

LEE'S SUMMIT LOGISTICS BUILDING B LOT 2 LEE'S SUMMIT, MO PROJECT NO. 2220003

STRUCTURAL CALCULATIONS



04/22/2022

JAMES GRANICH, P.E. ENGINEER OF RECORD

> wallace design collective, pc structural - civil - landscape - survey 1703 wyandotte street, suite 200 kansas city, missouri 64108 816.421.8282 - 800.364.5858 wallace.design

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of

Sheet No.

Date	4/19/2022
Job	Project Birkdale
Subject	Roof Loads

DEAD AND LIVE LOADS ASCE 7-16 Table C3.1-1a and Table 4.3.1

ROOF LOADS

DEAD

ITEM	DESCRIPTION	UNIT NEIGHT (PSF)	TOTAL
Roof Covering Insulation Deck Ceiling HVAC Sprinklers Fire-Proofing Waterproofing Miscellaneous	EPDM Membrane Polystyrene Foam (per inch thickness) Metal Deck, 22 gage, 1.5" B None HVAC Allowance (except sprinklers) Branch Lines Only	1.0 x 0.50 3.5 x 0.20 1.0 x 2.00 1.0 x 0.00	0.5 0.7 2.0 0.0 1.5 2.0 0.00 0.00 1.50
Sub-Total			8.2
Secondary Fram	ni K-Series (30K10) joists at 6'-0" O.C.	1	2.0
Sub-Total			10.2
Primary Framing	j Joist Girders (60' max. span)	1	1.5
Total			11.7

ROOF LIVE/SNOW LOADS

ITEM	LOAD (PSF)
Roof Live Load	20.0
Factored Roof Snow Load, Pf	20.0

	Date 3/10/2022		Sheet		of	
	Job 2220003 Building 2					
	Subject E/W Wind TOW: 39.041ft					
ICE 7-16. Chapters 26, 27 and 30						
,	1. Input					
windward Leews	Design Parameters					
Pressure Pressure	Basic Wind Speed, V =			109 r	mph (Section 26	6.5, Fig. 1A-2D)
odre	Exposure Category (B, C, or D) =			C (Section 26.7)	
oof	Building Risk Category (I, II, III, IV) =			II ((Table 1.5-1)	
	Civil finished floor elevation (if unknown input 0) =			0.00 f	eet (Sect. 26.9,	Table 26.9-1)
val 📕 🚽 📲						
	Eave Height, He =			33.63 f	eet	
	4 Max Building Height of Ridge Height above ground Parapet Height above ground level. Hp =	ievei, Hr =		37.92 I 29 50 f	eel	
	Building Width Perpendicular to Wind B =			210 00 f	eet (max bldg c	lim)
L	Building Width Parallel to Wind, L =			540.00 f	eet)
	Enclosure Classification =		Enclosed	Buildings	(Section 26.12)	
EFER TO FIGURE 27.3-1	Roof Configuration = Gabled, Hi	pped or Mond	oslope Ro	ofs (ø ≤ 7)		
	Angle of Plane of Roof From Horizontal, θ =			1.20	degrees	
	Is building on or near a hill, ridge, or escarpment?			N ((Y or N) (Section	n 26.8)
	Height of Hill or Escarpment relative to upwind terra	ain, H =		10.00 f	eet (Section 26	.8, Fig. 26.8-1)
x (upwind) x	Horiz. Dist. Upwind to Point Where Elevation = H/2	, Lh =		10.00 f	eet (Section 26	.8, Fig. 26.8-1)
	Horiz. Dist. from Crest to Building Site, x =			10.00 f	eet (Section 26	.8, Fig. 26.8-1)
	2D Ridge, 2D Escarpment, or Axisymmetrical Hill =			E ((R, E, or H)	
V(z) Î	Is the building site upwind or downwind of the crest	?		DOWN	(up, down)	
	2. Calculations - Main Wind Force Resisting System					
	Equivalent Allowable Stress Design Wind Speed, V	asd =		84.43 r	mph (IBC 2018,	1609.3.1)
	Mean roof height, h =			33.63 f	eet	
	Kz, velocity pressure exposure coefficient at hz = 3	7.92ft =		1.03	Table 26.10-1	(use with qz)
	Kz, velocity pressure exposure coefficient at hh = 3	3.63ft =		1.00	Table 26.10-1	(use with qh)
	Kz, velocity pressure exposure coefficient at hp = 3	8.5ft =		1.03	Table 26.10-1	(use with qp)
2-D Ridge or Axisymmetrical Hill	Kzt,topographic factor at hz = 37.92ft =			1.00	Figure 26.8-1	(use with qz)
	Kzt,topographic factor at hh = 33.63ft =			1.00	Figure 26.8-1	(use with qh)
EFER TO FIGURE 26.8-1	Kzt,topographic factor at hp = 38.5ft =			1.00	Figure 26.8-1	(use with qp)
	Kd, wind directionality factor =			0.85	Table 26.6-1	
	Ke, ground elevation factor at			1.00	Table 26.9-1	
	G, gust factor =			0.85 \$	Section 26.11.4	
	az velecity pressure at bz = 27.02ft =			26.62	oof /Eq. 26.10.1	`
	db velocity pressure at hb = 33.63ft =			25.85	osf (Eq. 20.10-1 osf (Eq. 26.10-1)
	qp, velocity pressure at hp = 38.5ft =			26.63	osf (Eq. 26.10-1)
	Walls: P = q(GCpf-GCpi) Eqn. 27.3-1	qz	GCp	GCpi	(1.0)P	(0.6)P
	Windward pressure	26.63	0.68		18.1 psf	10.9 psf
			00	00	4 000	/A A/F
	Looward Prossure	qh	GCp	ссрі	(1.0)P	(0.6)P
	Sidewall pressure	20.00 25.05	-0.23	0 1 9	-o psi	-3.0 pSI
	Internal Pressure	25.00	-0.00	0.10	-20 psi 4 7 nef	-1∠ µsi 2 8 nef
	internal i ressure	20.00		0.10	4.7 por	2.0 por
	(1.0)W = (1.0)(Windward + Leeward Pressure) =	18	.11 psf +	5.96 psf =	24.1 psf	
	(0.6)W =(0.6)(Windward + Leeward Pressure) =	10	.86 psf +	3.58 psf =	14.4 psf	
			•	•		
	Parapets: Pp =qp(GCpn) Eqn. 27.3-3	qp	GCpn		(1.0)Pp	(0.6)Pp
	Windward parapet pressure	26.63	1.5	_	39.9 psf	24 psf
	Leeward parapet pressure	26.63	-1.0		-26.6 psf	-16 psf
	Windward + Looward Processing	06.60	2 50		66 6 maf	20.0
	winuwaru + Leewaru Pressure	20.03	2.00		00.0 psr	sara bai
	Roof Normal to Ridge (8>10 degrees)	ah	GCn	GCni	(1.0)P	(0.6)P
	Windward Pressure case i	25.85	-0.60	0.18	-20 psf	-12 psf
	case ii	25.85	-0.15	0.18	-8.6 psf	-5.2 psf
	Leeward Pressure	25.85	-0.26	0.18	-11.2 psf	-6.7 psf
	Roof All Other Conditions	qh	GCp	GCpi	(1.0)P	(0.6)P
	For U to $h/2 = 0$ ft to 16.81 ft	25.85	-0.77	0.18	-24.4 psf	-14.7 psf
	11/2 to $11 = 10.01$ it to 33.03 lt b to $2b = 33.63$ ft to 67.25 ft	20.00	-0.//	0.10	-24.4 psr	-14.7 pst
	Π 10 2 Π = 33.03 II 10 07.25 Π	25.85	-0.43	0.18	-15.6 pst	-9.4 pst
	~211 - ~07.23 IL	20.00	-0.20	0.10	-11.2 psi	-o./ hst
	Roof Overhangs Section 27.3.3	ah	GCn	GCni	(1.0)P	(0.6)P
	Maximum pressures	25.85	-0.77	0.68	-37.4 psf	-22.4 psf

	Date 3/10/2022		Sheet		of	
	Job 2220003 Building 2					
	Subject E/W Wind TOW: 42.50ft					
ND ANALYSIS: ANALYTICAL - ALL HEIGHTS METHOD						
CE 7-16, Chapters 26, 27 and 30	4 Janué					
undward kee	Design Parameters					
pressure Press	Basic Wind Speed, V =			109 r	mph (Section 26	6.5, Fig. 1A-2D)
Pris Sure	Exposure Category (B, C, or D) =			С ((Section 26.7)	
of the second se	Building Risk Category (I, II, III, IV) =			II ((Table 1.5-1)	
	Civil finished floor elevation (if unknown input 0) =			0.00 f	feet (Sect. 26.9,	Table 26.9-1)
	Eave Height, He = Max Building Height or Ridge Height above ground	level Hr =		33.63	eet	
///////////////////////////////////////	Parapet Height above ground level. Hp =			40.00 f	eet	
	Building Width Perpendicular to Wind, B =			210.00 f	eet (max bldg o	lim)
	Building Width Parallel to Wind, L =			540.00 f	eet	
	Enclosure Classification =		Enclosed	Buildings ((Section 26.12)	
FER TO FIGURE 27.3-1	Angle of Plane of Roof From Horizontal, A =	ppea or wond	osiope Ro	$\frac{1}{20} (0 \le 7)$	learees	
				1.20	legrees	
	Is building on or near a hill, ridge, or escarpment?			N (Y or N) (Section	n 26.8)
	Height of Hill or Escarpment relative to upwind terra	ain, H =		10.00 f	eet (Section 26	.8, Fig. 26.8-1)
x (upwind) x	Horiz. Dist. Upwind to Point Where Elevation = H/2,	, Lh =		10.00 f	eet (Section 26	.8, Fig. 26.8-1)
	Horiz. Dist. from Crest to Building Site, x =			10.00 f	eet (Section 26	.8, Fig. 26.8-1)
	2D Ridge, 2D Escarpment, or Axisymmetrical Hill =			Ε ((R, E, or H)	
V(z) I	Is the building site upwind or downwind of the crest	?		DOWN	(up, down)	
	2. Colordations Main Wind Force Desisting System					
	2. Calculations - Main Wind Force Resisting System			04.42		4000 2 4)
	Equivalent Allowable Stress Design Wind Speed, V	aso =		84.43	прп (IBC 2018,	1609.3.1)
	Mean root neignt, n = Kz. velecity pressure expessive coefficient at bz = 2	7 02# -		33.03	eel Tabla 26 10 1	(uco with az)
	Kz, velocity pressure exposure coefficient at hz = 3	7.9211 - 3.63ft =		1.03	Table 20.10-1	(use with qb)
	Kz, velocity pressure exposure coefficient at hn = 4	0ft =		1.00	Table 26.10-1	(use with gn)
2-D Ridge or Axisymmetrical Hill	Kzt.topographic factor at hz = 37.92ft =	on		1.00	Figure 26.8-1	(use with qz)
	Kzt,topographic factor at hh = 33.63ft =			1.00	Figure 26.8-1	(use with qh)
ER TO FIGURE 26.8-1	Kzt,topographic factor at hp = 40ft =			1.00	Figure 26.8-1	(use with qp)
	Kd, wind directionality factor =			0.85	Table 26.6-1	、
	Ke, ground elevation factor at			1.00	Table 26.9-1	
	G, gust factor =			0.85 \$	Section 26.11.4	
	az velocity pressure at bz = 37.92ft =			26.63	osf (Eq. 26.10-1)
	qh, velocity pressure at hh = 33.63ft =			25.85	osf (Eq. 26.10-1)
	qp, velocity pressure at hp = 40ft =			26.89 p	osf (Eq. 26.10-1)
	Walls: $\mathbf{P} = \alpha(\mathbf{GCnf} \cdot \mathbf{GCni})$ Eqn. 27.3.1	07	GCn	GCni	(1 0)P	(0.6)P
	Windward pressure	26.63	0.68	oopi	18.1 psf	10.9 psf
		qh	GCp	GCpi	(1.0)P	(0.6)P
	Leeward Pressure	25.85	-0.23	0.40	-6 psf	-3.6 psf
	Sidewali pressure	25.85	-0.60	0.18	-20 pst	-12 psf
	internal Pressure	20.00		0.10	4.7 psi	2.0 psi
	(1.0)W = (1.0)(Windward + Leeward Pressure) =	18	.11 psf +	5.96 psf =	24.1 psf	
	(0.6)W =(0.6)(Windward + Leeward Pressure) =	10	.86 psf +	3.58 psf =	14.4 psf	
			cc		(4 O)D	(0.0)
	Parapets: Pp =qp(GCpn) Eqn. 27.3-3 Windward parapet pressure	26.80	GCpn 15	-	(1.0)Pp 40.3 pef	(0.6)Pp 24.2 pcf
	Leeward parapet pressure	26.89	-1.0		-26.9 psf	-16.1 ps
	Windward + Leeward Pressure	26.89	2.50		67.2 psf	40.3 psf
	Roof Normal to Ridge (A>10 degrees)	ah	GCn	GCni	(1 N)P	(0.6)P
	Windward Pressure case i	25.85	-0.60	0.18	-20 psf	-12 psf
	case ii	25.85	-0.15	0.18	-8.6 psf	-5.2 psf
	Leeward Pressure	25.85	-0.26	0.18	-11.2 psf	-6.7 psf
	Poof All Other Conditions		<u> </u>	CC-1	(4 A) D	(0.0)5
	FOOT ALL OTHER CONDITIONS For 0 to $b/2 = 0$ ft to 16.81 ft	25.85	GCp _0 77	0 18	(1.0)P -24.4 pef	(0.6)P
	h/2 to h = 16.81 ft to 33.63 ft	25.85	-0.77	0.18	-24.4 psi	-14.7 ps
	h to $2h = 33.63$ ft to 67.25 ft	25.85	-0.43	0.18	-15.6 psf	-9.4 psf
	>2h = >67.25 ft	25.85	-0.26	0.18	-11.2 psf	-6.7 psf
					-	-
	Roof Overhangs Section 27.3.3	qh	GCp	GCpi	(1.0)P	(0.6)P
	Maximum pressures	25.85	-0.77	0.68	-37.4 psf	-22.4 psf

	Date 3/10/2022		Sheet		of	
	Job 2220003 Building 2					
	Subject E/W Wind TOW: 40.542ft					
VIND ANALYSIS: ANALYTICAL - ALL HEIGHTS METHOD						
SCE 7-10, Chapters 20, 27 and 50	1 Input					
ward Leo	Design Parameters					
Pressure Pressure	Basic Wind Speed, V =			109 r	mph (Section 26	6.5, Fig. 1A-2D)
Plo	Exposure Category (B, C, or D) =			C (Section 26.7)	
	Building Risk Category (I, II, III, IV) =			II (Table 1.5-1)	
	Civil finished floor elevation (if unknown input 0) =			0.00 f	eet (Sect. 26.9	Table 26.9-1)
voll ه العامي الم	±					
	Eave Height, He =			33.63 f	eet	
	Max Building Height or Ridge Height above ground le	vel, Hr =		37.92 f	eet	
	Parapet Height above ground level, Hp =			42.00 T	eet oot (max bldg (lim)
L	Building Width Parallel to Wind, L =			540.00 f	eet (max blug t	
	Enclosure Classification =		Enclosed	Buildings (Section 26.12)	
REFER TO FIGURE 27.3-1	Roof Configuration = Gabled, Hipp	ed or Mond	slope Ro	ofs (ø ≤ 7)	,	
	Angle of Plane of Roof From Horizontal, θ =			1.20	degrees	
	Is building on or near a hill, ridge, or escarpment?			N (Y or N) (Sectio	n 26.8)
	Height of Hill or Escarpment relative to upwind terrain	, H =		10.00 f	eet (Section 26	.8, Fig. 26.8-1)
x (upwind) x	Horiz. Dist. Upwind to Point Where Elevation = H/2, L	h =		10.00 f	eet (Section 26	.8, Fig. 26.8-1)
	 Horiz. Dist. from Crest to Building Site, x = 			10.00 f	eet (Section 26	.8, Fig. 26.8-1)
	2D Ridge, 2D Escarpment, or Axisymmetrical Hill =			E (R, E, or H)	
V(z) I	Is the building site upwind or downwind of the crest?			DOWN (up, down)	
	2. Calculations - Main Wind Force Resisting System					
	Equivalent Allowable Stress Design Wind Speed, Vas	sd =		84.43 r	mph (IBC 2018,	1609.3.1)
	Mean roof height, h =			33.63 f	eet	<i>,</i>
` `	Kz, velocity pressure exposure coefficient at hz = 37.	92ft =		1.03	able 26.10-1	(use with qz)
	Kz, velocity pressure exposure coefficient at hh = 33.	63ft =		1.00	Table 26.10-1	(use with qh)
	Kz, velocity pressure exposure coefficient at hp = 42f	t =		1.05	Table 26.10-1	(use with qp)
2-D Ridge or Axisymmetrical Hill	Kzt, topographic factor at hz = 37.92ft =			1.00	-igure 26.8-1	(use with qz)
	Kzt, topographic factor at hn = 33.631 =			1.00 1	-igure 26.8-1	(use with an)
EFER TO FIGURE 20.0-1	Kzt, topographic factor at hp = 42ft =			1.00	-igure 26.8-1	(use with qp)
	Ko, ground elevation factor at			1.00	Table 20.0-1	
	G guet factor =			0.85 9	Section 26 11 /	
	o, gust lustor			0.00	500001120.11.4	
	qz, velocity pressure at hz = 37.92ft =			26.63 p	osf (Eq. 26.10-1)
	qh, velocity pressure at hh = 33.63ft =			25.85 p	osf (Eq. 26.10-1)
	qp, velocity pressure at hp = 42ft =			27.15	ost (Eq. 26.10-1)
	Walls: P = g(GCpf-GCpi) Egn. 27.3-1	az	GCp	GCpi	(1.0)P	(0.6)P
	Windward pressure	26.63	0.68		18.1 psf	10.9 psf
	·				-	
		qh	GCp	GCpi	(1.0)P	(0.6)P
	Leeward Pressure	25.85	-0.23		-6 psf	-3.6 psf
	Sidewall pressure	25.85	-0.60	0.18	-20 psf	-12 psf
	Internal Pressure	25.85		0.18	4.7 psf	2.8 psf
	(1 N)W = (1 N)(Windward + 1 poward Pressure) =	18	11 nef +	5 96 nef =	24.1 nef	
	(0.6)W = (0.6)(Windward + Leeward Pressure) =	10	.86 psf + 3	3.58 psf =	14.4 psf	
					•	
	Parapets: Pp =qp(GCpn) Eqn. 27.3-3	qp	GCpn	_	(1.0)Pp	(0.6)Pp
	Windward parapet pressure	27.15	1.5		40.7 psf	24.4 psf
	Leeward parapet pressure	27.15	-1.0		-27.1 psf	-16.3 psf
	Windward + Leeward Pressure	27 15	2.50		67.9 nsf	40.7 nsf
		27.10	2.50		0.10 poi	par
	Roof Normal to Ridge (θ≥10 degrees)	qh	GCp	GCpi	(1.0)P	(0.6)P
	Windward Pressure case i	25.85	-0.60	0.18	-20 psf	-12 psf
	case ii	25.85	-0.15	0.18	-8.6 psf	-5.2 psf
	Leeward Pressure	25.85	-0.26	0.18	-11.2 psf	-6.7 psf
	Roof All Other Conditions	ah	GCn	GCni	(1.0)P	(0 6)P
	For 0 to $h/2 = 0$ ft to 16.81 ft	25.85	-0.77	0,18	-24.4 psf	-14.7 psf
	h/2 to $h = 16.81$ ft to 33.63 ft	25.85	-0.77	0.18	-24.4 psf	-14.7 psf
	h to 2h = 33.63 ft to 67.25 ft	25.85	-0.43	0.18	-15.6 psf	-9.4 psf
	>2h = >67.25 ft	25.85	-0.26	0.18	-11.2 psf	-6.7 psf
					-	-
	Roof Overhangs Section 27.3.3	qh	GCp	GCpi	(1.0)P	(0.6)P
	Maximum pressures	25.85	-0.77	0.68	-37.4 psf	-22.4 psf

	Date 3/10/2022		Sheet		of	
	Job 2220003 Building 2					
	Subject North Wind TOW: 40.542ft					
ND ANALYSIS: ANALYTICAL - ALL HEIGHTS METHOD						
CE 7-16, Chapters 26, 27 and 30	4 Janué					
undward Lea	1. Input Design Parameters					
Wild Pressure	Basic Wind Speed, V =			109 r	mph (Section 26	6.5, Fig. 1A-2D)
Pie	Exposure Category (B, C, or D) =			С (Section 26.7)	
	Building Risk Category (I, II, III, IV) =			II ((Table 1.5-1)	
	Civil finished floor elevation (if unknown input 0) =			0.00 f	eet (Sect. 26.9,	Table 26.9-1)
레이 이 이용 기						
	Eave Height, He =			33.63 f	eet	
	Parapet Height above ground level Hp =	ievei, ni –		40 00 f	eel eet	
	Building Width Perpendicular to Wind, B =			540.00 f	eet (max bldg c	lim)
L L	Building Width Parallel to Wind, L =			210.00 f	feet	
	Enclosure Classification =		Enclosed	Buildings (Section 26.12)	
EFER TO FIGURE 27.3-1	Roof Configuration = Gabled, Hi	oped or Mono	oslope Ro	ofs (ø ≤ 7)		
	Angle of Plane of Roof From Horizontal, θ =			1.20	degrees	
	Is building on or pear a bill ridge or escaroment?			N (Y or N) (Section	n 26 8)
	Height of Hill or Escaroment relative to upwind terra	in. H =		10.00 f	eet (Section 26	.8. Fig. 26.8-1)
	Horiz, Dist, Upwind to Point Where Elevation = H/2	Lh =		10.00 f	eet (Section 26	.8. Fig. 26.8-1)
x (upwind)	Horiz, Dist, from Crest to Building Site, x =			10.00 f	eet (Section 26	.8. Fig. 26.8-1)
	2D Ridge, 2D Escaroment, or Axisymmetrical Hill =			E	(R. E. or H)	
►	Is the building site upwind or downwind of the crest	?		DOWN	(up. down)	
	5 1				(1)	
	2. Calculations - Main Wind Force Resisting System					
	Equivalent Allowable Stress Design Wind Speed, V	asd =		84.43 r	mph (IBC 2018,	1609.3.1)
I I	Mean roof height, h =			33.63 f	eet	
	Kz, velocity pressure exposure coefficient at hz = 3	7.92ft =		1.03	Table 26.10-1	(use with qz)
	Kz, velocity pressure exposure coefficient at hh = 3	3.63ft =		1.00	Table 26.10-1	(use with qh)
	Kz, velocity pressure exposure coefficient at hp = 4	Oft =		1.04	Table 26.10-1	(use with qp)
2-D Ridge or Axisymmetrical Hill	Kzt,topographic factor at hz = 37.92ft =			1.00	Figure 26.8-1	(use with qz)
	Kzt,topographic factor at hh = 33.63ft =			1.00	Figure 26.8-1	(use with qh)
ER TO FIGURE 26.8-1	Kzt,topographic factor at hp = 40ft =			1.00	Figure 26.8-1	(use with qp)
	Kd, wind directionality factor =			0.85	Table 26.6-1	
	Ke, ground elevation factor at			1.00	Table 26.9-1	
	G, gust factor =			0.85 \$	Section 26.11.4	
	gz, velocity pressure at hz = 37.92ft =			26.63	osf (Ea. 26.10-1)
	qh, velocity pressure at hh = 33.63ft =			25.85	osf (Eq. 26.10-1	ý
	qp, velocity pressure at hp = 40ft =			26.89 p	osf (Eq. 26.10-1)
	Walls: $P = q(GCpf-GCpi)$ Eqn. 27.3-1	az	GCp	GCpi	(1.0)P	(0.6)P
	Windward pressure	26.63	0.68		18.1 psf	10.9 psf
	Leoward Pressure	qh	GCp	GCpi	(1.0)P	(0.6)P
	Leeward Pressure	25.85	-0.43	0.19	-11 pst	-6.6 pst
	Internal Pressure	25.05	-0.00	0.18	4.7 nsf	-1∠µsi 28.nsf
		20.00		2.10	po.	2.0 001
	(1.0)W = (1.0)(Windward + Leeward Pressure) =	18.11 psf + 10.99 psf = 29.1		29.1 psf		
	(0.6)W =(0.6)(Windward + Leeward Pressure) =	10).86 psf +	6.59 psf =	17.5 psf	
	Parapets: Pp =qp(GCpp) Eqp 27.3-3	an	GCnn		(1.0)Pn	(0.6)Pp
	Windward parapet pressure	26.89	1.5	-	40.3 psf	24.2 nsf
	Leeward parapet pressure	26.89	-1.0		-26.9 psf	-16.1 pst
			_			·
	Windward + Leeward Pressure	26.89	2.50		67.2 psf	40.3 psf
	Roof Normal to Ridge (A>10 degrees)	ah	GCn	GCni	(1.0)P	(0.6)P
	Windward Pressure case i	25.85	-0.60	0.18	-20 psf	-12 psf
	case ii	25.85	-0.15	0.18	-8.6 psf	-5.2 psf
	Leeward Pressure	25.85	-0.26	0.18	-11.2 psf	-6.7 psf
	Poof All Other Conditions	~h	66-	6C~i	(1 A)D	(0.6)5
	For 0 to $b/2 = 0$ ft to 16.81 ft	25.85	-0 77	0.18	(1.0)P -24.4 nef	(U.6)P
	h/2 to $h = 16.81$ ft to 33.63 ft	25.85	-0,77	0.18	-24,4 psf	-14.7 nst
	h to 2h = 33.63 ft to 67.25 ft	25.85	-0.43	0.18	-15.6 psf	-9.4 psf
	>2h = >67.25 ft	25.85	-0.26	0.18	-11.2 psf	-6.7 psf
					-	-
	Roof Overhangs Section 27.3.3	qh	GCp	GCpi	(1.0)P	(0.6)P
	Maximum pressures	25.85	-0.77	0.68	-37.4 psf	-22.4 psf

	Date 3/10/2022		Sheet		of	
	Job 2220003 Building 2					
	Subject North Wind TOW: 42.5ft					
	חר					
E 7-16, Chapters 26, 27 and 30						
, ., 	1. Input					
Windward Leewa	Design Parameters					
Pressure	Basic Wind Speed, V =			109	mph (Section 26	6.5, Fig. 1A-2D)
	Exposure Category (B, C, or D) =				(Section 26.7) (Table 1.5-1)	
	Civil finished floor elevation (if unknown input 0) =		0.00	feet (Sect 26.9	Table 26 9-1)
)		0.00	1001 (0001. 20.0,	10010 20.0 1)
	Eave Height, He =			33.63	feet	
	Max Building Height or Ridge Height above gro	und level, Hr =		37.92	feet	
	Parapet Height above ground level, Hp =			42.00	feet	
L	Building Width Perpendicular to Wind, B =			540.00 1	feet (max bldg d	lim)
·	Enclosure Classification =		Enclosed	Buildings	(Section 26 12)	
	Roof Configuration = Gabled	, Hipped or Mon	oslope Ro	ofs (ø ≤ 7)	(000000120.12)	
FER TO FIGURE 27.3-1	Angle of Plane of Roof From Horizontal, $\theta =$	· · ·	•	1.20	degrees	
	Is building on or near a hill, ridge, or escarpmer	nt?		N	(Y or N) (Section	n 26.8)
	Height of Hill or Escarpment relative to upwind	terrain, H =		10.00	teet (Section 26	.8, Fig. 26.8-1)
x (upwind) x	Horiz. Dist. Upwind to Point Where Elevation =	H/2, Lh =		10.00	teet (Section 26	.8, Fig. 26.8-1)
	Horiz. Dist. from Crest to Building Site, x =			10.00	teet (Section 26	.8, Fig. 26.8-1)
► 2 2	2D Ridge, 2D Escarpment, or Axisymmetrical F	111 =		E	(R, E, or H)	
V(z) I	is the building site upwind or downwind of the c	rest?		DOWN	(up, down)	
	2 Calculations Main Wind Force Resisting System					
	Equivalent Allowable Stress Design Wind Shee	d Vasd =		84 43	mph (IBC 2018	1609 3 1)
	Mean roof beight h =	u, vasu –		33 63 1	feet	1003.3.1)
	Kz velocity pressure exposure coefficient at bz	= 37 92ft =		1 03	Table 26 10-1	(use with az)
	Kz velocity pressure exposure coefficient at hb	= 33 63ft =		1.00	Table 26 10-1	(use with ah)
	Kz, velocity pressure exposure coefficient at hp	= 42ft =		1.05	Table 26.10-1	(use with ap)
2-D Ridge or Axisymmetrical Hill	Kzt,topographic factor at hz = 37.92ft =			1.00	Figure 26.8-1	(use with qz)
	Kzt,topographic factor at hh = 33.63ft =			1.00	Figure 26.8-1	(use with qh)
ER TO FIGURE 26.8-1	Kzt,topographic factor at hp = 42ft =			1.00	Figure 26.8-1	(use with qp)
	Kd, wind directionality factor =			0.85	Table 26.6-1	
	Ke, ground elevation factor at			1.00	Table 26.9-1	
	G, gust factor =			0.85	Section 26.11.4	
	az velocity pressure at hz = 37 92ft =			26.63	nef (Eg. 26.10-1)
	qh, velocity pressure at hh = 33.63ft =			25.85	psf (Eq. 26.10-1 psf (Eq. 26.10-1)
	qp, velocity pressure at hp = 42ft =			27.15	psf (Eq. 26.10-1)
					(1.0)	(0.0)
	Windward processor	qz	GCp	GCpi	(1.0)P	(0.6)P
	windward pressure	20.03	0.00		io.i psi	iu.a bst
		qh	GCp	GCpi	(1.0)P	(0.6)P
	Leeward Pressure	25.85	-0.43		-11 psf	-6.6 psf
	Sidewall pressure	25.85	-0.60	0.18	-20 psf	-12 psf
	Internal Pressure	25.85		0.18	4.7 psf	2.8 psf
			11	0 00	20 4	
	(1.0)W = (1.0)(Windward + Leeward Pressure) (0.6)W = (0.6)(Windward + Leeward Pressure)	= 18. = 10	11 pst + 1 86 nef + 1	0.99 pst = 6 59 nef =	29.1 pst 17.5 psf	
	(0.0/11 -(0.0/(Windward + Leeward Plessure)		psi +	0.00 pai -	11.5 psi	
	Parapets: Pp =qp(GCpn) Eqn. 27.3-3	qp	GCpn		(1.0)Pp	(0.6)Pp
	Windward parapet pressure	27.15	1.5	-	40.7 psf	24.4 psf
	Leeward parapet pressure	27.15	-1.0		-27.1 psf	-16.3 pst
	Windward + Leoward Proseuro	07 1E	2 50		67 9 pcf	10.7 -
	Willuwalu + Leewalu Flessule	21.15	2.00		07.9 psi	40.7 pst
	Roof Normal to Ridge (θ≥10 degrees)	qh	GCp	GCpi	(1.0)P	(0.6)P
	Windward Pressure case i	25.85	-0.60	0.18	-20 psf	-12 psf
	case ii	25.85	-0.15	0.18	-8.6 psf	-5.2 psf
	Leeward Pressure	25.85	-0.26	0.18	-11.2 psf	-6.7 psf
	Roof All Other Conditions	ab	GCn	GCni	(1.0)P	(0 6)P
	For 0 to $h/2 = 0$ ft to 16.81 ft	25.85	-0.77	0.18	-24.4 psf	-14.7 pst
	h/2 to h = 16.81 ft to 33.63 ft	25.85	-0.77	0.18	-24.4 psf	-14.7 ps1
	h to 2h = 33.63 ft to 67.25 ft	25.85	-0.43	0.18	-15.6 psf	-9.4 psf
	>2h = >67.25 ft	25.85	-0.26	0.18	-11.2 psf	-6.7 psf
					(4 ···-	<i></i>
	KOOT Overnangs Section 27.3.3	qh	GCp	GCpi	(1.0)P	(0.6)P
	waximum pressures	∠0.85	-0.77	0.00	-37.4 psr	-22.4 pst

	Date 3/10/2022		Sheet		of	
	Job 2220003 Building 2					
	Subject South Wind TOW: 38.980ft					
SCE 7-16 Chapters 26 27 and 30						
,	1. Input					
windward Leeu	Design Parameters					
Pressure Pressure	Basic Wind Speed, V =			109 r	mph (Section 26	6.5, Fig. 1A-2D)
	Exposure Category (B, C, or D) =			C (Section 26.7)	
Roof	Building Risk Category (I, II, III, IV) =			II (Table 1.5-1)	
	Civil finished floor elevation (if unknown input 0) =			0.00 f	eet (Sect. 26.9,	Table 26.9-1)
Wall 🖳 🗜 નြષ્ટ્ર				05.07		
Wea	Eave Height, He =			35.27	eet	
	Parapet Height above ground level Hn =	н, пі –		37.92 I	eel eet	
	Building Width Perpendicular to Wind, B =			540.00 f	eet (max bldg o	lim)
	Building Width Parallel to Wind, L =			210.00 f	eet	,
	Enclosure Classification =	i	Enclosed	Buildings (Section 26.12)	
REFER TO FIGURE 27.3-1	Roof Configuration = Gabled, Hippe	d or Mono	slope Ro	ofs (ø ≤ 7)		
	Angle of Plane of Roof From Horizontal, θ =			1.20	degrees	
	la building og som en bill sides og som en 10				() NI) (C+	- 00 0)
	Is building on or near a nill, ridge, or escarpment?	u -		N ((1 ULIN) (Section	9 Eig 26 9 4)
	Heriz Diet Upwind to Deint Where Elevation = 1/2	-		10.00	eet (Section 26	.u, FIY. 20.8-1)
x (upwind) x	Horiz. Dist. Upwind to Point where Elevation = H/2, Ln	-		10.00	eet (Section 26	.u, FIY. 20.8-1)
	Portz. Dist. Irom Crest to Building Site, X =			10.00	B F or L	.o, rig. 20.8-1)
	2D Ridge, 2D Escarpment, or Axisymmetrical Hill =			E ($(\pi, E, 0, H)$	
	is the building site upwind or downwind of the crest?			DOWN	up, down)	
	2 Calculations - Main Wind Force Resisting System					
	Equivalent Allowable Stress Design Wind Speed Vasd	_		84 43 7	mph (IBC 2018	1609 3 1)
	Moon roof height h =	-		25 27 f	npri (ibo 2010,	1003.3.1)
	K_{z} velocity pressure exposure coefficient at $hz = 37.02$	oft =		1 02 1	Cel Table 26 10-1	(use with az)
	K_{z} velocity pressure exposure coefficient at hz = 37.32	/ft =		1.03	Table 20.10-1	(use with gh)
	K_z , velocity pressure exposure coefficient at $hh = 37.5f$	+ -		1.01	Table 20.10-1	(use with gn)
2 D Dideo er Avis metricel Hill	Kzt topographic factor at hz = 37 92ft =	. –		1.00	Figure 26.8-1	(use with gz)
2-D Ridge of Axisynmetrical Hill	Kzt topographic factor at hb = 35 27ft =			1.00	Figure 26 8-1	(use with gh)
REFER TO FIGURE 26.8-1	Kzt topographic factor at hn = 37 5ft =			1.00	Figure 26 8-1	(use with an)
	Kd wind directionality factor =			0.85	Table 26 6-1	(use min qp)
	Ke, ground elevation factor at			1.00	Table 26.9-1	
	G, gust factor =			0.85 \$	Section 26.11.4	
	qz, velocity pressure at hz = 37.92ft =			26.63 p	osf (Eq. 26.10-1)
	qh, velocity pressure at hh = 35.2/ft =			26.11	ost (Eq. 26.10-1)
	qp, velocity pressure at hp = 57.5it =			20.03	551 (Eq. 20.10-1)
	Walls: P = q(GCpf-GCpi) Eqn. 27.3-1	qz	GCp	GCpi	(1.0)P	(0.6)P
	Windward pressure	26.63	0.68		18.1 psf	10.9 psf
		qh	GCp	GCpi	(1.0)P	(0.6)P
	Leeward Pressure	26.11	-0.43	0.40	-11.1 psf	-6.7 psf
	Sidewall pressure	20.11	-0.60	0.18	-20.2 pst	- 12.1 pst
	internal Flessure	20.11		0.18	4.7 psi	∠.o psi
	(1.0)W = (1.0)(Windward + Leeward Pressure) =	18.	.11 psf +	11.1 psf =	29.2 psf	
	(0.6)W =(0.6)(Windward + Leeward Pressure) =	10.	.86 psf +	6.66 psf =	17.5 psf	
			-	-	-	
	Parapets: Pp =qp(GCpn) Eqn. 27.3-3	qp	GCpn	_	(1.0)Pp	(0.6)Pp
	Windward parapet pressure	26.63	1.5		39.9 psf	24 psf
	Leeward parapet pressure	26.63	-1.0		-26.6 psf	-16 psf
	Windward + Leeward Pressure	26.63	2.50		66.6 psf	39.9 nsf
	timatiana - Essinard Problem	20.00	2.00		poi	53.0 por
	Roof Normal to Ridge (θ≥10 degrees)	qh	GCp	GCpi	(1.0)P	(0.6)P
	Windward Pressure case i	26.11	-0.60	0.18	-20.2 psf	-12.1 psf
	case ii	26.11	-0.15	0.18	-8.7 psf	-5.2 psf
	Leeward Pressure	26.11	-0.26	0.18	-11.4 psf	-6.8 psf
	Boof All Other Conditions	~h	60-	6C~i	(1 0) D	(A 6)D
	For 0 to $h/2 = 0$ ft to 17.64 ft	26 11	-0 77	0.18	(1.0)P -24.7 nef	(U.0)P _14 8 nef
	h/2 to h = 17.64 ft to 35.27 ft	26.11	-0.77	0.18	-24,7 psf	-14.8 psf
	h to 2h = 35.27 ft to 70.54 ft	26.11	-0.43	0.18	-15.8 psf	-9.5 psf
	>2h = >70.54 ft	26.11	-0.26	0.18	-11.4 psf	-6.8 psf
			. ===			
	Roof Overhangs Section 27.3.3	qh	GCp	GCpi	(1.0)P	(0.6)P
	Maximum pressures	26.11	-0.77	0.68	-37.7 psf	-22.6 psf

	Date 3/10/2022		Sheet		of	
	Job 2220003 Building 2					
	Subject South Wind TOW: 38.980ft					
ND ANALYSIS: ANALYTICAL - ALL HEIGHTS METHOD						
CE 7-16, Chapters 26, 27 and 30	4 Innut					
undward Lee.	Design Parameters					
Pressure Pressure	Basic Wind Speed, V =			109 r	mph (Section 26	6.5, Fig. 1A-2D)
	Exposure Category (B, C, or D) =			C (Section 26.7)	
	Building Risk Category (I, II, III, IV) =			II (Table 1.5-1)	
	Civil finished floor elevation (if unknown input 0) =			0.00 f	eet (Sect. 26.9,	Table 26.9-1)
'all 위 기울 - 1월	Eave Height He -			22.62 (oot	
Meeting and a second se	Max Building Height or Ridge Height above ground	evel. Hr =		37.92 f	eet	
	Parapet Height above ground level, Hp =			38.50 f	eet	
	Building Width Perpendicular to Wind, B =			540.00 f	eet (max bldg o	lim)
	Building Width Parallel to Wind, L =		Fueleed	210.00 f	eet	
	Enclosure Classification =	ned or Mond	Enclosed	Buildings (of $a \le 7$)	Section 26.12)	
EFER TO FIGURE 27.3-1	Angle of Plane of Roof From Horizontal, θ =		slope ito	1.20 c	degrees	
	· · · · · · · · · · · · · · · · · · ·				3	
	Is building on or near a hill, ridge, or escarpment?			N ((Y or N) (Sectio	n 26.8)
	Height of Hill or Escarpment relative to upwind terra	n, H =		10.00 f	eet (Section 26	.8, Fig. 26.8-1)
x (upwind) x	Horiz. Dist. Upwind to Point Where Elevation = H/2,	Lh =		10.00 f	eet (Section 26	.8, Fig. 26.8-1)
	Horiz. Dist. from Crest to Building Site, x =			10.00 f	eet (Section 26	.8, Fig. 26.8-1)
► / Q	2D Ridge, 2D Escarpment, or Axisymmetrical Hill =			E ((R, E, or H)	
V(z) I	Is the building site upwind or downwind of the crest?	,		DOWN	up, down)	
	2 Calculations - Main Wind Force Resisting System					
	Equivalent Allowable Stress Design Wind Speed V	ed =		84 43 7	mph (IBC 2018	1609 3 1)
	Mean roof height h =	130 -		33 63 f	ieet	1003.3.1)
	Kz velocity pressure exposure coefficient at bz = 37	92ft =		1 03	Table 26 10-1	(use with az)
	Kz. velocity pressure exposure coefficient at hh = 33	3.63ft =		1.00	Table 26.10-1	(use with ah)
	Kz. velocity pressure exposure coefficient at hp = 38	3.5ft =		1.03	Table 26.10-1	(use with ap)
2-D Ridge or Axisymmetrical Hill	Kzt,topographic factor at hz = 37.92ft =			1.00	Figure 26.8-1	(use with qz)
	Kzt,topographic factor at hh = 33.63ft =			1.00 F	Figure 26.8-1	(use with qh)
FER TO FIGURE 26.8-1	Kzt,topographic factor at hp = 38.5ft =			1.00	Figure 26.8-1	(use with qp)
	Kd, wind directionality factor =			0.85	Table 26.6-1	
	Ke, ground elevation factor at			1.00	Table 26.9-1	
	G, gust factor =			0.85	Section 26.11.4	
	gz, velocity pressure at hz = 37.92ft =			26.63 p	osf (Eq. 26.10-1)
	qh, velocity pressure at hh = 33.63ft =			25.85 p	osf (Eq. 26.10-1)
	qp, velocity pressure at hp = 38.5ft =			26.63 p	osf (Eq. 26.10-1)
	Walls: P = q(GCpf-GCpi) Eqn. 27.3-1	qz	GCp	GCpi	(1.0)P	(0.6)P
	Windward pressure	26.63	0.68	• -	18.1 psf	10.9 psf
		-	• •			
	Looward Prossure	qh	GCp	GCpi	(1.0)P	(0.6)P
	Sidewall pressure	20.85 25.85	-0.43	0.18	-11 psi -20 nef	-0.0 pSI -12 nef
	Internal Pressure	25.85	0.00	0.18	4.7 psf	2.8 psf
				-		
	(1.0)W = (1.0)(Windward + Leeward Pressure) =	18.11 psf + 10.99 psf = 29.1 ps		29.1 psf		
	(0.6)W =(0.6)(Windward + Leeward Pressure) =	10	.86 psf +	6.59 psf =	17.5 psf	
	Parapets: Pn =qn(GCnn) Fgn 27 3-3	an	GCnn		(1.0)Pn	(0.6)Pp
	Windward parapet pressure	26.63	1.5	-	39.9 psf	24 psf
	Leeward parapet pressure	26.63	-1.0		-26.6 psf	-16 psf
			0.50			 -
	winawara + Leeward Pressure	26.63	2.50		66.6 pst	39.9 psf
	Roof Normal to Ridge (θ≥10 degrees)	ah	GCp	GCpi	(1.0)P	(0.6)P
	Windward Pressure case i	25.85	-0.60	0.18	-20 psf	-12 psf
	case ii	25.85	-0.15	0.18	-8.6 psf	-5.2 psf
	Leeward Pressure	25.85	-0.26	0.18	-11.2 psf	-6.7 psf
	Roof All Other Conditions	ah	GCn	GCni	(1.0\P	(0 6)P
	For 0 to $h/2 = 0$ ft to 16.81 ft	25.85	-0.77	0.18	-24.4 psf	-14.7 psf
	h/2 to $h = 16.81$ ft to 33.63 ft	25.85	-0.77	0.18	-24.4 psf	-14.7 psf
	h to 2h = 33.63 ft to 67.25 ft	25.85	-0.43	0.18	-15.6 psf	-9.4 psf
	>2h = >67.25 ft	25.85	-0.26	0.18	-11.2 psf	-6.7 psf
	Deef Overhaume Deef to 07 0.0		00	00.1	(4	(A A) E
	Moor Overnangs Section 27.3.3	25.85	-0 77	0.68	(1.0)P -37.4 pef	(U.6)P _22 A pef
	Maximum pressures	20.00	-0.11	0.00	-31.4 psi	-22.4 psi

Site Class:



Hazards by Location

Search Information

Address:	NE Tudor Rd & NW Main St, Lee's Summit, MO 64086, USA
Coordinates:	38.9307532, -94.3853697
Elevation:	994 ft
Timestamp:	2022-01-05T13:53:22.471Z
Hazard Type:	Seismic
Reference Document:	ASCE7-16
Risk Category:	II



MCER Horizontal Response Spectrum

С



Design Horizontal Response Spectrum



Basic Parameters

Name	Value	Description
S _S	0.099	MCE _R ground motion (period=0.2s)
S ₁	0.068	MCE _R ground motion (period=1.0s)
S _{MS}	0.129	Site-modified spectral acceleration value
S _{M1}	0.102	Site-modified spectral acceleration value
S _{DS}	0.086	Numeric seismic design value at 0.2s SA
S _{D1}	0.068	Numeric seismic design value at 1.0s SA

Additional Information

Name	Value	Description
SDC	В	Seismic design category
Fa	1.3	Site amplification factor at 0.2s
Fv	1.5	Site amplification factor at 1.0s

•	1/6/22, 8:02 AM		ATC Hazards by Location
	CR _S	0.927	Coefficient of risk (0.2s)
	CR ₁	0.877	Coefficient of risk (1.0s)
	PGA	0.047	MCE _G peak ground acceleration
	F _{PGA}	1.3	Site amplification factor at PGA
	PGA _M	0.061	Site modified peak ground acceleration
	ΤL	12	Long-period transition period (s)
	SsRT	0.099	Probabilistic risk-targeted ground motion (0.2s)
	SsUH	0.107	Factored uniform-hazard spectral acceleration (2% probability of exceedance in 50 years)
	SsD	1.5	Factored deterministic acceleration value (0.2s)
	S1RT	0.068	Probabilistic risk-targeted ground motion (1.0s)
	S1UH	0.078	Factored uniform-hazard spectral acceleration (2% probability of exceedance in 50 years)
	S1D	0.6	Factored deterministic acceleration value (1.0s)
	PGAd	0.5	Factored deterministic acceleration value (PGA)

The results indicated here DO NOT reflect any state or local amendments to the values or any delineation lines made during the building code adoption process. Users should confirm any output obtained from this tool with the local Authority Having Jurisdiction before proceeding with design.

Disclaimer

Hazard loads are provided by the U.S. Geological Survey Seismic Design Web Services.

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SEISMIC LOAD SUMMARY 2018 IBC (Ch: 16) and ASCE 7-16 (Ch: 11 to 13)

Date 3/10/2022 Sheet No. of Job 2220003 Building 2 Subject Seismic Design Criteria

1. Input			
Spectral Response Acceleration for Short Periods, Ss = Spectral Response Acceleration for 1-second Periods, S1 =	0.099 0.068		
Risk Category = Site Classification (A,B,C,D,E,F) =	II C	(IBC Table 1604. (ASCE 7 Ch.20 T	5 & ASCE: Table 1-5-1) able 20.3-1)
Basic Structural System BEARING W Lateral Force Resisting System Ordinary Prec	ALL SYSTEMS	(Table 12.2-1) (Table 12 2-1)	
ox redundancy in x-dir = (Redundancy is either 1.0 or 1.3	3) 1.00	(ASCE 7 Section	12.3.4)
ov. redundancy in v-dir.= (Redundancy is either 1.0 or 1.3	3) 1.00	(ASCE 7 Section	12.3.4)
r, =1.0 for Seismic Design Category B and C, RE: ASCE 7 Section 12.3.4.1 for additional exceptions.	,	X -	- /
Is Structure regular with a period < .5 sec?	Yes	(Yes or No, ASCI	E 7 Section 12.8.1.3)
Is Structure short period with a rigid diaphragm?	No	(Yes or No, ASC	E 7 Section 11.6)
Is Structure short period w/ non-rigid diaphragm & vertical elements of seismic force-resisting system spaced at 40' d	oc No	(Yes or No, ASCI	E 7 Section 11.6)
Does Structure have a flexible diaphragm?	Yes	(Yes or No, ASCI	E 7 Section 11.6)
(For Wall anchorage requirements per Section 12.11.2.1)			
Span length of flexible diaphragm -x dir. =	540	feet (input 0 for	rigid diaphragm)
Span length of flexible diaphragm -y-dir. =	210	feet	5 1 5 /
2. Determine Design Spectral Response Accelerations and Seismic Design Category, Section 11.6:		-	
Response Modification Factor, R =	3	(Table 12.2-1)	
Overstrength Factor, $\Omega o =$ (refer to footnote b for .5 reduction for Flexible Diaphragms)	2	(Table 12.2-1)	
Deflection Amplification Factor, Cd =	3	(Table 12.2-1)	
Acceleration for Short Period			
Site Coefficient, Fa =	1.30	(IBC Table 1613.	2.3(1), ASCE 7 Table 11.4-1)
Site Adjusted Spectral Response Acceleration for Short Periods, Sms =	0.129	(IBC Section 161	3.2.3, ASCE 7 Section 11.4.4)
Acceleration for 1-Second Period			
Site Coefficient, Fv =	1.50	(IBC Table 1613.	2.3(2), ASCE 7 Table 11.4-2)
Site Adjusted Spectral Response Acceleration for 1-second Periods, Sm1 =	0.102	(IBC Section 161	3.2.3, ASCE 7 Section 11.4.4)
Design Spectral Response Acceleration for Short Periods, Sds =	0.086	(IBC Section 161	3.2.4 and ASCE 7 Section 11.4.5;
Seismic Design Category based on short period =	Α		
Design Spectral Response Acceleration for 1-second Periods, Sd1 =	0.068	(IBC Section 161	3.2.4 and ASCE 7 Section 11.4.5)
Seismic Design Category based on 1-second period =	В		
Design Response Spectrum, Ts =	0.793	seconds (Section	11.4.6)
Approximate Fundamental Period, Ta =	0.500	seconds (Section	12.8.2.1)
Fundamental Period, T, shall not exceed Ta * Cu =	0.850	seconds (Section	12.8.2)
Can the Seismic Design Category be based on the short period alone?	No	(IBC Section 161	3.2.5.1, ASCE 7 Section 11.6)
Seismic Design Category =	В	(Most severe cas	e except as allowed by Sect 11.6;
3. Seismic Base Shear for the Lateral Force Resisting System using the Equivalent Lateral Force Procedure, Se	ction 12.8:		
a. Calculation of Seismic Base Shear Coefficient:			
Salamia Importance Easter I -	4 00		5.2.)
$Cs = (Sds/(R / I_e)) =$	0.029	(ASCE 7 Table 1 (ASCE Equation	.5-2) 12.8-2, Section 12.8.1.3)
b. Seismic Base Shear, Section 12.8.1:	Strength (1.0E)	ASD (0.7E)	
V = Cs W =	0.029 W	0.020 W	
c. Horizontal Seismic Load. Section 12.4.2.1=	Strength (1 OF)	ASD (0.7F)	
For the X-direction Fh=	0.029 W	0.020 W	
For the Y-direction: Eh=	0.029 W	0.020 W	
d. Vertical Seismic Load Component, Section 12.4.2.2:	0.047 D	0.040 D	
EV = 0.2 Solve D = For structures in SDC B and for the design of foundations using ASD. Ev may be taken as zero. (Section 12.4.2.2)	0.017 D	0.012 D	
e. Find the Design Seismic Shear for the Diaphragm, Section 12.10.1.1:	Strength (1.0E)	ASD (0.7E)	
Force shall not be less than 0.2*Sds*le*wpx =	0.017 W	0.012 W	
hut need not exceed 0.4 *le*Sde wry =	U.U34 VV	U.U24 W	
For a one story building, Fpx =	0.029 W	0.020 W	
6 Executive demonstration Defaulty Decision of the Defaulty Defaulty Defaulty Defaulty Defaulty Defaulty Defaulty			
t. For collector elements in Seismic Design Categories C through F, Section 12.10.2 Emb = Oc V =	0.057 14	0.040.144	
E1111 - 220 V -	0.057 W	0.040 VV	
Natas			

Notes: 1. A building that is low rise (one or two story) building with a short period is assumed for calculation of Seismic Response Coefficient, Cs. 1

2. The values for design spectral response acceleration assume a regular structure of 5 stories or less with a period, T < 0.5 seconds
3. The values for design forces for the diaphragm assume no offsets or changes in the stiffness of the vertical components
4. Section 1613.1 of 2018 IBC excludes the detailing requirements of Chapter 14 of ASCE 7.
5. Per Section 1613.2.2 and 11.4.3, if site investigations performed per ASCE 7 Chpt 20 reveal rock conditions consistent with Site Class B, but site-specific velocity measurements are not made, Fa and Fv shall = 1.0.

1

WALLACE DESIGN PROGRAM							
Revised 12/27/18, Sheila Butcher							
Copyright			0	Date	3/10/2022	Sheet No.	of
			J	lob			
			5	Subject			
SEISMIC LOAD SUMMARY							
2018 IBC (Ch: 16) and ASCE 7-16 (Ch: 11 to 1	3)						
4. Minimum Continuous Load Path, Interconne	ection and Connection to supp	orts, Section 12.1	1.3 and 12	.1.4:			
a. Continuous Load Path and Interconnection	ons, Section 12.1.3:			5	Strength (1.0E) ASD (0.7E)	
$\mathrm{F_{p}}\text{=}$ 0.133 $\mathrm{S}_{\mathrm{ds}}\mathrm{W}_{\mathrm{p}}$ or .05 W_{p} min. =	·				0.050 W	0.035 Wp (Section 12.1.3)
b. Connection to Supports, Section 12.1.4 : F = 05 * dead + live reaction =					0 050 Rd+	0 035 Rd+L (Section 12.1 (1)
r _p loo usuu litereusion					0.000 110		00000112.1.4)
5. Structural Walls and Anchorage, Section 12	11				Strongth (1 0E		
a. Minimum Out-of-Plane Forces on Structur	ral Walls, Section 12.11.1:				Suengui (1.0L) ADD (0.72)	
F_p = 0.40 le Sds wp or .10 wp min =	·				0.100 Wp	o 0.070 Wp (Section 12.11.1)
b. Minimum anchorage connection of struct	ural walls to supporting constr	uction. Section 1	12.11.2.1 a	nd 12.11.2			
g.	Per 12.11.2.2, the strength]					
For loading in the x-direction: $k = 1.0 \pm 1/100$ or max 2.0 =	design force for steel elements with the exception			5	Strength (1.0E) ASD (0.7E)	
$R_a = 1.0 + L_{f} + 100 \text{ of max } 2.0 =$ Fp= 0.4 S _{de} $R_a I_a W_b \text{ or } .2 R_a I_a Wp \text{ min.} =$	of anchor bolts and	2.00 Connections at	Elexible Di	aphragms.	0.400 Wr	0.280 Wp	
$F_p * 1.4$ for steel elements per 12.11.2.2.2	reinforcing steel shall be increased by 1.4 times.			apinagino.	0.560 W	0.392 Wp	
$k_a = 1.0$		_					
$Fp= 0.4 S_{ds} K_a I_e W_p \text{ or } .2 K_a I_e Wp \text{ min.} =$ $Fp * 1.4 \text{ for steel elements per 12 11 2 2 2}$	For C	onnections not at	Flexible Di	aphragms:	0.200 Wp	0.140 Wp	
					0.200 11	0.100 110	
For loading in the y-direction: $k = 1.0 \pm 1./100$ or max 2.0 =		2.00					
$R_a = 1.0 + L_{f} + 100 \text{ of max 2.0} =$ Fp= 0.4 S _{ds} $R_a I_e W_p \text{ or .2} R_a I_e Wp \text{ min.} =$	Fo	or Connections at	Flexible Dia	aphragms:	0.400 Wr	0.280 Wp	
Fp * 1.4 for steel elements per 12.11.2.2.2					0.560 W	0.392 Wp	
к _a = 1.0 Fp= 0.4 S _{da} k _a I _a W _a or .2 k _a I _a Wp min. =	For C	onnections not at	Elexible Di	aphragms.	0.200 Wr	0.140 Wp	
Fp * 1.4 for steel elements per 12.11.2.2.2		ermeenene net ut		apinagino	0.280 W	0.196 Wp	
The minimum well enchances load for each	ata ar maganny walla ia 0.2* th	wall waight or F	nof nor 4				
The minimum wan anchorage load for conci-	ete of masoning waits is 0.2 the	wan weight of 5	parper i.				
6. Horizontal Seismic Design Force on Nonstr	uctural Architectural Compone	nts, Section 13.3	3:		Evela 40		
Fp max = 1.6 Sds lp Wp=					For ip =1.0 0.137 Wr	• 0.206 Wp (I	Equation 13.3-2)
Fp min = 0.3 Sds Ip Wp=					0.026 Wp	0.039 Wp (Equation 13.3-3)
The Seismic Design Force is based on Equat	ion 13 3-1 with the minimum and	l maximum limits r	noted above	e			
Fp= 0.4 ap Sds Wp $(1 + 2 z/h)/(Rp/lp)$				0.			
Solomia Docian Force Summary on Architec	tural Componente Section 12	- .					
Seisinic Design Force Summary on Architec	tural components, Section 13.	ap=	Rp=	lp=	z/h=	Strength (1.0E)	ASD (0.7E)
1. Cantilevered (Unbraced) Parapets and Chim	neys	2.50	2.50	1.00	1.00	0.103 Wp	0.072 Wp (Table 13.5-1)
2 Braced Interior Non-masonry walls and partit	lions						
Fp at floor=		1.00	2.50	1.00	0.00	0.026 Wp	0.018 Wp (Table 13.5-1)
Fp at roof=		1.00	2.50	1.00	1.00	0.041 Wp	0.029 Wp (Table 13.5-1)
rp average at roor and hoor.						0.033 WP	0.023 WP
3. Braced Interior Unreinforced masonry walls a	and partitions						
Fp at floor= Fp at roof=		1.00 1.00	1.50 1.50	1.00 1.00	0.00	0.026 Wp 0.069 Wp	0.018 Wp (Table 13.5-1) 0.048 Wp (Table 13.5-1)
Fp average at roof and floor:		1.00	1.00	1.00		0.047 Wp	0.033 Wp
	alalla	2 50	2 50	4.00	0.00	0.024.18/-	
4. Cantilevered (Onbraced) Intenor Nonstructur		2.50	2.50	1.00	0.00	0.034 WP	0.024 WP (Table 13.5-1)
5. Braced Parapets and Chimneys		1.00	2.50	1.00	1.00	0.041 Wp	0.029 Wp (Table 13.5-1)
6. Exterior Nonstructural Wall Elements							
Fp at floor=		1.00	2.50	1.00	0.00	0.026 Wp	0.018 Wp (Table 13.5-1)
Fp at roof= En average at roof and floor:		1.00	2.50	1.00	1.00	0.041 Wp 0.033 Wp	0.029 Wp (Table 13.5-1)
. p atotago at roor and noor.						5.000 WP	0.020 110
For the Body of the Wall Panel Connection:		4.00	0.50	4.00		0.000.10/-	
Fp at noor= Fp at roof=		1.00 1.00	2.50 2.50	1.00 1.00	0.00 1.00	0.026 Wp 0.041 Wp	0.018 Wp (Table 13.5-1) 0.029 Wp (Table 13.5-1)
For the fasteners of the connecting system: En at floor=		1 25	1 00	1 00	0.00	0.043 Wp	0 030 Wp (Table 13 5 1)
Fp at roof=		1.25	1.00	1.00	1.00	0.129 Wp	0.090 Wp (Table 13.5-1)
7 Annondoges and One sector		0.50	2.52	4.00	4.00	0 400 144	0.070 M/m (T) / 10.5
r. Appendages and Ornamentation		2.50	2.50	1.00	1.00	0.103 Wp	U.U/2 WP (Table 13.5-1)

Notes:

Refer to Section 13.4.2 for additional requirements for anchors in concrete and masonry.
 Section 1613.1 of 2018 IBC excludes the detailing requirements of Chapter 14 of ASCE 7.

Date	3/10/2022	Sheet No.	of	
Job				
Subject				

SEISMIC LOAD SUMMARY

2018 IBC (Ch: 16) and ASCE 7-16 (Ch: 11 to 13)

Table 11.4-1 and IBC 1613.2.3(1)

Site Coefficient, Fa

		Sile Co	enicient, Fa				
Site	Mapped Spe	ectral Respo	nse Accelerat	ion at Short P	eriods (Ss)		Distance
Class	Ss<=0.25	0.5	0.75	1	1.25	Ss>=1.5	Value
Α	0.80	0.80	0.80	0.80	0.80	0.80	0.80
В	0.90	0.90	0.90	0.90	0.90	0.90	0.90
С	1.30	1.30	1.20	1.20	1.20	1.20	1.30
D	1.60	1.40	1.20	1.10	1.00	1.00	1.60
Е	2.40	1.70	1.30	1.20	1.20	1.20	2.40
F							

Minimum of 1.2 per Section 11.4.4 considered. Exceptions per Section 11.4.8 included.

Table 11.4-2 and IBC 1613.2.3(2) Site Coefficient Ev

Site	Mapped Spec	tral Respons	se Acceleratio	n at 1 Second	d Period (S1)		Distance
Class	S1<=0.1	0.2	0.3	0.4	0.5	S1>=0.6	Value
Α	0.80	0.80	0.80	0.80	0.80	0.80	0.80
В	0.80	0.80	0.80	0.80	0.80	0.80	0.80
С	1.50	1.50	1.50	1.50	1.50	1.40	1.50
D	2.40	2.20	2.00	1.90	1.80	1.70	2.40
Е	4.20	3.30	2.80	2.40	2.20	2.00	4.20
F							

IBC Table 1613.2.5(1) and 11.6-1 eismic Design Category based on Short Period Response Acceleratic

Value of	Occ	upancy Cate	gory	Design
Sds	l or ll	III	IV	Category Category
Sds <= 0.167	А	А	Α	A A
0.167 <= Sds < 0.33	В	В	С	В
0.33 <= Sds < 0.5	С	С	D	С
0.5 <= Sds	D	D	D	D
S1 >= 0.75	E	E	F	E

Table 1613.2.5(2) and 11.6-2

Seismic Design Category Based on 1-Second Period Response Acceleration

Value of	Occ	upancy Cate	gory	Design
Sd1	l or ll	III	IV	Category Category
Sd1 <= 0.067	А	А	А	A B
0.067 <= Sd1 < 0.133	В	В	С	В
0.133 <= Sd1 < 0.2	С	С	D	С
0.2 <= Sd1	D	D	D	D
S1 >= 0.75	Е	E	F	E

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Date	3/10/2022	Sheet No.	of	
Project	2220003			
Subject	bject Snow Drift East/West - 4.93ft Parapet			

FLAT ROOF SNOW DRIFT - Joists Parallel to Wall

ASCE 7-16



Configuration

Note: For projections, input width of projection for Wb1 and input maximum of leeward and windward drift length for Wb2. For parapets, input the parapet width as Wb1.



1. Input

1. input			
Dead Load = Roof Live Load = Pg, Ground Snow Load = Drift for parapet, projection, or Risk Categoy (I, II, III, or IV) = Ce, Exposure Factor = Ct, Thermal Factor = Use Pg minimum for drift calc's	upper roof? s (Pf = Pg)?	10 20 9 11 1.0 1.0 N	psf psf (P), (PR) or (U) Table 1.5-1 Table 7.3-1 Table 7.3-2 (Y or N)
Geometry T.O.W., Top of Parapet Elevati J.B.E., Joist Bearing Elevation td, Thickness of Joist, Deck, ar Wb1, length of upper roof = Wb2, length of lower roof = x, Joist #1 dist. from wall = S1, First Joist Spacing = S2, Second Joist Spacing = S3, Third Joist Spacing = S4, Fourth Joist Spacing = S5, Fifth Joist Spacing =	on = = nd Insulation =	40.00 34.42 4.00 0.80 540.00 6.00 6.00 6.00 6.00 6.00 6.00	feet feet feet feet feet feet feet feet
2. Balanced Snow Load Check			
Is, Importance Factor = Pf = 0.7 Ce Ct Is Pg = Pm = Is Pg = Rain on snow surcharge = Pmin =		1.0 14.00 20.00 5.00 20.00	Table 1.5-2 psf (7.3-1) psf (7.3.4) psf (7.10) psf
3. Drifted Snow Load Check			
$\begin{array}{l} Pf = 0.7 \ Ce \ Ct \ Is \ Pg = \\ D = 0.13 \ Pg + 14.0 \leq 30 \ pcf = \\ hb = Pf/D = \\ Wb = \\ hd = 0.75[0.43 \ Wb2^{1/3} \ (Pg+1) \\ hd + hb = \\ hr = \\ hc = hr - hb = \\ Wd = 4 \ hd \ or \ 4 \ [\ hd^{2/hc}] \leq 8 \ H \\ Pmax = D \ (hd + hb) \leq D \ hr = \\ Pd = D \ hd \leq D \ hc = \end{array}$	0)^1/4-1.5] ls^1. nc =	14.00 16.60 0.84 540.00 5.01 5.85 5.25 4.40 22.79 87.09 73.09	psf (7.3-1) pcf (7.7-1) feet feet feet (Fig. 7.6-1) feet feet feet feet psf psf
4. Uniform Load Summary			
Drifted Snow Load w, wall w, Joist #1 w, Joist #2 w, Joist #3 w, Joist #4 w, Joist #5	Snow 246.8 407.1 291.6 176.1 101.2 84.0	Total 276.8 467.1 351.6 236.1 161.2 144.0	plf * plf * plf * plf * plf plf
Balanced Load Check			
w, wall w, Joist #1 w, Joist #2 w, Joist #3 w, Joist #4 w, Joist #5	Pmin (20 psf) 60.0 120.0 120.0 120.0 120.0 120.0 120.0	Total 90.0 180.0 180.0 180.0 180.0 180.0	plf plf plf plf plf * plf *

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Date	3/10/2022	Sheet No.	of
Project	2220003		
Subject	North Drift 40'-0"		

FLAT ROOF SNOW DRIFT - Joists Perpendicular to Wall

ASCE 7-16



Configuration

Note: For projections, input width of projection for Wb1 and input maximum of leeward and windward drift length for Wb2. For parapets, input the parapet width as Wb1.





1. mput

Dead Load = Roof Live Load =	10 20	psf psf
Pg, Ground Snow Load =	20 D	(P) (PR) or (U)
Risk Categoy (I, II, III, or IV) =		Table 1.5-1
Ce, Exposure Factor =	1.00	Table 7.3-1
Ct, Thermal Factor =	1.00	Table 7.3-2
Use Pg minimum for drift calc's (Pf = Pg)?	N	(Y or N)
Geometry		
T.O.W., Top of Parapet Elevation =	40.00	feet
J.B.E., Joist Bearing Elevation =	37.58	feet
td, Thickness of Joist, Deck, and Insulation =	4.00	inches
Wb1, length of upper roof =	0.67	feet
Wb2, length of lower roof =	210.00	feet
S, Joist Spacing =	6.00	feet

2. Balanced Snow Load Check

L, Joist Span =

ls, Importance Factor =	1.00 Table 1.5-2
Pf = 0.7 Ce Ct Is Pg =	14.00 psf (7.3-1)
Pm = Is Pg =	20.00 psf (7.3.4)
Rain on snow surcharge =	5.00 psf (7.10)
Pmin =	20.00 psf

50.00 feet

3. Drifted Snow Load Check

Pf = 0.7 Ce Ct Is Pg =	14.00 psf
D = 0.13 Pg + 14.0 ≤ 30 pcf =	16.60 pcf
hb = Pf/D =	0.84 ft
Wb =	210.00 ft
hd = 0.75[0.43 Wb2^1/3 (Pg+10)^1/4-1.5] Is^2	3.35 ft
hd + hb =	4.20 ft
hr =	2.09 ft
hc = hr - hb =	1.24 ft
Wd = 4 hd or 4 [hd^2/hc] ≤ 8 hc =	9.95 ft
$Pmax = D (hd + hb) \le D hr =$	34.64 psf
Pd =D hd ≤ D hc =	20.64 psf

4. Uniform Load Summary

Drifted Snow Load

1

	Snow	Total
R left =	2675.0	4175.0 lbs
R right =	2140.8	3640.8 lbs
M max =	27280.8	46026.7 ft-lbs
w base =	84.0	144.0 plf
w drift =	123.8	183.8 plf
w equiv =	107.0	167.0 plf
Load Without Drift		
	Live	Total
w (Live = 20 psf) =	120.0	180.0 plf *

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Date	3/10/2022	Sheet No.	of
Project	2220003		
Subject	North Drift 40.542ft wall		

FLAT ROOF SNOW DRIFT - Joists Perpendicular to Wall

ASCE 7-16



Configuration

Note: For projections, input width of projection for Wb1 and input maximum of leeward and windward drift length for Wb2. For parapets, input the parapet width as Wb1.





1. mput

Dead Load = Boof Live Load =	10 20	psf psf
Pg. Ground Snow Load =	20	psf
Drift for parapet, projection, or upper roof?	 P	(P), (PR) or (U)
Risk Categoy (I, II, III, or IV) =	11	Table 1.5-1
Ce, Exposure Factor =	1.00	Table 7.3-1
Ct, Thermal Factor =	1.00	Table 7.3-2
Use Pg minimum for drift calc's (Pf = Pg)?	N	(Y or N)
Geometry		
	40.00	f
	42.00	leel
J.B.E., Joist Bearing Elevation =	37.58	feet
td, Thickness of Joist, Deck, and Insulation =	4.00	inches
Wb1, length of upper roof =	0.67	feet
Wb2, length of lower roof =	210.00	feet
S, Joist Spacing =	6.00	feet
L, Joist Span =	50.00	feet

2. Balanced Snow Load Check

ls, Importance Factor =	1.00 Table 1.5-2
Pf = 0.7 Ce Ct Is Pg =	14.00 psf (7.3-1)
Pm = Is Pg =	20.00 psf (7.3.4)
Rain on snow surcharge =	5.00 psf (7.10)
Pmin =	20.00 psf

3. Drifted Snow Load Check

Pf = 0.7 Ce Ct Is Pg =	14.00 psf
D = 0.13 Pg + 14.0 ≤ 30 pcf =	16.60 pcf
hb = Pf/D =	0.84 ft
Wb =	210.00 ft
hd = 0.75[0.43 Wb2^1/3 (Pg+10)^1/4-1.5] Is^2	3.35 ft
hd + hb =	4.20 ft
hr =	4.09 ft
hc = hr - hb =	3.24 ft
Wd = 4 hd or 4 [hd^2/hc] ≤ 8 hc =	13.87 ft
$Pmax = D (hd + hb) \le D hr =$	67.84 psf
Pd =D hd ≤ D hc =	53.84 psf

4. Uniform Load Summary

Drifted Snow Load

	Snow	Total
R left =	4132.8	5632.8 lbs
R right =	2307.1	3807.1 lbs
M max =	31682.3	50325.9 ft-lbs
w base =	84.0	144.0 plf
w drift =	323.0	383.0 plf
w equiv =	165.3	225.3 plf *
Load Without Drift		
	Live	Total

w (Live = 20 psf) = **120.0 180.0** plf

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Date	3/10/2022	Sheet No.	of
Project	2220003		
Subject	South Drift 37.542ft wall		

FLAT ROOF SNOW DRIFT - Joists Perpendicular to Wall

ASCE 7-16



Configuration

Note: For projections, input width of projection for Wb1 and input maximum of leeward and windward drift length for Wb2. For parapets, input the parapet width as Wb1.





4	1	
	IN	DUI
		_

Dead Load = Roof Live Load =	10 20	psf psf
Pg, Ground Snow Load =	20	, psf
Drift for parapet, projection, or upper roof?	Р	(P), (PR) or (U)
Risk Categoy (I, II, III, or IV) =	II	Table 1.5-1
Ce, Exposure Factor =	1.00	Table 7.3-1
Ct, Thermal Factor =	1.00	Table 7.3-2
Use Pg minimum for drift calc's (Pf = Pg)?	N	(Y or N)
Geometry		
T O W Top of Parapet Elevation =	37 50	feet
J.B.E., Joist Bearing Elevation =	34.42	feet
td, Thickness of Joist, Deck, and Insulation =	4.00	inches
Wb1, length of upper roof =	0.67	feet
Wb2, length of lower roof =	210.00	feet
S, Joist Spacing =	6.00	feet
L, Joist Span =	50.00	feet

2. Balanced Snow Load Check

ls, Importance Factor =	1.00 Table 1.5-2
Pf = 0.7 Ce Ct Is Pg =	14.00 psf (7.3-1)
Pm = Is Pg =	20.00 psf (7.3.4)
Rain on snow surcharge =	5.00 psf (7.10)
Pmin =	20.00 psf

3. Drifted Snow Load Check

Pf = 0.7 Ce Ct Is Pg =	14.00 psf
D = 0.13 Pg + 14.0 ≤ 30 pcf =	16.60 pcf
hb = Pf/D =	0.84 ft
Wb =	210.00 ft
hd = 0.75[0.43 Wb2^1/3 (Pg+10)^1/4-1.5] Is^2	3.35 ft
hd + hb =	4.20 ft
hr =	2.75 ft
hc = hr - hb =	1.90 ft
Wd = 4 hd or 4 [hd^2/hc] ≤ 8 hc =	15.23 ft
$Pmax = D (hd + hb) \le D hr =$	45.59 psf
Pd =D hd ≤ D hc =	31.59 psf

4. Uniform Load Summary

Drifted Snow Load

	Snow	Total
R left =	3396.7	4896.7 lbs
R right =	2246.5	3746.5 lbs
M max =	30040.2	48737.0 ft-lbs
w base =	84.0	144.0 plf
w drift =	189.6	249.6 plf
w equiv =	135.9	195.9 plf *
Load Without Drift		
	Live	Total

	LIVO	1 Ottai
w (Live = 20 psf) =	120.0	180.0 plf
1		

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Date	3/10/2022	Sheet No.	of
Project	2220003		
Subject	South Drift 37.542ft wall		

FLAT ROOF SNOW DRIFT - Joists Perpendicular to Wall

ASCE 7-16



Configuration

Note: For projections, input width of projection for Wb1 and input maximum of leeward and windward drift length for Wb2. For parapets, input the parapet width as Wb1.





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Dead Load = Roof Live Load = Pq, Ground Snow Load =	10 20 20	psf psf psf
Drift for parapet, projection, or upper roof?	Р	(P), (PR) or (U)
Risk Categoy (I, II, III, or IV) =	II	Table 1.5-1
Ce, Exposure Factor =	1.00	Table 7.3-1
Ct, Thermal Factor =	1.00	Table 7.3-2
Use Pg minimum for drift calc's (Pf = Pg)?	N	(Y or N)
Geometry	07 50	c .
I.O.W., Top of Parapet Elevation =	37.50	feet
J.B.E., Joist Bearing Elevation =	33.29	feet
td, Thickness of Joist, Deck, and Insulation =	4.00	inches
Wb1, length of upper roof =	0.67	feet
Wb2, length of lower roof =	210.00	feet
S, Joist Spacing =	6.00	feet
L, Joist Span =	60.00	feet

2. Balanced Snow Load Check

ls, Importance Factor =	1.00 Table 1.5-2
Pf = 0.7 Ce Ct Is Pg =	14.00 psf (7.3-1)
Pm = Is Pg =	20.00 psf (7.3.4)
Rain on snow surcharge =	5.00 psf (7.10)
Pmin =	20.00 psf

3. Drifted Snow Load Check

Pf = 0.7 Ce Ct Is Pg =	14.00 psf
D = 0.13 Pg + 14.0 ≤ 30 pcf =	16.60 pcf
hb = Pf/D =	0.84 ft
Wb =	210.00 ft
hd = 0.75[0.43 Wb2^1/3 (Pg+10)^1/4-1.5] Is^2	3.35 ft
hd + hb =	4.20 ft
hr =	3.88 ft
hc = hr - hb =	3.03 ft
Wd = 4 hd or 4 [hd^2/hc] ≤ 8 hc =	14.83 ft
$Pmax = D (hd + hb) \le D hr =$	64.35 psf
Pd =D hd ≤ D hc =	50.35 psf

4. Uniform Load Summary

Drifted Snow Load

	Snow	Total
R left =	4575.4	6375.4 lbs
R right =	2704.5	4504.5 lbs
M max =	43538.1	70453.6 ft-lbs
w base =	84.0	144.0 plf
w drift =	302.1	362.1 plf
w equiv =	152.5	212.5 plf *
Load Without Drift		
	Live	Total

w (Live = 20 psf) = **120.0 180.0** plf

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Date	3/10/2022	Sheet No.	of	
Project	2220003			
Subject	South Drift 38.98ft wall			

FLAT ROOF SNOW DRIFT - Joists Perpendicular to Wall

ASCE 7-16



Configuration

Note: For projections, input width of projection for Wb1 and input maximum of leeward and windward drift length for Wb2. For parapets, input the parapet width as Wb1.





1. mput

Dead Load =	10	psf
Roof Live Load =	20	psf
Pg, Ground Snow Load =	20	psf
Drift for parapet, projection, or upper roof?	Р	(P), (PR) or (U)
Risk Categoy (I, II, III, or IV) =	II	Table 1.5-1
Ce, Exposure Factor =	1.00	Table 7.3-1
Ct, Thermal Factor =	1.00	Table 7.3-2
Use Pg minimum for drift calc's (Pf = Pg)?	N	(Y or N)
Geometry		
T.O.W., Top of Parapet Elevation =	38.50	feet
J.B.E., Joist Bearing Elevation =	34.42	feet
td, Thickness of Joist, Deck, and Insulation =	4.00	inches
Wb1, length of upper roof =	0.67	feet
Wb2, length of lower roof =	210.00	feet
S, Joist Spacing =	6.00	feet
L, Joist Span =	50.00	feet

2. Balanced Snow Load Check

ls, Importance Factor =	1.00 Table 1.5-2
Pf = 0.7 Ce Ct Is Pg =	14.00 psf (7.3-1)
Pm = Is Pg =	20.00 psf (7.3.4)
Rain on snow surcharge =	5.00 psf (7.10)
Pmin =	20.00 psf

3. Drifted Snow Load Check

Pf = 0.7 Ce Ct Is Pg =	14.00 psf
D = 0.13 Pg + 14.0 ≤ 30 pcf =	16.60 pcf
hb = Pf/D =	0.84 ft
Wb =	210.00 ft
hd = 0.75[0.43 Wb2^1/3 (Pg+10)^1/4-1.5] Is^2	3.35 ft
hd + hb =	4.20 ft
hr =	3.75 ft
hc = hr - hb =	2.90 ft
Wd = 4 hd or 4 [hd^2/hc] ≤ 8 hc =	15.49 ft
$Pmax = D (hd + hb) \le D hr =$	62.19 psf
Pd =D hd ≤ D hc =	48.19 psf

4. Uniform Load Summary

Drifted Snow Load

	Snow	Total
R left =	4108.5	5608.5 lbs
R right =	2331.3	3831.3 lbs
M max =	32351.8	50969.1 ft-lbs
w base =	84.0	144.0 plf
w drift =	289.2	349.2 plf
w equiv =	164.3	224.3 plf *
Load Without Drift		
	Live	Total

w (Live = 20 psf) = **120.0 180.0** plf

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Date	3/10/2022	Sheet No.	of	
Project	2220003			
Subject	South Drift 38.98ft wall			

FLAT ROOF SNOW DRIFT - Joists Perpendicular to Wall

ASCE 7-16



Configuration

Note: For projections, input width of projection for Wb1 and input maximum of leeward and windward drift length for Wb2. For parapets, input the parapet width as Wb1.





1. mput

Dead Load =	10	psf
Roof Live Load =	20	pst
Pg, Ground Snow Load =	20	psf
Drift for parapet, projection, or upper roof?	Р	(P), (PR) or (U)
Risk Categoy (I, II, III, or IV) =	II	Table 1.5-1
Ce, Exposure Factor =	1.00	Table 7.3-1
Ct, Thermal Factor =	1.00	Table 7.3-2
Use Pg minimum for drift calc's (Pf = Pg)?	N	(Y or N)
Geometry		
T.O.W., Top of Parapet Elevation =	38.50	feet
J.B.E., Joist Bearing Elevation =	33.29	feet
td, Thickness of Joist, Deck, and Insulation =	4.00	inches
Wb1, length of upper roof =	0.67	feet
Wb2, length of lower roof =	210.00	feet
S, Joist Spacing =	6.00	feet
L, Joist Span =	50.00	feet

2. Balanced Snow Load Check

ls, Importance Factor =	1.00 Table 1.5-2
Pf = 0.7 Ce Ct Is Pg =	14.00 psf (7.3-1)
Pm = Is Pg =	20.00 psf (7.3.4)
Rain on snow surcharge =	5.00 psf (7.10)
Pmin =	20.00 psf

3. Drifted Snow Load Check

Pf = 0.7 Ce Ct Is Pg =	14.00 psf
D = 0.13 Pg + 14.0 ≤ 30 pcf =	16.60 pcf
hb = Pf/D =	0.84 ft
Wb =	210.00 ft
hd = 0.75[0.43 Wb2^1/3 (Pg+10)^1/4-1.5] Is^2	3.35 ft
hd + hb =	4.20 ft
hr =	4.88 ft
hc = hr - hb =	4.03 ft
Wd = 4 hd or 4 [hd^2/hc] ≤ 8 hc =	13.41 ft
$Pmax = D (hd + hb) \le D hr =$	69.66 psf
Pd =D hd ≤ D hc =	55.66 psf

4. Uniform Load Summary

Drifted Snow Load

	Snow	Total
R left =	4139.6	5639.6 lbs
R right =	2300.3	3800.3 lbs
M max =	31496.0	50146.5 ft-lbs
w base =	84.0	144.0 plf
w drift =	334.0	394.0 plf
w equiv =	165.6	225.6 plf *
Load Without Drift		
	Live	Total

w (Live = 20 psf) = **120.0 180.0** plf



<u>Date</u>	03/10/2022
Job	2220003 Building 2
<u>Subject</u>	MWFRS Analysis

BUILDING GEOMETRY

Building Width	<i>W</i> := 540 <i>ft</i>	
Building Depth	<i>D</i> ≔ 210 <i>ft</i> + 10 <i>in</i>	
	North Elevation	South Elevation
Bottom of Deck	<i>BOD_N</i> := 37 <i>ft</i> + 11 <i>in</i>	BOD _S ≔ 33 <i>ft</i> + 7.5 <i>in</i>
Bottom of Wall	$BOW_N := 0 ft$	BOW _S := -4 ft
Top of Wall (Max)	<i>TOW_{N_MAX}</i> := 42 <i>ft</i>	<i>TOW</i> _{S_MAX} := 38.5 <i>ft</i>
	East Elevation	West Elevation
Bottom of Deck - Eave	$BOD_{EE} := BOD_S$	$BOD_{WE} \coloneqq BOD_S$
Bottom of Deck - Ridge	$BOD_{ER} \coloneqq BOD_N$	$BOD_{WR} := BOD_N$
Bottom of Wall	<i>BOW_E</i> := 0 <i>ft</i>	<i>BOW_W</i> :=0 <i>ft</i>
Top of Wall (Max)	<i>TOW_{E_MAX}</i> := 42 <i>ft</i>	<i>TOW_{W_MAX}</i> := 42 <i>ft</i>

<u>MWFRS - NORTH/SOUTH</u>

ROOF REACTIONS (1.0W, LRFD)		
FOR NORTH WALL	$w_w := 29.1 \ psf$ $w_p := 67.9 \ psf$	
Typical Wall	$parapet := 40 \ ft - BOD_N = 2.08 \ ft$	
	$R_{N_typ} := w_w \cdot \frac{(BOD_N - BOW_N)}{2} + w_p \cdot parapet = 693.15 \text{ plf}$	
Max. Parapet	$parapet := TOW_{N_MAX} - BOD_N = 4.08 \ ft$	
	$R_{N_max} \coloneqq w_w \cdot \frac{(BOD_N - BOW_N)}{2} + w_p \cdot parapet = 828.95 \text{ plf}$	
FOR SOUTH WALL	$w_w := 29.1 \ psf$ $w_p := 66.6 \ psf$	
Typical Wall	parapet := 37.5 <i>ft</i> - BOD _S = 3.88 <i>ft</i>	
	$R_{S_{typ}} := w_w \cdot \frac{(BOD_S - BOW_S)}{2} + w_p \cdot parapet = 805.52 \text{ plf}$	
Corner Wall	parapet := TOW _{S_MAX} - BOD _S = 4.88 ft	
	$R_{S_typ} := w_w \cdot \frac{(BOD_S - 0 ft)}{2} + w_p \cdot parapet = 813.92 plf$	

SOUTH EXPOSURE CONTROLS ROOF REACTION



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<u>Subject</u>	MWFRS Analysis

DIAPHRAGM FORCES

GRID A-L



Reactions	<i>R</i> _{<i>u1</i>} := 217.7 <i>kip</i>	(LRFD)	R _{u2} :=217.7 kip	(LRFD)
	$R_1 \coloneqq 0.6 \cdot R_{u1} = 130.62 \ kip$	(ASD)	$R_2 := 0.6 \cdot R_{u2} = 130.62 \ kip$	(ASD)
	<i>M_{u1}</i> := 29379.3 <i>kip</i> ⋅ <i>ft</i>	(LRFD)	M ₁ := 17627.6 <i>kip • ft</i>	(ASD)

Chord Force
$$P_{u1} := \frac{M_{u1}}{D} = 139.35 \text{ kip}$$
 (LRFD) $P_1 := 0.6 \cdot P_{u1} = 83.61 \text{ kip}$ (ASD)

DIAPHRAGM SHEAR

Grid A	$v_u := \frac{R_{u1}}{D} = 1032.57$ plf	(LRFD)	$v := \frac{R_1}{D} = 619.54 \ plf$	(ASD)
Grid L	$v_u := \frac{R_{u2}}{D} = 1032.57 \text{ plf}$	(LRFD)	$v := \frac{R_2}{D} = 619.54 \ plf$	(ASD)



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MWFRS - EAST/WEST

ROOF REACTIONS (1.0W, LRFD)

	$w_w := 24.1 \text{ psf}$ $w_\rho := 67.9 \text{ psf}$
Roof Slope	$m \coloneqq \frac{BOD_{ER} - BOD_{EE}}{D} = 0.02 \qquad \qquad \theta \coloneqq \operatorname{atan}(m) = 1.17 \text{ deg}$
EVALUATE STEPPED WAL	L PRESSURES FROM EAVE END
Typical Wall	$TOW_{typ} := 40 \ ft$
Wall Panel 1	$L_1 := 54 \ \text{ft} + 10 \ \text{in} H := 38.5 \ \text{ft}$
Start	$BOD_i := BOD_{EE}$
	$R_i \coloneqq w_w \cdot \frac{(BOD_i - BOW_E)}{2} + w_p \cdot (H - BOD_i) = 736.19 \text{ plf}$
End	<i>BOD_j</i> := 34.75 <i>ft</i>
	$R_j \coloneqq w_w \cdot \frac{(BOD_j - BOW_E)}{2} + w_p \cdot (H - BOD_j) = 673.36 \text{ plf}$
Wall Panel 2	$L_2 := 96 \ ft \qquad \qquad H := TOW_{typ} = 40 \ ft$
Start	<i>BOD</i> _i := 34.75 <i>ft</i>
	$R_i \coloneqq w_w \cdot \frac{(BOD_i - BOW_E)}{2} + w_p \cdot (H - BOD_i) = 775.21 \text{ plf}$
End	$BOD_j := 34.75 \ ft + m \cdot L_2 = 36.7 \ ft$
	$R_j \coloneqq w_w \cdot \frac{(BOD_j - BOW_E)}{2} + w_p \cdot (H - BOD_j) = 666.07 \text{ plf}$
Wall Panel 3	$L_3 := 60 \ \text{ft}$ $H := TOW_{E_MAX} = 42 \ \text{ft}$
Start	$BOD_i := 34.75 \ \text{ft} + m \cdot L_2 = 36.7 \ \text{ft}$
	$R_i \coloneqq w_w \cdot \frac{(BOD_i - BOW_E)}{2} + w_p \cdot (H - BOD_i) = 801.87 \text{ plf}$
End	$BOD_j := BOD_{ER} = 37.92$ ft
	$R_j \coloneqq w_w \cdot \frac{(BOD_j - BOW_E)}{2} + w_p \cdot (H - BOD_j) = 734.15 \text{ plf}$



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<u>Subject</u>	MWFRS Analysis

DIAPHRAGM FORCES

GRID 1-5



Reactions

$$R_{u1} = 76.2 \text{ kip}$$
 (LRFD)
 $R_{u2} = 77.7 \text{ kip}$
 (LRFD)

$$R_1 := 0.6 \cdot R_{u1} = 45.72 \text{ kip}$$
 (ASD) $R_2 := 0.6 \cdot R_{u2} = 46.62 \text{ kip}$ (ASD)

$$M_{u1} := 4033.8 \ kip \cdot ft$$
 (LRFD) $M_1 := 0.6 \cdot M_{u1} = 2420.28 \ kip \cdot ft$ (ASD)

Chord Force
$$P_{u1} := \frac{M_{u1}}{W} = 7.47 \text{ kip}$$
 (LRFD) $P_1 := 0.6 \cdot P_{u1} = 4.48 \text{ kip}$ (ASD)

DIAPHRAGM SHEAR

Grid 1	$v_u := \frac{R_{u2}}{W} = 143.89 \text{ plf}$	(LRFD)	$v \coloneqq \frac{R_2}{W} = 86.33 \text{ plf}$	(ASD)
Grid 5	$v_u := \frac{R_{u1}}{W} = 141.11 \ plf$	(LRFD)	$v \coloneqq \frac{R_1}{W} = 84.67 \text{ plf}$	(ASD)



Date	03/10/2022
Job	2220003 Building 2
<u>Subject</u>	Seismic Load

CHECK FOR TOTAL SEISMIC FORCE

		OL 14/11 · ·
<u>SDC C</u> by ASCE 7-10,	Intermediate Precast	Shear Walls required

1.0E = 0.029W	$C_s := 0.029$	(LRFD)	
0.7E = 0.020W	$C_s := 0.020$	(ASD)	
ASSUME PRECAST CONCRETE WALL	t:=9.25 <i>in</i> w _w :=150 pcf ⋅t	= 115.63 psf	

	Average Wall Height	Average Tributary
NORTH ELEVATION	<i>h_N</i> := 41.16 <i>ft</i>	<i>t_N</i> := 21.97 <i>ft</i>
SOUTH ELEVATION	<i>h</i> _S :=41.37 <i>ft</i>	t _S ≔22.65 ft
EAST/WEST ELEVATION	<i>h_{EW}</i> :=40.18 <i>ft</i>	<i>t_{EW}</i> := 22.3 <i>ft</i>

FOR NORTH-SOUTH SEISMIC DIRECTION

Total Weight	$W_{NS} \coloneqq 12 \text{ psf} \cdot W \cdot D + w_w \cdot W \cdot (t_N + t_S)$	s) = 4152.16 <i>kip</i>
	$V := C_s \cdot W_{NS} = 83.04$ kip	$w := \frac{V}{W} = 153.78 \text{ plf}$
Diaphragm Forces Grid (A) + (L)	$R := w \cdot \frac{W}{2} = 41.52 \text{ kip}$	$v := \frac{R}{D} = 196.94 \ plf$
Chord Force	$M \coloneqq \frac{w \cdot W^2}{8} = 5605.42 \text{ kip} \cdot \text{ft}$	$P \coloneqq \frac{M}{D} = 26.59 \text{ kip}$

WIND CONTROLS DESIGN FOR NORTH-SOUTH DIRECTION

FOR EAST-WEST SEISMIC DIRECTION

Total Weight	$W_{EW} := 12 \text{ psf} \cdot W \cdot D + w_w \cdot D \cdot 2 \cdot t_{EW} = 2453.44 \text{ kip}$		
	$V \coloneqq C_s \cdot W_{EW} = 49.07 \ kip$	$w \coloneqq \frac{V}{D} = 232.74 \text{ plf}$	
Diaphragm Forces Grid (1) + (5)	$R := w \cdot \frac{D}{2} = 24.53 \text{ kip}$	$v \coloneqq \frac{R}{W} = 45.43 \text{ plf}$	
Chord Force	$M \coloneqq \frac{w \cdot D^2}{8} = 1293.17 \text{ kip} \cdot \text{ft}$	$P := \frac{M}{W} = 2.39 \text{ kip}$	

WIND CONTROLS DESIGN FOR EAST-WEST DIRECTION



Date	03/10/2022
Job	2220003 Building 2
<u>Subject</u>	Diaphragm Chord

DIAPHRAGM CHORD

Maximum Force	<i>P_u</i> ≔ 139.35 <i>kip</i> <i>P</i> ≔ 83.61 <i>kip</i>	from MWFRS (North/South)
Tension	$P_n = F_y \cdot A_g$ $\Omega := 1.67$	$F_y := 36$ ksi for A36 angle
	$A_{req} \coloneqq \frac{P \cdot \Omega}{F_y} = 3.879 \text{ in}^2$	
Compression	L = 6'-0" max. Refe	er to AISC Table 4-11
	Try L6x6x3/8 $\frac{P_n}{\Omega}$	= 72.2 kip < P
	Select L6x6x7/16 $\frac{P_n}{\Omega}$	= 88.1 <i>kip</i> > <i>P</i> OK
Splice	Check (2) 1/2x 5 Plate	
	$A \coloneqq \frac{1}{2} i \mathbf{n} \cdot 5 i \mathbf{n} \cdot 2 = 5 i \mathbf{n}^2$	A > A _{req}
	For 1/4 fillet weld	
	$L_{req} \coloneqq \frac{P}{0.928 \text{ ksi} \cdot 4 \text{ in} \cdot 2} = 1$	1.262 <i>in</i>
	Provide 12 in weld (min.)	
	OR Utilize square weld to <i>F_{Exx} > Fy</i> in lieu of splice	splice angles as plates



Date	03/10/2022
Job	2220003 Building 2
<u>Subject</u>	Diaphragm Chord

DIAPHRAGM CHORD

Maximum Force	P _u := 7.47 kip P:= 4.48 kip	from MWFRS	S (East/West)
Tension	$P_n = F_y \cdot A_g$ $\Omega := 1.67$	<i>F_y</i> :=36 <i>ksi</i>	for A36 angle
	$A_{req} \coloneqq \frac{P \cdot \Omega}{F_{y}} = 0.208 \text{ in}^{2}$		
Compression	L = 4'-0" by maximum embed spacing Refer to AISC Table 4-12		
	Try L4x4x3/8 $\frac{P_n}{\Omega} = 2$	9.6 kip>P	ОК
Splice	Check 3/8 x 3 Plate		
	$A := \frac{3}{8}$ in • 3 in = 1.125 in ²	A > A _{req}	ОК
	For 3/16 fillet weld		
	$L_{req} := \frac{P}{0.928 \ ksi \cdot 3 \ in \cdot 2} = 0.80$	05 <i>in</i>	

Provide 3 in weld (min.)



Date	03/10/2022
Job	2220003 Building 2
<u>Subject</u>	Wall Anchorage

WALL PANEL ANCHORAGE

Concrete Wall Thickness	<i>t</i> := 9.25 <i>in</i>				
	$w_T := 150 \ pcf \cdot t = 115.625 \ psf$				
	$P \coloneqq \max\left(0.2 \cdot w_T, 5 \text{ psf}\right) =$	23.125 psf			
Wind Wall Pressure	$A_T := 500 \ ft^2$				
	<i>w_{wall}</i> ≔ 23.3 psf				
	<i>w_{parapets}</i> := 51.3 <i>psf</i>				
Lateral Loads	<i>w_{wind}</i> := 0.6 ⋅ 829 <i>plf</i> = 497.4	plf 1	from North	n elevation	max parapet
	$w_{seismic} \coloneqq 233 \ plf$	1	from East/	/West eleva	ation
Gravity Loads	<i>w_{deck}</i> ≔ 287 <i>plf</i>		Joist Para	llel	
	$w_{joist} := \frac{6.8 \text{ kip}}{6 \text{ ft}} = 1133.333$	plf .	Joist Perp	endicular	
JOIST BEARING CONDITION					
Max. Joist Spacing	<i>s</i> := 6 <i>ft</i>				
Max. Diaphragm Shear	$R_{dia} := 86.33 \ \text{plf} \cdot \text{s} = 0.518 \ \text{s}$	kip			
Anchorage of Walls	$R_{anchor} \coloneqq W_{wind} \cdot s = 2.984$ ki	p			
Check standard seat weld (1/8" fillet x 2" long ea. side	e)			
	$R_{tot} \coloneqq R_{dia} + R_{anchor} = 3.502$	kip		_	
	$R_{n\Omega} \coloneqq 0.928 \ ksi \cdot 2 \ in \cdot 2 \ in$	• 2 = 7.424 <i>kij</i>	σ	$\frac{R_n}{\Omega} > R_{tot}$	ОК
JOIST PARALLEL CONDITION	1				
ANCHORAGE EMBED AT 4	4'-0"o.c. MAX - CHECK L4	x4x3/8			
$W_{grav} := W_{det}$	_{ck} =287 plf				
<i>w_{axial}</i> := 619	9.54 plf	ANGLE IS	ADEQUA	TE	
$W_{anchor} := 80$	01.87 <i>plf</i> max.				
Check attachment to embed	l for 3/16 fillet				
$P_{grav} := W_{grav}$	w • 4 <i>ft</i> = 1.148 <i>kip</i>	$P_{anchor} := w_{anchor}$	_{chor} •4 ft =3	3.207 kip	
$P_{axial} := w_{axi}$	_{ial} •4 ft =2.478 kip	$P_{moment} := \frac{P_{g}}{P_{g}}$	_{nrav} ∙3 in 4 in +	P _{anchor} ∙3 ii 4 in	n —=3.267 <i>kip</i>
$P_{tot} := P_{grav}$	$+P_{axial}+P_{anchor}+P_{moment}=1$	0.1 <i>kip</i>			
$L_{req} := \frac{1}{0.92}$	$\frac{P_{tot}}{8 \text{ ksi} \cdot 3 \text{ in}} = 3.628 \text{ in}$	USE 4in M	IN. WELD	1	



Date 3/10/22	Sheet No.	of
Job 2220003		
Subject ODNET D	AGRAMS	



amin = 13.5' Mmax = 37.92'

L	
DL=3psf	
0.64	0.6W-D.6D
le 8 psf	15.0 psf
29.2psf	27.21 psf
38.Spsf	36.7 psf
82.4psf	DO. Lepst
9 Lepsf	_
	DL = 3psf O. GW He & psf 29.2psf 38.Spsf 52.4psf 9.6psf

ROOF JOISTS DL - 10psf AT MAX (6'.50', (50')') = 833 ft2 CONSERVATIVELY USE 3 C.C. ZONE 1' 9.6 psf 0.60-0.60 3.lepst 12.3 psf 18.3psf ZONDEI 241.5 psf 18. Spsf ZONE 2 18.5psf 24.5psf ZONE 3 POSITINE 9. Le psf ROOF GIRDERS DL - 12psf AT MAX (541-0" × 50', (541)= 2700 A2 > 700 A2 0.600 0.600-0.60 OTD 16.8' 14.7psf 7:5pst 7:5pst 2:2pst 121.7psf 16.8 10 33.6' 9.4 psf 33.6' TO 67.3' 6.7 pst NO UPLIFT 167.3'

2023 Sheet No. 2 of Date 0003 BUILDING I Job Subject ROVE STRUCTURE

TYP. RECEF JUISTS SPROING G'O" $m_{n} = 10 \text{ BF}$, 115 HEF (FOR L>50') $m_{n} = 30 \text{ BF}$ $m_{n} = (10+20)(G'-0) = 100 \text{ HF}$, 109 HF (FOR L>50') $m_{n} = [12(10) + 1.G(20)] \text{ G.o}' = 2G4 \text{ HF}$, 274.5 HF (FOR L>50') JOIST REACTIONS $L = GO' \quad V = wL/2 = 5.G7 \text{ K}$ ASD, 8.244 K UPFO $L = 50' \quad V = wL/2 = 4.50 \text{ K}$ ASD, 6.GO0 K UPFO TYP. GIRDER POINT LONDS (ASD) AND BAY NORTH P = 50'[180 HF] = 9.00 KHSEND HAY SOMAL P = 50'[180 HF] = 11.340 KHSTYP. BAY MODULE P = 50'[180 HF] = 9.00 KHSAT SP1 P = 5087.7 HS + 50/2[180 HF] = 9.59 KHS, 14.740 KHS LADD AT SP2 P = 4983.4 HS + 50/2[180 HF] = 9.483 KHS, 14.573 KHS (AFD)

Date	3/3/2022	Sheet No.	G	of	
Job	2220003	BUILDING I			
Subje	et ROCE STI	PUCTURE			

	EVALU	ATE FA. J	GIBIOER I OIST DIVE	TO ORIFT					
0		(5		<u>,</u>			
		S S S	PI 84' Joists I-14			SPI			
	COMBIN	EL+/	1 SNOW						
		R _L (K)	Pig(K)	Pu (K)	R. (K)			
	SPI	1.998	0.288	3.196	8 0.46				
	SPZ	1.524	0.150	2.438	5 a240	C			
GRID	1-2	Rs, (K)	Rsun (K)	BSTOT_ (K)	PISTOTR(K)	Ru(k)	R _{up} (K) 17.586		
	2-	9,210	14.136	11208	9493	17.333	14.597		
	2	J. 343	11.148	9 341	7 631	14 345	11.609		
	2~4	5.475	8.160	7.473	5.763	11. 357	8.621		
	net	4.807	7.080	(. 798	5,088	9518	7.32		
GRIC) 4-5								
	\sim	10.101	15.330	11.605	10.251	17.763	15.57		
	No	7.8GO	11.748	9.384	8.010	14.18G	11.988		
	Wst CUT CV	5.760	8,496	7.284	5.910	10.934	8.736	1000	
	CRID	JUST UM	ATD V-1		JUISTS 1 :	11.365	+ 9,858 += 21.	223K, 17.586 + 1	5.172=
	Grill	a, x-p:) K-L	0	а	9.498	+7.390 = 17	488K, 14.597 ++ 12	2.137 =
		2346		(9)	3	7.631	4 6.123 k = 13.	754 K , 11. 609 + + 9	196=
		EA Er	<u> </u>		4	5.7C31	4 4.800 K = 10	563K, 8.621 + =	7.005
	A 1 2	58.490 K ASD		1 53.97	7 KASD 5-9	5,088 1	$(+ 4.800 \ \text{k} = 9)$	838K, 7.320 + +7	108) = 14 40 K
	124	33. 586 x Lp1	íp	2B 86.3	63× Lefo				
	Chio	1 2 10			JOISTS 1 :	10.251 +	9.858 = 20.109K	, 15,57 + 15,172	= 30,74a K
	GHID	4; A-B,	, K-1.		2	8.010 + -	7.990 = 16.09K	, 11.983 + 12,184 =	= 24.167
Y				9	3	5,910 + 6	.123 = 12.0334	2 8.736+ 9.196=	= 17.932 *
			<u> </u>	¥,	4-9	5.910 +	4.800 = 10.7104	2 8.736 + 7.080 =	= 15.8164
	<u></u> ∱, ∧ c	57 PT	JZ III	A =	321 16				
	14A "	90,018 = LA		14B 20	969k LEFA				

)				Di Jo Su	ate 3/ ob 33 ubject p	3/2022 10003 3009 ST	Sheet PullON RUCTUR	No. 7 I. E 6 C	obumn	DEGIGN	
GRID	2 4	HECK ALL JOIST	GIRDEPS CI P	TO DEC	IGN COLU	MINS FL	61	H	J	K.	L -,4
		L=Str TR BAY	 	4	f	1	1	1	4	4	
	£510 ·	53.9774.850 + 9.594(4.75) = 99.534	9.59(9) = 86.31K	86.314	1 8G314	86.31k	86.31 K	8G.31k	8G.31#	1 99.53K	
	LEFD	86,963 K + 14,740(4.75) = (56,378 K	14.740(5) = 132.664	132,66	132.GGr	132,664	13266K	132GG1×	132664	15G,378 K	
GRI	23 A	B	C	0	1	ę	Q	H	J	K	L _ 4
		6=54' TYP. BAY	ł	1	ł	2	1	**	F	1	
	ASD	51,424 K+ 9.03 K(4.75) = 94,174 K	9.00.9 = 81.0K	81,004	81,004	81,004	81,004	81, Ook	81.004	94,174K	
	14分	82,278 K + 13.200 K (4.75) = 144.973 K	13200(9 = /18,8) 118.8 K	3e 118	.8⊭ /18.9	3r 113,8r	/18.8r	118.34	118,84	
GRI	D 4 3-	L= 54'			A						- 4

52.231K+ 9.483 (475) 97.275K 9.483× 88 ×(9) = 88.047× 88.047 × 88.047 × 88.047 × 88.047 × 88.047 × 88.047 × 97.275 × ASD + 500

131.157 K

1

 $\begin{array}{r} 89.969 \\ + 14.573(4.75) \\ = 159.19075 \\ \end{array}$ H.573(9)= 131.157k131.1574 131.157K 131.157K 131.157K 131.157K 131.157K 159.191K LAFD

Sheet No. 8 of Date BUILDING I Job 003 Subject COLUMN DESIGN

COLUMNS: GRID B/K 52-4M.: 38. 458' NWX NWX LOND: $97.275 \times (AGD)$ HSS 10 × 10 × 916 $P_0/Q_0 = 121,25 \times B_0, 40.4 \ 16/FT$ TYPICAL COLUMNS: $-121,25 \times B_0, 40.4 \ 16/$



Date	03/10/2021
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DESIGN CRITERIA

Max. Allowable Bearing Capacity	<i>q_{a.max}</i> ≔ 2500 <i>psf</i>
Concrete Strength	f' _c := 3000 psi

TYPICAL SPREAD FOOTING

Minimum Column Load	$P_{min} \coloneqq min\left(P_{c}, P_{e}, P_{t}\right) = 88$ kip
Maximum Column Load	$P_{max} \coloneqq \max \left(P_c, P_e, P_t \right) = 101 \ kip$

$$\frac{P}{A} \le q_{a.max} \qquad \qquad A_{req.min} \coloneqq \frac{P_{min}}{q_{a.max}} = 35.154 \ \text{ft}^2 \qquad \qquad B_{min} \coloneqq \sqrt{A_{req.min}} = 5.929 \ \text{ft}$$

$$A_{req} \ge \frac{P}{q_{a.max}} \qquad \qquad A_{req.max} \coloneqq \frac{P_{max}}{q_{a.max}} = 40.415 \ \text{ft}^2 \qquad \qquad B_{max} \coloneqq \sqrt{A_{req.max}} = 6.357 \ \text{ft}$$

Select 6'-0"x6'-0"x1'-6" footing w/ (6)-#6 bottom bars ea. way

TYPICAL WALL FOOTING

Assumed precast wa	all thickness	t := 9.25 in $w_w := 150$ pcf ·	t = 115.625 psf			
Maximum Wall Heigh Minimum Wall Heigh	nt t	$H_{max} := 42 \ ft + 6$ $H_{min} := 39 \ ft + 6$	S in S in	$w_{w.max} := w_w \cdot H_{max} = 4914 \text{ plf}$ $w_{w.min} := w_w \cdot H_{min} = 4567 \text{ plf}$		
Joist Perpindicular		R _{typ} := 5.5 kip		$w_{j.typ} \coloneqq \frac{R_{typ}}{6 \ ft} = 91$	6.667 plf	
Joist Parallel		w:= 182 plf		<i>w_{j.∥}</i> := 98 <i>plf</i>		
Max Wall Height	Joist Perpindi Joist Parallel	cular	$w := w_{w.max} + w_{j.}$ $w := w_{w.max} + w_{j.}$	_{typ} = 5831 plf _{II} = 5012 plf	$B_{min} := w \cdot q_{a.max}^{-1} = 2.332 \text{ ft}$ $B_{min} := w \cdot q_{a.max}^{-1} = 2.005 \text{ ft}$	
Min Wall Height	Joist Perpindi Joist Parallel	cular	$w := w_{w.min} + w_{j.t}$ $w := w_{w.min} + w_{j.t}$	_{typ} = 5484 plf ₁ = 4665 plf	$B_{min} := w \cdot q_{a.max}^{-1} = 2.194 \ ft$ $B_{min} := w \cdot q_{a.max}^{-1} = 1.866 \ ft$	

Provide 3'-0" typical strip footing with (3)-#5 T&B longitudinal bars



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FOUNDATION AT JOIST GIRDER/WALL LOCATION

*Assume the cont. wall footing has no reserve capacity

Max. Joist Girder Reaction at Wall
$$P \coloneqq 60.2 \text{ kip}$$

 $A_{req} \coloneqq \frac{P}{q_{a.max}} = 24.08 \ \text{ft}^2$ $A_{req} = B^2 - 3 \cdot B$

$$B := 3 \ ft$$
 min.
 $B := root (B^2 - 3 \ ft \cdot B - A_{reg}, B) = 6.631 \ ft$

Provide 7'-0" ftg at joist/wall intersection at corners

Typ. Joist Girder Reaction at Wall

P ≔ 57.9 **kip**

$$A_{req} \coloneqq \frac{P}{q_{a.max}} = 23.16 \ \text{ft}^2$$

$$A_{req} = B^2 - 3 \cdot B$$

$$B := 3 \ ft \qquad \text{min.}$$

$$B := \operatorname{root} \left(B^2 - 3 \ ft \cdot B - A_{req}, B \right) = 6.541 \ ft$$

Provide 7'-0" ftg at typical joist/wall intersection