<u>CXT Inc. (Precast Division)</u>

Calculations
Navajo N-284
Structural Analysis

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

<u>Design Standards</u>
2018 International Building Code
ASCE 7-16/ ACI 318-14

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

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OKLAHOMA

September 25, 2022

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

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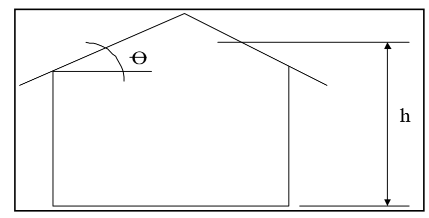
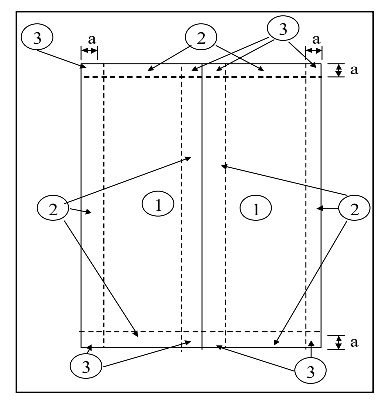
Main Wind Force Resisting System Loads (ASCE 7-16)

Navajo N-284		
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 26.5-1A thru 26.5-2D: Basic Wind Speed (3 second Gust)
h.wind	8.00 ft	Windward wall height
h.lee	8.00 ft	Leeward wall height
W.building	26 ft	Width of the building
L.building	20 ft	Length of the building
H.building	11.55 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



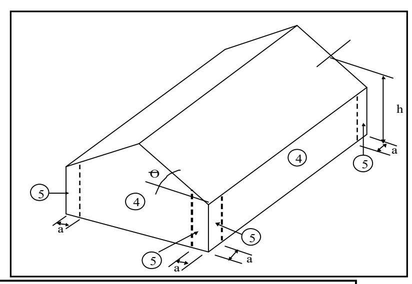


Kzt	1	Topographic factor
h	9.775 ft	Mean roof height
n_a	7.67	Natural frequency
Flexibility	Rigid	Building flexibility
α	9.5	Terrain factor
Z_g	900 ft	Terrain factor

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	160.0 sq ft	1188.3 sq ft	0 sq ft	0.00	0.00	0.00	0.00	Enclosed
Windward endwall	0 sq ft	254.2 sq ft	1094.2 sq ft	0 sq ft	0.00	0.00	0.00	0.00	Enclosed
Leeward sidewall	0 sq ft	160.0 sq ft	1188.3 sq ft	0 sq ft	0.00	0.00	0.00	0.00	Enclosed
Leeward endwall	0 sq ft	254.2 sq ft	1094.2 sq ft	0 sq ft	0.00	0.00	0.00	0.00	Enclosed
Roof	0 sq ft	520.0 sq ft	828.3 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 27.3.3 Roof Overhangs		
Cp	0.8	Windward wall (Use with qz) Fig. 27.3-1		
	-0.440	Leeward wall (wind normal to ridge) (Use with qh)	L/B =	1.30
	-0.500	Leeward wall (wind parallel to ridge) (Use with qh)	L/B =	0.77
	-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 27.3-1) Normal to Ridge when Theta >= 10degrees	-0.108	-0.639	-0.481

Roof Pressures Wind Perpendicular to Ridge w/ ϑ >= 10 deg	
w/ Negative Internal	3.67 psf
w/ Positive Internal	-30.07 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	-0.90	-0.90	-0.50	-0.30

Wall Pressures:	w/ Negative	w/ Positive Internal
Windward	35.74 psf	20.78 psf
Leeward (wind normal)	-16.00 psf	-23.02 psf
Leeward (wind parallel)	-16.00 psf	-25.14 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	-39.28 psf
h/2 to h	-39.28 psf
h to 2h	-25.14 psf
Over 2h	-18.08 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:	**9.78 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

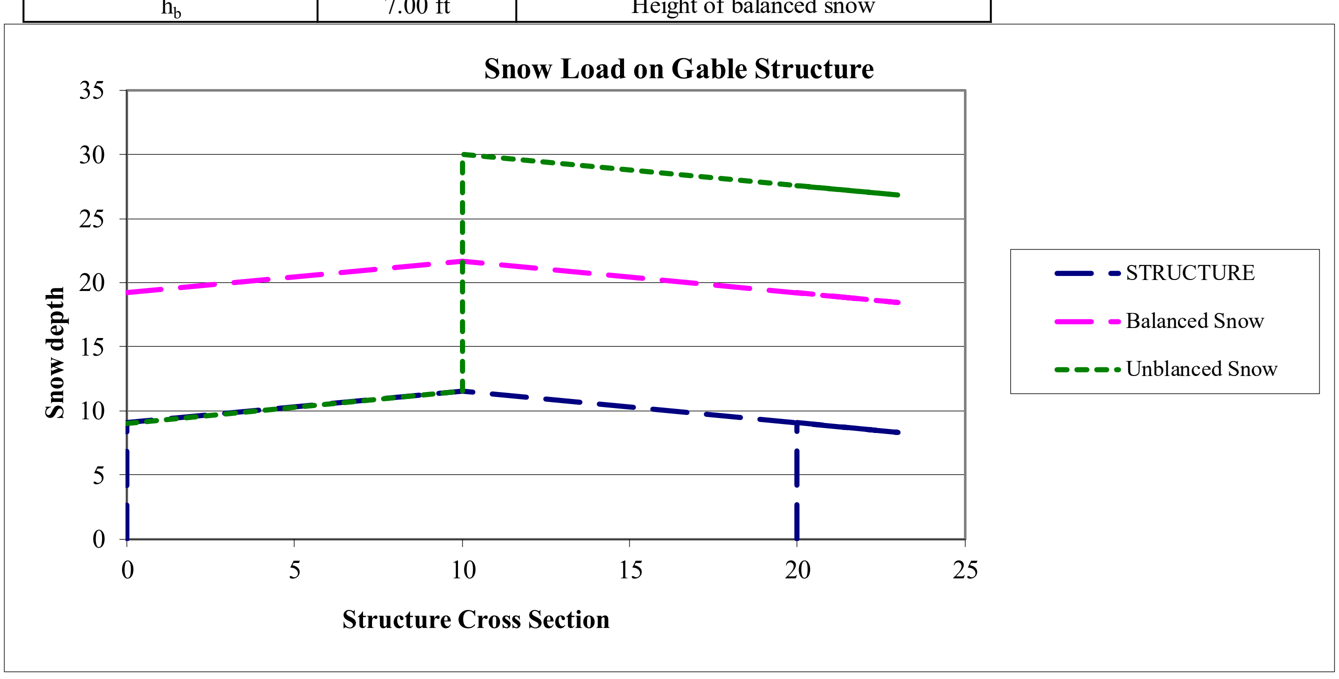
ASCE 7-16 SNOW LOAD CALCULATION

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	26 ft	Length of the building
L.building	20 ft	Width of the building
H.building	11.55 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 7.3-2 - 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	13 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	13.0 ft	ASCE Figure 7.6-2

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 7-16)

Navajo N-284			
Category	II	IBC TABLE 1604.5: Risk Category of Buildings and Other Structures.	
Ss	0.50 g	Max. Earthquake Ground Motion of 0.2 sec Spectral Response Acceleration	*ASCE Figure 22-1*
S_1	0.14 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	9.98 ft	Height in feet from base to highest level of structure	

				Value 1*	Value 2*
F_a	1.4016	Interpolated Value	*ASCE Table 11.4-1*	1.6	1.4
F_v	2.2	Interpolated Value	*ASCE Table 11.4-2*	2.2	2.2

*=Used for interpolation

Sms = Fa * S_S	0.698 g	Adjusted MCE Spectral Response Acceleration at short periods	*ASCE 11.4-1*
S_{ml} = F_v * S_1	0.310 g	Adjusted MCE Spectral Response Acceleration at 1 sec period	*ASCE 11.4-2*

(MCE = Maximum considered earthquake)

S_{DS} = 2/3 Sm_s	0.465 g	Design Spectral Acceleration Parameters	*ASCE 11.4-3*
S_{D1} = 2/3 Sm_1	0.207 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}	C	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	0.11 sec	Approximate fundamental period	*ASCE 12.8-7*
T_s = S_{D1}/S_{DS}	0.44 sec		
T	0.11 sec	Fundamental period of the structure (can be taken as Ta per ASCE 12.8.2)	

C_s = S_{DS}/(R/I)	0.116	*ASCE 12.8-2*	
$C_{s.min}$	0.021	*ASCE 12.8-5 & 12.8-6*	
$C_{s.max}$	0.460	*ASCE 12.8-3 & 12.8-4*	
C_s	0.116		
k	1.000	*ASCE 12.8.3*	
W	170.99 kip		
V = C_s * W	49.73 kip	*ASCE 12.8-1*	*Shear **with** snow load*
M_o =	490.2 k-ft		*Overturning Moment **with** snow load*
V = C_s * W	41.58 kip		*Shear **without** snow load*
M_o =	408.0 k-ft		*Overturning Moment **without** snow load*

WITH SNOW LOAD

						12.8-12	*12.8-11;11.7*			*12.10-1*
Level	Story Height	h_i or h_x	P_f (flat roof snow load)	w_i	w_i*h_i^k	C_{vx}	F_x	V_x (Story shear)	M_x	F_{px} (diaphragm force)
Roof	9.78 ft	9.98 ft	210 psf	105.09 kip	1049.2 k-ft	0.987	49.09 kip	49.09 kip	0.0 k-ft	19.56 kip
Walls	0.00 ft	0.00 ft								
Floor	0.21 ft	0.21 ft		65.90 kip	13.7 k-ft	0.013	0.64 kip	49.73 kip	479.8 k-ft	12.27 kip
Base	0 ft	0.00 ft	W=	170.99 kip	1062.9 k-ft			M_o =	490.2 k-ft	

WITHOUT SNOW LOAD

						12.8-12	*12.8-11;11.7*			*12.10-1*
Level	Story Height	h_i or h_x	P_f (flat roof snow load)	w_i	w_i*h_i^k	C_{vx}	F_x	V_x (Story shear)	M_x	F_{px} (diaphragm force)
Roof	9.78 ft	9.98 ft	0 psf	77.08 kip	769.5 k-ft	0.982	40.85 kip	40.85 kip	0.0 k-ft	14.35 kip
Walls	0.00 ft	0.00 ft								
Floor	0.21 ft	0.21 ft		65.90 kip	13.7 k-ft	0.018	0.73 kip	41.58 kip	399.3 k-ft	12.27 kip
Base	0 ft	0.00 ft	W=	142.98 kip	783.2 k-ft			Mo =	408.0 k-ft	

Center of Mass & Rigidity

Navajo N-284

Wall	Upper Left = 0,0		Lower Right	X: 348	Y: 276	
	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%	21.33%	2,608	103	12.284	118.000
W2	0.00%	28.67%	3,505	138	12.280	26.000
W3	18.71%	0.00%	2,288	229	166.279	69.077
W4	22.77%	0.00%	2,783	278	141.721	59.083
W5	8.46%	0.00%	1,034	141	12.279	72.000
W6	0.00%	21.33%	2,608	103	12.284	118.000
W7	0.00%	28.67%	3,505	138	12.280	26.000
W8	18.85%	0.00%	2,304	230	166.279	68.793
W9	22.77%	0.00%	2,783	278	141.721	59.083
W10	8.46%	0.00%	1,034	141	12.279	72.000
P1-1	0.00%	0.00%	-	-	27.721	41.000
P1-2	0.00%	0.00%	-	-	52.279	41.000
P1-3	0.00%	0.00%	-	-	27.721	41.000
P1-4	0.00%	0.00%	-	-	52.279	41.000

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	10700	174	0	348	138	210	261.0	69.0	17704	10700
R2	4.5	10700	0	0	174	138	210	87.0	69.0	17704	10700
R3	4.5	10700	174	138	348	276	210	261.0	207.0	17704	10700
R4	4.5	10700	0	138	174	276	210	87.0	207.0	17704	10700
F1	5	15813	18	18	330	138	400	174.0	78.0	15813	0
F2	5	15813	18	138	330	258	400	174.0	198.0	15813	0
Totals		68554						173.0	138.0		

Torsional Eccentricity ex	Torsional Eccentricity ey	Wgt (w snow)	Wgt (w/o snow)		wgt (w snow)	wgt (w/o snow)
11.25	0.01	170,994	142,980	roof	105,091	77,077
Center of Gravity				floor	65,903	
X	Y					
173.0	138.0					
Center of Rigidty						
X	Y					
161.7	138.0					

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 check	Toward Upper Left Anchor Resistance Moment (kip-ft)	Toward Upper Left check	Overturning status using just connection to adjacent walls
W1	-108.46	146.05	OK	146.05	OK	None Required
W2	-97.66	253.56	OK	253.56	OK	None Required
W3	20.00	43.25	OK	64.87	OK	None Required
W4	28.75	43.25	OK	64.87	OK	None Required
W5	12.66	22.45	OK	22.45	OK	None Required
W6	-108.46	146.05	OK	146.05	OK	None Required
W7	-97.66	253.56	OK	253.56	OK	None Required
W8	20.32	64.87	OK	43.25	OK	None Required
W9	28.75	64.87	OK	43.25	OK	None Required
W10	12.66	22.45	OK	22.45	OK	None Required
P1-1	-1.16	6.63	OK	0.00	OK	None Required
P1-2	-1.16	6.63	OK	0.00	OK	None Required
P1-3	-1.16	0.00	OK	6.63	OK	None Required
P1-4	-1.16	0.00	OK	6.63	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 check	Toward Upper Left Anchor Resistance Moment (kip-ft)	Toward Upper Left check	Combined Loading Unity Check	Required Tension Capacity per Base Anchor (lb)
W1	-108.46	174.25	OK	167.89	OK	OK	(3966)
W2	-97.66	201.67	OK	201.67	OK	OK	(4621)
W3	20.00	42.22	OK	42.90	OK	OK	(1567)
W4	28.75	39.93	OK	45.41	OK	OK	(1074)
W5	12.66	22.64	OK	22.64	OK	OK	(1336)
W6	-108.46	174.25	OK	167.89	OK	OK	(3966)
W7	-97.66	201.67	OK	201.67	OK	OK	(4621)
W8	20.32	42.90	OK	42.22	OK	OK	(1545)
W9	28.75	39.93	OK	45.41	OK	OK	(879)
W10	12.66	22.64	OK	22.64	OK	OK	(1336)
P1-1	-1.16	3.31	OK	4.52	OK	OK	(930)
P1-2	-1.16	3.31	OK	4.52	OK	OK	(930)
P1-3	-1.16	4.52	OK	3.31	OK	OK	(930)
P1-4	-1.16	4.52	OK	3.31	OK	OK	(930)

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	0.0%	100.0%	320.30	313.94	OK	OK
W2	0.0%	100.0%	455.23	455.23	OK	OK
W3	0.0%	100.0%	85.46	107.77	OK	OK
W4	0.0%	100.0%	83.18	110.28	OK	OK
W5	0.0%	100.0%	45.10	45.10	OK	OK
W6	0.0%	100.0%	320.30	313.94	OK	OK
W7	0.0%	100.0%	455.23	455.23	OK	OK
W8	0.0%	100.0%	107.77	85.46	OK	OK
W9	0.0%	100.0%	104.80	88.66	OK	OK
W10	0.0%	100.0%	45.10	45.10	OK	OK
P1-1	0.0%	100.0%	9.95	4.52	OK	OK
P1-2	0.0%	100.0%	9.95	4.52	OK	OK
P1-3	0.0%	100.0%	4.52	9.95	OK	OK
P1-4	0.0%	100.0%	4.52	9.95	OK	OK

ID: **Navajo N-284**

DESIGN OF ROOF PANELS MARK R1, R2, R3, & R4

Notes:

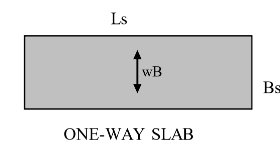
Material Properties

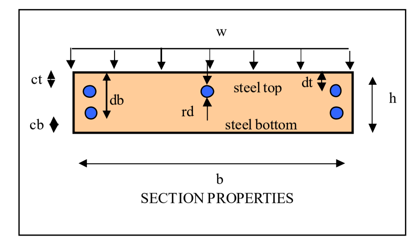
f ' c	5000 psi	
Steel Reinforcement	Plain WWF Grade 80	
Fy	80000 psi	
Lightweight?	No	
C_d (Concrete density)	150 pcf	O.K.
E (Steel)	29000000 psi	
E (Concrete)	4286826 psi	*ACI 8.5.1*
n (modular ratio)	6.76	*ACI 14.0*

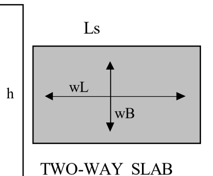
Geometric Properties

Ls (overall length of slab)	15.04 ft		
Bs (overall width of slab)	11.5 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)	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)	18 in	2	*[qty of overhangs in Bs direction]*
oh2 (overhang length and qty for ls)	18 in	1	*[qty of overhangs in ls direction]*
Cs (% of DL used for Seismic)	0.116	*[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	0.776 in	
I.cr	11.45 in^4	
c	0 in	
a	0.32 in	







bottom mesh		
$\rho_{provided}$ (reinforcement ratio provided)		0.0049
top mesh	ω =	0.0784 psi
$\rho_{provided}$ (reinforcement ratio provided)		0.0119
both layers	ω =	0.1904 psi
$\rho_{provided}$ (reinforcement ratio provided)		0.0138
	ω =	0.2208 psi

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	

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

		Wire Mesh (Bottom)		
		Wire Size	W6.7	
Trial Ast required		spacing	4 in	
0.136 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)	108.86 psf
E (Earthquake Load)	7.09 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	0.48 klf	0.08 klf

8.500 ft — 2.76 kip, 2.76 kip
13.54 ft — 0.60 kip, 0.60 kip

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	0.29 klf	0.04 klf

8.500 ft — 1.67 kip, 1.67 kip
13.54 ft — 0.30 kip, 0.30 kip

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-2(oh1)		8.500 ft	
L (Span in the long direction) = Ls-1(oh2)		13.54 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	0.16 klf	0.025 klf

8.500 ft — 0.92 kip, 0.92 kip
13.54 ft — 0.19 kip, 0.19 kip

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

ID: **Navajo N-284**

DESIGN OF ROOF PANELS MARK R1, R2, R3, & R4

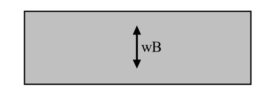
Notes:

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*

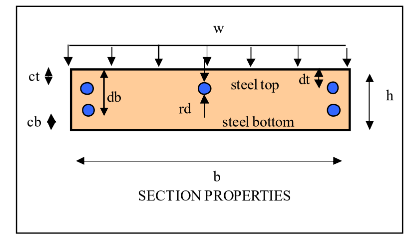
fr (rupture modulus)	530.3 psi	*ACI 9.5.2.3*
Ig = (b*h^3)/12	91.125 in^4	
Ag = (b*h)	54 in^4	
Yt = h/2	2.25 in^4	
Mcr	21.48	*ACI 9.5.2.3*
Beta_1	0.8	*ACI 10.2.7.3*
delta_initial	180	
delta_longterm	480	
B	8.830	
kd	0.776 in	
I.cr	11.45 in^4	
c	0 in	
a	0.32 in	

Ls



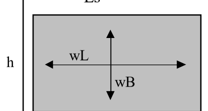
Bs

ONE-WAY SLAB



SECTION PROPERTIES

Ls



Bs

TWO-WAY SLAB

Geometric Properties

Ls (overall length of slab)	15.04 ft		
Bs (overall width of slab)	11.5 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)	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)	18 in	2	*{qty of overhangs in Bs direction}*
oh2 (overhang length and qty for ls)	18 in	1	*{qty of overhangs in ls direction}*
Cs (% of DL used for Seismic)	0.116	*{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)		0.0049
top mesh	ω =	0.0784 psi
$\rho_{provided}$ (reinforcement ratio provided)		0.0119
both layers	ω =	0.1904 psi
$\rho_{provided}$ (reinforcement ratio provided)		0.0138
	ω =	0.2208 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	4.34 kip-ft	0.036	Tension	0.9	3.96 kip-ft	0.38 kip-ft	5.06 kip-ft	O.K.	85.77%

		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	0.540 kip-ft	0.023 ft^3	0.55	0.941 kip-ft	O.K.	57.39%
Mneg (negative Moment) = (wB*oh2^2) / 2	0.540 kip-ft	0.023 ft^3	0.55	0.941 kip-ft	O.K.	57.39%

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	1.833 kip-ft	0.036	Tension	0.9	3.96 kip-ft		3.96 kip-ft	O.K.	46.30%

		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) = (wL*oh1^2) / 2	0.09	0.023 ft^3	0.55	0.941 kip-ft	O.K.	9.56%
Mneg (negative Moment) = (wL*oh2^2) / 2	0.09	0.023 ft^3	0.55	0.941 kip-ft	O.K.	9.56%

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)	2.04 kip	0.85	5.82 kip	4.95 kip	O.K.	41.22%
			per ACI 22.8			
Vu for side overhang 1 = wB*oh1	0.72 kip	0.55	2.98 kip	1.64 kip	O.K.	43.91%
Vu for side overhang 1 = wB*oh1	0.72 kip	0.55	2.98 kip	1.64 kip	O.K.	43.91%

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)	0.17 kip	0.85	5.82 kip	4.95 kip	O.K.	3.42%
			per ACI 22.8			
Vu for end overhang 2 = wL*oh2	0.04 kip	0.55	2.98 kip	1.64 kip	O.K.	2.29%
Vu for end overhang 2 = wL*oh2	0.04 kip	0.55	2.98 kip	1.64 kip	O.K.	2.29%

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	4.34 kip-ft	1.445 kip-ft	17.04 in^4	91.13 in^4	0.048 in	0.0049	0.9639	0.046 in	0.094 in	0.5667 in	0.2125 in	O.K.	O.K.	8.49%	44.46%
L	1.833 kip-ft	0.573 kip-ft	85.63 in^4	91.13 in^4	0.000 in	0.0049	0.9639	0.046 in	0.094 in	0.9027 in	0.3385 in	O.K.	O.K.	0.00%	27.91%

ID:	Navajo N-284
	DESIGN OF WALL MARKED W1

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

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	

Geometric Properties

X Corridinate	22
Y Corridinate	20
Direction of Wall	X
Center of gravity X	174.005
Center of gravity Y	20.000
Wall Weight	9728.000 lbs.
Central wall?	Yes
Wall that supports 2 roof panels?	Yes
lop (length of opening on wall)	0 ft
H (height of wall)	114.94 in
Lh (length of wall)	25.333 ft
Analyisis 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)	0.116
Eccentricity - Axial Load	1 in
Is wall Split	No

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)	7.09 psf	Earthquake Load (EL.lat)	5.82 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	560.213

Axial Pressure on Section	
PuB	2.66 kip

Assumption check	
Pu/Ag	55.417 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	94.38 psf

Lateral Pressure on Section	
Lw = W*(L^4 / H^4 + L^4)	0 klf
Hw = W*(L^4 / H^4 + L^4)	0.09 klf

Unfactored Laterally Applied Loads

Unfactored Pressure on Wall uWw	58.99 psf

Lateral Pressure on Section	
Lw = W*(L^4 / H^4 + L^4)	0 klf
Hw = W*(L^4 / H^4 + L^4)	0.06 klf

Unfactored Axially Applied Loads

Unfactored Pressure on Roof uWr	330.0825 psf

Axial Pressure on Section	
PB	1.68 kip

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	1.68 kip
Lateral	0 klf
Allowed service deflection	0.77 in
Msa	0.840 kip-in
M	0.847 kip-in
Ds	0.004 in
Check deflection	O.K.

Allowable Capacity

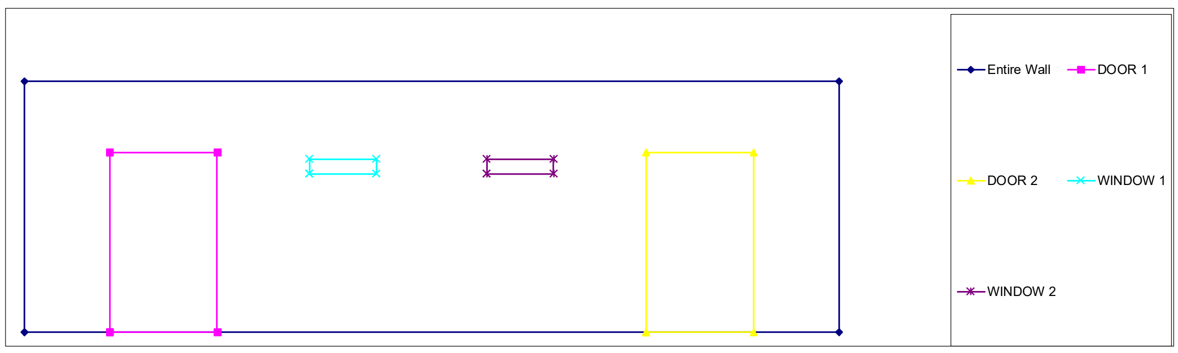
Ig = (b*h^3)/12	64 in^4
Ag = (b*h)	48 in^2
Yt = h/2	2
fr (rupture modulus)	530.330 psi
Mcr	16.971 kip-in
Beta_1	0.8
Trial Ast req'd	0.073 in^2
B	8.836162648
kd	0.542 in
I.cr	2.92 in^4
e_c	0.003
e_s	0.005
a	0.33483 psi
c	0.419 in
Ase	0.23 in^2
Icrdeflection	3.61 in^4
Ie	64.00 in^4
delta	150
r_t (maximum 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	1.141 kip-ft	

ACI eq. (14-6)		
Mu	1.670 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	82.84%	0.00%



REINFORCEMENT AT OPENINGS

Loading	
Pu (factorized load from roof)	0.48 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	2.66 ft	0 ft	3.34 ft	2.72 ft	1145.34	0.14 klf	0.62 klf	0.58 kip-ft
DOOR 2	19.33 ft	0 ft	3.34 ft	2.72 ft	1145.34	0.14 klf	0.62 klf	0.58 kip-ft
WINDOW 1	8.88 ft	6.05 ft	2.08 ft	2.96 ft	59.11	0.15 klf	0.63 klf	0.23 kip-ft
WINDOW 2	14.38 ft	6.05 ft	2.08 ft	2.96 ft	59.11	0.15 klf	0.63 klf	0.23 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	0.004 in^2	No. 3	1	15.39 kip-ft	O.K.
DOOR 2	0.9	0.004 in^2	No. 3	1	15.39 kip-ft	O.K.
WINDOW 1	0.9	0.002 in^2	No. 3	1	16.83 kip-ft	O.K.
WINDOW 2	0.9	0.002 in^2	No. 3	1	16.83 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
5	290	290	61.045	174.25	167.89	146.05	146.05

Total Tension	Base Anchors					
18.205	Dist	Tension (kip)	Shear	L - Dist	Moment +	Moment -
Base Anchor 1	14 in	3.64	12.21	290 in	0.205 kip*ft	87.991 kip*ft
Base Anchor 2	82 in	3.64	12.21	222 in	7.035 kip*ft	51.564 kip*ft
Base Anchor 3	162 in	3.64	12.21	142 in	27.458 kip*ft	21.097 kip*ft
Base Anchor 4	222 in	3.64	12.21	82 in	51.564 kip*ft	7.035 kip*ft
Base Anchor 5	290 in	3.64	12.21	14 in	87.991 kip*ft	0.205 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	5.961	40.00%	W4	0	304.000	3.062	0.000	77.571
Wall Connection 2	2	2.703	5.685	50.00%	W5	152	152.000	5.406	68.476	68.476
Wall Connection 3	2	1.531	5.731	40.00%	W3	304	0.000	3.062	77.571	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
7935	61045	53110	257	14582	OK	1587	(53110)	OK

RIGIDITY

CALCULATED VALUES	72%	Final	12.05120779

	Pier Label	Length (inches)	Height (inches)	Fixed Top? (Y/N)	Useable? (Y/N)	Stiffness (k) (1000 kip / IN)	Deflection (in / 1000 kip)
	Entire Wall	304	114.94	Y	Y	16.830	0.059
DOOR 1	A'	304	82.3	Y	Y	24.038	0.042
	A	31.92	82.3	Y	Y	0.804	1.244
	B	232	82.3	Y	Y	18.036	0.055
DOOR 2	B'	304	82.3	Y	Y	24.038	0.042
	C	231.96	82.3	Y	Y	18.033	0.055
	D	31.96	82.3	Y	Y	0.806	1.240
WINDOW 1	C'	304	6.82	Y	Y	297.115	0.003
	E	106.56	6.82	Y	Y	104.022	0.010
	F	172.48	6.82	Y	Y	168.514	0.006
WINDOW 2	D'	304	6.82	Y	Y	297.115	0.003
	G	172.56	6.82	Y	Y	168.593	0.006
	H	106.48	6.82	Y	Y	103.944	0.010

	Combine Logic					
	First Segment	Second Segment	Re-Name	Combine/Subtract	Method	Combined
DOOR 1	Entire Wall	A'	A'a	-	Deflection	0.018
	A	B	AB	+	Stiffness	18.840
	A'a	AB	A'b	+	Deflection	0.071
DOOR 2	A'b	B'	B'a	-	Deflection	0.029
	C	D	CD	+	Stiffness	18.840
	B'a	CD	B'b	+	Deflection	0.082
WINDOW 1	B'b	C'	C'a	-	Deflection	0.079
	E	F	EF	+	Stiffness	272.537
	C'a	EF	C'b	+	Deflection	0.083
WINDOW 2	C'b	D'	D'a	-	Deflection	0.079
	G	H	GH	+	Stiffness	272.536
	D'a	GH	Final	+	Deflection	0.083

ID:	Navajo N-284
	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 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*

Geometric Properties

X Corridinate	22
Y Corridinate	112
Direction of Wall	X
Center of gravity X	174.001
Center of gravity Y	112.000
Wall Weight	11814.000 lbs.
Central wall?	Yes
Wall that supports 2 roof panels?	Yes
lop (length of opening on wall)	0 ft
H (height of wall)	114.94 in
Lh (length of wall)	25.333 ft
Analyisis 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)	0.116
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)	7.09 psf	Earthquake Load (EL.lat)	5.82 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	560.213

Axial Pressure on Section	
PuB	2.72 kip

Assumption check	
Pu/Ag	56.667 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	1.74 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 / H^4 + L^4)	0 klf
Hw = W*(L^4 / H^4 + L^4)	0.09 klf

Unfactored Laterally Applied Loads

Unfactored Pressure on Wall uWw	58.99 psf

Lateral Pressure on Section	
Lw = W*(L^4 / H^4 + L^4)	0 klf
Hw = W*(L^4 / H^4 + L^4)	0.06 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	1.74 kip
Lateral	0 klf
Allowed service deflection	0.77 in
Msa	0.870 kip-in
M	0.878 kip-in
Ds	0.004 in
Check deflection	O.K.

Allowable Capacity

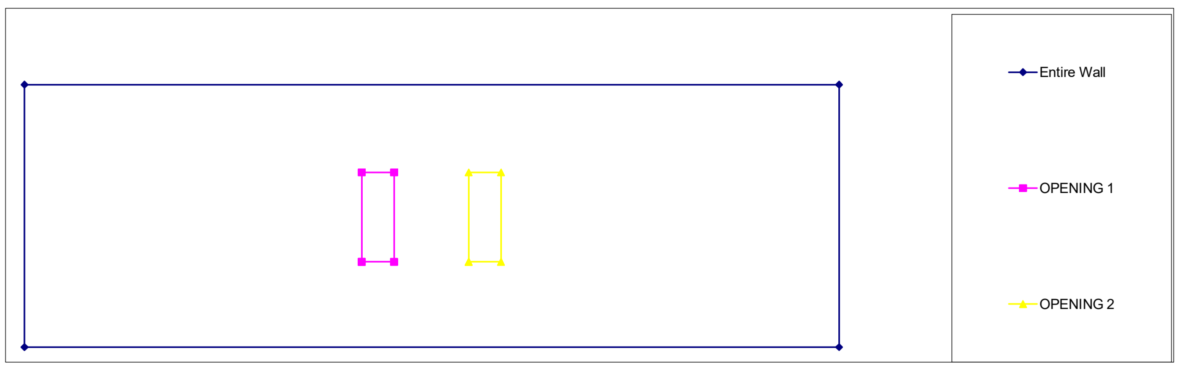
Ig = (b*h^3)/12	64 in^4
Ag = (b*h)	48 in^2
Yt = h/2	2
fr (rupture modulus)	530.330 psi
Mcr	16.971 kip-in
Beta_1	0.8
Trial Ast req'd	0.073 in^2
B	8.836162648
kd	0.542 in
I.cr	2.92 in^4
e_c	0.003
e_s	0.005
a	0.33483 psi
c	0.419 in
Ase	0.23 in^2
Icrdeflection	3.61 in^4
Ie	64.00 in^4
delta	150
r_t (maximum 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	1.143 kip-ft	

ACI eq. (14-6)		
Mu	1.690 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	83.83%	0.00%



REINFORCEMENT AT OPENINGS

Loading	
Pu (factorized load from roof)	0.48 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
OPENING 1	10.5 ft	3.13 ft	1 ft	3.2 ft	162.42	0.16 klf	0.64 klf	0.05 kip-ft
OPENING 2	13.83 ft	3.13 ft	1 ft	3.2 ft	162.42	0.16 klf	0.64 klf	0.05 kip-ft

Flexure

Opening	ϕb	As req'd	Bar size	qty req'd:	**ϕMn** = ϕAsFy(db - a/2)	Check ϕMn > Mu
OPENING 1	0.9	0 in^2	No. 3	0	0 kip-ft	O.K.
OPENING 2	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
6	298	298	61.420	201.67	201.67	253.56	253.56

Total Tension	Base Anchors					
21.502	Dist	Tension (kip)	Shear	L - Dist	Moment +	Moment -
Base Anchor 1	6 in	***3.47***	***6.29***	298 in	0.035 kip*ft	86.147 kip*ft
Base Anchor 2	54 in	3.64	12.21	250 in	2.969 kip*ft	63.636 kip*ft
Base Anchor 3	122 in	3.64	12.21	182 in	15.155 kip*ft	33.726 kip*ft
Base Anchor 4	182 in	3.64	12.21	122 in	33.726 kip*ft	15.155 kip*ft
Base Anchor 5	250 in	3.64	12.21	54 in	63.636 kip*ft	2.969 kip*ft
Base Anchor 6	298 in	***3.47***	***6.29***	6 in	86.147 kip*ft	0.035 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***	8.941	60.00%	W4	0	304.000	3.062	0.000	77.571
Wall Connection 2	2	***2.703***	4.244	100.00%	P1-1	112	192.000	4.244	39.611	67.904
Wall Connection 3	2	***2.703***	5.685	50.00%	W5	152	152.000	5.406	68.476	68.476
Wall Connection 4	2	***2.703***	4.244	100.00%	P1-2	192	112.000	4.244	67.904	39.611
Wall Connection 5	2	***1.531***	8.597	60.00%	W3	304	0.000	3.062	77.571	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
10479	61420	50941	346	19594	OK	1747	(50941)	OK

RIGIDITY

CALCULATED VALUES	96%	Final	16.19320523

	Pier Label	Length (inches)	Height (inches)	Fixed Top? (Y/N)	Useable? (Y/N)	Stiffness (k) (1000 kip / IN)	Deflection (in / 1000 kip)
	Entire Wall	304	114.94	Y	Y	16.830	0.059
OPENING 1	A'	304	38.98	Y	Y	51.709	0.019
	A	126	38.98	Y	Y	20.883	0.048
	B	166	38.98	Y	Y	27.878	0.036
OPENING 2	B'	304	38.98	Y	Y	51.709	0.019
	C	165.96	38.98	Y	Y	27.871	0.036
	D	126.04	38.98	Y	Y	20.890	0.048

	Combine Logic					
	First Segment	Second Segment	Re-Name	Combine/Subtract	Method	Combined
OPENING 1	Entire Wall	A'	A'a	-	Deflection	0.040
	A	B	AB	+	Stiffness	48.762
	A'a	AB	A'b	+	Deflection	0.061
OPENING 2	A'b	B'	B'a	-	Deflection	0.041
	C	D	CD	+	Stiffness	48.762
	B'a	CD	Final	+	Deflection	0.062

ID:	Navajo N-284
	DESIGN OF WALL MARKED W3

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

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	

Geometric Properties	
X Corridinate	328
Y Corridinate	18
Direction of Wall	Y
Center of gravity X	328.000
Center of gravity Y	68.923
Wall Weight	3405.000 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)	10.000 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)	0.116
Eccentricity - Axial Load	1 in
Is wall Split	No

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)	7.09 psf	Earthquake Load (EL.lat)	5.82 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	560.213

Axial Pressure on Section	
PuB	2.63 kip

Assumption check	
Pu/Ag	54.792 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	1.65 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)	0.07 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	1.65 kip	
Lateral	0.02 klf	
Allowed service deflection	0.64 in	
Msa	2.745 kip-in	
M	2.761 kip-in	
Ds	0.010 in	
Check deflection	O.K.	

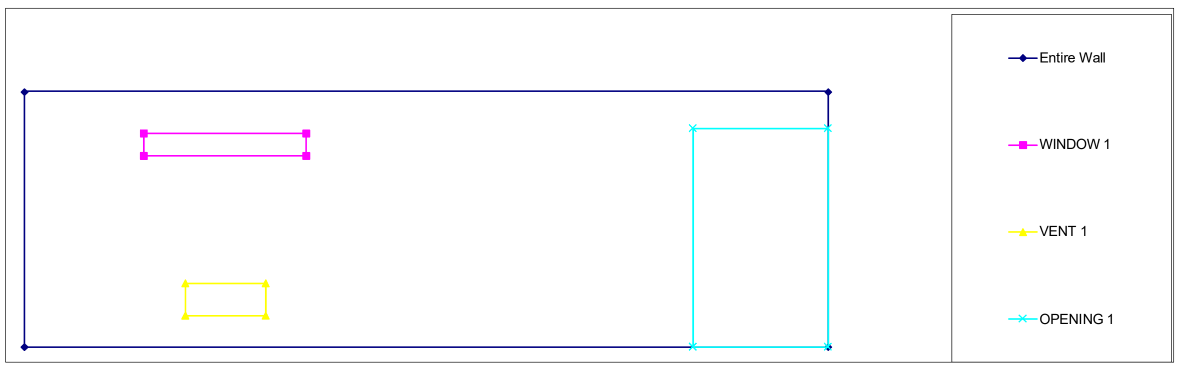
Allowable Capacity	
Ig = (b*h^3)/12	64 in^4
Ag = (b*h)	48 in^2
Yt = h/2	2
fr (rupture modulus)	530.330 psi
Mcr	16.971 kip-in
Beta_1	0.8
Trial Ast req'd	0.073 in^2
B	8.836162648
kd	0.542 in
I.cr	2.92 in^4
e_c	0.003
e_s	0.005
a	0.33483 psi
c	0.419 in
Ase	0.23 in^2
Icrdeflection	3.61 in^4
Ie	64.00 in^4
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	0.670 kip-ft	

ACI eq. (14-6)		
Mu	0.860 kip-ft	0.380 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	42.66%	18.85%



REINFORCEMENT AT OPENINGS

Loading	
Pu (factorized load from roof)	0.48 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
WINDOW 1	1.49 ft	5.99 ft	2.02 ft	1.32 ft	69.69	0.07 klf	0.55 klf	0.19 kip-ft
VENT 1	2 ft	1 ft	1 ft	6 ft	50.00	0.3 klf	0.78 klf	0.07 kip-ft
OPENING 1	8.32 ft	0 ft	1.68 ft	1.15 ft	575.40	0.06 klf	0.54 klf	0.13 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	0.003 in^2	No. 3	1	7.04 kip-ft	O.K.
VENT 1	0.9	0 in^2	No. 3	0	0 kip-ft	O.K.
OPENING 1	0.9	0.002 in^2	No. 3	1	6.03 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
3	88	90	36.627	42.22	42.90	43.25	64.87

Total Tension	Base Anchors					
10.923	Dist	Tension (kip)	Shear	L - Dist	Moment +	Moment -
Base Anchor 1	30 in	3.64	12.21	90 in	3.103 kip*ft	27.308 kip*ft
Base Anchor 2	60 in	3.64	12.21	60 in	12.413 kip*ft	12.137 kip*ft
Base Anchor 3	88 in	3.64	12.21	32 in	26.701 kip*ft	3.452 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	***2.703***	6.983	25.00%	W1	2	118.000	5.406	0.901	53.159
Wall Connection 2	2	***2.703***	6.137	18.42%	W2	94	26.000	5.406	42.347	11.713

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
6214	36627	30413	572	16204	OK	2071	(30413)	OK

RIGIDITY

CALCULATED VALUES	80%	Final	5.464884811

	Pier Label	Length (inches)	Height (inches)	Fixed Top? (Y/N)	Useable? (Y/N)	Stiffness (k) (1000 kip / IN)	Deflection (in / 1000 kip)
	Entire Wall	120	96	Y	Y	6.868	0.146
WINDOW 1	A'	120	8.28	Y	Y	96.465	0.010
	A	17.88	8.28	Y	Y	13.436	0.074
	B	77.88	8.28	Y	Y	62.470	0.016
VENT 1	B'	120	12	Y	Y	66.445	0.015
	C	24	12	Y	Y	12.308	0.081
	D	84	12	Y	Y	46.351	0.022
OPENING 1	C'	120	82.2	Y	Y	8.416	0.119
	E	99.84	82.2	Y	Y	6.605	0.151
	F	0	82.2	Y	N	0.000	0.000

	Combine Logic					
	First Segment	Second Segment	Re-Name	Combine/Subtract	Method	Combined
WINDOW 1	Entire Wall	A'	A'a	-	Deflection	0.135
	A	B	AB	+	Stiffness	75.906
	A'a	AB	A'b	+	Deflection	0.148
VENT 1	A'b	B'	B'a	-	Deflection	0.133
	C	D	CD	+	Stiffness	58.659
	B'a	CD	B'b	+	Deflection	0.150
OPENING 1	B'b	C'	C'a	-	Deflection	0.032
	E	F	EF	+	Stiffness	6.605
	C'a	EF	Final	+	Deflection	0.183

ID:	Navajo N-284
	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 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

Geometric Properties

X Corridinate	20
Y Corridinate	18
Direction of Wall	Y
Center of gravity X	20.000
Center of gravity Y	78.917
Wall Weight	3979.000 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)	10.000 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)	0.116
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)	7.09 psf	Earthquake Load (EL.lat)	5.82 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	560.213

Axial Pressure on Section	
PuB	2.67 kip

Assumption check	
Pu/Ag	55.625 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	1.69 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)	0.07 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	1.69 kip
Lateral	0.02 klf
Allowed service deflection	0.64 in
Msa	2.765 kip-in
M	2.781 kip-in
Ds	0.010 in
Check deflection	O.K.

Allowable Capacity

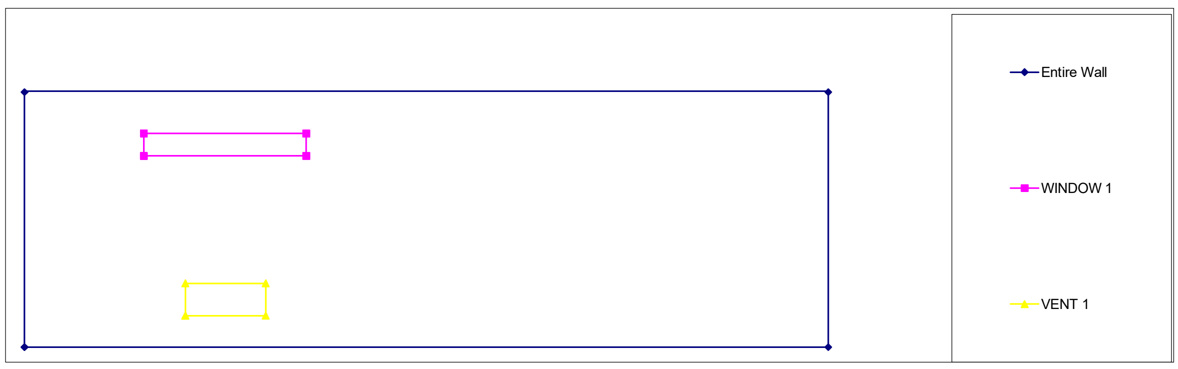
Ig = (b*h^3)/12	64 in^4
Ag = (b*h)	48 in^2
Yt = h/2	2
fr (rupture modulus)	530.330 psi
Mcr	16.971 kip-in
Beta_1	0.8
Trial Ast req'd	0.073 in^2
B	8.836162648
kd	0.542 in
I.cr	2.92 in^4
e_c	0.003
e_s	0.005
a	0.33483 psi
c	0.419 in
Ase	0.23 in^2
Icrdeflection	3.61 in^4
Ie	64.00 in^4
delta	150
r_t (maximum 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	0.671 kip-ft	

ACI eq. (14-6)		
Mu	0.860 kip-ft	0.380 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	42.66%	18.85%



REINFORCEMENT AT OPENINGS

Loading	
Pu (factorized load from roof)	0.48 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
WINDOW 1	1.49 ft	5.99 ft	2.02 ft	1.32 ft	69.69	0.07 klf	0.55 klf	0.19 kip-ft
VENT 1	2 ft	1 ft	1 ft	6 ft	50.00	0.3 klf	0.78 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	0.003 in^2	No. 3	1	7.04 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	90	108	36.627	39.93	45.41	43.25	64.87

Total Tension	Base Anchors					
10.923	Dist	Tension (kip)	Shear	L - Dist	Moment +	Moment -
Base Anchor 1	12 in	3.64	12.21	108 in	0.485 kip*ft	32.769 kip*ft
Base Anchor 2	60 in	3.64	12.21	60 in	12.137 kip*ft	10.114 kip*ft
Base Anchor 3	90 in	3.64	12.21	30 in	27.308 kip*ft	2.528 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	2.703	6.983	25.00%	W1	2	118.000	5.406	0.901	53.159
Wall Connection 2	2	2.703	6.137	18.42%	W2	94	26.000	5.406	42.347	11.713

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
7537	36627	29090	696	19714	OK	2512	(29090)	OK

RIGIDITY

CALCULATED VALUES	97%	Final	6.648693271

	Pier Label	Length (inches)	Height (inches)	Fixed Top? (Y/N)	Useable? (Y/N)	Stiffness (k) (1000 kip / IN)	Deflection (in / 1000 kip)
	Entire Wall	120	96	Y	Y	6.868	0.146
WINDOW 1	A'	120	8.28	Y	Y	96.465	0.010
	A	17.88	8.28	Y	Y	13.436	0.074
	B	77.88	8.28	Y	Y	62.470	0.016
VENT 1	B'	120	12	Y	Y	66.445	0.015
	C	24	12	Y	Y	12.308	0.081
	D	84	12	Y	Y	46.351	0.022

	Combine Logic					
	First Segment	Second Segment	Re-Name	Combine/Subtract	Method	Combined
WINDOW 1	Entire Wall	A'	A'a	-	Deflection	0.135
	A	B	AB	+	Stiffness	75.906
	A'a	AB	A'b	+	Deflection	0.148
VENT 1	A'b	B'	B'a	-	Deflection	0.133
	C	D	CD	+	Stiffness	58.659
	B'a	CD	Final	+	Deflection	0.150

ID:	Navajo N-284
	DESIGN OF WALL MARKED W5

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

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	

Geometric Properties

X Corridinate	174
Y Corridinate	22
Direction of Wall	Y
Center of gravity X	174.000
Center of gravity Y	66.000
Wall Weight	4087.000 lbs.
Central wall?	Yes
Wall that supports 2 roof panels?	Yes
lop (length of opening on wall)	0 ft
H (height of wall)	134 in
Lh (length of wall)	7.333 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)	0.116
Eccentricity - Axial Load	1 in
Is wall Split	No

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)	7.09 psf	Earthquake Load (EL.lat)	5.82 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	560.213

Axial Pressure on Section	
PuB	2.79 kip

Assumption check	
Pu/Ag	58.125 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	1.8 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)	0.08 klf
Hw = W*(H^4 / H^4 + L^4)	0.01 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)	0.05 klf
Hw = W*(H^4 / H^4 + L^4)	0.01 klf

Shear

Factored Loading per ACI	ACI eq. 9-3
Vu = wuB*(Bw-2db) / 2	0.15
Phi*Vc/2	1.33
Check Shear ACI 11.5.5.1	O.K.

Deflection *ACI 14.8.4*

Service Loads	
Axial	1.80 kip
Lateral	0.05 klf
Allowed service deflection	0.89 in
Msa	10.252 kip-in
M	10.379 kip-in
Ds	0.071 in
Check deflection	O.K.

Allowable Capacity

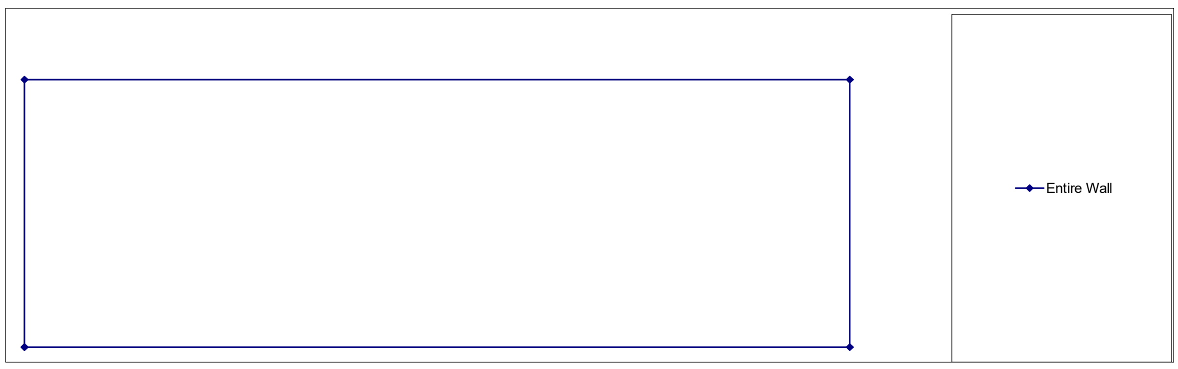
Ig = (b*h^3)/12	64 in^4
Ag = (b*h)	48 in^2
Yt = h/2	2
fr (rupture modulus)	530.330 psi
Mcr	16.971 kip-in
Beta_1	0.8
Trial Ast req'd	0.073 in^2
B	8.836162648
kd	0.542 in
I.cr	2.92 in^4
e_c	0.003
e_s	0.005
a	0.33483 psi
c	0.419 in
Ase	0.24 in^2
Icrdeflection	3.76 in^4
Ie	64.00 in^4
delta	150
r_t (maximum 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	0.276 kip-ft	

ACI eq. (14-6)		
Mu	0.490 kip-ft	0.540 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	24.31%	26.79%



REINFORCEMENT AT OPENINGS

Loading	
Pu (factorized load from roof)	0.48 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

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 in Shear	Maximum R - Distance	Maximum L - Distance	Shear kip	Moment + kip - ft	Moment - kip - ft	Moment + kip - ft	Moment - kip - ft
2	70	70	24.418	22.64	22.64	22.45	22.45

Total Tension	Base Anchors					
7.282	Dist	Tension (kip)	Shear	L - Dist	Moment +	Moment -
Base Anchor 1	18 in	3.64	12.21	70 in	1.404 kip*ft	21.239 kip*ft
Base Anchor 2	70 in	3.64	12.21	18 in	21.239 kip*ft	1.404 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***	13.967	50.00%	W1	0	88.000	3.062	0.000	22.455
Wall Connection 2	2	***1.531***	4.384	13.16%	W2	88	0.000	3.062	22.455	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
3179	24418	21239	352	20365	OK	1589	(21239)	OK

RIGIDITY

CALCULATED VALUES	100%	Final	2.469462921

Pier Label	Length (inches)	Height (inches)	Fixed Top? (Y/N)	Useable? (Y/N)	Stiffness (k) (1000 kip / IN)	Deflection (in / 1000 kip)
Entire Wall	88	134	Y	Y	2.469	0.405

Combine Logic					
First Segment	Second Segment	Re-Name	Combine/Subtract	Method	Combined
Entire Wall	0	Final			2.469

ID:	Navajo N-284
	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	

Geometric Properties

X Corridinate	22
Y Corridinate	256
Direction of Wall	X
Center of gravity X	174.005
Center of gravity Y	256.000
Wall Weight	9728.000 lbs.
Central wall?	Yes
Wall that supports 2 roof panels?	Yes
lop (length of opening on wall)	0 ft
H (height of wall)	114.94 in
Lh (length of wall)	25.333 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)	0.116
Eccentricity - Axial Load	1 in
Is wall Split	No

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)	7.09 psf	Earthquake Load (EL.lat)	5.82 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	560.213

Axial Pressure on Section	
PuB	2.66 kip

Assumption check	
Pu/Ag	55.417 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	1.68 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 / H^4 + L^4)	0 klf
Hw = W*(L^4 / H^4 + L^4)	0.09 klf

Unfactored Laterally Applied Loads

Unfactored Pressure on Wall uWw	58.99 psf

Lateral Pressure on Section	
Lw = W*(L^4 / H^4 + L^4)	0 klf
Hw = W*(L^4 / H^4 + L^4)	0.06 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	1.68 kip
Lateral	0 klf
Allowed service deflection	0.77 in
Msa	0.840 kip-in
M	0.847 kip-in
Ds	0.004 in
Check deflection	O.K.

Allowable Capacity

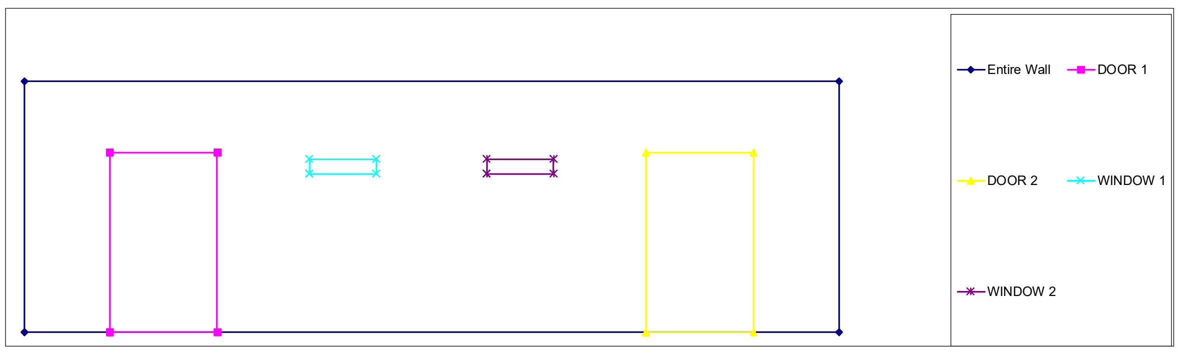
Ig = (b*h^3)/12	64 in^4
Ag = (b*h)	48 in^2
Yt = h/2	2
fr (rupture modulus)	530.330 psi
Mcr	16.971 kip-in
Beta_1	0.8
Trial Ast req'd	0.073 in^2
B	8.836162648
kd	0.542 in
I.cr	2.92 in^4
e_c	0.003
e_s	0.005
a	0.33483 psi
c	0.419 in
Ase	0.23 in^2
Icrdeflection	3.61 in^4
Ie	64.00 in^4
delta	150
r_t (maximum 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	1.141 kip-ft	

ACI eq. (14-6)		
Mu	1.670 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	82.84%	0.00%



REINFORCEMENT AT OPENINGS

Loading	
Pu (factorized load from roof)	0.48 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	2.66 ft	0 ft	3.34 ft	2.72 ft	1145.34	0.14 klf	0.62 klf	0.58 kip-ft
DOOR 2	19.33 ft	0 ft	3.34 ft	2.72 ft	1145.34	0.14 klf	0.62 klf	0.58 kip-ft
WINDOW 1	8.88 ft	6.05 ft	2.08 ft	2.96 ft	59.11	0.15 klf	0.63 klf	0.23 kip-ft
WINDOW 2	14.38 ft	6.05 ft	2.08 ft	2.96 ft	59.11	0.15 klf	0.63 klf	0.23 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	0.004 in^2	No. 3	1	15.39 kip-ft	O.K.
DOOR 2	0.9	0.004 in^2	No. 3	1	15.39 kip-ft	O.K.
WINDOW 1	0.9	0.002 in^2	No. 3	1	16.83 kip-ft	O.K.
WINDOW 2	0.9	0.002 in^2	No. 3	1	16.83 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
5	290	290	61.045	174.25	167.89	146.05	146.05

Total Tension	Base Anchors					
18.205	Dist	Tension (kip)	Shear	L - Dist	Moment +	Moment -
Base Anchor 1	14 in	3.64	12.21	290 in	0.205 kip*ft	87.991 kip*ft
Base Anchor 2	82 in	3.64	12.21	222 in	7.035 kip*ft	51.564 kip*ft
Base Anchor 3	162 in	3.64	12.21	142 in	27.458 kip*ft	21.097 kip*ft
Base Anchor 4	222 in	3.64	12.21	82 in	51.564 kip*ft	7.035 kip*ft
Base Anchor 5	290 in	3.64	12.21	14 in	87.991 kip*ft	0.205 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***	5.961	40.00%	W9	0	304.000	3.062	0.000	77.571
Wall Connection 2	2	***2.703***	5.685	50.00%	W10	152	152.000	5.406	68.476	68.476
Wall Connection 3	2	***1.531***	5.731	40.00%	W8	304	0.000	3.062	77.571	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
7935	61045	53110	257	14582	OK	1587	(53110)	OK

RIGIDITY

CALCULATED VALUES	72%	Final	12.05120779

	Pier Label	Length (inches)	Height (inches)	Fixed Top? (Y/N)	Useable? (Y/N)	Stiffness (k) (1000 kip / IN)	Deflection (in / 1000 kip)
	Entire Wall	304	114.94	Y	Y	16.830	0.059
DOOR 1	A'	304	82.3	Y	Y	24.038	0.042
	A	31.92	82.3	Y	Y	0.804	1.244
	B	232	82.3	Y	Y	18.036	0.055
DOOR 2	B'	304	82.3	Y	Y	24.038	0.042
	C	231.96	82.3	Y	Y	18.033	0.055
	D	31.96	82.3	Y	Y	0.806	1.240
WINDOW 1	C'	304	6.82	Y	Y	297.115	0.003
	E	106.56	6.82	Y	Y	104.022	0.010
	F	172.48	6.82	Y	Y	168.514	0.006
WINDOW 2	D'	304	6.82	Y	Y	297.115	0.003
	G	172.56	6.82	Y	Y	168.593	0.006
	H	106.48	6.82	Y	Y	103.944	0.010

	Combine Logic					
	First Segment	Second Segment	Re-Name	Combine/Subtract	Method	Combined
DOOR 1	Entire Wall	A'	A'a	-	Deflection	0.018
	A	B	AB	+	Stiffness	18.840
	A'a	AB	A'b	+	Deflection	0.071
DOOR 2	A'b	B'	B'a	-	Deflection	0.029
	C	D	CD	+	Stiffness	18.840
	B'a	CD	B'b	+	Deflection	0.082
WINDOW 1	B'b	C'	C'a	-	Deflection	0.079
	E	F	EF	+	Stiffness	272.537
	C'a	EF	C'b	+	Deflection	0.083
WINDOW 2	C'b	D'	D'a	-	Deflection	0.079
	G	H	GH	+	Stiffness	272.536
	D'a	GH	Final	+	Deflection	0.083

ID:	Navajo N-284
	DESIGN OF WALL MARKED W7

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.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	22
Y Corridinate	164
Direction of Wall	X
Center of gravity X	174.001
Center of gravity Y	164.000
Wall Weight	11814.000 lbs.
Central wall?	Yes
Wall that supports 2 roof panels?	Yes
lop (length of opening on wall)	0 ft
H (height of wall)	114.94 in
Lh (length of wall)	25.333 ft
Analyisis 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)	0.116
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)	7.09 psf	Earthquake Load (EL.lat)	5.82 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	560.213

Axial Pressure on Section	
PuB	2.72 kip

Assumption check	
Pu/Ag	56.667 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	1.74 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 / H^4 + L^4)	0 klf
Hw = W*(L^4 / H^4 + L^4)	0.09 klf

Unfactored Laterally Applied Loads

Unfactored Pressure on Wall uWw	58.99 psf

Lateral Pressure on Section	
Lw = W*(L^4 / H^4 + L^4)	0 klf
Hw = W*(L^4 / H^4 + L^4)	0.06 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	1.74 kip
Lateral	0 klf
Allowed service deflection	0.77 in
Msa	0.870 kip-in
M	0.878 kip-in
Ds	0.004 in
Check deflection	O.K.

Allowable Capacity

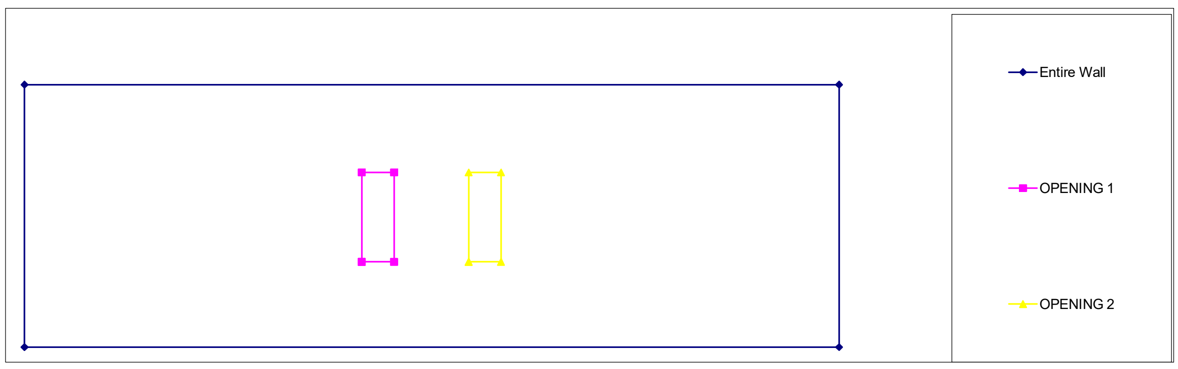
Ig = (b*h^3)/12	64 in^4
Ag = (b*h)	48 in^2
Yt = h/2	2
fr (rupture modulus)	530.330 psi
Mcr	16.971 kip-in
Beta_1	0.8
Trial Ast req'd	0.073 in^2
B	8.836162648
kd	0.542 in
I.cr	2.92 in^4
e_c	0.003
e_s	0.005
a	0.33483 psi
c	0.419 in
Ase	0.23 in^2
Icrdeflection	3.61 in^4
Ie	64.00 in^4
delta	150
r_t (maximum 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	1.143 kip-ft	

ACI eq. (14-6)		
Mu	1.690 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	83.83%	0.00%



REINFORCEMENT AT OPENINGS

Loading	
Pu (factorized load from roof)	0.48 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
OPENING 1	10.5 ft	3.13 ft	1 ft	3.2 ft	162.42	0.16 klf	0.64 klf	0.05 kip-ft
OPENING 2	13.83 ft	3.13 ft	1 ft	3.2 ft	162.42	0.16 klf	0.64 klf	0.05 kip-ft

Flexure

Opening	ϕb	As req'd	Bar size	qty req'd:	**ϕMn** = ϕAsFy(db - a/2)	Check ϕMn > Mu
OPENING 1	0.9	0 in^2	No. 3	0	0 kip-ft	O.K.
OPENING 2	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	Maximum	Maximum	Shear	Moment +	Moment -	Moment +	Moment -
in Shear	R - Distance	L - Distance	kip	kip - ft	kip - ft	kip - ft	kip - ft
6	298	298	61.420	201.67	201.67	253.56	253.56

Total Tension	Base Anchors					
21.502	Dist	Tension (kip)	Shear	L - Dist	Moment +	Moment -
Base Anchor 1	6 in	***3.47***	***6.29***	298 in	0.035 kip*ft	86.147 kip*ft
Base Anchor 2	54 in	3.64	12.21	250 in	2.969 kip*ft	63.636 kip*ft
Base Anchor 3	122 in	3.64	12.21	182 in	15.155 kip*ft	33.726 kip*ft
Base Anchor 4	182 in	3.64	12.21	122 in	33.726 kip*ft	15.155 kip*ft
Base Anchor 5	250 in	3.64	12.21	54 in	63.636 kip*ft	2.969 kip*ft
Base Anchor 6	298 in	***3.47***	***6.29***	6 in	86.147 kip*ft	0.035 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***	8.941	60.00%	W9	0	304.000	3.062	0.000	77.571
Wall Connection 2	2	***2.703***	4.244	100.00%	P1-3	112	192.000	4.244	39.611	67.904
Wall Connection 3	2	***2.703***	5.685	50.00%	W10	152	152.000	5.406	68.476	68.476
Wall Connection 4	2	***2.703***	4.244	100.00%	P1-4	192	112.000	4.244	67.904	39.611
Wall Connection 5	2	***1.531***	8.597	60.00%	W8	304	0.000	3.062	77.571	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
10479	61420	50941	346	19594	OK	1747	(50941)	OK

RIGIDITY

CALCULATED VALUES	96%	Final	16.19320523

	Pier Label	Length (inches)	Height (inches)	Fixed Top? (Y/N)	Useable? (Y/N)	Stiffness (k) (1000 kip / IN)	Deflection (in / 1000 kip)
	Entire Wall	304	114.94	Y	Y	16.830	0.059
OPENING 1	A'	304	38.98	Y	Y	51.709	0.019
	A	126	38.98	Y	Y	20.883	0.048
	B	166	38.98	Y	Y	27.878	0.036
OPENING 2	B'	304	38.98	Y	Y	51.709	0.019
	C	165.96	38.98	Y	Y	27.871	0.036
	D	126.04	38.98	Y	Y	20.890	0.048

	Combine Logic					
	First Segment	Second Segment	Re-Name	Combine/Subtract	Method	Combined
OPENING 1	Entire Wall	A'	A'a	-	Deflection	0.040
	A	B	AB	+	Stiffness	48.762
	A'a	AB	A'b	+	Deflection	0.061
OPENING 2	A'b	B'	B'a	-	Deflection	0.041
	C	D	CD	+	Stiffness	48.762
	B'a	CD	Final	+	Deflection	0.062

ID:	Navajo N-284
	DESIGN OF WALL MARKED W8

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*

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*

Geometric Properties

X Corridinate	328
Y Corridinate	138
Direction of Wall	Y
Center of gravity X	328.000
Center of gravity Y	206.793
Wall Weight	3405.000 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)	10.000 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)	0.116
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)	7.09 psf	Earthquake Load (EL.lat)	5.82 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	560.213

Axial Pressure on Section

PuB	2.63 kip

Assumption check

Pu/Ag	54.792 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	1.65 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)	0.07 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	1.65 kip
Lateral	0.02 klf
Allowed service deflection	0.64 in
Msa	2.745 kip-in
M	2.761 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	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	0.23 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

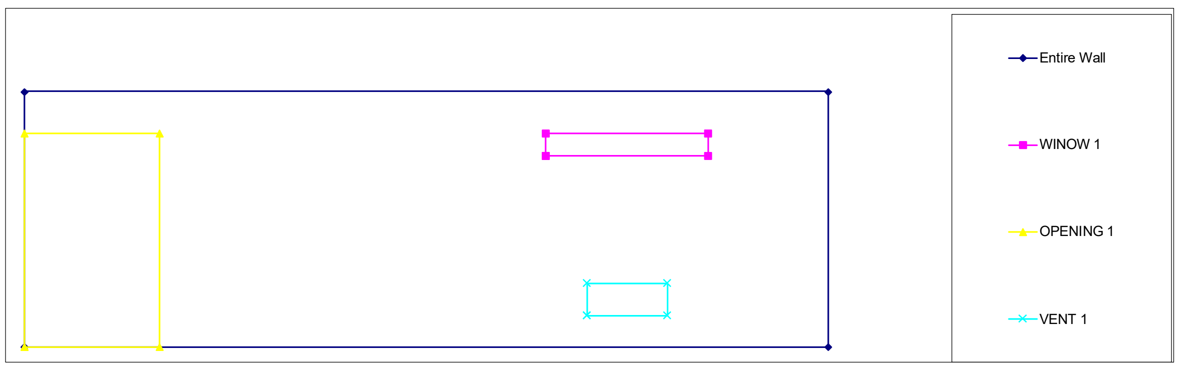
Span	Hw	Lw
net Tensile Strain	0.010	0.010
Check ACI 14.8.2.3	Tension	Tension
Mua	0.670 kip-ft	

ACI eq. (14-6)

Mu	0.860 kip-ft	0.380 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	42.66%	18.85%



REINFORCEMENT AT OPENINGS

Loading	
Pu (factorized load from roof)	0.48 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
WINOW 1	6.49 ft	5.99 ft	2.02 ft	1.32 ft	69.69	0.07 klf	0.55 klf	0.19 kip-ft
OPENING 1	0 ft	0 ft	1.68 ft	1.31 ft	561.96	0.07 klf	0.55 klf	0.13 kip-ft
VENT 1	7 ft	1 ft	1 ft	6 ft	50.00	0.3 klf	0.78 klf	0.07 kip-ft

Flexure

Opening	ϕb	As req'd	Bar size	qty req'd:	**ϕMn** = ϕAsFy(db - a/2)	Check ϕMn > Mu
WINOW 1	0.9	0.003 in^2	No. 3	1	7.04 kip-ft	O.K.
OPENING 1	0.9	0.002 in^2	No. 3	1	6.98 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	Maximum	Maximum	Shear	Moment +	Moment -	Moment +	Moment -
in Shear	R - Distance	L - Distance	kip	kip - ft	kip - ft	kip - ft	kip - ft
3	90	88	36.627	42.90	42.22	64.87	43.25

Total Tension	Base Anchors					
10.923	Dist	Tension (kip)	Shear	L - Dist	Moment +	Moment -
Base Anchor 1	32 in	3.64	12.21	88 in	3.452 kip*ft	26.701 kip*ft
Base Anchor 2	60 in	3.64	12.21	60 in	12.137 kip*ft	12.413 kip*ft
Base Anchor 3	90 in	3.64	12.21	30 in	27.308 kip*ft	3.103 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***	6.137	18.42%	W7	26	94.000	5.406	11.713	42.347
Wall Connection 2	2	***2.703***	6.983	25.00%	W6	118	2.000	5.406	53.159	0.901

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
6255	36627	30372	576	16319	OK	2085	(30372)	OK

RIGIDITY

CALCULATED VALUES	80%	Final	5.503823436

	Pier Label	Length (inches)	Height (inches)	Fixed Top? (Y/N)	Useable? (Y/N)	Stiffness (k) (1000 kip / IN)	Deflection (in / 1000 kip)
	Entire Wall	120	96	Y	Y	6.868	0.146
WINOW 1	A'	120	8.28	Y	Y	96.465	0.010
	A	77.88	8.28	Y	Y	62.470	0.016
	B	17.88	8.28	Y	Y	13.436	0.074
OPENING 1	B'	120	80.28	Y	Y	8.671	0.115
	C	0	80.28	Y	N	0.000	0.000
	D	99.84	80.28	Y	Y	6.821	0.147
VENT 1	C'	120	12	Y	Y	66.445	0.015
	E	84	12	Y	Y	46.351	0.022
	F	24	12	Y	Y	12.308	0.081

	Combine Logic					
	First Segment	Second Segment	Re-Name	Combine/Subtract	Method	Combined
WINOW 1	Entire Wall	A'	A'a	-	Deflection	0.135
	A	B	AB	+	Stiffness	75.906
	A'a	AB	A'b	+	Deflection	0.148
OPENING 1	A'b	B'	B'a	-	Deflection	0.033
	C	D	CD	+	Stiffness	6.821
	B'a	CD	B'b	+	Deflection	0.180
VENT 1	B'b	C'	C'a	-	Deflection	0.165
	E	F	EF	+	Stiffness	58.659
	C'a	EF	Final	+	Deflection	0.182

ID:	Navajo N-284	
	DESIGN OF WALL MARKED	W9

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.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	138
Direction of Wall	Y
Center of gravity X	20.000
Center of gravity Y	197.083
Wall Weight	3979.000 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)	10.000 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)	0.116
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)	7.09 psf	Earthquake Load (EL.lat)	5.82 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	560.213

Axial Pressure on Section	
PuB	2.67 kip

Assumption check	
Pu/Ag	55.625 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	1.69 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)	0.07 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	1.69 kip
Lateral	0.02 klf
Allowed service deflection	0.64 in
Msa	2.765 kip-in
M	2.781 kip-in
Ds	0.010 in
Check deflection	O.K.

Allowable Capacity

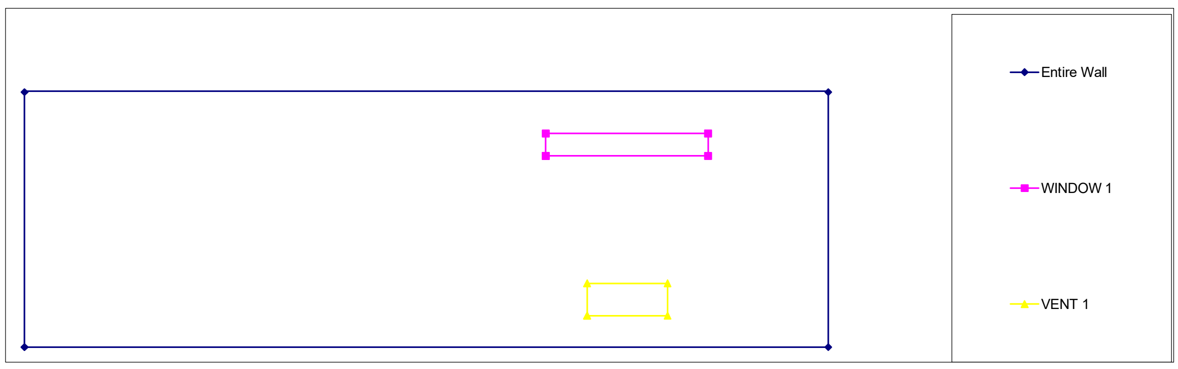
Ig = (b*h^3)/12	64 in^4
Ag = (b*h)	48 in^2
Yt = h/2	2
fr (rupture modulus)	530.330 psi
Mcr	16.971 kip-in
Beta_1	0.8
Trial Ast req'd	0.073 in^2
B	8.836162648
kd	0.542 in
I.cr	2.92 in^4
e_c	0.003
e_s	0.005
a	0.33483 psi
c	0.419 in
Ase	0.23 in^2
Icrdeflection	3.61 in^4
Ie	64.00 in^4
delta	150
r_t (maximum 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	0.671 kip-ft	

ACI eq. (14-6)		
Mu	0.860 kip-ft	0.380 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	42.66%	18.85%



REINFORCEMENT AT OPENINGS

Loading	
Pu (factorized load from roof)	0.48 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
WINDOW 1	6.49 ft	5.99 ft	2.02 ft	1.32 ft	69.69	0.07 klf	0.55 klf	0.19 kip-ft
VENT 1	7 ft	1 ft	1 ft	6 ft	50.00	0.3 klf	0.78 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	0.003 in^2	No. 3	1	7.04 kip-ft	O.K.
VENT 1	0.9	0 in^2	No. 3	0	0 kip-ft	O.K.

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
3	90	108	36.627	39.93	45.41	64.87	43.25

Total Tension	Base Anchors					
10.923	Dist	Tension (kip)	Shear	L - Dist	Moment +	Moment -
Base Anchor 1	12 in	3.64	12.21	108 in	0.485 kip*ft	32.769 kip*ft
Base Anchor 2	60 in	3.64	12.21	60 in	12.137 kip*ft	10.114 kip*ft
Base Anchor 3	90 in	3.64	12.21	30 in	27.308 kip*ft	2.528 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***	6.137	18.42%	W7	26	94.000	5.406	11.713	42.347
Wall Connection 2	2	***2.703***	6.983	25.00%	W6	118	2.000	5.406	53.159	0.901

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
7537	36627	29090	696	19714	OK	2512	(29090)	OK

RIGIDITY

CALCULATED VALUES	97%	Final	6.648693271

	Pier Label	Length (inches)	Height (inches)	Fixed Top? (Y/N)	Useable? (Y/N)	Stiffness (k) (1000 kip / IN)	Deflection (in / 1000 kip)
	Entire Wall	120	96	Y	Y	6.868	0.146
WINDOW 1	A'	120	8.28	Y	Y	96.465	0.010
	A	77.88	8.28	Y	Y	62.470	0.016
	B	17.88	8.28	Y	Y	13.436	0.074
VENT 1	B'	120	12	Y	Y	66.445	0.015
	C	84	12	Y	Y	46.351	0.022
	D	24	12	Y	Y	12.308	0.081

	Combine Logic					
	First Segment	Second Segment	Re-Name	Combine/Subtract	Method	Combined
WINDOW 1	Entire Wall	A'	A'a	-	Deflection	0.135
	A	B	AB	+	Stiffness	75.906
	A'a	AB	A'b	+	Deflection	0.148
VENT 1	A'b	B'	B'a	-	Deflection	0.133
	C	D	CD	+	Stiffness	58.659
	B'a	CD	Final	+	Deflection	0.150

ID:	Navajo N-284
	DESIGN OF WALL MARKED W10

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.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	174
Y Corridinate	166
Direction of Wall	Y
Center of gravity X	174.000
Center of gravity Y	210.000
Wall Weight	4087.000 lbs.
Central wall?	Yes
Wall that supports 2 roof panels?	Yes
lop (length of opening on wall)	0 ft
H (height of wall)	134 in
Lh (length of wall)	7.333 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)	0.116
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)	7.09 psf	Earthquake Load (EL.lat)	5.82 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	560.213

Axial Pressure on Section	
PuB	2.79 kip

Assumption check	
Pu/Ag	58.125 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	1.8 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)	0.08 klf
Hw = W*(H^4 / H^4 + L^4)	0.01 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)	0.05 klf
Hw = W*(H^4 / H^4 + L^4)	0.01 klf

Shear

Factored Loading per ACI	ACI eq. 9-3
Vu = wuB*(Bw-2db) / 2	0.15
Phi*Vc/2	1.33
Check Shear ACI 11.5.5.1	O.K.

Deflection ACI 14.8.4

Service Loads	
Axial	1.80 kip
Lateral	0.05 klf
Allowed service deflection	0.89 in
Msa	10.252 kip-in
M	10.379 kip-in
Ds	0.071 in
Check deflection	O.K.

Allowable Capacity

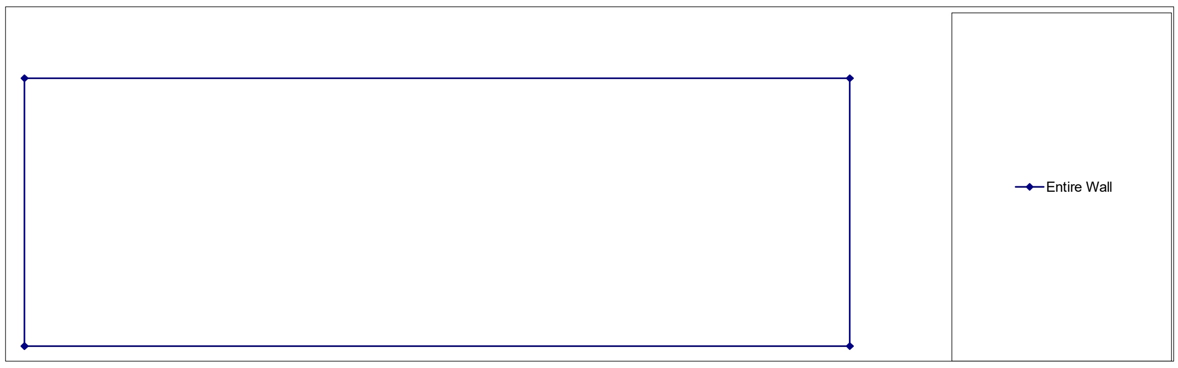
Ig = (b*h^3)/12	64 in^4
Ag = (b*h)	48 in^2
Yt = h/2	2
fr (rupture modulus)	530.330 psi
Mcr	16.971 kip-in
Beta_1	0.8
Trial Ast req'd	0.073 in^2
B	8.836162648
kd	0.542 in
I.cr	2.92 in^4
e_c	0.003
e_s	0.005
a	0.33483 psi
c	0.419 in
Ase	0.24 in^2
Icrdeflection	3.76 in^4
Ie	64.00 in^4
delta	150
r_t (maximum 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	0.276 kip-ft	

ACI eq. (14-6)		
Mu	0.490 kip-ft	0.540 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	24.31%	26.79%



REINFORCEMENT AT OPENINGS

Loading	
Pu (factorized load from roof)	0.48 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

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	70	70	24.418	22.64	22.64	22.45	22.45

Total Tension	Base Anchors					
7.282	Dist	Tension (kip)	Shear	L - Dist	Moment +	Moment -
Base Anchor 1	18 in	3.64	12.21	70 in	1.404 kip*ft	21.239 kip*ft
Base Anchor 2	70 in	3.64	12.21	18 in	21.239 kip*ft	1.404 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***	4.384	13.16%	W7	0	88.000	3.062	0.000	22.455
Wall Connection 2	2	***1.531***	13.967	50.00%	W6	88	0.000	3.062	22.455	0.000

Wall Shear Checks

Shear Connections at Base Design Force (lb)	Capacity (lb)	Reserve Capacity	Wall Shear Capacity Design (PLF)	Resistance (PLF)	check	Required Shear Capacity (lb) per Base Connector		Reserve Capacity
3179	24418	21239	352	20365	OK	1589	(21239)	OK

RIGIDITY

CALCULATED VALUES	100%	Final	2.469462921

Pier Label	Length (inches)	Height (inches)	Fixed Top? (Y/N)	Useable? (Y/N)	Stiffness (k) (1000 kip / IN)	Deflection (in / 1000 kip)
Entire Wall	88	134	Y	Y	2.469	0.405

Combine Logic					
First Segment	Second Segment	Re-Name	Combine/Subtract	Method	Combined
Entire Wall	0	Final			2.469

ID:	Navajo N-284
	DESIGN OF WALL MARKED P1-1

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.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	134
Y Corridinate	84
Direction of Wall	Y
Center of gravity X	134.000
Center of gravity Y	97.000
Wall Weight	632.000 lbs.
Central wall?	No
Wall that supports 2 roof panels?	No
lop (length of opening on wall)	0 ft
H (height of wall)	70 in
Lh (length of wall)	2.167 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)	0.116
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)	7.09 psf	Earthquake Load (EL.lat)	5.82 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	560.213

Axial Pressure on Section	
PuB	0.2 kip

Assumption check	
Pu/Ag	4.167 psi
0.06*f`c	300 psi
Check ACI 14.8.2.6	O.K.

Unfactored Axially Applied Loads

Unfactored Pressure on Roof uWr	330.0825 psf

Axial Pressure on Section	
PB	0.2 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)	0.09 klf
Hw = W*(H^4 / H^4 + L^4)	0 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)	0.06 klf
Hw = W*(H^4 / H^4 + L^4)	0 klf

Shear

Factored Loading per ACI	ACI eq. 9-3
Vu = wuB*(Bw-2db) / 2	-0.07
Phi*Vc/2	1.33
Check Shear ACI 11.5.5.1	O.K.

Deflection ACI 14.8.4

Service Loads	
Axial	0.20 kip
Lateral	0.06 klf
Allowed service deflection	0.47 in
Msa	3.163 kip-in
M	3.164 kip-in
Ds	0.006 in
Check deflection	O.K.

Allowable Capacity

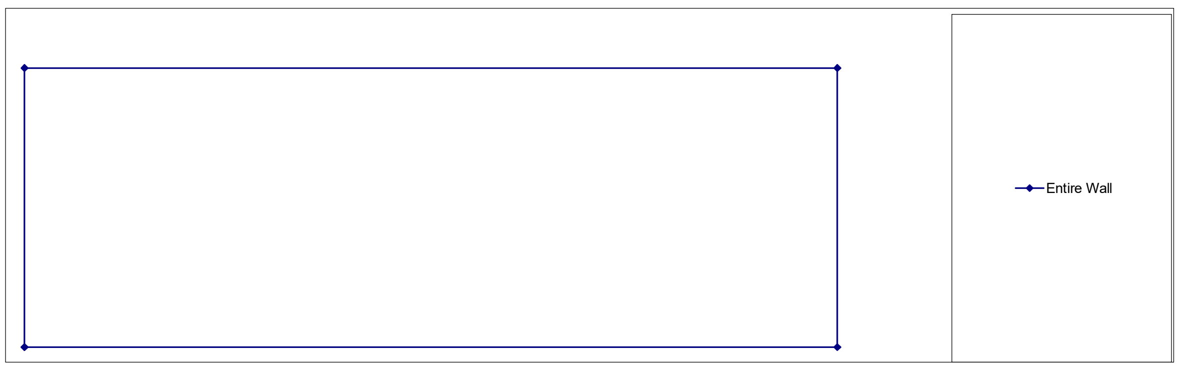
Ig = (b*h^3)/12	64 in^4
Ag = (b*h)	48 in^2
Yt = h/2	2
fr (rupture modulus)	530.330 psi
Mcr	16.971 kip-in
Beta_1	0.8
Trial Ast req'd	0.073 in^2
B	8.836162648
kd	0.542 in
I.cr	2.92 in^4
e_c	0.003
e_s	0.005
a	0.33483 psi
c	0.419 in
Ase	0.2 in^2
Icrdeflection	3.19 in^4
Ie	64.00 in^4
delta	150
r_t (maximum 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	0.008 kip-ft	

ACI eq. (14-6)		
Mu	0.010 kip-ft	0.050 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	0.50%	2.48%



REINFORCEMENT AT OPENINGS

Loading	
Pu (factorized load from roof)	0.48 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

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
1	11	15	11.223	3.31	4.52	6.63	0.00

Total Tension	Base Anchors					
3.612	Dist	Tension (kip)	Shear	L - Dist	Moment +	Moment -
Base Anchor 1	11 in	***3.61***	***11.22***	15 in	3.311 kip*ft	4.515 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***	8.329	25.00%	W2	26	0.000	3.062	6.634	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
92	11223	11131	0	20365	OK	92	(11131)	OK

RIGIDITY

CALCULATED VALUES	100%	Final	0

Pier Label	Length (inches)	Height (inches)	Fixed Top? (Y/N)	Useable? (Y/N)	Stiffness (k) (1000 kip / IN)	Deflection (in / 1000 kip)
Entire Wall	26	70	N	Y	0.232	4.307

Combine Logic					
First Segment	Second Segment	Re-Name	Combine/Subtract	Method	Combined
Entire Wall	0	Final			0.232

ID:	**Navajo N-284**
	DESIGN OF WALL MARKED P1-2

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.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	214
Y Corridinate	84
Direction of Wall	Y
Center of gravity X	214.000
Center of gravity Y	97.000
Wall Weight	632.000 lbs.
Central wall?	No
Wall that supports 2 roof panels?	No
lop (length of opening on wall)	0 ft
H (height of wall)	70 in
Lh (length of wall)	2.167 ft
Analyisis 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)	0.116
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)	7.09 psf	Earthquake Load (EL.lat)	5.82 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	560.213

Axial Pressure on Section	
PuB	0.2 kip

Assumption check	
Pu/Ag	4.167 psi
0.06*f`c	300 psi
Check ACI 14.8.2.6	O.K.

Unfactored Axially Applied Loads

Unfactored Pressure on Roof uWr	330.0825 psf

Axial Pressure on Section	
PB	0.2 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)	0.09 klf
Hw = W*(H^4 / H^4 + L^4)	0 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)	0.06 klf
Hw = W*(H^4 / H^4 + L^4)	0 klf

Shear

Factored Loading per ACI	ACI eq. 9-3
Vu = wuB*(Bw-2db) / 2	-0.07
Phi*Vc/2	1.33
Check Shear ACI 11.5.5.1	O.K.

Deflection *ACI 14.8.4*

Service Loads	
Axial	0.20 kip
Lateral	0.06 klf
Allowed service deflection	0.47 in
Msa	3.163 kip-in
M	3.164 kip-in
Ds	0.006 in
Check deflection	O.K.

Allowable Capacity

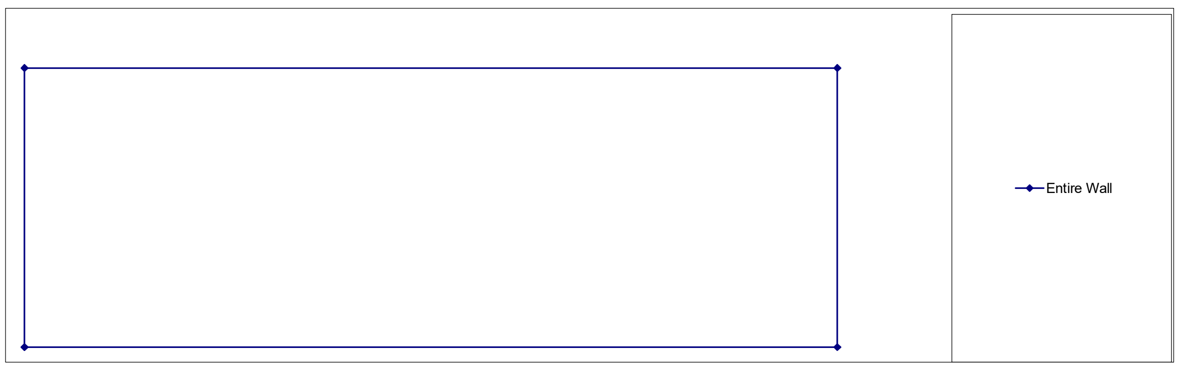
Ig = (b*h^3)/12	64 in^4
Ag = (b*h)	48 in^2
Yt = h/2	2
fr (rupture modulus)	530.330 psi
Mcr	16.971 kip-in
Beta_1	0.8
Trial Ast req'd	0.073 in^2
B	8.836162648
kd	0.542 in
I.cr	2.92 in^4
e_c	0.003
e_s	0.005
a	0.33483 psi
c	0.419 in
Ase	0.2 in^2
Icrdeflection	3.19 in^4
Ie	64.00 in^4
delta	150
r_t (maximum 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	0.008 kip-ft	

ACI eq. (14-6)		
Mu	0.010 kip-ft	0.050 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	0.50%	2.48%



REINFORCEMENT AT OPENINGS

Loading	
Pu (factorized load from roof)	0.48 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

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
1	11	15	11.223	3.31	4.52	6.63	0.00

Total Tension	Base Anchors					
3.612	Dist	Tension (kip)	Shear	L - Dist	Moment +	Moment -
Base Anchor 1	11 in	3.61	11.22	15 in	3.311 kip*ft	4.515 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	8.329	25.00%	W2	26	0.000	3.062	6.634	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
92	11223	11131	0	20365	OK	92	(11131)	OK

RIGIDITY

CALCULATED VALUES	100%	Final	0

Pier Label	Length (inches)	Height (inches)	Fixed Top? (Y/N)	Useable? (Y/N)	Stiffness (k) (1000 kip / IN)	Deflection (in / 1000 kip)
Entire Wall	26	70	N	Y	0.232	4.307

Combine Logic					
First Segment	Second Segment	Re-Name	Combine/Subtract	Method	Combined
Entire Wall	0	Final			0.232

ID:	Navajo N-284
	DESIGN OF WALL MARKED P1-3

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.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	134
Y Corridinate	166
Direction of Wall	Y
Center of gravity X	134.000
Center of gravity Y	179.000
Wall Weight	632.000 lbs.
Central wall?	No
Wall that supports 2 roof panels?	No
lop (length of opening on wall)	0 ft
H (height of wall)	70 in
Lh (length of wall)	2.167 ft
Analyisis 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)	0.116
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)	7.09 psf	Earthquake Load (EL.lat)	5.82 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	560.213

Axial Pressure on Section	
PuB	0.2 kip

Assumption check	
Pu/Ag	4.167 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	94.38 psf

Lateral Pressure on Section	
Lw = W*(L^4 / L^4 + H^4)	0.09 klf
Hw = W*(H^4 / H^4 + L^4)	0 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)	0.06 klf
Hw = W*(H^4 / H^4 + L^4)	0 klf

Unfactored Axially Applied Loads

Unfactored Pressure on Roof uWr	330.0825 psf

Axial Pressure on Section	
PB	0.2 kip

Shear

Factored Loading per ACI	ACI eq. 9-3
Vu = wuB*(Bw-2db) / 2	-0.07
Phi*Vc/2	1.33
Check Shear ACI 11.5.5.1	O.K.

Deflection ACI 14.8.4

Service Loads	
Axial	0.20 kip
Lateral	0.06 klf
Allowed service deflection	0.47 in
Msa	3.163 kip-in
M	3.164 kip-in
Ds	0.006 in
Check deflection	O.K.

Allowable Capacity

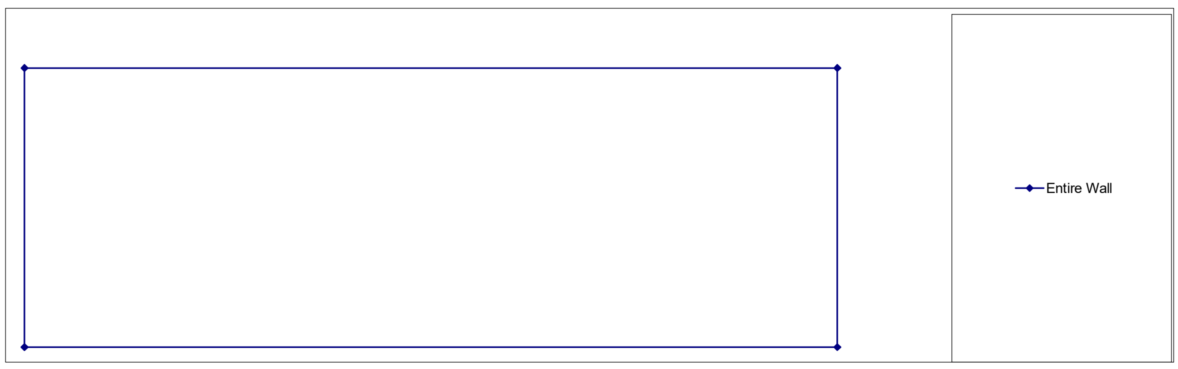
Ig = (b*h^3)/12	64 in^4
Ag = (b*h)	48 in^2
Yt = h/2	2
fr (rupture modulus)	530.330 psi
Mcr	16.971 kip-in
Beta_1	0.8
Trial Ast req'd	0.073 in^2
B	8.836162648
kd	0.542 in
I.cr	2.92 in^4
e_c	0.003
e_s	0.005
a	0.33483 psi
c	0.419 in
Ase	0.2 in^2
Icrdeflection	3.19 in^4
Ie	64.00 in^4
delta	150
r_t (maximum 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	0.008 kip-ft	

ACI eq. (14-6)		
Mu	0.010 kip-ft	0.050 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	0.50%	2.48%



REINFORCEMENT AT OPENINGS

Loading	
Pu (factorized load from roof)	0.48 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

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	Maximum	Maximum	Shear	Moment +	Moment -	Moment +	Moment -
in Shear	R - Distance	L - Distance	kip	kip - ft	kip - ft	kip - ft	kip - ft
1	15	11	11.223	4.52	3.31	0.00	6.63

Total Tension	Base Anchors					
3.612	Dist	Tension (kip)	Shear	L - Dist	Moment +	Moment -
Base Anchor 1	15 in	3.61	11.22	11 in	4.515 kip*ft	3.311 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	8.329	25.00%	W7	0	26.000	3.062	0.000	6.634

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
92	11223	11131	0	20365	OK	92	(11131)	OK

RIGIDITY

CALCULATED VALUES	100%	Final	0

Pier Label	Length (inches)	Height (inches)	Fixed Top? (Y/N)	Useable? (Y/N)	Stiffness (k) (1000 kip / IN)	Deflection (in / 1000 kip)
Entire Wall	26	70	N	Y	0.232	4.307

Combine Logic					
First Segment	Second Segment	Re-Name	Combine/Subtract	Method	Combined
Entire Wall	0	Final			0.232

ID:	Navajo N-284
	DESIGN OF WALL MARKED P1-4

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.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	214
Y Corridinate	166
Direction of Wall	Y
Center of gravity X	214.000
Center of gravity Y	179.000
Wall Weight	632.000 lbs.
Central wall?	No
Wall that supports 2 roof panels?	No
lop (length of opening on wall)	0 ft
H (height of wall)	70 in
Lh (length of wall)	2.167 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)	0.116
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)	7.09 psf	Earthquake Load (EL.lat)	5.82 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	560.213

Axial Pressure on Section	
PuB	0.2 kip

Assumption check	
Pu/Ag	4.167 psi
0.06*f`c	300 psi
Check ACI 14.8.2.6	O.K.

Unfactored Axially Applied Loads

Unfactored Pressure on Roof uWr	330.0825 psf

Axial Pressure on Section	
PB	0.2 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)	0.09 klf
Hw = W*(H^4 / H^4 + L^4)	0 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)	0.06 klf
Hw = W*(H^4 / H^4 + L^4)	0 klf

Shear

Factored Loading per ACI	ACI eq. 9-3
Vu = wuB*(Bw-2db) / 2	-0.07
Phi*Vc/2	1.33
Check Shear ACI 11.5.5.1	O.K.

Deflection ACI 14.8.4

Service Loads	
Axial	0.20 kip
Lateral	0.06 klf
Allowed service deflection	0.47 in
Msa	3.163 kip-in
M	3.164 kip-in
Ds	0.006 in
Check deflection	O.K.

Allowable Capacity

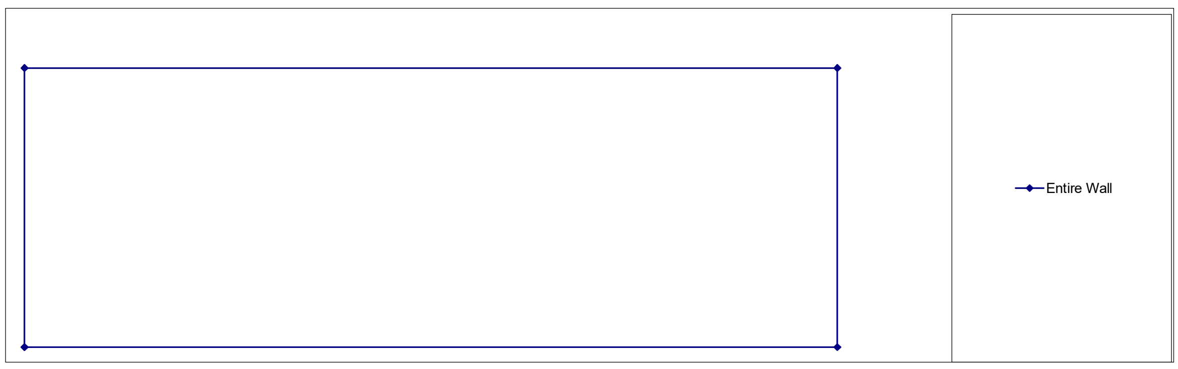
Ig = (b*h^3)/12	64 in^4
Ag = (b*h)	48 in^2
Yt = h/2	2
fr (rupture modulus)	530.330 psi
Mcr	16.971 kip-in
Beta_1	0.8
Trial Ast req'd	0.073 in^2
B	8.836162648
kd	0.542 in
I.cr	2.92 in^4
e_c	0.003
e_s	0.005
a	0.33483 psi
c	0.419 in
Ase	0.2 in^2
Icrdeflection	3.19 in^4
Ie	64.00 in^4
delta	150
r_t (maximum 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	0.008 kip-ft	

ACI eq. (14-6)		
Mu	0.010 kip-ft	0.050 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	0.50%	2.48%



REINFORCEMENT AT OPENINGS

Loading	
Pu (factorized load from roof)	0.48 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

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	Maximum	Maximum	Shear	Moment +	Moment -	Moment +	Moment -
in Shear	R - Distance	L - Distance	kip	kip - ft	kip - ft	kip - ft	kip - ft
1	15	11	11.223	4.52	3.31	0.00	6.63

Total Tension	Base Anchors					
3.612	Dist	Tension (kip)	Shear	L - Dist	Moment +	Moment -
Base Anchor 1	15 in	3.61	11.22	11 in	4.515 kip*ft	3.311 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	8.329	25.00%	W7	0	26.000	3.062	0.000	6.634

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
92	11223	11131	0	20365	OK	92	(11131)	OK

RIGIDITY

CALCULATED VALUES	100%	Final	0

Pier Label	Length (inches)	Height (inches)	Fixed Top? (Y/N)	Useable? (Y/N)	Stiffness (k) (1000 kip / IN)	Deflection (in / 1000 kip)
Entire Wall	26	70	N	Y	0.232	4.307

Combine Logic					
First Segment	Second Segment	Re-Name	Combine/Subtract	Method	Combined
Entire Wall	0	Final			0.232

ID: **Navajo N-284**

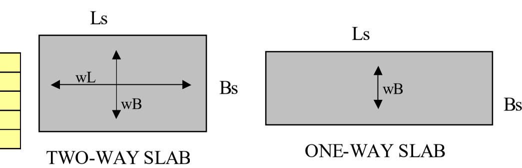
DESIGN OF FLOOR PANEL F1 & F2

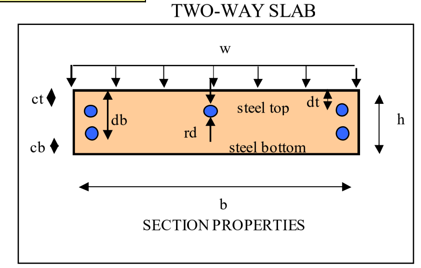
Notes:

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*

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	





Geometric Properties

Ls (overall length of slab)	26 ft		
Bs (overall width of slab)	10 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)	44.571 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)	0.116	*{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}*	

$\rho_{provided}$ (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

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	

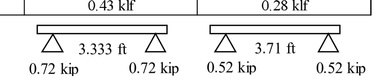
Trial As' req'd	**Wire Mesh (Top)**		
	Wire Size	W6.7	
	spacing	4 in	
0.13 in^2	Mesh Area	**0.20 in^2**	**= As'**

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

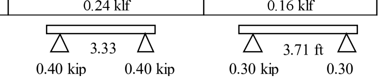
Loading

Design Loads	
Pressure on Slab	**w**
D (Dead load))	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)	7.27 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	0.43 klf	0.28 klf



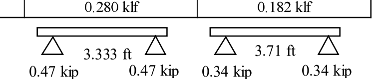
Unfactored Design Loads	Pressure on Section	Pressure on Section
Factored Pressure on Slab **W**	**wB** = W*(L^4 / B^4 + L^4)*be	**wL** = W*(B^4 / B^4 + L^4)*be
400 psf	0.24 klf	0.16 klf



B (Span in the short direction) = (Bs/NsB-1) - 0(oh1)	3.333 ft
L (Span in the long direction) = (Ls/NsL-1) - 0(oh2)	3.71 ft

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

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	0.280 klf	0.182 klf



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

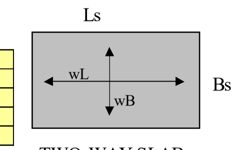
ID: **Navajo N-284**

DESIGN OF FLOOR PANEL F1 & F2

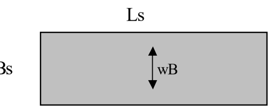
Material Properties

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*

Notes:

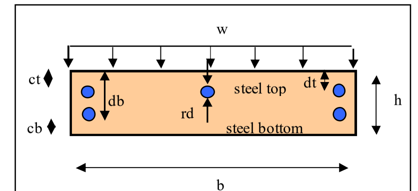


TWO-WAY SLAB



ONE-WAY SLAB

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	



SECTION PROPERTIES

Geometric Properties

Ls (overall length of slab)	26 ft		
Bs (overall width of slab)	10 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)	44.571 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)	0.116		*[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]*

$\rho_{provided}$ (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	0.38 kip-ft	0.036	Tension	0.9	3.64 kip-ft		3.64 kip-ft	O.K.	10.50%
**	Mneg (negative Moment) = (wB*B^2)*0.1	0.48 kip-ft	0.036	Tension	0.9	4.44 kip-ft		4.44 kip-ft	O.K.	10.76%

***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	0.30 kip-ft	0.036	Tension	0.9	3.64 kip-ft		3.64 kip-ft	O.K.	8.26%
**	Mneg (negative Moment) = (wB*B^2)*0.106	0.63 kip-ft	0.036	Tension	0.9	4.44 kip-ft		4.44 kip-ft	O.K.	14.13%

***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	0.86 kip	0.85	5.40 kip	4.59 kip	O.K.	18.74%
				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	0.41 kip	0.85	5.40 kip	4.59 kip	O.K.	8.91%
				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	0.38 kip-ft	0.39 kip-ft	125.00 in	125.00 in	0.001 in	0.0053	0.9500	0.001 in	0.003 in	0.2222 in	0.0833 in	O.K.	O.K.	0.65%	1.65%
L	0.30 kip-ft	0.31 kip-ft	125.00 in	125.00 in	0.001 in	0.0053	0.9500	0.001 in	0.003 in	0.2473 in	0.0928 in	O.K.	O.K.	0.59%	1.48%

ID: **Navajo N-284**

Geometric properties	
Bs (width of roof panel)	23.00 ft
Ls (Length of roof panel)	29.00 ft
Ar Area of Roof	667.00 ft^2
H (height of building)	11.55 ft
Lb (length of building)	20 ft
Wb (width of building)	26 ft
Ab (Area of building)	520 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)	504.78 ft^2
Hw (depth of floodwater)	1 ft

Loading	
Wv (weight of vault)**	0 lb
Wtr (roof panel weight)	42800 lb
Ww (total walls panel weight)	68554 lb
Fw (floor panel weight)	31626 lb
We (estimated weight of building)	142980 lb
Wev (estimated weight of building w/ vault)	142980 lb
PSFr (roof snow load)	210 psf
PSFf (Floor Live Load)	400 psf
Pmax (Maximum allowable pressure)	1500 psf
Fupmw (MWFRS Uplift Force)	43.81 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)	13924.5 lb
	.7*Vseismic (from seismic analysis without snow)	11643.3 lb
	Vwind = WLlat * max(Wb,Lb)*H	15538.4 lb

* Load adjustment per IBC 1605.3 load combinations.

Sliding Resistance with Snow	Pslide = u*(.6*We+.75*PSFr*Ar)	Pslide =	76336.2 lb

					Fsreqd	
Factor of Safety	FSwind = Pslide / Vwind	FSwind =	4.9	≥	1.0	O.K.
	FSseismic = Pslide / Vseismic	Fseismic =	5.5	≥	1.0	O.K.

Sliding Resistance with No Snow	Pslide = u*.6*We	Pslide =	34315.2 lb

					Fsreqd	
Factor of Safety	FSwind = Pslide / Vwind	Fswind =	2.2	≥	1.0	O.K.
	FSseismic = Pslide / Vseismic	Fseismic =	2.9	≥	1.0	O.K.

CHECK OVERTURNING RESISTANCE

Shear	.7*Otseismic (from seismic analysis with snow)	137.255 kip-ft
	.7*Otseismic (from seismic analysis without snow)	114.244 kip-ft
	Otwind = (WLlat*Lb*H^2 / 2) + (Fupmw*Lb*Wb^2 / 2)	365.153 kip-ft

* Load adjustment per IBC 1605.3 load combinations.

Overturning Resistance with Snow	Otrsnow = (.6*We+.75*PSFr*Ar)*(Wb/2)	Otrsnow =	1138.893 kip-ft

					Fsreqd	
Factor of Safety	FSwind = Otrsnow / Otwind	FSwind =	3.12	≥	1.0	O.K.
	FSseismic = Otrsnow / Vseismic	Fseismic =	8.30	≥	1.0	O.K.

Overturning Resistance with No Snow	Otr = .6*We*Wb/2	Otr	1115.244 kip-ft

					Fsreqd	
Factor of Safety	FSwind = Otr / Vwind	Fswind =	3.05	≥	1.0	O.K.
	FSseismic = Otr / Vseismic	Fseismic =	9.76	≥	1.0	O.K.

CHECK BEARING PRESSURE CONDITION

Net Pressure	Pnet = (Wev + PSFr*Ar + PSFf*Af) / Ab	944.33 psf

Allowable	Pmax ≥ Pnet	1500 psf ≥ 944.33 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 =**	31498.13 lb

Factor of Safety	FSb = We / Fb	**FSb =**	4.54	≥	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.