
	B	43.12	82.25	Y	Y	1.579	0.633

	Combine Logic					
	First Segment	Second Segment	Re-Name	Combine/Subtract	Method	Combined
Door 1	Entire Wall	A'	A'a	-	Deflection	0.044
	A	B	AB	+	Stiffness	2.844
	A'a	AB	Final	+	Deflection	0.395

ID:	Ozark I OZI-252
	DESIGN OF WALL MARKED W4

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 >= W1.2-A185	
Fy wire mesh	65000 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*

Geometric Properties

X Corridinate	18
Y Corridinate	10.5
Direction of Wall	X
Center of gravity X	90.983
Center of gravity Y	10.500
Wall Weight	4490.000 lbs.
Central wall?	Yes
Wall that supports 2 roof panels?	No
lop (length of opening on wall)	0 ft
H (height of wall)	96 in
Lh (length of wall)	12.000 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)	0.278
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)	108.86 psf	Wind Load (WL.lat)	58.99 psf
E (Earthquake Load)	16.95 psf	Earthquake Load (EL.lat)	13.91 psf

Wire Mesh		
Wire Size	W8	
spacing	4 in	
Mesh Area	0.24 in^2	= As

Factored Axially Applied Loads

Factored Loading per ACI	ACI eq. 9-3
Factored Pressure on Roof Wr	560.213

Axial Pressure on Section	
PuB	2.56 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	330.0825 psf

Axial Pressure on Section	
PB	1.61 kip

Factored Laterally Applied Loads

Factored Loading per ACI	*ACI eq. 9-4*
Factored Pressure on Wall Ww	94.38 psf

Lateral Pressure on Section	
Lw = W*(L^4 / L^4 + H^4)	0.02 klf
Hw = W*(H^4 / H^4 + L^4)	0.08 klf

Unfactored Laterally Applied Loads

Unfactored Pressure on Wall uWw	58.99 psf

Lateral Pressure on Section	
Lw = W*(L^4 / L^4 + H^4)	0.01 klf
Hw = W*(H^4 / H^4 + L^4)	0.05 klf

Shear

Factored Loading per ACI	ACI eq. 9-3
Vu = wuB*(Bw-2db) / 2	0.08
Phi*Vc/2	1.33
Check Shear ACI 11.5.5.1	O.K.

Deflection *ACI 14.8.4*

Service Loads	
Axial	1.61 kip
Lateral	0.01 klf
Allowed service deflection	0.64 in
Msa	1.765 kip-in
M	1.775 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	7.403686795
kd	0.583 in
I.cr	3.35 in^4
e_c	0.003
e_s	0.005
a	0.32469 psi
c	0.406 in
Asc	0.28 in^2
Icrdeflection	4.35 in^4
Ie	64.00 in^4
delta	150
r_t (maximum tensile reinforcement)	0.0225
r_{temp} (min. temperature reinforcement)	0.0017
r_{min} (minimum tensile reinforcement)	0.0033
r_{trial} (trial reinforcement ratio bottom)	0.0033
$\rho_{provided}$ (reinforcement ratio provided)	0.0110

Flexure

Assumption check		
Span	Hw	Lw
net Tensile Strain	0.011	0.011
Check ACI 14.8.2.3	Tension	Tension
Mua	0.747 kip-ft	

ACI eq. (14-6)		
Mu	0.910 kip-ft	0.360 kip-ft

ACI 9.3.2		
fb	0.9	0.9
fMn trial = ϕAsFy(dt - a/2)	1.960 kip-ft	1.960 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	0.24 in^2	0.24 in^2
fMn = ϕAsFy(db - a/2)	1.961 kip-ft	1.961 kip-ft
Check ϕMn > Mu	O.K.	O.K.
% allowed	46.40%	18.36%

REINFORCEMENT AT OPENINGS

Loading	
Pu (factorized load from roof)	0.56 klf
Ww (weight of panel per sq ft)	0.05 ksf

Material Properties	
db (effective depth bottom)	1.84 in
a (block of strain)	0.32469 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
Window 1	2.54 ft	5.33 ft	3.17 ft	1.5 ft	185.45	0.08 klf	0.64 klf	0.54 kip-ft
Vent 1	5.4 ft	0.71 ft	1 ft	6.29 ft	50.00	0.31 klf	0.87 klf	0.07 kip-ft

Flexure

Opening	ϕb	As req'd	Bar size	qty req'd:	**ϕMn** = ϕAsFy(db - a/2)	Check ϕMn > Mu
Window 1	0.9	0.008 in^2	No. 3	1	8.12 kip-ft	O.K.
Vent 1	0.9	0 in^2	No. 3	0	0 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	132	132	36.627	52.30	52.30	148.29	135.86

Total Tension	Base Anchors					
10.923	Dist	Tension (kip)	Shear	L - Dist	Moment +	Moment -
Base Anchor 1	12 in	3.64	12.21	132 in	0.331 kip*ft	40.051 kip*ft
Base Anchor 2	72 in	3.64	12.21	72 in	11.916 kip*ft	11.916 kip*ft
Base Anchor 3	132 in	3.64	12.21	12 in	40.051 kip*ft	0.331 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	3	***2.703***	8.712	50.00%	W1	2	142.000	8.109	1.352	95.957
Wall Connection 2	3	***2.703***	7.462	50.00%	W2	82	62.000	7.462	50.987	38.551
Wall Connection 3	3	***2.703***	8.712	50.00%	W3	142	2.000	8.109	95.957	1.352

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
11233	36627	25394	806	19192	OK	3744	(25394)	OK

RIGIDITY

	CALCULATED VALUES	94%	Final	8.208108712

	Pier Label	Length (inches)	Height (inches)	Fixed Top? (Y/N)	Useable? (Y/N)	Stiffness (k) (1000 kip / IN)	Deflection (in / 1000 kip)
	Entire Wall	144	96	Y	Y	8.710	0.115
Window 1	A'	144	14.04	Y	Y	68.160	0.015
	A	30.48	14.04	Y	Y	13.517	0.074
	B	75.48	14.04	Y	Y	35.432	0.028
Vent 1	B'	144	12	Y	Y	79.815	0.013
	C	64.8	12	Y	Y	35.593	0.028
	D	67.2	12	Y	Y	36.941	0.027

	Combine Logic					
	First Segment	Second Segment	Re-Name	Combine/Subtract	Method	Combined
Window 1	Entire Wall	A'	A'a	-	Deflection	0.100
	A	B	AB	+	Stiffness	48.949
	A'a	AB	A'b	+	Deflection	0.121
Vent 1	A'b	B'	B'a	-	Deflection	0.108
	C	D	CD	+	Stiffness	72.534
	B'a	CD	Final	+	Deflection	0.122

ID:	Ozark I OZI-252
	DESIGN OF WALL MARKED W5

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 >= W1.2-A185	
Fy wire mesh	65000 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*

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)	108.86 psf	Wind Load (WL.lat)	58.99 psf
E (Earthquake Load)	16.95 psf	Earthquake Load (EL.lat)	13.91 psf

Factored Axially Applied Loads

Factored Loading per ACI	ACI eq. 9-3
Factored Pressure on Roof Wr	560.213

Axial Pressure on Section	
PuB	2.56 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	330.0825 psf

Axial Pressure on Section	
PB	1.61 kip

Shear

Factored Loading per ACI	ACI eq. 9-3
Vu = wuB*(Bw-2db) / 2	0.08
Phi*Vc/2	1.33
Check Shear ACI 11.5.5.1	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	7.403686795
kd	0.583 in
I.cr	3.35 in^4
e_c	0.003
e_s	0.005
a	0.32469 psi
c	0.406 in
Asc	0.28 in^2
Icrdeflection	4.35 in^4
Ie	64.00 in^4
delta	150
r_t (maximum tensile reinforcement)	0.0225
r_{temp} (min. temperature reinforcement)	0.0017
r_{min} (minimum tensile reinforcement)	0.0033
r_{trial} (trial reinforcement ratio bottom)	0.0033
$\rho_{provided}$ (reinforcement ratio provided)	0.0110

Geometric Properties

X Corridinate	18
Y Corridinate	132.5
Direction of Wall	X
Center of gravity X	91.577
Center of gravity Y	132.500
Wall Weight	4490.000 lbs.
Central wall?	Yes
Wall that supports 2 roof panels?	No
lop (length of opening on wall)	0 ft
H (height of wall)	96 in
Lh (length of wall)	12.000 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)	0.278
Eccentricity - Axial Load	1 in
Is wall Split	No

Wire Mesh		
Wire Size	W8	
spacing	4 in	
Mesh Area	0.24 in^2	= As

Factored Laterally Applied Loads

Factored Loading per ACI	*ACI eq. 9-4*
Factored Pressure on Wall Ww	94.38 psf

Lateral Pressure on Section	
Lw = W*(L^4 / L^4 + H^4)	0.02 klf
Hw = W*(H^4 / H^4 + L^4)	0.08 klf

Unfactored Laterally Applied Loads

Unfactored Pressure on Wall uWw	58.99 psf

Lateral Pressure on Section	
Lw = W*(L^4 / L^4 + H^4)	0.01 klf
Hw = W*(H^4 / H^4 + L^4)	0.05 klf

Deflection *ACI 14.8.4*

Service Loads	
Axial	1.61 kip
Lateral	0.01 klf
Allowed service deflection	0.64 in
Msa	1.765 kip-in
M	1.775 kip-in
Ds	0.006 in
Check deflection	O.K.

Flexure

Assumption check		
Span	Hw	Lw
net Tensile Strain	0.011	0.011
Check ACI 14.8.2.3	Tension	Tension
Mua	0.747 kip-ft	

ACI eq. (14-6)		
Mu	0.910 kip-ft	0.360 kip-ft

ACI 9.3.2		
fb	0.9	0.9
fMn trial = ϕAsFy(dt - a/2)	1.960 kip-ft	1.960 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	0.24 in^2	0.24 in^2
fMn = ϕAsFy(db - a/2)	1.961 kip-ft	1.961 kip-ft
Check ϕMn > Mu	O.K.	O.K.
% allowed	46.40%	18.36%

REINFORCEMENT AT OPENINGS

Loading	
Pu (factorized load from roof)	0.56 klf
Ww (weight of panel per sq ft)	0.05 ksf

Material Properties	
db (effective depth bottom)	1.84 in
a (block of strain)	0.32469 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
Window 1	2.54 ft	5.33 ft	3.17 ft	1.5 ft	185.45	0.08 klf	0.64 klf	0.54 kip-ft
Vent 1	1 ft	0.71 ft	1 ft	6.29 ft	50.00	0.31 klf	0.87 klf	0.07 kip-ft

Flexure

Opening	ϕb	As req'd	Bar size	qty req'd:	**ϕMn** = ϕAsFy(db - a/2)	Check ϕMn > Mu
Window 1	0.9	0.008 in^2	No. 3	1	8.12 kip-ft	O.K.
Vent 1	0.9	0 in^2	No. 3	0	0 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	132	132	36.627	52.30	52.30	148.29	135.86

Total Tension	Base Anchors					
10.923	Dist	Tension (kip)	Shear	L - Dist	Moment +	Moment -
Base Anchor 1	12 in	3.64	12.21	132 in	0.331 kip*ft	40.051 kip*ft
Base Anchor 2	72 in	3.64	12.21	72 in	11.916 kip*ft	11.916 kip*ft
Base Anchor 3	132 in	3.64	12.21	12 in	40.051 kip*ft	0.331 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	3	***2.703***	8.712	50.00%	W1	2	142.000	8.109	1.352	95.957
Wall Connection 2	3	***2.703***	7.462	50.00%	W2	82	62.000	7.462	50.987	38.551
Wall Connection 3	3	***2.703***	8.712	50.00%	W3	142	2.000	8.109	95.957	1.352

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
11217	36627	25410	805	19159	OK	3739	(25410)	OK

RIGIDITY

	CALCULATED VALUES	94%	Final	8.193979962

	Pier Label	Length (inches)	Height (inches)	Fixed Top? (Y/N)	Useable? (Y/N)	Stiffness (k) (1000 kip / IN)	Deflection (in / 1000 kip)
	Entire Wall	144	96	Y	Y	8.710	0.115
Window 1	A'	144	14.04	Y	Y	68.160	0.015
	A	30.48	14.04	Y	Y	13.517	0.074
	B	75.48	14.04	Y	Y	35.432	0.028
Vent 1	B'	144	12	Y	Y	79.815	0.013
	C	12	12	Y	Y	5.000	0.200
	D	120	12	Y	Y	66.445	0.015

	Combine Logic					
	First Segment	Second Segment	Re-Name	Combine/Subtract	Method	Combined
Window 1	Entire Wall	A'	A'a	-	Deflection	0.100
	A	B	AB	+	Stiffness	48.949
	A'a	AB	A'b	+	Deflection	0.121
Vent 1	A'b	B'	B'a	-	Deflection	0.108
	C	D	CD	+	Stiffness	71.445
	B'a	CD	Final	+	Deflection	0.122

ID: **Ozark I OZI-252**

DESIGN OF FLOOR PANEL F1

Material Properties

f'c	5000 psi	
Steel Reinforcement	I. Plain WWF >= W1.2-A185	
Fy	65000 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*

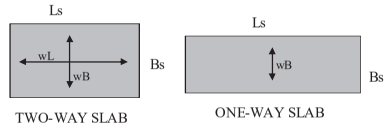
Geometric Properties

Ls (overall length of slab)	12.17 ft		
Bs (overall width of slab)	10.67 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)*
I (length of slab for deflection)	20.863 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)	0.278		*{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}*

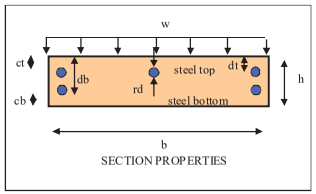
Reinforcement Limits

ρ_t (maximum tensile reinforcement)	0.0225	*ACI 10.3.3*
ρ_{temp} (min. temperature reinforcement)	0.0017	*ACI 7.12.2*
ρ_{min} (minimum tensile reinforcement)	0.0033	*ACI 10.5.1*
ρ_{trial} (trial reinforcement ratio bottom)	0.0033	
ρ_{trial} (trial reinforcement ratio top)	0.0033	

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	7.398	
kd	0.441 in	
I.cr	1.18 in^4	
c	0 in	
a	0.31 in	



$\rho_{provided}$ (reinforcement ratio provided)	0.0063	
ω =	0.0817	
$\rho_{provided}$ (reinforcement ratio provided)	0.0184	
ω =	0.2394	
$\rho_{provided}$ (reinforcement ratio provided)	0.0052	
ω =	0.0676	
$\rho_{provided}$ (reinforcement ratio provided)	0.0141	
ω =	0.1836	

	Wire Mesh (Top)		
	Wire Size	W8	
Trial As' req'd	spacing	4 in	
0.13 in^2	Mesh Area	**0.24 in^2**	**= As'**

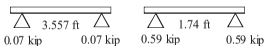
	Wire Mesh (Bottom)		
	Wire Size	W8	
Trial Ast required	spacing	4 in	
0.126 in^2	Mesh Area	**0.24 in^2**	**= As**

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)	17.39 psf

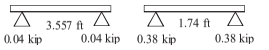
Sustained Loading	
Pressure on slab	**W**
D (Dead load)	62.5 psf
S (Snow Load)	0 psf
Lr (Live Floor Load)	400 psf

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	715 psf	0.04 klf	0.68 klf



B (Span in the short direction) = (Bs/NsB-1) - 0(oh1)	3.557 ft
L (Span in the long direction) = (Ls/NsL-1) - 0(oh2)	1.74 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	0.025 klf	0.437 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	0.02 klf	0.38 klf

Eficiency can be enhanced if Ast is diminished

SUMMARY

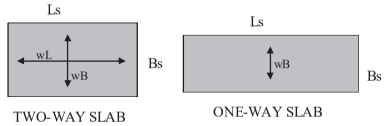
Use 1 Layer of Wire Mesh on Top W8 x W8 x 4 x 4

Use 1 Layer of Wire Mesh on Bottom W8 x W8 x 4 x 4

ID: **Ozark I OZI-252**

DESIGN OF FLOOR PANEL F1

Notes:



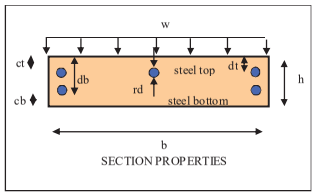
Material Properties

f ' c	5000 psi	
Steel Reinforcement	I. Plain WWF >= W1.2-A185	
Fy	65000 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)	12.17 ft		
Bs (overall width of slab)	10.67 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)	20.863 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)	0.278		*{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}*

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	7.398	
kd	0.441 in	
I.cr	1.18 in^4	
c	0 in	
a	0.31 in	



$\rho_{provided}$ (reinforcement ratio provided)	0.0063
ω =	0.0817
$\rho_{provided}$ (reinforcement ratio provided)	0.0184
ω =	0.2394
$\rho_{provided}$ (reinforcement ratio provided)	0.0052
ω =	0.0676
$\rho_{provided}$ (reinforcement ratio provided)	0.0141
ω =	0.1836

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	0.04 kip-ft	0.045	Tension	0.9	3.54 kip-ft		3.54 kip-ft	O.K.	1.14%
** Mneg (negative Moment) = (wB*B^2)*0.1	0.05 kip-ft	0.045	Tension	0.9	4.31 kip-ft		4.31 kip-ft	O.K.	1.17%

***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	0.16 kip-ft	0.045	Tension	0.9	3.54 kip-ft		3.54 kip-ft	O.K.	4.54%
** Mneg (negative Moment) = (wB*B^2)*0.106	0.01 kip-ft	0.045	Tension	0.9	4.31 kip-ft		4.31 kip-ft	O.K.	0.30%

***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	0.09 kip	0.85	5.40 kip	4.59 kip	O.K.	1.86%
			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	0.46 kip	0.85	5.40 kip	4.59 kip	O.K.	10.04%
			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

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 long term deflection	% allowed - short term	% allowed - long term
B	0.04 kip-ft	0.04 kip-ft	125.00 in	125.00 in	0.000 in	0.0063	0.9132	0.000 in	0.000 in	0.2371 in	0.0889 in	O.K.	O.K.	0.07%	0.17%
L	0.16 kip-ft	0.17 kip-ft	125.00 in	125.00 in	0.000 in	0.0063	0.9132	0.000 in	0.000 in	0.1160 in	0.0435 in	O.K.	O.K.	0.14%	0.35%

ID: **Ozark I OZI-252**

Geometric properties	
Bs (width of roof panel)	11.92 ft
Ls (Length of roof panel)	15.00 ft
Ar Area of Roof	178.75 ft^2
H (height of building)	9.75 ft
Lb (length of building)	12 ft
Wb (width of building)	10.5 ft
Ab (Area of building)	126 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)	118.61 ft^2
Hw (depth of floodwater)	1 ft

Loading	
Wv (weight of vault)**	0 lb
Wtr (roof panel weight)	10590 lb
Ww (total walls panel weight)	19390 lb
Fw (floor panel weight)	7650 lb
We (estimated weight of building)	37630 lb
Wev (estimated weight of building w/ vault)	37630 lb
PSFr (roof snow load)	210 psf
PSFf (Floor Live Load)	400 psf
Pmax (Maximum allowable pressure)	1500 psf
Fupmw (MWFRS Uplift Force)	44.92 psf
WLlat (MWFRS lateral wind pressure)	51.74 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)	8789.0 lb
	.7*Vseismic (from seismic analysis without snow)	7327.2 lb
	Vwind = WLlat * max(Wb,Lb)*H	6053.9 lb

* Load adjustment per IBC 1605.3 load combinations.

Sliding Resistance with Snow	Pslide = u*(.6*We+.75*PSFr*Ar)	Pslide =	20292.45 lb				
						Fsreqd	
Factor of Safety	FSwind = Pslide / Vwind	FSwind =	3.4	≥	1.0		O.K.
	FSseismic = Pslide / Vseismic	Fseismic =	2.3	≥	1.0		O.K.
Sliding Resistance with No Snow	Pslide = u*.6*We	Pslide =	9031.2 lb				
						Fsreqd	
Factor of Safety	FSwind = Pslide / Vwind	Fswind =	1.5	≥	1.0		O.K.
	FSseismic = Pslide / Vseismic	Fseismic =	1.2	≥	1.0		O.K.

CHECK OVERTURNING RESISTANCE

Shear	.7*Otseismic (from seismic analysis with snow)	78.733 kip-ft
	.7*Otseismic (from seismic analysis without snow)	47.913 kip-ft
	Otwind = (WLlat*Lb*H^2 / 2) + (Fupmw*Lb*Wb^2 / 2)	59.225 kip-ft

* Load adjustment per IBC 1605.3 load combinations.

Overturning Resistance with Snow	Otrsnow = (.6*We+.75*PSFr*Ar)*(Wb/2)	Otrsnow =	126.597 kip-ft				
						Fsreqd	
Factor of Safety	FSwind = Otrsnow / Otwind	FSwind =	2.14	≥	1.0		O.K.
	FSseismic = Otrsnow / Vseismic	Fseismic =	1.61	≥	1.0		O.K.
Overturning Resistance with No Snow	Otr = .6*We*Wb/2	Otr	118.535 kip-ft				
						Fsreqd	
Factor of Safety	FSwind = Otr / Vwind	Fswind =	2.00	≥	1.0		O.K.
	FSseismic = Otr / Vseismic	Fseismic =	2.47	≥	1.0		O.K.

CHECK BEARING PRESSURE CONDITION

Net Pressure	Pnet = (Wev + PSFr*Ar + PSFf*Af) / Ab	996.57 psf	
Allowable	Pmax ≥ Pnet	1500 psf ≥ 996.57 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** =	7401.33 lb			
Factor of Safety	FSb = We / Fb	**FSb =**	5.08	≥	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.