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

Calculations
Denali DN-389 and DN-390
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

September 30, 2022

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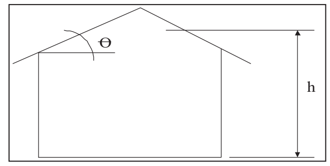
Appendix: (Provided Upon Request) UL-752 Bullet Resistance Testing

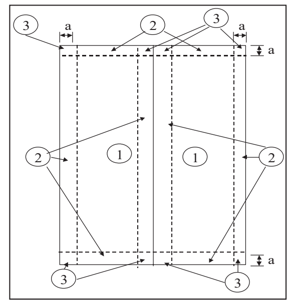
All attached documents are for reference only and designed or approved by others.

September 30, 2022

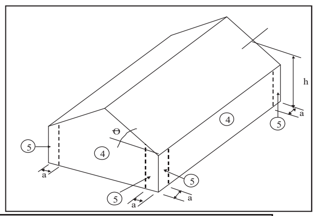
Main Wind Force Resisting System Loads (ASCE 7-16)

Denali DN-389 and DN-390		
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	7.25 ft	Windward wall height
h.lee	7.25 ft	Leeward wall height
W.building	17 ft	Width of the building
L.building	9.5 ft	Length of the building
H.building	12.58 ft	Height of the building (to the ridge). Enter 0 if unknown.
Roof Rise	7.1875	Roof pitch (per foot)
ϑ	30.92 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	9.915 ft	Mean roof height
n_a	7.56	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	68.9 sq ft	567.5 sq ft	0 sq ft	0.00	0.00	0.00	0.00	Enclosed
Windward endwall	0 sq ft	168.6 sq ft	467.8 sq ft	0 sq ft	0.00	0.00	0.00	0.00	Enclosed
Leeward sidewall	0 sq ft	68.9 sq ft	567.5 sq ft	0 sq ft	0.00	0.00	0.00	0.00	Enclosed
Leeward endwall	0 sq ft	168.6 sq ft	467.8 sq ft	0 sq ft	0.00	0.00	0.00	0.00	Enclosed
Roof	0 sq ft	161.5 sq ft	474.9 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		
	-0.342	Leeward wall (wind normal to ridge) (Use with qh)	L/B =	1.79
	-0.500	Leeward wall (wind parallel to ridge) (Use with qh)	L/B =	0.56
	-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	0.215	-0.214	-0.600

Roof Pressures Wind Perpendicular to Ridge w/ ϑ >= 10 deg	
w/ Negative Internal	15.09 psf
w/ Positive Internal	-28.68 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.	-0.97	-0.87	-0.53	-0.37
Roof Pressure Coefficients (Fig 27.3-1) PARALLEL to Ridge	-1.30	-0.70	-0.70	-0.70

Wall Pressures:	w/ Negative	w/ Positive Internal
Windward	35.74 psf	20.78 psf
Leeward (wind normal)	-16.00 psf	-19.57 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	-53.41 psf
h/2 to h	-32.21 psf
h to 2h	-32.21 psf
Over 2h	-32.21 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:	**7.19 : 12**	**COMPONENTS & CLADDING**	
Exposure:	**C**	Mean Roof Height:	**9.92 ft**		
	Effective Area				
Zone	10.0 sq ft		100.0 sq ft		500.0 sq ft	
1	-42.36 psf	36.61 psf	-34.05 psf	32.45 psf	-34.05 psf	32.45 psf
2	-50.67 psf	36.61 psf	-42.36 psf	32.45 psf	-42.36 psf	32.45 psf
2oh	-83.12 psf	-	-74.81 psf	-	-74.81 psf	-
3	-50.67 psf	36.61 psf	-42.36 psf	32.45 psf	-42.36 psf	32.45 psf
3oh	-83.12 psf	-	-74.81 psf	-	-74.81 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

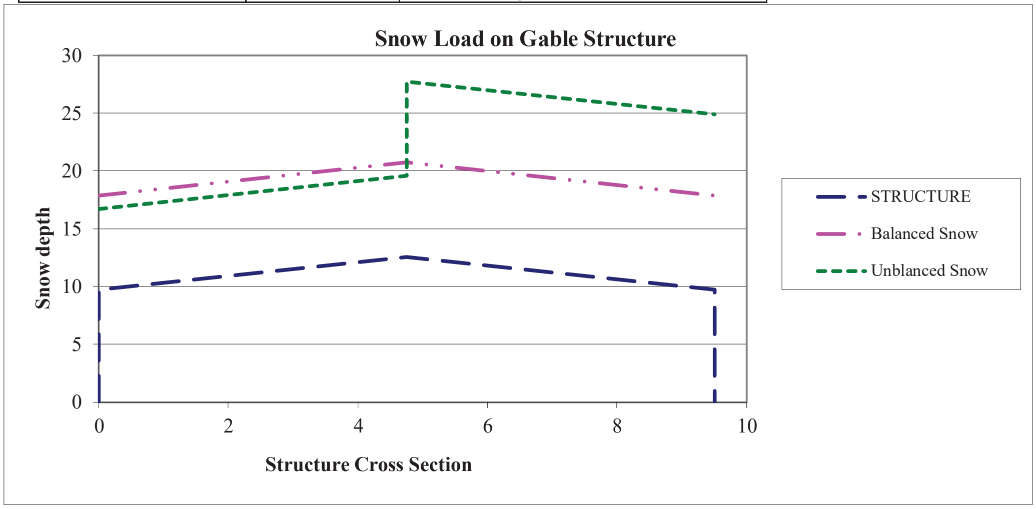
ASCE 7-16 SNOW LOAD CALCULATION

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	17 ft	Length of the building
L.building	9.5 ft	Width of the building
H.building	12.58 ft	Height of the building (to the ridge). Enter 0 if unknown.
Roof Rise (per foot)	7.1875	Roof pitch
ϑ	30.92 deg	Roof Angle

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

C_t	1.2	(Choose from table above)
Is	1	ASCE Table 1.5-2
Surface	Unobstructed	ASCE § 7.4
Roof type	Gable	
Hor. Eave to Ridge Distance - windward	8.5 ft	
Roof Exposure	Partially exposed	ASCE Table 7.3-1
C_e	1	ASCE Table 7.3-1
Cs	0.71054815	Slope Factor from Figure 7.4-1
Low Sloped?:	No	ASCE § 7.3.4
P_f	210.00 psf	Flat Roof Snow Load
P_s	149.22 psf	Sloped Roof Snow Load
Use unbalanced?	No	ASCE § 7.6.1
$P_{windward}$	210.00 psf	ASCE § 7.6.1
$P_{leeward\ 1}$	210.00 psf	ASCE § 7.6.1
$P_{leeward\ 2}$	210.00 psf	ASCE § 7.6.1
Distance from Ridge to Edge of $P_{leeward1}$ loading	0.0 ft	ASCE Figure 7.6-2

γ	30.00 pcf	Snow density	Eq. 7.7-1 of ASCE 7
S	1.669565217	Run per rise of 1	ASCE § 7.1
h_d	8.16 ft	Height of drifting snow on leeward side	
h_b	4.97 ft	Height of balanced snow	



Seismic Loads (ASCE 7-16)

Denali DN-389 and DN-390				
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	10.12 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*
$S_{m1} = F_v * S_1$	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*	
$C_{s.max}$	0.455	*ASCE 12.8-3 & 12.8-4*	
C_s	0.116		
k	1.000	*ASCE 12.8.3*	
W	60.13 kip		
$V = C_s * W$	17.49 kip	*ASCE 12.8-1*	*Shear **with** snow load*
M_o =	174.7 k-ft		*Overturning Moment **with** snow load*
$V = C_s * W$	15.11 kip		*Shear **without** snow load*
M_o =	150.3 k-ft		*Overturning Moment **without** snow load*

WITH SNOW LOAD

Level	Story Height	h_i or h_x	P_f (flat roof snow load)	w_i	$w_i*h_i^k$	C_{vx} *12.8-12*	F_x *12.8-11;11.7*	V_x (Story shear)	M_x	F_{px} (diaphragm force) *12.10-1*
Roof	9.92 ft	10.12 ft	210 psf	36.13 kip	365.7 k-ft	0.987	17.25 kip	17.25 kip	0.0 k-ft	6.72 kip
Walls	0.00 ft	0.00 ft								
Floor	0.21 ft	0.21 ft		24.00 kip	5.0 k-ft	0.013	0.24 kip	17.49 kip	171.0 k-ft	4.47 kip
Base	0 ft	0.00 ft	W=	60.13 kip	370.7 k-ft			M_o =	174.7 k-ft	

WITHOUT SNOW LOAD

Level	Story Height	h_i or h_x	P_f (flat roof snow load)	w_i	$w_i*h_i^k$	C_{vx} *12.8-12*	F_x *12.8-11;11.7*	V_x (Story shear)	M_x	F_{px} (diaphragm force) *12.10-1*
Roof	9.92 ft	10.12 ft	0 psf	27.95 kip	282.9 k-ft	0.983	14.85 kip	14.85 kip	0.0 k-ft	5.20 kip
Walls	0.00 ft	0.00 ft								
Floor	0.21 ft	0.21 ft		24.00 kip	5.0 k-ft	0.017	0.26 kip	15.11 kip	147.2 k-ft	4.47 kip
Base	0 ft	0.00 ft	W=	51.95 kip	287.9 k-ft			Mo =	150.3 k-ft	

Center of Mass & Rigidity

Denali DN-389 and DN-390

				X	Y	
	Upper Left = 0,0		Lower Right	228	123	
Wall	X Relative Stiffness	Y Relative Stiffness	Shear Force lbs	Shear Force plf	Dist to CoRx dx (IN)	Dist to CoRy dy (IN)
W1	37.01%	0.00%	1,555	164	100.000	10.770
W2	37.01%	0.00%	1,555	164	100.000	10.770
W3	12.99%	0.00%	546	74	18.000	2.270
W4	12.99%	0.00%	546	74	18.000	2.270
W5	0.00%	52.45%	2,204	135	0.007	43.980
W6	0.00%	47.55%	1,998	122	0.011	48.520

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	7140	0	0	114	123	210	57.0	61.5	11230	7140
R2	4.5	7140	114	0	228	123	210	171.0	61.5	11230	7140
F1	5	10335	11	4	217	123	400	114.0	63.5	10335	0
Totals		27333						114.0	58.0		

Torsional Eccentricity ex	Torsional Eccentricity ey	Wgt (w snow)	Wgt (w/o snow)		wgt (w snow)	wgt (w/o snow)
0.00	7.06	60,128	51,948	roof	36,126	27,947
Center of Gravity				floor	24,002	
X	Y					
114.0	58.0					
Center of Rigidty						
X	Y					
114.0	51.0					

Wall	Wall Overturning Checks Using Weight of Adjacent Walls / Force Transferred by Connections Between Walls: Anchorage Required to Resist Overturning From Design Moment (kip-ft)	Toward Lower Right Anchor Resistance: Moment (kip-ft)	Toward Lower Right Anchor Resistance: check	Toward Upper Left Anchor Resistance: Moment (kip-ft)	Toward Upper Left Anchor Resistance: check	Overturning status using just connection to adjacent walls
W1	5.95	32.30	OK	44.95	OK	None Required
W2	5.95	32.30	OK	44.95	OK	None Required
W3	0.00	33.87	OK	33.87	OK	None Required
W4	0.00	33.87	OK	33.87	OK	None Required
W5	-19.07	163.32	OK	163.32	OK	None Required
W6	-18.94	163.32	OK	163.32	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	Wall Overturning Checks Using Base Anchors Only / Must investigate ONLY if connection to adjacent walls is insufficient: Design Moment (kip-ft)	Toward Lower Right Anchor Resistance: Moment (kip-ft)	Toward Lower Right Anchor Resistance: check	Toward Upper Left Anchor Resistance: Moment (kip-ft)	Toward Upper Left Anchor Resistance: check	Combined Loading Unity Check	Required Tension Capacity per Base Anchor (lb)
W1	5.95	40.03	OK	40.81	OK	OK	(1893)
W2	5.95	40.03	OK	40.81	OK	OK	(1893)
W3	0.00	23.80	OK	23.80	OK	OK	(4593)
W4	0.00	23.80	OK	23.80	OK	OK	(4593)
W5	-19.07	87.64	OK	87.64	OK	OK	(5583)
W6	-18.94	87.64	OK	87.64	OK	OK	(5579)

Wall	Wall Overturning Checks Using Base Anchors and Connection to Adjacent Walls / Must investigate ONLY if both base anchor alone and adjacent walls alone are insufficient: Base Anchor Shear Required (% 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%	72.32	85.76	OK	OK
W2	0.0%	100.0%	72.32	85.76	OK	OK
W3	0.0%	100.0%	57.67	57.67	OK	OK
W4	0.0%	100.0%	57.67	57.67	OK	OK
W5	0.0%	100.0%	250.96	250.96	OK	OK
W6	0.0%	100.0%	250.96	250.96	OK	OK

ID: **Denali DN-389 and DN-390**

DESIGN OF ROOF PANELS MARK R1 & R2

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)	11.33 ft		
Bs (overall width of slab)	10.25 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 in		
cb (cover bottom)	1 1/4 in		
rd (assumed reinf. diameter)	0.319 in	*(if centered enter 0)*	
dt (effective depth top)	1.1595 in		
db (effective depth bottom)	2.931 in		
oh1 (overhang length and qty for Bs)	11 in	1	*{qty of overhangs in Bs direction}*
oh2 (overhang length and qty for ls)	12.25 in	2	*{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}*	

Reinforcement Limits

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

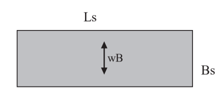
Loading

Design Loads	
Pressure on Slab	**w**
D (Dead load))	60.938 psf
S (Snow Load)	210 psf
L (Live Load)	0 psf
Lr (Live Roof Load)	30 psf
W (Wind Load)	50.67 psf
E (Earthquake Load)	7.09 psf

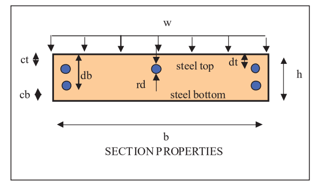
Sustained Loading	
Pressure on slab	**W**
D (Dead load)	60.938 psf
S (Snow Load)	210 psf
Lr (Live Roof Load)	30 psf

Notes:

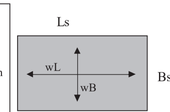
fr (rupture modulus)	530.3 psi	*ACI 9.5.2.3*
Ig = (b*h^3)/12	91.125 in^4	
Ag = (b*h)	54 in^4	
Yt = h/2	2.25 in^4	
Mcr	21.48	*ACI 9.5.2.3*
Beta_1	0.8	*ACI 10.2.7.3*
delta_initial	180	
delta_longterm	480	
B	8.830	
kd	0.709 in	
I.cr	8.14 in^4	
c	0 in	
a	0.32 in	



ONE-WAY SLAB



SECTION PROPERTIES



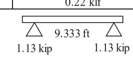
TWO-WAY SLAB

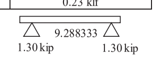
bottom mesh		
$\rho_{provided}$ (reinforcement ratio provided)		0.0057
top mesh	ω =	0.0912 psi
$\rho_{provided}$ (reinforcement ratio provided)		0.0144
both layers	ω =	0.2304 psi
$\rho_{provided}$ (reinforcement ratio provided)		0.0164
	ω =	0.2624 psi

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

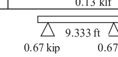
	Wire Mesh (Bottom)		
	Wire Size	W6.7	
Trial Ast required	spacing	4 in	
0.116 in^2	Mesh Area	**0.20 in^2**	= As

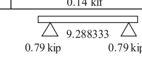
Factored Design Loads		Pressure on Section	Pressure on Section
Factored Loading per ACI equation indicated	Factored Pressure on Slab **W**	**wB** = W*(L^4 / B^4 + L^4)*be	**wL** = W*(B^4 / B^4 + L^4)*be
ACI eq. 9-3	449.661 psf	0.22 klf	0.23 klf



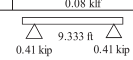


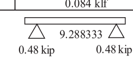
Unfactored Design Loads	Pressure on Section	Pressure on Section
Unfactored Pressure on Slab **W**	**wB** = W*(L^4 / B^4 + L^4)*be	**wL** = W*(B^4 / B^4 + L^4)*be
270.9375 psf	0.13 klf	0.14 klf





B (Span in the short direction) = Bs-1(oh1)		9.333 ft	
L (Span in the long direction) = Ls-2(oh2)		9.288333333 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	165.938 psf	0.08 klf	0.084 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

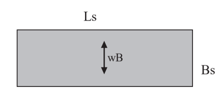
ID: **Denali DN-389 and DN-390**

DESIGN OF ROOF PANELS MARK R1 & R2

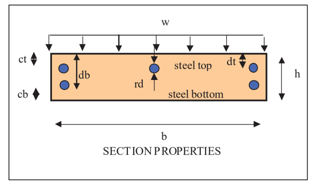
Notes:

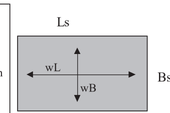
Material Properties

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



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	0.709 in	
I.cr	8.14 in^4	
c	0 in	
a	0.32 in	





Geometric Properties

Ls (overall length of slab)	11.33 ft		
Bs (overall width of slab)	10.25 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 in		
cb (cover bottom)	1 1/4 in		
rd (assumed reinf. diameter)	0.319 in	*(if centered enter 0)*	
dt (effective depth top)	1.1595 in		
db (effective depth bottom)	2.931 in		
oh1 (overhang length and qty for Bs)	11 in	1	*{qty of overhangs in Bs direction}*
oh2 (overhang length and qty for ls)	12.25 in	2	*{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)		0.0057
top mesh	ω =	0.0912 psi
$\rho_{provided}$ (reinforcement ratio provided)		0.0144
both layers	ω =	0.2304 psi
$\rho_{provided}$ (reinforcement ratio provided)		0.0164
	ω =	0.2624 psi

Flexure

Flexural Moments for Bs	Mu	Tensile Strain	Check ACI 14.8.2.3	ϕb	ϕMn trial = ϕf'cbd^2ω(1-0.59ω)	ΔM = Mu - ϕM	ϕMn =	Check ϕMn > Mu	% allowed
Mpos (positive Moment) = (wB*B^2) / 8	2.4 kip-ft	0.036	Tension	0.9	3.34 kip-ft		3.34 kip-ft	O.K.	71.86%

	Mu	Structural Plain Concrete per ACI 22.5: S Elastic Section Modulus	ϕb	ϕMn = ϕ5*(f 'c*S)^0.5	Check ϕMn > Mu	% allowed
Mneg (negative Moment) = (wB*oh1^2) / 2	0.092 kip-ft	0.023 ft^3	0.55	0.941 kip-ft	O.K.	9.82%
Mneg (negative Moment) = (wB*oh2^2) / 2	0.092 kip-ft	0.023 ft^3	0.55	0.941 kip-ft	O.K.	9.82%

Flexural Moments for ls	Mu	Tensile Strain	Check ACI 14.8.2.3	ϕb	ϕMn trial = ϕf'cbd^2ω(1-0.59ω)	ΔM = Mu - ϕM	ϕMn =	Check ϕMn > Mu	% allowed
Mpos (positive Moment) = (wL*L^2) / 8	2.480 kip-ft	0.036	Tension	0.9	3.34 kip-ft		3.34 kip-ft	O.K.	74.26%

	Mu	Structural Plain Concrete per ACI 22.5: S Elastic Section Modulus	ϕb	ϕMn = ϕ5*(f 'c*S)^0.5	Check ϕMn > Mu	% allowed
Mneg (negative Moment) = (wL*oh1^2) / 2	0.1198416	0.023 ft^3	0.55	0.941 kip-ft	O.K.	12.74%
Mneg (negative Moment) = (wL*oh2^2) / 2	0.1198416	0.023 ft^3	0.55	0.941 kip-ft	O.K.	12.74%

Shear

Maximum Shear for Bs	Vu	ϕv per ACI 9.3.2.3	Vc per ACI 11.3.1.1	ϕVc	Check ϕVc > Vu	% allowed
Vu = wB (B/2)	1.03 kip	0.85	4.97 kip	4.23 kip	O.K.	24.28%
			per ACI 22.8			
Vu for side overhang 1 = wB*oh1	0.20 kip	0.55	2.98 kip	1.64 kip	O.K.	12.30%
Vu for side overhang 1 = wB*oh1	0.20 kip	0.55	2.98 kip	1.64 kip	O.K.	12.30%

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)	0.39 kip	0.85	4.97 kip	4.23 kip	O.K.	9.23%
			per ACI 22.8			
Vu for end overhang 2 = wL*oh2	0.09 kip	0.55	2.98 kip	1.64 kip	O.K.	5.23%
Vu for end overhang 2 = wL*oh2	0.09 kip	0.55	2.98 kip	1.64 kip	O.K.	5.23%

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 ΔI-t	Δ total long-term deflection (Δi + ΔI-t)	Δ allow (immediate)	Δ allow (long term)	Check short term deflection	Check total long term deflection	% allowed - short term	% allowed - total long term
B	2.4 kip-ft	0.871 kip-ft	42.56 in^4	91.13 in^4	0.035 in	0.0057	0.9339	0.033 in	0.068 in	0.6222 in	0.2333 in	O.K.	O.K.	5.62%	28.98%
L	2.48 kip-ft	0.906 kip-ft	39.33 in^4	91.13 in^4	0.000 in	0.0057	0.9339	0.033 in	0.068 in	0.6192 in	0.2322 in	O.K.	O.K.	0.00%	29.12%

ID:	Denali DN-389 and DN-390
	DESIGN OF WALL MARKED W1

Notes	

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

Material Properties

f'c	5000 psi	
Steel Reinforcement	Plain WWF 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	14
Y Corridinate	4
Direction of Wall	Y
Center of gravity X	14.000
Center of gravity Y	61.750
Wall Weight	3608.000 lbs.
Central wall?	Yes
Wall that supports 2 roof panels?	No
lop (length of opening on wall)	0 ft
H (height of wall)	87 in
Lh (length of wall)	9.500 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)	210 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)	50.67 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	449.661

Axial Pressure on Section	
PuB	1.58 kip

Assumption check	
Pu/Ag	32.917 psi
0.06*f'c	300 psi
Check ACI 14.8.2.6	O.K.

Unfactored Axially Applied Loads

Unfactored Pressure on Roof uWr	270.9375 psf

Axial Pressure on Section	
PB	1.06 kip

Factored Laterally Applied Loads

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

Lateral Pressure on Section	
Lw= W*(L^4 / L^4 + H^4)	0.02 klf
Hw= W*(H^4 / H^4 + L^4)	0.07 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)	0.01 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	0.06
Phi*Vc/2	1.33
Check Shear ACI 11.5.5.1	O.K.

Deflection ACI 14.8.4

Service Loads	
Axial	1.06 kip
Lateral	0.01 klf
Allowed service deflection	0.58 in
Msa	1.318 kip-in
M	1.322 kip-in
Ds	0.004 in
Check deflection	O.K.

Allowable Capacity

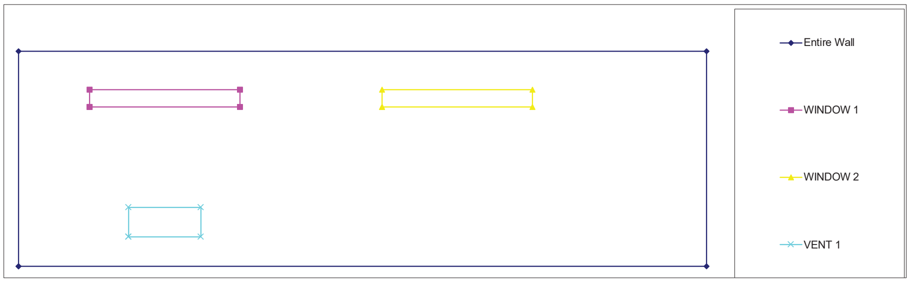
Ig = (b*h^3)/12	64 in^4
Ag = (b*h)	48 in^2
Yt = h/2	2
fr (rupture modulus)	530.330 psi
Mcr	16.971 kip-in
Beta_1	0.8
Trial Ast req'd	0.073 in^2
B	8.836162648
kd	0.542 in
I.cr	2.92 in^4
e_c	0.003
e_s	0.005
a	0.33483 psi
c	0.419 in
Ase	0.22 in^2
Icrdeflection	3.47 in^4
Ie	64.00 in^4
delta	150
r_t (maximum tensile reinforcement)	0.0166
r_{temp} (min. temperature reinforcement)	0.0014
r_{min} (minimum tensile reinforcement)	0.0027
r_{trial} (trial reinforcement ratio bottom)	0.0033
$\rho_{provided}$ (reinforcement ratio provided)	0.0090

Flexure

Assumption check		
Span	Hw	Lw
net Tensile Strain	0.010	0.010
Check ACI 14.8.2.3	Tension	Tension
Mua	0.526 kip-ft	

ACI eq. (14-6)		
Mu	0.590 kip-ft	0.230 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	29.27%	11.41%



REINFORCEMENT AT OPENINGS

Loading	
Pu (factorized load from roof)	0.23 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	0.98 ft	5.38 ft	2.08 ft	1.29 ft	60.32	0.06 klf	0.29 klf	0.1 kip-ft
WINDOW 2	5.02 ft	5.38 ft	2.08 ft	1.29 ft	60.32	0.06 klf	0.29 klf	0.1 kip-ft
VENT 1	1.52 ft	1 ft	1 ft	5.25 ft	50.00	0.26 klf	0.49 klf	0.04 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	0.002 in^2	No. 3	1	6.86 kip-ft	O.K.
WINDOW 2	0.9	0.002 in^2	No. 3	1	6.86 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	93	98	36.627	40.03	40.81	32.30	44.95

Total Tension	Base Anchors					
10.923	Dist	Tension (kip)	Shear	L - Dist	Moment +	Moment -
Base Anchor 1	16 in	3.64	12.21	98 in	0.835 kip*ft	29.735 kip*ft
Base Anchor 2	58 in	3.64	12.21	56 in	10.975 kip*ft	9.709 kip*ft
Base Anchor 3	93 in	3.64	12.21	21 in	28.218 kip*ft	1.365 kip*ft

	Wall Connections									
	Quantity of Anchors	Capacity of each Anchor	Countering Dead Load from Adjoining Wall	% of wall to use	Adjoining Wall	Dist (inches)	L - Dist	Allowable Force	Overturning Moment Resistance (kip-ft) Up Left	Overturning Moment Resistance (kip-ft) Low Right
Wall Connection 1	3	***2.703***	4.208	20.41%	W5	4	110.000	4.208	1.403	38.575
Wall Connection 2	3	***2.703***	3.923	20.41%	W6	94.5	19.500	3.923	30.894	6.375

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
4413	36627	32214	409	19276	OK	1471	(32214)	OK

RIGIDITY

CALCULATED VALUES	95%	Final	6.924257696

	Pier Label	Length (inches)	Height (inches)	Fixed Top? (Y/N)	Useable? (Y/N)	Stiffness (k) (1000 kip / IN)	Deflection (in / 1000 kip)
	Entire Wall	114	87	Y	Y	7.315	0.137
WINDOW 1	A'	114	6.96	Y	Y	109.060	0.009
	A	11.76	6.96	Y	Y	10.087	0.099
	B	77.28	6.96	Y	Y	73.823	0.014
WINDOW 2	B'	114	6.96	Y	Y	109.060	0.009
	C	60.24	6.96	Y	Y	57.446	0.017
	D	28.8	6.96	Y	Y	27.059	0.037
VENT 1	C'	114	12	Y	Y	63.100	0.016
	E	18.24	12	Y	Y	8.856	0.113
	F	83.76	12	Y	Y	46.217	0.022

	Combine Logic					
	First Segment	Second Segment	Re-Name	Combine/Subtract	Method	Combined
WINDOW 1	Entire Wall	A'	A'a	-	Deflection	0.128
	A	B	AB	+	Stiffness	83.910
	A'a	AB	A'b	+	Deflection	0.139
WINDOW 2	A'b	B'	B'a	-	Deflection	0.130
	C	D	CD	+	Stiffness	84.505
	B'a	CD	B'b	+	Deflection	0.142
VENT 1	B'b	C'	C'a	-	Deflection	0.126
	E	F	EF	+	Stiffness	55.073
	C'a	EF	Final	+	Deflection	0.144

ID:	Denali DN-389 and DN-390
	DESIGN OF WALL MARKED W2

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	214
Y Corridinate	4
Direction of Wall	Y
Center of gravity X	214.000
Center of gravity Y	61.750
Wall Weight	3608.000 lbs.
Central wall?	Yes
Wall that supports 2 roof panels?	No
lop (length of opening on wall)	0 ft
H (height of wall)	87 in
Lh (length of wall)	9.500 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)	210 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)	50.67 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	449.661

Axial Pressure on Section	
PuB	1.58 kip

Assumption check	
Pu/Ag	32.917 psi
0.06*f'c	300 psi
Check ACI 14.8.2.6	O.K.

Unfactored Axially Applied Loads	
Unfactored Pressure on Roof uWr	270.9375 psf

Axial Pressure on Section	
PB	1.06 kip

Factored Laterally Applied Loads	
Factored Loading per ACI	ACI eq. 9-4
Factored Pressure on Wall Ww	94.38 psf

Lateral Pressure on Section	
Lw = W*(L^4 / L^4 + H^4)	0.02 klf
Hw = W*(H^4 / H^4 + L^4)	0.07 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)	0.01 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	0.06
Phi*Vc/2	1.33
Check Shear ACI 11.5.5.1	O.K.

Deflection		ACI 14.8.4
Service Loads		
Axial	1.06 kip	
Lateral	0.01 klf	
Allowed service deflection	0.58 in	
Msa	1.318 kip-in	
M	1.322 kip-in	
Ds	0.004 in	
Check deflection	O.K.	

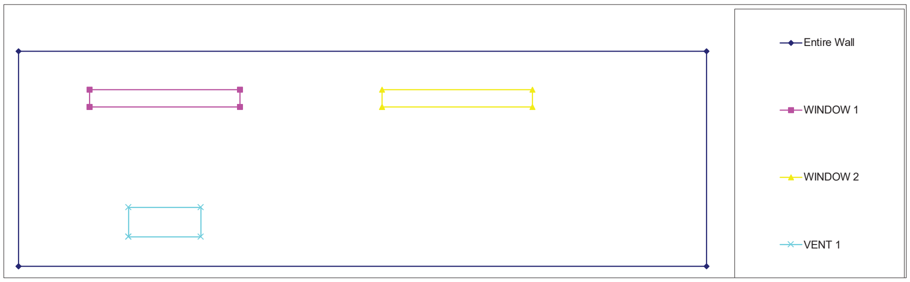
Allowable Capacity	
Ig = (b*h^3)/12	64 in^4
Ag = (b*h)	48 in^2
Yt = h/2	2
fr (rupture modulus)	530.330 psi
Mcr	16.971 kip-in
Beta_1	0.8
Trial Ast req'd	0.073 in^2
B	8.836162648
kd	0.542 in
I.cr	2.92 in^4
e_c	0.003
e_s	0.005
a	0.33483 psi
c	0.419 in
Ase	0.22 in^2
Icrdeflection	3.47 in^4
Ie	64.00 in^4
delta	150
r_t (maximum tensile reinforcement)	0.0166
r_{temp} (min. temperature reinforcement)	0.0014
r_{min} (minimum tensile reinforcement)	0.0027
r_{trial} (trial reinforcement ratio bottom)	0.0033
$\rho_{provided}$ (reinforcement ratio provided)	0.0090

Flexure

Assumption check		
Span	Hw	Lw
net Tensile Strain	0.010	0.010
Check ACI 14.8.2.3	Tension	Tension
Mua	0.526 kip-ft	

ACI eq. (14-6)		
Mu	0.590 kip-ft	0.230 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	29.27%	11.41%



REINFORCEMENT AT OPENINGS

Loading	
Pu (factorized load from roof)	0.23 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	0.98 ft	5.38 ft	2.08 ft	1.29 ft	60.32	0.06 klf	0.29 klf	0.1 kip-ft
WINDOW 2	5.02 ft	5.38 ft	2.08 ft	1.29 ft	60.32	0.06 klf	0.29 klf	0.1 kip-ft
VENT 1	1.52 ft	1 ft	1 ft	5.25 ft	50.00	0.26 klf	0.49 klf	0.04 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	0.002 in^2	No. 3	1	6.86 kip-ft	O.K.
WINDOW 2	0.9	0.002 in^2	No. 3	1	6.86 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				
			Lateral	Overturning			
				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	93	98	36.627	40.03	40.81	32.30	44.95

Total Tension	Base Anchors					
10.923	Dist	Tension (kip)	Shear	L - Dist	Moment +	Moment -
Base Anchor 1	16 in	3.64	12.21	98 in	0.835 kip*ft	29.735 kip*ft
Base Anchor 2	58 in	3.64	12.21	56 in	10.975 kip*ft	9.709 kip*ft
Base Anchor 3	93 in	3.64	12.21	21 in	28.218 kip*ft	1.365 kip*ft

	Wall Connections									
	Quantity of Anchors	Capacity of each Anchor	Countering Dead Load from Adjoining Wall	% of wall to use	Adjoining Wall	Dist (inches)	L - Dist	Allowable Force	Overturning Moment Resistance (kip-ft) Up Left	Low Right
Wall Connection 1	3	2.703	4.208	20.41%	W5	4	110.000	4.208	1.403	38.575
Wall Connection 2	3	2.703	3.923	20.41%	W6	94.5	19.500	3.923	30.894	6.375

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
4413	36627	32214	409	19276	OK	1471	(32214)	OK

RIGIDITY

CALCULATED VALUES	95%	Final	6.924257696

	Pier Label	Length (inches)	Height (inches)	Fixed Top? (Y/N)	Useable? (Y/N)	Stiffness (k) (1000 kip / IN)	Deflection (in / 1000 kip)
	Entire Wall	114	87	Y	Y	7.315	0.137
WINDOW 1	A'	114	6.96	Y	Y	109.060	0.009
	A	11.76	6.96	Y	Y	10.087	0.099
	B	77.28	6.96	Y	Y	73.823	0.014
WINDOW 2	B'	114	6.96	Y	Y	109.060	0.009
	C	60.24	6.96	Y	Y	57.446	0.017
	D	28.8	6.96	Y	Y	27.059	0.037
VENT 1	C'	114	12	Y	Y	63.100	0.016
	E	18.24	12	Y	Y	8.856	0.113
	F	83.76	12	Y	Y	46.217	0.022

	Combine Logic					
	First Segment	Second Segment	Re-Name	Combine/Subtract	Method	Combined
WINDOW 1	Entire Wall	A'	A'a	-	Deflection	0.128
	A	B	AB	+	Stiffness	83.910
	A'a	AB	A'b	+	Deflection	0.139
WINDOW 2	A'b	B'	B'a	-	Deflection	0.130
	C	D	CD	+	Stiffness	84.505
	B'a	CD	B'b	+	Deflection	0.142
VENT 1	B'b	C'	C'a	-	Deflection	0.126
	E	F	EF	+	Stiffness	55.073
	C'a	EF	Final	+	Deflection	0.144

ID:	Denali DN-389 and DN-390
	DESIGN OF WALL MARKED W3

Notes	

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

Material Properties

f'c	5000 psi	
Steel Reinforcement	Plain WWF Grade 80	
Fy wire mesh	80000 psi	
Fy rebar	60000 pcf	
Lightweight?	No	
Concrete density	150 pcf	O.K.
E (Steel)	29000000 psi	
E (Concrete)	4290000 psi	ACI 8.5.1
n (modular ratio)	6.76	

Shear Parameters

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

Minimum Wall Reinforcement Requirements

roe.min.vert	0.0012	ACI 14.3.2
roe.min.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	96
Y Corridinate	9
Direction of Wall	Y
Center of gravity X	96.000
Center of gravity Y	53.250
Wall Weight	4185.000 lbs.
Central wall?	Yes
Wall that supports 2 roof panels?	No
lop (length of opening on wall)	0 ft
H (height of wall)	135.88 in
Lh (length of wall)	7.375 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)	210 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)	50.67 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	449.661

Axial Pressure on Section	
PuB	1.72 kip

Assumption check	
Pu/Ag	35.833 psi
0.06*f'c	300 psi
Check ACI 14.8.2.6	O.K.

Unfactored Axially Applied Loads

Unfactored Pressure on Roof uWr	270.9375 psf

Axial Pressure on Section	
PB	1.19 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	1.19 kip
Lateral	0.05 klf
Allowed service deflection	0.91 in
Msa	10.211 kip-in
M	10.297 kip-in
Ds	0.072 in
Check deflection	O.K.

Allowable Capacity

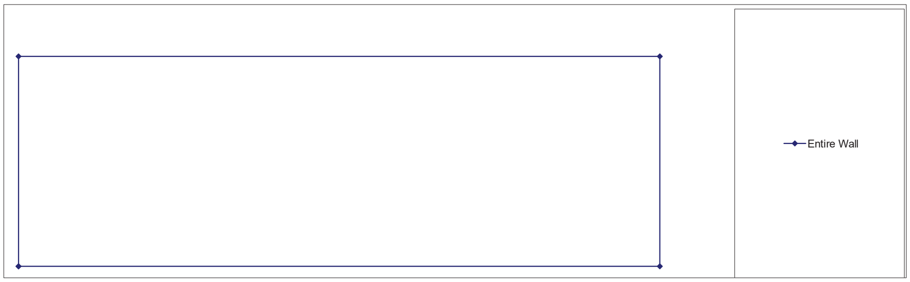
Ig = (b*h^3)/12	64 in^4
Ag = (b*h)	48 in^2
Yt = h/2	2
fr (rupture modulus)	530.330 psi
Mcr	16.971 kip-in
Beta_1	0.8
Trial Ast req'd	0.073 in^2
B	8.836162648
kd	0.542 in
I.cr	2.92 in^4
e_c	0.003
e_s	0.005
a	0.33483 psi
c	0.419 in
Ase	0.22 in^2
Icrdeflection	3.48 in^4
Ie	64.00 in^4
delta	150
r_t (maximum 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	0.232 kip-ft	

ACI eq. (14-6)		
Mu	0.330 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	16.37%	26.79%



REINFORCEMENT AT OPENINGS

Loading	
Pu (factorized load from roof)	0.23 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	77.5	77.5	22.446	23.80	23.80	33.87	33.87

Total Tension	Base Anchors					
7.224	Dist	Tension (kip)	Shear	L - Dist	Moment +	Moment -
Base Anchor 1	11 in	***3.61***	***11.22***	77.5 in	0.470 kip*ft	23.328 kip*ft
Base Anchor 2	77.5 in	***3.61***	***11.22***	11 in	23.328 kip*ft	0.470 kip*ft

	Wall Connections									
	Quantity of Anchors	Capacity of each Anchor	Countering Dead Load from Adjoining Wall	% of wall to use	Adjoining Wall	Dist (inches)	L - Dist	Allowable Force	Overturning Moment Resistance (kip-ft) Up Left	Overturning Moment Resistance (kip-ft) Low Right
Wall Connection 1	3	***1.531***	6.102	29.59%	W5	0	88.500	4.593	0.000	33.873
Wall Connection 2	3	***1.531***	5.688	29.59%	W6	88.5	0.000	4.593	33.873	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
1974	22446	20472	185	20365	OK	987	(20472)	OK

RIGIDITY

CALCULATED VALUES	100%	Final	2.431462089

Pier Label	Length (inches)	Height (inches)	Fixed Top? (Y/N)	Useable? (Y/N)	Stiffness (k) (1000 kip / IN)	Deflection (in / 1000 kip)
Entire Wall	88.5	135.88	Y	Y	2.431	0.411

Combine Logic					
First Segment	Second Segment	Re-Name	Combine/Subtract	Method	Combined
Entire Wall	0	Final			2.431

ID:	Denali DN-389 and DN-390
	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	132
Y Corridinate	9
Direction of Wall	Y
Center of gravity X	132.000
Center of gravity Y	53.250
Wall Weight	4185.000 lbs.
Central wall?	Yes
Wall that supports 2 roof panels?	No
lop (length of opening on wall)	0 ft
H (height of wall)	135.88 in
Lh (length of wall)	7.375 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)	210 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)	50.67 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	449.661

Axial Pressure on Section	
PuB	1.72 kip

Assumption check	
Pu/Ag	35.833 psi
0.06*f'c	300 psi
Check ACI 14.8.2.6	O.K.

Unfactored Axially Applied Loads

Unfactored Pressure on Roof uWr	270.9375 psf

Axial Pressure on Section	
PB	1.19 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	1.19 kip
Lateral	0.05 klf
Allowed service deflection	0.91 in
Msa	10.211 kip-in
M	10.297 kip-in
Ds	0.072 in
Check deflection	O.K.

Allowable Capacity

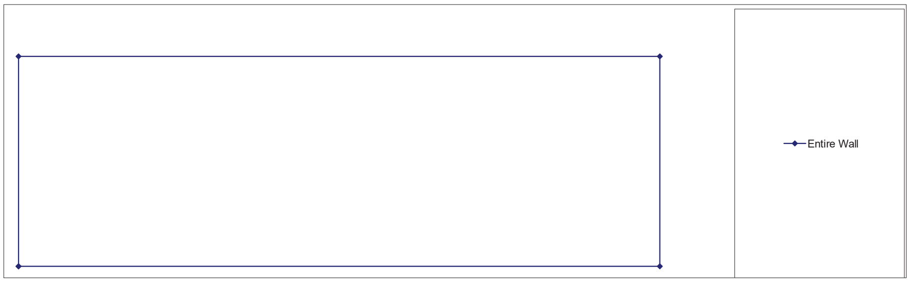
Ig = (b*h^3)/12	64 in^4
Ag = (b*h)	48 in^2
Yt = h/2	2
fr (rupture modulus)	530.330 psi
Mcr	16.971 kip-in
Beta_1	0.8
Trial Ast req'd	0.073 in^2
B	8.836162648
kd	0.542 in
I.cr	2.92 in^4
e_c	0.003
e_s	0.005
a	0.33483 psi
c	0.419 in
Ase	0.22 in^2
Icrdeflection	3.48 in^4
Ie	64.00 in^4
delta	150
r_t (maximum 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	0.232 kip-ft	

ACI eq. (14-6)		
Mu	0.330 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	16.37%	26.79%



REINFORCEMENT AT OPENINGS

Loading	
Pu (factorized load from roof)	0.23 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	77.5	77.5	22.446	23.80	23.80	33.87	33.87

Total Tension	Base Anchors					
7.224	Dist	Tension (kip)	Shear	L - Dist	Moment +	Moment -
Base Anchor 1	11 in	3.61	11.22	77.5 in	0.470 kip*ft	23.328 kip*ft
Base Anchor 2	77.5 in	3.61	11.22	11 in	23.328 kip*ft	0.470 kip*ft

	Wall Connections									
	Quantity of Anchors	Capacity of each Anchor	Countering Dead Load from Adjoining Wall	% of wall to use	Adjoining Wall	Dist (inches)	L - Dist	Allowable Force	Overturning Moment Resistance (kip-ft) Up Left	Low Right
Wall Connection 1	3	1.531	6.102	29.59%	W5	0	88.500	4.593	0.000	33.873
Wall Connection 2	3	1.531	5.688	29.59%	W6	88.5	0.000	4.593	33.873	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
1974	22446	20472	185	20365	OK	987	(20472)	OK

RIGIDITY

CALCULATED VALUES	100%	Final	2.431462089

Pier Label	Length (inches)	Height (inches)	Fixed Top? (Y/N)	Useable? (Y/N)	Stiffness (k) (1000 kip / IN)	Deflection (in / 1000 kip)
Entire Wall	88.5	135.88	Y	Y	2.431	0.411

Combine Logic					
First Segment	Second Segment	Re-Name	Combine/Subtract	Method	Combined
Entire Wall	0	Final			2.431

ID:	Denali DN-389 and DN-390
	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	

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	16
Y Corridinate	7
Direction of Wall	X
Center of gravity X	113.993
Center of gravity Y	7.000
Wall Weight	6572.000 lbs.
Central wall?	Yes
Wall that supports 2 roof panels?	Yes
lop (length of opening on wall)	0 ft
H (height of wall)	104.25 in
Lh (length of wall)	16.333 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)	210 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)	50.67 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	449.661

Axial Pressure on Section	
PuB	1.6 kip

Assumption check	
Pu/Ag	33.333 psi
0.06*f'c	300 psi
Check ACI 14.8.2.6	O.K.

Unfactored Axially Applied Loads

Unfactored Pressure on Roof uWr	270.9375 psf

Axial Pressure on Section	
PB	1.08 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)	0.01 klf
Hw = W*(H^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 / L^4 + H^4)	0 klf
Hw = W*(H^4 / H^4 + L^4)	0.05 klf

Shear

Factored Loading per ACI	ACI eq. 9-3
Vu = wuB*(Bw-2db) / 2	0.06
Phi*Vc/2	1.33
Check Shear ACI 11.5.5.1	O.K.

Deflection ACI 14.8.4

Service Loads	
Axial	1.08 kip
Lateral	0 klf
Allowed service deflection	0.70 in
Msa	0.540 kip-in
M	0.542 kip-in
Ds	0.002 in
Check deflection	O.K.

Allowable Capacity

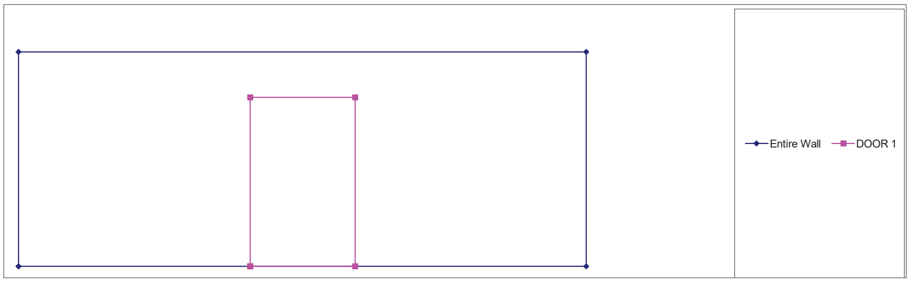
Ig = (b*h^3)/12	64 in^4
Ag = (b*h)	48 in^2
Yt = h/2	2
fr (rupture modulus)	530.330 psi
Mcr	16.971 kip-in
Beta_1	0.8
Trial Ast req'd	0.073 in^2
B	8.836162648
kd	0.542 in
I.cr	2.92 in^4
e_c	0.003
e_s	0.005
a	0.33483 psi
c	0.419 in
Ase	0.22 in^2
Icrdeflection	3.47 in^4
Ie	64.00 in^4
delta	150
r_t (maximum tensile reinforcement)	0.0166
r_{temp} (min. temperature reinforcement)	0.0014
r_{min} (minimum tensile reinforcement)	0.0027
r_{trial} (trial reinforcement ratio bottom)	0.0033
$\rho_{provided}$ (reinforcement ratio provided)	0.0090

Flexure

Assumption check		
Span	Hw	Lw
net Tensile Strain	0.010	0.010
Check ACI 14.8.2.3	Tension	Tension
Mua	0.917 kip-ft	

ACI eq. (14-6)		
Mu	1.090 kip-ft	0.330 kip-ft

ACI 9.3.2		
fb	0.9	0.9
fMn trial = ϕAsFy(dt - a/2)	2.020 kip-ft	2.020 kip-ft
DM =Mpos - ϕM	0.000 kip-ft	0.000 kip-ft
As Add'l req'd	0.00 in^2	0.00 in^2
Additional reinf req`d	0.00 in^2	0.00 in^2
Add'l bar size:	3	3
qty req`d	0	0
or spacing of:	0	0
As add'l =	0.000 kip-ft	0.000 kip-ft
Ast = As + As add'l	0.20 in^2	0.20 in^2
fMn = ϕAsFy(db - a/2)	2.016 kip-ft	2.016 kip-ft
Check ϕMn > Mu	O.K.	O.K.
% allowed	54.07%	16.37%



REINFORCEMENT AT OPENINGS

Loading	
Pu (factorized load from roof)	0.23 klf
Ww (weight of panel per sq ft)	0.05 ksf

Material Properties	
db (effective depth bottom)	1.84 in
a (block of strain)	0.33483 psi
	a=As * fy / (0.85 * f'c *b)

Factorized Moment

Opening	Horizontal Location	Vertical Location	L length of opening	H height above opening	(-) Weight of Opening (LBS)	Pw total factorized panel load	wu total factorized load	**Mu** (wu*L^2)/12
DOOR 1	6.66 ft	0 ft	3.02 ft	1.83 ft	1035.48	0.09 klf	0.32 klf	0.24 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	0.003 in^2	No. 3	1	10.09 kip-ft	O.K.

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
4	193	193	31.085	87.64	87.64	163.32	163.32

Total Tension	Base Anchors					
14.048	Dist	Tension (kip)	Shear	L - Dist	Moment +	Moment -
Base Anchor 1	3 in	3.38	3.33	193 in	0.013 kip*ft	54.410 kip*ft
Base Anchor 2	67 in	3.64	12.21	129 in	7.057 kip*ft	26.161 kip*ft
Base Anchor 3	129 in	3.64	12.21	67 in	26.161 kip*ft	7.057 kip*ft
Base Anchor 4	193 in	3.38	3.33	3 in	54.410 kip*ft	0.013 kip*ft

	Wall Connections									
	Quantity of Anchors	Capacity of each Anchor	Countering Dead Load from Adjoining Wall	% of wall to use	Adjoining Wall	Dist (inches)	L - Dist	Allowable Force	Overturning Moment Resistance (kip-ft) Up Left	Low Right
Wall Connection 1	3	1.531	6.278	43.20%	W1	0	196.000	4.593	0.000	75.019
Wall Connection 2	2	2.703	5.705	50.00%	W3	80	116.000	5.406	36.040	52.258
Wall Connection 3	2	2.703	5.705	50.00%	W4	116	80.000	5.406	52.258	36.040
Wall Connection 4	3	1.531	6.278	43.20%	W2	196	0.000	4.593	75.019	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
6467	31085	24618	337	14198	OK	1617	(24618)	OK

RIGIDITY

CALCULATED VALUES	70%	Final	7.985693594

	Pier Label	Length (inches)	Height (inches)	Fixed Top? (Y/N)	Useable? (Y/N)	Stiffness (k) (1000 kip / IN)	Deflection (in / 1000 kip)
	Entire Wall	196	104.25	Y	Y	11.454	0.087
DOOR 1	A'	196	82.29	Y	Y	14.998	0.067
	A	79.92	82.29	Y	Y	4.784	0.209
	B	79.84	82.29	Y	Y	4.777	0.209

	Combine Logic					
	First Segment	Second Segment	Re-Name	Combine/Subtract	Method	Combined
DOOR 1	Entire Wall	A'	A'a	-	Deflection	0.021
	A	B	AB	+	Stiffness	9.561
	A'a	AB	Final	+	Deflection	0.125

ID:	Denali DN-389 and DN-390
	DESIGN OF WALL MARKED W6
Notes	

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

Material Properties

f'c	5000 psi	
Steel Reinforcement	Plain WWF Grade 80	
Fy wire mesh	80000 psi	
Fy rebar	60000 pcf	
Lightweight?	No	
Concrete density	150 pcf	O.K.
E (Steel)	29000000 psi	
E (Concrete)	4290000 psi	ACI 8.5.1
n (modular ratio)	6.76	

Shear Parameters

Phi.v	0.85	ACI 9.3.2.3
Vc	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	16
Y Corridinate	99.5
Direction of Wall	X
Center of gravity X	114.011
Center of gravity Y	99.500
Wall Weight	5175.000 lbs.
Central wall?	Yes
Wall that supports 2 roof panels?	Yes
lop (length of opening on wall)	0 ft
H (height of wall)	104.25 in
Lh (length of wall)	16.333 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)	210 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)	50.67 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	449.661

Axial Pressure on Section	
PuB	1.54 kip

Assumption check	
Pu/Ag	32.083 psi
0.06*f'c	300 psi
Check ACI 14.8.2.6	O.K.

Unfactored Axially Applied Loads

Unfactored Pressure on Roof uWr	270.9375 psf

Axial Pressure on Section	
PB	1.02 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)	0.01 klf
Hw = W*(H^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 / L^4 + H^4)	0 klf
Hw = W*(H^4 / H^4 + L^4)	0.05 klf

Shear

Factored Loading per ACI	ACI eq. 9-3
Vu = wuB*(Bw-2db) / 2	0.06
Phi*Vc/2	1.33
Check Shear ACI 11.5.5.1	O.K.

Deflection ACI 14.8.4

Service Loads	
Axial	1.02 kip
Lateral	0 klf
Allowed service deflection	0.70 in
Msa	0.510 kip-in
M	0.512 kip-in
Ds	0.002 in
Check deflection	O.K.

Allowable Capacity

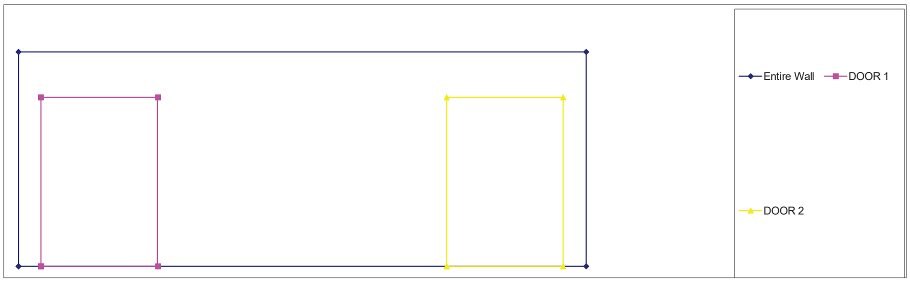
Ig = (b*h^3)/12	64 in^4
Ag = (b*h)	48 in^2
Yt = h/2	2
fr (rupture modulus)	530.330 psi
Mcr	16.971 kip-in
Beta_1	0.8
Trial Ast req'd	0.073 in^2
B	8.836162648
kd	0.542 in
I.cr	2.92 in^4
e_c	0.003
e_s	0.005
a	0.33483 psi
c	0.419 in
Ase	0.22 in^2
Icrdeflection	3.47 in^4
Ie	64.00 in^4
delta	150
r_t (maximum tensile reinforcement)	0.0166
r_{temp} (min. temperature reinforcement)	0.0014
r_{min} (minimum tensile reinforcement)	0.0027
r_{trial} (trial reinforcement ratio bottom)	0.0033
$\rho_{provided}$ (reinforcement ratio provided)	0.0090

Flexure

Assumption check		
Span	Hw	Lw
net Tensile Strain	0.010	0.010
Check ACI 14.8.2.3	Tension	Tension
Mua	0.914 kip-ft	

ACI eq. (14-6)		
Mu	1.080 kip-ft	0.330 kip-ft

ACI 9.3.2		
fb	0.9	0.9
fMn trial = ϕAsFy(dt - a/2)	2.020 kip-ft	2.020 kip-ft
DM =Mpos - ϕM	0.000 kip-ft	0.000 kip-ft
As Add'l req'd	0.00 in^2	0.00 in^2
Additional reinf req`d	0.00 in^2	0.00 in^2
Add'l bar size:	3	3
qty req`d	0	0
or spacing of:	0	0
As add'l =	0.000 kip-ft	0.000 kip-ft
Ast = As + As add'l	0.20 in^2	0.20 in^2
fMn = ϕAsFy(db - a/2)	2.016 kip-ft	2.016 kip-ft
Check ϕMn > Mu	O.K.	O.K.
% allowed	53.57%	16.37%



REINFORCEMENT AT OPENINGS

Loading	
Pu (factorized load from roof)	0.23 klf
Ww (weight of panel per sq ft)	0.05 ksf

Material Properties	
db (effective depth bottom)	1.84 in
a (block of strain)	0.33483 psi
	a=As * fy / (0.85 * f'c *b)

Factorized Moment

Opening	Horizontal Location	Vertical Location	L length of opening	H height above opening	(-) Weight of Opening (LBS)	Pw total factorized panel load	wu total factorized load	**Mu** (wu*L^2)/12
DOOR 1	0.66 ft	0 ft	3.35 ft	1.83 ft	1148.63	0.09 klf	0.32 klf	0.3 kip-ft
DOOR 2	12.32 ft	0 ft	3.35 ft	1.83 ft	1148.63	0.09 klf	0.32 klf	0.3 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	0.003 in^2	No. 3	1	10.09 kip-ft	O.K.
DOOR 2	0.9	0.003 in^2	No. 3	1	10.09 kip-ft	O.K.

CONNECTIONS

			Full Resistance Value				
			Overturning				
Base Anchors			Lateral	Base Anchors		Wall-Wall Connection	
Quantity	Maximum	Maximum	Shear	Moment +	Moment -	Moment +	Moment -
in Shear	R - Distance	L - Distance	kip	kip - ft	kip - ft	kip - ft	kip - ft
4	193	193	31.085	87.64	87.64	163.32	163.32

Total Tension	Base Anchors					
14.048	Dist	Tension (kip)	Shear	L - Dist	Moment +	Moment -
Base Anchor 1	3 in	3.38	3.33	193 in	0.013 kip*ft	54.410 kip*ft
Base Anchor 2	67 in	3.64	12.21	129 in	7.057 kip*ft	26.161 kip*ft
Base Anchor 3	129 in	3.64	12.21	67 in	26.161 kip*ft	7.057 kip*ft
Base Anchor 4	193 in	3.38	3.33	3 in	54.410 kip*ft	0.013 kip*ft

	Wall Connections									
	Quantity of Anchors	Capacity of each Anchor	Countering Dead Load from Adjoining Wall	% of wall to use	Adjoining Wall	Dist (inches)	L - Dist	Allowable Force	Overturning Moment Resistance (kip-ft)	
									Up Left	Low Right
Wall Connection 1	3	1.531	8.253	56.80%	W1	0	196.000	4.593	0.000	75.019
Wall Connection 2	2	2.703	5.705	50.00%	W3	80	116.000	5.406	36.040	52.258
Wall Connection 3	2	2.703	5.705	50.00%	W4	116	80.000	5.406	52.258	36.040
Wall Connection 4	3	1.531	8.253	56.80%	W2	196	0.000	4.593	75.019	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
5748	31085	25337	306	12870	OK	1437	(25337)	OK

RIGIDITY

CALCULATED VALUES	63%	Final	7.238396929

	Pier Label	Length (inches)	Height (inches)	Fixed Top? (Y/N)	Useable? (Y/N)	Stiffness (k) (1000 kip / IN)	Deflection (in / 1000 kip)
	Entire Wall	196	104.25	Y	Y	11.454	0.087
DOOR 1	A'	196	82.29	Y	Y	14.998	0.067
	A	7.92	82.29	Y	N	0.000	0.000
	B	147.88	82.29	Y	Y	10.860	0.092
DOOR 2	B'	196	82.29	Y	Y	14.998	0.067
	C	147.84	82.29	Y	Y	10.856	0.092
	D	7.96	82.29	Y	N	0.000	0.000

	Combine Logic					
	First Segment	Second Segment	Re-Name	Combine/Subtract	Method	Combined
DOOR 1	Entire Wall	A'	A'a	-	Deflection	0.021
	A	B	AB	+	Stiffness	10.860
	A'a	AB	A'b	+	Deflection	0.113
DOOR 2	A'b	B'	B'a	-	Deflection	0.046
	C	D	CD	+	Stiffness	10.856
	B'a	CD	Final	+	Deflection	0.138

ID: **Denali DN-389 and DN-390**

DESIGN OF FLOOR PANEL F1

Material Properties

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

Geometric Properties

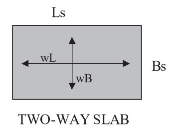
Ls (overall length of slab)	17.18 ft		
Bs (overall width of slab)	9.92 ft		
Design will be performed as :	**Two-way slab**		
tfr (floor finish thickness)	0 in		
b (section width)	12 in		*(typically 12 inches)*
h (section thickness)	5 in		
ct (cover top)	1 in		
cb (cover bottom)	1 1/2 in		
rd (assumed reinf. diameter)	0.319 in		*(if centered enter 0)*
l (length of slab for deflection)	29.451 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}*

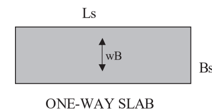
Reinforcement Limits

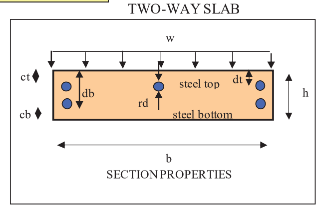
ρ_t (maximum tensile reinforcement)	0.0166	*ACI 10.3.3*
ρ_{temp} (min. temperature reinforcement)	0.0014	*ACI 7.12.2*
ρ_{min} (minimum tensile reinforcement)	0.0027	*ACI 10.5.1*
ρ_{trial} (trial reinforcement ratio bottom)	0.0033	
ρ_{trial} (trial reinforcement ratio top)	0.0033	

Notes:

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







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

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	0.17 klf	0.55 klf

3.307 ft 0.28 kip 0.28 kip

2.45 ft 0.67 kip 0.67 kip

B (Span in the short direction) = (Bs/NsB-1) - 0(oh1)	3.307 ft
L (Span in the long direction) = (Ls/NsL-1) - 0(oh2)	2.45 ft

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

3.307 ft 0.18 kip 0.18 kip

2.45 ft 0.44 kip 0.44 kip

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

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

3.30 0.15 kip 0.15 kip

2.45 ft 0.38 kip 0.38

Eficiency can be enhanced if Ast is diminished

SUMMARY

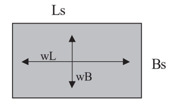
Use 1 Layer of Wire Mesh on Top W6.7 x W6.7 x 4 x 4

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

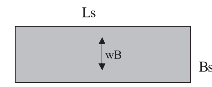
ID: **Denali DN-389 and DN-390**

DESIGN OF FLOOR PANEL F1

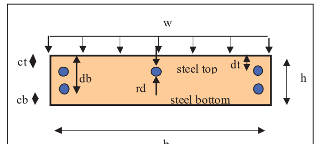
Notes:



TWO-WAY SLAB



ONE-WAY SLAB



SECTION PROPERTIES

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)	17.18 ft		
Bs (overall width of slab)	9.92 ft		
Design will be performed as :	**Two-way slab**		
tfr (floor finish thickness)	0 in		
b (section width)	12 in	*(typically 12 inches)*	
h (section thickness)	5 in		
ct (cover top)	1 in		
cb (cover bottom)	1 1/2 in		
rd (assumed reinf. diameter)	0.319 in	*(if centered enter 0)*	
l (length of slab for deflection)	29.451 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	0.15 kip-ft	0.036	Tension	0.9	3.64 kip-ft		3.64 kip-ft	O.K.	4.09%
**	Mneg (negative Moment) = (wB*B^2)*0.1	0.19 kip-ft	0.036	Tension	0.9	4.44 kip-ft		4.44 kip-ft	O.K.	4.19%

***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	0.26 kip-ft	0.036	Tension	0.9	3.64 kip-ft		3.64 kip-ft	O.K.	7.07%
**	Mneg (negative Moment) = (wB*B^2)*0.106	0.11 kip-ft	0.036	Tension	0.9	4.44 kip-ft		4.44 kip-ft	O.K.	2.44%

***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	0.34 kip	0.85	5.40 kip	4.59 kip	O.K.	7.35%
				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	0.53 kip	0.85	5.40 kip	4.59 kip	O.K.	11.48%
				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	0.15 kip-ft	0.15 kip-ft	125.00 in	125.00 in	0.001 in	0.0053	0.9500	0.001 in	0.001 in	0.2204 in	0.0827 in	O.K.	O.K.	0.24%	0.62%
L	0.26 kip-ft	0.27 kip-ft	125.00 in	125.00 in	0.001 in	0.0053	0.9500	0.001 in	0.001 in	0.1633 in	0.0613 in	O.K.	O.K.	0.33%	0.83%

ID: **Denali DN-389 and DN-390**

Geometric properties	
Bs (width of roof panel)	10.25 ft
Ls (Length of roof panel)	19.00 ft
Ar Area of Roof	194.75 ft^2
H (height of building)	12.58 ft
Lb (length of building)	9.5 ft
Wb (width of building)	17 ft
Ab (Area of building)	161.5 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)	128.47 ft^2
Hw (depth of floodwater)	1 ft

Loading	
Wv (weight of vault)**	0 lb
Wtr (roof panel weight)	14280 lb
Ww (total walls panel weight)	27333 lb
Fw (floor panel weight)	10335 lb
We (estimated weight of building)	51948 lb
Wev (estimated weight of building w/ vault)	51948 lb
PSFr (roof snow load)	210 psf
PSFf (Floor Live Load)	400 psf
Pmax (Maximum allowable pressure)	1500 psf
Fupmw (MWFRS Uplift Force)	40.35 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)	4896.4 lb
	.7*Vseismic (from seismic analysis without snow)	4230.3 lb
	Vwind = WLlat * max(Wb,Lb)*H	11065.7 lb

* Load adjustment per IBC 1605.3 load combinations.

Sliding Resistance with Snow	Pslide = u*(.6*We+.75*PSFr*Ar)	Pslide =	24736.77 lb

					Fsreqd	
Factor of Safety	FSwind = Pslide / Vwind	FSwind =	2.2	≥	1.0	O.K.
	FSseismic = Pslide / Vseismic	Fseismic =	5.1	≥	1.0	O.K.

Sliding Resistance with No Snow	Pslide = u*.6*We	Pslide =	12467.52 lb

					Fsreqd	
Factor of Safety	FSwind = Pslide / Vwind	Fswind =	1.1	≥	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)	48.913 kip-ft
	.7*Otseismic (from seismic analysis without snow)	42.096 kip-ft
	Otwind = (WLlat*Lb*H^2 / 2) + (Fupmw*Lb*Wb^2 / 2)	94.283 kip-ft

* Load adjustment per IBC 1605.3 load combinations.

Overturning Resistance with Snow	Otrsnow = (.6*We+.75*PSFr*Ar)*(Wb/2)	Otrsnow =	281.776 kip-ft

					Fsreqd	
Factor of Safety	FSwind = Otrsnow / Otwind	FSwind =	2.99	≥	1.0	O.K.
	FSseismic = Otrsnow / Vseismic	Fseismic =	5.76	≥	1.0	O.K.

Overturning Resistance with No Snow	Otr = .6*We*Wb/2	Otr	264.935 kip-ft

					Fsreqd	
Factor of Safety	FSwind = Otr / Vwind	Fswind =	2.81	≥	1.0	O.K.
	FSseismic = Otr / Vseismic	Fseismic =	6.29	≥	1.0	O.K.

CHECK BEARING PRESSURE CONDITION

Net Pressure	Pnet = (Wev + PSFr*Ar + PSFf*Af) / Ab	974.89 psf

Allowable	Pmax ≥ Pnet	1500 psf ≥ 974.89 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** =	8016.67 lb

Factor of Safety	FSb = We / Fb	**FSb =**	6.48	≥	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.