ELLISON GAGE
& ASSOCIATES, PLLC
Consulting Structural Engineers

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STRUCTURAL CALCULATIONS

Salad and Go #2001

Lee’s Summit (Chipman & NW Ward), MO
Project No. 24-011

ELLISON GAGE
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LOAD CRITERIA

- 2018 INTERNATIONAL BUILDING CODE (ASCE 7-16)

- DEAD LOADS:
  - ROOF

	MAX.	MIN. (UPLIFT)
INSULATION, AND METAL DECK	5.0 PSF	2.0 PSF
FRAMING	3.0 PSF	2.0 PSF
CEILING, MECH, ELEC, AND PLUMBING	12.0 PSF	2.0 PSF
TOTAL	**20 PSF**	**6 PSF**

- LIVE LOADS
  - ROOF **20 PSF**
  - FLOOR (SLAB-ON-GRADE) **150 PSF**

- SNOW LOAD: (ASCE 7-16)
  GROUND SNOW LOAD, P_g = 20 psf

ATC Hazards by Location

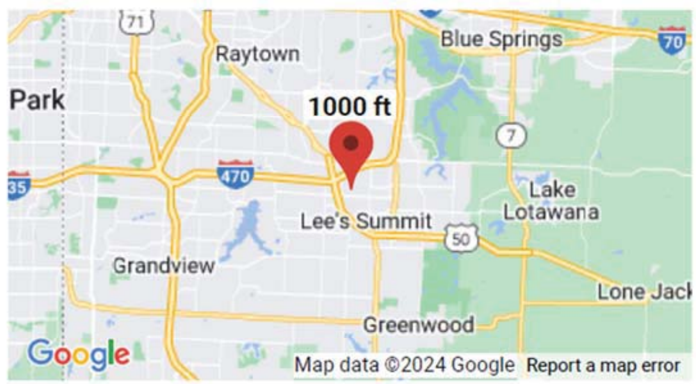
Search Information

Coordinates: 38.926402, -94.394617

Elevation: 1000 ft

Timestamp: 2024-04-05T19:18:01.814Z

Hazard Type: Snow



ASCE 7-16	ASCE 7-10	ASCE 7-05
Ground Snow Load 20 lb/sqft	Ground Snow Load 20 lb/sqft	Ground Snow Load 20 lb/sqft

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SNOW DRIFT ANALYSIS:

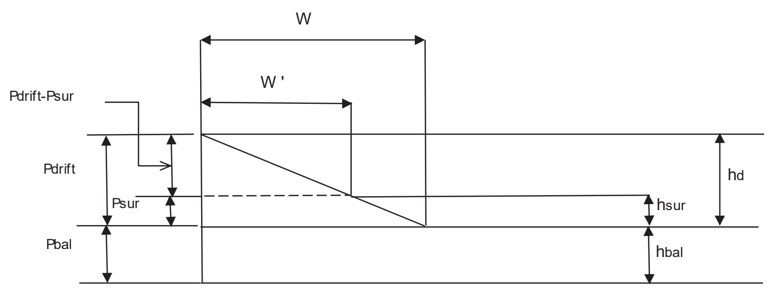
Snow Drift Loads (Low Slope Roofs)

Code: ASCE 7-16

Input Data			
Ground Snow Load, p_g =	20	*psf*	
Exposure Factor, C_e =	1.0		[TABLE 7.3-1]
Thermal Factor, C_t =	1.0		[TABLE 7.3-2]
Importance Factor, I =	1.0		[TABLE 1.5-2]
Flat Roof Snow Load, p_f =	14.0	*psf*	[EQ. 7.3-1]
Minimum Snow Load, p_m =	20.0	*psf*	[7.3.4]
Rain-On-Snow Surcharge Load =	5.0	*psf*	[7.10]
Design Roof Snow Load, $p_{bal}+p_{fsur}$ =	20.0	*psf*	

This is the "Total Design Roof Snow Load" for Sheet S0

Output Information		
Snow Density, γ =	16.6	*pcf*
Snow depth, h_{bal} =	0.84	*ft*
Snow depth, $h_{bal}+h_{sur}$ =	1.20	*ft <= BALANCED SNOW LOAD DEPTH INCLUDING RAIN-ON-SNOW SURCHARGE OR MINIMUM SNOW LOAD*



Parapet	h_r (ft)	l_u (ft)	h_c (ft)	h_c/h_b	h_d (ft)	W (ft)	p_{drift} (psf)	$p_{drift}+p_{bal}$ (psf)	$p_{drift}-p_{sur}$ (psf)	W' (ft)	
FRONT WALL	6.0	54	5.16	6.1	1.72	6.9	29	43	23	5.4	
REAR WALL	4.0	54	3.16	3.7	1.72	6.9	29	43	23	5.4	
SIDE WALLS	6.0	16	5.16	6.1	0.78	3.1	13	27	7	1.7	
AWNINGS	10.0	16	8.99	8.9	1.04	4.1	17	34	14	3.4	*LOWER ROOF, COLD ROOF

ELLISON GAGE & ASSOCIATES, PLLC Consulting Structural Engineers 5068 W. Plano Parkway Suite 200 Plano, TX 75093 Ofc: (972) 354-8855 Fax: (972) 354-8856 www.ellisongage.com	Project SALAD AND GO #2001 – LEE'S SUMMIT, MO			Job Ref. 24-011
	Description LOAD CRITERIA DESIGN (SEC. LC)	Calc. by BG	Date 4/11/2024	Sheet No. LC3

o WIND LOADS: (ASCE 7-16, SECS. 26 THROUGH 31, DIRECTIONAL PROCEDURE)

RISK CATEGORY II, V_{ULT} = 110 MPH

ATC Hazards by Location

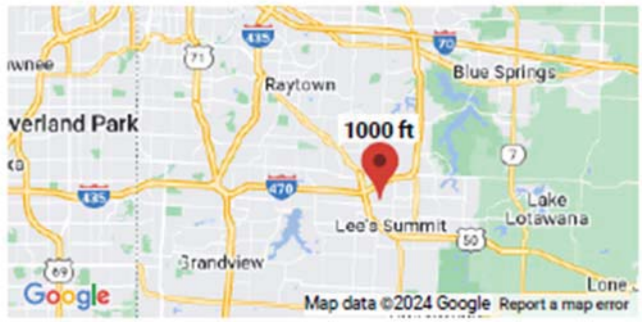
Search Information

Coordinates: 38.926402, -94.394617

Elevation: 1000 ft

Timestamp: 2024-04-05T19:16:08.076Z

Hazard Type: Wind



ASCE 7-16

MRI 10-Year	76 mph
MRI 25-Year	83 mph
MRI 50-Year	88 mph
MRI 100-Year	94 mph
Risk Category I	103 mph
Risk Category II	109 mph
Risk Category III	117 mph
Risk Category IV	122 mph

ASCE 7-10

MRI 10-Year	76 mph
MRI 25-Year	84 mph
MRI 50-Year	90 mph
MRI 100-Year	96 mph
Risk Category I	105 mph
Risk Category II	115 mph
Risk Category III-IV	120 mph

ASCE 7-05

ASCE 7-05 Wind Speed	90 mph

USE 110 MPH

WIND LOADS (Components & Cladding, ASCE 7-16)

Wind Load Parameters [CHAPTER 26]

Risk Category =	II		[TABLE 1.5-1]
Basic Wind Speed, V =	110	mph	[FIG. 26.5-1, ATC WEBSITE]
Service Wind Speed, V_{asd} =	85	mph	
K_d =	0.85		[TABLE 26.6-1]
Exposure =	C		[26.7]
K_{zt} =	1.00		[26.8.2]
Elevation =	900	ft	[26.9]
G =	0.85		[26.11.1]
Enclosure Classification =	E		[26.12]
GC_{pi} =	0.18		[TABLE 26.13-1]
α =	9.5		[TABLE 26.11-1]
z_g =	900	ft	[TABLE 26.11-1]
Mean Roof Height, h =	14.0	ft	
K_h =	0.85		[TABLE 26.10-1]
K_e =	0.968		[TABLE 26.9-1]
q_h =	21.6	psf (at h ft)	[EQ. 26.10-1]
Roof Angle, θ =	1.2	deg	

Enclosed and Partially Enclosed Low-Rise Buildings with h < 60 ft (Part 1) [CHAPTER 30] [30.3]

Enclosed and Partially Enclosed Low-Rise Buildings with h<60 ft

$p = q_h[(GC_p) - (GC_{pi})]$ psf [EQ. 30.3-1]

Area	Effective Wind Area (ft^2)	Zone	+GCp	-GCp	LRFD + Pressure (psf)	LRFD - Pressure (psf)	ASD + Pressure (psf) * 0.6	ASD - Pressure (psf) * 0.6	Reference
Walls (general)	0.0	4	0.90	-0.99	**23.4**	**-25.3**	**14.0**	**-15.2**	FIG. 30.3-1
	0.0	5	0.90	-1.26	**23.4**	**-31.2**	**14.0**	**-18.7**	FIG. 30.3-1
Walls 10 ft^2	10.0	4	0.90	-0.99	**23.4**	**-25.3**	**14.0**	**-15.2**	FIG. 30.3-1
	10.0	5	0.90	-1.26	**23.4**	**-31.2**	**14.0**	**-18.7**	FIG. 30.3-1
Walls 20 ft^2	20.0	4	0.85	-0.94	**22.3**	**-24.3**	**13.4**	**-14.6**	FIG. 30.3-1
	20.0	5	0.85	-1.16	**22.3**	**-29.1**	**13.4**	**-17.5**	FIG. 30.3-1
Walls 50 ft^2	50.0	4	0.79	-0.88	**21.0**	**-22.9**	**12.6**	**-13.7**	FIG. 30.3-1
	50.0	5	0.79	-1.04	**21.0**	**-26.3**	**12.6**	**-15.8**	FIG. 30.3-1
Walls 100 ft^2	100.0	4	0.74	-0.83	**19.9**	**-21.9**	**12.0**	**-13.1**	FIG. 30.3-1
	100.0	5	0.74	-0.94	**19.9**	**-24.3**	**12.0**	**-14.6**	FIG. 30.3-1
Walls 200 ft^2	200.0	4	0.69	-0.78	**18.9**	**-20.8**	**11.3**	**-12.5**	FIG. 30.3-1
	200.0	5	0.69	-0.85	**18.9**	**-22.2**	**11.3**	**-13.3**	FIG. 30.3-1
Walls > 500 ft^2	500.0	4	0.63	-0.72	**17.5**	**-19.5**	**10.5**	**-11.7**	FIG. 30.3-1
	500.0	5	0.63	-0.72	**17.5**	**-19.5**	**10.5**	**-11.7**	FIG. 30.3-1
Roof (general)	82.0	1'	0.21	-0.90	**8.4**	**-23.4**	**5.0**	**-14.0**	FIG.30.3-2A
	82.0	1	0.21	-1.32	**8.4**	**-32.5**	**5.0**	**-19.5**	FIG.30.3-2A
	82.0	2	0.21	-1.82	**8.4**	**-43.2**	**5.0**	**-25.9**	FIG.30.3-2A
	82.0	3	0.21	-2.23	**8.4**	**-43.2**	**5.0**	**-25.9**	FIG.30.3-2A
Roof 10 ft^2	10.0	1'	0.30	-0.90	**10.4**	**-23.4**	**6.2**	**-14.0**	FIG.30.3-2A
	10.0	1	0.30	-1.70	**10.4**	**-40.7**	**6.2**	**-24.4**	FIG.30.3-2A
	10.0	2	0.30	-2.30	**10.4**	**-53.7**	**6.2**	**-32.2**	FIG.30.3-2A
	10.0	3	0.30	-3.20	**10.4**	**-53.7**	**6.2**	**-32.2**	FIG.30.3-2A
Roof 20 ft^2	20.0	1'	0.27	-0.90	**9.7**	**-23.4**	**5.8**	**-14.0**	FIG.30.3-2A
	20.0	1	0.27	-1.58	**9.7**	**-38.0**	**5.8**	**-22.8**	FIG.30.3-2A
	20.0	2	0.27	-2.14	**9.7**	**-50.2**	**5.8**	**-30.1**	FIG.30.3-2A
	20.0	3	0.27	-2.88	**9.7**	**-50.2**	**5.8**	**-30.1**	FIG.30.3-2A
Roof 50 ft^2	50.0	1'	0.23	-0.90	**8.9**	**-23.4**	**5.3**	**-14.0**	FIG.30.3-2A
	50.0	1	0.23	-1.41	**8.9**	**-34.4**	**5.3**	**-20.7**	FIG.30.3-2A
	50.0	2	0.23	-1.93	**8.9**	**-45.6**	**5.3**	**-27.4**	FIG.30.3-2A
	50.0	3	0.23	-2.46	**8.9**	**-45.6**	**5.3**	**-27.4**	FIG.30.3-2A
Roof 100 ft^2	100.0	1'	0.20	-0.90	**8.2**	**-23.4**	**4.9**	**-14.0**	FIG.30.3-2A
	100.0	1	0.20	-1.29	**8.2**	**-31.8**	**4.9**	**-19.1**	FIG.30.3-2A
	100.0	2	0.20	-1.77	**8.2**	**-42.2**	**4.9**	**-25.3**	FIG.30.3-2A
	100.0	3	0.20	-2.14	**8.2**	**-42.2**	**4.9**	**-25.3**	FIG.30.3-2A
Roof 500 ft^2	500.0	1'	0.20	-0.55	**8.2**	**-15.8**	**4.9**	**-9.5**	FIG.30.3-2A
	500.0	1	0.20	-1.00	**8.2**	**-25.5**	**4.9**	**-15.3**	FIG.30.3-2A
	500.0	2	0.20	-1.40	**8.2**	**-34.2**	**4.9**	**-20.5**	FIG.30.3-2A
	500.0	3	0.20	-1.40	**8.2**	**-34.2**	**4.9**	**-20.5**	FIG.30.3-2A
Roof > 1000 ft^2	1000.0	1'	0.20	-0.40	**8.2**	**-12.5**	**4.9**	**-7.5**	FIG.30.3-2A
	1000.0	1	0.20	-1.00	**8.2**	**-25.5**	**4.9**	**-15.3**	FIG.30.3-2A
	1000.0	2	0.20	-1.40	**8.2**	**-34.2**	**4.9**	**-20.5**	FIG.30.3-2A
	1000.0	3	0.20	-1.40	**8.2**	**-34.2**	**4.9**	**-20.5**	FIG.30.3-2A

COMBINED WALL WIND PRESSURES $_{N\text{-}S\ WIND}$ (PSF)				
HT (FT)	$P_{WINDWARD}$	$P_{LEEWARD}$	P_{TOTAL}	0.6 * P_{TOTAL}
0-15	15.2	-9.2	**24.4**	**14.6**
15-20	16.1	-9.2	**25.3**	**15.2**
20-25	16.9	-9.2	**26.1**	**15.7**
25-30	17.6	-9.2	**26.8**	**16.1**
30-40	18.7	-9.2	**27.9**	**16.7**
40-50	19.6	-9.2	**28.8**	**17.3**
50-60	20.3	-9.2	**29.5**	**17.7**
60-70	21.0	-9.2	**30.2**	**18.1**

COMBINED WALL WIND PRESSURES $_{E\text{-}W\ WIND}$ (PSF)				
HT (FT)	$P_{WINDWARD}$	$P_{LEEWARD}$	P_{TOTAL}	0.6 * P_{TOTAL}
0-15	15.2	-4.5	**19.7**	**11.8**
15-20	16.1	-4.5	**20.6**	**12.4**
20-25	16.9	-4.5	**21.4**	**12.8**
25-30	17.6	-4.5	**22.1**	**13.2**
30-40	18.7	-4.5	**23.2**	**13.9**
40-50	19.6	-4.5	**24.1**	**14.4**
50-60	20.3	-4.5	**24.8**	**14.9**
60-70	21.0	-4.5	**25.5**	**15.3**

ROOF WIND PRESSURES (PSF)				
Horiz. dist. from windward edge	$P_{N\text{-}S\ WIND}$	$P_{E\text{-}W\ WIND}$	$0.6{*}P_{N\text{-}S\ WIND}$	$0.6{*}P_{E\text{-}W\ WIND}$
0 to h/2	**-24.8**	**-20.4**	**-14.9**	**-12.3**
h/2 to h	**-18.3**	**-20.4**	**-11.0**	**-12.3**
h to 2h	**-15.3**	**-13.1**	**-9.2**	**-7.9**
>2h	**-13.8**	**-9.4**	**-8.3**	**-5.6**

PARAPETS [27.3.4]

$p_p = q_p(GC_{pn})$ psf [EQ. 27.3-3]

Top of parapet = 20.17 ft

$K_{z,\ top\ of\ parapet}$ = 0.90 [TABLE 26.10-1]

$q_{z,\ top\ of\ parapet}$ = 23.0 psf

$+GC_{pn}$ = 1.5 [27.3.4]

$-GC_{pn}$ = -1.0 [27.3.4]

$+p_{windward}$ = 34.5 psf

$-p_{leeward}$ = -23.0 psf

p_p = 57.6 psf (Factored)

0.6 * p_p = 34.5 psf (Unfactored)

PARAPET (LEEWARD)

p_p = 23.0 psf (Factored)

0.6 * p_p = 13.8 psf (Unfactored)

PARAPET (WINDWARD)

p_p = 34.5 psf (Factored)

0.6 * p_p = 20.7 psf (Unfactored)

WIND LOADS (Components & Cladding, ASCE 7-16)

Wind Load Parameters [CHAPTER 26]

Risk Category =	II		[TABLE 1.5-1]
Basic Wind Speed, V =	110	mph	[FIG. 26.5-1, ATC WEBSITE]
Service Wind Speed, V_{asd} =	85	mph	
K_d =	0.85		[TABLE 26.6-1]
Exposure =	C		[26.7]
K_{zt} =	1.00		[26.8.2]
Elevation =	900	ft	[26.9]
G =	0.85		[26.11.1]
Enclosure Classification =	E		[26.12]
GC_{pi} =	0.18		[TABLE 26.13-1]
α =	9.5		[TABLE 26.11-1]
z_g =	900	ft	[TABLE 26.11-1]
Mean Roof Height, h =	14.0	ft	
K_h =	0.85		[TABLE 26.10-1]
K_e =	0.968		[TABLE 26.9-1]
q_h =	21.6	psf (at h ft)	[EQ. 26.10-1]
Roof Angle, θ =	1.2	deg	

Enclosed and Partially Enclosed Low-Rise Buildings with h < 60 ft (Part 1) [CHAPTER 30] [30.3]

Enclosed and Partially Enclosed Low-Rise Buildings with h<60 ft

$p = q_h[(GC_p) – (GC_{pi})]$ psf [EQ. 30.3-1]

Area	Effective Wind Area (ft^2)	Zone	+GCp	-GCp	LRFD + Pressure (psf)	LRFD - Pressure (psf)	ASD + Pressure (psf) * 0.6	ASD - Pressure (psf) * 0.6	Reference
Walls (general)	0.0	4	0.90	-0.99	**23.4**	**-25.3**	**14.0**	**-15.2**	FIG. 30.3-1
	0.0	5	0.90	-1.26	**23.4**	**-31.2**	**14.0**	**-18.7**	FIG. 30.3-1
Walls 10 ft^2	10.0	4	0.90	-0.99	**23.4**	**-25.3**	**14.0**	**-15.2**	FIG. 30.3-1
	10.0	5	0.90	-1.26	**23.4**	**-31.2**	**14.0**	**-18.7**	FIG. 30.3-1
Walls 20 ft^2	20.0	4	0.85	-0.94	**22.3**	**-24.3**	**13.4**	**-14.6**	FIG. 30.3-1
	20.0	5	0.85	-1.16	**22.3**	**-29.1**	**13.4**	**-17.5**	FIG. 30.3-1
Walls 50 ft^2	50.0	4	0.79	-0.88	**21.0**	**-22.9**	**12.6**	**-13.7**	FIG. 30.3-1
	50.0	5	0.79	-1.04	**21.0**	**-26.3**	**12.6**	**-15.8**	FIG. 30.3-1
Walls 100 ft^2	100.0	4	0.74	-0.83	**19.9**	**-21.9**	**12.0**	**-13.1**	FIG. 30.3-1
	100.0	5	0.74	-0.94	**19.9**	**-24.3**	**12.0**	**-14.6**	FIG. 30.3-1
Walls 200 ft^2	200.0	4	0.69	-0.78	**18.9**	**-20.8**	**11.3**	**-12.5**	FIG. 30.3-1
	200.0	5	0.69	-0.85	**18.9**	**-22.2**	**11.3**	**-13.3**	FIG. 30.3-1
Walls > 500 ft^2	500.0	4	0.63	-0.72	**17.5**	**-19.5**	**10.5**	**-11.7**	FIG. 30.3-1
	500.0	5	0.63	-0.72	**17.5**	**-19.5**	**10.5**	**-11.7**	FIG. 30.3-1
Roof (general)	82.0	1'	0.21	-0.90	**8.4**	**-23.4**	**5.0**	**-14.0**	FIG.30.3-2A
	82.0	1	0.21	-1.32	**8.4**	**-32.5**	**5.0**	**-19.5**	FIG.30.3-2A
	82.0	2	0.21	-1.82	**8.4**	**-43.2**	**5.0**	**-25.9**	FIG.30.3-2A
	82.0	3	0.21	-2.23	**8.4**	**-43.2**	**5.0**	**-25.9**	FIG.30.3-2A
Roof 10 ft^2	10.0	1'	0.30	-0.90	**10.4**	**-23.4**	**6.2**	**-14.0**	FIG.30.3-2A
	10.0	1	0.30	-1.70	**10.4**	**-40.7**	**6.2**	**-24.4**	FIG.30.3-2A
	10.0	2	0.30	-2.30	**10.4**	**-53.7**	**6.2**	**-32.2**	FIG.30.3-2A
	10.0	3	0.30	-3.20	**10.4**	**-53.7**	**6.2**	**-32.2**	FIG.30.3-2A
Roof 20 ft^2	20.0	1'	0.27	-0.90	**9.7**	**-23.4**	**5.8**	**-14.0**	FIG.30.3-2A
	20.0	1	0.27	-1.58	**9.7**	**-38.0**	**5.8**	**-22.8**	FIG.30.3-2A
	20.0	2	0.27	-2.14	**9.7**	**-50.2**	**5.8**	**-30.1**	FIG.30.3-2A
	20.0	3	0.27	-2.88	**9.7**	**-50.2**	**5.8**	**-30.1**	FIG.30.3-2A
Roof 50 ft^2	50.0	1'	0.23	-0.90	**8.9**	**-23.4**	**5.3**	**-14.0**	FIG.30.3-2A
	50.0	1	0.23	-1.41	**8.9**	**-34.4**	**5.3**	**-20.7**	FIG.30.3-2A
	50.0	2	0.23	-1.93	**8.9**	**-45.6**	**5.3**	**-27.4**	FIG.30.3-2A
	50.0	3	0.23	-2.46	**8.9**	**-45.6**	**5.3**	**-27.4**	FIG.30.3-2A
Roof 100 ft^2	100.0	1'	0.20	-0.90	**8.2**	**-23.4**	**4.9**	**-14.0**	FIG.30.3-2A
	100.0	1	0.20	-1.29	**8.2**	**-31.8**	**4.9**	**-19.1**	FIG.30.3-2A
	100.0	2	0.20	-1.77	**8.2**	**-42.2**	**4.9**	**-25.3**	FIG.30.3-2A
	100.0	3	0.20	-2.14	**8.2**	**-42.2**	**4.9**	**-25.3**	FIG.30.3-2A
Roof 500 ft^2	500.0	1'	0.20	-0.55	**8.2**	**-15.8**	**4.9**	**-9.5**	FIG.30.3-2A
	500.0	1	0.20	-1.00	**8.2**	**-25.5**	**4.9**	**-15.3**	FIG.30.3-2A
	500.0	2	0.20	-1.40	**8.2**	**-34.2**	**4.9**	**-20.5**	FIG.30.3-2A
	500.0	3	0.20	-1.40	**8.2**	**-34.2**	**4.9**	**-20.5**	FIG.30.3-2A

Parapets [30.8]

$p = q_p((GC_p)-(GC_{pi}))$ psf [EQ. 30.8-1]

Top of Parapet =	20.17	ft	
$K_{z,\ top\ of\ parapet}$ =	0.90		[TABLE 30.3-1]
$q_{p,\ top\ of\ parapet}$ =	23.8	psf	[EQ. 30.3-1]
Effective Wind Area =	50.0	ft^2	
GC_{pi} =	0.00		[TABLE 26.11-1]
Min. Parapet Height Around Bldg. =	4.00	ft	[FIGURE 30.4-2A, NOTE 5]

Pressure Coefficients, GC_p			
Area	Corner Zone	Int. Zone	
Negative Wall	-1.04	-0.88	[FIG. 30.3-1]
Positive Wall	0.79	0.79	[FIG. 30.3-1]
Negative Roof	-1.93	-1.93	[FIG. 30.4-2A]

PARAPET WIND PRESSURES (PSF) [FIG. 30.9-1]				
Load Case A	p_1 (psf)	p_2 (psf)	p_{TOTAL} (psf)	p_{TOTAL} (psf) * 0.6
Interior Zone	18.8	-45.9	**64.7**	**38.8**
Corner Zone	18.8	-45.9	**64.7**	**38.8**
Load Case B	p_3 (psf)	p_4 (psf)	p_{TOTAL} (psf)	p_{TOTAL} (psf) * 0.6
Interior Zone	18.8	-20.9	**39.7**	**23.8**
Corner Zone	18.8	-24.7	**43.5**	**26.1**

Area	Effective Wind Area (ft^2)	Zone	+GCp	-GCp	+ Pressure (psf)	- Pressure (psf)	- Pressure (psf) * 0.6	Dead Load (psf)	Net Pressure (psf)
Joists	84	1'	0.21	-0.90	8.4	-23.4	-14.0	6.0	**8.0**
	84	1	0.21	-1.32	8.4	-32.5	-19.5	6.0	**13.5**
	84	2	0.21	-1.81	8.4	-43.1	-25.9	6.0	**19.9**
	84	3	0.21	-2.22	8.4	-43.1	-25.9	6.0	**19.9**

o SEISMIC LOADS: (ASCE 7-16, EQUIVALENT LATERAL FORCE PROCEDURE)

(SEE ATTACHED SPREADSHEET FOR CALCULATION OF SEISMIC BASE SHEAR)

S_{DS} = 0.106 I_e = 1.00

ρ: REDUNDANCY (REF LATERAL DESIGN SECTION)   [12.3.4]

ρ = 1.0 FOR SEISMIC DESIGN CATEGORY B

SEISMIC LOADS (ASCE 7-16)

GENERAL INFORMATION:

S_s =	0.099	[FROM USGS WEBSITE]	
S_1 =	0.068	[FROM USGS WEBSITE]	
Site Class =	D	[FROM SOILS REPORT, OR ASSUMED]	
F_a =	1.6	[TABLE 11.4-1, or software]	
F_v =	2.4	[TABLE 11.4-2, or software]	
S_{MS} =	0.158	[EQ. 11.4-1]	
S_{M1} =	0.163	[EQ. 11.4-2]	
S_{DS} =	0.106	[EQ. 11.4-3]	
S_{D1} =	0.109	[EQ. 11.4-4]	
Risk Category	II	[TABLE 1.5-1]	
I_e =	1.00	[TABLE 1.5-2]	
Seismic Design Category =	**B**	[TABLES 11.6-1 & 11.6-2]	
R =	3.0	[TABLE 12.2-1]	*STEEL SYSTEM NOT SPECIFICALLY DETAILED FOR
Ω_o =	2.5	[TABLE 12.2-1]	SEISMIC RESISTANCE

SEISMIC BASE SHEAR: [Equivalent Lateral Force Procedure, SECTION 12.8]

C_s BOUND EQUATIONS:

			C_t =	0.020	[TABLE 12.8-2]
C_s =	0.035	[EQ. 12.8-2]	x =	0.75	[TABLE 12.8-2]
C_s =	0.251	[EQ. 12.8-3, 12.8-4]	h_n =	14.0	ft [12.8.2.1]
C_s =	0.010	[EQ. 12.8-5]	T_a =	0.14	sec [EQ. 12.8-7]
C_s =	NA	[EQ. 12.8-6]	T_L =	6.00	sec [FIG. 22-14]
C_s =	0.035	(GOVERNING VALUE)			
$V = C_sW$ =	**0.035**	**W**	[EQ. 12.8-1]	ULTIMATE LOAD (FACTORED)	
$V = C_sW$ =	**0.025**	**W**	[EQ. 12.8-1]	SERVICE LEVEL LOAD (ULTIMATE LOAD x 0.7)	

SEISMIC LOADS ON WALLS:

Building Wall [Design for Out-of-Plane Forces, SECTION 12.11.1]

F_p =	$0.4S_{DS}I_eW_p$	[12.11.1]			
S_{DS} =	0.106	[SEE ABOVE]	Ie =	1.00	[SEE ABOVE]
F_p =	0.042 Wp				
Min F_p =	0.100 Wp	[12.11.1]			
F_p =	0.100 Wp	(GOVERNING VALUE, FACTORED)			
W_p =	15	(PSF) 6" STUD WALL WITH EIFS/STUCCO			
W_p =	55	(PSF) 6" STUD WALL WITH BRICK VENEER			
6" WALL WITH EIFS/STUCCO F_p =	**1.1**	PSF	BY INSPECTION, WIND CONTROLS		
6" WALL WITH BRICK F_p =	**3.9**	PSF	BY INSPECTION, WIND CONTROLS		

Parapet [Seismic Demands on Nonstructural Components, SECTION 13.3]

$$F_p = \frac{0.4a_pS_{DS}W_p\,(1+2z/h)}{R_P/I_P} \quad \text{[EQ. 13.3-1]}$$

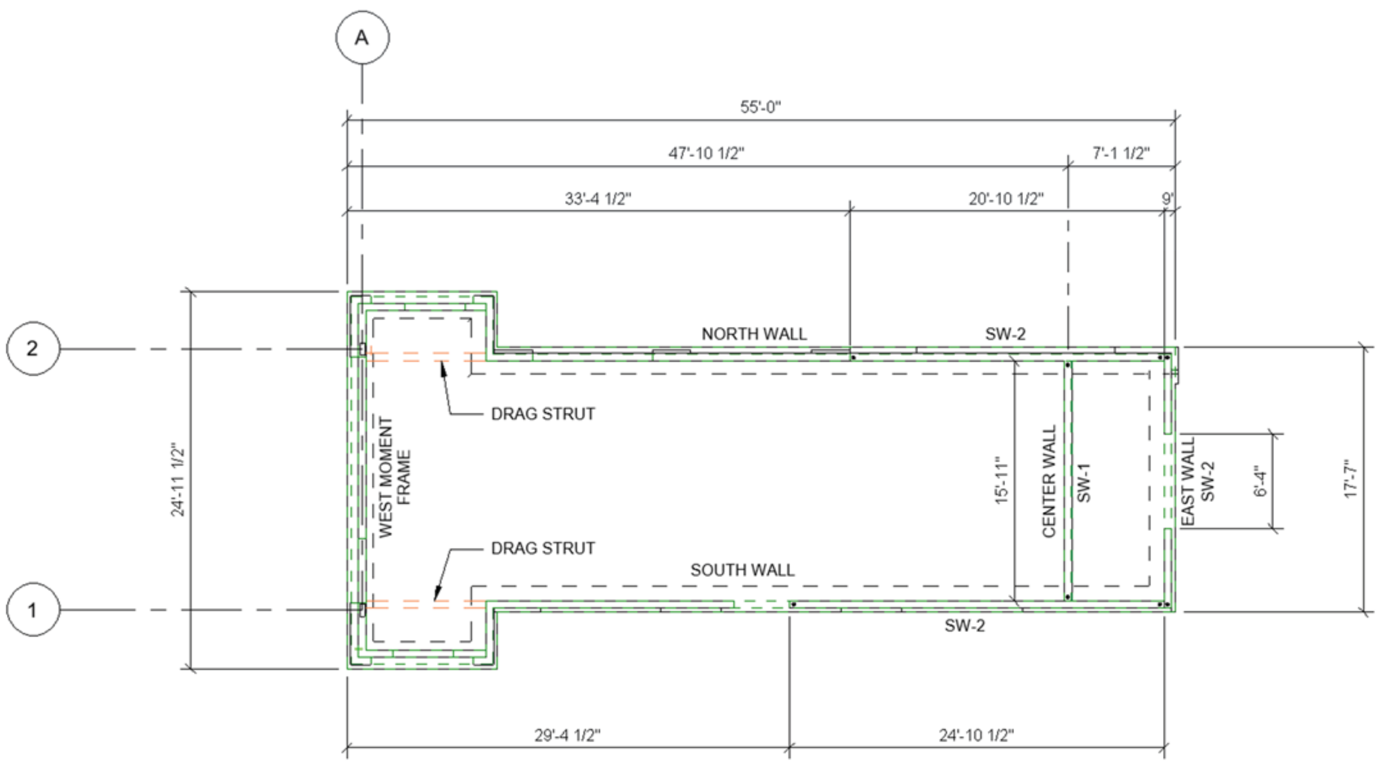
S_{DS} =	0.106	[See Above]	z/h =	1.00	
a_P =	2.5	[TABLE 13.5-1]	I_P =	1.0	[SECTION 13.1.3]
R_P =	2.5	[TABLE 13.5-1]			
F_p =	0.127 Wp	[EQ. 13.3-1]			
Max F_p = $1.6S_{DS}I_PW_P$ =	0.169 Wp	[EQ. 13.3-2]			
Min F_p = $0.3S_{DS}I_PW_P$ =	0.032 Wp	[EQ. 13.3-3]			
F_p =	0.127 Wp	(GOVERNING VALUE, FACTORED)			
W_p =	15	(PSF) 6" STUD WALL WITH EIFS/STUCCO			
W_p =	55	(PSF) 6" STUD WALL WITH BRICK VENEER			
6" WALL WITH EIFS/STUCCO F_p =	**1.3**	PSF	BY INSPECTION, WIND CONTROLS		
6" WALL WITH BRICK F_p =	**4.9**	PSF	BY INSPECTION, WIND CONTROLS		

LATERAL DESIGN

o DESIGN PHILOSOPHY:

- LATERAL DESIGN IN THE PLAN NORTH-SOUTH DIRECTION:
  LATERAL DESIGN IN THE PLAN NORTH-SOUTH DIRECTION CONSISTS OF A RECTANGULAR, FLEXIBLE SHEATHED DIAPHRAGM SPANNING BETWEEN A STEEL MOMENT FRAME AT THE WEST WALL AND SHEAR WALLS SHEATHED WITH WOOD STRUCTURAL PANELS ALONG THE CENTER AND EAST WALLS.

- LATERAL DESIGN IN THE PLAN EAST-WEST DIRECTION:
  LATERAL DESIGN IN THE PLAN EAST-WEST DIRECTION CONSISTS OF A RECTANGULAR, FLEXIBLE SHEATHED DIAPHRAGM SPANNING BETWEEN SHEAR WALLS SHEATHED WITH WOOD STRUCTURAL PANELS ALONG THE NORTH AND SOUTH WALLS.



o LATERAL ANALYSIS FOR WIND LOADS

LATERAL ANALYSIS FOR MWFRS WIND LOADS

EAST-WEST DIRECTION

MWFRS WALL WIND PRESSURES

HT (FT)	$P_{WINDWARD}$	$P_{LEEWARD}$
0-15	15.2	-4.5
15-20	16.1	-4.5
20-25	16.9	-4.5
25-30	17.6	-4.5

MWFRS PARAPET WIND PRESSURES

$P_{WINDWARD}$	$P_{LEEWARD}$
34.5	-23.0

BUILDING LENGTH = 55 ft
BUILDING WIDTH = 23 ft

WEST WALL

Wall Height	h (ft)	L (ft)	Avg. Roof Elev. (ft)	Parapet Ht. (ft)	Windward (0-15')	Windward (15-20')	Windward (20-25')	Windward (25-30')	Leeward	Parapet Windward	Parapet Leeward	Windward Force (plf)	Windward Force (kips)	Leeward Force (plf)	Leeward Force (kips)	Windward Moment	Leeward Moment
20'-2"	20.17	25.00	13.8	6.34	1454	0	0	0	-429	3723	-2482	374	**9.4**	-210	**-5.3**	117	-66
			0.0	0.00	0	0	0	0	0	0	0	0	**0.0**	0	**0.0**	0	0
			0.0	0.00	0	0	0	0	0	0	0	0	**0.0**	0	**0.0**	0	0
		25.0											**9.4**		**-5.3**	117	-66

EAST WALL

Wall Height	h (ft)	L (ft)	Avg. Roof Elev. (ft)	Parapet Ht. (ft)	Windward (0-15')	Windward (15-20')	Windward (20-25')	Windward (25-30')	Leeward	Parapet Windward	Parapet Leeward	Windward Force (plf)	Windward Force (kips)	Leeward Force (plf)	Leeward Force (kips)	Windward Moment	Leeward Moment
20'-2"	20.17	4.00	13.8	6.34	1454	0	0	0	-429	3723	-2482	374	**1.5**	-210	**-0.8**	3	-2
18'-2"	18.17	17.00	13.8	4.34	1454	0	0	0	-429	2398	-1599	279	**4.7**	-147	**-2.5**	59	-31
20'-2"	20.17	4.00	13.8	6.34	1454	0	0	0	-429	3723	-2482	374	**1.5**	-210	**-0.8**	34	-19
		25.0											**7.7**		**-4.2**	97	-52

WEST WIND TOTAL FORCE = 13.5 K
EAST WIND TOTAL FORCE = 13.0 K

WEST FORCES (LRFD)			EAST FORCES (LRFD)		
V_{SOUTH} =	6.8	K	V_{SOUTH} =	6.5	K
V_{NORTH} =	6.8	K	V_{NORTH} =	6.5	K
MOMENT =	38	K-FT	MOMENT =	37	K-FT
WEST DIAPHRAGM SHEARS (LRFD)			**EAST DIAPHRAGM SHEARS (LRFD)**		
v_{SOUTH} =	124	plf	v_{SOUTH} =	119	plf
v_{NORTH} =	124	plf	v_{NORTH} =	119	plf

WEST FORCES (ASD)			EAST FORCES (ASD)		
V_{SOUTH} =	4.1	K	V_{SOUTH} =	3.9	K
V_{NORTH} =	4.1	K	V_{NORTH} =	3.9	K
MOMENT =	23	K-FT	MOMENT =	22	K-FT
WEST DIAPHRAGM SHEARS (ASD)			**EAST DIAPHRAGM SHEARS (ASD)**		
v_{SOUTH} =	74	plf	v_{SOUTH} =	71	plf
v_{NORTH} =	74	plf	v_{NORTH} =	71	plf

NORTH-SOUTH DIRECTION

MWFRS WALL WIND PRESSURES

HT (FT)	$P_{WINDWARD}$	$P_{LEEWARD}$
0-15	15.2	-9.2
15-20	16.1	-9.2
20-25	16.9	-9.2
25-30	17.6	-9.2

MWFRS PARAPET WIND PRESSURES

$P_{WINDWARD}$	$P_{LEEWARD}$
34.5	-23.0

AREA 1

SOUTH WALL

Wall Height	h (ft)	L (ft)	Avg. Roof Elev. (ft)	Parapet Ht. (ft)	Windward (0-15')	Windward (15-20')	Windward (20-25')	Windward (25-30')	Leeward	Parapet Windward	Parapet Leeward	Windward Force (plf)	Windward Force (kips)	Leeward Force (plf)	Leeward Force (kips)	Windward Moment	Leeward Moment
20'-2"	20.17	10.00	14.0	6.17	1489	0	0	0	-901	3641	-2427	366	**3.7**	-238	**-2.4**	18	-12
18'-2"	18.17	37.80	14.0	4.17	1489	0	0	0	-901	2317	-1545	272	**10.3**	-175	**-6.6**	297	-191
			0.0	0.00	0	0	0	0	0	0	0	0	**0.0**	0	**0.0**	0	0
		47.8											**13.9**		**-9.0**	315	-203

NORTH WALL

Wall Height	h (ft)	L (ft)	Avg. Roof Elev. (ft)	Parapet Ht. (ft)	Windward (0-15')	Windward (15-20')	Windward (20-25')	Windward (25-30')	Leeward	Parapet Windward	Parapet Leeward	Windward Force (plf)	Windward Force (kips)	Leeward Force (plf)	Leeward Force (kips)	Windward Moment	Leeward Moment
20'-2"	20.17	10.00	13.7	6.50	1420	0	0	0	-859	3799	-2533	382	**3.8**	-248	**-2.5**	19	-12
18'-2"	18.17	23.30	13.7	4.50	1420	0	0	0	-859	2474	-1650	285	**6.6**	-184	**-4.3**	144	-93
20'-2"	20.17	14.50	13.7	6.50	1420	0	0	0	-859	3799	-2533	382	**5.5**	-248	**-3.6**	224	-146
		47.8											**16.0**		**-10.4**	387	-251

SOUTH WIND TOTAL FORCE = 24.3 K

NORTH WIND TOTAL FORCE = 25.0 K

SOUTH FORCES (LRFD)			NORTH FORCES (LRFD)		
V_{CENTER} =	11.8	K	V_{CENTER} =	11.8	K
V_{WEST} =	12.5	K	V_{WEST} =	13.2	K
MOMENT =	166	K-FT	MOMENT =	171	K-FT
SOUTH DIAPHRAGM SHEARS (LRFD)			NORTH DIAPHRAGM SHEARS (LRFD)		
V_{CENTER} =	697	plf	V_{CENTER} =	695	plf
v_{WEST} =	553	plf	v_{WEST} =	585	plf

SOUTH FORCES (ASD)			NORTH FORCES (ASD)		
V_{CENTER} =	7.1	K	V_{CENTER} =	7.1	K
V_{WEST} =	7.5	K	V_{WEST} =	7.9	K
MOMENT =	100	K-FT	MOMENT =	102	K-FT
SOUTH DIAPHRAGM SHEARS (ASD)			NORTH DIAPHRAGM SHEARS (ASD)		
V_{CENTER} =	418	plf	V_{CENTER} =	417	plf
v_{WEST} =	332	plf	v_{WEST} =	351	plf

AREA 2

SOUTH WALL

Wall Height	h (ft)	L (ft)	Avg. Roof Elev. (ft)	Parapet Ht. (ft)	Windward (0-15')	Windward (15-20')	Windward (20-25')	Windward (25-30')	Leeward	Parapet Windward	Parapet Leeward	Windward Force (plf)	Windward Force (kips)	Leeward Force (plf)	Leeward Force (kips)	Windward Moment	Leeward Moment
18'-2"	18.17	7.20	14.0	4.17	1489	0	0	0	-901	2317	-1545	272	**2.0**	-175	**-1.3**	7	-5
		7.2											**2.0**		**-1.3**	7	-5

NORTH WALL

Wall Height	h (ft)	L (ft)	Avg. Roof Elev. (ft)	Parapet Ht. (ft)	Windward (0-15')	Windward (15-20')	Windward (20-25')	Windward (25-30')	Leeward	Parapet Windward	Parapet Leeward	Windward Force (plf)	Windward Force (kips)	Leeward Force (plf)	Leeward Force (kips)	Windward Moment	Leeward Moment
20'-2"	20.17	7.20	13.7	6.50	1420	0	0	0	-859	3799	-2533	382	**2.7**	-248	**-1.8**	10	-6
		7.2											**2.7**		**-1.8**	10	-6

SOUTH WIND TOTAL FORCE = 3.7 K

NORTH WIND TOTAL FORCE = 4.0 K

SOUTH FORCES (LRFD)			NORTH FORCES (LRFD)		
V_{EAST} =	1.9	K	V_{EAST} =	2.0	K
V_{CENTER} =	1.9	K	V_{CENTER} =	2.0	K
MOMENT =	26	K-FT	MOMENT =	27	K-FT
SOUTH DIAPHRAGM SHEARS (LRFD)			NORTH DIAPHRAGM SHEARS (LRFD)		
v_{EAST} =	110	plf	v_{EAST} =	89	plf
v_{CENTER} =	110	plf	v_{CENTER} =	89	plf

SOUTH FORCES (ASD)			NORTH FORCES (ASD)		
V_{EAST} =	1.1	K	V_{EAST} =	1.2	K
V_{CENTER} =	1.1	K	V_{CENTER} =	1.2	K
MOMENT =	15	K-FT	MOMENT =	16	K-FT
SOUTH DIAPHRAGM SHEARS (ASD)			NORTH DIAPHRAGM SHEARS (ASD)		
v_{EAST} =	66	plf	v_{EAST} =	53	plf
v_{CENTER} =	66	plf	v_{CENTER} =	53	plf

SUMMARY OF WIND FORCES INTO SHEAR WALLS

LRFD		ASD	
V_{NORTH} =	**6.8 K**	V_{NORTH} =	**4.1 K**
V_{SOUTH} =	**6.8 K**	V_{SOUTH} =	**4.1 K**
V_{WEST} =	**13.2 K**	V_{WEST} =	**7.9 K**
V_{CENTER} =	**13.8 K**	V_{CENTER} =	**8.3 K**
V_{EAST} =	**2.00 K**	V_{EAST} =	**1.20 K**

- SEISMIC LOADS IN DIAPHRAGM:
  (SEE ATTACHED CALCULATION, "SEISMIC EFFECTIVE WEIGHT CALCULATIONS" FOR APPLICABLE WALL AND ROOF WEIGHTS IN THE FOLLOWING CALCULATIONS, PER ASCE 7-16, SEC. 12.7.2)

  STRUCTURAL IRREGULARITY [TABLE 12.3-1]
  RE-ENTRANT CORNER IRREGULARITY DOES NOT EXIST.

  DIAPHRAGMS, CHORDS, AND COLLECTORS [12.10.1.1]
  PER ASCE 7-16, DIAPHRAGM FORCE $F_p = V = C_sW$
  FROM LOAD CRITERIA SHEET, $C_s = 0.035$
  $V = C_sW = 0.035\ W$

  THIS FORCE NEED NOT EXCEED $0.4S_{DS}Iw_{px} = 0.4(0.106)(1.0)w_{px} = 0.042w_{px}$
  BUT SHALL NOT BE LESS THAN $0.2S_{DS}Iw_{px} = 0.2(0.106)(1.0)w_{px} = 0.021w_{px}$
  ∴ USE 0.035W

  REDUNDANCY [12.3.4]
  $\rho = 1.0$ PER SEC. 12.3.4.1

- TOTAL SEISMIC LOAD IN E-W DIRECTION
  $V = 0.035 \times W \times \rho$
  $V_{EW} = 0.035 \times 35.5\ \text{kip} \times 1.0 = 1.24\ \text{K (LRFD)}$

- FORCES IN LINES OF RESISTANCE DUE TO SEISMIC (LRFD)
  $V_{NORTH} = V_{SOUTH} = 1.24\ K / 2 =$ **0.62 K**

- DIAPHRAGM SHEARS DUE TO SEISMIC (LRFD)
  $v_{NORTH} = v_{SOUTH} = 620\ \text{lbs} / 54.5\ \text{ft} = 11\ \text{lb/ft}$

∴ BY INSPECTION, WIND CONTROLS THE DESIGN IN THIS DIRECTION

- TOTAL SEISMIC LOAD IN N-S DIRECTION
  $V = 0.035 \times W \times \rho$
  $V_{NS} = 0.035 \times 49.5\ \text{kip} \times 1.0 = 1.74\ \text{K (LRFD)}$

- FORCES IN LINES OF RESISTANCE DUE TO SEISMIC (LRFD)
  $V_{WEST} = V_{CENTER} = 1.74\ K / 2 =$ **0.87 K**

- DIAPHRAGM SHEARS DUE TO SEISMIC (LRFD)
  $v_{WEST} = v_{CENTER} = 870\ \text{lbs} / 17.0\ \text{ft} = 52\ \text{lb/ft}$

∴ BY INSPECTION, WIND CONTROLS THE DESIGN IN THIS DIRECTION

SEISMIC EFFECTIVE WEIGHT CALCULATIONS

North-South Wall Effective Weight, W [A7-16, 12.7.2]							
Location	Wall Type	h (ft)	L (ft)	Weight (psf)	Avg. Roof Elev. (ft)	Full Wall Weight (lb)	Weight Trib. To Roof (lb)
20'-2" South Wall	6" Stud	20.17	10.00	20	14.00	4034	2634
18'-2" South Wall	6" Stud	18.17	45.000	20	14.00	16353	10053
20'-2" North Wall	6" Stud	20.17	33.300	20	13.67	13433	8881
18'-2" North Wall	6" Stud	18.17	21.700	20	13.67	7886	4919
						0	0
						0	0
						0	0
			55.0			TOTAL	**26.5 K**

East-West Wall Effective Weight, W [A7-16, 12.7.2]							
Location	Wall Type	h (ft)	L (ft)	Weight (psf)	Avg. Roof Elev. (ft)	Full Wall Weight (lb)	Weight Trib. To Roof (lb)
20'-2" West Wall	6" Stud	20.17	25.000	20	13.83	10085	6628
18'-2" East Wall	6" Stud	18.17	17.700	20	13.83	6432	3984
20'-2" East Wall	6" Stud	20.17	7.300	20	13.83	2945	1935
						0	0
						0	0
						0	0
			25.0			TOTAL	**12.5 K**

Roof Effective Weight, W [A7-16, 12.7.2]					
Area	Surface Area (SF)	DL (psf)	SL (psf)	Permanent Equipment (lb)	Total Weight (K)
1	900	20.0	0.0	5000	**23.0**

TOTAL WEIGHT (K) - DIAPHRAGM + OUT OF PLANE SHEAR WALLS	
North-South	**49.5**
East-West	**35.5**

LATERAL ANALYSIS FOR SEISMIC LOADS

$V = C_S W$ $C_S =$ 0.035

EAST-WEST DIRECTION

E-W WEIGHT, W = 35.5 K
E-W TOTAL SEISMIC FORCE, V = **1.24** K
E-W TOTAL WIND FORCE = 13.53 K <- FROM WIND ANALYSIS
--WIND CONTROLS--

NORTH-SOUTH DIRECTION

N-S WEIGHT, W = 49.5 K
N-S TOTAL SEISMIC FORCE, V = **1.73** K
N-S TOTAL WIND FORCE = 29.0 K <- FROM WIND ANALYSIS
--WIND CONTROLS--

o ROOF DECK DESIGN

DESIGN DECK FOR GRAVITY LOADS

MAX. UNIFORM GRAVITY LOAD = 20 PSF DL + 35 PSF SL
= 55 PSF TL

JOIST SPACING = 2'-8" MAX
WIND UPLIFT FORCE (ASD) = 32.2 PSF

TRY 19/32" WOOD STRUCTURAL I PANELS WITH A 40/20 SPAN RATING
DOWNWARD ALLOWABLE UNIFORM LOAD = 78 PSF > 55 PSF ***OK***

TABLE 2b

UNIFORM LOADS (psf) ON APA RATED OSB STRUCTURAL I SHEATHING.
MULTI-SPAN, NORMAL DURATION OF LOAD, DRY CONDITIONS, PANELS 24 INCHES OR WIDER

Span Rating[b]	Load Governed By[c]	Strength Axis[a]: Perpendicular to Supports, Span Center-to-Center of Supports (inches)										Parallel to Supports, Span Center-to-Center of Supports (inches)		
		12	16	19.2	24	30	32	36	40	48	60	12	16	24
24/0	L/360	261	98	54	26	13	10					77	29	10
	L/240	392	147	81	39	19	16					115	43	15
	L/180	522	196	107	52	26	21					153	58	19
	Bending	250	141	98	63	40	35					121	68	24
	Shear	248	179	147	116	91	85					248	179	111
24/16	L/360	339	128	70	34	17	14	12				111	42	14
	L/240	509	191	105	51	25	20	18				167	63	21
	L/180	679	255	140	68	33	27	24				223	84	28
	Bending	321	180	125	80	51	45	29				144	81	29
	Shear	286	207	169	133	105	98	83				286	207	128
32/16	L/360	500	188	103	50	24	20	18	13			174	65	22
	L/240	750	282	154	75	37	30	26	19			261	98	33
	L/180	1,001	376	206	100	49	40	35	25			348	131	44
	Bending	371	209	145	93	59	52	33	27			206	116	41
	Shear	314	228	186	147	116	108	92	82			314	228	141
40/20	L/360	979	368	201	98	48	39	34	25	16		390	147	50
	L/240	1,468	552	302	146	72	58	51	37	24		585	220	74
	L/180	1,958	736	403	195	96	78	69	49	32		780	293	99
	Bending	625	352	244	156	100	88	56	45	31		338	190	68
	Shear	390	283	232	182	144	134	114	102	88		390	283	175

UPWARD ALLOWABLE LOAD = 150 PSF / 1.6 = 94 PSF > 32.2 PSF ***OK***

Table 3.2.2 Nominal Uniform Load Capacities (psf) for Roof Sheathing Resisting Out-of-Plane Wind Loads[1,2,6]

Sheathing Type[5]	Span Rating or Grade	Minimum Thickness (in.)	Strength Axis[7] Applied Perpendicular to Supports						Strength Axis[7] Applied Parallel to Supports		
			Rafter/Truss Spacing (in.)						Rafter/Truss Spacing (in.)		
			12	16	19.2	24	32	48	12	16	24
			Nominal Uniform Loads (psf)						Nominal Uniform Loads (psf)		
Wood Structural Panels (Sheathing Grades, C-C, C-D, C-C Plugged, OSB)	24/0	3/8	425	240	165	105	-	-	90	50	30[3]
	24/16	7/16	540	305	210	135	-	-	110	60	35[3]
	32/16	15/32	625	355	245	155	90	-	155	90	45[3]
	40/20	19/32	955	595	415	265	150	-	255	145	75[3]
	48/24	23/32	1160[3]	840[3]	615[3]	395[3]	220[3]	100[3]	455[3]	255[3]	115[3]
Wood Structural Panels (Single Floor Grades, Underlayment, C-C Plugged)	16 o.c.	19/32	705	395	275	175	100	-	170	95	50[3]
	20 o.c.	19/32	815	455	320	205	115	-	235	135	70[3]
	24 o.c.	23/32	1160[3]	670[3]	465[3]	300[3]	170[3]	-	440[3]	250[3]	110[3]
	32 o.c.	7/8	1395[4]	1000[4]	695[4]	445[4]	250[4]	110[4]	1160[4]	655[4]	290[4]
	48 o.c.	1-1/8	1790[4]	1295[4]	1060[4]	805[4]	455[4]	200[4]	1790[4]	1145[4]	510[4]

ROOF SHEATHING TO BE ATTACHED WITH 10d COMMON NAILS (DIA=0.148") AT 8" OC MAX
MAX UPLIFT LOAD PER SCREW = 32.2 PSF x 2.67' OC x 8"/12 SPACING = 55 LBS
ALLOWABLE WITHDRAWAL OF 10d NAIL WITH 1.5" PENETRATION

W'=WCdCmCtCegCtn [NDS TABLE 11.3.1]

W = 1380 $G^{5/2}$ D [EQ. 12.2-3]

G = 0.42 (ASSUME SPF)

D = 0.148" (10d NAILS)

W = 1380$(0.42)^{5/2}$ (0.148)(1.5") = 35 LBS

Cd = 1.6

Cm = 1.0 Ceg = N/A

Ct = 1.0 Ctn = N/A

W' = 35 LBS x 1.6 x 1.0 x 1.0 = 56 LBS > 55 LBS ***OK***

DESIGN DECK FOR DIAPHRAGM SHEARS

ROOF DIAPHRAGM ASPECT RATIO = 46.5 FT / 16.92 FT = 2.7 < 3.0 ***OK*** [SDPWS, TABLE 4.2.4]

MAX DIAPHRAGM SHEAR v_{MAX} = 418 lb/ft (WIND, ASD, N-S DIRECTION, CENTER WALL)

IN N-S DIRECTION, LONG DIMENSION OF SHEATHING PANELS IS PERPENDICULAR TO SUPPORTS, SO USE CASE 1

FROM SDPWS TABLE 4.2A BLOCKED DIAPHRAGM,

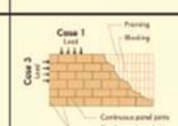
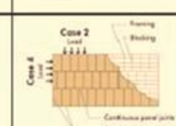
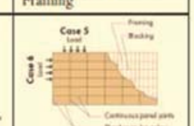
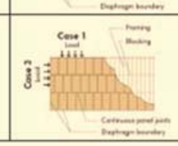
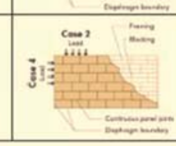
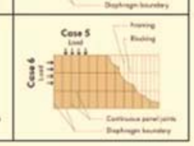
Table 4.2A Nominal Unit Shear Capacities for Wood-Frame Diaphragms

Blocked Wood Structural Panel Diaphragms[1,2,3,4,5]

A SEISMIC / B WIND — Nail Spacing (in.) at diaphragm boundaries (all cases), at continuous panel edges parallel to load (Cases 3 & 4), and at all panel edges (Cases 5 & 6): 6 / 4 / 2-1/2 / 2; Nail Spacing (in.) at other panel edges (Cases 1, 2, 3, & 4): 6 / 6 / 4 / 3

Sheathing Grade	Common Nail Size	Minimum Fastener Penetration in Framing Member or Blocking (in.)	Minimum Nominal Panel Thickness (in.)	Minimum Nominal Width of Nailed Face at Adjoining Panel Edges and Boundaries (in.)	A Seismic 6/6 v_s (plf)	6/6 G_a (kips/in.) OSB	6/6 G_a PLY	4/6 v_s (plf)	4/6 G_a OSB	4/6 G_a PLY	2-1/2/4 v_s (plf)	2-1/2/4 G_a OSB	2-1/2/4 G_a PLY	2/3 v_s (plf)	2/3 G_a OSB	2/3 G_a PLY	B Wind 6/6 v_w (plf)	4/6 v_w (plf)	2-1/2/4 v_w (plf)	2/3 v_w (plf)
Structural I	6d	1-1/4	5/16	2	370	15	12	500	8.5	7.5	750	12	10	840	20	15	520	700	1050	1175
				3	420	12	9.5	560	7.0	6.0	840	9.5	8.5	950	17	13	590	785	1175	1330
	8d	1-3/8	3/8	2	540	14	11	720	9.0	7.5	1060	13	10	1200	21	15	755	1010	1485	1680
				3	600	12	10	800	7.5	6.5	1200	10	9.0	1350	18	13	840	1120	1680	1890
	10d	1-1/2	15/32	2	640	24	17	850	15	12	1280	20	15	1460	31	21	895	1190	1790	2045
				3	720	20	15	960	12	9.5	1440	16	13	1640	26	18	1010	1345	2015	2295
Sheathing and Single-Floor	6d	1-1/4	5/16	2	340	15	10	450	9.0	7.0	670	13	9.5	760	21	13	475	630	940	1065
				3	380	12	9.0	500	7.0	6.0	760	10	8.0	860	17	12	530	700	1065	1205
			3/8	2	370	13	9.5	500	7.0	6.0	750	10	8.0	840	18	12	520	700	1050	1175
				3	420	10	8.0	560	5.5	5.0	840	8.5	7.0	950	14	10	590	785	1175	1330
	8d	1-3/8	3/8	2	480	15	11	640	9.5	7.5	960	13	9.5	1090	21	13	670	895	1345	1525
				3	540	12	9.5	720	7.5	6.0	1080	11	8.5	1220	18	12	755	1010	1510	1710
			7/16	2	510	14	10	680	8.5	7.0	1010	12	9.5	1150	20	13	715	950	1415	1610
				3	570	11	9.0	760	7.0	6.0	1140	10	8.0	1290	17	12	800	1065	1595	1805
			15/32	2	540	13	9.5	720	7.5	6.5	1060	11	8.5	1200	19	13	755	1010	1485	1680
				3	600	10	8.5	800	6.0	5.5	1200	9.0	7.5	1350	15	11	840	1120	1680	1890
	10d	1-1/2	15/32	2	580	25	15	770	15	11	1150	21	14	1310	33	18	810	1080	1610	1835
				3	650	21	14	860	12	9.5	1300	17	12	1470	28	16	910	1205	1820	2060
			19/32	2	640	21	14	850	13	9.5	1280	18	12	1460	28	17	895	1190	1790	2045
				3	720	17	12	960	10	8.0	1440	14	11	1640	24	15	1010	1345	2015	2295

1. Nominal unit shear capacities shall be adjusted in accordance with 4.2.3 to determine ASD allowable unit shear capacity and LRFD factored unit resistance. For general construction requirements see 4.2.6. For specific requirements, see 4.2.7.1 for wood structural panel diaphragms. See Appendix A for common nail dimensions.
2. For species and grades of framing other than Douglas-Fir-Larch or Southern Pine, reduced nominal unit shear capacities shall be determined by multiplying the tabulated nominal unit shear capacity by the Specific Gravity Adjustment Factor = [1-(0.5-G)], where G = Specific Gravity of the framing lumber from the *NDS* (Table 12.3.3A). The Specific Gravity Adjustment Factor shall not be greater than 1.
3. Apparent shear stiffness values, G_a, are based on nail slip in framing with moisture content less than or equal to 19% at time of fabrication and panel stiffness values for diaphragms constructed with either OSB or 3-ply plywood panels. When 4-ply or 5-ply plywood panels or composite panels are used, G_a values shall be permitted to be multiplied by 1.2.
4. Where moisture content of the framing is greater than 19% at time of fabrication, G_a values shall be multiplied by 0.5.
5. Diaphragm resistance depends on the direction of continuous panel joints with respect to the loading direction and direction of framing members, and is independent of the panel orientation.

	Cases 1&3: Continuous Panel Joints Perpendicular to Framing	**Cases 2&4**: Continuous Panel Joints Parallel to Framing	**Cases 5&6**: Continuous Panel Joints Perpendicular and Parallel to Framing
Long Panel Direction Perpendicular to Supports			
Long Panel Direction Parallel to Supports[a]			

(a) Panel span rating for out-of-plane loads may be lower than the span rating with the long panel direction perpendicular to supports (See Section 3.2.2 and Section 3.2.3)

DECK DIAPHRAGM CAPACITY = 895 PLF (ULTIMATE)

ALLOWABLE DIAPHRAGM CAPACITY = 895 PLF / 2.0 = 448 PLF < 418 PLF ***OK***

FROM SDPWS TABLE 4.2C UNBLOCKED DIAPHRAGM,

Table 4.2C Nominal Unit Shear Capacities for Wood-Frame Diaphragms

Unblocked Wood Structural Panel Diaphragms[1,2,3,4,5]

Sheathing Grade	Common Nail Size	Minimum Fastener Penetration in Framing (in.)	Minimum Nominal Panel Thickness (in.)	Minimum Nominal Width of Nailed Face at Supported Edges and Boundaries (in.)	A SEISMIC – 6 in. Nail Spacing at diaphragm boundaries and supported panel edges – Case 1 – v_s (plf)	Case 1 – G_a (kips/in.) OSB	Case 1 – G_a (kips/in.) PLY	Cases 2,3,4,5,6 – v_s (plf)	Cases 2,3,4,5,6 – G_a (kips/in.) OSB	Cases 2,3,4,5,6 – G_a (kips/in.) PLY	B WIND – 6 in. Nail Spacing at diaphragm boundaries and supported panel edges – Case 1 – v_w (plf)	Cases 2,3,4,5,6 – v_w (plf)
Structural I	6d	1-1/4	5/16	2	330	9.0	7.0	250	6.0	4.5	460	350
				3	370	7.0	6.0	280	4.5	4.0	520	390
	8d	1-3/8	3/8	2	480	8.5	7.0	360	6.0	4.5	670	505
				3	530	7.5	6.0	400	5.0	4.0	740	560
	10d	1-1/2	15/32	2	570	14	10	430	9.5	7.0	800	600
				3	640	12	9.0	480	8.0	6.0	895	670
Sheathing and Single-Floor	6d	1-1/4	5/16	2	300	9.0	6.5	220	6.0	4.0	420	310
				3	340	7.0	5.5	250	5.0	3.5	475	350
			3/8	2	330	7.5	5.5	250	5.0	4.0	460	350
				3	370	6.0	4.5	280	4.0	3.0	520	390
	8d	1-3/8	3/8	2	430	9.0	6.5	320	6.0	4.5	600	450
				3	480	7.5	5.5	360	5.0	3.5	670	505
			7/16	2	460	8.5	6.0	340	5.5	4.0	645	475
				3	510	7.0	5.5	380	4.5	3.5	715	530
			15/32	2	480	7.5	5.5	360	5.0	4.0	670	505
				3	530	6.5	5.0	400	4.0	3.5	740	560
	10d	1-1/2	15/32	2	510	15	9.0	380	10	6.0	715	530
				3	580	12	8.0	430	8.0	5.5	810	600
			19/32	2	570	13	8.5	430	8.5	5.5	800	600
				3	640	10	7.5	480	7.0	5.0	895	670

DECK DIAPHRAGM CAPACITY = 800 PLF (ULTIMATE)
ALLOWABLE DIAPHRAGM CAPACITY = 800 PLF / 2.0 = 400 PLF < 395 PLF ***OK***

BLOCKED DIAPHRAGM REQUIRED ON PLAN NORTH SIDE OF CENTER SHEAR WALL. DETERMINE DISTANCE REQUIRED TO TRANSITION FROM BLOCKED TO UNBLOCKED DIAPHRAGM.

DIAPHRAGM SHEAR OVER DISTANCE
DISTANCE FROM GRID = 1.3 FT

SOUTH FORCES (ASD)		NORTH FORCES (ASD)	
V_{CENTER} =	0.4 K	V_{CENTER} =	0.4 K
V_{WEST} =	0.4 K	V_{WEST} =	0.4 K
V_{CENTER} =	395 plf	V_{CENTER} =	393 plf
v_{WEST} =	314 plf	v_{WEST} =	333 plf

∴ **<u>USE 19/32" WOOD STRUCTURAL I PANELS WITH A SPAN RATING OF 40/20, BLOCKED WITH SCREWS SPACED AT 6" O.C. MAX AT PANEL EDGES AND 8" OC MAX IN THE FIELD OF THE PANEL AT INTERMEDIATE SUPPORTS IN JOIST SPACE PLAN NORTH OF CENTER SHEAR WALL</u>**

∴ **<u>AT ALL OTHER LOCATIONS, USE 19/32" WOOD STRUCTURAL I PANELS WITH A SPAN RATING OF 40/20, UNBLOCKED WITH SCREWS SPACED AT 6" O.C. MAX AT PANEL EDGES AND 8" OC MAX IN THE FIELD OF THE PANEL AT INTERMEDIATE SUPPORTS</u>**

o FIND CHORD FORCES:

EAST-WEST WIND

$M_{EW,\ MAX}$ = 23 kip_ft (ASD)

$T_{EW,\ MAX} = M_{EW,\ MAX}$ / 53.0 ft = 0.44 kips

NORTH-SOUTH WIND

$M_{NS,\ MAX}$ = 102 kip_ft (ASD)

$T_{NS,\ MAX} = M_{NS,\ MAX}$ / 17 ft = 6.0 kips

o FIND COLLECTOR FORCES:

NORTH WALL FORCE = 4.1 K (ASD)

NORTH WALL COLLECTOR FORCE = 2.52 K (ASD)

Drag / Collector Force Diagram Generator

INPUT DATA

TOTAL SHEAR FORCE (ASD) F_p = 4.1 kips

NUMBER OF SEGMENTS n = 2

Segment	1	2
Length, ft	33.3	20.9
Shear Wall ?	NO	YES

ANALYSIS

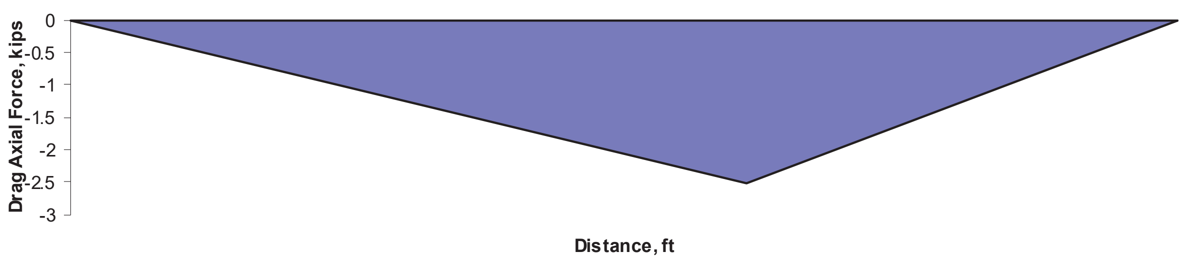
TOTAL DRAG LENGTH L_{drag} = 54.2 ft

TOTAL SHEAR WALL LENGTH L_{wall} = 20.9 ft

DIAPHRAGM SHEAR STRESS $v_{diaphragm} = F_p / v_{drag}$ = 76 plf

SHEAR WALL SHEAR STRESS $v_{shear\ wall} = F_p / v_{wall}$ = 196 plf

Section Point	0	1	2
Distance, ft	0	33.3	54.2
Axial Force	0	-2.52	0.00



DRAG / COLLECTOR FORCE DIAGRAM

SOUTH WALL FORCE = 3.4 K (ASD)
SOUTH WALL COLLECTOR FORCE = 2.22 K (ASD)

Drag / Collector Force Diagram Generator

INPUT DATA

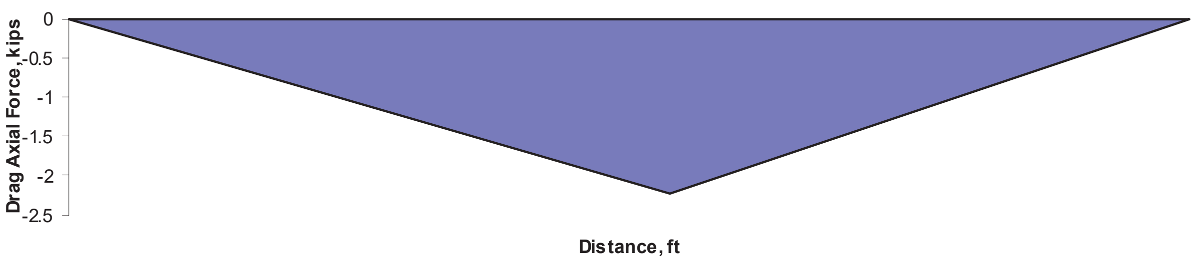
TOTAL SHEAR FORCE (ASD)	F_p =	4.1	kips
NUMBER OF SEGMENTS	n =	2	

Segment	1	2	
Length, ft	29.3	24.8	
Shear Wall ?	NO	YES	0

ANALYSIS

TOTAL DRAG LENGTH	L_{drag} =	54.1	ft
TOTAL SHEAR WALL LENGTH	L_{wall} =	24.8	ft
DIAPHRAGM SHEAR STRESS	$v_{diaphragm} = F_p / v_{drag}$ =	76	plf
SHEAR WALL SHEAR STRESS	$v_{shear\ wall} = F_p / v_{wall}$ =	165	plf

Section Point	0	1	2
Distance, ft	0	29.3	54.1
Axial Force	0	-2.22	0.00



DRAG / COLLECTOR FORCE DIAGRAM

EAST WALL FORCE = 1.20 K (ASD)
EAST WALL COLLECTOR FORCE = 0.22 K (ASD)

Drag / Collector Force Diagram Generator

INPUT DATA

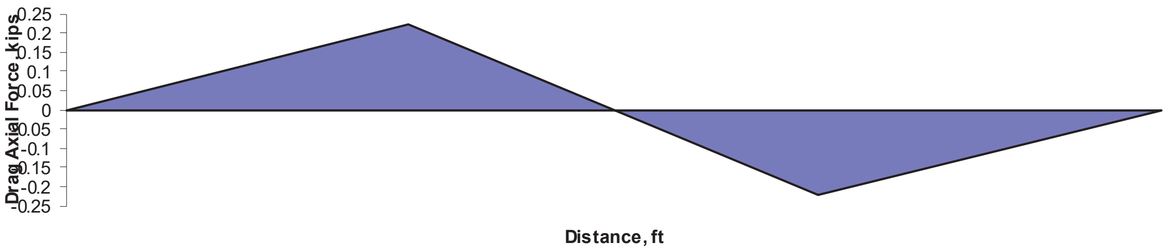
TOTAL SHEAR FORCE (ASD)	F_p =	1.2	kips
NUMBER OF SEGMENTS	n =	3	

Segment	1	2	3
Length, ft	5.3	6.3	5.3
Shear Wall ?	YES	NO	YES

ANALYSIS

TOTAL DRAG LENGTH	L_{drag} =	16.9	ft
TOTAL SHEAR WALL LENGTH	L_{wall} =	10.6	ft
DIAPHRAGM SHEAR STRESS	$v_{diaphragm} = F_p / v_{drag}$ =	71	plf
SHEAR WALL SHEAR STRESS	$v_{shear\ wall} = F_p / v_{wall}$ =	113	plf

Section Point	0	1	2	3
Distance, ft	0	5.3	11.6	16.9
Axial Force	0	0.22	-0.22	0.00



DRAG / COLLECTOR FORCE DIAGRAM

WEST WALL COLLECTOR FORCE = NONE, MOMENT FRAME HERE

CENTER WALL COLLECTOR FORCE = NONE, ENTIRE WALL IS A SHEAR WALL o DESIGN CHORDS/COLLECTORS:

WORST CASE CHORD/COLLECTOR FORCE = 6.0 K (N-S CHORD FORCE)

∴ **USE CONT 2x6 CHORD/COLLECTOR ON ALL WALLS**

COMPRESSION: P_A = (1,350 psi x 1.6 x 1.0 x 1.1 x 1.0 x 1.0 x 0.39) x 1.5" x 5.5" = 7.7 kip
TENSION: T_A = (575 psi x 1.6 x 1.0 x 1.0 x 1.3) x 1.5" x 5.5" = 9.9 kip

o DESIGN OF CHORD/COLLECTOR SPLICES:

WORST CASE CHORD/COLLECTOR FORCE = 6.0 K (N-S CHORD FORCE)

∴ **SIMPSON MSTI48 STRAP WITH (48) 0.148 x 1½" NAILS**

STRAP CAPACITY = 3,800 x 2 = 7.6 K

MSTI26	12	2 1/16	26	(26) 0.148 x 1½	2,745	2,380
MSTI36		2 1/16	36	(36) 0.148 x 1½	3,800	3,295
MSTI48		2 1/16	48	(48) 0.148 x 1½	5,070	4,390
MSTI60		2 1/16	60	(60) 0.148 x 1½	5,070	5,070
MSTI72		2 1/16	72	(72) 0.148 x 1½	5,070	5,070

o SUMMARY OF FORCES IN SHEAR WALLS, SEE ATTACHED OUTPUT:

- NORTH WALL
  L = 20.8'
  H = 13.7'
  F = 4.06 K (ASD)
  v = 195 PLF
  END POST, C = 2.7 K
  HOLD DOWN FORCE, T = 1.0 K

- SOUTH WALL
  L = 24.0'
  H = 14.0'
  F = 4.06 K (ASD)
  v = 164 PLF
  END POST, C = 2.3 K
  HOLD DOWN FORCE, T = 0.3 K

- WEST WALL
  SEE MOMENT FRAME DESIGN

- CENTER WALL
  L = 15.9'
  H = 13.8'
  F = 8.29 K (ASD)
  v = 521 PLF
  END POST, C = 7.2 K
  HOLD DOWN FORCE, T = 7.0 K

- EAST WALL (PERFORATED SHEAR WALL)
  L = 16.9'
  H = 13.8'
  F = 1.20 K (ASD)
  v = 157 PLF
  END POST, C = 2.6 K
  HOLD DOWN FORCE, T = 2.6 K

o DESIGN OF SHEAR WALLS:

Wood Shear Wall Without Openings

Location: **North Wall**

Shear Wall Geometry:

h =	13.7	ft	Height to Roof
hp =	4.5	ft	Parapet Height
L =	20.8	ft	Wall Length
Ld =	20.3	ft	Distance Between Hold-Downs

Loads:

V =	4.06	K	Lateral Force on Wall (ASD)
Ww =	12.0	psf	Dead Weight of Wall (Unfactored)
WDL =	48.0	plf	Roof Dead Load (Unfactored)
WLL =	160.0	plf	Roof Live Load (Unfactored)

Analysis:

Check Aspect Ratio

h / L = 0.66 OK — Max Aspect Ratio = 3.5 (SDPWS, Table 4.3.4)

Check In-Plane Shear Load in Wall — Aspect Ratio Factor = 1.00

v = V / L = 195 plf

USE 15/32" RATED SHEATHING (OSB), ONE SIDE WITH
6 IN PANEL EDGE SPACING

Capacity = 870 PLF (SDPWS, Table 4.3A)

Vall = 435 plf OK

Determine Tension/Compression Chord Force

T = C = vh = 2670 lb

Determine Hold-down Force

MOT =	56	K-FT	Overturning Moment V x h
Dwall =	2.7	K	0.6 x DL of Wall
Droof =	0.6	K	0.6 x DL of Roof
MResist =	34.7	K-FT	Resisting Moment DL x L / 2
T =	**1.0**	**K**	Hold-Down Force (Tension)
C =	**2.7**	**K**	End-Post Force (Compression)

Wood Shear Wall Without Openings

Location: **South Wall**

Shear Wall Geometry:

h =	14.0	ft	Height to Roof
hp =	4.2	ft	Parapet Height
L =	24.8	ft	Wall Length
Ld =	24.3	ft	Distance Between Hold-Downs

Loads:

V =	4.06	K	Lateral Force on Wall (ASD)
Ww =	12.0	psf	Dead Weight of Wall (Unfactored)
WDL =	48.0	plf	Roof Dead Load (Unfactored)
WLL =	160.0	plf	Roof Live Load (Unfactored)

Analysis:

Check Aspect Ratio

h / L = 0.56 OK Max Aspect Ratio = 3.5

(SDPWS, Table 4.3.4)

Check In-Plane Shear Load in Wall — Aspect Ratio Factor = 1.00

v = V / L = 164 plf

USE 15/32" RATED SHEATHING (OSB), ONE SIDE WITH — Capacity = 870 PLF

6 IN PANEL EDGE SPACING — (SDPWS, Table 4.3A)

Vall = 435 plf OK

Determine Tension/Compression Chord Force

T = C = vh = 2292 lb

Determine Hold-down Force

MOT =	57	K-FT	Overturning Moment V x h
Dwall =	3.2	K	0.6 x DL of Wall
Droof =	0.7	K	0.6 x DL of Roof
MResist =	49.2	K-FT	Resisting Moment DL x L / 2
T =	**0.3**	**K**	Hold-Down Force (Tension)
C =	**2.3**	**K**	End-Post Force (Compression)

Wood Shear Wall Without Openings

Location: **Center Wall**

Shear Wall Geometry:

h =	13.8	ft	Height to Roof
hp =	0.0	ft	Parapet Height
L =	15.9	ft	Wall Length
Ld =	15.4	ft	Distance Between Hold-Downs

Loads:

V =	8.29	K	Lateral Force on Wall (ASD)
Ww =	6.0	psf	Dead Weight of Wall (Unfactored)
WDL =	9.0	plf	Roof Dead Load (Unfactored)
WLL =	30.0	plf	Roof Live Load (Unfactored)

Analysis:

Check Aspect Ratio

h / L = 0.87 OK Max Aspect Ratio = 3.5

(SDPWS, Table 4.3.4)

Check In-Plane Shear Load in Wall Aspect Ratio Factor = 1.00

v = V / L = 521 plf

USE 15/32" RATED SHEATHING (OSB), ONE SIDE WITH Capacity = 1290 PLF

4 IN PANEL EDGE SPACING (SDPWS, Table 4.3A)

Vall = 645 plf OK

Determine Tension/Compression Chord Force

T = C = vh = 7200 lb

Determine Hold-down Force

MOT =	115	K-FT	Overturning Moment V x h
Dwall =	0.8	K	0.6 x DL of Wall
Droof =	0.1	K	0.6 x DL of Roof
MResist =	7.0	K-FT	Resisting Moment DL x L / 2
T =	**7.0**	**K**	Hold-Down Force (Tension)
C =	**7.2**	**K**	End-Post Force (Compression)

Perforated Wood Shear Wall

Location: **East Wall**

Shear Wall Geometry:

h = 13.8 ft Height to Roof
hp = 4.4 ft Parapet Height

Force in Shear Wall = **1202** lbs
Type = **Perforated**
Blocking = **Blocked**

SDPWS TABLE 4.3.3.5

SHEAR WALL PART	LENGTH (FT)	HEIGHT (FT)	FORCE IN PART (LB)	ΣL_i (FT)	% FULL HEIGHT SHEATHING	MAX OPENING HEIGHT (FT)	C_o	v (PLF)	v_{total} (PLF)	END POST COMPRESSION FORCE (LBS)	MIN FLOOR/WALL DL (PLF)	HOLD-DOWN FORCE, T (LBS)	WOOD STRUCTURAL PANEL	EDGE NAILING (IN)
1	16.92	13.8	1202	8.12	0.48	7.00	0.79	187	187	2590	0	2590	15/32" ONE SIDE	**6**
			1202											

NOMINAL SHEAR CAPACITY (PLF) = 870
ALLOWABLE SHEAR CAPACITY (PLF) = 435 OK

PER SDPWS TABLE 4.3A
15/32 Wood Structural Panels - Sheathing
10d nails w/ 1.5" penetration

Aspect Ratio, h / L = 0.82 OK
Max Allowable Aspect Ratio = 3.5
Aspect Ratio Factor = 1.00
Unblocked Shear Wall Adjustment Factor = 1.0

SDPWS, Table 4.3.4

Perforated Shear Wall Segment	Length, b_s (ft)	h/Li	Adjusted Length (ft)	
L_1	5.30	2.6	4.06	OK
L_2	5.30	2.6	4.06	OK
		ΣLi (FT) =	8.12	

Table 4.3A Nominal Unit Shear Capacities for Wood-Frame Shear Walls[1,3,6,7]

Wood-based Panels[4]

Sheathing Material	Minimum Nominal Panel Thickness (in.)	Minimun Fastener Penetration in Framing Member or Blocking (in.)	Fastener Type & Size	A SEISMIC Panel Edge Fastener Spacing (in.) 6 v_s (plf)	6 G_a (kips/in.) OSB	6 G_a PLY	4 v_s (plf)	4 G_a (kips/in.) OSB	4 G_a PLY	3 v_s (plf)	3 G_a (kips/in.) OSB	3 G_a PLY	2 v_s (plf)	2 G_a (kips/in.) OSB	2 G_a PLY	B WIND Panel Edge Fastener Spacing (in.) 6 v_w (plf)	4 v_w (plf)	3 v_w (plf)	2 v_w (plf)
Wood Structural Panels - Structural I[4,5]	5/16	1-1/4	Nail (common or galvanized box) 6d	400	13	10	600	18	13	780	23	16	1020	35	22	560	840	1090	1430
	3/8[2]	1-3/8	8d	460	19	14	720	24	17	920	30	20	1220	43	24	645	1010	1290	1710
	7/16[2]			510	16	13	790	21	16	1010	27	19	1340	40	24	715	1105	1415	1875
	15/32			560	14	11	860	18	14	1100	24	17	1460	37	23	785	1205	1540	2045
	15/32	1-1/2	10d	680	22	16	1020	29	20	1330	36	22	1740	51	28	950	1430	1860	2435
Wood Structural Panels – Sheathing[4,5]	5/16	1-1/4	6d	360	13	9.5	540	18	12	700	24	14	900	37	18	505	755	980	1260
	3/8			400	11	8.5	600	15	11	780	20	13	1020	32	17	560	840	1090	1430
	3/8[2]	1-3/8	8d	440	17	12	640	25	15	820	31	17	1060	45	20	615	895	1150	1485
	7/16[2]			480	15	11	700	22	14	900	28	17	1170	42	21	670	980	1260	1640
	15/32			520	13	10	760	19	13	980	25	15	1280	39	20	730	1065	1370	1790
	15/32	1-1/2	10d	620	22	14	920	30	17	1200	37	19	1540	52	23	870	1290	1680	2155
	19/32			680	19	13	1020	26	16	1330	33	18	1740	48	22	950	1430	1860	2435
Plywood Siding	5/16	1-1/4	Nail (galvanized casing) 6d	280	13		420	16		550	17		720	21		390	590	770	1010
	3/8	1-3/8	8d	320	16		480	18		620	20		820	22		450	670	870	1150
Particleboard Sheathing - (M-S "Exterior Glue" and M-2 "Exterior Glue")	3/8		Nail (common or galvanized box) 6d	240	15		360	17		460	19		600	22		335	505	645	840
	3/8		8d	260	18		380	20		480	21		630	23		365	530	670	880
	1/2			280	18		420	20		540	22		700	24		390	590	755	980
	1/2		10d	370	21		550	23		720	24		920	25		520	770	1010	1290
	5/8			400	21		610	23		790	24		1040	26		560	855	1105	1455
Structural Fiberboard Sheathing	1/2		Nail (galvanized roofing) 11 ga. galv. roofing nail (0.120" x 1-1/2" long x 7/16" head)				340	4.0		460	5.0		520	5.5			475	645	730
	25/32		11 ga. galv. roofing nail (0.120" x 1-3/4" long x 3/8" head)				340	4.0		460	5.0		520	5.5			475	645	730

ELLISON GAGE & ASSOCIATES, PLLC Consulting Structural Engineers 5068 W. Plano Parkway Suite 200 Plano, TX 75093 Ofc: (972) 354-8855 Fax: (972) 354-8856 www.ellisongage.com	Project SALAD AND GO #2001 – LEE'S SUMMIT, MO			Job Ref. 24-011
	Description LATERAL DESIGN (SEC. L)	Calc by. BG	Date 4/19/2024	Sheet No. L19

- o SHEAR WALL SUMMARY

NORTH, SOUTH AND EAST SHEAR WALLS NOTED AS TYPE SW-2, REFER TO SHEET S004
SHEATHING SCREW SPACING IS 6" OC AT EDGES AND 12" OC INTERMEDIATE

CENTER SHEAR WALL NOTED AS TYPE SW-1, REFER TO SHEET S004
SHEATHING SCREW SPACING IS 4" OC AT EDGES AND 12" OC INTERMEDIATE

- o DESIGN SHEAR WALL CONNECTION TO FOUNDATION:

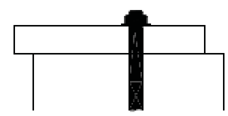
CONNECTION TO FOUNDATION WILL BE THE SAME AT ALL SHEAR WALL LOCATIONS
IN-PLANE SHEAR LOAD, v = 521 PLF (ASD, WORST CASE AT CENTER WALL)
TRY ½" DIAMETER BOLTS AT 24" OC AT 2x SILL PLATE
SHEAR CAPACITY OF BOLT = Z' = 650 LBS x 1.6 = 1,050 LBS/BOLT [NDS TABLE 12E]

v_{ALL} = 1040 LBS*(12/24) = 525 PLF > 521 PLF → *ok*

USE 2x6 SILL PLATE ATTACHED WITH ½" DIAMETER BOLTS AT 24" OC

12E BOLTS: Reference Lateral Design Values, Z, for Single Shear (two member) Connections[1,2,3,4]

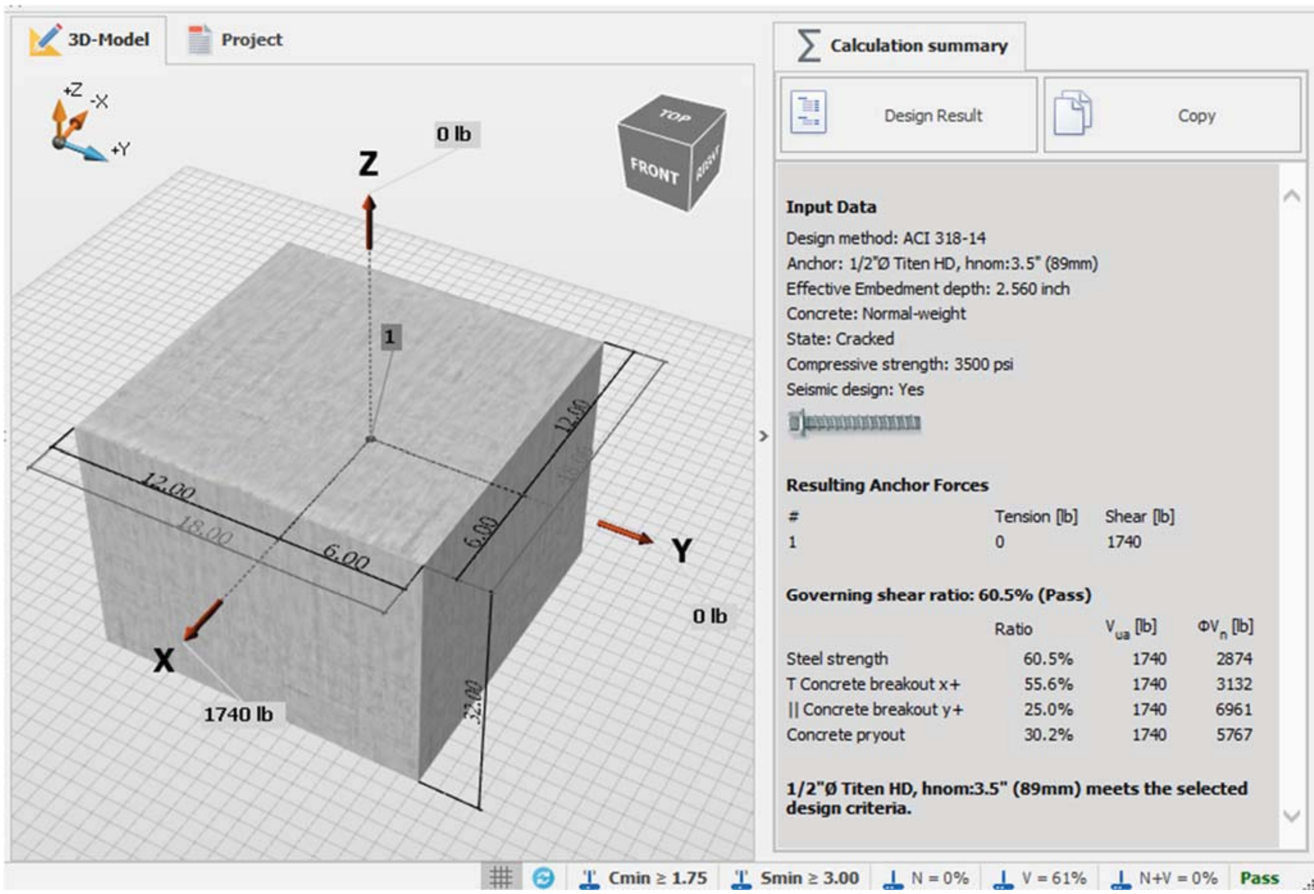
for sawn lumber or SCL to concrete



Thickness: Embedment Depth in Concrete t_m in.	Thickness: Side Member t_s in.	Bolt Diameter D in.	G=0.67 Red Oak $Z_{\parallel}$ lbs.	G=0.67 Red Oak $Z_{\perp}$ lbs.	G=0.55 Mixed Maple Southern Pine $Z_{\parallel}$ lbs.	G=0.55 Mixed Maple Southern Pine $Z_{\perp}$ lbs.	G=0.50 Douglas Fir-Larch $Z_{\parallel}$ lbs.	G=0.50 Douglas Fir-Larch $Z_{\perp}$ lbs.	G=0.49 Douglas Fir-Larch(N) $Z_{\parallel}$ lbs.	G=0.49 Douglas Fir-Larch(N) $Z_{\perp}$ lbs.	G=0.46 Douglas Fir(S) Hem-Fir(N) $Z_{\parallel}$ lbs.	G=0.46 Douglas Fir(S) Hem-Fir(N) $Z_{\perp}$ lbs.
6.0 and greater	1-1/2	1/2	770	480	680	410	650	380	640	380	620	360
		5/8	1070	660	970	580	930	530	920	520	890	470
		3/4	1450	890	1330	660	1270	590	1260	560	1230	520
		7/8	1890	960	1750	720	1690	630	1680	600	1640	550
		1	2410	1020	2250	770	2100	680	2060	650	1930	600
	1-3/4	1/2	830	510	740	430	700	400	690	390	670	370
		5/8	1160	680	1030	600	980	550	970	550	940	530
		3/4	1530	900	1390	770	1330	680	1310	660	1270	600
		7/8	1970	1120	1800	840	1730	740	1720	700	1680	640
		1	2480	1190	2290	890	2210	790	2200	750	2150	700
	2-1/2	1/2	830	590	790	520	770	470	760	460	750	440
		5/8	1290	800	1230	670	1180	610	1170	610	1120	570
		3/4	1840	1000	1630	850	1540	800	1520	780	1460	750
		7/8	2290	1240	2050	1080	1940	1020	1920	1000	1860	920
		1	2800	1520	2530	1280	2410	1130	2390	1080	2310	1000
	3-1/2	1/2	830	590	790	540	770	510	760	500	750	490
		5/8	1290	880	1230	810	1200	730	1190	720	1170	670
		3/4	1860	1190	1770	980	1720	900	1720	880	1680	830
		7/8	2540	1410	2410	1190	2320	1100	2290	1070	2200	1020
		1	3310	1670	2970	1420	2800	1330	2770	1300	2660	1260

CHECK ½" DIA SIMPSON TITEN HD ANCHORS FOR IN-PLANE SHEAR LOAD, v = 521 PLF (ASD)
FORCE IN ANCHOR = 521 PLF x 2 FT OC SPACING = 1040 LBS (ASD)
FORCE IN ANCHOR = 1040 / 0.6 = 1,740 LBS (LRFD)

<u>USE ½" DIA TITEN HD (hnom=3.5") SPACED AT 24" ON CENTER</u>



- DESIGN SHEAR WALL HOLD DOWNS:

USE ONE TYPE OF HOLD DOWN FOR ALL SHEAR WALLS (CONSERVATIVE)

WORST CASE IS CENTER SHEAR WALL, MAX T = 7,000 LBS (SERVICE)

<u>USE SIMPSON HDU8-SDS2.5 WITH 7/8" DIA ANCHOR BOLT AND (20) ¼" x 2½" SCREWS TO POST</u>

T_{ALL} = 7,875 LBS < 7,000 LBS **<u>OK</u>**

SIMPSON Strong-Tie

HDU/DTT

Holdowns

VALUE ENGINEERED

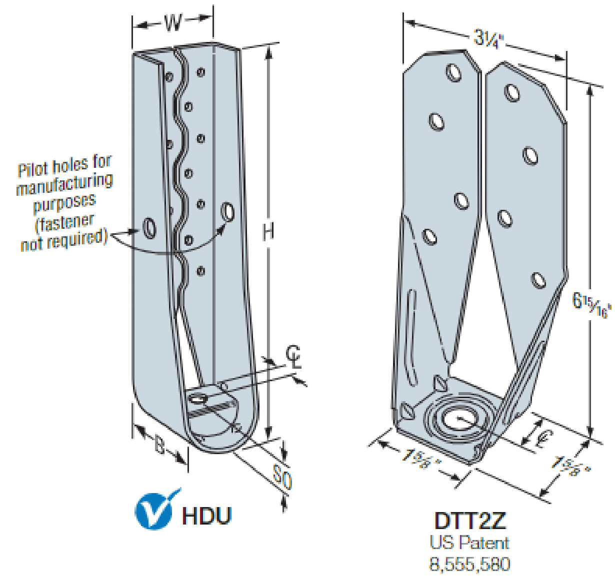
This product is preferable to similar connectors because of (a) easier installation, (b) higher loads, (c) lower installed cost, or a combination of these features.

HDU holdowns are pre-deflected during the manufacturing process, virtually eliminating deflection under load due to material stretch. They use Strong-Drive® SDS Heavy-Duty Connector screws which install easily, reduce fastener slip and provide a greater net section when compared to bolts.

The DTT tension ties are designed for lighter-duty holdown applications on single 2x posts. The DTT1Z is installed with nails or Strong-Drive SD Connector screws and the DTT2 installs easily with the Strong-Drive SDS Heavy-Duty Connector screws (included). The DTT1Z holdowns have been tested for use in designed shearwalls and prescriptive braced wall panels as well as prescriptive wood-deck applications (see p. 295 for deck applications).

For more information on holdown options, contact Simpson Strong-Tie.

HDU Features:



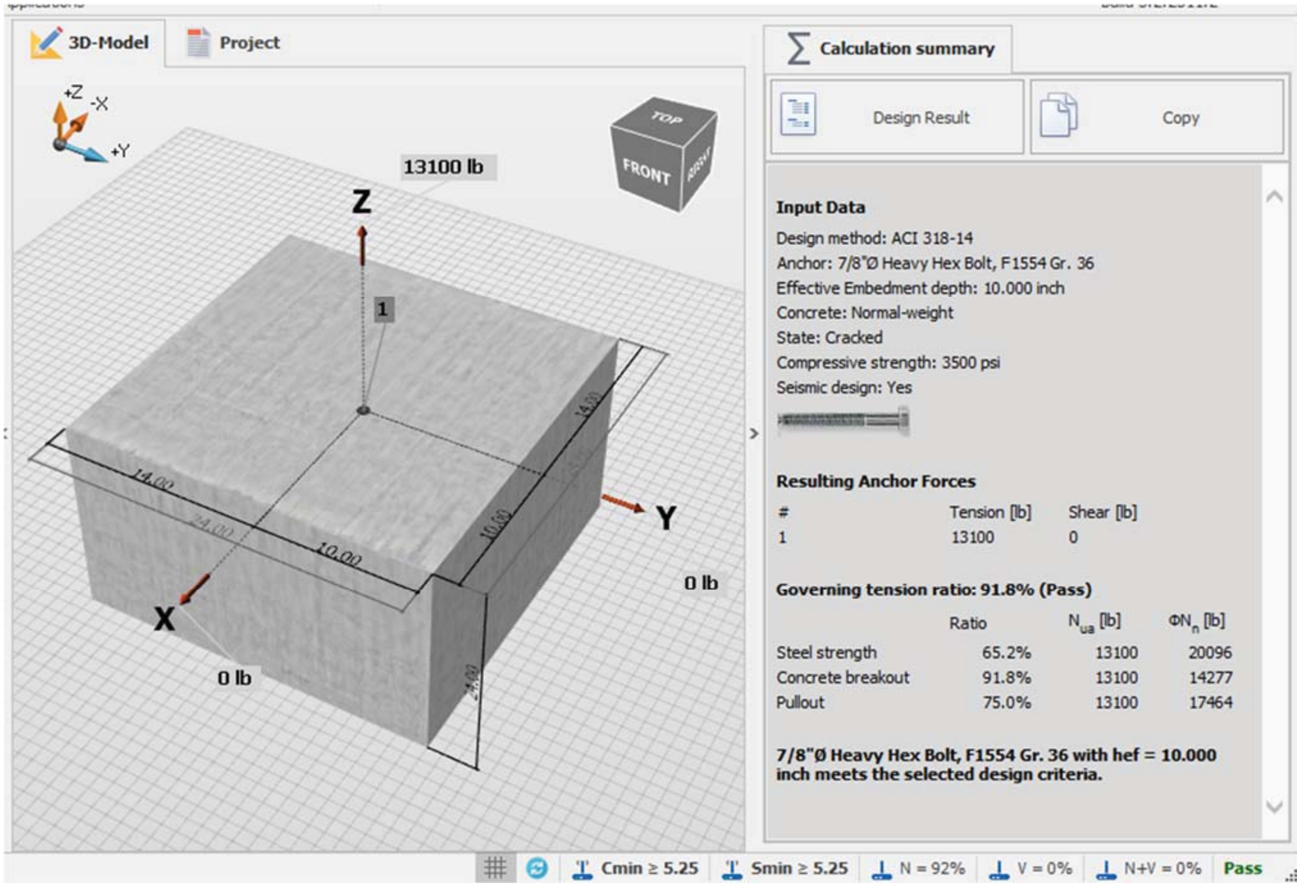
Model No.	Ga.	Dimensions (in.)					Fasteners (in.)		Minimum Wood Member Size (in.)	Allowable Tension Loads (160)			Code Ref.
		W	H	B	CL	SO	Anchor Bolt Dia. (in.)	Wood Fasteners		DF/SP	SPF/HF	Deflection at Allowable Load (in.)	
DTT1Z	14	1½	7⅛	1 7/16	¾	3/16	⅜	(6) #9 x 1½" SD	1½ x 3½	840	840	0.17	IBC®, FL, LA
								(6) 0.148 x 1½		910	640	0.167	
								(8) 0.148 x 1½		910	850	0.167	
DTT2Z	14	3¼	6 15/16	1⅝	13/16	3/16	½	(8) ¼ x 1½ SDS	1½ x 3½	1,825	1,800	0.105	
								(8) ¼ x 1½ SDS	3 x 3½	2,145	1,835	0.128	
DTT2Z-SDS2.5								(8) ¼ x 2½ SDS	3 x 3½	2,145	2,105	0.128	
HDU2-SDS2.5	14	3	8 11/16	3¼	1 5/16	1⅜	⅝	(6) ¼ x 2½ SDS	3 x 3½	3,075	2,215	0.088	
HDU4-SDS2.5	14	3	10 15/16	3¼	1 5/16	1⅜	⅝	(10) ¼ x 2½ SDS	3 x 3½	4,565	3,285	0.114	
HDU5-SDS2.5	14	3	13 3/16	3¼	1 5/16	1⅜	⅝	(14) ¼ x 2½ SDS	3 x 3½	5,645	4,340	0.115	
HDU8-SDS2.5	10	3	16⅝	3½	1⅜	1½	⅞	(20) ¼ x 2½ SDS	3 x 3½	6,765	5,820	0.11	
									3½ x 3½	6,970	5,995	0.116	
									3½ x 4½	7,870	6,580	0.113	
HDU11-SDS2.5	10	3	22¼	3½	1⅜	1½	1	(30) ¼ x 2½ SDS	3½ x 5½	9,535	8,030	0.137	
									3½ x 7¼	11,175	9,610	0.137	
HDU14-SDS2.5	7	3	25 11/16	3½	1 9/16	1 9/16	1	(36) ¼ x 2½ SDS	3½ x 5½	10,770	9,260	0.122	—
									3½ x 7¼	14,390	12,375	0.177	IBC, FL, LA
									5½ x 5½	14,445	12,425	0.172	

1. HDU14 requires heavy-hex anchor nut to achieve tabulated loads (supplied with holdown).
2. HDU14 loads on 4x6 post are applicable to installation on either the narrow or the wide face of the post.
3. **Fasteners:** Nail dimensions are listed diameter by length. SD and SDS screws are Simpson Strong-Tie Strong-Drive SD Connector and SDS Heavy-Duty Connector screws. See pp. 23–24 for fastener information.

o DESIGN ATTACHMENT OF TENSION HOLD DOWNS TO FOUNDATION

FOR SIMPSON HDU8, MAX T = 7,870 LBS = 13,100 LBS (LRFD)

USE 7/8" DIA HEAVY HEX BOLT (F1554, GR 36) EMBED 10" INTO FOUNDATION (18" TOTAL FROM TOP OF SLAB)



o DESIGN SHEAR WALL END POSTS:

WORST CASE IS CENTER SHEAR WALL, MAX C = 7,200 LBS (ASD)

∴ **USE (2) 2x6 STUDS**

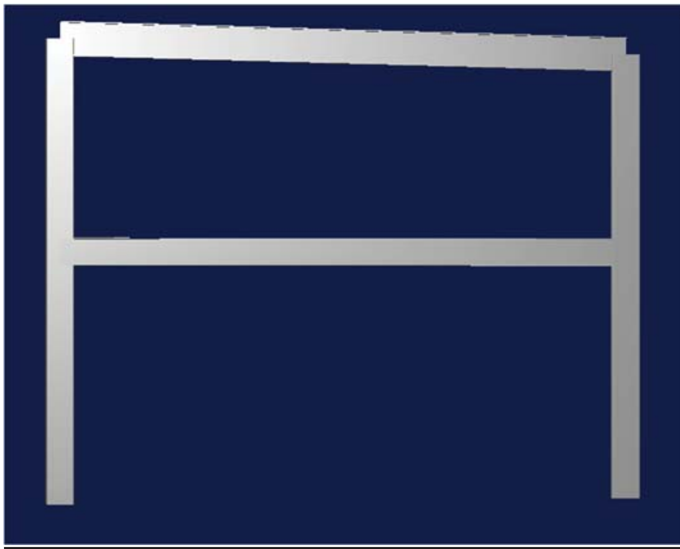
(2) 2x6, P_{ALL} = 2 x 4,318 LBS = 8,636 LBS > 7,200 LBS **OK**

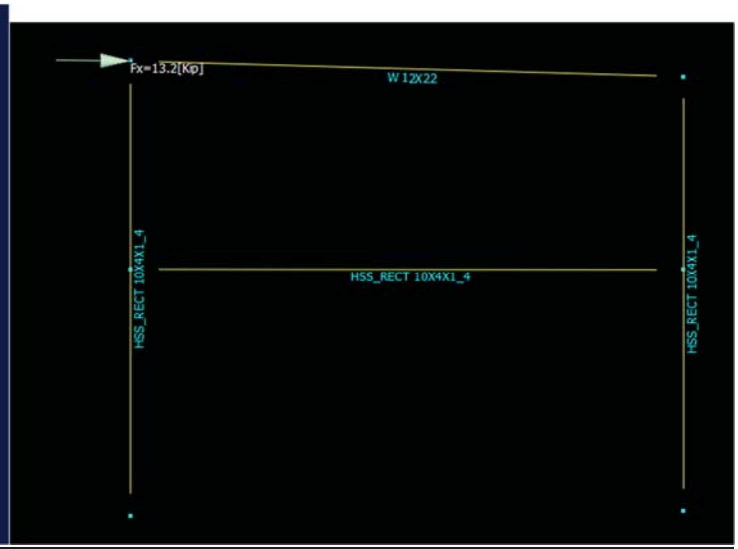
WOOD COLUMN DESIGN (NDS 2015)

MEMBER NOMINAL THICKNESS, t =	2	IN	
MEMBER NOMINAL WIDTH, w =	6	IN	
WOOD SPECIES =	DFL NO. 2	(VISUALLY GRADED LUMBER, 2"-4" THICK)	
ℓ_1 =	161.5	IN	[FIG. 3F]
d_1 =	5.5	IN	[FIG. 3F]
K_{e1} =	1.00		[TABLE G1]
$K_{e1}\ell_1/d_1$ =	29.36		CONTROLS
ℓ_2 =	12	IN	[FIG. 3F]
d_2 =	1.5	IN	[FIG. 3F]
K_{e2} =	1.00		[TABLE G1]
$K_{e2}\ell_2/d_2$ =	8.00		
E_{min}' =	580000	PSI	[NDS TABLE 4A]
c =	0.80		[3.7.1]
$F_c^* = F_c \times C_D \times C_M \times C_t \times C_F \times C_i$			[3.7.1 & NDS TABLE 4.3.1]
F_C =	1350	PSI	[NDS TABLE 4A]
C_D =	1.60		[4.3.2 & TABLE 2.3.2]
C_M =	1.00		[4.3.3 & TABLE 4A ADJUSTMENT FACTORS]
IN-SERVICE MOISTURE CONDITION =	Dry		[TABLE 2.3.3]
IN-SERVICE TEMP. =	<100	deg F	[TABLE 2.3.3]
C_t =	1.0		[4.3.4 & TABLE 2.3.3]
C_F =	1.10		[4.3.6 & TABLE 4A ADJUSTMENT FACTORS]
C_i =	1.00		[4.3.8]
F_C^* =	2376	PSI	[3.7.1]
$F_{cE} = (0.822 \times E_{min}') / ((\ell_e/d)^2)$ =	553	PSI	[3.7.1]
F_{cE} / F_C^* =	0.23		
C_P =	**0.22**		[EQ. 3.7-1]
F'_C =	523.4 PSI		
P_{ALL} =	**4318 LBS**		

o DESIGN OF MOMENT FRAME AT GRID A

FORCE INTO FRAME. V_{WEST} = 13.2 K (LRFD)





Steel Code Check
Summary - Group by member

Load conditions to be included in design :

D1=1.4DL
D2=1.2DL+0.5LLr
D3=1.2DL+1.6LLr
D4=1.2DL+0.5WL
D5=1.2DL+1.6LLr+0.5WL
D6=1.2DL+WL
D7=1.2DL+0.5LLr+WL
D8=0.9DL+WL

Description	Section	Member	Ctrl Eq.	Ratio	Status	Reference
	HSS_RECT 10X4X1_4	3	D6 at 100.00%	0.43	OK	
	W 12X22	4	D6 at 100.00%	0.51	OK	
COLUMN	*HSS_RECT 10X4X1_4*	1	D8 at 0.00%	0.50	OK	
		5	D6 at 0.00%	0.53	OK	

Analysis result

Nodes

Nodal displacements envelope

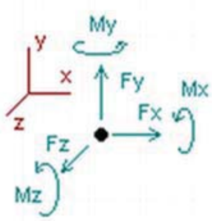
Note.- **lc** is the controlling load condition

Nodal displacements envelope for :

S1=DL
S2=DL+LLr
S3=DL+0.75LLr
S4=DL+0.6WL
S5=0.6DL+0.6WL

Node		Translation X [in]	lc	Y [in]	lc	Z [in]	lc	Rotation Rx [Rad]	lc	Ry [Rad]	lc	Rz [Rad]	lc
3	Max	0.507	S5	0.002	S5	0.000	S1	0.00000	S1	0.00000	S1	-0.00038	S1
	Min	-0.001	S1	-0.002	S1	0.000	S1	0.00000	S1	0.00000	S1	-0.00187	S4
8	Max	0.502	S5	-0.002	S1	0.000	S1	0.00000	S1	0.00000	S1	0.00039	S1
	Min	-0.002	S1	-0.005	S4	0.000	S1	0.00000	S1	0.00000	S1	-0.00140	S5

Reactions

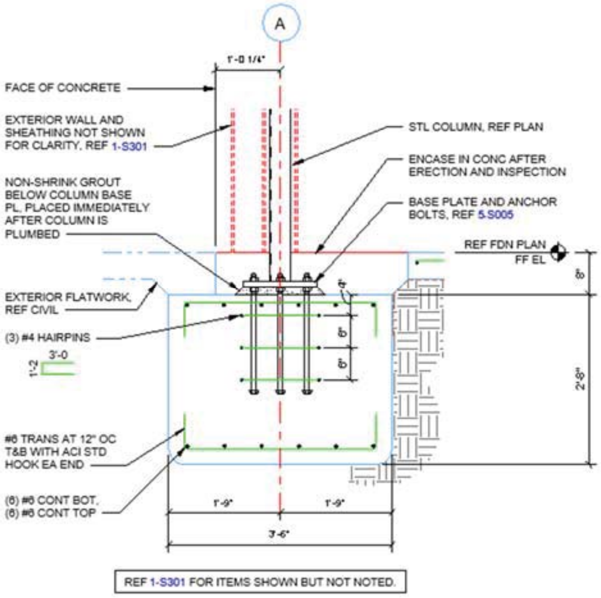


Direction of positive forces and moments

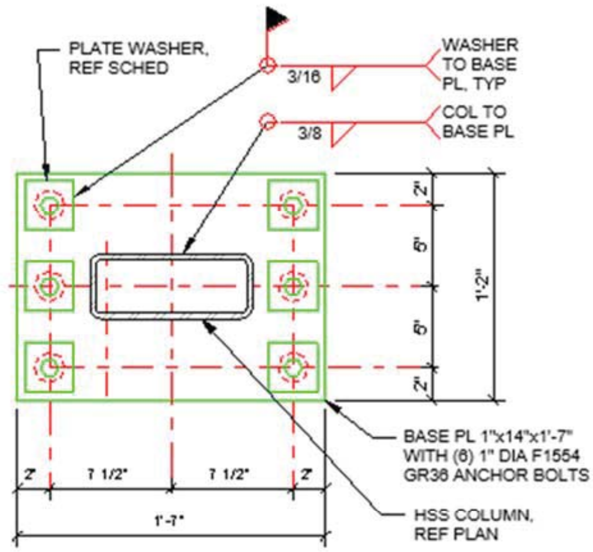
Node	Forces [Kip] FX	FY	FZ	Moments [Kip*ft] MX	MY	MZ
Condition **DL=Dead Load**						
1	0.09363	2.58533	0.00000	0.00000	0.00000	-0.26846
4	-0.09363	2.59019	0.00000	0.00000	0.00000	0.22654
SUM	0.00000	5.17552	0.00000	0.00000	0.00000	-0.04192
Condition **LLr=Roof Live Load**						
SUM	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
Condition **WL=Wind Load**						
1	-6.66188	-6.44465	0.00000	0.00000	0.00000	35.63568
4	-6.53812	6.44465	0.00000	0.00000	0.00000	35.35408
SUM	-13.20000	0.00000	0.00000	0.00000	0.00000	70.98976

o DESIGN OF BASE PLATE AT MOMENT FRAME

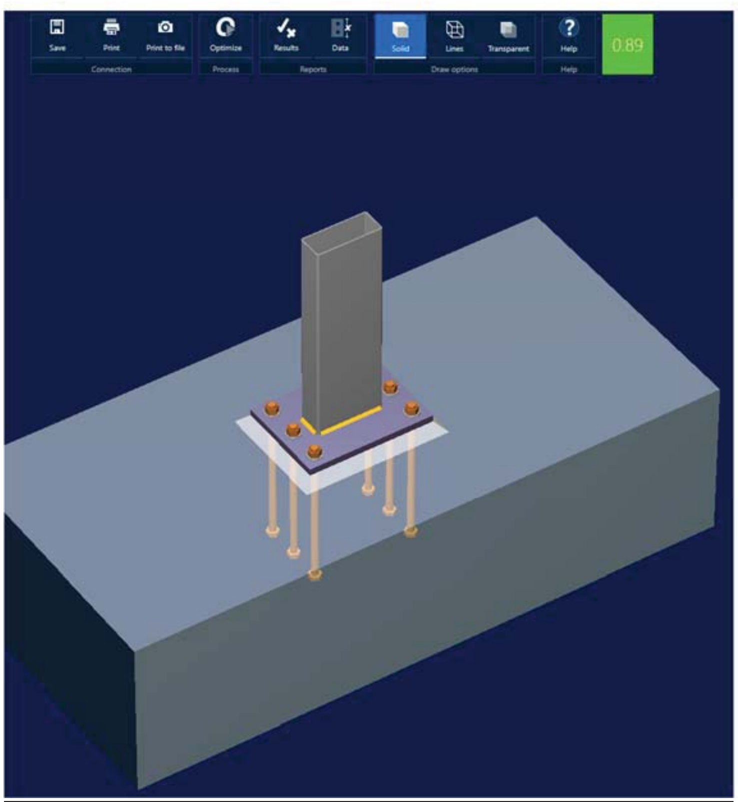
REFER TO 2-S501 AND 5-S005



2 FOUNDATION AT MOMENT FRAME
3/4" = 1'-0"



BASE PLATE



RAM Connection **Windows User**

Current Date: 4/24/2024 12:02 PM
Units system: English
File name: Z:\24-001_24-999\24-011 SAG 2001 Lees Summit-MO\02-Design Info\Calcs\Moment Frame\2001-FIXED BASE PLATE CONNECTION.rcnx

Steel Connections Results

Connection: 1 - Fixed uniaxial major axis BP

Family: Column - Base (CB)
Type: Base plate

Design code: AISC 360-16 LRFD, ACI 318-11

Demands

Description	Pu [kip]	Mu22 [kip*ft]	Mu33 [kip*ft]	Vu2 [kip]	Vu3 [kip]	Load type
DL	-2.60	0.00	0.30	0.10	0.00	Design
LLr	0.00	0.00	0.00	0.00	0.00	Design
W	6.40	0.00	35.60	6.70	0.00	Design
D1	-3.64	0.00	0.42	0.14	0.00	Design
D2	-3.12	0.00	0.36	0.12	0.00	Design
D3	-3.12	0.00	0.36	0.12	0.00	Design
D4	0.08	0.00	18.16	3.47	0.00	Design
D5	0.08	0.00	18.16	3.47	0.00	Design
D6	3.28	0.00	35.96	6.82	0.00	Design
D7	3.28	0.00	35.96	6.82	0.00	Design
D8	4.06	0.00	35.87	6.79	0.00	Design

Design calculations

Design for major axis
Base plate (AISC 360-16 LRFD)

Geometric Considerations

Dimensions	Unit	Value	Min.	Max.	Sta.	References
Base plate						
Distance from anchor to edge	[in]	1.50	0.25	--	✔	
Weld size	[1/16in]	6	2	--	✔	table J2.4

Design Check

Verification	Unit	Capacity	Demand	Ctrl EQ	Ratio	References
Concrete base						
Axial bearing	[Kip/in2]	4.42	0.49	D6	0.11	DG1 3.1.1

Base plate						
Flexural yielding (bearing interface)	[Kip*ft/ft]	8.10	6.19	W	0.76	DG1 Sec 3.1.2
Flexural yielding (tension interface)	[Kip*ft/ft]	8.10	6.38	W	0.79	DG1 Eq. 3.3.13
Column						
Weld capacity	[Kip/ft]	150.35	25.19	W	0.17	HSS Manual p. 7-10
Elastic method weld shear and axial capacity	[Kip/ft]	100.23	88.83	W	0.89	Sec. J2.4

Ratio **0.89**

Anchors

Geometric Considerations

Dimensions	Unit	Value	Min.	Max.	Sta.	References
Anchors						
Anchor spacing	[in]	5.00	4.00	--	✓	Sec. D.8.1
Concrete cover	[in]	15.50	3.00	--	✓	Sec. 7.7.1
Effective length	[in]	18.65	--	23.35	✓	

Design Check

Verification	Unit	Capacity	Demand	Ctrl EQ	Ratio	References
Anchor tension	[Kip]	26.35	10.50	W	0.40	Eq. D-2
Breakout of anchor in tension	[Kip]	59.79	10.50	W	0.18	Sec. D.3.3.4.4
Breakout of group of anchors in tension	[Kip]	59.79	31.49	W	0.53	Sec. D.3.3.4.4
Pullout of anchor in tension	[Kip]	26.06	10.50	W	0.40	Sec. D.3.3.4.4
Anchor shear	[Kip]	10.96	1.14	D6	0.10	Eq. D-29, Sec. D.6.1.3
Breakout of anchor in shear	[Kip]	19.74	1.14	D6	0.06	Table D.4.1.1, Sec. D.4.3
Breakout of group of anchors in shear	[Kip]	20.08	6.82	D6	0.34	Table D.4.1.1, Sec. D.4.3
Pryout of anchor in shear	[Kip]	119.57	1.14	D6	0.01	Table D.4.1.1, Sec. D.4.3
Pryout of group of anchors in shear	[Kip]	152.79	6.82	D6	0.04	Table D.4.1.1, Sec. D.4.3
Interaction of tensile and shear forces	[Kip]	1.20	0.86	W	0.72	Eq. D-42

Ratio **0.72**

Global critical strength ratio **0.89**

Major axis

ROOF AND WALL FRAMING DESIGN

ROOF FRAMING DESIGN

- ROOF FRAMING IS PREFABRICATED WOOD TRUSSES BY TRUSS MFR

- STEEL FRAME AT GRID A
  REFER TO LATERAL ANALYSIS FOR DESIGN OF MOMENT FRAME COMPONENTS

  DESIGN HSS GIRT AT EL = 107'-10" FOR OUT OF PLANE WIND LOAD
  L = 16.5' MAX
  ASD WIND LOAD = 15.8 PSF x 12.6' / 2 = 100 PLF
  WEIGHT OF WINDOW AND FRAMING ABOVE HSS GIRD
  DL = 25 PSF x (12.6' – 7.8') = 120 PLF

  USE HSS10x4x1/4

Wind Girt Design (Biaxial Bending)

General Input	
Girt Span (ft) =	**16.50**
Lateral Pressure (psf) =	**15.8**
Tributary Width (ft) =	**6.30**
Lateral Load (plf) =	**100**
Vertical Load (plf) =	**120**
End Conditions	**Pinned**

Girt Section Properties	
Girt Selection =	**1**
Area (in^2) =	6.17
I_X (in^4) =	74.70
S_X (in^4) =	14.90
I_y (in^4) =	17.70
S_y (in^3) =	8.87
F_y (ksi) =	46
E (ksi) =	29000

Model Girt Analysis Results	
Vert. Moment, M_X (k-ft) =	4.08
Lat. Moment, M_Y (k-ft) =	3.39
Column Reaction (k) =	0.82
Max. Lat. Deflection (in) =	0.23

Girt Check		
f_{bx} (ksi) =	3.29	OK
F_{bx} (ksi) =	27.60	
f_{by} (ksi) =	4.58	OK
F_{by} (ksi) =	27.60	
Interaction =	0.29	OK
L/240 (in) =	0.83	OK

Girt Properties Table:

	Girt Size							
Property	**HSS10x4x1/4** **1**	**HSS10x4x5/16** **2**	**HSS10x4x3/8** **3**	**HSS8x4x1/4** **4**	**HSS8x4x5/16** **5**	**HSS8x4x3/8** **6**	**7**	**8**
A (in^2)	6.17	7.59	8.97	5.24	6.43	7.58		
I_X (in^4)	74.7	90.1	104.0	42.5	51.0	58.7		
S_X (in^4)	14.9	18.0	20.8	10.6	12.8	14.7		
I_y (in^4)	17.7	21.2	24.3	14.4	17.2	19.6		
S_y (in^3)	8.9	10.6	12.1	7.2	8.6	9.8		

WALL STUD DESIGN

WALL TRIB AREA FOR WIND = 14'*14'/3 = 65 SF, USE WIND LOADS FOR 50 PSF TRIB AREA
LRFD WALL WIND PRESSURE = **26.3 PSF** (WORST CASE)
LRFD PARAPET WIND PRESSURE = **64.7 PSF** (WORST CASE)
LIMIT WALL DEFLECTION TO L/360

- LOAD-BEARING NORTH AND SOUTH WALLS, 1-S401
  TRUSSES BEAR ON TOP PLATE
  h = 12.3'
  LOAD FROM ROOF JOISTS:
  ROOF DEAD LOAD = 25 PSF x 24' / 2 = 270 PLF + 100 PLF (RTU) = 400 PLF
  ROOF LIVE LOAD = 20 PSF x 24' / 2 = 240 PLF
  ROOF SNOW LOAD = 35 PSF x 24' / 2 = 420 PLF (WORST CASE WALL DRIFT)
  LOAD FROM PARAPET WALL = 20 PSF x 20.2'-12.3' = 160 PLF

USE 2x6 SYP #2 WALL STUDS SPACED AT 16" OC

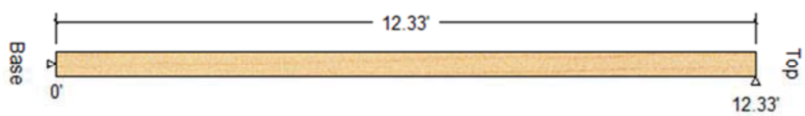
Design Check Calculation Sheet

WoodWorks Sizer 11.1

Loads:

Load	Type	Distribution	Location [ft] Start End	Magnitude Start End	Unit
WIND	Wind C&C	Full Area		26.30(16.0")	psf
DL	Dead	Axial UDL	(Ecc. = 0.92")	560	plf
LR	Roof live	Axial UDL	(Ecc. = 0.92")	240	plf
SL	Snow	Axial UDL	(Ecc. = 0.92")	420	plf

Lateral Reactions (lbs):



Unfactored:			
Dead	5		-5
Snow	3		-3
Wind	216		216
Roof Live	2		-2
Factored:			
R->L			-8
Load comb			#4
L->R	134		127
Load comb	#7		#6

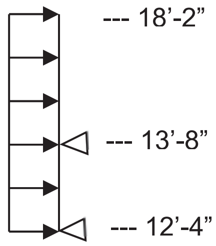
Lumber Stud, S. Pine, No.2, 2x6 (1-1/2"x5-1/2")

Support: Lumber Stud Bottom plate, S. Pine No.2; Bearing length = stud thickness; continuous lower support
Spaced at 16.0" c/c; Total length: 12.33'; Clear span: 12.205'; volume = 0.7 cu.ft.
Pinned base; Load face = width(b); Ke x Lb: 1.0 x 0.0 = 0.0 [ft]; Ke x Ld: 1.0 x 12.33 = 12.33 [ft]; Repetitive factor: applied where permitted (refer to online help);

Analysis vs. Allowable Stress and Deflection using NDS 2015 :

Criterion	Analysis Value	Design Value	Unit	Analysis/Design
Shear	fv = 24	Fv' = 280	psi	fv/Fv' = 0.09
Bending(+)	fb = 681	Fb' = 2159	psi	fb/Fb' = 0.32
Axial	fc = 158	Fc' = 528	psi	fc/Fc' = 0.30
Combined	(axial + eccentric &	side load bending)		Eq.15.4-1= 0.45
Axial Bearing	fc = 158	Fc* = 1610	psi	fc/Fc* = 0.10
Support Bearing	fcp = 158	Fcp = 706	psi	fcp/Fcp = 0.22
Live Defl'n	0.26 = L/562	0.41 = L/360	in	0.64
Total Defl'n	0.30 = L/501	0.62 = L/240	in	0.48

- LOAD-BEARING NORTH AND SOUTH WALLS, 1-S401
  2x FRAMING ABOVE TOP PLATE



TOP OF WALL = 18'-2"

USE 2x6 SYP #2 WALL STUDS SPACED AT 16" OC

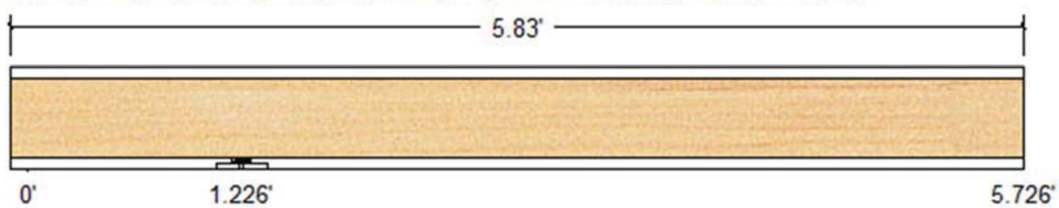
Design Check Calculation Sheet

WoodWorks Sizer 11.1

Loads:

Load	Type	Distribution	Pat-tern	Location [ft] Start	End	Magnitude Start	End	Unit
WL	Wind C&C	Partial Area	No	0.00	1.33	26.30(16.0")		psf
PARAPET WL	Wind C&C	Partial Area	No	1.33	5.83	64.70(16.0")		psf

Maximum Reactions (lbs), Bearing Capacities (lbs) and Bearing Lengths (in) :



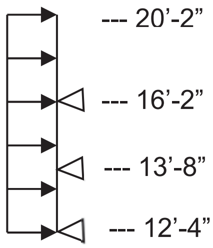
Lumber-soft, S. Pine, No.2, 2x6 (1-1/2"x5-1/2")

Supports: 1 - Non-wood; 2 - Lumber-soft Sill plate, D.Fir-L No.2;
Roof joist spaced at 16.0" c/c; Total length: 5.83'; Clear span: 1.329', 4.5'; volume = 0.3 cu.ft.
Lateral support: top= full, bottom= full; Repetitive factor: applied where permitted (refer to online help);

Analysis vs. Allowable Stress and Deflection using NDS 2015 :

Criterion	Analysis Value	Design Value	Unit	Analysis/Design
Shear	fv = 79	Fv' = 280	psi	fv/Fv' = 0.28
Bending(-)	fb = 832	Fb' = 1840	psi	fb/Fb' = 0.45
Deflection:				
Interior Live	-0.00 = <L/999	0.07 = L/240	in	0.03
Total	-0.00 = <L/999	0.07 = L/240	in	0.03
Cantil. Live	0.15 = L/360	0.45 = L/120	in	0.33
Total	0.15 = L/360	0.45 = L/120	in	0.33

- LOAD-BEARING NORTH AND SOUTH WALLS, 4 & 5-S401
  2x FRAMING ABOVE TOP PLATE, 20'-2" WALLS TO RECEIVE DIAGONAL BRACE



TOP OF WALL = 20'-2"

USE 2x6 SYP #2 WALL STUDS SPACED AT 16" OC

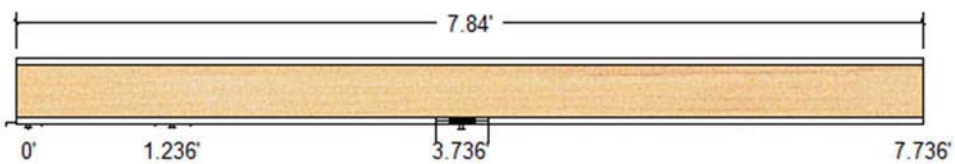
Design Check Calculation Sheet

WoodWorks Sizer 11.1

Loads:

Load	Type	Distribution	Pat-tern	Location [ft] Start	End	Magnitude Start	End	Unit
WL	Wind C&C	Partial Area	No	0.00	1.33	26.30(16.0")		psf
PARAPET WL	Wind C&C	Partial Area	No	1.33	7.84	64.70(16.0")		psf

Maximum Reactions (lbs), Bearing Capacities (lbs) and Bearing Lengths (in) :



Lumber-soft, S. Pine, No.2, 2x6 (1-1/2"x5-1/2")

Supports: 1,2 - Lumber-soft Sill plate, D.Fir-L No.2; 3 - Lumber Stud Wall, D.Fir-L No.2;
Roof joist spaced at 16.0" c/c; Total length: 7.84'; Clear span: 1.339', 2.498', 3.998'; volume = 0.4 cu.ft.
Lateral support: top= full, bottom= full; Repetitive factor: applied where permitted (refer to online help);

Analysis vs. Allowable Stress and Deflection using NDS 2015 :

Criterion	Analysis Value	Design Value	Unit	Analysis/Design
Shear	fv = 44	Fv' = 280	psi	fv/Fv' = 0.16
Bending(+)	fb = 175	Fb' = 1840	psi	fb/Fb' = 0.10
Bending(-)	fb = 657	Fb' = 1840	psi	fb/Fb' = 0.36
Deflection:				
Interior Live	-0.00 = <L/999	0.12 = L/240	in	0.03
Total	-0.00 = <L/999	0.08 = L/360	in	0.05
Cantil. Live	0.11 = L/425	0.40 = L/120	in	0.28
Total	0.11 = L/425	0.27 = L/180	in	0.42

- BALLOON-FRAMED EAST WALL, 2-S401

  TOW = 118'-2"
  h = 13.8'
  PARAPET HT = 4.4'
  ROOF DEAD LOAD = 25 PSF x 2.7' / 2 = 35 PLF + 65 PLF (MISC) = 100 PLF
  ROOF LIVE LOAD = 20 PSF x 2.7' / 2 = 30 PLF
  ROOF SNOW NOAD = 45 PSF x 2.7' / 2 = 60 PLF

  USE 2x6 SYP #2 WALL STUDS SPACED AT 16" OC

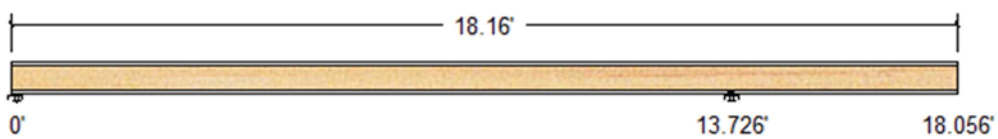
Design Check Calculation Sheet

WoodWorks Sizer 11.1

Loads:

Load	Type	Distribution	Pat-tern	Location [ft] Start	End	Magnitude Start	End	Unit
WL PARAPET	Wind C&C	Partial Area	No	13.83	18.16	64.70(16.0")		psf
WL	Wind C&C	Partial Area	No	1.00	13.83	26.30(16.0")		psf

Maximum Reactions (lbs), Bearing Capacities (lbs) and Bearing Lengths (in) :



Lumber-soft, S. Pine, No.2, 2x6 (1-1/2"x5-1/2")

Supports: All - Lumber-soft Sill plate, D.Fir-L No.2
Roof joist spaced at 16.0" c/c; Total length: 18.16'; Clear span: 13.829', 4.33'; volume = 1.0 cu.ft.
Lateral support: top= full, bottom= full; Repetitive factor: applied where permitted (refer to online help);

WARNING: Member length exceeds typical stock length of 18.0 [ft]

Analysis vs. Allowable Stress and Deflection using NDS 2015 :

Criterion	Analysis Value	Design Value	Unit	Analysis/Design
Shear	fv = 36	Fv' = 280	psi	fv/Fv' = 0.13
Bending(+)	fb = 440	Fb' = 1840	psi	fb/Fb' = 0.24
Bending(-)	fb = 770	Fb' = 1840	psi	fb/Fb' = 0.42
Deflection:				
Interior Live	0.17 = L/985	0.46 = L/360	in	0.37
Total	0.17 = L/985	0.69 = L/240	in	0.24
Cantil. Live	0.09 = L/580	0.29 = L/180	in	0.31
Total	0.09 = L/580	0.43 = L/120	in	0.21

- INTERIOR WALL, 6-S401
  - TOW = 114'-1"
  - ROOF DEAD LOAD = 25 PSF x 2.7' / 2 = 35 PLF + 65 PLF (MISC) = 100 PLF
  - ROOF LIVE LOAD = 20 PSF x 2.7' / 2 = 30 PLF
  - ROOF SNOW LOAD = 21 PSF x 2.7' / 2 = 30 PLF
  - INTERIOR WIND LOAD = 10 PSF

  <u>USE 2x4 SYP #2 WALL STUDS SPACED AT 16" OC</u>

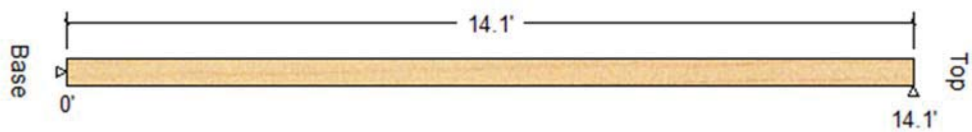
Design Check Calculation Sheet

WoodWorks Sizer 11.1

Loads:

Load	Type	Distribution	Location [ft] Start End	Magnitude Start End	Unit
WIND	Wind C&C	Full Area		10.00(16.0")	psf
DL	Dead	Axial UDL	(Ecc. = 0.92")	100	plf
LR	Roof live	Axial UDL	(Ecc. = 0.92")	30	plf
SL	Snow	Axial UDL	(Ecc. = 0.92")	30	plf

Lateral Reactions (lbs):



Unfactored:			
Dead	1		-1
Snow	0		-0
Wind	94		94
Roof Live	0		-0
Factored:			
R->L			-1
Load comb			#2
L->R	57		56
Load comb	#7		#6

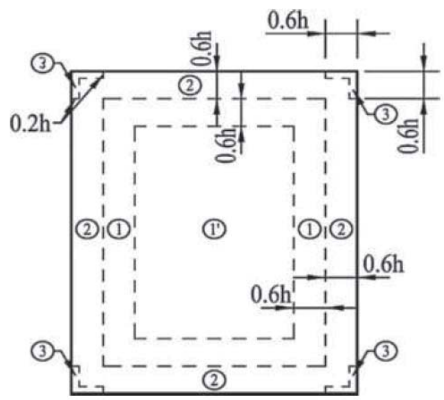
Lumber Stud, D.Fir-L, No.2, 2x6 (1-1/2"x5-1/2")

Support: Lumber Stud Bottom plate, D.Fir-L No.2; Bearing length = stud thickness; continuous lower support
Spaced at 16.0" c/c; Total length: 14.1'; Clear span: 13.975'; volume = 0.8 cu.ft.
Pinned base; Load face = width(b); Ke x Lb: 1.0 x 0.0 = 0.0 [ft]; Ke x Ld: 1.0 x 14.1 = 14.1 [ft]; Repetitive factor: applied where permitted (refer to online help);

Analysis vs. Allowable Stress and Deflection using NDS 2015 :

Criterion	Analysis Value	Design Value	Unit	Analysis/Design
Shear	fv = 10	Fv' = 288	psi	fv/Fv' = 0.04
Bending(+)	fb = 324	Fb' = 2527	psi	fb/Fb' = 0.13
Axial	fc = 21	Fc' = 462	psi	fc/Fc' = 0.05
Combined	(axial + eccentric & side load bending)			Eq.15.4-1= 0.14
Axial Bearing	fc = 21	Fc* = 1485	psi	fc/Fc* = 0.01
Support Bearing	fcp = 21	Fcp = 781	psi	fcp/Fcp = 0.03
Live Defl'n	0.15 = <L/999	0.70 = L/240	in	0.21
Total Defl'n	0.16 = <L/999	0.70 = L/240	in	0.22

ROOF UPLIFT ANALYSIS AND CONNECTION DESIGN



PLAN

TRUSS SPAN = 15'-10"
TRUSS TRIB AREA = 15.83 x 15.83 / 3 = 84 FT^2
NET ROOF UPLIFTS:

ZONE 1' = 14.0 PSF (ASD)
ZONE 1 = 19.5 PSF (ASD)
ZONE 2 = 25.9 PSF (ASD)
ZONE 3 = ZONE 2 = 25.9 PSF

0.6h = 8.4'

- FIND TRUSS UPLIFT, SEE 1-S401
  SINCE BUILDING ROOF WIDTH IS ONLY 15.83', USE ZONE 2 UPLIFT LOAD ON ENTIRE ROOF

  WORST CASE TRUSS SPAN = 23.0'
  F_{up} = (25.9 PSF – 6 PSF ROOF DL) x 23.0'/2 = 230 PLF

- TRUSS CONNECTION LOADS
  TRUSS SPACING = 32" = 2.67'
  UPLIFT = 230 PLF x 32"/12 = **610 LBS** AT EACH TRUSS
  USE SIMPSON H2.5A AT EACH TRUSS (P_{all} = 625 LBS > 610 LBS → *ok*)

- FIND WALL UPLIFT AT TOP PLATE, SEE 1-S401

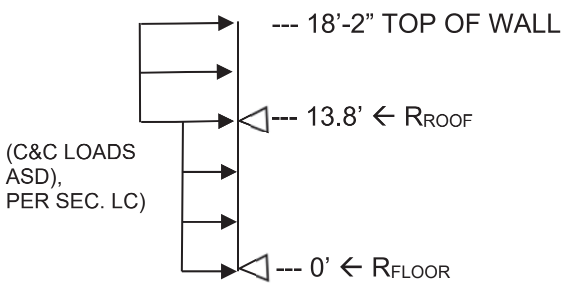
  F_{up} = (25.9 PSF – 10 PSF ROOF DL) x 23.0'/2 = 185 PLF
  PARAPET WT = 5 PSF (18.17-14) = 20 PLF
  F_{up} = 165 PLF

- WALL CONNECTION LOADS
  TIE PLATE SPACING = 48" = 4'
  UPLIFT = 165 PLF x 40"/12 = **550 LBS**
  USE SIMPSON SP1 AT 48" OC (P_{all} = 555 LBS > 550 LBS → *ok*)

OUT-OF-PLANE ANALYSIS AND CONNECTION DESIGN

- DETAIL 2-S401, EAST WALL

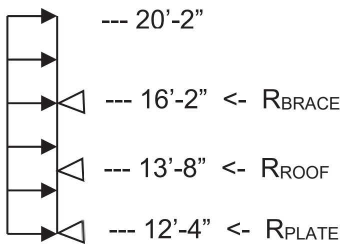
WALL TRIB AREA FOR WIND = 14'*14'/3 = 65 SF, USE WIND LOADS FOR 50 PSF TRIB AREA
ASD WALL WIND PRESSURE = **15.8 PSF** (WORST CASE)
ASD PARAPET WIND PRESSURE = **38.8 PSF** (WORST CASE)



R_{ROOF} = 38.8 PSF*(4.5') + 15.8 PSF*(13.8'/2) = 280 PLF
R_{FLOOR} = 15.8 PSF*(13.8'/2) = 100 PLF

WALL-TO-ROOF CONNECTION DESIGN:
TRY SIMPSON H8 AT 32" OC
F = 280 PLF x 32"/12 = 750 LBS
USE SIMPSON H8 AT 32" OC (P_{all} = 780 LBS > 750 LBS → *ok*)

- PARAPET BRACING DESIGN, REF 3, 4, AND 5-S401



TOP OF WALL = 20'-2"

R_{BRACE} = 38.8 PSF x [(20.2'-16.2') + (16.2'-13.7')/2] = 210 PLF
R_{ROOF} = 38.8 PSF x (16.2'-13.7')/2 + 15.8 PSF (13.7'-12.3')/2 = 60 PLF
R_{PLATE} = 15.8 PSF x [(13.7'-12.3')/2 + (12.3'/2)] = 110 PLF

- 2x4 DIAGONAL BRACE DESIGN
BRACE SPACING = 16" = 1.33'
BRACE FORCE = 210 PLF x 1.33' / 0.707 = 400 LBS

UPPER BRACE CONNECTION:
LAP 2x4 BRACE WITH VERT STUDS AND PROVIDE (6) 10d NAILS
LATERAL CAPACITY OF NAILS = 118 LBS x 1.6 = 188 LBS x 6 = 1130 LBS > 400 LBS → *ok*

BRACE CONNECTION AT ROOF:
SHEAR FORCE = 210 PLF x 16"/12 = 280 LBS
USE SIMPSON A23 ANGLE AT 16" OC (P_{all} = 535 LBS > 280 LB → *ok*)
UPLIFT FORCE = 210 PLF x 16"/12 = 280 LBS
USE SIMPSON CS20 AT 16" OC WITH (6) 10d NAILS (P_{all} = 1,030 LBS x 6/12 > 515 LBS → *ok*)

FIND VERT FORCE ACTING ON TRUSS 4 AND 5-S401:
F = 210 PLF x 32"/12 = **600 LBS** (WIND, ASD)

DESIGN OF FRAMING AT OPENINGS

REFER TO HEADER SCHEDULE, 5-S003.

- HEADER AT LOAD-BEARING WALLS UP TO 4'-0"
  WALL DL = 20 PSF x (20.2' – 7') = 265 PLF
  ROOF DL = 25 PSF x 23'/2 = 290 PLF
  ROOF LL = 20 PSF x 23'/2 = 230 PLF
  ROOF SL = 35 PSF x 23'/2 = 400 PLF

  USE H-1 (3) 2x6 SYP #2

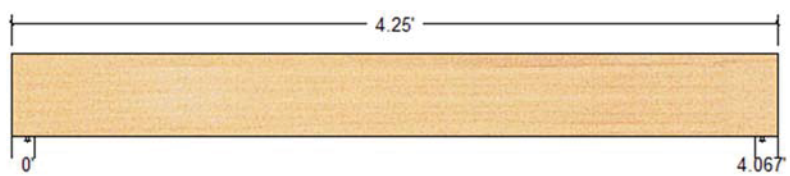
Design Check Calculation Sheet

WoodWorks Sizer 11.1

Loads:

Load	Type	Distribution	Pattern	Location [ft] Start	End	Magnitude Start	End	Unit
DL	Dead	Full UDL				555.0		plf
RL	Roof constr.	Full UDL				230.0		plf
SL	Snow	Full UDL				400.0		plf
Self-weight	Dead	Full UDL				6.4		plf

Maximum Reactions (lbs), Bearing Capacities (lbs) and Bearing Lengths (in) :



Unfactored:			
Dead	1192		1192
Snow	850		850
Roof Live	489		489
Factored:			
Total	2042		2042
Bearing:			
Capacity			
Beam	3814		3814
Support	11527		11527
Des ratio			
Beam	0.54		0.54
Support	0.18		0.18
Load comb	#3		#3
Length	1.50		1.50
Min req'd	0.80		0.80
Cb	1.00		1.00
Cb min	1.00		1.00
Cb support	1.00		1.00
Fc sup	1350		1350

Lumber n-ply, S. Pine, No.2, 2x6, 3-ply (4-1/2"x5-1/2")
Supports: All - Lumber n-ply Column, D.Fir-L No.2
Total length: 4.0'; Clear span: 4.0'; volume = 0.7 cu.ft.
Lateral support: top= at supports, bottom= at supports; Repetitive factor: applied where permitted (refer to online help);

Analysis vs. Allowable Stress and Deflection using NDS 2015 :

Criterion	Analysis Value	Design Value	Unit	Analysis/Design
Shear	fv = 90	Fv' = 201	psi	fv/Fv' = 0.45
Bending(+)	fb = 1051	Fb' = 1263	psi	fb/Fb' = 0.83
Live Defl'n	0.03 = <L/999	0.14 = L/360	in	0.21
Total Defl'n	0.09 = L/557	0.20 = L/240	in	0.43

- JAMB AT LOAD-BEARING WALLS UP TO 4'-0"
  DL = 1200 LBS
  LL = 500 LBS
  SL = 850 LBS
  WIND LOAD = 26.3 PSF x 4'/2 = 50 PLF

  <u>USE H-1 2x6 SYP #2, (1) KING AND (1) JACK STUD</u>

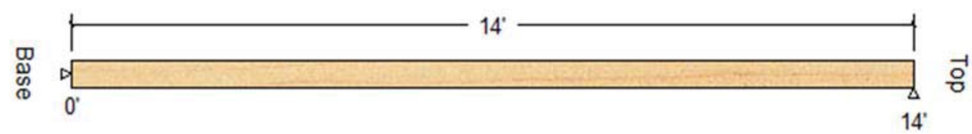
Design Check Calculation Sheet

WoodWorks Sizer 11.1

Loads:

Load	Type	Distribution	Location [ft] Start End	Magnitude Start End	Unit
DL	Dead	Axial	(Ecc. = 0.00")	1200	lbs
LR	Roof constr.	Axial	(Ecc. = 0.00")	500	lbs
WL	Wind C&C	Full UDL		50.0	plf
SL	Roof constr.	Axial	(Ecc. = 0.00")	850	lbs
Self-weight	Dead	Axial		60	lbs

Lateral Reactions (lbs):



Unfactored:			
Dead			
Wind	350		350
Roof Live			
Factored:			
L->R	350		350
Load comb	#4		#4

Lumber n-ply, S. Pine, No.2, 2x6, 2-ply (3"x5-1/2")

Support: Non-wood

Total length: 14.0'; Clear span: 14.0'; volume = 1.6 cu.ft.

Pinned base; Load face = width(b); Built-up fastener: nails; Ke x Lb: 1.0 x 0.0 = 0.0 [ft]; Ke x Ld: 1.0 x 14.0 = 14.0 [ft]; Repetitive factor: applied where permitted (refer to online help);

Analysis vs. Allowable Stress and Deflection using NDS 2015 :

Criterion	Analysis Value	Design Value	Unit	Analysis/Design
Shear	fv = 32	Fv' = 280	psi	fv/Fv' = 0.11
Bending(+)	fb = 972	Fb' = 1600	psi	fb/Fb' = 0.61
Axial	fc = 158	Fc' = 422	psi	fc/Fc' = 0.37
Axial Bearing	fc = 158	Fc* = 1750	psi	fc/Fc* = 0.09
Combined	(axial compression + side load bending)			Eq.3.9-3 = 0.76
Live Defl'n	0.52 = L/323	0.93 = L/180	in	0.56
Total Defl'n	0.52 = L/323	0.93 = L/180	in	0.56

- HEADER AT NON LOAD-BEARING EAST WALL, SPAN = 6'-4"
  - WALL DL = 20 PSF x (18.2' – 7') = 250 PLF
  - ROOF DL = 25 PSF x 2' = 50 PLF
  - ROOF LL = 20 PSF x 2' = 40 PLF
  - ROOF SL = 35 PSF x 2' = 70 PLF

  <u>USE H-2 (3) 2x8 SYP #2</u>

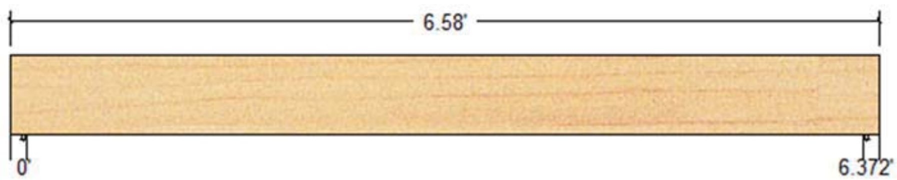
Design Check Calculation Sheet

WoodWorks Sizer 11.1

Loads:

Load	Type	Distribution	Pat- tern	Location [ft] Start	End	Magnitude Start	End	Unit
DL	Dead	Full UDL				300.0		plf
RL	Roof constr.	Full UDL				40.0		plf
SL	Snow	Full UDL				70.0		plf
Self-weight	Dead	Full UDL				8.5		plf

Maximum Reactions (lbs), Bearing Capacities (lbs) and Bearing Lengths (in) :



Unfactored:			
Dead	1014		1014
Snow	230		230
Roof Live	132		132
Factored:			
Total	1244		1244
Bearing:			
Capacity			
Beam	3814		3814
Support	9021		9021
Des ratio			
Beam	0.27		0.27
Support	0.11		0.11
Load comb	#1		#3
Length	1.50		1.50
Min req'd	0.50*		0.50*
Cb	1.00		1.00
Cb min	1.00		1.00
Cb support	1.00		1.00
Fc sup	1350		1350

*Minimum bearing length setting used: 1/2" for end supports
Maximum reaction on at least one support is from a different load combination than the critical one for bearing design, shown here, due to Kd factor. See Analysis results for reaction from critical load combination.

Lumber n-ply, S. Pine, No.2, 2x8, 3-ply (4-1/2"x7-1/4")

Supports: All - Lumber n-ply Column, D.Fir-L No.2
Total length: 6.33'; Clear span: 6.33'; volume = 1.4 cu.ft.
Lateral support: top= at supports, bottom= at supports; Repetitive factor: applied where permitted (refer to online help);

- JAMB AT NON LOAD-BEARING EAST WALL, SPAN = 6'-4"
  DL = 1000 LBS
  LL = 150 LBS
  SL = 250 LBS
  WIND LOAD = 26.3 PSF x 6.33'/2 = 80 PLF

  USE H-2 2x6 SYP #2, (2) KING AND (1) JACK STUD

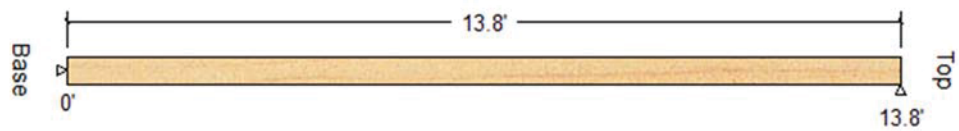
Design Check Calculation Sheet

WoodWorks Sizer 11.1

Loads:

Load	Type	Distribution	Location [ft] Start End	Magnitude Start End	Unit
DL	Dead	Axial	(Ecc. = 0.00")	1000	lbs
LR	Roof constr.	Axial	(Ecc. = 0.00")	150	lbs
WL	Wind C&C	Full UDL		70.0	plf
SL	Snow	Axial	(Ecc. = 0.00")	250	lbs
Self-weight	Dead	Axial		59	lbs

Lateral Reactions (lbs):



Unfactored:			
Dead			
Snow			
Wind	483		483
Roof Live			
Factored:			
L->R	483		483
Load comb	#6		#6

Lumber n-ply, S. Pine, No.2, 2x6, 2-ply (3"x5-1/2")

Support: Non-wood
Total length: 13.8'; Clear span: 13.8'; volume = 1.6 cu.ft.
Pinned base; Load face = width(b); Built-up fastener: nails; Ke x Lb: 1.0 x 0.0 = 0.0 [ft]; Ke x Ld: 1.0 x 13.8 = 13.8 [ft]; Repetitive factor: applied where permitted (refer to online help);

Analysis vs. Allowable Stress and Deflection using NDS 2015 :

Criterion	Analysis Value	Design Value	Unit	Analysis/Design
Shear	fv = 44	Fv' = 280	psi	fv/Fv' = 0.16
Bending(+)	fb = 1322	Fb' = 1600	psi	fb/Fb' = 0.83
Axial	fc = 79	Fc' = 431	psi	fc/Fc' = 0.18
Axial Bearing	fc = 79	Fc* = 1610	psi	fc/Fc* = 0.05
Combined	(axial compression + side load bending)			Eq.3.9-3 = 0.98
Live Defl'n	0.69 = L/241	0.92 = L/180	in	0.75
Total Defl'n	0.69 = L/241	0.92 = L/180	in	0.75

- HEADER AT LOAD-BEARING WALLS UP TO 7'-0"
  WALL DL = 20 PSF x (20.2' – 7') = 265 PLF
  ROOF DL = 25 PSF x 23'/2 = 290 PLF
  ROOF LL = 20 PSF x 23'/2 = 230 PLF
  ROOF SL = 35 PSF x 23'/2 = 400 PLF

  USE H-3 (3) 2x12 SYP #1

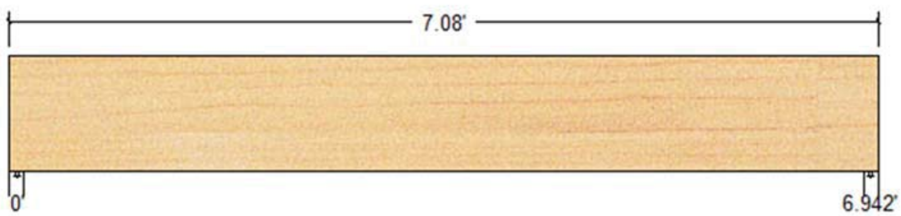
Design Check Calculation Sheet

WoodWorks Sizer 11.1

Loads:

Load	Type	Distribution	Pat-tern	Location [ft] Start	Location [ft] End	Magnitude Start	Magnitude End	Unit
DL	Dead	Full UDL				555.0		plf
RL	Roof constr.	Full UDL				230.0		plf
SL	Snow	Full UDL				400.0		plf
Self-weight	Dead	Full UDL				13.1		plf

Maximum Reactions (lbs), Bearing Capacities (lbs) and Bearing Lengths (in) :



Unfactored:			
Dead	2010		2010
Snow	1416		1416
Roof Live	814		814
Factored:			
Total	3426		3426
Bearing:			
Capacity			
Beam	3814		3814
Support	11527		11527
Des ratio			
Beam	0.90		0.90
Support	0.30		0.30
Load comb	#3		#3
Length	1.50		1.50
Min req'd	1.35		1.35
Cb	1.00		1.00
Cb min	1.00		1.00
Cb support	1.00		1.00
Fc sup	1350		1350

Lumber n-ply, S. Pine, No. 1, 2x12, 3-ply (4-1/2"x11-1/4")

Supports: All - Lumber n-ply Column, D.Fir-L No.2

Total length: 6.83'; Clear span: 6.83'; volume = 2.4 cu.ft.

Lateral support: top= at supports, bottom= at supports; Repetitive factor: applied where permitted (refer to online help);

- JAMB AT LOAD-BEARING WALLS UP TO 7'-0"
  - DL = 2100 LBS
  - LL = 850 LBS
  - SL = 1400 LBS
  - WIND LOAD = 26.3 PSF x 7'/2 = 90 PLF

  USE H-3 2x6 SYP #2, (3) KING AND (1) JACK STUD

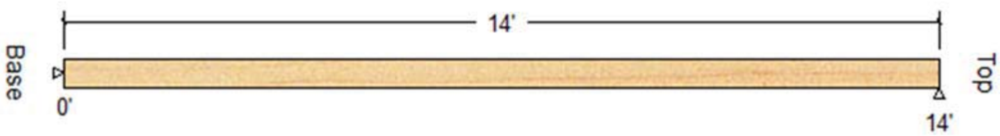
Design Check Calculation Sheet

WoodWorks Sizer 11.1

Loads:

Load	Type	Distribution	Location [ft] Start End	Magnitude Start End	Unit
DL	Dead	Axial	(Ecc. = 0.00")	2100	lbs
LR	Roof constr.	Axial	(Ecc. = 0.00")	850	lbs
WL	Wind C&C	Full UDL		90.0	plf
SL	Snow	Axial	(Ecc. = 0.00")	1400	lbs
Self-weight	Dead	Axial		90	lbs

Lateral Reactions (lbs):



Unfactored:			
Dead			
Snow			
Wind	630		630
Roof Live			
Factored:			
L->R	630		630
Load comb	#6		#6

Lumber n-ply, S. Pine, No.2, 2x6, 3-ply (4-1/2"x5-1/2")

Support: Non-wood

Total length: 14.0'; Clear span: 14.0'; volume = 2.4 cu.ft.

Pinned base; Load face = width(b); Built-up fastener: nails; Ke x Lb: 1.0 x 0.0 = 0.0 [ft]; Ke x Ld: 1.0 x 14.0 = 14.0 [ft]; Repetitive factor: applied where permitted (refer to online help);

Analysis vs. Allowable Stress and Deflection using NDS 2015 :

Criterion	Analysis Value	Design Value	Unit	Analysis/Design
Shear	fv = 38	Fv' = 280	psi	fv/Fv' = 0.14
Bending(+)	fb = 1166	Fb' = 1840	psi	fb/Fb' = 0.63
Axial	fc = 145	Fc' = 420	psi	fc/Fc' = 0.35
Axial Bearing	fc = 145	Fc* = 1610	psi	fc/Fc* = 0.09
Combined	(axial compression + side load bending)			Eq.3.9-3 = 0.83
Live Defl'n	0.62 = L/269	0.93 = L/180	in	0.67
Total Defl'n	0.62 = L/269	0.93 = L/180	in	0.67

- HEADER AT LOAD-BEARING WALLS UP TO 8'-0"

  WALL DL = 20 PSF x (18.2' – 8') = 205 PLF
  ROOF DL = 25 PSF x 16'/2 = 200 PLF
  ROOF LL = 20 PSF x 16'/2 = 160 PLF
  ROOF SL = 25 PSF x 16'/2 = 200 PLF

  USE H-3 (3) 2x12 SYP #1

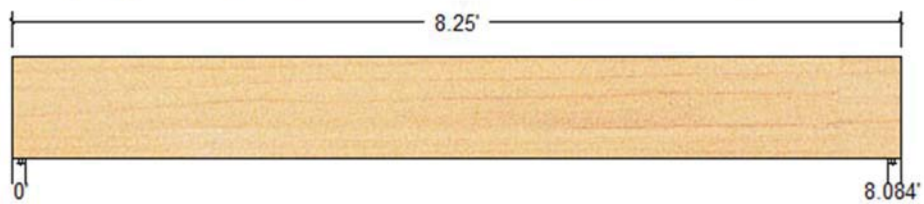
Design Check Calculation Sheet

WoodWorks Sizer 11.1

Loads:

Load	Type	Distribution	Pat-tern	Location [ft] Start	End	Magnitude Start	End	Unit
DL	Dead	Full UDL				405.0		plf
RL	Roof constr.	Full UDL				160.0		plf
SL	Snow	Full UDL				200.0		plf
Self-weight	Dead	Full UDL				13.1		plf

Maximum Reactions (lbs), Bearing Capacities (lbs) and Bearing Lengths (in) :



Unfactored:		
Dead	1724	1724
Snow	825	825
Roof Live	660	660
Factored:		
Total	2549	2549
Bearing:		
Capacity		
Beam	3814	3814
Support	11527	11527
Des ratio		
Beam	0.67	0.67
Support	0.22	0.22
Load comb	#3	#3
Length	1.50	1.50
Min req'd	1.00	1.00
Cb	1.00	1.00
Cb min	1.00	1.00
Cb support	1.00	1.00
Fc sup	1350	1350

Lumber n-ply, S. Pine, No. 1, 2x12, 3-ply (4-1/2"x11-1/4")

Supports: All - Lumber n-ply Column, D.Fir-L No.2
Total length: 8.0'; Clear span: 8.0'; volume = 2.8 cu.ft.
Lateral support: top= at supports, bottom= at supports; Repetitive factor: applied where permitted (refer to online help);

- JAMB AT LOAD-BEARING WALLS UP TO 8'-0"
  DL = 1750 LBS
  LL = 600 LBS
  SL = 830 LBS
  WIND LOAD = 26.3 PSF x 8'/2 = 100 PLF

  USE H-3 2x6 SYP #2, (3) KING AND (1) JACK STUD

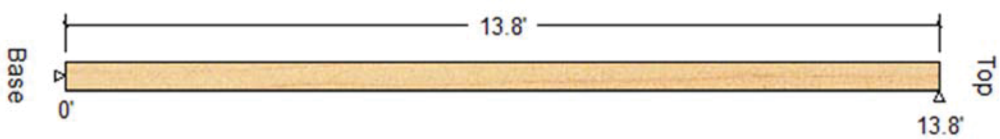
Design Check Calculation Sheet

WoodWorks Sizer 11.1

Loads:

Load	Type	Distribution	Location [ft] Start End	Magnitude Start End	Unit
DL	Dead	Axial	(Ecc. = 0.00")	1800	lbs
WL	Wind C&C	Full UDL		100.0	plf
LR	Roof constr.	Axial	(Ecc. = 0.00")	600	lbs
SL	Snow	Axial	(Ecc. = 0.00")	830	lbs
Self-weight	Dead	Axial		89	lbs

Lateral Reactions (lbs):



Unfactored:			
Dead			
Snow			
Wind	690		690
Roof Live			
Factored:			
L->R	690		690
Load comb	#6		#6

Lumber n-ply, S. Pine, No.2, 2x6, 3-ply (4-1/2"x5-1/2")

Support: Non-wood

Total length: 13.8'; Clear span: 13.8'; volume = 2.4 cu.ft.

Pinned base; Load face = width(b); Built-up fastener: nails; Ke x Lb: 1.0 x 0.0 = 0.0 [ft]; Ke x Ld: 1.0 x 13.8 = 13.8 [ft]; Repetitive factor: applied where permitted (refer to online help);

Analysis vs. Allowable Stress and Deflection using NDS 2015 :

Criterion	Analysis Value	Design Value	Unit	Analysis/Design
Shear	fv = 42	Fv' = 280	psi	fv/Fv' = 0.15
Bending(+)	fb = 1259	Fb' = 1840	psi	fb/Fb' = 0.68
Axial	fc = 110	Fc' = 431	psi	fc/Fc' = 0.25
Axial Bearing	fc = 110	Fc* = 1610	psi	fc/Fc* = 0.07
Combined	(axial compression + side load bending)			Eq.3.9-3 = 0.85
Live Defl'n	0.65 = L/253	0.92 = L/180	in	0.71
Total Defl'n	0.65 = L/253	0.92 = L/180	in	0.71

- HEADER AT NON LOAD-BEARING WEST WINDOW, SPAN = 16'-3"
  WALL DL = 20 PSF x (20.2' – 13') = 140 PLF

  <u>USE H-4 (3) 1 3/4x9 1/2 LVL'S</u>

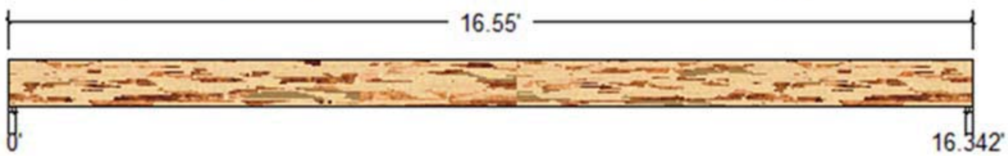
Design Check Calculation Sheet

WoodWorks Sizer 11.1

Loads:

Load	Type	Distribution	Pat-tern	Location [ft] Start	End	Magnitude Start	End	Unit
DL	Dead	Full UDL				140.0		plf
Self-weight	Dead	Full UDL				14.4		plf

Maximum Reactions (lbs), Bearing Capacities (lbs) and Bearing Lengths (in) :



Unfactored:			
Dead	1276		1276
Factored:			
Total	1276		1276
Bearing:			
Capacity			
Beam	5906		5906
Support	10525		10525
Des ratio			
Beam	0.22		0.22
Support	0.12		0.12
Load comb	#1		#1
Length	1.50		1.50
Min req'd	0.50*		0.50*
Cb	1.00		1.00
Cb min	1.00		1.00
Cb support	1.00		1.00
Fc sup	1350		1350

*Minimum bearing length setting used: 1/2" for end supports

LVL n-ply, 1.8E, 2200Fb, 1-3/4"x9-1/2", 3-ply (5-1/4"x9-1/2")

Supports: All - Lumber n-ply Column, D.Fir-L No.2
Total length: 16.3'; Clear span: 16.3'; volume = 5.6 cu.ft.
Lateral support: top= at supports, bottom= at supports; Repetitive factor: applied where permitted (refer to online help);

Analysis vs. Allowable Stress and Deflection using NDS 2015 :

Criterion	Analysis Value	Design Value	Unit	Analysis/Design
Shear	fv = 34	Fv' = 256	psi	fv/Fv' = 0.13
Bending(+)	fb = 783	Fb' = 961	psi	fb/Fb' = 0.82
Live Defl'n	negligible			
Total Defl'n	0.37 = L/534	0.82 = L/240	in	0.45

- JAMB AT NON LOAD-BEARING WEST WINDOW, SPAN = 16'-3"
  DL = 1500 LBS
  WIND LOAD = 26.30 PSF x 1' = 25 PLF

  USE H-4 2x6 SYP #2, (3) KING AND (1) JACK STUD

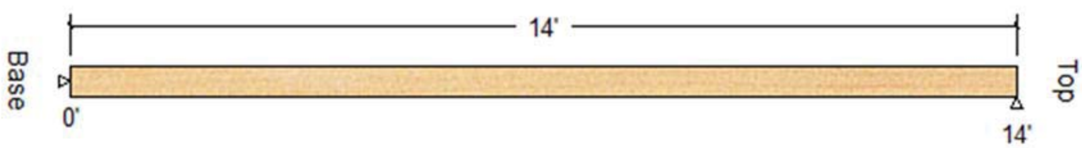
Design Check Calculation Sheet

WoodWorks Sizer 11.1

Loads:

Load	Type	Distribution	Location [ft] Start End	Magnitude Start End	Unit
DL	Dead	Axial	(Ecc. = 0.00")	1500	lbs
WL	Wind C&C	Full UDL		25.0	plf
Self-weight	Dead	Axial		82	lbs

Lateral Reactions (lbs):



Unfactored:			
Dead			
Wind	175		175
Factored:			
L->R	175		175
Load comb	#2		#2

Lumber n-ply, D.Fir-L, No.2, 2x6, 3-ply (4-1/2"x5-1/2")

Support: Non-wood

Total length: 14.0'; Clear span: 14.0'; volume = 2.4 cu.ft.

Pinned base; Load face = width(b); Built-up fastener: nails; Ke x Lb: 1.0 x 0.0 = 0.0 [ft]; Ke x Ld: 1.0 x 14.0 = 14.0 [ft]; Repetitive factor: applied where permitted (refer to online help);

Analysis vs. Allowable Stress and Deflection using NDS 2015 :

Criterion	Analysis Value	Design Value	Unit	Analysis/Design
Shear	fv = 11	Fv' = 288	psi	fv/Fv' = 0.04
Bending(+)	fb = 324	Fb' = 2153	psi	fb/Fb' = 0.15
Axial	fc = 64	Fc' = 462	psi	fc/Fc' = 0.14
Axial Bearing	fc = 64	Fc* = 1336	psi	fc/Fc* = 0.05
Combined	(axial compression + side load bending)			Eq.3.9-3 = 0.19
Live Defl'n	0.15 = <L/999	0.93 = L/180	in	0.16
Total Defl'n	0.15 = <L/999	0.93 = L/180	in	0.16

BACKUP INFORMATION:

Simpson Strong-Tie® Wood Construction Connectors

H/TSP

SIMPSON Strong-Tie

Seismic and Hurricane Ties (cont.)

These products are available with additional corrosion protection. For more information, see p. 14.

SS For stainless-steel fasteners, see p. 21.

SD Many of these products are approved for installation with Strong-Drive® SD Connector screws. See pp. 348–352 for more information.

Model No.	Ga.	Fasteners (in.) To Rafters/Truss	To Plates	To Studs	DF/SP Allowable Loads Uplift (160)	Lateral (160) F_1	Lateral (160) F_2	Uplift with 0.131" x 1½" Nails (160)	SPF/HF Allowable Loads Uplift (160)	Lateral (160) F_1	Lateral (160) F_2	Uplift with 0.131" x 1½" Nails (160)	Code Ref.
H1	18	(6) 0.131 x 1½	(4) 0.131 x 2½	—	480	510	190	455	425	440	165	370	IBC, FL, LA
H1.81Z	18	(6) 0.131 x 1½	(4) 0.131 x 2½	—	540	440	170	460	465	380	130	395	—
H2A	18	(5) 0.131 x 1½	(2) 0.131 x 1½	(5) 0.131 x 1½	525	130	55	—	495	130	55	—	IBC, FL, LA
H2ASS	18	(5) 0.131 x 1½	(2) 0.131 x 1½	(5) 0.131 x 1½	400	130	55	400	345	130	55	345	—
H2.5A	18	(5) 0.131 x 2½	(5) 0.131 x 2½	—	700	110	110	625	615	110	110	540	IBC, FL, LA
H2.5ASS	18	(5) 0.131 x 2½	(5) 0.131 x 2½	—	440	75	70	365	380	75	70	310	—
H2.5T	18	(5) 0.131 x 2½	(5) 0.131 x 2½	—	590	135	145	480	565	135	145	475	IBC, FL, LA
H3	18	(4) 0.131 x 2½	(4) 0.131 x 2½	—	400	210	170	400	365	180	145	290	

Simpson Strong-Tie® Wood Construction Connectors

DSP/SSP/SP/SPH/RSP4/TSP/CS

SIMPSON Strong-Tie

Stud Plate Ties (cont.)

These products are available with additional corrosion protection. For more information, see p. 14.

SS For stainless-steel fasteners, see p. 21.

SD Many of these products are approved for installation with Strong-Drive® SD Connector screws. See pp. 348–352 for more information.

Model No.	Dimensions (in.) W	L	Stud	Plate Width	Fasteners (in.) Stud	Plate	Allowable Uplift Loads DF/SP Side[8] (160)	DF/SP Center[9] (160)	SPF/HF Side[8] (160)	SPF/HF Center[9] (160)	Code Ref.
SP1	3½	5⁵⁄₁₆	2x	—	(6) 0.148 x 3	(4) 0.148 x 3	555	555	535	535	
SP2	3½	6⅝	2x	—	(6) 0.148 x 3	(6) 0.148 x 3	1,010	1,010	605	605	
SP4	3⁹⁄₁₆	7¼	2x	4x	(6) 0.148 x 1½	—	415	825	355	710	
SP6	5⁹⁄₁₆	7¾	2x	6x	(6) 0.148 x 1½	—	415	825	355	710	
SP8	7⁵⁄₁₆	8⁵⁄₁₆	2x	8x	(6) 0.148 x 1½	—	415	825	355	710	

Simpson Strong-Tie® Wood Construction Connectors

SIMPSON Strong-Tie

H/TSP

Seismic and Hurricane Ties (cont.)

These products are available with additional corrosion protection. For more information, see p. 14.

SS For stainless-steel fasteners, see p. 21.

SD Many of these products are approved for installation with Strong-Drive® SD Connector screws. See pp. 348–352 for more information.

Model No.	Ga.	Fasteners (in.) To Rafters/Truss	Fasteners (in.) To Plates	Fasteners (in.) To Studs	DF/SP Allowable Loads Uplift (160)	DF/SP Lateral (160) F_1	DF/SP Lateral (160) F_2	Uplift with 0.131" x 1½" Nails (160)	SPF/HF Allowable Loads Uplift (160)	SPF/HF Lateral (160) F_1	SPF/HF Lateral (160) F_2	Uplift with 0.131" x 1½" Nails (160)	Code Ref.
H1	18	(6) 0.131 x 1½	(4) 0.131 x 2½	—	480	510	190	455	425	440	165	370	IBC, FL, LA
H1.81Z	18	(6) 0.131 x 1½	(4) 0.131 x 2½	—	540	440	170	460	465	380	130	395	—
H2A	18	(5) 0.131 x 1½	(2) 0.131 x 1½	(5) 0.131 x 1½	525	130	55	—	495	130	55	—	IBC, FL, LA
H2ASS	18	(5) 0.131 x 1½	(2) 0.131 x 1½	(5) 0.131 x 1½	400	130	55	400	345	130	55	345	—
H2.5A	18	(5) 0.131 x 2½	(5) 0.131 x 2½	—	700	110	110	625	615	110	110	540	IBC, FL, LA
H2.5ASS	18	(5) 0.131 x 2½	(5) 0.131 x 2½	—	440	75	70	365	380	75	70	310	—
H2.5T	18	(5) 0.131 x 2½	(5) 0.131 x 2½	—	590	135	145	480	565	135	145	475	IBC, FL, LA
H3	18	(4) 0.131 x 2½	(4) 0.131 x 2½	—	400	210	170	400	365	180	145	290	IBC, FL, LA
H3SS	18	(4) 0.131 x 2½	(4) 0.131 x 2½	—	280	145	120	275	225	100	85	210	—
H6 (to Plates)	16	—	(8) 0.131 x 2½	(8) 0.131 x 2½	930	—	—	—	800	—	—	—	IBC, FL, LA
H6 (to Rim)	16	(8) 0.131 x 2½	—	(8) 0.131 x 2½	1,230	—	—	—	1,065	—	—	—	IBC, FL, LA
H7Z	16	(4) 0.131 x 2½	(2) 0.131 x 1½	(8) 0.131 x 2½	830	410	—	—	715	355	—	—	IBC, FL, LA
H8	18	(5) 0.148 x 1½	(5) 0.148 x 1½	—	780	95	90	630	710	95	90	510	IBC, FL, LA
H8SS	18	(5) 0.148 x 1½	(5) 0.148 x 1½	—	610	90	120	440	370	90	55	335	—

Simpson Strong-Tie® Wood Construction Connectors

SIMPSON Strong-Tie

A

Angle

Our line of angles provides a way to make a wide range of 90° connections.

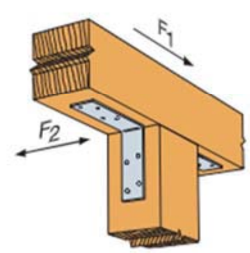
Material: A21 and A23 — 18 ga.; all other A angles — 12 ga.

Finish: Galvanized. Some products available in stainless steel or ZMAX® coating. See Corrosion Information, pp. 12–15.

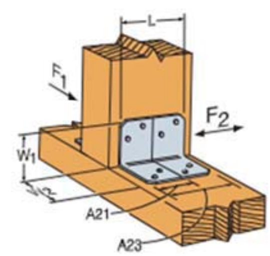
Installation:

- Use all specified fasteners; see General Notes

Codes: See p. 11 for Code Reference Key Chart



A44 Installation
(A33 similar)



A21/A23 Installation

These products are available with additional corrosion protection. For more information, see p. 14.

SD Many of these products are approved for installation with Strong-Drive® SD Connector screws. See pp. 348–352 for more information.

Model No.	Dimensions (in.) W_1	Dimensions (in.) W_2	Dimensions (in.) L	Fasteners (in.) Base Bolts	Fasteners (in.) Base Nails	Fasteners (in.) Post Bolts	Fasteners (in.) Post Nails	Allowable Loads DF/SP (160) F_1[3]	Allowable Loads DF/SP (160) F_2	Code Ref.
A21	2	1½	1⅜	—	(2) 0.148 x 1½	—	(2) 0.148 x 1½	330	150	IBC, FL, LA
A23	2	1½	2¾	—	(4) 0.148 x 1½	—	(4) 0.148 x 1½	680	535	IBC, FL, LA
A33	3	3	1½	—	(4) 0.148 x 3	—	(4) 0.148 x 3	765	340	IBC, FL, LA

Simpson Strong-Tie® Wood Construction Connectors

SIMPSON Strong-Tie

CS/CMST/CMSTC/CSHP

Coiled Straps (cont.)

These products are available with additional corrosion protection. For more information, see p. 14.

SS For stainless-steel fasteners, see p. 21.

SD Many of these products are approved for installation with Strong-Drive® SD Connector screws. See pp. 348–352 for more information.

Model No.	Total L	Ga.	DF/SP Fasteners (in.)	DF/SP End Length (in.)	SPF/HF Fasteners (in.)	SPF/HF End Length (in.)	Allowable Tension Loads (160)	Code Ref.
CMST12	40'	12	(74) 0.162 x 2½	33	(84) 0.162 x 2½	38	9,215	IBC, FL, LA
			(86) 0.148 x 2½	39	(98) 0.148 x 2½	44	9,215	
CMST14	52½'	14	(56) 0.162 x 2½	26	(66) 0.162 x 2½	30	6,475	
			(66) 0.148 x 2½	30	(76) 0.148 x 2½	34	6,475	
CMSTC16	54'	16	(50) 0.148 x 3¼	20	(58) 0.148 x 3¼	25	4,690	
CS14	100'	14	(26) 0.148 x 2½	15	(30) 0.148 x 2½	16	2,490	
			(30) 0.131 x 2½	16	(36) 0.131 x 2½	19	2,490	
SS CS16	150'	16	(20) 0.148 x 2½	11	(22) 0.148 x 2½	13	1,705	
			(22) 0.131 x 2½	13	(26) 0.131 x 2½	15	1,705	
CS20	250'	20	(12) 0.148 x 2½	7	(14) 0.148 x 2½	9	1,030	
			(14) 0.131 x 2½	9	(16) 0.131 x 2½	9	1,030	

	Project SALAD AND GO #2001 – LEE'S SUMMIT, MO			Job Ref. 24-011
ELLISON GAGE & ASSOCIATES, PLLC Consulting Structural Engineers 5068 W. Plano Parkway Suite 200 Plano, TX 75093 Ofc: (972) 354-8855 Fax: (972) 354-8856 www.ellisongage.com	Description ROOF AND WALL DESIGN (SEC. RW)	Calc. by BG	4/9/2024	Description RW23

NDS TABLE 12N NAIL SHEAR VALUES:

Side Member Thickness t_s in.	Nail Diameter D in.	Common Wire Nail (Pennyweight)	Box Nail (Pennyweight)	Sinker Nail (Pennyweight)	G=0.67 Red Oak lbs.	G=0.55 Mixed Maple Southern Pine lbs.	G=0.5 Douglas Fir-Larch lbs.	G=0.49 Douglas Fir-Larch (N) lbs.	G=0.46 Douglas Fir(S) Hem-Fir(N) lbs.	G=0.43 Hem-Fir lbs.	G=0.42 Spruce-Pine-Fir lbs.
3/4	0.099		6d	7d	73	61	55	54	51	48	47
	0.113	6d	8d	8d	94	79	72	71	65	58	57
	0.120			10d	107	89	80	77	71	64	62
	0.128		10d		121	101	87	84	78	70	68
	0.131	8d			127	104	90	87	80	73	70
	0.135		16d	12d	135	108	94	91	84	76	74
	0.148	10d	20d	16d	154	121	105	102	94	85	83
	0.162	16d	40d		183	138	121	117	108	99	96
	0.177			20d	200	153	134	130	121	111	107
	0.192	20d		30d	206	157	138	134	125	114	111
	0.207	30d		40d	216	166	147	143	133	122	119
	0.225	40d			229	178	158	154	144	132	129
	0.244	50d		60d	234	182	162	158	147	136	132
1	0.099		6d	7d	73	61	55	54	51	48	47
	0.113	6d[4]	8d	8d	94	79	72	71	67	63	61
	0.120			10d	107	89	81	80	76	71	69
	0.128		10d		121	101	93	91	86	80	79
	0.131	8d			127	106	97	95	90	84	82
	0.135		16d	12d	135	113	103	101	96	89	86
	0.148	10d	20d	16d	154	128	118	115	109	99	96
	0.162	16d	40d		184	154	141	137	125	113	109
	0.177			20d	213	178	155	150	138	125	121
	0.192	20d		30d	222	183	159	154	142	128	124
	0.207	30d		40d	243	192	167	162	149	135	131
	0.225	40d			268	202	177	171	159	144	140
	0.244	50d		60d	274	207	181	175	162	148	143
1-1/4	0.099		6d[4]	7d[4]	73	61	55	54	51	48	47
	0.113	6d[4]	8d	8d[4]	94	79	72	71	67	63	61
	0.120			10d	107	89	81	80	76	71	69
	0.128		10d		121	101	93	91	86	80	79
	0.131	8d[4]			127	106	97	95	90	84	82
	0.135		16d	12d	135	113	103	101	96	89	88
	0.148	10d	20d	16d	154	128	118	115	109	102	100
	0.162	16d	40d		184	154	141	138	131	122	120
	0.177			20d	213	178	163	159	151	141	136
	0.192	20d		30d	222	185	170	166	157	145	140
	0.207	30d		40d	243	203	186	182	169	152	147
	0.225	40d			268	224	200	193	177	160	155
	0.244	50d		60d	276	230	204	197	181	163	158
1-1/2	0.099			7d[4]	73	61	55	54	51	48	47
	0.113		8d[4]	8d[4]	94	79	72	71	67	63	61
	0.120			10d	107	89	81	80	76	71	69
	0.128		10d		121	101	93	91	86	80	79
	0.131	8d[4]			127	106	97	95	90	84	82
	0.135		16d	12d	135	113	103	101	96	89	88
	0.148	10d	20d	16d	154	128	118	115	109	102	100
	0.162	16d	40d		184	154	141	138	131	122	120
	0.177			20d	213	178	163	159	151	141	138
	0.192	20d		30d	222	185	170	166	157	147	144
	0.207	30d		40d	243	203	186	182	172	161	158
	0.225	40d			268	224	205	201	190	178	172
	0.244	50d		60d	276	230	211	206	196	181	175

AWNING DESIGN

- DEAD LOADS:

	MAX	MIN (UPLIFT)
METAL DECK	2.0 PSF	2.0 PSF
FRAMING	4.0 PSF	2.0 PSF
MISC	4.0 PSF	0.0 PSF
TOTAL	**10 PSF**	**4 PSF**

- LIVE LOADS:
  ROOF **20 PSF** (NOT TO BE REDUCED)

- SNOW LOADS:
  (SEE LOAD CRITERIA SECTION)
  DESIGN ROOF SNOW LOAD, P_f = 20 PSF x 1.2 (THERMAL FACTOR) = 21.8 PSF

- SNOW DRIFT:
  PER LOAD CRITERIA SHEET PAGE LC2, ADD 15 PSF x 3.4' LONG TRIANGULAR SNOW DRIFT TO AWNINGS.

- WIND LOADS:
  (SEE NEXT PAGE)

- SEISMIC LOADS

HORIZONTAL SEISMIC FORCE:

$F_p = 0.4a_pS_{DS}W_p\ (1+2z/h)\ /\ R_PI_P$ [EQ. 13.3-1]

a_p = 1.0
S_{DS} = 0.106
R_P = 2.5
I_P = 1.0
z/h = 1.0

$F_p = 0.4(1.0)(0.106)W_p\ (1+2(1))\ /\ 2.5(1.0) = 0.051\ W_p$ **<- CONTROLS**

F_p SHALL NOT BE GREATER THAN,
$F_p = 1.6\ S_{DS}I_PW_p$ [EQ. 13.3-1]
$F_P = 0.170\ W_p$

F_p SHALL NOT BE LESS THAN,
$F_p = 0.3\ S_{DS}I_PW_p$ [EQ. 13.3-1]
$F_P = 0.032\ W_p$

F_P = 0.051 (30 PSF) = 1.5 PSF **<- WIND CONTROLS**

VERTICAL SEISMIC FORCE:
$F_V = \pm 0.2\ S_{DS}W_p$

$F_V = \pm 0.021\ W_p$

F_V = ±0.021 (30 PSF) = ±0.64 PSF **<- WIND CONTROLS**

WIND DESIGN PRESSURES

HORIZONTAL WIND FORCE:

PER ASCE 7-16, SECTION 27.3.4
$p_{LAT} = q_pGC_{pn}$ [EQ. 27.3-2]
WINDWARD $GC_{pn} = 1.5$
LEEWARD $GC_{pn} = -1.0$

q_p = VELOCITY PRESSURE = $0.00256K_ZK_{ZT}K_dK_EV^2$

BASIC WIND SPEED (V)	= 110 MPH
EXPOSURE	= C
H	= 10 FT
K_Z	= 0.85
K_{ZT}	= 1.0
K_d	= 0.85
K_E	= 0.968

$q_h = 0.00256(0.85)(1.0)(0.85)(0.959)(110)^2$ = 21.6 PSF

$p_{LAT} = q_hGC_{pn}$ = 1.5(21.6) = 32.4 PSF (WINDWARD, LRFD)
= -1.0(21.6) = -21.6 PSF (LEEWARD, LRFD)

VERTICAL WIND FORCE:

PER ASCE 7-16, SECTION 30.11

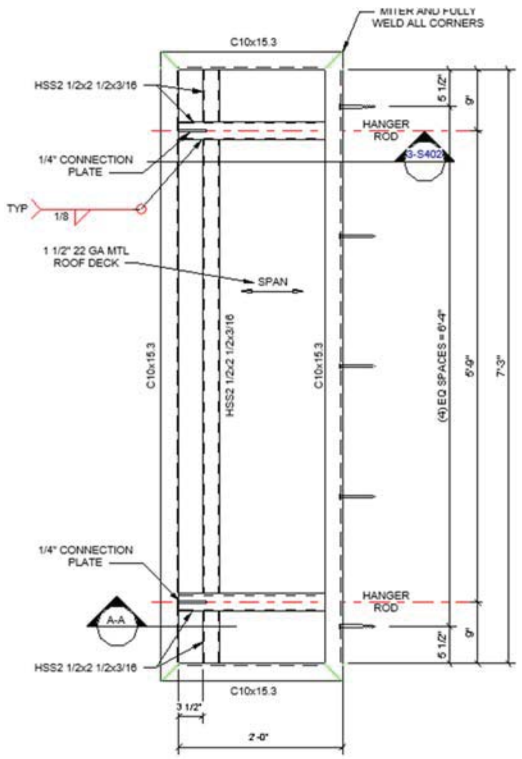
$P = q_h (GC_P)$ [EQ. 30.11-1]

q_h = 21.6 PSF
Hc / He = 10' / 14' = 0.71
GC_P = +0.90, -0.90 (FIGURE 30.11-1B)

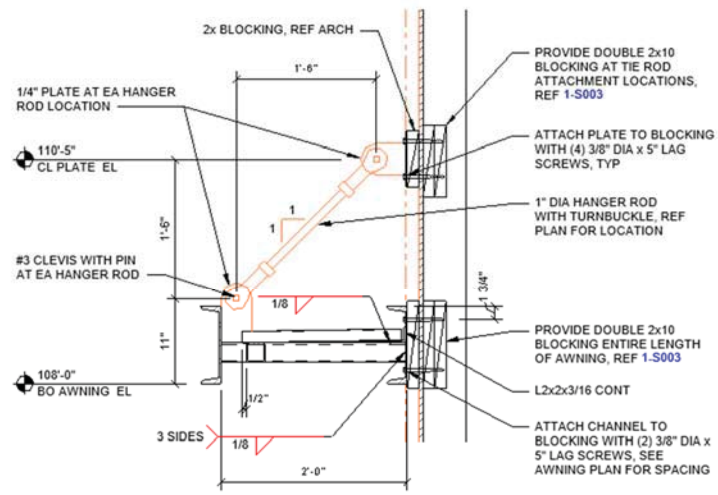
P = +19.4 PSF (LRFD), +11.7 PSF (ASD)
P = -19.4 PSF (LRFD), -11.7 PSF (ASD)

ELLISON GAGE & ASSOCIATES, PLLC Consulting Structural Engineers 5068 W. Plano Parkway Suite 200 Plano, TX 75093 Ofc: (972) 354-8855 Fax: (972) 354-8856 www.ellisongage.com	Project SALAD AND GO #2001 – LEE'S SUMMIT, MO			Job Ref. 24-011
	Description AWNING DESIGN (SEC. A)	Calc by. BG	Date 04/24/2024	Sheet No. A3

AWNING #1 DESIGN



AWNING - 1



3 SECTION AT AWNING
1" = 1'-0"

3-S401:

METAL DECK DESIGN:

MAX. UNIFORM GRAVITY LOAD = 10 PSF DL + 37 PSF SL
= 47 PSF TL

DECK SPAN = 1'-6"
WIND UPLIFT FORCE = 19.4 PSF (LRFD) = 11.7 PSF (ASD)

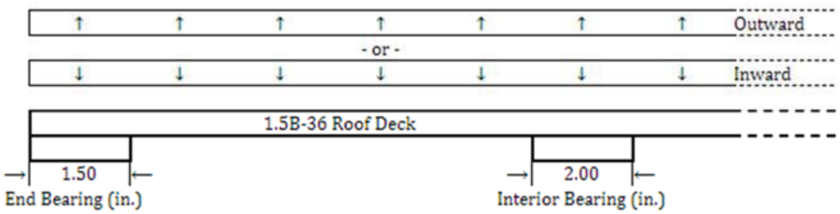
USE 1 ½" – 22 GA TYPE B ROOF DECK

22 Gage 1.5B-36 Grade 50
Uniform Allowable Load Table, ASD (psf)
For End Lapped Deck

NUCOR VULCRAFT GROUP

36 / 7 Connection Pattern to Supports with
5/8" Visible Dia. Arc Spot Weld

Support Member A36
$0.25 \le t_2$ (in.)



Inward Uniform Allowable Load Table, ASD (psf)

Span	(ft-in)	1'-6"	2'-0"	2'-6"	3'-0"	3'-6"	4'-0"	4'-6"	5'-0"
1	Wn/Ω	1076	807	540	375	275	211	167	135
	L/240	-	-	-	-	237	159	112	81
2	Wn/Ω	699	524	419	349	284	219	173	141
	L/240	-	-	-	-	-	-	-	-
3	Wn/Ω	794	596	476	397	340	271	215	175
	L/240	-	-	-	-	-	-	-	-

Uplift (Outward) Uniform Allowable Load Table, ASD (psf)

Span	(ft-in)	1'-6"	2'-0"	2'-6"	3'-0"	3'-6"	4'-0"	4'-6"	5'-0"
1	Wn/Ω	648	486	389	324	278	223	176	143.0
	Rn/Ω	648	486	389	324	278	243	216	194
	L/240	-	-	-	-	272	182	128	93
2	Wn/Ω	518	389	311	259	222	194	164	133
	Rn/Ω	518	389	311	259	222	194	173	155
	L/240	-	-	-	-	-	-	-	-
3	Wn/Ω	589	442	353	294	252	221	196	166
	Rn/Ω	589	442	353	294	252	221	196	177
	L/240	-	-	-	-	-	-	-	153

INWARD ALLOWABLE UNIFORM LOAD = 1076 PSF > 47 PSF *OK*
OUTWARD (UPLIFT) ALLOWABLE UNIFORM LOAD = 648 PSF > 11.7 PSF *OK*

DESIGN C10x15.3:

L = 5.75 FT

UNFACTORED LOADS			TRIB AREA		
DL =	30 PSF	X	0.75	=	22.5 PLF
L_R =	20 PSF	X	0.75	=	15.0 PLF
SL =	37 PSF	X	0.75	=	27.75 PLF
0.6W =	11.7 PSF	X	0.75	=	8.78 PLF
$0.6W_{UPLIFT}$ =	-11.7 PSF	X	0.75	=	-8.78 PLF

ASD LOAD COMBINATIONS		
D =	22.5 PLF	
D+L_R =	37.5 PLF	
D+S =	**50.3 PLF**	**<--CONTROLS**
D+.75L_R+.75(.6W) =	40.3 PLF	
D+.75SL+.75(.6W) =	49.9 PLF	

UNFACTORED UPLIFT		
0.6DL + 0.6W =	8.2 PLF	NO NET UPLIFT

USE **C10x15.3**

STEEL CODE: AISC 360-10 LRFD

SPAN INFORMATION (ft): I-End (0.00,0.00) J-End (7.25,0.00)

Beam Size (User Selected) = C10X15.3 Fy = 36.0 ksi
Total Beam Length (ft) = 7.25
Cantilever on left (ft) = 0.75
Cantilever on right (ft) = 0.75
Mp (kip-ft) = 47.70
Top flange not braced by decking.

LINE LOADS (k/ft):

Load	Dist (ft)	DL	LL	PartL
1	0.000	0.023	0.050	0.000
	0.750	0.023	0.050	0.000
2	0.750	0.023	0.050	0.000
	6.500	0.023	0.050	0.000
3	6.500	0.023	0.050	0.000
	7.250	0.023	0.050	0.000

SHEAR (Ultimate): Max Vu (1.2DL+1.6LL) = 0.31 kips 0.90Vn = 46.66 kips

MOMENTS (Ultimate):

Span	Cond	LoadCombo	Mu kip-ft	@ ft	Lb ft	Cb	Phi	Phi*Mn kip-ft
Left	Max -	1.2DL+1.6LL	-0.0	0.8	0.8	1.00	0.90	42.93
Center	Max +	1.2DL+1.6LL	0.4	3.6	5.8	1.14	0.90	42.00
	Max -	1.4DL	-0.0	3.6	5.8	1.15	0.90	42.32
Right	Max -	1.2DL+1.6LL	-0.0	6.5	0.8	1.00	0.90	42.93
Controlling		1.2DL+1.6LL	0.4	3.6	5.8	1.14	0.90	42.00

REACTIONS (kips):

	Left	**Right**
DL reaction	0.08	0.08
Max +LL reaction	0.18	0.18
Max -LL reaction	-0.00	-0.00
Max +total reaction (factored)	0.39	0.39

DESIGN HSS2 1/2x2 1/2x3/16:

L = 5.75 FT

UNFACTORED LOADS			TRIB AREA		
DL =	30 PSF	X	0.75	=	22.5 PLF
L_R =	20 PSF	X	0.75	=	15.0 PLF
SL =	37 PSF	X	0.75	=	27.75 PLF
0.6W =	11.7 PSF	X	0.75	=	8.78 PLF
$0.6W_{UPLIFT}$ =	-11.7 PSF	X	0.75	=	-8.78 PLF

ASD LOAD COMBINATIONS		
D =	22.5 PLF	
D+L_R =	37.5 PLF	
D+S =	**50.3 PLF**	**<--CONTROLS**
D+.75L_R+.75(.6W) =	40.3 PLF	
D+.75SL+.75(.6W) =	49.9 PLF	

UNFACTORED UPLIFT		
0.6DL + 0.6W =	8.2 PLF	NO NET UPLIFT

USE **HSS2 1/2x2 1/2x3/16**

INPUT DATA & DESIGN SUMMARY

MEMBER SHAPE (Tube, Pipe, or WF) & SIZE		HSS2-1/2X2-1/2X3/16	< == **Tube**
STEEL YIELD STRESS	F_y =	46	ksi
AXIAL COMPRESSION FORCE	P =	0	kips, ASD
STRONG AXIS EFFECTIVE LENGTH	KL_x =	5.75	ft
WEAK AXIS EFFECTIVE LENGTH	KL_y =	5.75	ft
STRONG AXIS BENDING MOMENT	M_{rx} =	0.4	ft-kips, ASD
STRONG AXIS BENDING UNBRACED LENGTH	L_b =	5.75	ft, (AISC 360-05 F2.2.c)
STRONG DIRECTION SHEAR LOAD, ASD	V_{strong} =	0.22	kips
WEAK AXIS BENDING MOMENT	M_{ry} =	0	ft-kips, ASD
WEAK DIRECTION SHEAR LOAD, ASD	V_{weak} =	0	kips

THE DESIGN IS ADEQUATE.

ANALYSIS

CHECK COMBINED COMPRESSION AND BENDING CAPACITY (AISC 360-05, H1)

$$\begin{cases} \frac{P_r}{P_c}+\frac{8}{9}\left(\frac{M_{rx}}{M_{cx}}+\frac{M_{ry}}{M_{cy}}\right) , & for \ \frac{P_r}{P_c}\geq 0.2 \\ \frac{P_r}{2P_c}+\left(\frac{M_{rx}}{M_{cx}}+\frac{M_{ry}}{M_{cy}}\right) , & for \ \frac{P_r}{P_c}< 0.2 \end{cases} = 0.13 < 1.0 \quad \textbf{[Satisfactory]}$$

Where $P_c = P_n / \Omega_c$ = 49 / 1.67 = 29.44 kips, (AISC 360-05 Chapter E)
> P_r **[Satisfactory]**

$M_{cx} = M_n / \Omega_b$ = 5.06 / 1.67 = 3.03 ft-kips, (AISC 360-05 Chapter F)
> M_{rx} **[Satisfactory]**

$M_{cy} = M_n / \Omega_b$ = 5.06 / 1.67 = 3.03 ft-kips, (AISC 360-05 Chapter F)
> M_{ry} **[Satisfactory]**

CHECK SHEAR CAPACITY (AISC 360-05, G2)

$V_{n,strong} / \Omega_v$ = 368.0 / 1.67 = 220.4 kips > V_{strong} = 0.2 kips **[Satisfactory]**

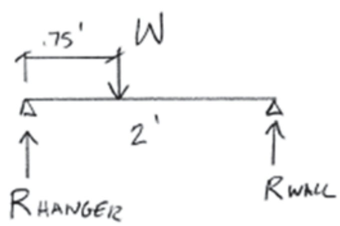
$V_{n,weak} / \Omega_v$ = 73.6 / 1.67 = 44.1 kips > V_{weak} = 0.0 kips **[Satisfactory]**

FIND LOAD ON HANGER ROD:

UNFACTORED LOADS			TRIB AREA		
DL =	30 PSF	X	16.3	=	489 LBS
L_R =	20 PSF	X	16.3	=	326 LBS
SL =	37 PSF	X	16.3	=	604 LBS
0.6W =	11.7 PSF	X	16.3	=	191 LBS
$0.6W_{UPLIFT}$ =	-11.7 PSF	X	16.3	=	-191 LBS

ASD LOAD COMBINATIONS		
D =	489 LBS	
D+L_R =	816 LBS	
D+S =	**1093 LBS**	**<--CONTROLS**
D+.75L_R+.75(.6W) =	877 LBS	
D+.75SL+.75(.6W) =	1085 LBS	

UNFACTORED UPLIFT		
0.6DL + 0.6W =	179 LBS	NO NET UPLIFT



W = LOAD AT HANGER = 1100 LBS / 2 = 550 LBS
R_{HANGER} = 550 x 1.25' / 2' = 345 LBS
R_{WALL} = 205 LBS

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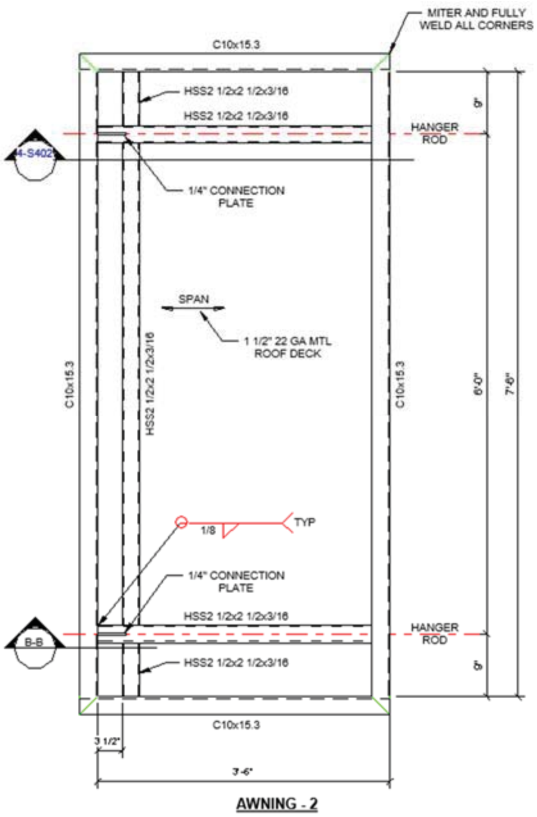
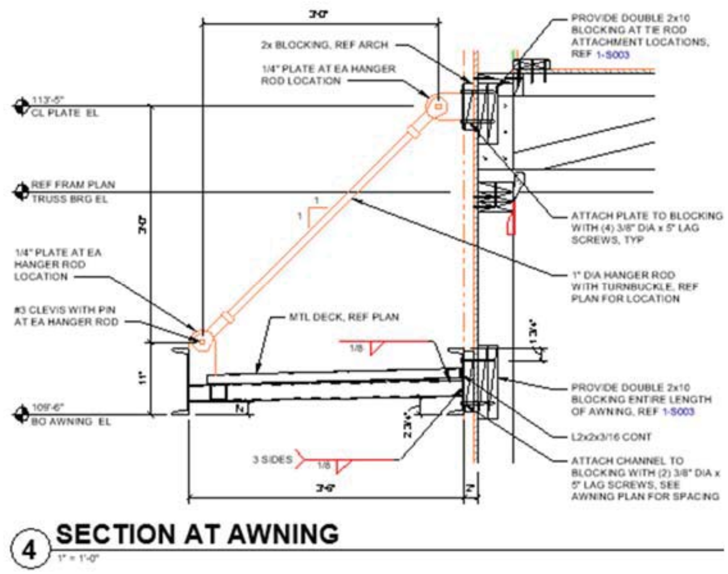
	Description AWNING DESIGN (SEC. A)	Calc by. BG	Date 04/24/2024	Sheet No. A8

AWNING #2 DESIGN





4-S401:

METAL DECK DESIGN:

MAX. UNIFORM GRAVITY LOAD = 10 PSF DL + 37 PSF SL
= 47 PSF TL

DECK SPAN = 3'-6"
WIND UPLIFT FORCE = 19.4 PSF (LRFD) = 11.7 PSF (ASD)

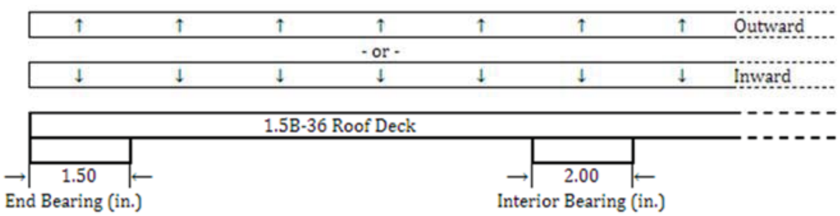
USE 1 ½" – 22 GA ROOF DECK

22 Gage 1.5B-36 Grade 50
Uniform Allowable Load Table, ASD (psf)
For End Lapped Deck

NUCOR VULCRAFT GROUP

36 / 7 Connection Pattern to Supports with
5/8" Visible Dia. Arc Spot Weld

Support Member A36
$0.25 \le t_2$ (in.)



Inward Uniform Allowable Load Table, ASD (psf)

Span	(ft-in)	1'-6"	2'-0"	2'-6"	3'-0"	3'-6"	4'-0"	4'-6"	5'-0"
1	Wn/Ω	1076	807	540	375	275	211	167	135
	L/240	-	-	-	-	237	159	112	81
2	Wn/Ω	699	524	419	349	284	219	173	141
	L/240	-	-	-	-	-	-	-	-
3	Wn/Ω	794	596	476	397	340	271	215	175
	L/240	-	-	-	-	-	-	-	-

Uplift (Outward) Uniform Allowable Load Table, ASD (psf)

Span	(ft-in)	1'-6"	2'-0"	2'-6"	3'-0"	3'-6"	4'-0"	4'-6"	5'-0"
1	Wn/Ω	648	486	389	324	278	223	176	143.0
	Rn/Ω	648	486	389	324	278	243	216	194
	L/240	-	-	-	-	272	182	128	93
2	Wn/Ω	518	389	311	259	222	194	164	133
	Rn/Ω	518	389	311	259	222	194	173	155
	L/240	-	-	-	-	-	-	-	-
3	Wn/Ω	589	442	353	294	252	221	196	166
	Rn/Ω	589	442	353	294	252	221	196	177
	L/240	-	-	-	-	-	-	-	153

INWARD ALLOWABLE UNIFORM LOAD = 275 PSF > 47 PSF *OK*
OUTWARD (UPLIFT) ALLOWABLE UNIFORM LOAD = 278 PSF > 11.7 PSF *OK*

DESIGN C10x15.3:

L = 6.0 FT

UNFACTORED LOADS			TRIB AREA		
DL =	20 PSF	X	1.5	=	30 PLF
L_R =	20 PSF	X	1.5	=	30.0 PLF
SL =	37 PSF	X	1.5	=	55.5 PLF
0.6W =	11.7 PSF	X	1.5	=	17.6 PLF
$0.6W_{UPLIFT}$ =	-11.7 PSF	X	1.5	=	-17.6 PLF

ASD LOAD COMBINATIONS		
D =	30.0 PLF	
D+L_R =	60.0 PLF	
D+S =	**85.5 PLF**	**<--CONTROLS**
D+.75L_R+.75(.6W) =	65.7 PLF	
D+.75SL+.75(.6W) =	84.8 PLF	

UNFACTORED UPLIFT		
0.6DL + 0.6W =	7.5 PLF	NO NET UPLIFT

USE **C10x15.3**

STEEL CODE: AISC 360-10 LRFD

SPAN INFORMATION (ft): I-End (0.00,0.00) J-End (7.50,0.00)

Beam Size (User Selected) = C10X15.3 Fy = 36.0 ksi
Total Beam Length (ft) = 7.50
Cantilever on left (ft) = 0.75
Cantilever on right (ft) = 0.75
Mp (kip-ft) = 47.70
Top flange not braced by decking.

LINE LOADS (k/ft):

Load	Dist (ft)	DL	LL	PartL
1	0.000	0.030	0.083	0.000
	0.750	0.030	0.083	0.000
2	0.750	0.030	0.083	0.000
	6.750	0.030	0.083	0.000
3	6.750	0.030	0.083	0.000
	7.500	0.030	0.083	0.000

SHEAR (Ultimate): Max Vu (1.2DL+1.6LL) = 0.51 kips 0.90Vn = 46.66 kips

MOMENTS (Ultimate):

Span	Cond	LoadCombo	Mu kip-ft	@ ft	Lb ft	Cb	Phi	Phi*Mn kip-ft
Left	Max -	1.2DL+1.6LL	-0.0	0.8	0.8	1.00	0.90	42.93
Center	Max +	1.2DL+1.6LL	0.7	3.8	6.0	1.14	0.90	41.36
	Max -	1.2DL+1.6LL	-0.0	3.8	6.0	1.20	0.90	42.93
Right	Max -	1.2DL+1.6LL	-0.0	6.8	0.8	1.00	0.90	42.93
Controlling		1.2DL+1.6LL	0.7	3.8	6.0	1.14	0.90	41.36

REACTIONS (kips):

	Left	**Right**
DL reaction	0.11	0.11
Max +LL reaction	0.32	0.32
Max -LL reaction	-0.00	-0.00
Max +total reaction (factored)	0.64	0.64

DESIGN HSS2 1/2x2 1/2x3/16:

L = 6.0 FT

UNFACTORED LOADS			TRIB AREA		
DL =	20 PSF	X	1.5	=	30 PLF
L_R =	20 PSF	X	1.5	=	30.0 PLF
SL =	37 PSF	X	1.5	=	55.5 PLF
0.6W =	11.7 PSF	X	1.5	=	17.6 PLF
$0.6W_{UPLIFT}$ =	-11.7 PSF	X	1.5	=	-17.6 PLF

ASD LOAD COMBINATIONS		
D =	30.0 PLF	
D+L_R =	60.0 PLF	
D+S =	**85.5 PLF**	**<--CONTROLS**
D+.75L_R+.75(.6W) =	65.7 PLF	
D+.75SL+.75(.6W) =	84.8 PLF	

UNFACTORED UPLIFT		
0.6DL + 0.6W =	7.5 PLF	NO NET UPLIFT

USE **HSS2 1/2x2 1/2x3/16**

INPUT DATA & DESIGN SUMMARY

MEMBER SHAPE (Tube, Pipe, or WF) & SIZE		HSS2-1/2X2-1/2X3/16	< == **Tube**
STEEL YIELD STRESS	F_y =	46	ksi
AXIAL COMPRESSION FORCE	P =	0	kips, ASD
STRONG AXIS EFFECTIVE LENGTH	KL_x =	6	ft
WEAK AXIS EFFECTIVE LENGTH	KL_y =	6	ft
STRONG AXIS BENDING MOMENT	M_{rx} =	0.6	ft-kips, ASD
STRONG AXIS BENDING UNBRACED LENGTH	L_b =	6	ft, (AISC 360-05 F2.2.c)
STRONG DIRECTION SHEAR LOAD, ASD	V_{strong} =	0.35	kips
WEAK AXIS BENDING MOMENT	M_{ry} =	0	ft-kips, ASD
WEAK DIRECTION SHEAR LOAD, ASD	V_{weak} =	0	kips

THE DESIGN IS ADEQUATE.

ANALYSIS

CHECK COMBINED COMPRESSION AND BENDING CAPACITY (AISC 360-05, H1)

$$\begin{cases} \frac{P_r}{P_c}+\frac{8}{9}\left(\frac{M_{rx}}{M_{cx}}+\frac{M_{ry}}{M_{cy}}\right), & for\ \frac{P_r}{P_c}\geq 0.2 \\ \frac{P_r}{2P_c}+\left(\frac{M_{rx}}{M_{cx}}+\frac{M_{ry}}{M_{cy}}\right), & for\ \frac{P_r}{P_c}< 0.2 \end{cases} = 0.20 < 1.0$$ **[Satisfactory]**

Where $P_c = P_n / \Omega_c$ = 48 / 1.67 = 28.50 kips, (AISC 360-05 Chapter E)
> P_r **[Satisfactory]**

$M_{cx} = M_n / \Omega_b$ = 5.06 / 1.67 = 3.03 ft-kips, (AISC 360-05 Chapter F)
> M_{rx} **[Satisfactory]**

$M_{cy} = M_n / \Omega_b$ = 5.06 / 1.67 = 3.03 ft-kips, (AISC 360-05 Chapter F)
> M_{ry} **[Satisfactory]**

CHECK SHEAR CAPACITY (AISC 360-05, G2)

$V_{n,strong} / \Omega_v$ = 368.0 / 1.67 = 220.4 kips > V_{strong} = 0.4 kips **[Satisfactory]**

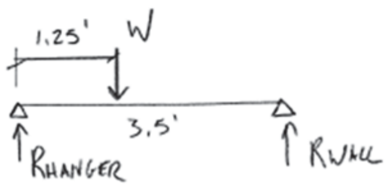
$V_{n,weak} / \Omega_v$ = 73.6 / 1.67 = 44.1 kips > V_{weak} = 0.0 kips **[Satisfactory]**

FIND LOAD ON HANGER ROD:

UNFACTORED LOADS			TRIB AREA		
DL =	20 PSF	X	25.4	=	508 LBS
L_R =	20 PSF	X	25.4	=	508 LBS
SL =	37 PSF	X	25.4	=	939 LBS
0.6W =	11.7 PSF	X	25.4	=	297 LBS
$0.6W_{UPLIFT}$ =	-11.7 PSF	X	25.4	=	-297 LBS

ASD LOAD COMBINATIONS		
D =	508 LBS	
D+L_R =	1015 LBS	
D+S =	**1446 LBS**	**<--CONTROLS**
D+.75L_R+.75(.6W) =	1111 LBS	
D+.75SL+.75(.6W) =	1434 LBS	

UNFACTORED UPLIFT		
0.6DL + 0.6W =	126 LBS	NO NET UPLIFT



W = LOAD AT HANGER = 1450 LBS / 2 = 725 LBS

R_{HANGER} = 725 x 2.25' / 3.5' = 470 LBS

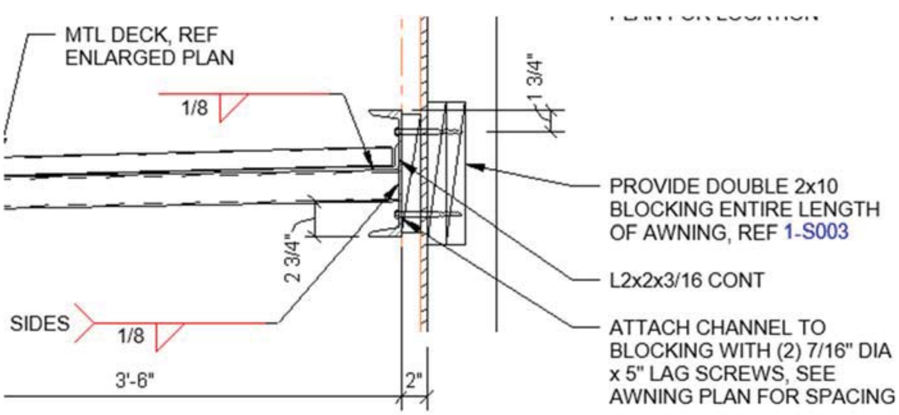
R_{WALL} = 725 - 470 = 255 LBS

HANGER ROD DESIGN

WORST CASE OCCURS AT AWNING #2
MAX ASD TENSILE FORCE T = 470 LBS / 0.707 = 670 LBS
MAX ASD COMPRESSIVE FORCE = C = 0 LBS (NO NET UPLIFT)

USE 1" DIAMETER ROD WITH NO. 3 CLEVIS
$A = 0.79\ IN^2$
L = 4.3'
r = 0.25 IN
TENSION:
$T_{ALL} = 36\ KSI \times 0.79\ IN^2 / 1.67 = 17.0\ K > 670\ LBS$ **OK**

AWNING CONNECTION TO WALL



WORST CASE LOADING OCCURS AT AWNING #2

VERTICAL FORCE AT WALL = 255 LBS x 2 = 510 LBS

ATTACH CHANNEL TO WALL WITH (2) 3/8" DIA LAG BOLTS AT (5) LOCATIONS (10 TOTAL BOLTS)

SHEAR FORCE PER ANCHOR V_{ANCHOR} = 600 LBS / 10 ANCHORS = 60 LBS

LAG SCREW CAPACITY (SEE CALC BELOW):

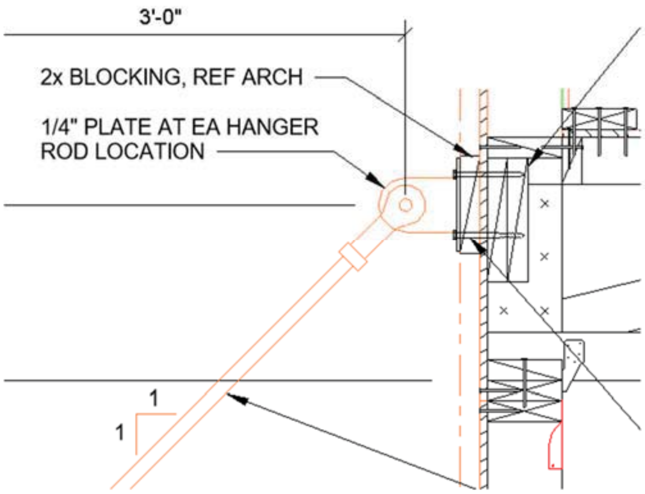
T_{ALL} = 543 LBS
V_{ALL} = 276 LBS

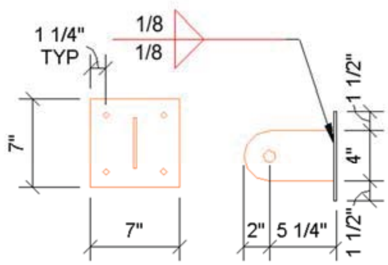
INTERACTION:

0 / 543 + 60 / 276 = 0.22 < 1.0 **OK**

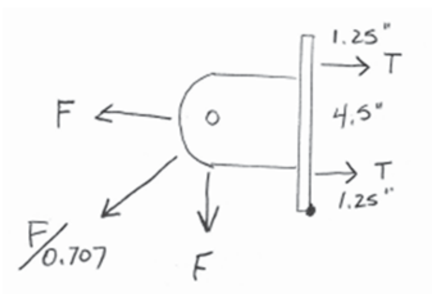
USE 3/8" DIA LAG SCREWS ARE OK

HANGER ROD CONNECTION TO WALL





TYPICAL UPPER CONNECTION PLATE



HANGER FORCE F = 470 LBS

ATTACH PLATE TO WALL WITH (4) 3/8” DIA LAG BOLTS

SOLVE FOR T,
1.25T + 5.75T = 5.25F + 3.5F
T = 590 LBS
T_{ANCHOR} = 590 LBS / 2 ANCHORS = 295 LBS PER ANCHOR

SHEAR FORCE PER ANCHOR V_{ANCHOR} = 470 LBS / 4 ANCHORS = 120 LBS

LAG SCREW CAPACITY (SEE CALC BELOW):

T_{ALL} = 588 LBS
V_{ALL} = 422 LBS

INTERACTION:

295 / 543 + 120 / 276 = 0.978 < 1.0 **OK**

USE (4) 3/8” DIA LAG SCREWS

LAG SCREW CAPACITY IN TENSION:

Design Method	Allowable Stress Design (ASD)
Connection Type	Withdrawal loading
Fastener Type	Lag Screw
Loading Scenario	N/A
	Submit Initial Values

Main Member Type	Douglas Fir-Larch
Main Member Thickness	3.5 in.
Side Member Type	Steel
Side Member Thickness	1/4 in.
Washer Thickness	0 in.
Nominal Diameter	3/8 in.
Length	3 in.
Load Duration Factor	C_D = 1.0
Wet Service Factor	C_M = 1.0
End Grain Factor	C_eg = 1.0
Temperature Factor	C_t = 1.0

Calculate Connection Capacity

Connection Yield Mode Descriptions | Limits of Use

Diaphragm Factor Help | Load Duration Factor Help | Technical Help

Show Printable View

Adjusted ASD Capacity	543 lbs.

LAG SCREW CAPACITY IN SHEAR:

Main Member Type	Douglas Fir-Larch
Main Member Thickness	3.5 in.
Main Member: Angle of Load to Grain	0
Side Member Type	Steel
Side Member Thickness	1/4 in.
Side Member: Angle of Load to Grain	0
Washer Thickness	0 in.
Nominal Diameter	3/8 in.
Length	3 in.
Load Duration Factor	C_D = 1.0
Wet Service Factor	C_M = 1.0
End Grain Factor	C_eg = 1.0
Temperature Factor	C_t = 1.0

Calculate Connection Capacity

Connection Yield Mode Descriptions | Limits of Use

Diaphragm Factor Help | Load Duration Factor Help | Technical Help

Show Printable View

Connection Yield Modes

Im	939 lbs.
Is	1441 lbs.
II	452 lbs.
IIIm	501 lbs.
IIIs	323 lbs.
IV	276 lbs.

Adjusted ASD Capacity	**276 lbs.**

	Project SALAD AND GO #2001 – LEE'S SUMMIT, MO			Job Ref. 24-011
ELLISON GAGE & ASSOCIATES, PLLC Consulting Structural Engineers 5068 W. Plano Parkway Suite 200 Plano, TX 75093 Ofc: (972) 354-8855 Fax: (972) 354-8856 www.ellisongage.com	Description AWNING DESIGN (SEC. A)	Calc by. BG	Date 04/24/2024	Sheet No. A17

CHECK WALL STUDS AT AWNINGS

WALL TRIB AREA FOR WIND = 14'*14'/3 = 65 SF, USE 50 SF TRIB AREA FOR WIND LOADS
LRFD WALL WIND PRESSURE = **23.6 PSF** (WORST CASE)
LRFD PARAPET WIND PRESSURE = **64.7 PSF** (WORST CASE)
LIMIT WALL DEFLECTION TO L/360

- LOAD-BEARING WALLS AT AWNING #1
  - TOP PLATE EL = 112'-4"
  - DL FROM WALL ABOVE = 20 PSF x 20.2' – 12.3' = 160 PLF
  - ROOF DL = 25 PSF x 17' / 2 = 220 PLF
  - ROOF LL = 20 PSF x 17' / 2 = 170 PLF
  - ROOF SL = 35 PSF x 17' / 2 = 300 PLF
  - HORIZ LOAD FROM AWNING HANGER ROD:
    - P_{HORIZ} = 345 LBS x 0.67 = 230 LBS (ASD) APPLIED AT 10.4'
  - VERT LOAD FROM HANGER ROD AND CANOPY
    - P_{VERT} = 345 LBS + 150 LBS = 500 LBS x 0.67 = 350 LBS

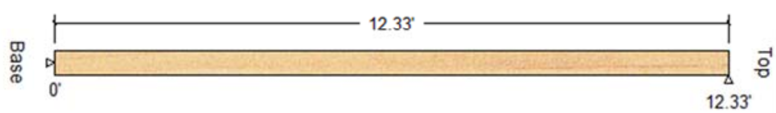
<u>USE 2x6 SYP #2 WALL STUDS SPACED AT 16" OC</u>

Design Check Calculation Sheet
WoodWorks Sizer 11.1

Loads:

Load	Type	Distribution	Location [ft] Start End	Magnitude Start End	Unit
WIND	Wind C&C	Full Area		23.60(16.0")	psf
DL	Dead	Axial UDL	(Ecc. = 0.92")	380	plf
LR	Roof live	Axial UDL	(Ecc. = 0.92")	170	plf
HANGER HORIZ L	Dead	Point	10.40	230	lbs
HANGER VERT DL	Dead	Axial UDL	(Ecc. = 0.92")	350	plf
SL	Snow	Axial UDL	(Ecc. = 0.92")	300	plf

Lateral Reactions (lbs):



Unfactored:			
Dead	42		188
Snow	2		-2
Wind	194		194
Roof Live	1		-1
Factored:			
L->R	158		304
Load comb	#7		#7

AWNING #1
Lumber Stud, S. Pine, No.2, 2x6 (1-1/2"x5-1/2")

Support: Lumber Stud Bottom plate, S. Pine No.2; Bearing length = stud thickness; continuous lower support Spaced at 16.0" c/c; Total length: 12.33'; Clear span: 12.205'; volume = 0.7 cu.ft.
Pinned base; Load face = width(b); Ke x Lb: 1.0 x 0.0 = 0.0 [ft]; Ke x Ld: 1.0 x 12.33 = 12.33 [ft]; Repetitive factor: applied where permitted (refer to online help);

Analysis vs. Allowable Stress and Deflection using NDS 2015 :

Criterion	Analysis Value	Design Value	Unit	Analysis/Design
Shear	fv = 34	Fv' = 157	psi	fv/Fv' = 0.22
Bending(+)	fb = 694	Fb' = 1035	psi	fb/Fb' = 0.67
Axial	fc = 166	Fc' = 528	psi	fc/Fc' = 0.32
Combined	(axial + eccentric & side load bending)			Eq.15.4-1= 0.95
Axial Bearing	fc = 166	Fc* = 1610	psi	fc/Fc* = 0.10
Support Bearing	fcp = 166	Fcp = 706	psi	fcp/Fcp = 0.24
Live Defl'n	0.23 = L/631	1.23 = L/120	in	0.19
Total Defl'n	0.52 = L/282	1.23 = L/120	in	0.42

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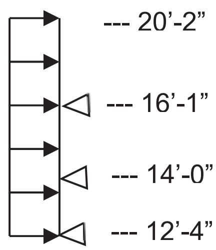
	Description AWNING DESIGN (SEC. A)	Calc by. BG	Date 04/24/2024	Sheet No. A18

- LOAD-BEARING WALLS AT AWNING #2

  HANGER ROD LOAD IS APPLIED TO 2x FRAMING ABOVE TOP PLATE (EL=13.4')



TOP OF WALL = 20'-2"

HORIZ LOAD FROM AWNING HANGER ROD:

P_{HORIZ} = 470 LBS (ASD) APPLIED AT 13.4'

VERT LOAD FROM HANGER ROD

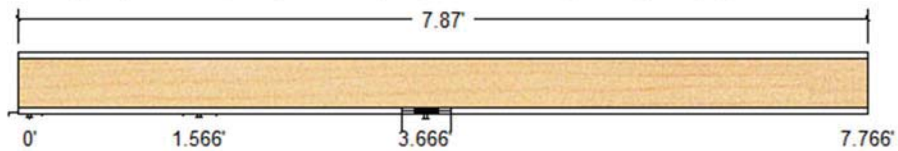
P_{VERT} = 470 LBS

<u>USE 2x6 SYP #2 WALL STUDS SPACED AT 16" OC</u>

Loads:

Load	Type	Distribution	Pat-tern	Location [ft] Start	End	Magnitude Start	End	Unit
WL	Wind C&C	Partial Area	No	0.00	1.67	23.60(16.0")		psf
HANGER ROD	Dead	Point	No	1.07		470		lbs
PARAPET WL	Wind C&C	Partial Area	No	1.67	7.87	64.70(16.0")		psf

Maximum Reactions (lbs), Bearing Capacities (lbs) and Bearing Lengths (in) :



Unfactored:							
Dead	142		357		-29		
Wind	141		-427		874		
Factored:							
Uplift			-42		-29		
Total	226		357		507		
Bearing:							
Capacity							
Joist	1271		1589		4979		
Support	1406		1758		6445		
Des ratio							
Joist	0.18		0.22		0.10		
Support	0.16		0.20		0.08		
Load comb	#3		#1		#2		
Length	1.50		1.50		5.50		
Min req'd	0.50*		0.50*		0.43**		
Cb	1.00		1.25		1.07		
Cb min	1.00		1.75		1.75		
Cb support	1.00		1.25		1.25		
Fcp sup	625		625		625		

**Minimum bearing length governed by the required width of the supporting member.
Maximum reaction on at least one support is from a different load combination than the critical one for bearing design, shown here, due to Kd factor. See Analysis results for reaction from critical load combination.
Bearing for wall supports is perpendicular-to-grain bearing on top plate. No stud design included.

Lumber-soft, S. Pine, No.2, 2x6 (1-1/2"x5-1/2")

Supports: 1,2 - Lumber-soft Sill plate, D.Fir-L No.2; 3 - Lumber Stud Wall, D.Fir-L No.2;
Roof joist spaced at 16.0" c/c; Total length: 7.87'; Clear span: 1.669', 2.098', 4.098'; volume = 0.5 cu.ft.
Lateral support: top= full, bottom= full; Repetitive factor: applied where permitted (refer to online help);

Analysis vs. Allowable Stress and Deflection using NDS 2015 :

Criterion	Analysis Value	Design Value	Unit	Analysis/Design
Shear	fv = 60	Fv' = 157	psi	fv/Fv' = 0.38
Bending(+)	fb = 217	Fb' = 1035	psi	fb/Fb' = 0.21
Bending(-)	fb = 690	Fb' = 1840	psi	fb/Fb' = 0.38
Deflection:				
Interior Live	-0.00 = <L/999	0.11 = L/240	in	0.03
Total	-0.00 = <L/999	0.11 = L/240	in	0.04
Cantil. Live	0.12 = L/416	0.41 = L/120	in	0.29
Total	0.12 = L/399	0.41 = L/120	in	0.30

FOUNDATION DESIGN

- FOUNDATION DESIGN REQUIREMENTS (FROM SOILS REPORT)

TYPE	= SHALLOW SPREAD FOOTINGS
ALLOWABLE BEARING PRESSURE	= 2,100 PSF
MINIMUM FOOTING DIMENSION	= 24" (INDIVIDUAL)
	= 18" (CONTINUOUS)
MINIMUM FOOTING EMBEDMENT	= 36" BELOW EXTERIOR GRADE (EXTERIOR FOOTINGS)

- CONTINUOUS FOOTING DESIGN (WORST CASE NORTH/SOUTH WALLS)

WALL DL = 20 PSF x 20.0' = 400 PLF
ROOF DL = 25 PSF x 24' / 2 = 300 PLF + 50 PLF (RTU) = 350 PLF
ROOF SL = 35 PSF x 24' / 2 = 420 PLF

TOTAL DL = 750 PLF
TOTAL LL = 420 PLF

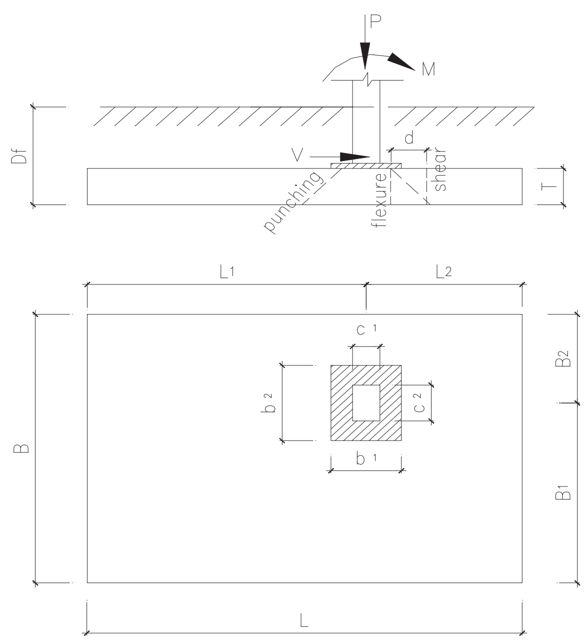
USE 2'-0" WIDE BY 2'-8" DEEP CONC FOOTING REINF WITH (3) #6 CONT TOP AND BOTTOM
REFER TO 1-S301

INPUT DATA

COLUMN WIDTH	c_1	=	6	in
COLUMN DEPTH	c_2	=	12	in
BASE PLATE WIDTH	b_1	=	6	in
BASE PLATE DEPTH	b_2	=	12	in
FOOTING CONCRETE STRENGTH	f_c'	=	4	ksi
REBAR YIELD STRESS	f_y	=	60	ksi
AXIAL DEAD LOAD	P_{DL}	=	0.75	k
AXIAL LIVE LOAD	P_{LL}	=	0.42	k
LATERAL LOAD (0=WIND, 1=SEISMIC)		=	0	Wind,ASD
WIND AXIAL LOAD	P_{LAT}	=	0	k, ASD
WIND MOMENT LOAD	M_{LAT}	=	0	ft-k, ASD
WIND SHEAR LOAD	V_{LAT}	=	0	k, ASD
SURCHARGE	q_s	=	0	ksf
SOIL WEIGHT	w_s	=	0	kcf
FOOTING EMBEDMENT DEPTH	D_f	=	3.33	ft
FOOTING THICKNESS	T	=	32	in
ALLOW SOIL PRESSURE	Q_a	=	2.1	ksf
FOOTING WIDTH	B_1	=	0.5	ft
	B_2	=	0.5	ft
FOOTING LENGTH	L_1	=	1	ft
	L_2	=	1	ft
REINFORCING SIZE		#	6	

DESIGN SUMMARY

FOOTING WIDTH	B	=	1.00	ft
FOOTING LENGTH	L	=	2.00	ft
FOOTING THICKNESS	T	=	32	in
LONGITUDINAL REINF., TOP	Not Required			
LONGITUDINAL REINF., BOT.	2 # 6 @ 6 in o.c.			
TRANSVERSE REINF., BOT.	2 # 6 @ 18 in o.c.			



THE FOOTING DESIGN IS ADEQUATE.

CHECK SOIL BEARING CAPACITY (ACI 318-05 SEC.15.2.2)

Service Loads	CASE 1		CASE 2		CASE 3		
P	1.2		1.2		1.2		k
e	0.0		0.0		0.0		ft (from center of footing)
q_s B L	0.0		0		0.0		k, (surcharge load)
(0.15-w_s)T B L	0.8		0.8		0.8		k, (footing increased)
Σ P	2.0		2.0		2.0		k
e_L	0.0	< L/6	0.0	< L/6	0.0	< L/6	ft
e_B	0.0	< B/6	0.0	< B/6	0.0	< B/6	ft
q_L	1.0		1.0		1.0		k / ft
q_{max}	1.0		1.0		1.0		ksf
q_{allow}	2.1		2.8		2.8		ksf

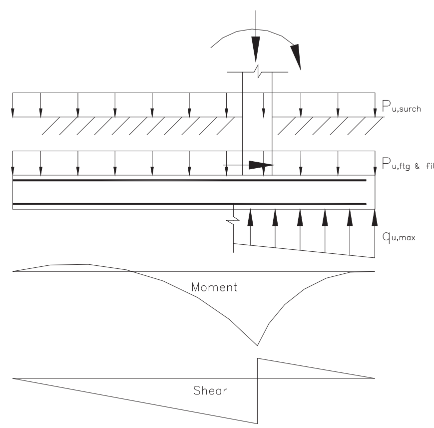
Where

$$q_L = \begin{cases} \dfrac{(\Sigma P)\left(1+\dfrac{6e_L}{L}\right)}{L}, & for \quad e_L \le \dfrac{L}{6} \\ \dfrac{2(\Sigma P)}{3(0.5L-e_L)}, & for \quad e_L > \dfrac{L}{6} \end{cases} \qquad q_{MAX} = \begin{cases} \dfrac{q_L\left(1+\dfrac{6e_B}{B}\right)}{B}, & for \quad e_B \le \dfrac{B}{6} \\ \dfrac{2q_L}{3(0.5B-e_B)}, & for \quad e_B > \dfrac{B}{6} \end{cases}$$

[Satisfactory]

DESIGN FLEXURE & CHECK FLEXURE SHEAR

(ACI 318-05 SEC.15.4.2, 10.2, 10.3.5, 10.5.4, 7.12.2, 12.2, 12.5, 15.5.2, 11.1.3.1, & 11.3)



$$q_{u,MAX} = \begin{cases} \dfrac{(\Sigma P_u)\left(1+\dfrac{6e_u}{L}\right)}{BL}, & for \quad e_u \le \dfrac{L}{6} \\ \dfrac{2(\Sigma P_u)}{3B(0.5L-e_u)}, & for \quad e_u > \dfrac{L}{6} \end{cases} \qquad \rho_{MAX} = \frac{0.85\beta_1 f_c'}{f_y}\frac{\varepsilon_u}{\varepsilon_u+\varepsilon_t}$$

$$\rho = \frac{0.85 f_c'\left(1-\sqrt{1-\dfrac{M_u}{0.383bd^2 f_c'}}\right)}{f_y} \qquad \rho_{MIN} = MIN\left(0.0018\frac{T}{d}\ ,\ \frac{4}{3}\rho\right)$$

FACTORED SOIL PRESSURE

Factored Loads	CASE 1		CASE 2		CASE 3		
P_u	1.6		1.3		0.7		k
e_u	0.0		0.0		0.0		ft
$\gamma\, q_s$ B L	0.0		0.0		0.0		k, (factored surcharge load)
γ[0.15T + w_s(D_f - T)]BL	1.0		1.0		0.7		k, (factored footing & backfill loads)
$\Sigma\, P_u$	2.5		2.3		1.4		k
e_u	0.0	< L/6	0.0	< L/6	0.0	< L/6	ft
$q_{u,\ max}$	1.266		1.140		0.698		ksf

DESIGN FLEXURE

Location	$M_{u,max}$		d (in)	ρ_{min}	ρ_{reqD}	ρ_{max}	s_{max}	use	ρ_{provD}
Top Longitudinal	0.0	ft-k	29.63	0.0000	0.0000	0.0206	no limit	Not Required	0.0000
Bottom Longitudinal	0.2	ft-k	28.63	0.0000	0.0000	0.0206	18	2 # 6 @ 6 in o.c.	0.0026
Bottom Transverse	0	ft-k / ft	28.25	0.0000	0.0000	0.0206	18	2 # 6 @ 18 in o.c.	0.0013

[Satisfactory]

CHECK FLEXURE SHEAR

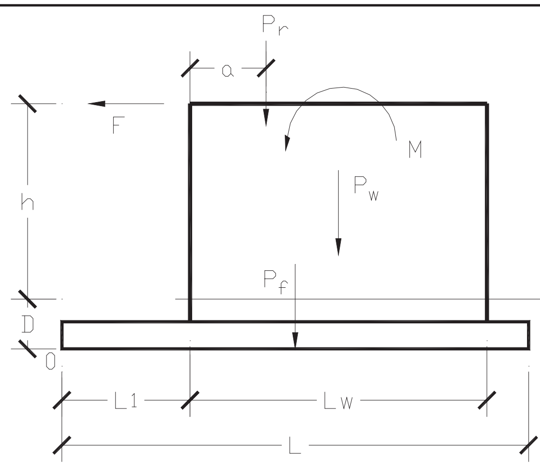
Direction	$V_{u,max}$		$\phi V_c = 2\ \phi b\ d\ (f_c')^{0.5}$		check $V_u < \phi V_c$
Longitudinal	0.6	k	33	k	**[Satisfactory]**
Transverse	0.3	k / ft	32	k / ft	**[Satisfactory]**

Project SALAD AND GO #2001 – LEE'S SUMMIT, MO			Job Ref. 24-011
Description FOUNDATION DESIGN (SEC. F)	Calc by. BG	Date 04/25/2024	Sheet No. F3

- WALL FOOTING DESIGN AT NORTH AND SOUTH SHEAR WALLS

  FORCE IN SHEAR WALL = 4.06 K AT 14.0 FT ABOVE GROUND

USE 2'-0" WIDE BY 2'-8" DEEP CONC FOOTING REINF WITH (3) #6 CONT TOP BOTTOM, REFER TO 1-S301



INPUT DATA

WALL LENGTH	L_w =	21.33	ft		
WALL HEIGHT	h =	14	ft		
WALL THICKNESS	t =	6	in		
FOOTING LENGTH	L =	47	ft		
	L_1 =	1	ft		
FOOTING WIDTH	B =	2	ft		
FOOTING THICKNESS	T =	32	in		
FOOTING EMBEDMENT DEPTH	D =	3.33	ft		
ALLOWABLE SOIL PRESSURE	q_a =	2.1	ksf		
DEAD LOAD AT TOP WALL	$P_{r,DL}$ =	0	kips		
LIVE LOAD AT TOP WALL	$P_{r,LL}$ =	0	kips		
TOP LOAD LOCATION	a =	0	ft		
WALL SELF WEIGHT	P_w =	11.5182	kips		
LATERAL LOAD TYPE (0=w ind,1=seismic)		0	w ind		
WIND LOADS AT WALL TOP	F =	4.06	kips		
	M =	0	ft-kips		
CONCRETE STRENGTH	f_c' =	4	ksi		
REBAR YIELD STRESS	f_y =	60	ksi		
TOP BARS, LONGITUDINAL		3	#	6	
BOTTOM BARS, LONGITUDINAL		3	#	6	
BOTTOM BARS, TRANSVERSE	#	3	@	24	in o.c.

THE FOOTING DESIGN IS ADEQUATE.

< == Not Required

ANALYSIS

CHECK OVERTURNING FACTOR (IBC 06 1605.2.1, 1801.2.1, & ASCE 7-05 12.13.4)

$F = M_R / M_O$ = 14.05 **>** 1.6 / 0.9 for w ind [Satisfactory]

Where P_f = 36.3467 kips (footing self w eight)

$M_O = F\,(h + D) + M$ = 70 ft-kips (overturning moment)

$M_R = (P_{r,DL})\,(L_1 + a) + P_f\,(0.5\,L) + P_w\,(L_1 + 0.5L_w)$ = 989 ft-kips (resisting moment w ithout live load)

CHECK SOIL CAPACITY (ALLOWABLE STRESS DESIGN)

P_s = 25.0667 kips (soil w eight in footing size)

$P = (P_{r,DL} + P_{r,LL}) + P_w + (P_f - P_s)$ = 22.80 kips (total vertical net load)

$M_R = (P_{r,DL} + P_{r,LL})\,(L_1 + a) + P_f\,(0.5\,L) + P_w\,(L_1 + 0.5L_w)$ = 989 ft-kips (resisting moment w ith live load)

$e = 0.5\,L - (M_R - M_O) / P$ = -16.77 ft (eccentricity from middle of footing)

$$q_{MAX} = \begin{cases} \dfrac{P\left(1+\dfrac{6e}{L}\right)}{BL}, & for \quad e \le \dfrac{L}{6} \\ \dfrac{2P}{3B(0.5L-e)}, & for \quad e > \dfrac{L}{6} \end{cases}$$ = -0.28 ksf **<** 4 / 3 q_a

[Satisfactory]

Where e = -16.77 ft, < (L / 6)

CHECK FOOTING CAPACITY (STRENGTH DESIGN)

$M_{u,R}$ = 1.2 [$P_{r,DL}$ (L_1 + a) + P_f (0.5 L) + P_w (L_1 + 0.5L_w)] + 0.5 $P_{r,\,LL}$(L_1 + a) = 1186 ft-kips

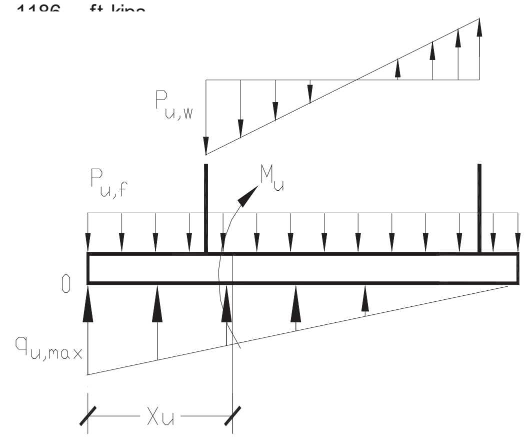
$M_{u,o}$ = 1.6 [F(h + D) + M] = 113 ft-kips

P_u = 1.2 ($P_{r,DL}$ + P_f + P_w) + 0.5 $P_{r,\,LL}$ = 57 kips

e_u = 0.5L - ($M_{u,R}$ - $M_{u,O}$) / P_u = 4.81 ft

$$q_{u,MAX}=\begin{cases}\dfrac{P_u\left(1+\dfrac{6e_u}{L}\right)}{BL}, & for\quad e_u\le\dfrac{L}{6}\\[2ex] \dfrac{2P_u}{3B(0.5L-e_u)}, & for\quad e_u>\dfrac{L}{6}\end{cases} =$$

0.99 ksf



Location	$M_{u,max}$		d (in)	ρ_{reqD}	$\rho_{prov\,D}$	$V_{u,max}$		$\phi V_c = 2\,\phi\,b\,d\,(f_c')^{0.5}$	
Top Longitudinal	-58	ft-k	28.63	0.0007	0.0019	5	kips	74	kips
Bottom Longitudinal	0	ft-k	28.63	0.0018	0.0019	5	kips	74	kips
Bottom Transverse	0	ft-k / ft	28.06	0.0000	0.0000	0	kips / ft	36	kips / ft

Where
$$\rho=\frac{0.85f'_c\left(1-\sqrt{1-\dfrac{M_u}{0.383bd^2f'_c}}\right)}{f_y}$$

ρ_{min} = 0.0018

$$\rho_{MAX}=\frac{0.85\beta_1 f'_c}{f_y}\frac{\varepsilon u}{\varepsilon u+\varepsilon t}=$$

0.0206 [Satisfactory]

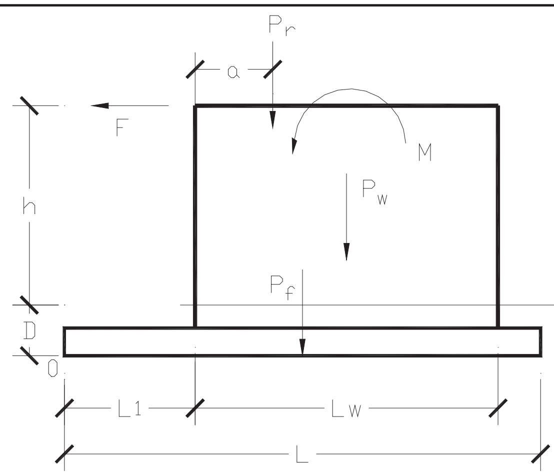
- WALL FOOTING DESIGN AT CENTER SHEAR WALL

  FORCE IN SHEAR WALL = 8.29 K AT 13.8 FT ABOVE GROUND

  USE 3’-0” WIDE BY 2’-8” DEEP CONC FOOTING REINF WITH (4) #6 CONT TOP AND (5) #6 CONT BOTTOM, REFER TO 6-S301

INPUT DATA

WALL LENGTH	L_w =	16.92	ft		
WALL HEIGHT	h =	13.8	ft		
WALL THICKNESS	t =	6	in		
FOOTING LENGTH	L =	18	ft		
	L_1 =	0.5	ft		
FOOTING WIDTH	B =	3	ft		
FOOTING THICKNESS	T =	32	in		
FOOTING EMBEDMENT DEPTH	D =	3.33	ft		
ALLOWABLE SOIL PRESSURE	q_a =	2.1	ksf		
DEAD LOAD AT TOP WALL	$P_{r,DL}$ =	0	kips		
LIVE LOAD AT TOP WALL	$P_{r,LL}$ =	0	kips		
TOP LOAD LOCATION	a =	0	ft		
WALL SELF WEIGHT	P_w =	6.0912	kips		
LATERAL LOAD TYPE (0=wind,1=seismic)		0	wind		
WIND LOADS AT WALL TOP	F =	4.06	kips		
	M =	0	ft-kips		
CONCRETE STRENGTH	f_c' =	4	ksi		
REBAR YIELD STRESS	f_y =	60	ksi		
TOP BARS, LONGITUDINAL		4	#	6	
BOTTOM BARS, LONGITUDINAL		5	#	6	
BOTTOM BARS, TRANSVERSE	#	3	@	24	in o.c.



THE FOOTING DESIGN IS ADEQUATE.

< == Not Required

ANALYSIS

CHECK OVERTURNING FACTOR (IBC 06 1605.2.1, 1801.2.1, & ASCE 7-05 12.13.4)

$F = M_R / M_O =$ 3.49 **>** 1.6 / 0.9 for wind [Satisfactory]

Where P_f = 20.88 kips (footing self weight)

$M_O = F\,(h + D) + M =$ 70 ft-kips (overturning moment)

$M_R = (P_{r,DL})\,(L_1 + a) + P_f\,(0.5\,L) + P_w\,(L_1 + 0.5L_w) =$ 242 ft-kips (resisting moment without live load)

CHECK SOIL CAPACITY (ALLOWABLE STRESS DESIGN)

P_s = 14.4 kips (soil weight in footing size)

$P = (P_{r,DL} + P_{r,LL}) + P_w + (P_f - P_s) =$ 12.57 kips (total vertical net load)

$M_R = (P_{r,DL} + P_{r,LL})\,(L_1 + a) + P_f\,(0.5\,L) + P_w\,(L_1 + 0.5L_w) =$ 242 ft-kips (resisting moment with live load)

$e = 0.5\,L - (M_R - M_O) / P =$ -4.76 ft (eccentricity from middle of footing)

$$q_{MAX} = \begin{cases} \dfrac{P\left(1+\dfrac{6e}{L}\right)}{BL}, & for\ \ e \le \dfrac{L}{6} \\[2ex] \dfrac{2P}{3B(0.5L-e)}, & for\ \ e > \dfrac{L}{6} \end{cases} = -0.14\ \text{ksf} < 4/3\,q_a$$

[Satisfactory]

Where e = -4.76 ft, < (L / 6)

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Suite 200 Fax: (972) 354-8856
Plano, TX 75093 www.ellisongage.com

CHECK FOOTING CAPACITY (STRENGTH DESIGN)

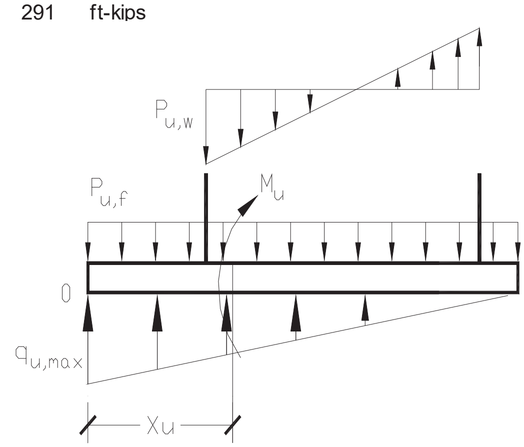
$M_{u,R}$ = 1.2 [$P_{r,DL}$ (L_1 + a) + P_f (0.5 L) + P_w (L_1 + 0.5L_w)] + 0.5 $P_{r,\,LL}$(L_1 + a) = 291 ft-kips

$M_{u,o}$ = 1.6 [F(h + D) + M] = 111 ft-kips

P_u = 1.2 ($P_{r,DL}$ + P_f + P_w) + 0.5 $P_{r,\,LL}$ = 32 kips

e_u = 0.5L - ($M_{u,R}$ - $M_{u,O}$) / P_u = 3.45 ft

$$q_{u,MAX} = \begin{cases} \dfrac{P_u\left(1+\dfrac{6e_u}{L}\right)}{BL}, & for \quad e_u \le \dfrac{L}{6} \\ \dfrac{2P_u}{3B(0.5L-e_u)}, & for \quad e_u > \dfrac{L}{6} \end{cases} =$$ 1.30 ksf



Location	$M_{u,max}$		d (in)	ρ_{reqD}	$\rho_{prov\,D}$	$V_{u,max}$		$\phi V_c = 2\,\phi\,b\,d\,(f_c')^{0.5}$	
Top Longitudinal	-1	ft-k	28.63	0.0000	0.0017	1	kips	111	kips
Bottom Longitudinal	2	ft-k	28.63	0.0018	0.0021	1	kips	111	kips
Bottom Transverse	0	ft-k / ft	28.06	0.0000	0.0000	1	kips / ft	36	kips / ft

Where $$\rho = \frac{0.85 f_c'\left(1-\sqrt{1-\dfrac{M_u}{0.383bd^2 f_c'}}\right)}{f_y}$$ ρ_{min} = 0.0018

$$\rho_{MAX} = \frac{0.85\beta_1 f_c'}{f_y}\frac{\varepsilon_u}{\varepsilon_u+\varepsilon_t} =$$ 0.0206 [Satisfactory]

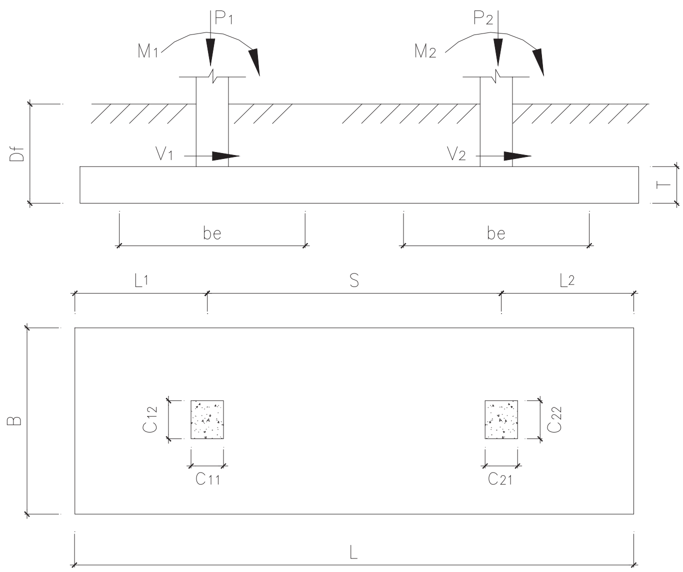
- CONTINUOUS FOOTING DESIGN AT MOMENT FRAME

FORCE IN SHEAR WALL = 7.90 K AT 13.8 FT ABOVE GROUND

USE 3'-6" WIDE BY 2'-8" DEEP CONC FOOTING REINF WITH (6) #6 CONT TOP BOTTOM, REFER TO 2-S501

INPUT DATA

			COL#1	COL#2	
COLUMN WIDTH	c_1	=	19	19	in
COLUMN DEPTH	c_2	=	14	14	in
AXIAL DEAD LOAD	P_{DL}	=	1	0	k
AXIAL LIVE LOAD	P_{LL}	=	1	0	k
LATERAL LOAD (0=WIND, 1=SEISMIC)		=	0	Wind ASD	
WIND AXIAL LOAD, ASD	P_{LAT}	=	-3.9	9.1	k
WIND SHEAR LOAD, ASD	V_{LAT}	=	-6.7	-6.7	k
WIND MOMENT, ASD	M_{LAT}	=	35.6	35.6	k-ft
CONCRETE STRENGTH	f_c'	=	4	ksi	
REBAR YIELD STRESS	f_y	=	60	ksi	
ALLOWABLE SOIL PRESSURE	Q_a	=	2.1	ksf	
DISTANCE TO LEFT EDGE	L_1	=	4.08	ft	
DISTANCE BETWEEN COLUMNS	S	=	17.3	ft	
DISTANCE TO RIGHT EDGE	L_2	=	4.08	ft	
FOOTING WIDTH	B	=	3.5	ft	
FTG EMBEDMENT DEPTH	D_f	=	3.33	ft	
FOOTING THICKNESS	T	=	32	in	
SURCHARGE	q_s	=	0	ksf	
SOIL WEIGHT	w_s	=	0	kcf	
LONGITUDINAL REINFORCING BAR SIZE		#	6		
TRANSVERSE REINFORCING BAR SIZE		#	6		



BAND WIDTH	b_e	=	3.5	ft
LONG. REINF AT TOP	2 # 6 @ 35 in o.c., cont.			
LONG. REINF AT BOTTOM	3 # 6 @ 18 in o.c., cont.			
TRANS. REINF. AT BAND WIDTH	4 # 6 @ 12 in o.c., bottom			

DESIGN SUMMARY

FOOTING LENGTH	L	=	25.46	ft
FOOTING WIDTH	B	=	3.50	ft
FOOTING THICKNESS	T	=	32	in

THE FOOTING DESIGN IS ADEQUATE.

CHECK OVERTURNING FACTOR (IBC 06 1605.2.1, 1801.2.1, & ASCE 7-05 12.13.4)

M_R / M_O = 5.81 **>** F = 1.6 / 0.9 = 1.78 **[Satisfactory]**

Where M_O = $M_{LAT\,1} + M_{LAT\,2} + (V_{LAT\,1} + V_{LAT\,2})\,T - P_{LAT\,1}(L - L_1) - P_{LAT\,2}L_2$ = 82 k-ft

P_{ftg} = (0.15 kcf) T B L = 35.64 k, footing weight

P_{soil} = $w_s\,(D_f - T)\,B\,L$ = 0.00 k, soil weight

M_R = $P_{DL\,1}(L - L_1) + P_{DL\,2}L_2 + 0.5\,(P_{ftg} + P_{soil})\,L$ = 475 k-ft

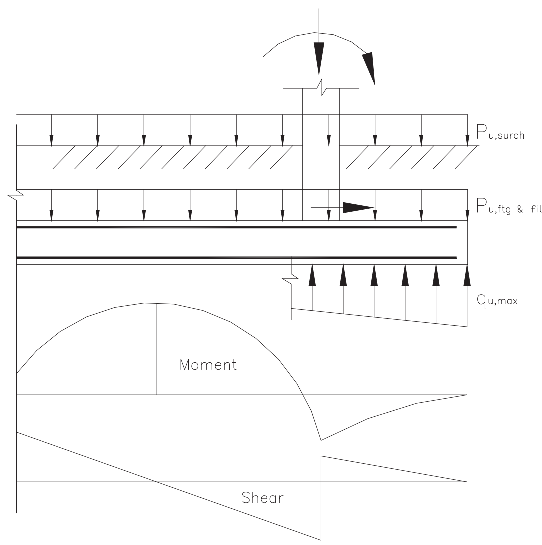
CHECK SOIL BEARING CAPACITY (ACI 318 SEC.15.2.2)

Service Loads	CASE 1		CASE 2		CASE 3		
P	2.0		8.8		5.4		k
e	-8.7		18.7		13.6		ft, (at base, including V T / P)
q_s B L	0.0		0.0		0.0		k, (surcharge load)
$(0.15-w_s)$T B L	35.6		35.6		35.6		k, (footing increased)
Σ P	37.6		44.4		41.0		k
e	-0.5	< L/6	3.7	< L/6	1.8	< L/6	ft
q_{max}	0.4		0.9		0.7		ksf
q_{allow}	2.1		2.8		2.8		ksf

Where

$$q_{MAX} = \begin{cases} \dfrac{(\Sigma P)\left(1+\dfrac{6e}{L}\right)}{BL}, & for \quad e \le \dfrac{L}{6} \\ \dfrac{2(\Sigma P)}{3B(0.5L-e)}, & for \quad e > \dfrac{L}{6} \end{cases}$$

[Satisfactory]



DESIGN FLEXURE & CHECK FLEXURE SHEAR

(ACI 318 SEC.15.4.2, 10.2, 10.5.4, 7.12.2, 12.2, 12.5, 15.5.2, 11.1.3.1, & 11.3)

$$q_{u,MAX} = \begin{cases} \dfrac{(\Sigma P_u)\left(1+\dfrac{6e_u}{L}\right)}{BL}, & for \quad e_u \le \dfrac{L}{6} \\ \dfrac{2(\Sigma P_u)}{3B(0.5L-e_u)}, & for \quad e_u > \dfrac{L}{6} \end{cases}$$

$$\rho_{MAX} = \frac{0.85\beta_1 f'_c}{f_y}\frac{\varepsilon_u}{\varepsilon_u+\varepsilon_t}$$

$$\rho_{MIN} = MIN\left(0.0018\frac{T}{d},\ \frac{4}{3}\rho\right)$$

$$\rho = \frac{0.85 f'_c\left(1-\sqrt{1-\dfrac{M_u}{0.383bd^2 f'_c}}\right)}{f_y}$$

FACTORED SOIL PRESSURE

Factored Loads	CASE 1		CASE 2		CASE 3		
P_u	2.8		10.5		9.2		k
e_u	-8.7		19.3		23.3		ft, (at base, including V_u T / P_u)
γ q_s B L	0.0		0.0		0.0		k, (factored surcharge load)
γ [0.15 T + w_s (D_f - T)] B L	42.8		42.8		32.1		k, (factored footing & backfill loads)
Σ P_u	45.6		53.3		41.3		k
e_u	-0.5	< L/6	3.8	< L/6	5.2	> L/6	ft
$q_{u, max}$	0.447		1.136		1.044		ksf

DESIGN FLEXURE

Location	$M_{u,max}$		d (in)	ρ_{min}	ρ_{reqD}	ρ_{max}	s_{max}(in)	use	ρ_{provD}
Top Longitudinal	-82	ft-k	29.63	0.0007	0.0005	0.0206	no limit	2 # 6 @ 35 in o.c., cont.	0.0007
Bottom Longitudinal	11	ft-k	28.63	0.0001	0.0001	0.0206	18	3 # 6 @ 18 in o.c., cont.	0.0011
Bottom Transverse, b_e	0	ft-k / ft	27.88	0.0000	4.3E-06	0.0206	18	4 # 6 @ 12 in o.c.	0.0015

[Satisfactory]

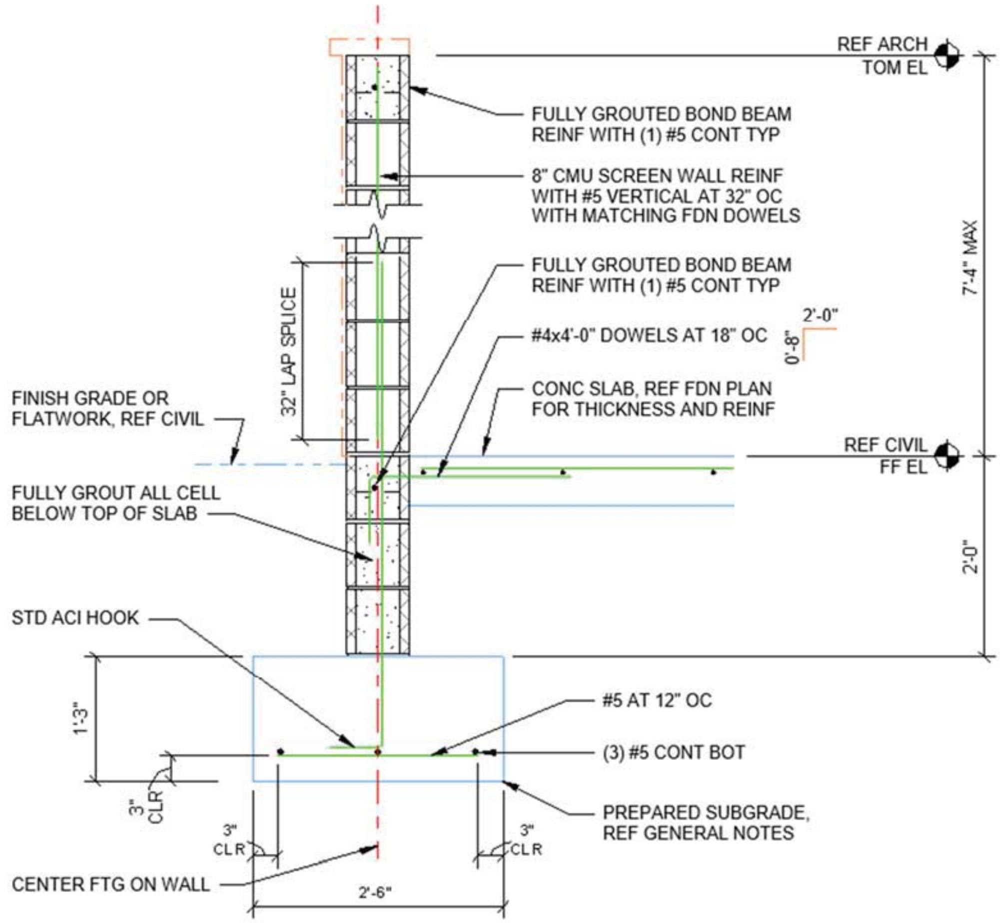
(cont'd)

CHECK FLEXURE SHEAR

Direction	$V_{u,max}$		$\phi V_c = 2\,\phi b\,d\,(f_c')^{0.5}$		check $V_u < \phi V_c$
Longitudinal	8	k	114	k	**[Satisfactory]**
Transverse	0	k / ft	32	k / ft	**[Satisfactory]**

DUMPSTER ENCLOSURE DESIGN

8-S004:



8 CMU DUMPSTER WALL
3/4" = 1'-0"

- DETERMINE WIND LOAD ON DUMPSTER WALL

NOTE: USE Cf = 1.4

DESIGN WIND LOADS: SOLID FREESTANDING WALLS AND SOLID SIGNS
(MWFRS - Directional Procedure, ASCE 7-16)

$F = q_h G C_f A_s$ (lb)			[EQ. 29.3-1]
Risk Category =	II		[TABLE 1.5-1]
Basic Wind Speed, V =	110	mph	[FIG. 26.5-1, ATC WEBSITE] https://hazards.atcouncil.org/
Service Wind Speed, V_{asd} =	85	mph	
Exposure =	C		[26.7]
Height to Top of Sign, h =	8.00	ft	[FIG. 29.3-1]
α =	9.5		[TABLE 26.11-1]
z_g =	900	ft	[TABLE 26.11-1]
K_h =	0.85		[TABLE 26.10-1]
K_{zt} =	1.00		[26.8.2]
K_d =	0.85		[TABLE 26.6-1]
Elevation =	900	ft	[26.9]
K_e =	0.97		[TABLE 26.9-1]
q_h =	21.6	psf	[EQ. 26.10-1]
G =	0.85		[26.11.1]
Height to Top of Sign, h =	8.00	ft	[FIG. 29.3-1]
Sign Height, s =	8.00	ft	[FIG. 29.3-1]
Sign Length, B =	13.33	ft	[FIG. 29.3-1]
Clearance Ratio, s/h =	1.00		
Return Corner Length, L_r =	13.33	ft	[FIG. 29.3-1]
Aspect Ratio, L_r/s =	1.67		
Aspect Ratio, B/s =	1.67		DISREGARD CASE C
C_f, Case A & Case B =	1.4		[FIG. 29.3-1]
C_f, Case C:			[FIG. 29.3-1]
0 to s =	2.25		
s to 2s =	1.50		
2s to 3s =	1.15		
3s to 10s =	0.00		
C_f, Case C (design) =		1.00	
Gross Area of Sign, A_s =	107	ft^2	
Maximum Force, F =	**2745**	lb	[EQ. 29.3-1]
Maximum Pressure, P =	**25.7**	psf	(ULTIMATE LEVEL)
Maximum Pressure, P =	**15.4**	psf	(SERVICE LEVEL)

- DETERMINE SEISMIC LOAD ON DUMPSTER WALL

SEISMIC LOADS ON FREESTANDING WALLS:

[SECTION 15.6.8, Ground Supported Cantilevered Walls and Fences]
Per 15.4.1, Design Walls Using Section 12.8

S_1 =	0.068			
S_{DS} =	0.106			
S_{D1} =	0.109			
R =	1.25	[TABLE 15.4-2]		
C_d =	2.50	[TABLE 15.4-2]		
I_e =	1.0	[TABLE 1.5-2]		
T_L =	6.00	sec, [FIG. 22-14]		
T =	1.030	sec, [Sec. 12.8.2]		
C_s BOUND EQUATIONS:				
C_s =	0.084	[EQ. 12.8-2]		
C_s =	0.084	[EQ. 12.8-3, 12.8-4]		
C_s =	0.030	[EQ. 15.4-1]		
C_s =	NA	[EQ. 12.8-6]		
C_s =	0.084	(GOVERNING VALUE)		
$V = C_sW$ =	0.084	W	[EQ. 12.8-1]	ULTIMATE LOAD (FACTORED)
$V = C_sW$ =	0.059	W	[EQ. 12.8-1]	SERVICE LEVEL LOAD (ULTIMATE LOAD x 0.7)
W =	50	PSF, 8" CMU WALLS		
	70	PSF, 12" CMU WALLS		
8" CMU WALL V =	**4.2**	PSF (ULTIMATE)		
12" CMU WALL V =	**5.9**	PSF (ULTIMATE)		

- DESIGN 8" CMU DUMPSTER WALL

GIVEN:
WIND PRESSURE = 25.7 PSF (LRFD), 15.4 PSF (ASD) ←**CONTROLS**
SEISMIC PRESSURE (8" CMU) = 5.9 PSF (LRFD), 4.2 PSF (ASD)
WALL HEIGHT = 7.5 FT
f'm = 1,500 PSI
MASONRY STRESS = 675 PSI
STEEL STRESS = 32 KSI

USE 8" CMU REINFORCED WITH #5 VERTICAL AT 32" ON CENTER

Dumpster Enclosure Screenwall

Masonry Wall Properties	
Weight of wall (psf) =	51
Wall type (s or c) =	**c**
Reinforcement (s or d)=	**s**
CMU thickness (in) =	**8**
Wall ht. (ft) =	**7.50**
Wall ht for critical axial load (ft) =	**7.50**
Design width, b (in) =	**12**
Grout Spacing (in) =	**32**
Bar size =	**5**
r (in) =	2.59
E_s (ksi) =	**29000**
f'_m (psi) =	**1500**
E_m (ksi) =	1350
n =	21.48
Equiv. Solid Thickness (in) =	4.9
d (in) =	3.81
A_e (in^2) =	58.8
A_{bar} (in^2) =	0.31
A_s (in^2) =	0.11625
ρ =	0.002541
ρn =	0.054584
k =	0.2803
j =	0.906567
I_g (in^4) =	117.65
I_{cr} (in^4) =	23.68
S_{cr} (in^3) =	22.16
S (in^3) =	48.02
f_r (psi) =	96.82
h/r=	69.50
F_a (psi)=	282.590
Critical axial load P_a (lbs)=	382.5

Load Info.	
Addtl. axial load (k)=	**0.00**
Eccentricity of load (in.) =	**0.0**
$M_{max,\ axial}$ (k-ft)=	0.000
Pilaster trib width (ft) =	**0.00**
Wind pressure (psf) =	**15.4**
Uniform load, w (lb/ft) =	15.4
$M_{max,\ lateral}$ (k-ft) =	0.433
$M_{max,\ analysis}$ (k-ft) =	**0.000**
M_{tot} (k-ft) =	0.433

Steel Stress Check		
f_s (ksi) =	12.94	
F_s (ksi) =	32.00	OK
Stress % =	40.42	OK

Masonry Stress Check		
f_b (psi) =	234.53	
F_b (psi) =	675.00	OK

Deflection Check		
Parapet Δ_{max} (in) =	0.23	
Wall Δ_{max} (in) =	N/A	
L/120 (in) =	0.75	OK
L/240 (in) =	N/A	N/A

Masonry Axial Check		
f_a (psi)=	6.51	
F_a (psi)=	282.59	O.K.

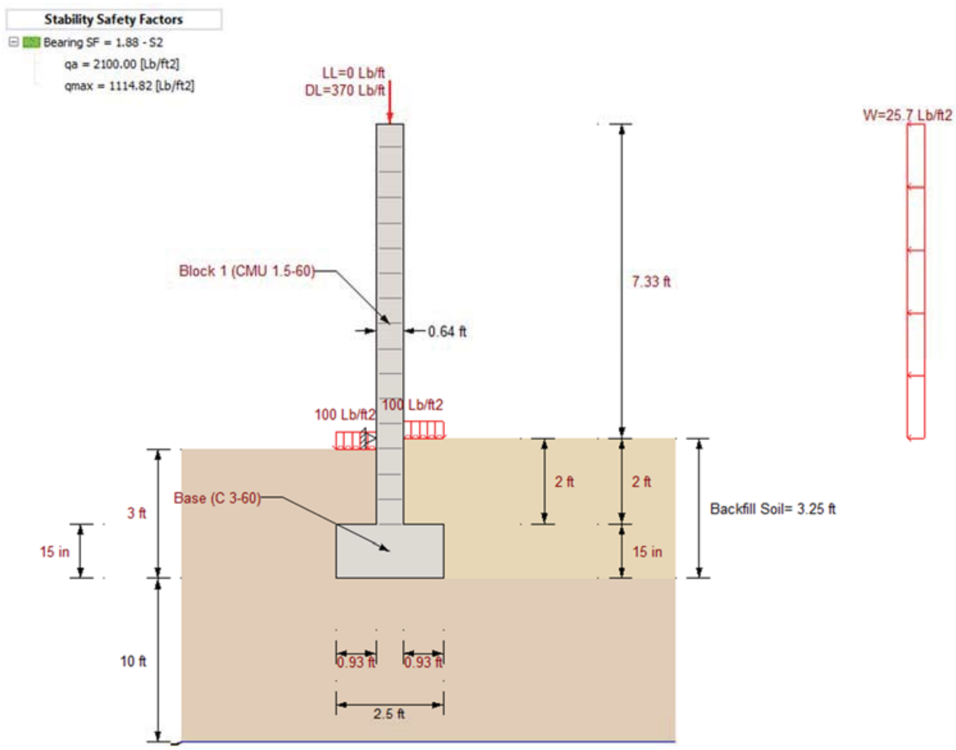
Interaction Equation		
$f_a/F_a + f_b/F_b$=	0.37	O.K.

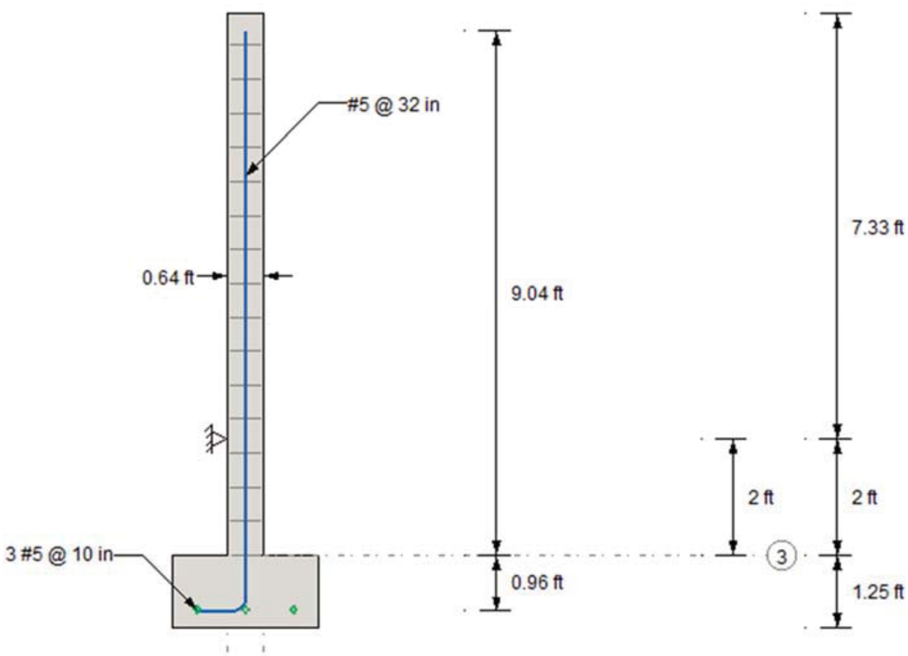
Ellison Gage & Associates, PLLC Consulting Structural Engineers 5068 W. Plano Parkway Suite 200 Plano, TX 75093 Ofc: (972) 354-8855 Fax: (972) 354-8856 www.ellisongage.com	Project SALAD AND GO #2001 – LEE'S SUMMIT, MO			Job Ref. 24-011
	Description CMU DUMPSTER DESIGN (SEC. D)	Calc. by BG	Date 04/29/2024	Sheet No. D5

- DESIGN CMU SCREEN WALL FOUNDATION

GIVEN:
WIND PRESSURE = 25.7 PSF (LRFD)

USE 2'-6" x 1'-3" THICK CONT FOOTING REINFORCED WITH (3) #4 CONT





RAM Elements **Windows User**

Current Date: 4/29/2024 10:17 AM
Units system: English
File name: Z:\24-001_24-999\24-011 SAG 2001 Lees Summit-MO\02-Design Info\Calcs\Masonry Screenwall\2001-Dumpster Enclosure Wall Footing.rtw

Design Results
Retaining wall

General Information

Design code	:	ACI 318-2014
Masonry design code	:	TMS 402-2016 ASD

Materials

Description	:	C 3-60
Concrete, f'c	:	3.00 [Kip/in2]
Steel, fy	:	60.00 [Kip/in2]
Elasticity modulus	:	3122.02 [Kip/in2]
Unit weight	:	0.15 [Kip/ft3]

Description	:	CMU 1.5-60
Masonry, F'm	:	1.50 [Kip/in2]
Steel, Fs	:	32.00 [Kip/in2]
Elasticity modulus	:	1350.00 [Kip/in2]
Unit weight	:	0.14 [Kip/ft3]
Grouting type	:	Partial grouting

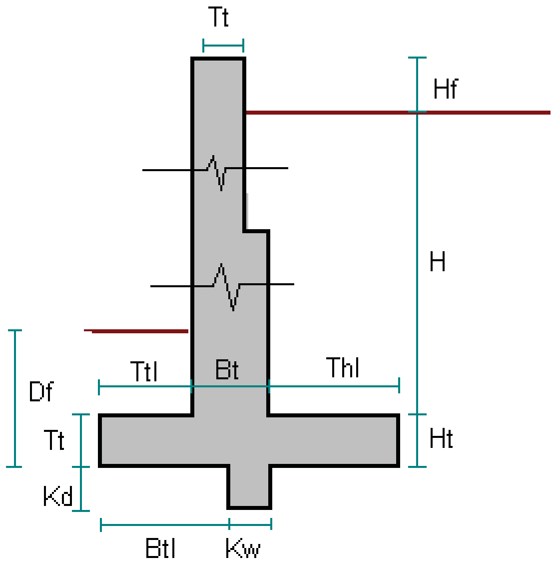
Soil

Modulus of subgrade reaction	:	150.00 [Kip/ft3]
Backfill slope	:	0.00 [°]

Description	U.W. [Kip/ft3]	Saturated U.W. [Kip/ft3]	phi [°]	c [Lb/ft2]	Friction wall/soil	Ko
Base Soil	0.11	0.14	27.00	0.00	22.78	--
Backfill Soil	0.11	--	27.00	0.00	0.00	0.00

Geometry

Wall type : Restrained



Retained height H	: 2.00 [ft]		Wall height above retained soil Hf	:	7.33 [ft]
Base depth Df	: 3.00 [ft]		Use key	:	No
Top toe length Ttl	: 0.93 [ft]		Toe thickness Tt	:	1.25 [ft]
Bottom toe length Btl	: 2.00 [ft]				
Top heel length Thl	: 0.93 [ft]		Heel thickness Ht	:	1.25 [ft]
Base material	: C 3-60				

Stem thickness at base Bt : 7.63 [in]
Stem blocks number : 1

Block	Thickness [in]	Height [ft]	Material
1	7.63	9.33	CMU 1.5-60

Loads

Backfill surcharge	:	0.10 [Kip/ft2]
Surcharge over toe	:	0.10 [Kip/ft2]
Wind pressure	:	0.03 [Kip/ft2]
Stem axial load (DL)	:	0.37 [Kip]

Load conditions included in the design:

Service Load Combinations:
S1 = DL+H
S2 = DL+LL+H
S3 = DL+LL+0.6H
S4 = DL+0.75LL+H
S5 = DL+0.75LL+0.45W+H
S6 = 0.6DL+0.6W+H
S7 = 0.6DL+0.6W+0.6H

Strength Design Load Combinations:
R1 = 1.4DL
R2 = 1.2DL+1.6LL
R3 = 1.2DL+0.5W
R4 = 1.2DL+W
R5 = 1.2DL+W+LL
R6 = 0.9DL+W
R7 = 0.9DL+W+1.6H

R8 = 0.9DL+W+0.9H

Masonry Design Load Combinations:
A1 = DL+LL+H
A2 = DL
A3 = DL+LL
A4 = DL+H+LL
A5 = DL+0.75LL
A6 = DL+W
A7 = DL+H+W
A8 = DL+0.75W+0.75LL
A9 = DL+H+0.75W+0.75LL
A10 = 0.6DL+W
A11 = 0.6DL+W+H

Reinforcement

Steel reinforcement bars:

Stem free cover	:	0.25 [ft]
Base free cover	:	0.25 [ft]
Maximum Rho/Rho balanced ratio	:	0.75
Round longitudinal bar lengths to	:	1.00 [in]

Longitudinal reinforcement

Element	Size	Spacing [in]	Pos	Axis	Dist1 [ft]	Dist2 [ft]	Hook1	Hook2
Stem	#5	32.00	Int.	3	-0.96	9.04	Yes	No

Development and splice lengths

Element	Diameter	Ld [in]	Ldh [in]	L. Splice [in]	L. total [ft]
Stem	#5	11.50	--	--	10.00

Horizontal reinforcement

Element	Diameter	Nr	@ [in]	Position
Base	#5	3	10.00	Int.

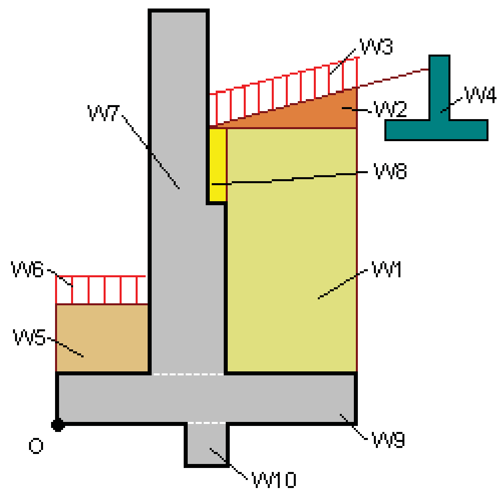
Assumptions

Active pressures calculation method	:	At rest
Calculation method for lateral soil pressures	:	Boussinesq
Calculation method for soil bearing pressures	:	Hansen
Frost depth	:	1.50 [ft]
Undermining depth	:	0.00 [ft]

Design

Status : OK

Calculation of resisting forces



Description	Force [Kip]	Distance [ft]	Moment [Kip*ft]
Weight of soil over heel (W1)	0.20	2.03	0.42
Surcharge over heel (W3)	0.09	2.03	0.19
Weight of soil over toe (W5)	0.18	0.47	0.08
Surcharge over toe (W6)	0.09	0.47	0.04
Stem weight (W7)	0.80	1.25	1.00
Base weight (W9)	0.47	1.25	0.58
Stem axial load (DL)	0.37	1.25	0.46
Total	2.21		2.78
Toe horizontal soil pressure against sliding (Pp)	0.66	0.67	0.44

Calculation of destabilizing forces

Description	Force [Kip]	Distance [ft]	Moment [Kip*ft]
Heel horizontal soil pressure (Pah)	0.49	1.28	0.63
Wind force (Pw)	0.19	6.92	1.30

Global stability

Minimum additional safety factor for soil pressures : 1.00

Load case	qmax [Lb/ft2]	qa [Lb/ft2]	Soil Pres. SF	RM [Kip*ft]	OTM [Kip*ft]	Overt. SF	Res F [Kip]	Slid F [Kip]	Slid. SF	Defl [in]
S1	1040.29	2100.00	2.02	-	-	N.A.	-	-	N.A.	-
S2	1114.82	2100.00	1.88	-	-	N.A.	-	-	N.A.	-
S3	1015.03	2100.00	2.07	-	-	N.A.	-	-	N.A.	-
S4	1096.19	2100.00	1.92	-	-	N.A.	-	-	N.A.	-
S5	977.81	2100.00	2.15	-	-	N.A.	-	-	N.A.	-
S6	732.45	2100.00	2.87	-	-	N.A.	-	-	N.A.	-
S7	832.24	2100.00	2.52	-	-	N.A.	-	-	N.A.	-

Bending and Shear per element

Element : Toe

Station Nr.	Dist	d [in]	Mu[Kip*ft] neg	pos	ϕ*Mn[Kip*ft] neg	pos	Asreq [in2] ext	int	Asprov [in2] ext	int	sb [in] ext	int	Mu/(ϕ*Mn)
1	0%	15.00	-0.27	0.48	-6.16	6.16	0.00	0.00	0.00	0.00	--	--	0.08
2	10%	15.00	-0.22	0.39	-6.16	6.16	0.00	0.00	0.00	0.00	--	--	0.06
3	20%	15.00	-0.17	0.31	-6.16	6.16	0.00	0.00	0.00	0.00	--	--	0.05
4	30%	15.00	-0.13	0.24	-6.16	6.16	0.00	0.00	0.00	0.00	--	--	0.04
5	40%	15.00	-0.10	0.17	-6.16	6.16	0.00	0.00	0.00	0.00	--	--	0.03
6	50%	15.00	-0.07	0.12	-6.16	6.16	0.00	0.00	0.00	0.00	--	--	0.02
7	60%	15.00	-0.04	0.08	-6.16	6.16	0.00	0.00	0.00	0.00	--	--	0.01
8	70%	15.00	-0.02	0.04	-6.16	6.16	0.00	0.00	0.00	0.00	--	--	0.01
9	80%	15.00	-0.01	0.02	-6.16	6.16	0.00	0.00	0.00	0.00	--	--	0.00
10	90%	15.00	0.00	0.00	-6.16	6.16	0.00	0.00	0.00	0.00	--	--	0.00
11	100%	15.00	0.00	0.00	-6.16	6.16	0.00	0.00	0.00	0.00	--	--	0.00
C	0%	15.00	-0.27	0.48	-6.16	6.16	0.00	0.00	0.00	0.00	--	--	0.08

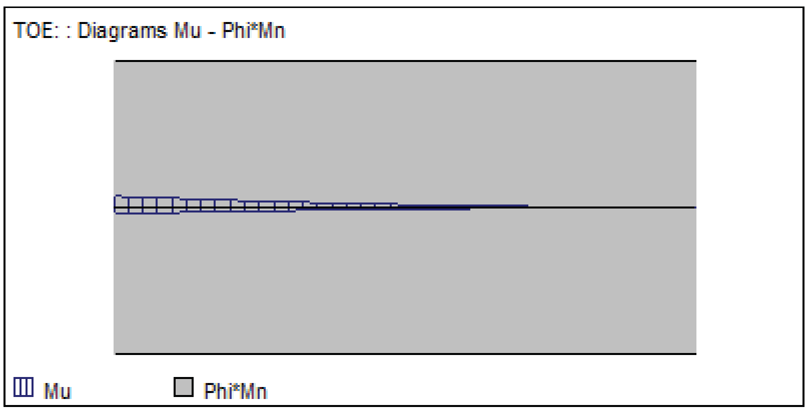
Maximum allowed spacing between bars : 18.00 [in]

Base transverse reinforcement:

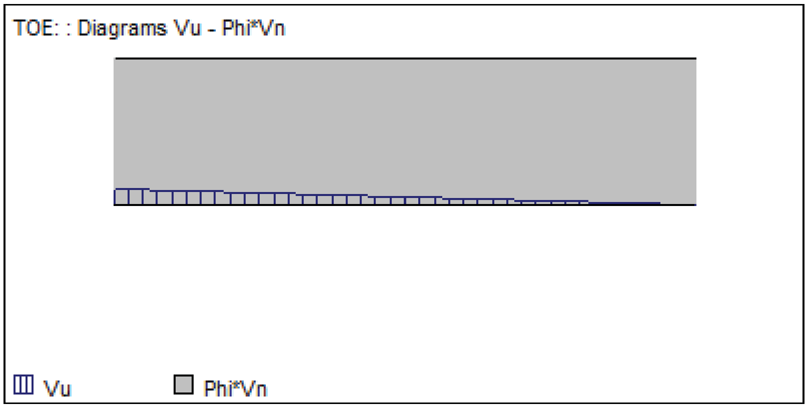
Top reinforcement : 0.00 [in2]

Bottom reinforcement : 0.37 [in2]

Minimum shrinkage and temperature reinforcement : 0.36 [in2]



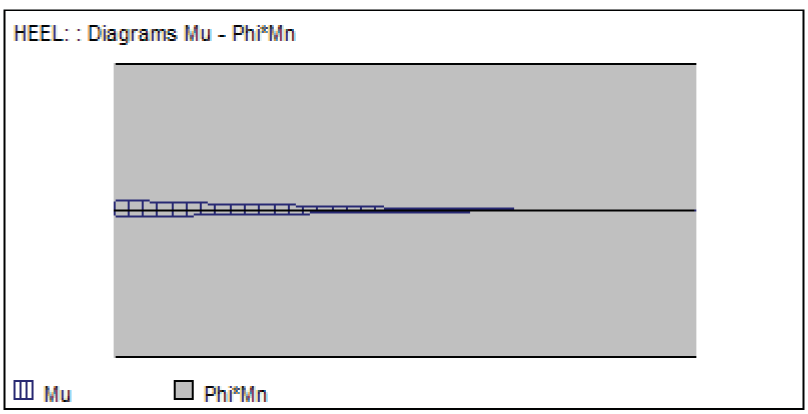
Station Nr.	Dist	Vu [Kip]	Vc [Kip]	ϕ*Vn [Kip]	Vu/(ϕ*Vn)
1	0%	1.03	15.77	9.46	0.11
2	10%	0.93	15.77	9.46	0.10
3	20%	0.83	15.77	9.46	0.09
4	30%	0.73	15.77	9.46	0.08
5	40%	0.62	15.77	9.46	0.07
6	50%	0.52	15.77	9.46	0.05
7	60%	0.42	15.77	9.46	0.04
8	70%	0.31	15.77	9.46	0.03
9	80%	0.21	15.77	9.46	0.02
10	90%	0.10	15.77	9.46	0.01
11	100%	0.00	15.77	9.46	0.00
C	0%	1.03	15.77	9.46	0.11



Element : Heel

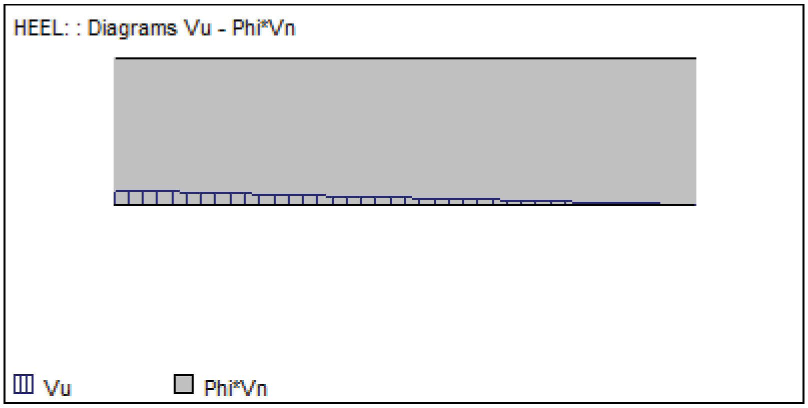
Station Nr.	Dist	d [in]	Mu[Kip*ft] neg	pos	ϕ*Mn[Kip*ft] neg	pos	Asreq [in2] ext	int	Asprov [in2] ext	int	sb [in] ext	int	Mu/(ϕ*Mn)
1	0%	15.00	-0.28	0.44	-6.16	6.16	0.00	0.00	0.00	0.00	--	--	0.07
2	10%	15.00	-0.23	0.35	-6.16	6.16	0.00	0.00	0.00	0.00	--	--	0.06
3	20%	15.00	-0.18	0.28	-6.16	6.16	0.00	0.00	0.00	0.00	--	--	0.05
4	30%	15.00	-0.14	0.21	-6.16	6.16	0.00	0.00	0.00	0.00	--	--	0.03
5	40%	15.00	-0.10	0.16	-6.16	6.16	0.00	0.00	0.00	0.00	--	--	0.03
6	50%	15.00	-0.07	0.11	-6.16	6.16	0.00	0.00	0.00	0.00	--	--	0.02
7	60%	15.00	-0.04	0.07	-6.16	6.16	0.00	0.00	0.00	0.00	--	--	0.01
8	70%	15.00	-0.03	0.04	-6.16	6.16	0.00	0.00	0.00	0.00	--	--	0.01
9	80%	15.00	-0.01	0.02	-6.16	6.16	0.00	0.00	0.00	0.00	--	--	0.00
10	90%	15.00	0.00	0.00	-6.16	6.16	0.00	0.00	0.00	0.00	--	--	0.00
11	100%	15.00	0.00	0.00	-6.16	6.16	0.00	0.00	0.00	0.00	--	--	0.00
C	0%	15.00	-0.28	0.44	-6.16	6.16	0.00	0.00	0.00	0.00	--	--	0.07

Maximum allowed spacing between bars : 18.00 [in]



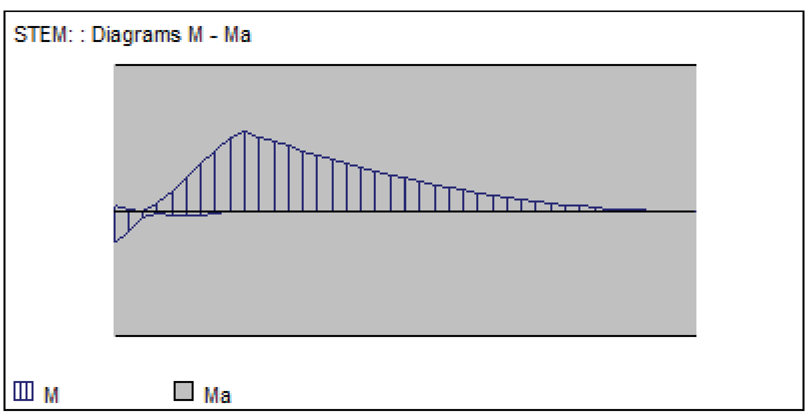
Station Nr.	Dist	Vu [Kip]	Vc [Kip]	ϕ*Vn [Kip]	Vu/(ϕ*Vn)
1	0%	0.94	15.77	9.46	0.10
2	10%	0.85	15.77	9.46	0.09
3	20%	0.75	15.77	9.46	0.08
4	30%	0.66	15.77	9.46	0.07
5	40%	0.56	15.77	9.46	0.06
6	50%	0.47	15.77	9.46	0.05

7	60%	0.37	15.77	9.46	0.04
8	70%	0.28	15.77	9.46	0.03
9	80%	0.19	15.77	9.46	0.02
10	90%	0.09	15.77	9.46	0.01
11	100%	0.00	15.77	9.46	0.00
C	0%	0.94	15.77	9.46	0.10

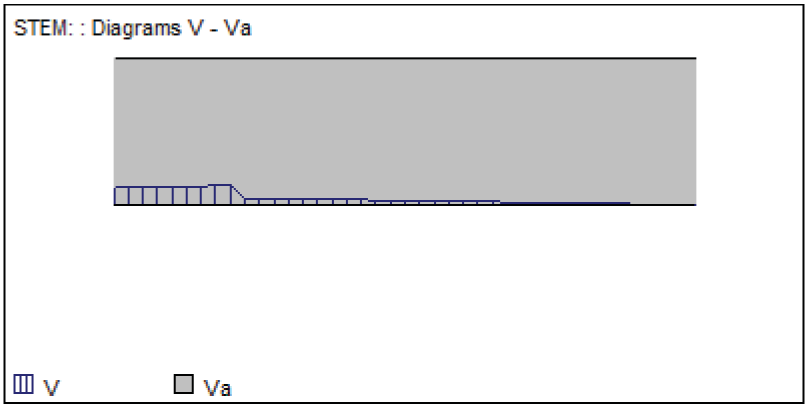


Element : Stem (Block 1, Masonry)

Station No.	Dist	d [in]	M [Kip*ft] neg	pos	Ma [Kip*ft]	Asreq [in2]	Asprov [in2]	P [Kip]	Pa [Kip]	Bending ratio
1	0%	4.13	-0.27	0.06	1.04	0.03	0.12	0.37	19.50	0.26
2	10%	4.13	-0.03	0.18	1.23	0.02	0.12	0.37	19.50	0.14
3	20%	4.13	-0.01	0.63	1.23	0.06	0.12	0.37	19.50	0.51
4	30%	4.13	0.00	0.55	1.23	0.05	0.12	0.37	19.50	0.45
5	40%	4.13	0.00	0.40	1.23	0.04	0.12	0.37	19.50	0.33
6	50%	4.13	0.00	0.28	1.23	0.03	0.12	0.37	19.50	0.23
7	60%	4.13	0.00	0.18	1.23	0.02	0.12	0.37	19.50	0.15
8	70%	4.13	0.00	0.10	1.23	0.01	0.12	0.37	19.50	0.08
9	80%	4.13	0.00	0.04	1.23	0.00	0.12	0.37	19.50	0.04
10	90%	4.13	0.00	0.01	1.23	0.00	0.12	0.37	19.50	0.02
11	100%	4.13	0.00	0.00	1.23	0.00	0.12	0.37	19.50	0.00



Station No.	Dist	**V** [Kip]	**Va** [Kip]	**V/Va**
1	0%	0.48	3.93	0.20
2	10%	0.48	3.93	0.16
3	20%	0.54	3.93	0.24
4	30%	0.17	3.93	0.08
5	40%	0.14	3.93	0.06
6	50%	0.12	3.93	0.05
7	60%	0.10	3.93	0.04
8	70%	0.07	3.93	0.03
9	80%	0.05	3.93	0.02
10	90%	0.02	3.93	0.01
11	100%	0.00	3.93	0.00



Notes

* The soil beneath the wall is considered elastic and homogeneous. A linear variation of pressures is adopted.
* The required reinforcement for bending takes into account the minimum reinforcement ratio given by Code.
* For bending and shear design, the critical section is adopted at the support faces and axial forces are not considered.
* Shear reinforcement is not considered.
* Values shown in red are not in compliance with a provision of the code
* Ld,Ldh = Development length of each bar. If the bar ends with a hook, it considers the Ldh length.
*qprom = Mean compression pressure on soil.
*qmax = Maximum compression pressure on soil.
* SF = Safety factor, RM = Resisting moment, OTM = Overturning moment.
* ResF = Resisting force, SlidF = Sliding force, Defl = Deflection.
* sb = Free distance between bars.
* If the section at which member flexural strength is being calculated is within the development length of a group of bars, the bars will contribute to the bending capacity an amount proportional to their actual length / their full development length.
* Asprov is the provided reinforcement, considering the reduction due to the development length as described previously.

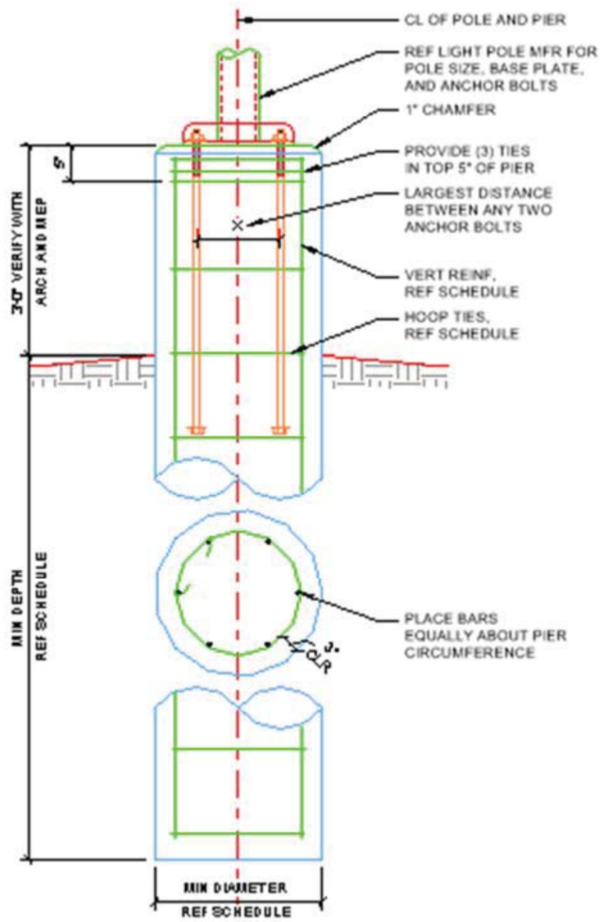
LIGHT POLE BASE DESIGN

5-S004:

LIGHT POLE BASE SCHEDULE

MAX POLE HT	MIN PIER DIAMETER	MIN DEPTH	VERTICAL REINF	HOOP TIES
15'-0"	24"	8'-0"	(6) #6	#3 AT 12" OC

NOTE(S):
1. REF MEP DRAWINGS FOR EMBEDDED CONDUIT, WIRES, ETC.
2. MINIMUM PIER DIAMETER SHALL BE THE GREATER OF 'X' PLUS 9" OR THE MINIMUM PIER DIAMETER IN THE SCHEDULE.
3. SPLICE VERTICAL PIER REINFORCING A MINIMUM OF 57 BAR DIAMETERS. LAP HOOP TIES A MINIMUM OF 48 BAR DIAMETERS.



5 TYPICAL LIGHT POLE BASE

3/4" = 1'-0"

- FIND WIND LOADS ON LIGHT POLE BASE

WIND LOAD CALCULATIONS ASCE 7-16 (Chapter 29)

DESIGN WIND LOADS - SOLID FREESTANDING WALLS AND SOLID SIGNS 29.3

V (mph) =	115	(Ultimate Wind Speed)
V_{asd} (mph) =	89	(Service Wind Speed)
Exposure =	C	
K_{zt} =	1.0	(26.8.2)
K_d =	0.85	(Table 26.6-1)
Elevation (ft) =	0	
K_e =	1.00	(Table 26.9-1)

Fixture Geometry

Height of Base (ft) =	3.00	
Height of Pole (ft) =	15.00	
Total Height, h (ft) =	18.00	
Diameter of Pole (ft) =	0.83	
Width of Fixture (ft) =	4.00	
Height of Fixture, s (ft) =	0.75	
Fixture Area (ft^2) =	3.00	
# Of Fixtures =	2	
Total Width of Fixtures, B (ft) =	8.83	
Total Area of Fixtures B x s (ft^2) =	6.62	

Velocity Pressure, q_z (psf)

$q_z = 0.00256 K_z K_{zt} K_d K_e V^2$ (psf) (Eq. 26.10-1)

α =	9.5	(Table 26.11-1)
z_g (ft) =	900	(Table 26.11-1)
K_h =	0.88	(Table 26.10-1)
q_h (psf) =	25.4	(Eq. 26.10-1)

Lateral Force

$F = q_h G C_f A_s$ (lb) (Eq. 29.3-1)

q_z (psf) =	25.38		
G =	0.85	(26.11)	
Clearance Ratio, s/h =	0.04	(Figure 29.3-1)	
Aspect Ratio, B/s =	11.77	(Figure 29.3-1)	
Case A & Case B, Cf =	1.86	(Figure 29.3-1)	
Case C, $C_{f\ 0\ to\ s}$ =	3.90	(Figure 29.3-1)	
Case C, $C_{f\ s\ to\ 2s}$ =	2.55	(Figure 29.3-1)	
Case C, $C_{f\ 2s\ to\ 3s}$ =	1.95	(Figure 29.3-1)	
Case C, $C_{f\ 3s\ to\ 10s}$ =	0.95	(Figure 29.3-1)	
Case C, $C_{f\ avg}$ =	2.80		2.80
$C_{f\ max}$ =	2.80		
$F_{fixture}$ (lbs) =	**400**	(Ultimate)	
$F_{fixture}$ (lbs) =	**240**	(Service, 0.6W)	

Reactions from Fixture(s)

Service Level Moment at Base of Pole (ft-lbs) =	3511
Service Level Moment at Ground (ft-lbs) =	4231
Applied Service Lateral Force (lbs) =	**240**
Distance from Ground to Force (ft) =	**17.6**

Light Pole/Base Geometry

Height of Base (ft) =	3.00
Height of Pole, s (ft) =	15.00
Total Height, h (ft) =	18.00
Diameter of Pole, B (ft) =	0.83
Diameter of Base (ft) =	2.00
Area of Light Pole B x s (ft^2) =	12.45
Area of Base (ft^2) =	6.00

Velocity Pressure, q_z (psf)

$q_z = 0.00256 K_z K_{zt} K_d K_e V^2$ (psf) (Eq. 26.10-1)

α =	9.5	(Table 26.11-1)
z_g (ft) =	900	(Table 26.11-1)
K_h =	0.88	(Table 26.10-1)
q_h (psf) =	25.4	(Eq. 26.10-1)

Lateral Force

$F = q_h G C_f A_s$ (lb) (Eq. 29.3-1)

q_z (psf) =	25.38	
G =	0.85	(26.11)
Clearance Ratio, s/h =	0.83	(Figure 29.3-1)
Aspect Ratio, B/s =	0.06	(Figure 29.3-1)
Case A & Case B, Cf =	1.90	(Figure 29.3-1)
Case C, $C_{f\ 0\ to\ s}$ =	1.00	(Figure 29.3-1)
Case C, $C_{f\ s\ to\ 2s}$ =	1.00	(Figure 29.3-1)
Case C, $C_{f\ 2s\ to\ 3s}$ =	1.00	(Figure 29.3-1)
Case C, $C_{f\ 3s\ to\ 10s}$ =	1.00	(Figure 29.3-1)
Case C, $C_{f\ avg}$ =	1.00	
$C_{f\ max}$ =	1.90	
F_{pole} (lbs) =	**510**	(Ultimate)
F_{base} (lbs) =	**246**	(Ultimate)
F_{pole} (lbs) =	**306**	(Service, 0.6W)
F_{base} (lbs) =	**148**	(Service, 0.6W)

Reactions from Pole and Base

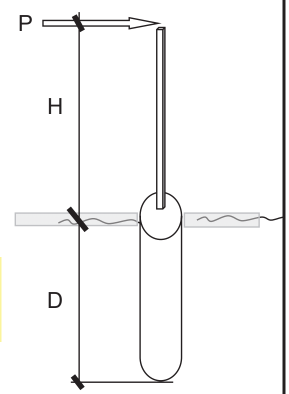
Service Level Moment at Base of Pole (ft-lbs) =	2527
Service Level Moment at Ground (ft-lbs) =	3459
Applied Service Lateral Force (lbs) =	**454**
Distance from Ground to Force (ft) =	**7.6**

Total Applied Service Level Moment at Ground (ft-lbs) =	**7690**
Total Applied Service Lateral Force (lbs) =	**694**
Distance from Ground to Force (ft) =	**11.1**

- DESIGN LIGHT POLE BASE

INPUT DATA & DESIGN SUMMARY

IS FOOTING RESTRAINED @ GRADE LEVEL ? (1=YES,0=NO)		0	no
LATERAL FORCE @ TOP OF POLE	P =	0.694	k
HEIGHT OF POLE ABOVE GRADE	H =	11.1	ft
DIAMETER OF POLE FOOTING	B =	2	ft
LATERAL SOIL BEARING CAPACITY	S =	0.1	ksf / ft
ISOLATED POLE FACTOR (IBC 1804.3.1 or UBC note 3 on Tab 18-I-	F =	1	
FIRST TRIAL DEPTH ===>	D =	6.5	ft



Use 2 ft dia x 7.86 ft deep footing unrestrained @ ground level

ANALYSIS

LATERAL BEARING @ BOTTOM : $S_3 = FS \quad Min(D \quad , \quad 12')$

LATERAL BEARING @ D/3 : $S_1 = FS \quad Min\left(\frac{D}{3} \quad , \quad 12'\right)$

$$A = \frac{2.34P}{BS_1}$$

REQUIRD DEPTH :

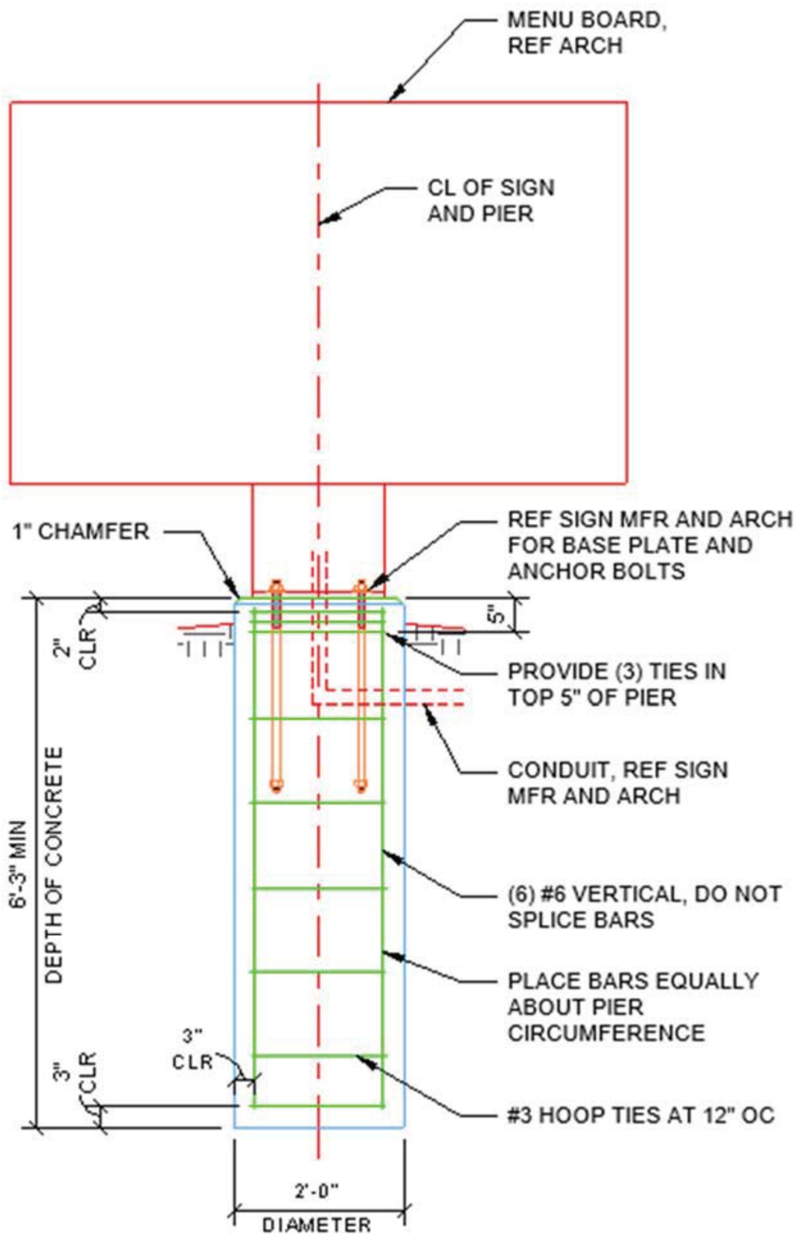
$$D = \begin{cases} \frac{A}{2}\left[1+\sqrt{1+\frac{4.36H}{A}}\right] & \text{, FOR NONCONSTRAINED} \\ \sqrt{\frac{4.25PH}{BS_3}} & \text{, FOR CONSTRAINED} \end{cases}$$

			NONCONSTRAINED		CONSTRAINED	
LATERAL FORCE @ TOP OF POLE	P	=>	0.69	k	0.69	k
HEIGHT OF POLE ABOVE GRADE	H	=>	11.1	ft	11.1	ft
DIAMETER OF POLE FOOTING	B	=>	2.00	ft	2.00	ft
LATERAL SOIL BEARING CAPACITY	FS	=>	0.10	ksf / ft	0.10	ksf / ft

USE 24" DIA BASE EMBEDDED 8' BELOW GRADE

MENU BOARD BASE DESIGN

10-S004:



10 MENU BOARD BASE

1/2" = 1'-0"

- FIND LOADS ON MENU BOARD BASE

WIND LOAD CALCULATIONS (ASCE 7-16, CHAPTER 29)

DESIGN WIND LOADS - SOLID FREESTANDING WALLS AND SOLID SIGNS 29.3

V (mph) =	115	(Ultimate Wind Speed)
V_{asd} (mph) =	89	(Service Wind Speed)
Exposure =	C	
K_{zt} =	1.0	(26.8.2)
K_d =	0.85	(Table 26.6-1)
Elevation (ft) =	0	(Assume Mean Sea Level)
K_e =	1.00	(Table 26.9-1)

Summary of Wind Forces:

Total Applied Service Level Moment at Ground (ft-lbs) =	**2352**
Total Applied Service Lateral Force (lbs) =	**687**
Distance from Ground to Force (ft) =	**3.42**

Sign Geometry

Vertical Dimension of Sign, s (ft) =	4.50	
Horizontal Dimension of Sign, B (ft) =	7.33	
Height of Sign, h (ft) =	5.83	
Area of Sign, A (ft^2) =	33.0	

Velocity Pressure, q_z (psf)

$q_z = 0.00256K_zK_{zt}K_dK_eV^2$(psf) (Eq. 26.10-1)

α =	9.5	(Table 26.11-1)
z_g (ft) =	900	(Table 26.11-1)
K_h =	0.85	(Table 26.10-1)
q_h (psf) =	24.4	(Eq. 26.10-1)

Lateral Force on Sign

$F = q_hGC_fA_s$ (lb) (Eq. 29.3-1)

q_h (psf)=	24.43		
G =	0.85	(26.11)	
Clearance Ratio, s/h =	0.77	(Figure 29.3-1)	
Aspect Ratio, B/s =	1.63	(Figure 29.3-1)	
Case A & Case B, Cf =	1.58	(Figure 29.3-1)	
Case C, $C_{f\ 0\ to\ s}$ =	1.00	(Figure 29.3-1)	
Case C, $C_{f\ s\ to\ 2s}$ =	1.00	(Figure 29.3-1)	
Case C, $C_{f\ 2s\ to\ 3s}$ =	1.00	(Figure 29.3-1)	
Case C, $C_{f\ 3s\ to\ 10s}$ =	1.00	(Figure 29.3-1)	
Case C, $C_{f\ avg}$ =	1.00		1.00
$C_{f\ max}$ =	1.58		
F_{sign} (lbs) =	**1082**	(Ultimate)	
F_{sign} (lbs) =	**649**	(Service)	

Reactions from Sign

Service Level Moment (ft-lbs) =	2325
Applied Service Lateral Force (lbs) =	649
Distance from Ground to Force (ft) =	3.58

Sign Base Geometry

Vertical Dimension of Base, s (ft) =	1.33
Horizontal Dimension of Base, B (ft) =	1.58
Height of Base, h (ft) =	1.33
Area of Base, A (ft^2) =	2.1

Velocity Pressure, q_z (psf)

$q_z = 0.00256K_zK_{zt}K_dK_eV^2$(psf) (Eq. 26.10-1)

α =	9.5	(Table 26.11-1)
z_g (ft) =	900	(Table 26.11-1)
K_h =	0.85	(Table 26.10-1)
q_h (psf) =	24.4	(Eq. 26.10-1)

Lateral Force on Base

$F = q_hGC_fA_s$ (lb) (Eq. 29.3-1)

q_z (psf)=	24.43	
G =	0.85	(26.11)
Clearance Ratio, s/h =	1.00	(Figure 29.3-1)
Aspect Ratio, B/s =	1.19	(Figure 29.3-1)
Case A & Case B, Cf =	1.44	(Figure 29.3-1)
Case C, $C_{f\ 0\ to\ s}$ =	1.00	(Figure 29.3-1)
Case C, $C_{f\ s\ to\ 2s}$ =	1.00	(Figure 29.3-1)
Case C, $C_{f\ 2s\ to\ 3s}$ =	1.00	(Figure 29.3-1)
Case C, $C_{f\ 3s\ to\ 10s}$ =	1.00	(Figure 29.3-1)
Case C, $C_{f\ avg}$ =	1.00	
$C_{f\ max}$ =	1.44	
F_{base} (lbs) =	**63**	(Ultimate)
F_{base} (lbs) =	**38**	(Service)

Reactions from Base

Service Level Moment (ft-lbs) =	28
Applied Service Lateral Force (lbs) =	38
Distance from Ground to Force (ft) =	0.73

Project SALAD AND GO #2001 – LEE'S SUMMIT, MO			Job Ref. 24-011
Description SITE ITEM BASE DESIGN (SEC. SB)	Calc. by BG	Date 4/29/2024	Sheet No. SB6

ELLISON GAGE & ASSOCIATES, PLLC
Consulting Structural Engineers
5068 W. Plano Parkway Suite 200 Plano, TX 75093
Ofc: (972) 354-8855 Fax: (972) 354-8856 www.ellisongage.com

SEISMIC LOADS ON NON-BUILDING STRUCTURES (ASCE 7-16, CHAPTER 15):

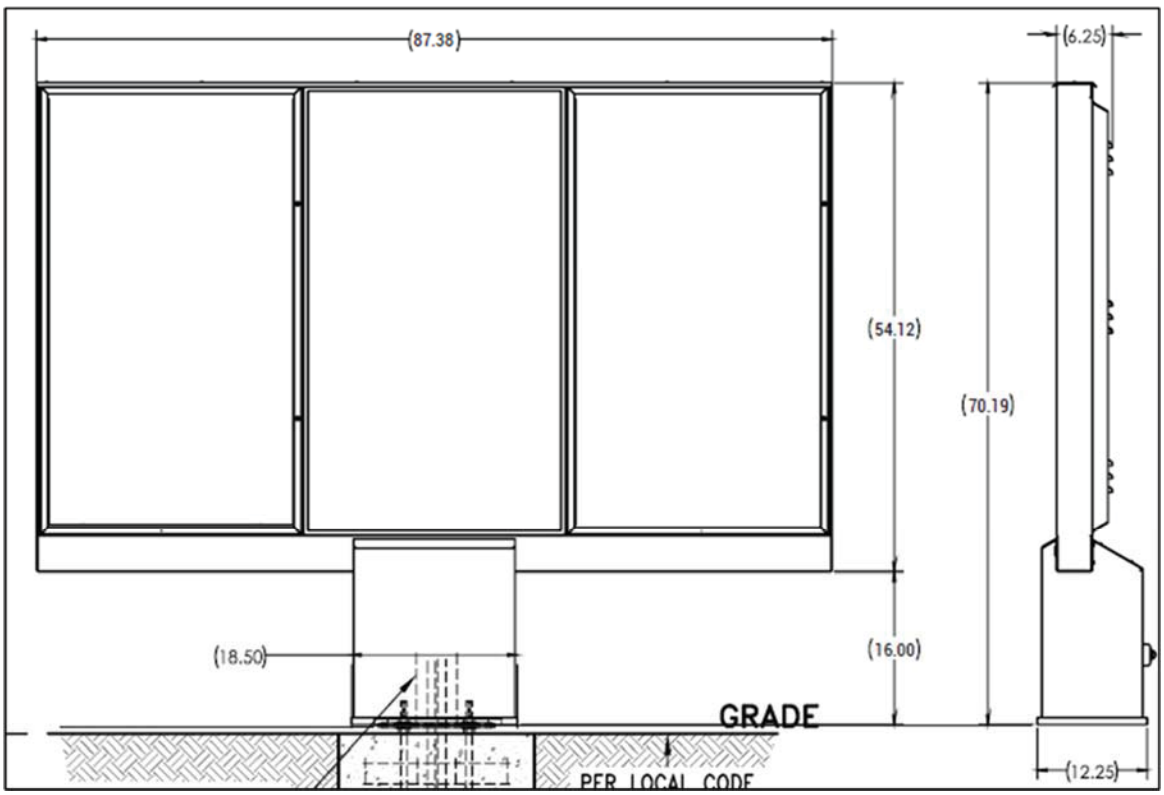
PER 15.4.1, DESIGN WALLS USING SECTION 12.8

S_1 =	0.099			
S_{DS} =	0.106			
S_{D1} =	0.109			
R =	3.00	[TABLE 15.4-2]		
C_d =	3.00	[TABLE 15.4-2]		
I_e =	1.0	[TABLE 1.5-2]		
T_L =	6.00	sec, [FIG. 22-14]		
T =	1.028	sec, [Sec. 12.8.2]		

C_s Bound Equations:

C_s =	0.035	[EQ. 12.8-2]		
C_s =	0.035	[EQ. 12.8-3, 12.8-4]		
C_s =	0.030	[EQ. 15.4-1]		
C_s =	NA	[EQ. 12.8-6]		
C_s =	0.035	(Governing Value)		
$V = C_sW$ =	0.035	W	[EQ. 12.8-1]	Ultimate Load (Factored)
$V = C_sW$ =	0.025	W	[EQ. 12.8-1]	Service Load (Ultimate Load x 0.7)
Menu Board Weight, W =	800	lbs		

Summary of Seismic Forces:			
Applied Service Lateral Force =	**20**	lbs	
Distance from Ground to Force =	**4.0**	ft	
Service Level Moment =	**79**	ft-lbs	**Wind Controls**

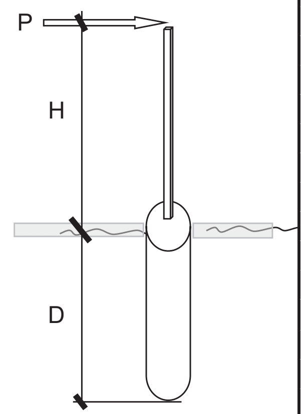


ELLISON GAGE & ASSOCIATES, PLLC Consulting Structural Engineers 5068 W. Plano Parkway Suite 200 Plano, TX 75093 Ofc: (972) 354-8855 Fax: (972) 354-8856 www.ellisongage.com	Project SALAD AND GO #2001 – LEE'S SUMMIT, MO			Job Ref. 24-011
	Description SITE ITEM BASE DESIGN (SEC. SB)	Calc. by BG	Date 4/29/2024	Sheet No. SB7

- DESIGN MENU BOARD BASE

INPUT DATA & DESIGN SUMMARY

IS FOOTING RESTRAINED @ GRADE LEVEL ? (1=YES,0=NO)		0	no
LATERAL FORCE @ TOP OF POLE	P =	0.687	k
HEIGHT OF POLE ABOVE GRADE	H =	3.42	ft
DIAMETER OF POLE FOOTING	B =	2	ft
LATERAL SOIL BEARING CAPACITY	S =	0.1	ksf / ft
ISOLATED POLE FACTOR (IBC 1804.3.1 or UBC note 3 on Tab 18-I-	F =	1	
FIRST TRIAL DEPTH ===>	D =	6.5	ft



Use 2 ft dia x 6.21 ft deep footing unrestrained @ ground level

ANALYSIS

LATERAL BEARING @ BOTTOM : $S_3 = FS \quad Min(D \quad , \quad 12')$

LATERAL BEARING @ D/3 : $S_1 = FS \quad Min\left(\frac{D}{3} \quad , \quad 12'\right)$

$$A = \frac{2.34P}{BS_1}$$

REQUIRD DEPTH :

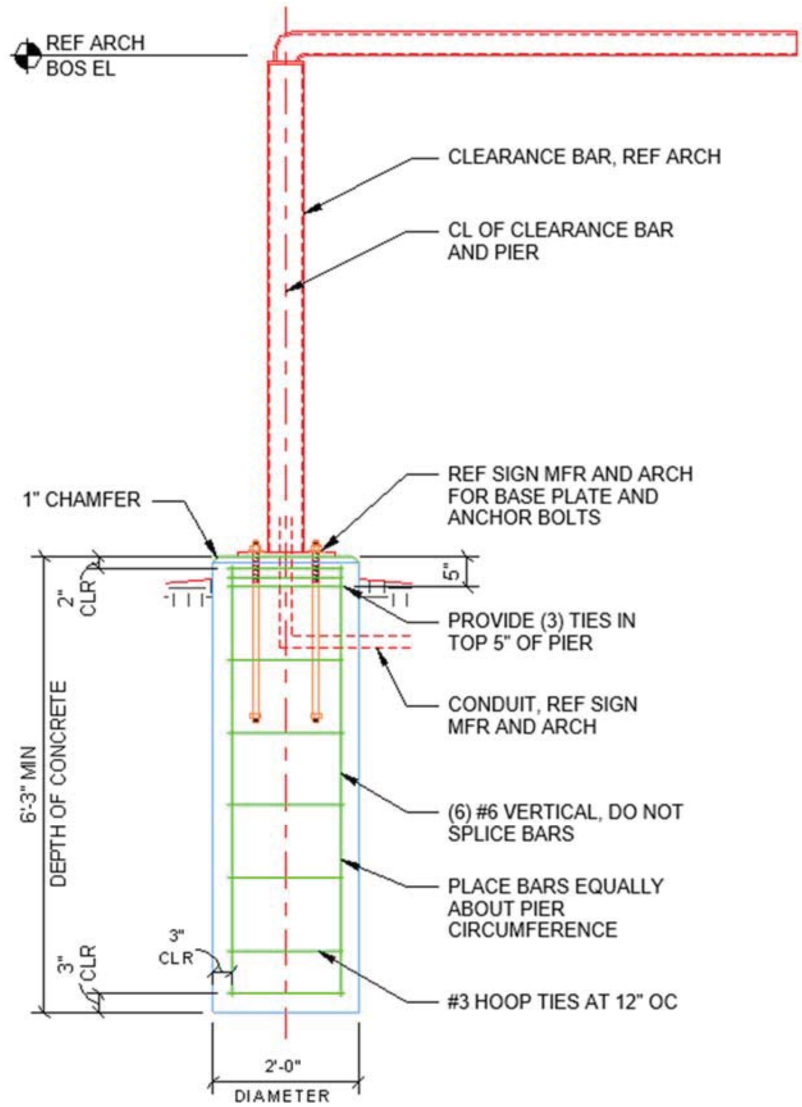
$$D = \begin{cases} \frac{A}{2}\left[1+\sqrt{1+\frac{4.36H}{A}}\right] & \text{, FOR NONCONSTRAINED} \\ \sqrt{\frac{4.25PH}{BS_3}} & \text{, FOR CONSTRAINED} \end{cases}$$

			NONCONSTRAINED		CONSTRAINED	
LATERAL FORCE @ TOP OF POLE	P	=>	0.69	k	0.69	k
HEIGHT OF POLE ABOVE GRADE	H	=>	3.4	ft	3.4	ft
DIAMETER OF POLE FOOTING	B	=>	2.00	ft	2.00	ft
LATERAL SOIL BEARING CAPACITY	FS	=>	0.10	ksf / ft	0.10	ksf / ft

USE 24" DIA BASE EMBEDDED 6'-3" BELOW GRADE

CLEARANCE BAR BASE DESIGN

11-S004:



11 CLEARANCE BAR BASE

1/2" = 1'-0"

- FIND LOADS ON CLEARANCE BAR BASE

WIND LOAD CALCULATIONS ASCE 7-16 (Chapter 29)

DESIGN WIND LOADS - SOLID FREESTANDING WALLS AND SOLID SIGNS 29.3

V (mph) =	115	(Ultimate Wind Speed)
V_{asd} (mph) =	89	(Service Wind Speed)
Exposure =	C	
K_{zt} =	1.0	(26.8.2)
K_d =	0.85	(Table 26.6-1)
Elevation (ft) =	0	(Assume Mean Sea Level)
K_e =	1.00	(Table 26.9-1)

Summary of Wind Forces:

Total Applied Service Level Moment at Ground (ft-lbs) =	**3414**
Total Applied Service Lateral Force (lbs) =	**457**
Distance from Ground to Force (ft) =	**7.48**

Vertical Column Geometry

Vertical Dimension of Column, s (ft) =	9.50	
Horizontal Dimension of Column, B (ft) =	0.67	
Height to top of Column, h (ft) =	9.50	
Area of Column, A (ft^2) =	6.4	

Velocity Pressure, q_z (psf)

$q_z = 0.00256 K_z K_{zt} K_d K_e V^2$(psf) (Eq. 26.10-1)

α =	9.5	(Table 26.11-1)
z_g (ft) =	900	(Table 26.11-1)
K_h =	0.85	(Table 26.10-1)
q_h (psf) =	24.4	(Eq. 26.10-1)

Lateral Force on Vertical Column

$F = q_h G C_f A_s$ (lb) (Eq. 29.3-1)

q_h (psf)=	24.43	
G =	0.85	(26.11)
Clearance Ratio, s/h =	1.00	(Figure 29.3-1)
Aspect Ratio, B/s =	0.07	(Figure 29.3-1)
Case A & Case B, Cf =	1.80	(Figure 29.3-1)
Case C, $C_{f\,0\text{ to }s}$ =	1.00	(Figure 29.3-1)
Case C, $C_{f\,s\text{ to }2s}$ =	1.00	(Figure 29.3-1)
Case C, $C_{f\,2s\text{ to }3s}$ =	1.00	(Figure 29.3-1)
Case C, $C_{f\,3s\text{ to }10s}$ =	1.00	(Figure 29.3-1)
Case C, $C_{f\,avg}$ =	1.00	1.00
$C_{f\,max}$ =	1.80	
F_{sign} (lbs) =	**238**	(Ultimate)
F_{sign} (lbs) =	**143**	(Service)

Reactions from Sign

Service Level Moment (ft-lbs) =	746
Applied Service Lateral Force (lbs) =	143
Distance from Ground to Force (ft) =	5.23

Horizontal Support Geometry

Vertical Dimension of Horiz. Support, s (ft) =	2.00	
Horizontal Dimension of Horiz. Support, B (ft) =	7.00	
Height to top of Horiz. Support, h (ft) =	9.50	
Area of Horiz. Support, A (ft^2) =	14.0	

Velocity Pressure, q_z (psf)

$q_z = 0.00256 K_z K_{zt} K_d K_e V^2$(psf) (Eq. 26.10-1)

α =	9.5	(Table 26.11-1)
z_g (ft) =	900	(Table 26.11-1)
K_h =	0.85	(Table 26.10-1)
q_h (psf) =	24.4	(Eq. 26.10-1)

Lateral Force on Base

$F = q_h G C_f A_s$ (lb) (Eq. 29.3-1)

q_z (psf)=	24.43	
G =	0.85	(26.11)
Clearance Ratio, s/h =	0.21	(Figure 29.3-1)
Aspect Ratio, B/s =	3.50	(Figure 29.3-1)
Case A & Case B, Cf =	1.80	(Figure 29.3-1)
Case C, $C_{f\,0\text{ to }s}$ =	2.75	(Figure 29.3-1)
Case C, $C_{f\,s\text{ to }2s}$ =	1.80	(Figure 29.3-1)
Case C, $C_{f\,2s\text{ to }3s}$ =	1.22	(Figure 29.3-1)
Case C, $C_{f\,3s\text{ to }10s}$ =	1.00	(Figure 29.3-1)
Case C, $C_{f\,avg}$ =	1.69	
$C_{f\,max}$ =	1.80	
F_{base} (lbs) =	**523**	(Ultimate)
F_{base} (lbs) =	**314**	(Service)

Reactions from Base

Service Level Moment (ft-lbs) =	2669
Applied Service Lateral Force (lbs) =	314
Distance from Ground to Force (ft) =	8.50

SEISMIC LOADS ON NON-BUILDING STRUCTURES (ASCE 7-16, CHAPTER 15):

PER 15.4.1, DESIGN WALLS USING SECTION 12.8

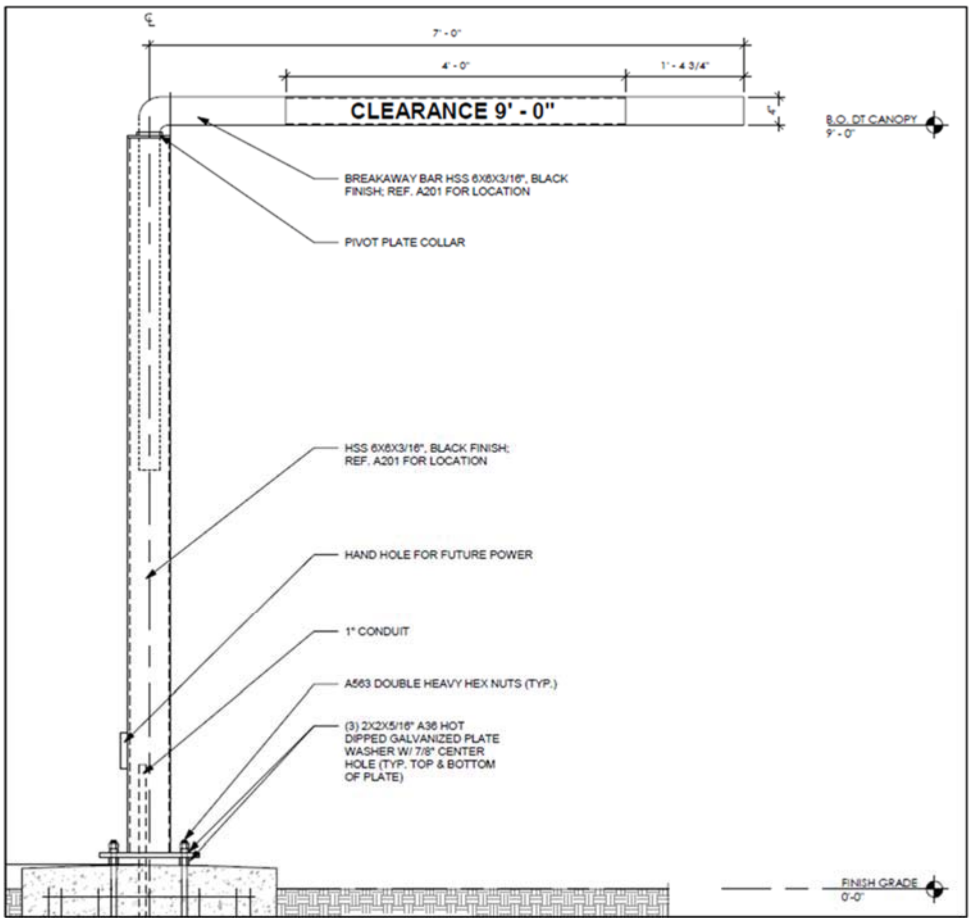
S_1 =	0.099		
S_{DS} =	0.106		
S_{D1} =	0.109		
R =	3.00	[TABLE 15.4-2]	
C_d =	3.00	[TABLE 15.4-2]	
I_e =	1.0	[TABLE 1.5-2]	
T_L =	6.00	sec, [FIG. 22-14]	
T =	1.028	sec, [Sec. 12.8.2]	

C_s Bound Equations:

C_s =	0.035	[EQ. 12.8-2]	
C_s =	0.035	[EQ. 12.8-3, 12.8-4]	
C_s =	0.030	[EQ. 15.4-1]	
C_s =	NA	[EQ. 12.8-6]	
C_s =	0.035	(Governing Value)	
$V = C_sW$ =	0.035	W	[EQ. 12.8-1 Ultimate Load (Factored)
$V = C_sW$ =	0.025	W	[EQ. 12.8-1 Service Load (Ultimate Load x 0.7)

Clearance Bar Weight, W = 400 lbs

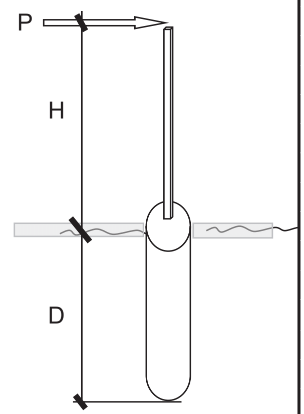
Summary of Seismic Forces:			
Applied Service Lateral Force =	10	lbs	
Distance from Ground to Force =	5.75	ft	
Service Level Moment =	57	ft-lbs	**--Wind Controls**



- DESIGN CLEARANCE BAR BASE

INPUT DATA & DESIGN SUMMARY

IS FOOTING RESTRAINED @ GRADE LEVEL ? (1=YES,0=NO)		0	no
LATERAL FORCE @ TOP OF POLE	P =	0.457	k
HEIGHT OF POLE ABOVE GRADE	H =	7.48	ft
DIAMETER OF POLE FOOTING	B =	2	ft
LATERAL SOIL BEARING CAPACITY	S =	0.1	ksf / ft
ISOLATED POLE FACTOR (IBC 1804.3.1 or UBC note 3 on Tab 18-I-	F =	1	
FIRST TRIAL DEPTH ===>	D =	6.5	ft



Use 2 ft dia x 6.12 ft deep footing unrestrained @ ground level

ANALYSIS

LATERAL BEARING @ BOTTOM : $S_3 = FS \quad Min\left(D \quad , \quad 12'\right)$

LATERAL BEARING @ D/3 :

$$S_1 = FS \quad Min\left(\frac{D}{3} \quad , \quad 12'\right)$$

$$A = \frac{2.34P}{BS_1}$$

REQUIRD DEPTH :

$$D = \begin{cases} \frac{A}{2}\left[1+\sqrt{1+\frac{4.36H}{A}}\right] & \text{, FOR NONCONSTRAINED} \\ \sqrt{\frac{4.25PH}{BS_3}} & \text{, FOR CONSTRAINED} \end{cases}$$

			NONCONSTRAINED		CONSTRAINED	
LATERAL FORCE @ TOP OF POLE	P	=>	0.46	k	0.46	k
HEIGHT OF POLE ABOVE GRADE	H	=>	7.5	ft	7.5	ft
DIAMETER OF POLE FOOTING	B	=>	2.00	ft	2.00	ft
LATERAL SOIL BEARING CAPACITY	FS	=>	0.10	ksf / ft	0.10	ksf / ft

USE 24" DIA BASE EMBEDDED 6'-3" BELOW GRADE